REPUBLIQUE DU CAMEROUN

Paix - Travail – Patrie

REPUBLIC OF CAMEROON

Peace – Work – Fatherland

DEPARTMENT OF CIVIL ENGINEERING

DÉPARTEMENT DE GENIE CIVIL

DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

STABILIZATION OF A LANDSLIDE ALONG THE BATIBO-NUMBA ROAD IN THE NORTH-WEST REGION OF CAMEROON

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

Curriculum: Geotechnical Engineering

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Academic Year: 2019-2020

DEDICATION

This work is dedicated to the Nupukuh's family

ACKNOWLEDGEMENTS

The achievement of this work is first and foremost thanks to God Almighty who accorded me the strength and ability to be able to complete my studies. It is also thanks to the collaboration of the efforts of several physical and moral actors who contributed either directly or indirectly. I address my sincere and deepest gratitude to:

- The **President of the Jury, Prof. Simonetta Cola** for having accepted to preside at my defense.
- The **Examiner of the Jury, Dr. NIMPA Giscard** who took out time to assess and correct this work
- My supervisor, **Prof. Simonetta Cola** for her guidance, appraisals and corrections throughout this work.
- The Director of the NASPW, **Pr. NKENG George ELAMBO, Pr. Carmelo MAIORANA** and **Pr. ESOH ELAME** of the University of Padova, Italy for all their academic and administrative support during my five years of studies at NASPW and also for giving me the opportunity participate in the academic program under the partnership with the University of Padova, Italy.
- **Pr. MBESSA Michel**, the head of the civil engineering department of the NASPW, for his availability, dynamism and strict follow up during my thesis writing and throughout my studies and for being a source of inspiration for hard work.
- **Eng. WEMBE BILL Jeferson** for his advice and help towards the achievement of this work
- **The staff of the NASPW University of Padova** for their dedication, continuous instruction and sacrifices to ensure that the knowledge and moral values we need for competence in the civil engineering field were instilled in us.
- **All the administrative personnel of the NASPW** for their expertise and combined efforts in ensuring a successful running of the academic year.
- **To all my classmates and seniors** for being a source of motivation through all the moments we shared and for helping me overcome the many difficulties during my studies.
- Finally, many thanks to my lovely parents, **Mr. NUPUKUH Edward** and **Mrs NUPUKUH Sophina** who through their continuous moral, spiritual and financial support gave me an unremitting source of inspiration for this work.

ABBREVIATIONS AND SYMBOLS

2D	2-dimensional
3D	3-dimensional
AASHTO	American Association of State Highway and Transportation
$\alpha_{bond\text{-}below}$	grout-to-ground ultimate bond strength below the critical slip surface
$\alpha_{bond\text{-}above}$	grout-to-ground ultimate bond strength above the critical slip surface
API	American Petroleum Institute
ASTM	American society for testing and materials
ASTM-USCS	Modification of the USCS system by ASTM
c	Cohesion
cu	Undrained shear strength
CSS	Circular slip surface
DA	Design action
DDH	Diameter of drillhole
DPH	Dynamic penetration heavy
Dx	Diameter for which x% is finer
ECR	Essai de cisaillement rectiligne (direct shear test)
EN	European standard
FEM	Finite element method
FHWA	Federal Highway Administration
FoS	Factor of safety
γ	Bulk unit weight
GEO	Geotechnical
IN	Interslice normal
IS	Interslice shear
JICA	Japan International Cooperation Agency
LE(M)	Limit equilibrium (method)
LL	Liquid limit
M-P	Morgenstern-Price
MSE	Mechanically stabilized earth
NR6	National road 6

ϕ	Angle of friction (ϕ' for drained conditions)
PL	Plastic limit
PI	Plasticity index
σ	Total stress
σ'	Effective stress
SL	Shrinkage limit
SRM	Strength reduction method
PEI	Disturbed soil sample
PER	Undisturbed soil sample
PVC	Polyvinyl Chloride
τm	Mobilized shear force
τf	Ultimate shear strength
STR	Structural
USCS	Unified soil classification system
ULS	Ultimate Limit State

ABSTRACT

This work has as general objective stabilize a recurrent landslide along the Batibo-Numba road by proposing a complementary and an alternative solution to the retaining wall already constructed by the road construction company. It is also of importance to identify the possible causes of the landslide so as to develop appropriate remediation measures. In order to tackle this problem and achieve this objective, an idealized cross-section of the slope at the kilometric point labeled PK19 was studied. SLOPE/W, a numerical modeling tool, was used to represent and analyze the study slope of the site with the geotechnical and geometrical data obtained from both field and laboratory investigations. After performing a numerical analysis of the slope in its natural state and also with the retaining walls which were already put in place for the landslide stabilization, reinforcements were included with the aim of increasing the initial safety factor to an acceptable value. A total of three techniques were proposed for the landslide stabilization: one meant to complement the retaining wall and two alternatives. With the retaining wall, the overall safety factor was at a value of 0.951 indicating instability. To complement the retaining wall was soil nailing which gave an overall safety factor of 1.308. The alternative techniques include stabilizing the soil slope with micropiles which yielded a factor of safety of 1.4 and using anchors to support the slope which also yielded a safety factor of 1.329. The various solutions were compared using a factor of safety and cost criteria. The costs considered were the material costs and they were estimated for a linear meter of the span of the affected slope. For the soil nailing coupled with the retaining wall, the cost was 2,473,343 FCFA. The micropiles estimated cost was at 1,029,410 FCFA and the anchored shotcrete support cost was 2,348,483 FCFA. On these grounds, the system of micropiles was found to be the most cost-saving and performant solution to the landslide.

Keywords: Landslide, slope stability, limit equilibrium methods, Factor of safety, stabilization techniques.

RESUME

Ce travail a pour objectif général de stabiliser un glissement de terrain récurrent le long de la route Batibo-Numba en proposant une solution complémentaire et une solution alternative au mur de soutènement déjà construit par l'entreprise qui était chargée de la construction de cette route. Il est également important d'identifier la cause de ce glissement de terrain afin de proposer des mesures correctives appropriées. Afin d'aborder ce problème et d'atteindre cet objectif, une section transversale idéalisée de la pente au point kilométrique désigné par PK19 a été étudiée après une analyse sélective des données collectées sur le site. SLOPE/W, un outil de modélisation numérique, a été utilisé pour représenter et analyser la pente avec les données géotechniques et géométriques obtenues sur le terrain et au laboratoire. Après avoir effectué une analyse numérique de la pente dans son état naturel et aussi avec les murs de soutènement qui sont déjà mis en place pour la stabilisation dudit glissement de terrain, les renforcements ont été inclus dans le but d'augmenter le facteur de sécurité à une valeur acceptable. Avec le mur de soutènement, le facteur de sécurité global était de 0,951, ce qui indique une instabilité. Trois techniques au total ont été proposées pour la stabilisation dudit glissement de terrain : une prévue pour être utilisée avec le mur de soutènement et deux alternatives. Couplé avec le mur, le clouage du sol a donné un facteur de sécurité global de 1,308. Les techniques alternatives comprennent la stabilisation de la pente avec des micropieux qui a donné un facteur de sécurité de 1,4 et un support en béton projeté ancré ce qui a donné un facteur de sécurité de 1,329. Les différentes solutions ont été comparées en utilisant un critère de facteur de sécurité et du coût. Les coûts ont été évalués pour un mètre linéaire de la pente. Pour le clouage du sol couplé au mur de soutènement, le coût est de 2 473 343 FCFA. Les micropieux coûtent 1 029 410 FCFA et le support en béton projeté ancré coût 2 348 483 FCFA. Sur la base de ces critères, le système de micropieux s'est avéré être la solution la plus économique et la plus efficace

Mots clés : Glissement de terrain, stabilité des pentes, méthodes d'équilibre limite, facteur de sécurité, techniques de stabilisation.

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GENERAL INTRODUCTION

Mankind is progressively being placed at the mercy of nature with the rapid increase in population, increasing demands for resources, urbanization and environmental changes. Considering the fact that a lot of construction activities are taking place to meet up these demands, a civil engineer is compelled to build on all sorts of lands regardless of the geometry of the landforms and the geological constituents of the soils in these areas. Unfortunately, in many cases the risk of geological hazards cannot be avoided during construction and some of these infrastructures may experience damage during or after their construction due to the occurrence of hazards such as earthquakes, landslides and floods. Landslides are responsible for thousands of deaths yearly and monetary damages worldwide (Sassa, 2004). Along national highways, they are the most frequently observed road slope disasters, severely threatening or damaging their traffic function (Japan International Cooperation Agency, 2007). Landslides are otherwise referred to as mass movements and they involve many different geological processes (Pradhan et al., 2019). They occur on slopes, in a variety of geological materials and develop through various mechanisms and causes (Bolt et al., 1977). Slope materials have a tendency to slide due to shearing stresses created in the soil by gravitation and other forces (Abramson et al., 2002). This sliding is resisted by the shear strength of the slope materials. Landslides are mainly caused by strength variations at the transition zones of soil layers (soft soil located on rock layer), heavy traffic loads, seasonal high-intensity precipitations, erosion at the toe of slopes, rapid snow melts and some other natural events (Usluogullari et al., 2016). Generally, this geological phenomenon involves a wide range of ground movement such as rock falls, deep failure of slopes and shallow debris flow which can occur offshore, in coastal and onshore environments (Werner et al., 2011). With time, measures have been developed for the stabilization of landslides in order to manage and control the potential damages and avoid fatalities. Slope stabilization methods have vital roles to prevent landslides and cease soil movement. The practice of stabilizing slopes adjacent to roadsides is called road-slope stabilization (Laura et al., 2012). In order to evaluate slope stabilities via the calculation of factor of safety, many researchers have developed several methods to be used for analysis including limit equilibrium and finite element methods. A slope may possess several factors of safety according to different methods of analysis (Cheng & Lau, 2014).

In Cameroon, landslides account for about 25% of natural hazards and have led to the loss of lives and property (Guedjeo et al., 2017). The Batibo-Numba road is one of the road infrastructures in Cameroon affected by recurrent landslides occurring on different sections along the road with the most recent landslide event in October 2020. During the period of construction of the road, a retaining wall was put in place to stabilize one of the road cut slopes but it failed with time as mass movements still took place thereafter. With debris falling on the road from the uphill side of the road and a portion of the road sliding along with the landslides, traffic is constantly interrupted. These conditions reduce the serviceability of the road and pose a serious threat to the safety of road users.

The major objective of this work is to identify the possible causes of the recurrent landslides along the Batibo-Numba road despite the initial solution already implemented by the road construction company, propose a complementary solution to the initial solution and also propose an alternative solution to the problem.

In order to achieve the aforementioned objective, this work is divided into three chapters. The first chapter is the literature review which presents the basic soil-related concepts and landslide terminologies, establishing the relationship between slope stability and landslide stabilization and then the available stabilization techniques. The second chapter focusses on the methodology used to achieve the main objective of this work and the third and final chapter presents the results obtained, their interpretations and the solutions proposed for the landslide stabilization.

CHAPTER 1. LITERATURE REVIEW

Introduction

Landslide occurrences pose a serious threat to both the natural and built environments. The attention of many researchers and geotechnical engineers has been drawn to this natural hazard as it demands that adequate measures are taken in order to avoid its risk on human lives and properties by an appropriate technical design of stabilization techniques (Shamsan et al. 2017). In order to design an appropriate, cost-effective remedial measure for a landslide, the processes leading to its occurrence need to be clearly understood (Mihail, 2002). Knowledge of the rock and soil profiles of the affected site helps in the idealization of the ground conditions and in subsequent development of realistic geotechnical models. Presented in the first part of this chapter will be the basic definition of soils, soil constituents, soil formation and types followed by soil properties and standard classification of soils. The second part gives the processes involved in landslides, the possible causes of landslides and the effects of this phenomenon on both the natural and built environments. The third part gives an overview of the notion of factor of safety, the methods available for slope stability analysis, the types of slope stability analyses and the conditions to be considered when performing slope stability analyses. The fourth part of this chapter will present the various design approaches used in performing a slope stability analysis. In the last part, an overview of various landslide stabilization techniques will be done.

1.1. Soil

To the civil engineer, soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks (aggregation of minerals into a hard mass), the void space between the particles containing water and/or air (Craig, 2004). Also, soil to an engineer is a material that can be built on, built in, built with and supported (Balasubramanian, 2017). This section aims at providing a general understanding of soils and related concepts essential to slope stability.

1.1.1. Soil constituents

A soil mass as it exists in nature is essentially composed of about 50% of solid particles with voids in-between them containing liquids or gases or both (Liao et al., 2020). This means that an element of soil may be a three-phase material as illustrated in Figure 1.1. The solid phase of the soil is generally stable in nature and may be made up of minerals or organic matter or both. Yet, if organic matter is not properly managed, it may be depleted from the soil through various processes (Soil management, 2021). The liquid and gas phases of the soil, which are water and air respectively, are the most dynamic properties of the soil. The relative amounts of water and air in the soil are constantly changing as the soil wets or dries. A soil is said to be saturated if all the voids are filled with water and unsaturated or dry if partially filled with water or air only.

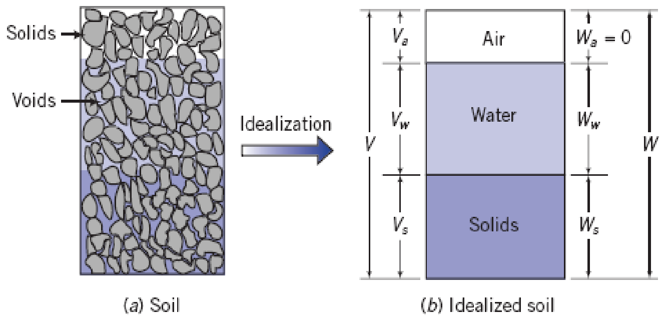

Figure 1.1. Soil phases (Budhu, 2015)

With reference to Figure 1.1, fundamental definitions known as phase relationships are essential in the characterization of the state of a soil. They include:

- Void ratio, $e = \frac{V_w + V_a}{V_s}$ (1.1)
- specific volume, $v = (Vs + V_w + V_a)/Vs = 1 + e$ (1.2)
- Porosity, $n = \frac{Vv}{Vs+Vv} = \frac{e}{1+e} = (v-1)v$ (1.3)
- Saturation ratio, $S_r = \frac{V_w}{V_v}$, $0 \leq S_r \leq 1$. (1.4)

  S_r=0 for dry soils and S_r=1 for fully saturated soils
- Water Content, $w = \frac{W_w}{W_s} \times 100\ (\%)$ (1.5)
- Bulk unit weight, $\gamma = \frac{W}{V}$ (1.6)
- Specific gravity, $Gs = \frac{W_s}{V_s\,\gamma_w}$, $\gamma_w = \frac{W_w}{V_w}$ (1.7)

Where Vw, Va and Vs are the volume of water, air and solids respectively and Ww, Wa and Ws are the weight of water, air and solids respectively

1.1.2. Soil formation

The earth's crust is made up of in-place rock and unconsolidated sediments. The sediments are derived from the rock formations through weathering processes which are either physical disintegration and chemical decomposition processes (Holtz, 1974). The weathered rock formations are either igneous (magmatic rocks), sedimentary (formed by accumulation of mineral or organic particles) or metamorphic rocks (formed from the alteration of preexisting rocks) (Venkratramiah, 2006). The soils may remain at their site of formation (residual soils) or are transported to other regions by agents such as wind, gravity, water or ice (transported soils). As transported soils are moved, reworked, abraded, further weathered, mixed with organic materials and soluble minerals, all in varying degrees, they eventually form the material that concerns the soils engineer. Depending on the chemical and physical properties of the parent rock and the geological processes, different types of soils are formed (Holtz, 1974).

1.1.2.1. Factors influencing soil formation

Soil forming factors integrate and drive soil forming processes to create unique and varied soil profiles across the local landscape and the world (Daniels et al., 2011). Several factors influence the soil formation process. Among these factors are the nature of the parent rock, climate, temperature, biota, topography and time (Jenny, 1941).

a. Nature of the parent material

The material from which a soil forms is called the parent material. Soils inherit many properties from their parent materials such as the mineral elements, color, particle size and the chemical composition. Residual soils have the same chemical composition as the parent material. Metamorphic rocks which are basically granite, gneiss and schist form red soils upon weathering because they contain iron oxide. Soils derived from igneous rocks are black colored while sandy soils are derived from sandstone.

b. Climate

Climate affects soil formation process by varying the input of heat and moisture. Temperature and moisture amounts cause different patterns of weathering and leaching. Wind redistributes sand and other particles especially in desert regions. The amount, intensity, timing, and type of precipitation influence soil formation. Temperature augments the rate of chemical reactions, evapotranspiration and biological processes. Wide variations in temperature, especially in the presence of water cause shrinking and swelling, frost action and general weathering in soils.

c. Biota

Plants, animals and microbes are all agents that play an active role in the soil formation process. Animals and micro-organisms form burrows and pores in the soil by mixing them. Plant roots create channels in soils and also add organic content to the soil through decomposition. Micro-organisms affect chemical exchanges between roots and soil. Animals are able to mix the soil so extensively that the soil material reverts to having the same properties as the parent material.

d. Topography

Slope and aspect affect the moisture and temperature of soil. Steep slopes facing the sun are warmer, just like the south-facing side of a building. Runoff from uplands creates wetter conditions on the lowlands, and organic soils in some cases. Thus, as a redistributor of the climate features, topography affects soil processes and soil distribution at a particular site.

e. Time

The length of time during which all the aforementioned factors interact is of great importance in determining the characteristics of the soil. Soil formation processes are continuous. Recently deposited material, such as the deposition by a water body or wind, show no features from soil development activities. The previous soil surface and underlying layers become buried and the time clock resets for these soils. Over time, as climate, vegetation, animals, and topography affect soil development, the vertical disparity becomes increasingly visible giving rise to a soil profile.

1.1.2.2. Soil formation processes

There are four different processes involved in soil formation. They include additions, losses, translocations and transformations (Balasubramanian, 2017)

a. Addition

In the top layer of the soil, organic matter from litter and roots are added to the soil and decompose giving rise to humus. Over time, the humus accumulates in poor drained soils due to limitation of the organic material breakdown by waterlogging. Materials deposited by the forces of ice, water or wind also accumulate over time. Some plants with the aid of bacteria fix atmospheric nitrogen and ammonia compounds into the soil as nitrates.

b. Losses

Losses can take place via the movement of wind or water, uptake by plants, soil particles or the erosion or leaching of chemical compounds from the soil. Leaching of the soil involves the removal of soluble components of the soil such as carbonates, magnesium and other minerals. As a result, the physical and chemical compositions of the new accumulated materials together with the soil parent material are altered.

c. Translocations

Translocations involve movement of soil constituents within the soil profile. Translocation usually involves the movement of solution or in suspension (eluviation) of clay, organic matter and hydrous oxides. Over time, this process is one of the more visibly noticeable as variations in color, texture, and structure become apparent.

d. Transformation

Transformation refers to the modification of soil mineral and organic fractions through biophysical and chemical processes such as podsolization, laterization, calcification, salinization and gleization. Podsolization takes place when strong acidic solutions breakdown clay minerals. Laterization is a pedogenic process common to soils found in tropical and subtropical environments which results in the formation of laterite. Calcification occurs when evapotranspiration exceeds precipitation causing the upward movement of dissolved alkaline salts from the groundwater. Salinization occurs with increasing salt content in the soil. Gleiing occurs in waterlogged, anaerobic conditions when iron compounds are reduced and either removed from the soil forming glei (blue, green or grey clay).

1.1.3. Basic soil types

Familiarity with basic soil types is essential for understanding the fundamentals of soil behavior. Soil can be divided into four basic types which include sand, clay, silt, and loam.

1.1.3.1. Clay

Clay is formed very slowly as a result of the weathering and erosion of rocks containing the mineral group known as feldspar. The small size, flat shape, and mineral composition of clay particles combine to produce a material that is both compressible and plastic. Clay soils have medium to high plasticity until they are dried to the point where they become hard and brittle. They have low resistance to deformation when wet, but they dry to hard, cohesive masses. Generally, clays are virtually impervious, difficult to compact when wet, and impossible to drain by ordinary means.

1.1.3.2. Silt

Silts are the non-plastic fines which have the tendency to become quick (a viscous fluid) when saturated. Silts are fairly impervious, difficult to compact, and undergo change of volume with change of shape (the property of dilatancy). When dry, silt can be pulverized easily under finger pressure and has a smooth feel between the fingers unlike the grittiness of fine sand. Generally, the higher the liquid limit of a silt, the more compressible it is.

1.1.3.3. Sand

Sandy soil is usually formed by the breakdown or fragmentation of rocks like granite, limestone and quartz. The precise composition of sand varies depending on its source and the conditions prevalent at that location. Sand is in very commonly used in construction, often providing bulk, strength and stability to other materials such as concrete, mortar, asphalt and cement. Sand particles are loose and thus free draining, making sand a very good material used in drainage systems.

1.1.3.4. Loam

Loam soil is a mixture of sand, silt and clay which incorporates the beneficial properties from each. These soils are easy to work with and drain easily. Depending on their predominant composition they can be either sandy or clay loam.

1.1.4. Soil properties

Visual description of a soil as it is found in the field as well as certain mechanical properties such as permeability and strength of soils can be of considerable importance to a civil engineer.

1.1.4.1. Particle shape

In coarse soils and silts, the soil grains are bulky in nature. The particle shape of bulky grains may be described by terms such as angular, sub-angular, sub-rounded, rounded and well-rounded. The more rounded the soil grains, the lower the friction between the particles whereas the more angular the soil particles are, the higher the friction between them (Venkratramiah, 2006). Coarse-grained soils with angular particles have higher strengths, higher compressibility, and lower densities compared to those with rounded particles (Cornforth, 2005).

1.1.4.2. Color

Soil color may vary widely, ranging from white to red and then black. Besides the degree of oxidation, the color of a soil depends on the mineral matter, the nature and amount of organic matter and the amount of coloring oxides of iron and manganese (Venkratramiah, 2006). An increase in the organic content of soil causes it to become darker while a decrease in moisture content leads to the lightening of soil color. A dark-colored soil turns lighter upon oven-drying.

1.1.4.3. Texture

The texture of soil is reflected largely by the particle size, shape, and gradation. The relative percentages of sand, clay and silt are what give soil its texture. The particle size and composition of soils influence their load-bearing capacity and settlement characteristics. The sieve analysis is used to determine the particle size distribution for coarse-grained soils. A hydrometer analysis or sedimentation test is usually carried out following a sieve analysis in order to analyze the finer grains ($d < 0.075$mm) when the percentage of fines in the soil sample exceeds 5%. Soil texture has a major influence on the form, stability and resilience of soil structure.

1.1.4.4. Soil consistency

As earlier mentioned, soil can be seen as a medium with predominantly the solid and liquid phases. If the solid phase is comprised of coarse particles such as coarse sand or gravel, then water can flow between the particles of the solid phase with ease. If the solid phase is comprised of fine-grained particles like clay or silt, then water cannot flow as freely as in the coarse-grained solid phase because pore spaces are smaller and solids react with water. The consistency of fine-grained soil refers to its firmness and it varies with the water content in the soil. The physical and mechanical behavior of fine-grained-soils is linked to four distinct states: solid, semi-solid, plastic and viscous liquid in order of increasing water content (FHWA-NHI, 2006). The shrinkage limit SL is the water content that causes a fine soil to change from a semisolid to a solid state. The plastic limit PL is the water content at which the soil changes from a semisolid to the plastic state. Below the plastic limit, the fine soil becomes brittle and crumbly. The water content above which the soil will behave as a liquid is known as the liquid limit, LL. The soil behaves as a plastic material if the water content is greater than the plastic limit but less than the liquid limit, and can be molded in the hand without cracking or running out between the fingers. The range of water content over which the soil behaves in this way is known as the plasticity index (PI) calculated from equation (1.8). Index properties help to assess the engineering behavior of a soil and assist in its classification. Figure 1.2 illustrates the various consistency limits.

$$PI = LL - PL \tag{1.8}$$

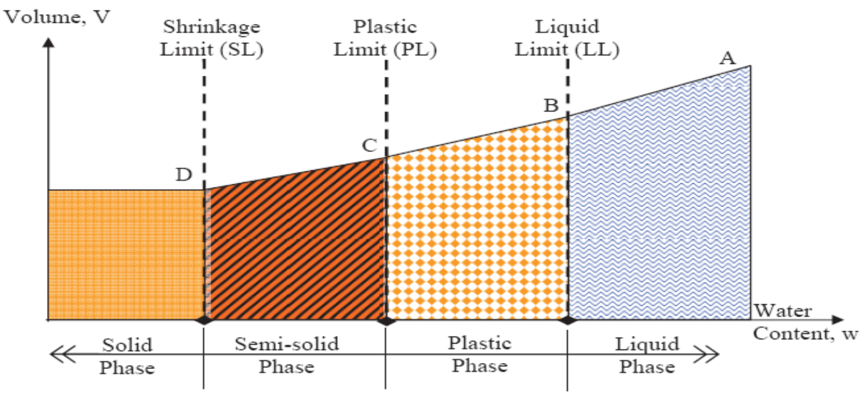

Figure 1.2. Conceptual changes in soil phases as a function of water content (FHWA-NHI, 2006)

1.1.4.5. Permeability and compressibility

The permeability of a soil refers to the rate at which water flows through it under the action of a hydraulic gradient. The passage of moisture via the inter-spaces of soils is known as percolation (Balasubramanian, 2017). Permeable soils are porous enough to allow for percolation while impermeable soils do not permit the passage of water. The knowledge of permeability is very essential for seepage, drainage and groundwater problems and also to determine the rate of settlement on saturated soils. The compressibility of a soil is the decrease in volume per unit increase of pressure. Sands, gravels and silts are incompressible because they do not undergo a significant volume change when subjected to compression. Clays on the other hand are compressible. The compressibility of sand and silt varies directly with the water content of the soil and inversely with cohesion.

1.1.4.6. Shrinkage and swell

Some soils suffer large volume changes with changes in the soil moisture content. They either shrink with decreasing moisture or swell with an increase in moisture. These changes can be problematic where structures are constructed on such soils as they can cause settlements and add extra loads to the structure which was not accounted for in the design.

1.1.4.7. Soil shear strength

Soils are made up of individual particles that can slide and roll relative to one another The strength of a soil is highly stress-dependent and frictional in nature and is also a function of the soil density. It is defined as the maximum value of shear stress that can be mobilized within a soil mass without failure taking place (Duncan et al.2014). Under external pressure, a saturated soil sample will contain water which is also under pressure. This external pressure is considered as the total stress σ and the pressure to which the water is subjected is called the pore water pressure u. The effective stress σ' is the force transmitted through particle contacts divided by the surface area. From the principle of effective stress (Terzaghi, 1936) the effective stress can also be obtained by subtracting the pore water pressure from the total stress (equation (1.9)).

$$\sigma' = \sigma - u \tag{1.9}$$

Shear strength parameters are used to analyze the stability of slopes, excavations, and footings. Compared to other building materials, the strength of soils can be classed as low, extremely variable, and subject to change with time and natural and operating conditions (Holtz, 1974). The shear strength of water is zero. For an effective stress analysis, given c' the effective cohesion, σ' the effective stress and ϕ' the effective angle of internal friction, the shear strength τf of the soil can be obtained as shown in equation (1.10) from the Mohr-Coulomb theory. The angle at which soil resists shearing is termed its friction angle. The cohesion of the soil is the internal bond within the soil, which is not a function of the overburden pressure.

$$\tau f = c' + \sigma' \tan \phi' \tag{1.10}$$

Depending on the conditions under which the soil is loaded to failure, the shear strength can either be drained or undrained (c_u). The drained shear strength of a soil is the shear strength under conditions where water flow into and out of the soil is as rapid as the loading or unloading rate. Describing a soil as being drained does not imply the soil is dry. A completely saturated soil can be drained even though its voids are completely filled with water. The shear strength for conditions in which the water content and volume of a soil under rapid loading does not change is known as the undrained shear strength. c_u is equal to the cohesion value of the Mohr-Coulomb envelope for total stresses and is influenced by the degree of over consolidation.

1.1.5. Soil classification

Soils have been grouped into categories based on certain physical characteristics and engineering properties. In order to describe and classify soils, a knowledge of their grading and plasticity is essential. Over the years, various classification systems to categorize soils based on their particle size have been developed.

1.1.5.1. ASTM-USCS

According to the ASTM-USCS classification system, soils are divided into two categories: coarse-grained (cohesionless) soils and fine-grained (cohesive) soils (Powrie, 2004). According to their particle sizes, coarse-grained soils have been further subdivided into gravels and sands while fine-grained soils include clays and silts. Fine-grained soils have low load-bearing capacities and poor drainage qualities and changes in moisture conditions strongly influence their volume-change characteristics and strength. Coarse-grained soils on the other hand are often described as free-draining soils and they exhibit high bearing capacities and shearing resistance. Two important coefficients are used to distinguish soils based on their particle size: the uniformity coefficient Cu (equation (1.11)) and the coefficient of curvature Cc (equation (1.12)). Poorly graded soils have uniformity coefficients less than 4 and steep gradation curves. Well-graded (wide assortment) soils have uniformity coefficients equal to 4, coefficients of curvature between 1 and 3, and flat gradation curves. Gap-graded soils have coefficients of curvature outside the range of 1 to 3, and a sudden change in the gradation curves.

$$Cu = \frac{D_{60}}{D_{10}} \tag{1.11}$$

$$Cc = \frac{(D_{30})^2}{D_{10}D_{60}} \tag{1.12}$$

Where D_x is the diameter of the soil particles for which x% of the particles are finer.

1.1.5.2. AASHTO

According to AASHTO, granular soils are soils in which 35% or less are finer than the No. 200 sieve (0.075 mm) in the sieve analysis. Fine soils are soils in which more than 35% are finer than the No. 200 sieve. The AASHTO system classifies soils into seven major groups, A-1 through A-7. The first three groups, A-1 through A-3, are granular or coarse-grained soils, while the last four groups (A-4 through A-7) are fine-grained soils. Table 1.1 shows the various soil types and their average grain sizes.

Table 1.1. Soil types, average grain size and description according to AASHTO (Budhu 2015)

Soil type	Average grain size
Gravel	75 mm to 2 mm (No. 10 sieve)
Sand	2 mm (No. 10 sieve) to 0.075 mm (No. 200 sieve)
Silt and clay	<0.0075 mm (No. 200 sieve) Silty: PI < 10% Clayey: PI > 11%

1.2. Landslides

Leroueil (as cited by Hungr et al., 2013) described a landslide as a physical system that develops in time through several stages. As reviewed by Skempton and Hutchinson (1969), the history of a mass movement comprises pre-failure deformations, failure itself and post-failure displacements. Several definitions of a landslide have been proposed by different authors. A simple definition of a landslide is "the movement of a mass of rock, debris or earth down a slope" (Cruden, 1991). Elaborated in this part are the various features associated with landslides, the classification of landslides, causes of landslides and the effects of landslide occurrences.

1.2.1. Landslide features

The different parts of a landslide depend on the type of mass movement in reality. Some of the common terms associated with the basic parts of a landslide are illustrated in Figure 1.3. They include the crown, the head, the main body, toe, foot, flank, main scarp, minor scarp, surface of rupture and toe of surface of rupture

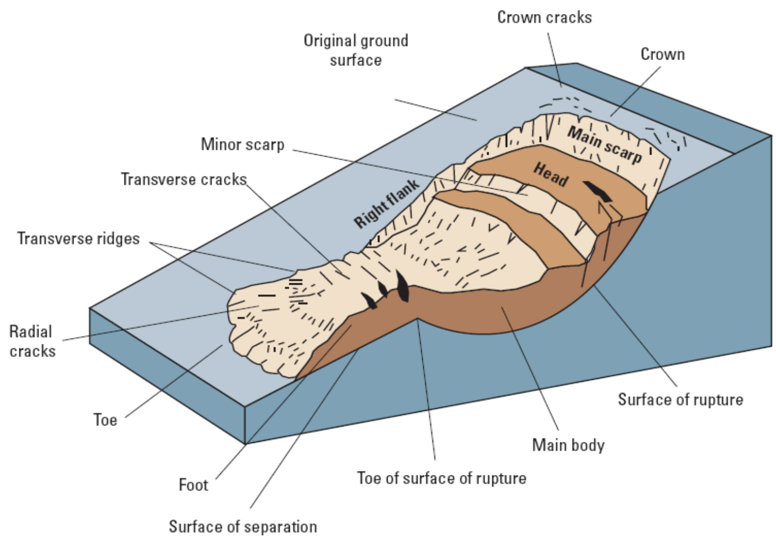

Figure 1.3. Parts of a landslide (Highland & Bobrowsky, 2008)

1.2.2. Landslide classification

Having an idea about the type of landslide will provide some information on the speed and volume of displacement of the slide masses, the potential damage the slide can cause and help in the decision of what mitigation measure to be adopted. Several authors have developed classification methods for slope movement. Bolt et al. (1977) elaborated the classification of landslides based on certain factors which are elaborated subsequently.

1.2.2.1. Classification based on the type of material involved

The most obvious difference between various landslide types is the nature of the participating material (Bolt et al., 1977). Material state and type play an important part in the classification of landslides. As for the state of the material, it could be intact or jointed, fresh or considerably weathered. The material in a landslide mass is either rock, soil, or ice or a mixture of these (Hungr et al. 2013). Slides composed of ice are known as avalanches. Soil is described as earth if mainly composed of sand-sized or finer particles and debris if composed of coarser fragments (Cruden and Varnes, 1996).

1.2.2.2. Classification based on the rate of movement

In order to take into account the effects on people and engineering works, the speed at which a landslide develops and moves is a very important feature to be considered. Various landslide velocities can be classified in terms of the time available for people to shelter, or take remedial action against the slide. Amongst these can be distinguished rapid, intermediate and slow-moving landslides. Rapid landslides are landslides that occur in seconds to minutes. Landslides that occur within a time frame of minutes to hours may be grouped as those with Intermediate rates of movement. Slow landslides develop and move in periods ranging from days to years. A landslide that spans over hundreds of meters to kilometers is classified as rapid even if it occurs over a period of minutes to hours because it is difficult to escape. The velocity of a landslide is related to the slope material behavior and to the processes leading to the occurrence of the landslide. For instance, in mountainous regions, earthquakes are usually accompanied by landslides and rockfalls which are extremely rapid.

1.2.2.3. Classification based on the shape of the slide surface

According to the shape of the slide surface, Savarenski (1939) divided landslides into *asequent*, *consequent* and *insequent* landslides. Asequent landslides develop in homogeneous cohesive soils along curved, approximately cylindrical surfaces. Consequent landslides move on bedding planes, joints or planes of schistosity dipping downslope. Insequent landslides run transversely to bedding and are generally of large dimensions and the slide surfaces extend deep into the slope.

1.2.2.4. Classification based on the mode of the sliding movement

Generally, in a landslide, the sliding mass of material can be clearly distinguished from an underlying stationary bedrock or stable soil layer which does not take part in the motion. In most cases, there exists a sliding or shearing surface along which the displacements occur.

Varnes (1978) established 5 principal types of mass movements describing the actual internal mechanics of how the landslide mass is displaced (Figure 1.4).

a. Falls

Here, a mass of rock or soil is detached from the slope along a slide surface with little or no shear displacement taking place. The loosened rock mass in this type of landslide is in a high-velocity free fall for the greater part of the distance covered during the movement and it goes down the slope with either a rolling or bouncing movement. The falling masses therefore possess high kinetic energy and the rock debris scatters upon impact with the ground and may begin moving on steeper slopes and continue until the terrain flattens or the mass encounters an obstacle.

b. Slides

With this type of landslide, the movement of the rock mass is about a point above its center of gravity. Slides occur mostly along surfaces of rupture or on relatively thin zones of intense shear strain. Usually, an indication of a slide movement is usually the occurrence of cracks in the original ground surface along which the slide will form. The volume of displacing material enlarges from an area of local failure. Depending on the mode of the slide movement, rotational and translational slides can be distinguished.

i. Translational slide

With translational slides, the rock mass moves predominantly along more or less planar or gently undulating surfaces. Translational slides usually follow discontinuities such as faults and the contact between rock and residual or transported soils. This slide movement will continue to progress if the surface of separation of the sliding mass and the slope is sufficiently inclined. A block slide is a translational slide that involves the displacement of a mass consisting of a single unit or a few closely related units that move downslope in the form of a relatively coherent mass. The slide head may be separating from stable rock along a deep, vertical tension crack. Block slides are usually extremely rapid.

ii. Rotational slide

Here, the mass moves along a curved rupture surface which could either be cylindrical or circular. In the case of a cylindrical rupture surface, the axis about which the slide rotates is parallel to the axis of the rupture surface and the slide extends considerably in the direction perpendicular to the direction of motion of the slide. A slide with a circular rupture surface implies very little internal deformation of the mass of soil or rock and the mass moves with its head almost vertically downward meanwhile the upper surface of the displaced material tilts back towards the scarp. The rotational mechanism is self-stabilizing as the gravitational driving forces diminish with increasing displacement and the movement tends to bring the sliding mass to conditions of equilibrium. For this reason, most rotational slides in rock tend to move at slow or moderate velocities.

c. Topples

The movement here is such that the displaced mass from the slope overturns about a point below its center of gravity. Topples range from extremely slow to rapid and sometimes, the mass accelerates throughout its movement. The driving force of the mass in a toppling movement is gravity exerted by the material upslope of the displaced mass. Sometimes toppling is due to water or ice in cracks in the mass.

d. Lateral spreads

Lateral spreads are landslides in which lateral extension movements occur in fractured mass. They may result from the flow of softer materials or from liquefaction. Two modes of spreading have been defined by Varnes (1978) which are block spreads and liquefaction spreads. These differ with respect to the slope material. With block spreads, a layer of hard rock overlying a soft material may crack, making way for the underlying material to be squeezed out alongside the fill broken material. Liquefaction spreads form in sensitive clays and silts that have lost their strength due to disturbances that damaged their structure.

e. Flows

The flow movement depends on the kind of surface in which the slide is occurring. In bedrock, flows include spatially continuous deformations and surficial deep creep involving extremely slow and generally non-accelerating movements among relatively intact units. In soil, flows occur within a displaced mass, and it moves like a viscous liquid. Five categories of flows can be distinguished:

i. Debris flow

Debris flows are mass movements that involve a combination of rock, loose soil, air, organic matter and water mobilized as slurry flowing downslope and forming thick muddy deposits on valley floors. They are rapid mass movements and are commonly caused by intense surface-water flow due to heavy precipitation or rapid snow melt that erodes and mobilizes loose soil or rock on steep slopes.

ii. Debris avalanche

This movement type involves large volumes of moving mass usually composed of unconsolidated material and occurs when the flank of a mountain or volcano collapses and slides downslope. It is a variety of very rapid to extremely rapid debris flow and is therefore highly destructive. Some debris avalanches can travel close to 100 m/s.

iii. Earthflow

Earthflows usually occur in fine-grained materials or clay-bearing rocks on moderate slopes and have a characteristic "hourglass" shape. The slope material liquefies and runs out, forming a bowl or depression at the head. Slides or lateral spreads may also evolve downslope into earthflows. Earthflows may be triggered by saturation of soil and can range from very slow to rapid and catastrophic.

iv. Mudflow

A mudflow is an earthflow made up of material that is wet enough to flow rapidly and that contains at least 50 percent sand, silt, and clay-sized particles. In some instances, mudflows and debris flows are commonly referred to as "mudslides."

v. Creep

Creep is the unnoticeably gradual, steady, downward movement of slope-forming soil or rock initiated when shear stress is sufficient to produce permanent deformation but too small to produce shear failure. Creep is the informal name for a slow earthflow and can be regarded as seasonal, where movement is within the depth of soil affected by seasonal changes in soil moisture and soil temperature; continuous, where shear stress continuously exceeds the strength of the material; and progressive, where slopes are reaching the point of failure as other types of mass movements. This mechanism has been used to explain many cases of slope failure occurring many years after the slope has been loaded either by excavation or filling (Bolt et al., 1977).

f. Complex slides

Varnes (1978) added a sixth class to accommodate landslides with characteristics of a combination of two or more principal types. Complex landslides exhibit two or more of the five principal types of movement illustrated above.

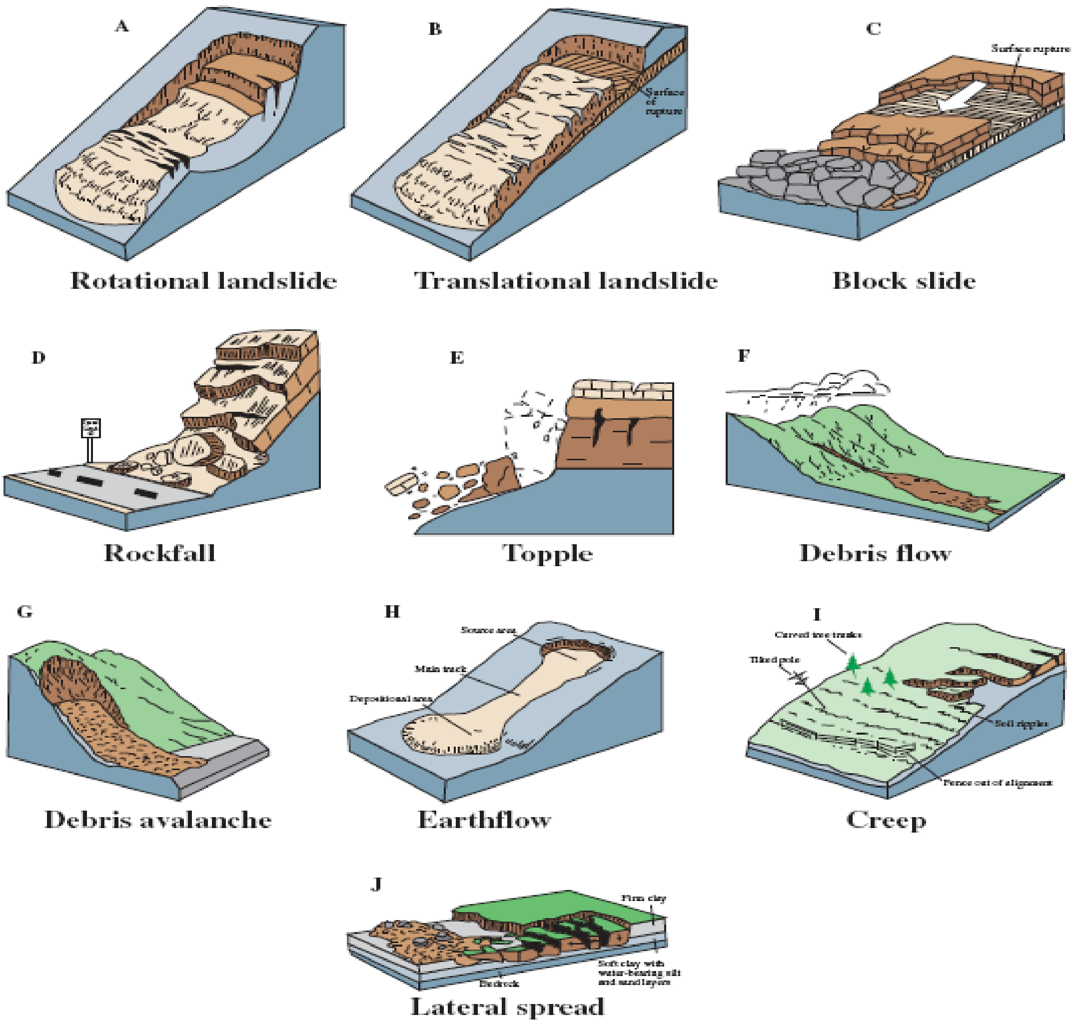

Figure 1.4. Classification of landslides according to their mode of movement (Highland & Bobrowsky, 2008)

1.2.3. Factors contributing to slope failures or landslide occurrence

Several processes underlie the changes that lead to slope failures and can either be natural or can be caused by human activity.

1.2.3.1. Natural Factors

The impact of all the natural causes varies widely and depends on several factors. The natural causes include geomorphological processes, geological factors and physical factors.

a. Geomorphological processes

The geomorphological causes of landslide occurrence include all causes related to morphological changes such as erosion and deposition.

i. Erosion

Erosion by running water, streams, rivers, waves or currents, ice or wind removes the toe and lateral slope support of regions prone to landslides. Erosion by water also leads to the weakening and exposure of slope materials to sliding.

ii. Vegetation removal

The vegetation of a slope plays an important role in holding the soil in place by providing a root system of trees and other plants. This vegetation can be lost to fire, erosion or drought making the slope more susceptible to land sliding.

iii. Tectonic or volcanic uplift

Terrain uplift associated with plate tectonics directly implies soil movement. Large earthquakes can cause mountainous topography by inducing rock uplift but can also erode mountains and lead to the occurrence of landslides. The shaking of the soil profile by either an earthquake or volcanic eruption decreases the resisting forces along the potential slip planes.

iv. Deposition loading of the slope or its crest

Deposition of materials such as earth, snow and hail on a slope subject it to additional loads leading to an increase in the driving forces and reduction in the slope’s stability thereby making it more susceptible to a slide.

b. Geological Factors

These are the causes related to the ground conditions or characteristics of the slope mass and materials. The soil may be weak or fractured, or its different layers may have varying strengths and stiffnesses. Geological factors may account for 43% of landslides (Ram & Maurizio, 2020).

i. Contrast in permeability and/or stiffness of materials

A good number of slides occur in geologic settings where permeable layers of sand and gravel overlie impermeable layers of silt and clay or bedrock. A zone of weakness is created when water seeps down the overlying permeable layers and accumulates on top of the underlying strata leading to the occurrence of the slide. Other slides could occur in slopes with dense material lying over plastic material.

ii. Weak or sensitive materials

Landslides are more likely to occur in weak materials due to their low shear strength. Materials may become weak due to change in water content, fissuring, weathering and other physiochemical reactions. Disturbance or remolding of sensitive materials such as dry or saturated loose sand and loess contributes to their low shear strength.

iii. Weathered materials

Rock and indurated soils are subject to strength loss as a result of weathering which involves various physical, biological and chemical processes (Mitchel, 1993). Materials that have been exposed to deterioration become weak and prone to landslides.

c. Physical factors

Slope saturation by water is a primary cause of landslides. Saturation can occur in the form of intense rainfall, snowmelt, changes in ground-water levels, and surface-water level changes along coastlines, earth dams, and in the banks of lakes, reservoirs, canals, and rivers.

i. Heavy rainfall and prolonged precipitation

Rainfall is recognized as one of the most important triggers of landslides. The increase in groundwater level leads to the building up of pore water pressure and decrease in shear strength, thus facilitating slope failure. Groundwater conditions responsible for slope failures are related to rainfall through infiltration, soil characteristics, antecedent moisture content, and rainfall history. Intense, short-period rainfall can lead to slope saturation by water which is a primary cause of landslides.

ii. Freeze-and-thaw weathering

Frequent freezing and thawing activity can lead to the development of fractures in rock, degradation of the physical and mechanical properties of rock-soil mass, reduce slope strength and thus easily result in landslide incidents when a trigger mechanism overloads the slope.

iii. Shrink-and-swell weathering of expansive soils

Expansive soils are soils that are capable of shrinking and/or swelling and thus change in volume with fluctuation in their moisture content. Repeated cycles of swelling and/or shrinkage of soil lead to the development of cracks and fissures which expose the subsoil or clay layers along the fissures to lubrication thus encouraging downhill mass movement.

1.2.3.2. Anthropogenic Factors

Anthropogenic or man-made processes include all kinds of human activities that influence slope stability. They include the construction of highways, development of housing infrastructure on hillsides, dam constructions, reservoirs, drainage and utility structures usually involve the movement of substantial amounts of soil or rock on slopes (Bolt et al., 1977). If the construction or operation involves addition of material to the top of the slope or removal of soil or rock from its base, then the slope is pushed toward failure as a result of an increase in the driving forces. The addition of fill material across a hillside for a road may not necessarily cause the failure of the slope due to its weight but may also cause artificial steepening of the slope or interfere with the natural water flow regime and drainage through the soil or rock. In this way, the weight of the slope material is perhaps increased or the water pressure is changed in the pores of the soil or the rock interstices giving rise to the occurrence of a slide within months to years after the completion of construction. Other human activities that can trigger landslide occurrences include the excavation of slope or its toe, creation of cuts in a slope, deforestation, irrigation, mining, artificial vibration and water leakage from utilities.

1.2.4. Effects of landslides

The occurrence of landslides is relevant to the built and natural elements. This section elaborates on some major effects of landslides.

1.2.4.1. Effects of landslides on the natural environment

Landslides like every other natural disaster are not without damaging effects. Pointed out in this section are some of the consequences of landslide occurrences.

a. Effects on the morphology of the earth surface

The main geomorphic effect of landslides is the reduction of slopes to angles at which they possess long-term stability. In mountain valleys or at slope toes, they have the effect of raising the earth's surface as a result of deposition of slope materials in these regions thereby changing their forms and shapes (Lawrence & Aaron, 2013).

b. Effects on forests and grasslands

Landslides can wipe out large tracts of forest and vegetation upon their occurrence while carrying downslope all of the fertile soil (Geertsema et al., 2009). Since no soil is left for new plants to grow, the evidence of deforestation is visible.

c. Effects on native fauna

The occurrence of landslides poses a threat to wildlife. They eliminate their food and water supplies, habitats, and in some cases lead to their eradication (Geertsema et al., 2009).

1.2.4.2. Effects of landslides on the built environment

On a construction site, a landslide occurrence is a great deficit to the project at hand. The project may face design modifications, additional expenditure and may take a longer time to get to completion since its progress has been interrupted. An indirect adverse effect of slope movement is the situation in which a landslide blocks a valley, thus giving rise to a temporary lake, which then endangers the downstream area with a possibility of flooding (Highland & Bobrowsky, 2008; Quido & Vojtech, 1982). If during the construction of highways that cross areas that are highly susceptible to landslides the stability of slopes was disturbed, these infrastructures frequently get interrupted by slope movements, be it natural or artificial slopes. Landslides can obstruct traffic and significantly increase road maintenance costs (Highland & Bobrowsky, 2008).

1.3. Slope stability

Landslide occurrences are directly linked to the instability of slopes, whether natural or man-made. The major indicator of slope stability is the factor of safety. This section gives an idea on the notion of factor of safety, an overview of slope stability analysis methods, the types of slope stability analysis and the conditions to be taken into consideration when doing a slope stability analysis.

1.3.1. Factor of Safety

Conventional analysis procedures characterize the stability of a slope by calculating a factor of safety. Appropriate factors of safety are required to ensure the adequate performance of slopes throughout their design lives. There exist several means of formulating the factor of safety (FoS). The choice of a design safety factor for a particular project depends on the method of stability analysis used, the method used to determine the shear strength, the degree of confidence in the reliability of subsurface data, the possible consequences of a failure and how critical the application is. The most common formulation for FoS assumes that the factor of safety is constant along the slip surface (Cheng & Lau, 2014), and is defined with respect to force equilibrium or moment equilibrium as shown in Figure 1.5.

1.3.1.1. Factor of safety in terms of shear strength

The factor of safety here is defined as the ratio between the ultimate shear strength (τf) and the mobilized shear stress (τm) of a slope at incipient failure (equation (1.13).

$$FoS = \frac{\tau f}{\tau m} \tag{1.13}$$

1.3.1.2. Factor of safety for moments

The moment equilibrium is generally applied for the analysis of landslides where the sliding movement type is rotational. With consideration of a slip surface, the safety factor FoS_m defined in terms of moments is given by equation (1.14). The reference point for the moment is chosen depending on the type of slip surface. For a circular failure surface, the center of the circle is usually taken as the moment point for convenience. For a non-circular failure surface, an arbitrary point for the moment consideration may be taken in the analysis.

$$FoS_m = \frac{\text{Sum of resisting moments}}{\text{Sum of driving moments}} \tag{1.14}$$

1.3.1.3. Force equilibrium

The force equilibrium is generally applied to rotational or translational failure types having plane or polygonal slip surfaces. The stability of a slope is estimated in terms of forces as the ratio of the forces acting to resist failure (resistive forces) and the forces acting to cause failure (driving forces) as in equation (1.15).

$$FoS = \frac{Resistive\ forces}{Driving\ forces} \tag{1.15}$$

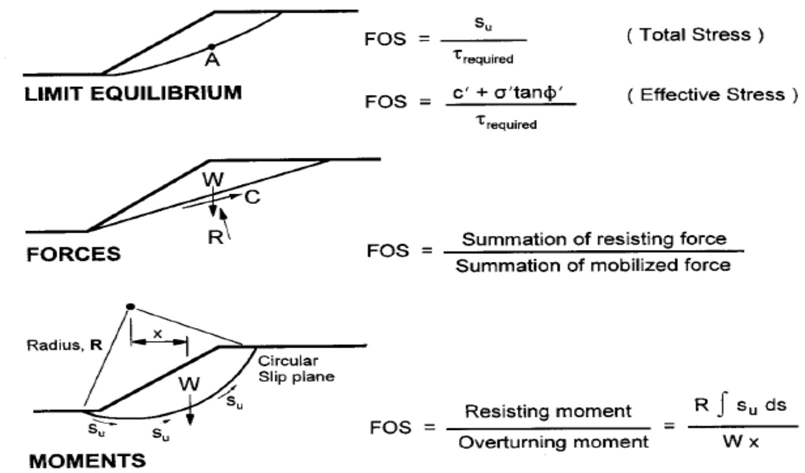

Figure 1.5. Various definitions of factor of safety FoS (Abramson et al., 2002)

The minimum allowable factor of safety of a slope depends on the following factors (Duncan et al., 1987):

- The degree of inaccuracy of the shear strength measurements, slope geometry and other conditions. The uncertainty of the strength parameters is greatest when the soil conditions are complex and when the available strength data do not give a complete, consistent or logical picture of the strength characteristics.
- The costs of flattening or lowering the slope in order to make it more stable
- The costs and consequences of a slope failure on the site
- Whether the slope is temporary or permanent

1.3.2. Slope stability analysis methods

All slope failures are 3D in nature, but 2D modeling is usually adopted as this greatly simplifies the analysis. Before setting out to check the stability of a slope, it is necessary to have an idea of what the slope is like and what it consists of, its height, slope angle, whether it has berms and is served by a drainage system or not and also it is essential to know its history, both in terms of its geological past, whether it has suffered failure or distress, and whether it has been engineered before (Cheng & Lau, 2014).

1.3.2.1. Limit equilibrium method

The traditional LEM is the most common technique for the analysis of slopes due to its simplicity and accuracy (Cheng & Lau, 2014). The LEMs consist of cutting the slope into fine slices which can be vertical, horizontal or inclined and then applying appropriate equilibrium equations. In the classical LE approach, the user has to define a trial slip surface before working out the stability.

The first slice technique (Fellenius, 1927) known as the Normal or Ordinary Method of Slices was based more on engineering intuition than on a rigorous mechanics principle. The Ordinary Method of Slices ignores both interslice shear (IS) and interslice normal (IN) forces and satisfies only moment equilibrium. It can easily be performed by hand calculations and is also a method by which the computation of driving and resisting forces is straightforward and easily demonstrated. The emergence of electronic computers in the 1960s made it possible to handle the iterative procedures involved in the method more readily, which led to mathematically more accurate formulations such as those developed by Morgenstern and Price (1965) and by Spencer (1967). The Bishop Simplified method ensures the equilibrium of moments and was initially developed for circular slip surfaces, but the assumptions inherent in this method can be applied to any noncircular slip surface. Janbu extended the circular slip to a generalized slip surface (Janbu, 1973). Morgenstern and Price (1965) developed a method that ensures that moments and forces equilibrium are achieved simultaneously. All limit equilibrium methods use the Mohr-Coulomb expression to determine the shear strength (τf) along the sliding surface. The limit equilibrium method is based on the following assumptions (Nash, 1987):

- The mass of soil is considered perfectly rigid
- Slope instability occurs for sliding of a mass of soil along a surface
- The resistance of the soil is fully mobilized along the entire surface (F=cost)

1.3.2.2. Infinite slope analysis

The infinite slope method is a 2D slope stability analysis like all limit equilibrium methods. A slope that extends for a relatively long distance and has a consistent subsurface profile may be analyzed as an infinite slope. It may also be described as a slope long in comparison with the thickness of potentially unstable material. It is also long in the sense that conditions at the top and bottom of the slope are far enough away to have little effect on the stability of the sloping surface. This situation usually occurs when the potentially unstable layer is relatively thin and overlies a much stronger material. The failure plane for this case is parallel to the surface of the slope (Figure 1.6) and the limit equilibrium method can be applied readily. The infinite slope procedure is applicable to homogeneous cohesionless slopes and slopes where the stratigraphy restricts the slip surface to shallow depths and parallel to the slope surface (Cheng & Lau, 2014).

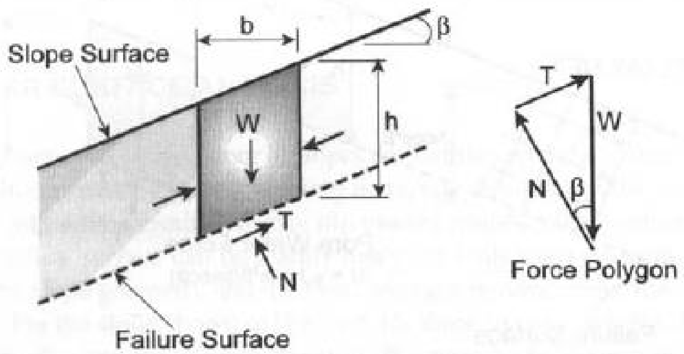

Figure 1.6. Infinite slope failure in dry sand (Abramson et al., 2002)

1.3.2.3. Finite element method

The application of finite element method (FEM) of analysis in geotechnical analysis has become increasingly common with the improvement of computer performance (Abramson et al., 2002). The strength reduction method (SRM) is generally used to perform stability analysis using the FEM code and is based on the reduction of the cohesion and the tangent of the friction angle of the soil (Cheng & Lau, 2014). The FEM essentially divides the soil into discrete units called finite elements, which are interconnected at their nodes and at predefined boundaries of the continuum (Figure 1.7). The parameters are reduced in steps until the soil mass fails. Finite element methods are advantageous in that, they model slopes with a degree of high realism (that is slopes with complex geometry, presence of material for reinforcement, action of water and have laws for complex soil behavior) and to better visualize the deformations of the soils in place.

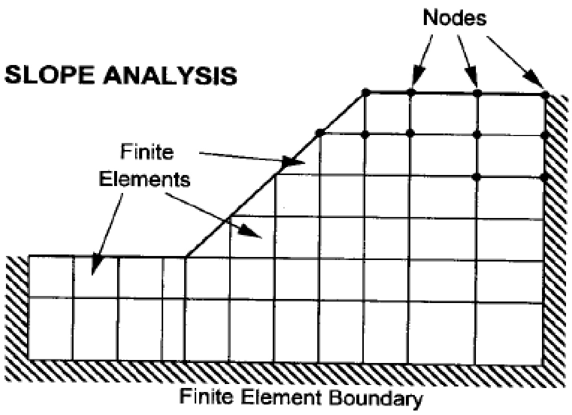

Figure 1.7. Definition of terms used for FEM (Abramson et al., 2002)

1.3.3. Types of slope stability analysis

Depending on the dominant conditions in the soil, there are two different ways of analyzing the stability of slopes (Cheng & Lau, 2014). They are the total stress approach and the effective stress approach.

1.3.3.1. Total stress approach

Otherwise known as the 'ϕ=o analysis', the total stress analysis is applicable only in undrained conditions for example in clayey slopes or slopes with saturated sandy soils under short-term loadings where the pore pressure is not dissipated. Here, the shear strengths are related to the total stresses and therefore it is not necessary to evaluate the pore pressures when performing the total stress analysis.

1.3.3.2. Effective stress approach

Applies to long-term stability analysis in which drained conditions prevail. With the effective stress analysis, the pore pressures along the shear surface are subtracted from the total stresses to determine the effective normal stresses, which are used along with c' and ϕ' to evaluate the shear strengths. Knowing the pore pressure at every point along the shearing surface is necessary when performing an effective stress analysis. Natural slopes and slopes in residual soils should be analyzed with the effective stress method considering the maximum water level under severe rainstorms (Cheng & Lau, 2014).

1.3.4. Drained and undrained conditions for stability analysis

The local geological details of a site are very essential and have a considerable influence on the performance of a slope. In order to understand and formulate slope stability problems, it is necessary to know the conditions under which the analysis will be performed. In the design of geotechnical systems, drained and undrained conditions should be considered in order to determine which of them is critical.

1.3.4.1. Drained Conditions

Water is an important factor in slope stability problems. It can influence the stability of a slope by generating pore water pressures which alter stress conditions in the soil and lead to a subsequent decrease in shear strength, developing both internal and external erosions, reducing apparent cohesion due to saturation and changing the bulk density and mineral constituents of slope forming materials. Drained conditions are those where load changes are slow enough or where they persist long enough so that the soil reaches a state of equilibrium without any excess pore pressures caused by the loads. The water within the soil may be static or it may be seeping steadily with no change in seepage over time and no increase or decrease in the amount of water in the soil (Duncan et al., 2014). The pore pressures here are controlled by the hydraulic boundary conditions. A drained analysis is applicable if these conditions prevail in all of the soils at the site or can be reasonably approximated by these conditions.

1.3.4.2. Undrained conditions

Undrained conditions are those in which changes in the load occur faster than the flow of water into or out of the soil. The behavior of the soil in response to changes in external loads governs the overall changes in pore pressure. Undrained conditions are typical of fine-grained soils due to their low permeability, and also in coarse-grained soils under very fast loading rates such as in earthquakes (Duncan et al., 2014).

1.4. Design approaches for slope stability

The provisions of EN 1997-1 for slope design are contained in section 11 of the code which applies for the overall stability of slopes and movements in the ground whether natural or fill, around foundations or retaining structures, embankments, natural slopes or excavations. The ULS is checked using the GEO/STR design values of actions, resistance and strength with the appropriate values for partial factors. Concerning serviceability, the slope must be designed such that deformations in the ground will not cause serviceability limit states of the infrastructures on or around the slope to be reached. This can be done either by limiting the mobilized shear or by using observational methods. All relevant failure modes are to be taken into account (Orr, 2013). DA1.C2 and DA3 are the same in the design of slopes as loads on the surface in DA3 are treated as geotechnical actions using the A2 set of partial factors on actions, as in DA1.C2. With slope stability, part of the soil weight is considered favorable while the other is considered unfavorable (Figure 1.8). The soil weight components are seen as coming from the same source and so the same partial factors are applied to the favorable and unfavorable weights alike.

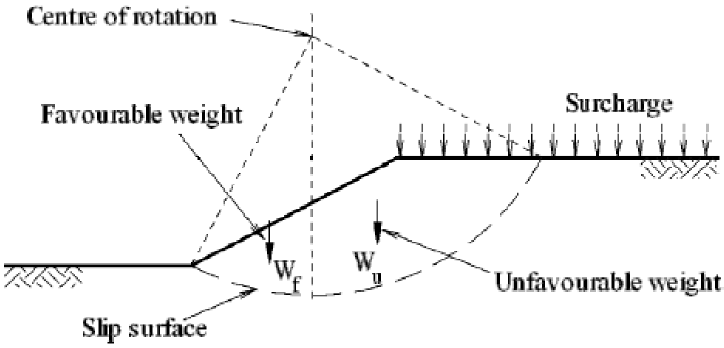

Figure 1.8. Illustration of the soil weight components in a sliding mass (Orr, 2013)

1.5. Slope stabilization techniques

The stabilization strategy focuses on the implementation of engineering works to reduce the likelihood of occurrence of landslides. This section presents some categories of slope stabilization techniques.

1.5.1. Unloading

Generally, the forces tending to cause movements downslope are gravitational therefore reducing the slope soil mass is a simple approach to increasing the slope stability. Two methods of unloading and are discussed here.

1.5.1.1. Excavation

Improving stability by excavation requires that an area at the top of the slope can be sacrificed to improve stability, that the site is accessible to construction equipment and that an area is available for disposal of the excavated material (Duncan et al., 2014). Included in this category are benching, reduction of the slope height and removal of soil from the head of a slide

a. Benching

The benching technique involves the cutting out of a series of steps into a deep soil or rock face. Benches are useful in providing protection for infrastructures situated beneath rockfall-prone cliffs, controlling surface drainage, or for providing a work area for installing drainpipes or other structures (Abramson et al., 2002). They are mainly effective in reducing the incidence of shallow failures but generally are not very efficient in improving the overall slope stability for which other methods are recommended (Highland & Bobrowsky, 2008).

b. Reducing the height of the slope

The reduction of the height of a slope decreases the weight of the soil mass thereby decreasing the driving forces acting on the failure surface. Additional modification of the land surface may be a complementary solution to this technique in order to make it more efficient (Highland & Bobrowsky, 2008). Flattening or reducing slope angle, or other slope modification reduces the weight of slope material and reduces the possibility of undercutting of the slope by rivers or streams or construction loading.

c. Removal of soil from the head of a slide

Partial removal or excavation of a sufficient quantity of slope material at the head of a slide helps ensure the stability of the potential sliding mass. In slopes prone to rotational landslides, removing soil by cutting deep into the slope is suitable to improve the slope stability. This method is ineffective if applied for translational failures on long, uniform or planar slopes or flow-type landslides (Highland & Bobrowsky, 2008).

1.5.1.2. Lightweight fill

Lightweight materials are especially used in embankment construction and play the role of reducing the gravitational driving forces tending to cause instability (Abramson et al., 2002). The technique consists of excavating the upper soil and replacing it with a lightweight backfill material such as encapsulated saw dust, expanded shale, cinders, shredded rubber tyres, polystyrene foam and seashells. The choice of the fill material depends on the cost and availability in local areas.

1.5.2. Drainage

Drainage systems are the most effective remedial measures against slope instability in saturated soils due to their capacity to reduce pore water pressure in the subsoil (Highland & Bobrowsky, 2008). Drainage improves slope stability by reducing the driving forces of water pressures in cracks. A number of drainage measures to treat the slope surface in order to promote rapid runoff, reduce erosion and improve stability are as presented here.

1.5.2.1. Surface drainage

Good surface drainage is strongly recommended as part of the treatment of any landslide or potential landslide (Cedergren, 1989). Surface drainage plays the role of preventing erosion of the face of the slope, reducing the probability of water ponding on the slope surface, and reducing groundwater pressures by preventing infiltration of water into the soil which if not controlled will lead to localized failures on the slope face (Duncan et al.,2014). Surface runoff is usually collected in permanent facilities such as V- or U-shaped concrete lined or semi-circular corrugated steel pipe channels and diverted away from the slope. These channels should be placed strategically at the head of the slope and along berms (Duncan et al., 2014). Redirection of surface runoff is commonly the first response to a rainfall-induced failure. Surface drainage measures include:

- Establishing lined or paved ditches to convey water away from the site.
- Minimizing water infiltration in the short term by covering the ground surface with plastic sheets and in the long term through the use of vegetation. Sealing cracks with surface coatings such as shotcrete, lean concrete, or bitumen is another way to reduce infiltration by water into the slope.
- Grading in order to eliminate low spots which can promote standing water.

1.5.2.2. Subsurface drainage

The provisions for subsurface drainage help regulate or prevent the excessive buildup of groundwater pressure due to seepage in a slope. Subsurface drainage systems are widely used also in combination with other stabilization works. Subsurface drainage consists of one or more of the techniques outlined below.

a. Horizontal drains

Horizontal drains are perforated pipes inserted in drilled holes in a slope to provide underground drainage. Horizontal drains usually slope upward, to permit drainage of groundwater by gravity (Figure 1.10). The drain pipes are commonly perforated or slotted PVC pipes, although steel pipes were used for early applications. The drains are installed by drilling into the slope with the use of a hollow stem auger or sacrificial drill bit, inserting the drain pipe and withdrawing the auger leaving the drain in place. The hole is allowed to collapse around the drain pipe and there is no filter between the pipe and the soil. Rahardjo et al. (2012) found that horizontal drains are most effective when placed low in the slope, provided that the slope does not contain distinct layers of high permeability above the drains.

b. Subsurface drainage blankets

In a situation where there is a thin layer of poor-quality saturated soil at shallow depth and where there are materials of better quality below that layer, it may be practical to remove the poor-quality layer and replace it with a well-draining soil fill. To minimize erosion on the surface, a drainage ditch may be installed to divert water flow from the pipe to a suitable discharge point.

c. Trenches

Trenches are usually excavated at the steepest stable side slopes for a construction period. Trench drains are excavated trenches backfilled with pervious material that serve as filters in the surrounding soil (Figure 1.9). They may contain a pipe to increase flow capacity and are sloped so that they drain by gravity. Manholes are usually provided at intervals for inspection and maintenance in cases where pipes are used. The maximum depth that a drain trench can extend below the ground surface is determined by the requirement that the sides of the trench must remain stable without support until they are backfilled with drain rock. Trench drains excavated perpendicular to a slope are known as finger or counterfort drains and do not affect the stability of the slope as much as would excavation of a trench drain parallel to the slope (Abramson et al., 2002).

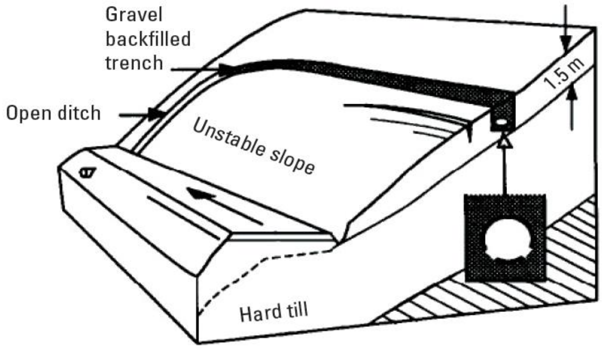

Figure 1.9. Schematic diagram of a ditch and drain trench (Highland & Bobrowsky, 2008)

d. Drainage tunnels or galleries

Where drainage is needed deep within a hillside, a drainage gallery can be used. Drainage tunnels may also be considered when a cut to be dewatered is so large that it requires a substantial number of horizontal drains, when groundwater is at a depth such that it is impossible to reach by open excavation methods, or even when the topography makes the application of horizontal drains impossible. Drains can be drilled outward from the tunnel, extending the drainage through the slope. Drainage tunnels have relatively high construction costs.

e. Drainage wells

Drainage wells have the principal function of lowering the water pressures in layers that are deep down the subsoil. They are vertical holes with perforated pipes placed in them and the annular space between the borehole and the pipe is filled with filter material. The water from the wells is disposed of by using a submersible pump or surface pump with discharge channels.

Alternatively, horizontal grains can be used to tap the drainage wells for water disposal as seen in Figure 1.10.

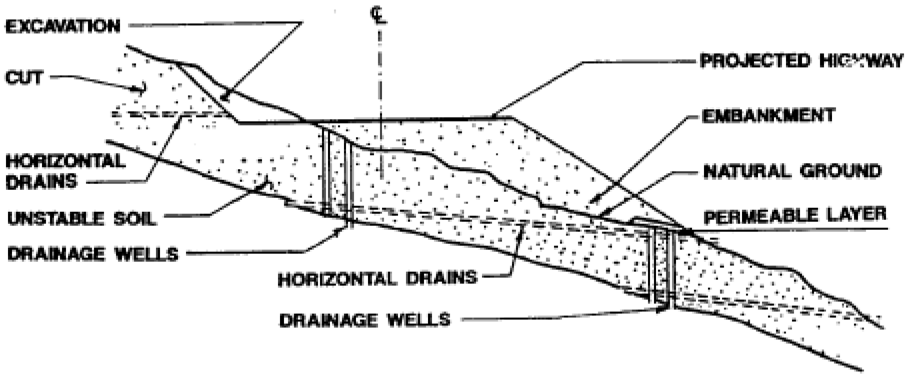

Figure 1.10. Drainage well combined with horizontal drain (Abramson et al., 2002)

f. Cut-off drains

Cut-off drains are very useful in intercepting groundwater flow in areas where shallow groundwater is encountered (Abramson et al., 2002). As illustrated in Figure 1.11, an impermeable zone or membrane is used as a cut-off downslope of the drain and the top zone of the trench is backfilled with impermeable material.

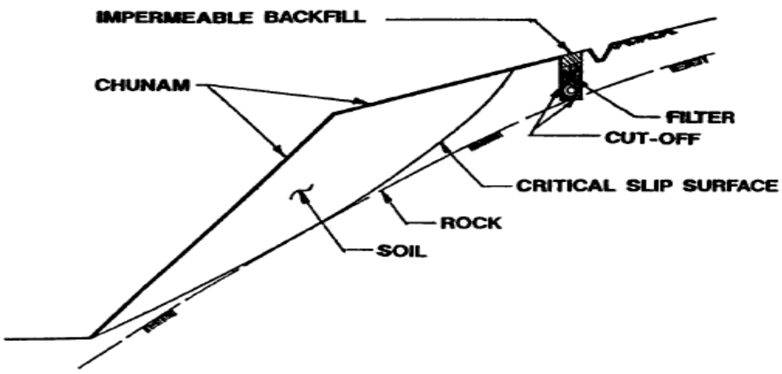

Figure 1.11. Cut-off drain (Abramson et al., 2002)

g. Geosynthetic drainage

Geosynthetics are factory-manufactured polymer materials that are used for earth stabilization and erosion control. Geosynthetics and vegetation are often integrated to provide combined benefits. These materials when used for drainage purposes function by allowing liquid flow and limiting the loss of soil within the plain of the geosynthetic. Geocomposites and geotextiles are the two main types of geosynthetics used for drainage purposes. Geotextiles are permeable fabrics typically made from polypropylene or polyester which can perform the functions of separation, filtration, reinforcement, protection, or drainage when associated with soil. Geocomposites combine the best features of geosynthetics with other materials for optimal performance and cost-effective techniques. Other types of geosynthetics include geonets, geogrids and geomembranes (Figure 1.12).

Figure 1.12. Photographs illustrating the various types of geosynthetics (Brezzi, 2019)

1.5.3. Internal reinforcement

Internal reinforcement can either be done by soil reinforcement or in situ reinforcement. Soil reinforcement involves the inclusion of tensile-resistant elements in a soil mass in order to improve its overall shearing resistance. It is a technically attractive and cost-effective technique for increasing a slope's stability (Abramson et al., 2002). In-situ reinforcement systems allow for the reinforcement of existing soil masses and include soil nailing, application of soil anchors and pin piles.

1.5.3.1. Soil nailing

Soil nailing is a method of in-situ reinforcement with the use of passive inclusions that are mobilized when movement occurs. It is prevalently applied for retaining excavated slope (as temporal or final support), but also in slope stabilization (FWHA, 2003). The nails are generally not pre-stressed (passive) and consequently they do not need particular maintenance care (Cola et al., 2012). The design of nailed slopes and excavations is based on limit equilibrium analysis where critical potential failure surfaces must be assumed. Figure 1.13

illustrates some applications of soil nailing and installation steps. Soil nails are adaptable in all soils and rock and have a low environmental impact (Cola et al., 2012).

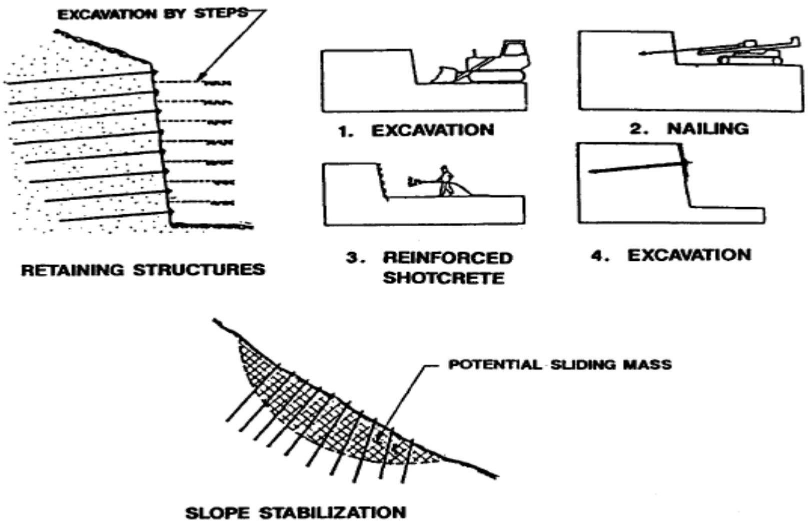

Figure 1.13. Main applications of soil nailing (Abramson et al., 2002)

1.5.3.2. Stone Columns

Stone columns are a series of closely spaced, large-diameter columns of compacted stone that function in increasing the average shear resistance of the soil along the potential slip surface. They are practically efficient in providing a path for the relief of pore water pressures, thereby increasing the strength of the surrounding clay soils. An illustration of a slope stabilized using stone columns is shown in Figure 1.14a.

1.5.3.3. Reticulated micropiles

Micropiles are cast-in-place reinforced concrete piles with diameters ranging from 9 to 25cm. Micropile installations cause less noise and vibration than conventional piling techniques, especially driven piles. The use of micropiles in old urban environments and industrial areas can prevent potential damage to adjacent sensitive structures and equipment due to vibrations pile driving. Micropiles are suitable in all soil types and do not obstruct groundwater flow (Abramson et al., 2002). Reticulated micropile systems are similar to soil nailing systems but differ in that the behavior of the micropiles is significantly influenced by their geometric alignment shown in Figure 1.14b. Field and model tests have demonstrated that the group and network effect of reticulated micropiles system provides higher load bearing and shearing capacity than those of closely spaced vertical piles (Lizzi & Carnevale, 1979). The crisscross pattern used subjects the micropiles to tension and compression forces that provide the required structural stability of the slope (Abramson et al., 2002).

1.5.3.4. Geosynthetically reinforced slopes

Geosynthetic soil reinforcement is a slope stabilization technique particularly used after a failure has occurred or if a steeper-than-safe unreinforced slope is desired. The design of geosynthetically reinforced slopes is based on modified classical LEMs for slope stability (Abramson et al., 2002). The length of the fabric layers is determined from analyses of both internal and external stability. Figure 1.14c illustrates geosynthetic soil reinforcement.

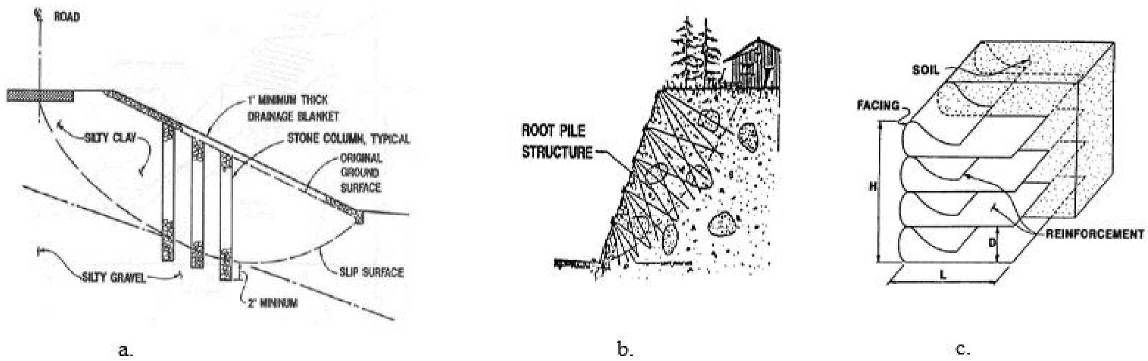

Figure 1.14. a. Stone columns; b. reticulated micropiles; c. reinforced soil using geosynthetics (Abramson et al., 2002)

As shown in Figure 1.15, there are three possible failure modes for reinforced slopes: internal failure where the failure plane passes through the reinforcing elements; external in which the failure surface passes behind and underneath the reinforced mass (the reinforced mass is the mass of soil that contains the reinforcements); and compound failure where the failure surface passes behind and through the reinforced soil mass.

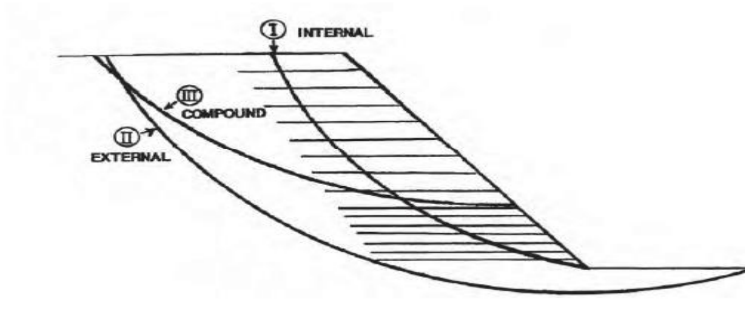

Figure 1.15. Failure modes for reinforced soil slopes (Samtani et al., 2006)

1.5.4. Retaining structures

Retaining structures provide physical resistance to slope movement and may be designed such that they are capable of opposing the destabilizing forces involved, including those due to water. It is considerably difficult for the retention system to block water flow therefore; the walls should be designed with good drainage systems behind or through them.

The major types of retaining structures used for slope stabilization are discussed in the following subsections.

1.5.4.1. Gravity and cantilever retaining walls

Retaining walls are commonly used for slope stabilization where a cut or fill is required and there is insufficient space or right-of-way available for just the slope itself. The wall should be deep enough so that the critical slip surfaces pass around it with an adequate safety factor (Abramson et al., 2002). The retaining wall must be able to withstand shearing, overturning and sliding at its base. Shear keys are sometimes required to provide adequate sliding resistance.

1.5.4.2. Tieback walls

Tieback wall designs use the principle of carrying the lateral earth pressure on the wall with the use of a 'tie' system that transfers the imposed load to a zone behind the existing or potential slip plane where satisfactory resistance can be reached. Tiebacks consist of post-tensioned steel cables, rods or wires attached to deadmen as used in embankments or grouted to a firm, bearing stratum as used in cuts. The length of a tieback is controlled by stability requirements (Abramson et al., 2002). Permanent tiebacks are routinely installed in cohesionless soils but rarely installed in soft cohesive soils due to concerns about their long-term holding ability.

1.5.4.3. Prestressed anchors and anchored walls

Long, prestressed anchors are used as elements to stabilize soil slopes. Prestressed anchors and anchored walls have the advantage that they do not require slope movement before they impose restraining forces. The improvement of stability offered by anchors and anchored walls can be evaluated using conventional limit equilibrium slope stability analysis.

1.5.4.4. Drilled shaft walls

Mostly applicable in urban locations where the presence of existing commercial and private structures limits the types of stabilization methods selected. The shafts are usually placed at 3 pile diameters apart due to arching effects between the drilled shafts. It should be noted that the driving forces on the wall increase as a function of the height squared and so higher walls necessitate greater embedment depth, larger drilled diameters and additional reinforcement to resist overturning moments thus rendering them costlier. Combining this technique with tiebacks or axial post-tensioning helps minimize cost (Abramson et al., 2002).

1.5.5. Buttressing

Buttressing is a technique used to counter or offset the driving forces of a slope by increasing the resisting forces through an externally applied force system (Abramson et al., 2002). Buttresses may consist of one of the following fills:

- MSE which involve the designed use of backfill soil and thin metallic strips, mesh or geosynthetic reinforcement mesh to form a gravity mass capable of supporting or restraining large imposed loads. Reinforced soil is applicable to cases in which the reinforcement and backfill are placed as the slope or wall is constructed. The MSE is usually confined using either metal, shortcrete or reinforced concrete facing.
- Counterberms used to provide weight at the slope toe and increase the shear strength of the soil below the toe.
- Soil and rock fill used to provide adequate dead weight near the toe of an unstable slope to prevent its movement.
- Tyresoil or pneusol which apply old automobile tires as inclusions in the soil mass in place of the metal or non-metal reinforcement in MSE walls

1.5.6. Vegetation and biotechnical stabilization

These techniques are eco-friendlier but vegetation needs time to reach maximum strength and so should be combined with other physical techniques in order to have a combination of immediate and long-term protection.

1.5.6.1. Vegetation

Vegetation contributes to the enhancement of slope stability in several ways but the two most important factors are root reinforcement and rainfall interception and evapotranspiration. Slope stabilization using vegetation only and vegetation combined with structural slope stabilization elements are discussed briefly in this part.

a. Tree root reinforcement

Planting shrubs adds vegetation cover and builds stronger root systems which will in turn enhance slope stability. Before seeding is done, control of surface-water drainage, removing cut-bank overhangs, reducing slope angles, and benching all should be done in order to make the slope more resistant to future erosion.

b. Rainfall interception and evapotranspiration

Rainfall interception and transpiration of moisture by trees tend to maintain drier soils and mitigate or delay the onset of waterlogged or saturated soil conditions which have been known to cause slope failure.

1.5.6.2. Biotechnical Slope Protection

Biotechnical slope stabilization is the stabilization of slopes by the combined use of vegetation and man-made structural elements working together in an integrated manner. Here, reinforced grass systems and woody plants can be used as a means to stabilize slopes. Biotechnical slope-protection systems blend into the landscape and emphasize the use of natural, locally available materials, such as soil, rock, timber, and vegetation, in contrast to manufactured materials such as steel and concrete. This technique is especially effective in the protection against shallow, rainfall-induced landslides.

1.5.7. Other slope stabilization techniques

The following techniques are also included in the category of measures for slope stabilization:

- The application of shotcrete (concrete with aggregates up to 2cm in size sprayed at high velocity) to a slope or cut that may erode over time can serve as a permanent covering. Unreinforced shotcrete can be used to prevent differential erosion between units.
- Masonry blocks or rip-rap to armor, stabilize and protect the soil surface against erosion and scour in areas of concentrated flow or wave energy.
- Hardening of cohesive soils that have low to very low permeability which can be done by mixing cement with the local soil material, drainage by electro-osmosis, thermal treatment, grouting and lime injection.

Conclusion

A necessity for addressing slope instabilities arises considering the significance of the effects of landslide occurrences in the world today. This chapter gave an overview of the basic soil-related concepts and processes involved in landslides, their possible causes and the aftermath of this phenomenon. Also outlined were the mechanisms of failure associated with slope instabilities, methods and design approaches for evaluating slope stability, giving the diverse techniques of slope stabilization. Having gotten an idea of the generalities of landslides and seeing the effects they have on both the natural environment and on man-made structures such as roads if not prevented or properly remediated, it is therefore important to design a stabilization technique for the case study of this work. With this, it is necessary to present the methodology adopted to attain the objectives of this work.

CHAPTER 2. METHODOLOGY

Introduction

The literature review done in chapter one gave a broad knowledge of landslide-related concepts and various stabilization techniques. This chapter elaborates the methodology adopted in order to achieve the main objective of this work which is to stabilize one of the recurrent landslides in our case study. For this study, the methodology begins with a site recognition which was done through an extensive documentary research. Next is the data collection which presents the types of data that were collected for the slope stability analyses conducted using the numerical modelling software Geostudio. The third part of this chapter presents the proposed stabilization techniques for the landslide stabilization and the procedures followed in order to analyze and assess the stability of the slope. Finally, the considerations taken in order to make an optimal choice on the slope stabilization technique will be outlined.

2.1. Site recognition

Recognition of the site was done by means of documentary research with the aim of obtaining information on the general physical characteristics (geographic location, climatic conditions, relief, geology, soil, hydrography and vegetation) on one hand and on the other hand the socio-economic characteristics (demography and economic activities) of the population in the case study area.

2.2. Data Collection

The data required to achieve this work were geometrical data and geotechnical data. These data were secondary data gotten from the results of a set of investigations supervised and conducted on the site by the companies BXTG and SOL SOLUTION.

2.2.1. Geotechnical data

The results of geotechnical tests give information on the various layer thicknesses, the groundwater level and serve as a useful tool in the estimation of the soil mechanical properties. A comprehensive field exploration program had been carried out by the companies to obtain the soil geotechnical parameters. The soil properties were characterized by conducting in-situ penetration tests including heavy dynamic penetration tests, laboratory tests including direct shear tests (for determining the soil strength parameters) and other soil characterization tests on the soil samples collected from the investigated area. The main geotechnical data that were collected for this study were the cohesion and friction angles of the various soil layers in the slope, the soil unit weights and water contents, the resistance to penetration at depths in the soil layers and information on the position of the groundwater level.

2.2.2. Geometrical data

The geometrical properties obtained give information on the slope that failed on the site. A transverse section of the slope showing the complexity of the slope geometry was obtained. This cross-section gives an indication of how steep the slope is by showing the slope angles, height and whether it has berms or not.

2.3. Proposed stabilization techniques and Numerical modelling

Different stabilization techniques were proposed initially for the stabilization of the landslide. Numerical models were made for the simulation of various conditions. Using numerical analyses, the stability of the slope was checked for each technique. The proposed stabilization techniques will be presented in this section alongside the numerical modelling procedures.

2.3.1. Proposed stabilization techniques

The methods used for the slope stabilization were selected on the basis of the groundwater levels, soil type, the geometry of the slope and the engineering feasibility. The stabilization techniques assessed in this study include retaining walls, soil nailing, ground anchors and micropiles. The preliminary design considerations and design steps for the techniques proposed for the landslide stabilization will be presented in this part.

2.3.1.1. Retaining wall

For the retaining wall, the wall dimensions gotten from the data collected were used to verify the sliding, overturning and bearing capacity conditions of the wall. An acceptable value for the FoS of the overall slope stability with the retaining wall would be greater than or equal to 1.3 (FHWA NHI, 2006).

a. Wall geometry

The wall type was a gabion wall with three interconnected units. Figure 2.1 shows a characteristic geometry of the retaining wall considered during the design.

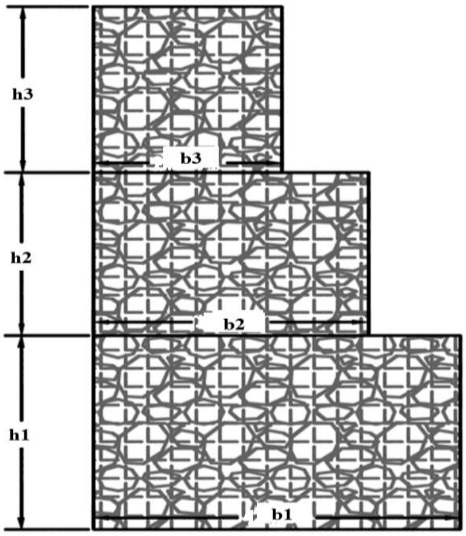

Figure 2.1. Wall geometry

b. Sliding verification

The sliding verifications were done using the partial coefficients on the geotechnical actions (GEO) in EC7. The wall was verified using Design action 1 combination 1. The FoS for sliding was computed from the ratio of the force resisting sliding of the wall $R_{d,slid}$ and the tangential force on the plane of sliding T. A FoS value greater than 1 indicates wall stability against sliding.

$$FoS_{sliding} = {R_{d,slid}}/{T} \tag{2.1}$$

$$T = \gamma_{Gu} \times Sa,h \tag{2.2}$$

$$R_{d,slid} = \frac{N \tan \delta_b}{\gamma_{r,slid}} \tag{2.3}$$

$$N = \gamma_{Gf} \times W_G + \gamma_{Gu} \times Sa,v \tag{2.4}$$

$$Sa = 0.5 \times Ka \times \gamma \times H^2 \tag{2.5}$$

$$Ka = \frac{1 - \sin \phi}{1 + \sin \phi} \tag{2.6}$$

$$Sa,h = Sa \times \cos \delta \tag{2.7}$$

$$Sa,v = Sa \times \sin \delta \tag{2.8}$$

Where:

ϕ = Internal friction angle of the soil behind the wall

H = wall height (m)

δ = wall-soil friction angle

$\delta b = k\phi$, k = 1 for cast-in place concrete

W = weight of the wall (kN/m)

α = base inclination

Sa = Active earth pressure resultant (kN/m)

$\gamma_{Gu} = 1.35$ and $\gamma_{Gf} = 1.0$

δ_b = wall-soil friction

N = Normal force on the plane of sliding (kN/m)

Sa,h = Horrizontal component of active earth pressure (kN/m)

Sa,v = Vertical component of active earth pressure (kN/m)

c. Overturning stability verification

For the overturning stability, the verifications were done using the EQU in EC7. The FoS was calculated from the ratio of the resisting moments Mr and overturning moments Mo (equation (2.9). A FoS greater than 1 implies overturning is verified.

$$FoS = {}^{Mr}/_{Mo} \tag{2.9}$$

$$Mr = Sa, v \times bv + W_G \times X_G \tag{2.10}$$

$$Mo = Sa, h \times bh \tag{2.11}$$

$$X_G = \frac{\sum_{i=1}^{n} st \times bi \times hi}{\sum_{i=1}^{n} bi \times hi} \tag{2.12}$$

Where:

Mr = Resisting moment

Mo = Overturning moment

bv = lever arm of Sa,v

X_G = horizontal distance of the center of gravity about the toe of the wall

bi = width of each wall unit

hi = height of each wall unit

st = offset from the wall toe

bh = lever arm of Sa,h

d. Bearing capacity verification

The total bearing pressure P on the base of the wall was gotten from the eccentricity e of the resultant force N on the wall base B (equation (2.13)). The FoS was calculated by dividing the allowable bearing capacity of the soil σ_{adm} by P.

$$P = (\tfrac{4}{3})(\tfrac{N}{B-2e}) \tag{2.13}$$

$$FoS = \frac{\sigma_{adm}}{P} \tag{2.14}$$

Where:

P = total bearing pressure on the wall base

N = the resultant force on the wall base

σ_{adm} = allowable bearing capacity of the soil

B = length of wall base

2.3.1.2. Soil nailing

This part gives an outline of the procedures used for the analytical design of the model of the slope stabilized using soil nails.

a. Design procedure

The steps followed during the numerical analysis were derived from an algorithm suggested by Chris Bridges (2017). The procedure involves developing a ground model through the definition of the geometry, selection of the nail input parameters, determination of grout-ground bond strength and then performing numerical analyses.

b. Geometry definition

The first step of the design consisted of defining the geometry and the soil parameters and a selection of a limit equilibrium method that satisfies both force and moment equilibrium and appropriate critical surface search parameters.

c. Selection of soil nail parameters

Figure 2.2 shows the typical components of the soil nail that will be considered in the design. The selection of the bar properties and nail layout (nail length, inclination angles, position of each level of nails, nail spacing, bond stress and drill hole diameter) was conducted following the allowable stress design recommendations given in the FHWA report on soil nail design (2015). Soil nails when used to stabilize a slope work mainly in tension. Equation (2.15) was used to obtain the resistance to tensile failure of the nail bar.

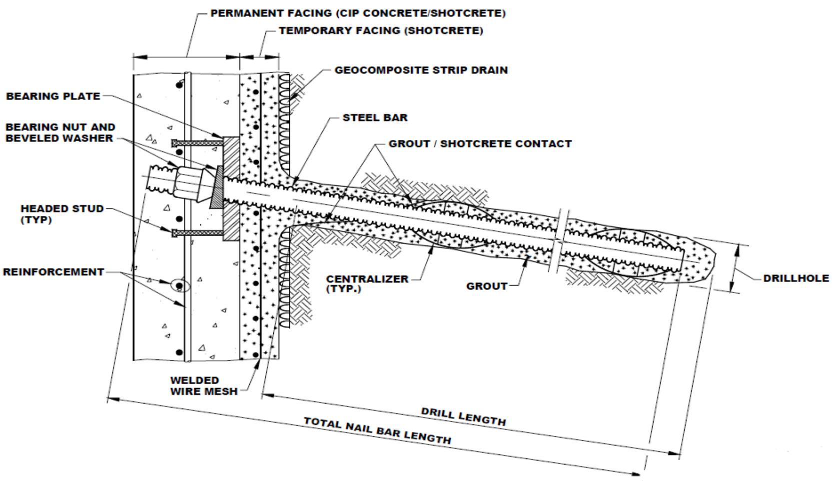

Figure 2.2. Main components of a typical soil nail with a concrete facing wall (Carlos et al., 2003)

$$Rt = At \ x \ fyk \tag{2.15}$$

Where:

At = cross-sectional area of nail tendon

fyk = Yield resistance of tendon

d. Determination of nail pullout resistance.

The resistance of the soil nail to pullout failure was calculated from the ultimate bond strength using equation (2.16).

$$r_{po} = \pi \times DDH \times q_u \tag{2.16}$$

Where:

r_{po} = pullout resistance per unit length of soil nail

At = cross-sectional area of the bar,

fyk = yield resistance of the bar.

DDH = Drill hole diameter

q_u = Bond strength which depends on the soil type and drilling method (Figure 2.3)

Table 4.4a: Estimated Bond Strength for Soil Nails in Coarse-Grained Soils (Modified after Elias and Juran 1991)

Drill-Hole Drilling Method	Soil Type	Bond Strength, q_u (psi)
Rotary Drilled	Sand/gravel	15 - 26
Rotary Drilled	Silty sand	15 - 22
Rotary Drilled	Silt	9 - 11
Rotary Drilled	Piedmont residual	6 - 17
Rotary Drilled	Fine Colluvium	11 - 22
Driven Casing	Sand/gravel w/low overburden (1)	28 - 35
Driven Casing	Sand/gravel w/high overburden (1)	41 - 62
Driven Casing	Dense Moraine	55 - 70
Driven Casing	Colluvium	15 - 26
Augered	Silty sand fill	3 - 6
Augered	Silty fine sand	8 - 13
Augered	Silty clayey sand	9 - 20

Figure 2.3. Estimated bond strength for soil nails (FHWA-NHI, 2015)

e. Analysis

For each time the design produced an unsatisfactory FoS, the preceding steps were performed again until an acceptable FoS was reached. Following FHWA specifications, the target FoS was set at 1.3.

2.3.1.3. Ground anchors

Ground anchors when used to stabilize a slope have the aim of producing a clamping effect (by applying stress normal to the sliding surface) and/or a straining effect (by using steel members as anchors) on the slope (JICA, 2009). The main steps of the design are discussed here.

a. Anchor function and required preventive force

The first step in the design was conducting the stability analysis of the unsupported slope. From the stability analysis, the force required to prevent the sliding P was calculated using equation (2.17). Figure 2.4 shows the interaction of an anchor in a slope. The forces P1 and P2 act parallel and perpendicular to the sliding surface respectively and work to increase the shear resistance and decrease the sliding force of the landslide. The selected anchor function was the straining (shearing) effect for a gentle slope with a deep sliding surface.

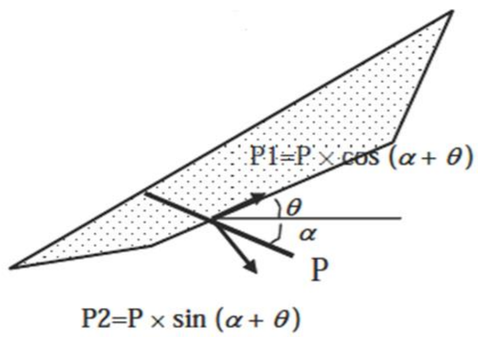

Figure 2.4. Illustration of the anchor effects (JICA, 2009)

$$P = \frac{(Fs - Fs_{\ (initial)}) \times \sum T}{\cos(\alpha + \theta) + \sin(\alpha + \theta) \times \tan\phi} \tag{2.17}$$

Where:

Fs is the target factor of safety

$Fs_{(initial)}$ = FoS of the slope before anchorage

α = angle of inclination of the anchors with respect to the horizontal

θ = slope of the slip surface

ϕ = internal friction angle of the slip surface

T = Tangential force attributable to gravity, T = W*sin θ

b. Determination of the design anchor power

The design power Td of the anchor was computed from equation (2.18).

$$Td = \frac{P}{\sin(\alpha+\theta).\tan\phi+\cos(\alpha+\theta)} . \frac{B}{N} \tag{2.18}$$

Where:

B = horizontal spacing between adjacent anchors in a row

N = Number of anchors in the vertical direction

c. Anchor layout

The arrangement of the anchors was chosen following the norms in the FHWA ground anchors design manual (1999). The analyses were performed while varying the layout (vertical and horizontal spacing, number of anchors and anchor inclination) of the anchors until a satisfactory value of FoS was obtained.

d. Calculation of anchor bond length

An illustration of the anchor structure is shown in Figure 2.5. The fixation (bonded) length L2 was gotten by choosing the maximum between the values of l_{sa} and l_a computed from equation (2.19) and equation (2.20) respectively. Typical anchor bearing plates include cribs, plates or cross-shaped blocks. The size of which is dependent on the allowable compressive stress of the shotcrete facing. The anchor should be placed perpendicularly to the bearing plate.

$$l_{sa} = \frac{T_d}{\pi \times D_s \times \tau_{ab}} \tag{2.19}$$

$$l_a = \frac{f \times T_d}{\pi \times D_a \times \tau_{ag}} \tag{2.20}$$

Where:

lsa= required length between the tendon and grout

la = required length of contact between the soil and grout

Ds = Diameter of tendon

f = safety factor which is generally assigned a value of 2.5

Da = anchor diameter

τab = Allowable adhesive stress between the soil and grout

τag = Skin frictional resistance

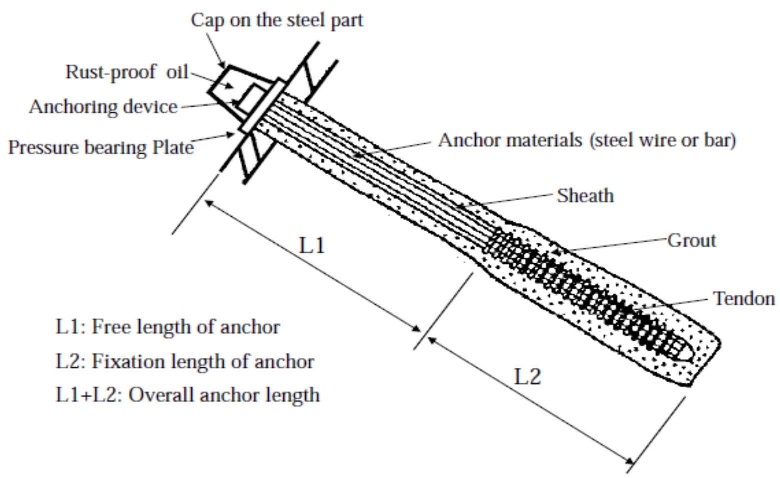

Figure 2.5. Ground anchor components

2.3.1.4. Micropile design

The FORCE=PASSIVE method described by Gutierrez et al (2021) was followed during the preliminary design process of the landslide stabilization with micropiles where the passive reinforcement considered were micropiles arranged in rows.

a. Design procedure

Figure 2.6 shows the flowchart from which the design procedures were drawn. The procedure begins with a definition of the target factor of safety, conducting stability analyses of the unreinforced slope, determining the required micropile resistance, selection of micropile layout and bar properties and the calculation of the design micropile shear resistance.

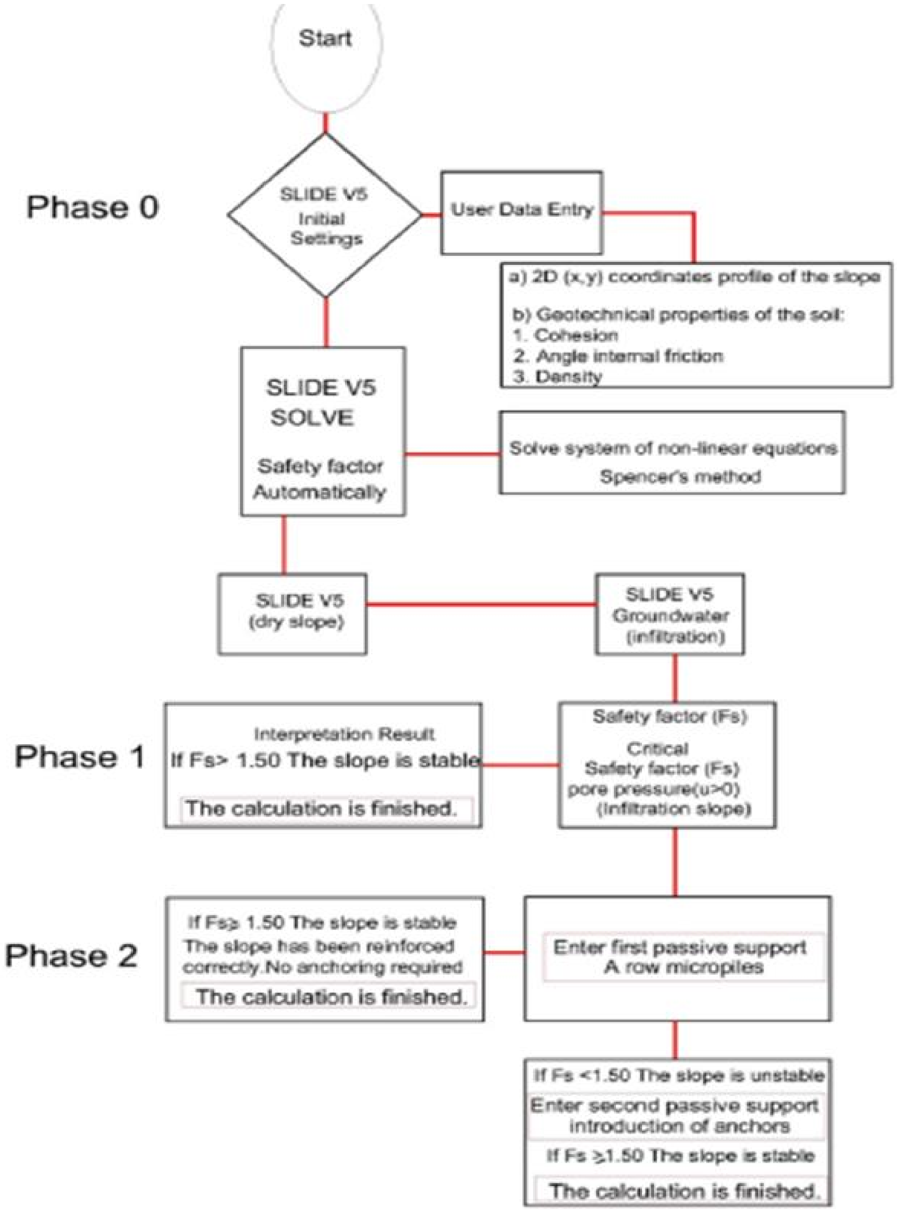

Figure 2.6. Flowchart for the phases involved in the design using the FORCE=PASSIVE method (Gutierrez et al.,2021)

b. Target safety factor

The target safety factor for this design was set at 1.3 following the minimum interval of 1.2 to 1.3 for slope stabilization with micropiles stipulated by FHWA (2005). Point E on Figure 2.10b coincides with the upper limit of the embedment of the support and determines the shear strength that each micropile in a single row of micropiles must support.

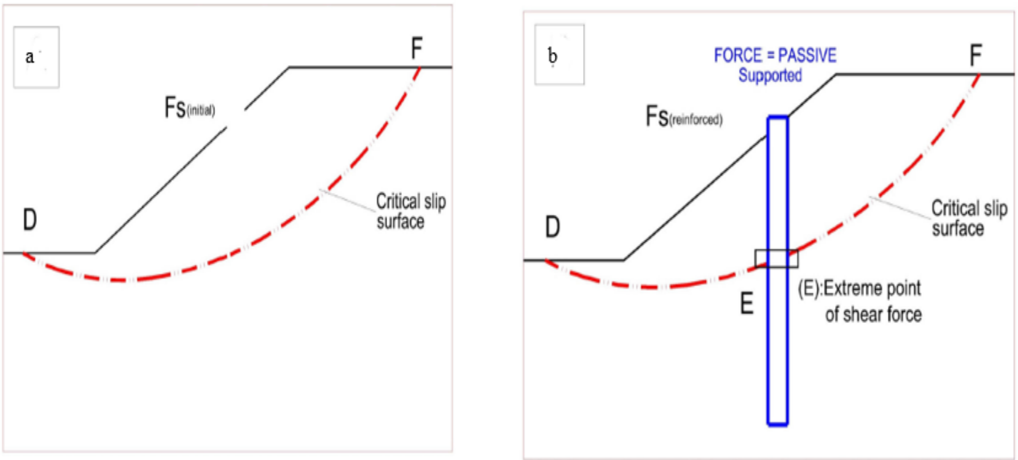

Figure 2.7. Illustrative example of the theoretical application of the model. (a) No support example and (b) critical surface with the FORCE introduction. (Gutierrez et al.,2021)

c. Stability analysis of the unreinforced slope

After defining the slope geometry and soil parameters, a stability analysis was performed on the unsupported slope. The support of the slope was done in two phases:

- An initial determination of the FoS of the unsupported slope $Fs_{(initial)}$ using the LEM. The value of $Fs_{(initial)}$ was compared to the target value of 1.3. If $Fs_{(initial)}<1.3$, the slope was considered unstable. If $Fs_{(initial)} >1.3$, the slope was considered stable. With reference to Figure 2.8, the $Fs_{(initial)}$ was determined by the LE software through the ratio of the resisting forces Fr and the driving forces Fd (Equation (2.21)).

$$Fs_{initial} = \frac{Fr}{Fd} \tag{2.21}$$

- For the case where $Fs_{(initial)}<1.3$, the reinforcements were introduced in order to increase the shear strength of the slope.

d. Determination of the required micropile resistance

The new FoS ($Fs_{(reinforced)} = 1.3$) of the slope after application of micropiles considers the total resistance provided by micropiles for a unit length of the slope F_{rm-p}. F_{rm-p} was calculated from equation (2.22) in order to get the total required micropile resisting force. Figure 2.8 shows the forces acting on the slope pre and post the micropile installation.

$$Fs_{(reinforced)} = \frac{F_r + F_{rm-p}}{F_d} \tag{2.22}$$

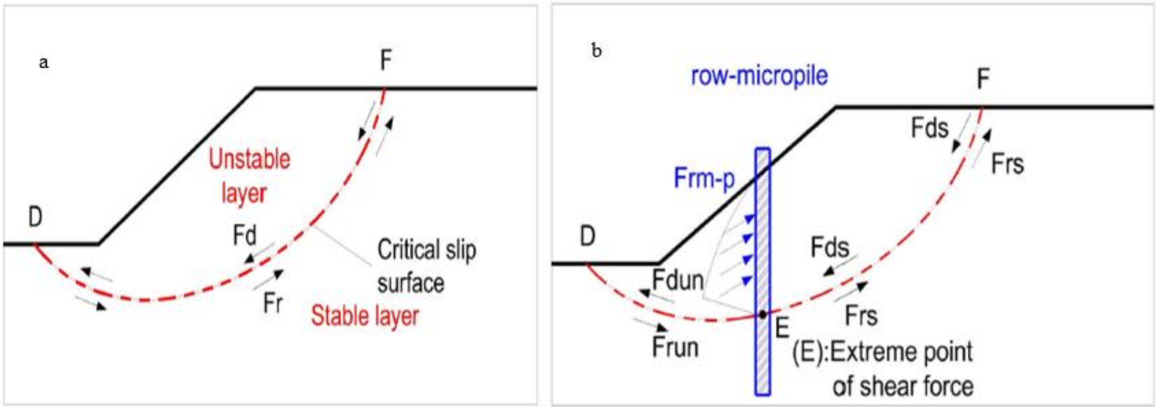

Figure 2.8. Slope stability pre-and post-micropile installation. a. Unstable layer and critical slip; b. Reinforcement to stabilize the slope by introducing passive force (Gutierrez et al.,2021)

e. Selection of micropile layout and bar properties

A choice of ideal micropile cross-sections and configurations was made. The spacing of the micropiles was considered to be the greater value between 760mm and three times the micropile diameter in accordance with FHWA (2005) specifications. The properties of the reinforcing bar and neat cement grout were gotten from the FHWA micropile design and construction manual (2005). The selection of the number of micropiles and their cross-section was done by choosing the diameter and reinforcement necessary to provide the required shear force per row of micropiles. The micropile type considered was the type A2 (gravity-grout) micropile reinforced with steel reinforcing bar as shown in Figure 2.9.

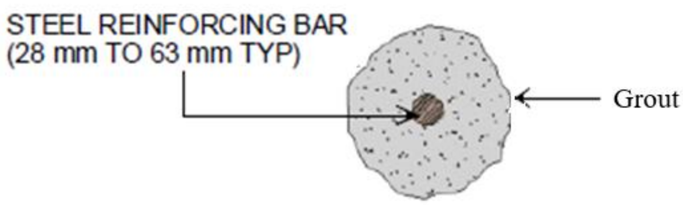

Figure 2.9. Micropile cross-section (FHWA, 2005)

f. Calculation of the design micropile shear resistance

One very essential parameter to the numerical model of the reinforcement is the shear resistance. Figure 2.10 shows the mechanism by which micropiles work which is basically resisting mobilized shear forces. The shear resistance (SR) of a single micropile was calculated using equation (2.23) from Babu (2017).

$$SR = 0.4\ x\ yield\ strength\ of\ the\ reinforcement \quad (2.23)$$

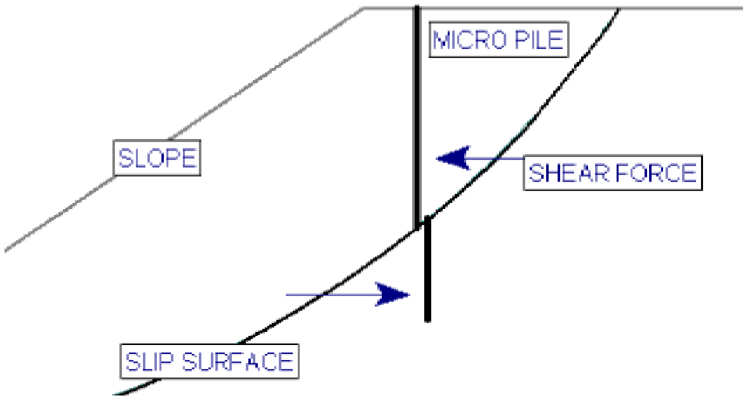

Figure 2.10. Shearing effect on micropile (Babu, 2017)

g. Minimum length of micropile below slip surface

According to Turner and Halvorson (2013), field observations show that the effects of axial loads is very much significant than for bending moments in a micropile. The magnitude of a uniformly distributed load P (in the direction of the sliding plane) due to the sliding mass assumed to act on the micropile above the slip surface is given in equation (2.26) (Turner & Halvorson, 2013). A sufficient length of the micropile below the slip surface L_{below} is required to resist the imposed axial tensile and compressive loads. The length L_{below} is calculated from equation (2.24).

$$L_{below} = \frac{P'}{f_{all} \times \pi \times D} \tag{2.24}$$

$$P' = Pcos\theta \tag{2.25}$$

$$P = \frac{F_{rm-p}}{L_{above}} \tag{2.26}$$

Where:

P' = Axial force

D = micropile reinforcement bar diameter

f_{all} = grout-to-ground bond strength below the slip surface (Figure 2.11)

L_{above} = Length of the micropile above the slip surface (from numerical analysis)

θ = Angle between the micropile (above slip surface) and the slip surface

Soil / Rock Description	Grout-to-Ground Bond Ultimate Strengths, kPa (psi)			
	Type A	Type B	Type C	Type D
Silt & Clay (some sand) (soft, medium plastic)	35-70 (5-10)	35-95 (5-14)	50-120 (5-17.5)	50-145 (5-21)
Silt & Clay (some sand) (stiff, dense to very dense)	50-120 (5-17.5)	70-190 (10-27.5)	95-190 (14-27.5)	95-190 (14-27.5)
Sand (some silt) (fine, loose-medium dense)	70-145 (10-21)	70-190 (10-27.5)	95-190 (14-27.5)	95- 240 (14-35)
Sand (some silt, gravel) (fine-coarse, med.-very dense)	95-215 (14-31)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Gravel (some sand) (medium-very dense)	95-265 (14-38.5)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	95-190 (14-27.5)	95-310 (14-45)	120-310 (17.5-45)	120-335 (17.5-48.5)
Soft Shales (fresh-moderate fracturing, little to no weathering)	205-550 (30-80)	N/A	N/A	N/A
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	515-1,380 (75-200)	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	1,035-2,070 (150-300)	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	520-1,725 (75.5-250)	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	1,380-4,200 (200-609)	N/A	N/A	N/A

Figure 2.11. Typical αbond values for micropile design (FHWA, 2005)

2.3.2. Numerical modeling

With the insitu and laboratory data assembled, a numerical analysis was performed using the numerical software Geostudio 2018. The principal aim of the modeling was to analyze the slope stability and to verify the functionality of the proposed remedial solutions. Geostudio being a complete suite of geotechnical software designed for modeling geotechnical problems has incorporated within its interface various modules including SLOPE/W. SLOPE/W is a limit equilibrium-based software developed by GEO-SLOPE International Canada to be a general tool for the stability analysis of earth structures. Using LE, SLOPE/W can model heterogeneous soil types, complex stratigraphic and slip surface geometry and variable pore water pressures. SLOPE/W can effectively analyze both simple and complex geometrical shapes for a variety of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods, and loading conditions. This part will begin with a presentation of the analysis methods available in SLOPE/W then the procedure followed during the modeling.

2.3.2.1. Limit equilibrium analysis methods available in SLOPE/W

The literature review in chapter one showed that the LE methods are very similar and are based on the principle of division of the slope mass into slices. Each slice has both normal and shear forces acting on the slice base and the slice sides. Figure 2.12 shows an illustration of the slice discretization in SLOPE/W. The forces acting on a slice in a sliding mass are shown in Figure 2.13a (without the presence of water) and Figure 2.2b (with the presence of water).

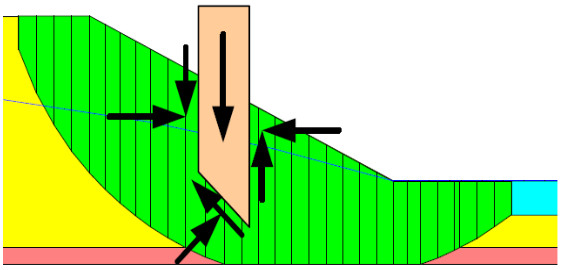

Figure 2.12. Slice discretization and slice forces in a sliding mass (SLOPE/W manual, 2012)

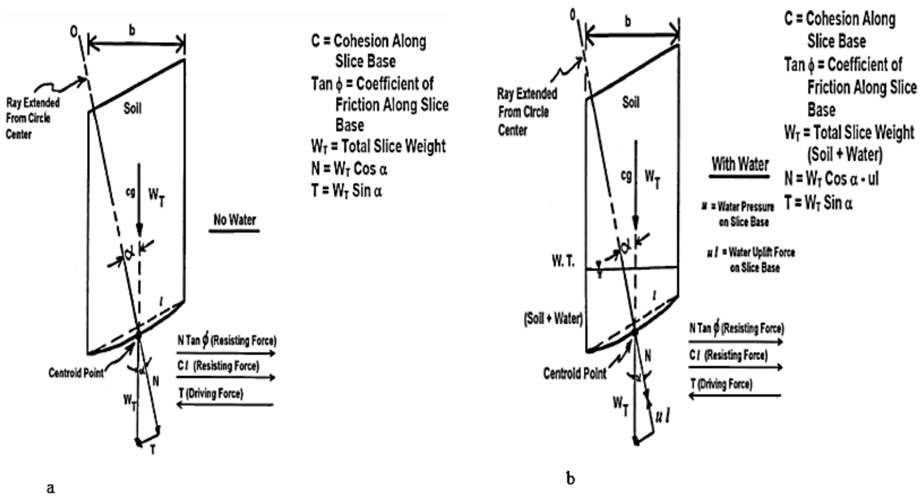

Figure 2.13. Forces acting on a slice in conditions: a. without water b. with water (Samtani et al., 2006)

The basic difference between the LE methods is due to the equations of statics they satisfy and how the interslice shear and normal forces are determined or assumed. Table 2.1 shows the methods available in SLOPE/W and summarizes the major differences between them. The peculiarity of each of the methods is as follows:

- With the Ordinary method of slices in SLOPE/W, ignoring the interslice forces makes the safety factor equation linear and so no iterative procedure is required to solve for the factor of safety.

- Bishop simplified method satisfies only moment equilibrium, considers only the interslice normal forces but ignores interslice shear forces. An iterative technique is required to solve the Bishop factor of safety since its equation is non-linear. The Bishop method is close to being in force equilibrium since it includes interslice normal forces in the force polygon.
- Janbu method of analysis is identical to the Bishop method, except it satisfies only horizontal force equilibrium. Janbu method includes interslice normal forces but not shear forces.
- Spencer method considers both normal and shear interslice forces and satisfies both force and moment equilibrium. The unique condition in Spencer method is that the ratio of shear to normal interslice forces is a constant and is therefore the same for each slice. SLOPE/W computes one factor of safety with respect to moment equilibrium and a second factor of safety with respect to horizontal force equilibrium for the various shear-to-normal ratios referred to as lambda (λ) in SLOPE/W (equation (2.27). The iterative process continues until both factors of safety are approximately the same. Within a specified tolerance, the solution is said to have converged to the Spencer factor of safety.

$$\lambda = \tan {}^{X}/_{E} \tag{2.27}$$

Where X and E are the interslice shear force and interslice normal force respectively

- The Morgenstern-Price method is very much similar to the Spencer method, except it allows the user to specify an interslice force function. For the analysis in SLOPE/W using the M-P method, the force functions available include the constant or half-sine function, clipped-sine, trapezoidal and data-point specified.
- The Corps of Engineers 1 & 2 methods only satisfy overall horizontal force equilibrium. Overall moment equilibrium is not satisfied. The methods are characterized by two different assumptions about the interslice force functions. One uses the slope of a line from crest to toe (slip surface entrance and exit points) and the other uses the slope of the ground surface at the top of the slice.
- The Sarma method in SLOPE/W is similar to the Spencer and the Morgenstern-Price methods. The computed interslice shear forces are adjusted with a global factor (similar to Lambda) until overall force and moment equilibrium are satisfied.
- The generalized limit equilibrium method embodies the concepts of all the other methods. SLOPE/W computes both moment and force safety factors for each specified λ value.

Table 2.1. Summary of LEMs available in SLOPE/W

Method	Moment equilibrium	Force equilibrium	Interslice shear	Interslice normal	Slip surface
Ordinary	Yes	No	No	No	Circular
Bishop simplified	Yes	No	No	Yes	Circular
Janbu Simplified	No	Yes	No	Yes	Non-circular
Janbu Generalized	Yes	Yes	Yes	Yes	Any shape
Lowe-Karafiath	No	Yes	Yes	Yes	Non-circular
Corps of Engineers	No	Yes	Yes	Yes	Non-circular
Sarma	Yes	Yes	Yes	Yes	Any shape
Spencer	Yes	Yes	Yes	Yes	Any shape
Morgenstern-Price	Yes	Yes	Yes	Yes	Any shape

2.3.2.2. Simulation procedure

Performing the numerical simulation basically consisted of entering a set of input data and solving the analysis in order to obtain the output which was mainly the FoS. Beginning the LE analysis in SLOPE/W was with a definition of the problem. Defining the problem on SLOPE/W was through the definition of the slope geometry along with the soil mechanical properties of the soil layers, ground surface line, strength model and reinforcements and the LE analysis method. The procedures followed will be presented herein.

a. Geometry

The slope geometry defined in closed regions the various soil layer arrangements, slope angle and in the case of the presence of groundwater, the piezometric surface orientation. After setting the working area by defining the working scale and grid, the regions were drawn directly in the software paying careful attention to the slope angles. The shape of the slip surface was also specified. The circular slip surface was used. It is worth noting that with the CSS, the moment equilibrium is completely independent of the interslice shear forces because the sliding mass can rotate without any slippage between the slice but the force equilibrium however is sensitive to interslice shear.

b. Ground surface line

In SLOPE/W, the ground surface line, bounded by ground surface limits (Figure 2.3) was used to control and filter trial slip surfaces. The entry and exit method of creating trial slip surfaces was chosen with the slip surfaces entering and exiting along the ground surface line. The ground surface line was computed automatically by the software from the definition of the slope geometry.

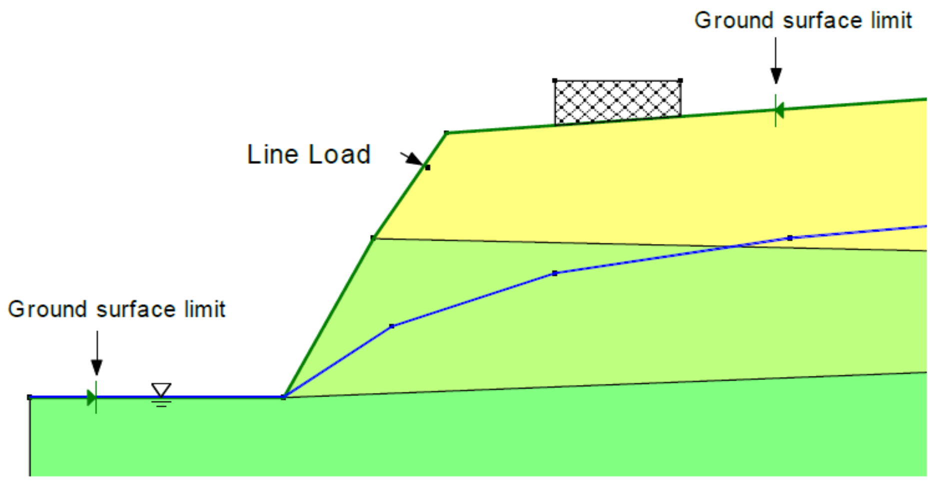

Figure 2.14. Example showing ground surface line and limit markers (SLOPE/W, 2012)

c. Strength model

The Mohr-Coulomb model was used to define the strength of the slope-forming materials. The parameters that were inputted for the Mohr-Coulomb model of each material include the unit weight, angle of internal friction and cohesion. Equation (1.9) gave a relationship between the shear strength of the soil with respect to the soil frictional angle, cohesion and the effective stress within the soil which is dependent on the unit weight of the soil. SLOPE/W makes no distinction between total and effective stress parameters so the parameters used were selected based on the ground conditions.

d. Reinforcement

The inclusion of reinforcements where necessary was done by defining the type of reinforcement load and inputting the computed design parameters for the reinforcement. The reinforcements were modelled differently, depending on the reinforcement type and position.

e. Analysis method

The analysis using SLOPE/W was based on the Morgenstern-Price half-sine function method. Given that the LEM approaches are based on the equation of statics, the M-P method was considered as a choice method for the analysis since it deals with both force and moment equilibrium equations and considers both interslice shear and normal forces.

f. Verification and computation

After defining the geometry, analysis method, material and reinforcement properties, the verify data command in the Tools menu was used to make the program run several checks on the inputted data. When the verification was completed and there were no errors, the factor of safety was computed using the selected LE method which was the Morgenstern Price method. A half-sine function was used to compute the interslice forces based on assumptions of a particular number of slices, specification of the presence or absence of tension cracks and non-optimization of the critical failure surface. Even though the chosen method was Morgenstern-Price with the half-sine function for the analyses, the analyses were also performed with the Spencer method as a control to verify the results gotten using the M-P method. The minimum factor of safety and the associated critical slip surface were obtained for each analysis performed.

g. Factor of safety

As earlier mentioned, the main aim of conducting a stability analysis is to verify that the factor of safety is greater than a certain minimum value as per the stabilization technique. For the cases analyzed, getting a satisfactory FoS value was necessary in order to ensure that the slope is considered stable. In SLOPE/W, the minimum FoS was computed based on an assumption of 30 slices, no tension cracks and no optimization of CSS. The minimum values for the factor of safety for the LE analyses that were considered were specified for each case.

h. Analyzed cases

A total of four situations were studied. These were modeled by specifying the material properties and defining the preliminary design parameters for the reinforcement loads which were used. The analyses were conducted for each case and the results obtained were examined to guide the judgment of the performances of the remedial solutions. The cases include the following:

- Stability analysis of the unreinforced slope plus a sensitivity analysis to evaluate the FoS sensitivity to water. The sensitivity analysis produced a sensitivity graph which shows the change in FoS with variations in groundwater level.
- Stability analysis of the slope with retaining wall.
- Stability analysis of the slope with a complementary solution.

- Stability analysis of the modified slope with the alternative solutions.

2.4. Criteria of comparison of the proposed stabilization techniques

The geotechnical works aimed at slope stabilization have to take into account technical aspects such as the landslide morphology, the economic impact and at the same time, ensure efficiency and performance for long-term safety conditions. Hence, in this study a comprehensive analysis of common slope stabilization methods was done in order to give an insight on the behavior of the support systems. The technique considered here as optimal is that which will be more effective in the prevention of potential landslide occurrences (higher FoS), cost less and secure the road after its implementation. In essence, the proposed techniques were compared with respect to cost and also with respect to FoS.

Conclusion

After reviewing documents with information on the general site conditions of our case study and assembling the necessary data for the geotechnical study, an appropriate methodology was developed in order to achieve the main objective of the work. This methodology was divided into four main parts. The first being the general recognition of the site done through documentary research. The second part, which was the data collection gave a brief description of the two main types of data necessary for the study (geometrical and geotechnical data). The third part outlined the methods and procedures followed in the design of the stabilization techniques. Also presented were the procedures for the numerical modelling and analysis of the problem. The last part of the methodology presented the criteria adopted for the selection of the most efficient stabilization technique.

CHAPTER 3. RESULTS, ANALYSES AND INTERPRETATIONS

Introduction

A landslide study requires a careful study and assessment of several parameters. This becomes even more necessary when the landslide in question affects an infrastructure. In this case study of this work, the road is affected by recurrent landslides along different sections of the road. As a result of the unstable situation along the cut slopes, debris and loose boulders also are falling onto the road surface creating a hazard. In order to detect and know the causes and possible failure mechanisms taking place in the affected slopes and propose remedial solutions, slope stability analyses were conducted following the different procedures enumerated in chapter 2. This chapter entitled results, analyses and interpretations presents first and foremost the information gotten from the site recognition followed by the data obtained from field and laboratory test results. Next, the stability analyses of the slope with modeling of the proposed mitigation techniques will be conducted. Finally, based on the criteria given in chapter 2, the optimal solution will be selected.

3.1. Site presentation

The general physical features of the site and the socio-economic activities of the people living in the vicinity of the Batibo-Numba road will be presented in this section through a comprehensive description of the area's geographic location and landslide affected portions of the site, relief, climate, soil, vegetation, hydrography, demography and economic activities.

3.1.1. Location

The Batibo-Numba road stretch located in the North-West region of Cameroon is along the National road 6 (NR6) on the Bamenda-Mamfe-Ekok-Enugu corridor of the Trans-African highway which forms a link between Cameroon and Nigeria. The Batibo-Numba section of the road is approximately 20km long and serves different localities including the Widikum and Tiben areas. Batibo and Numba lie between latitudes 05°49'12" and 05°50'20" North and longitudes 09°42' 51" to 09°50' 00" East respectively. The road being one of the routes that opens up the North-West and South West regions promotes regional integration.

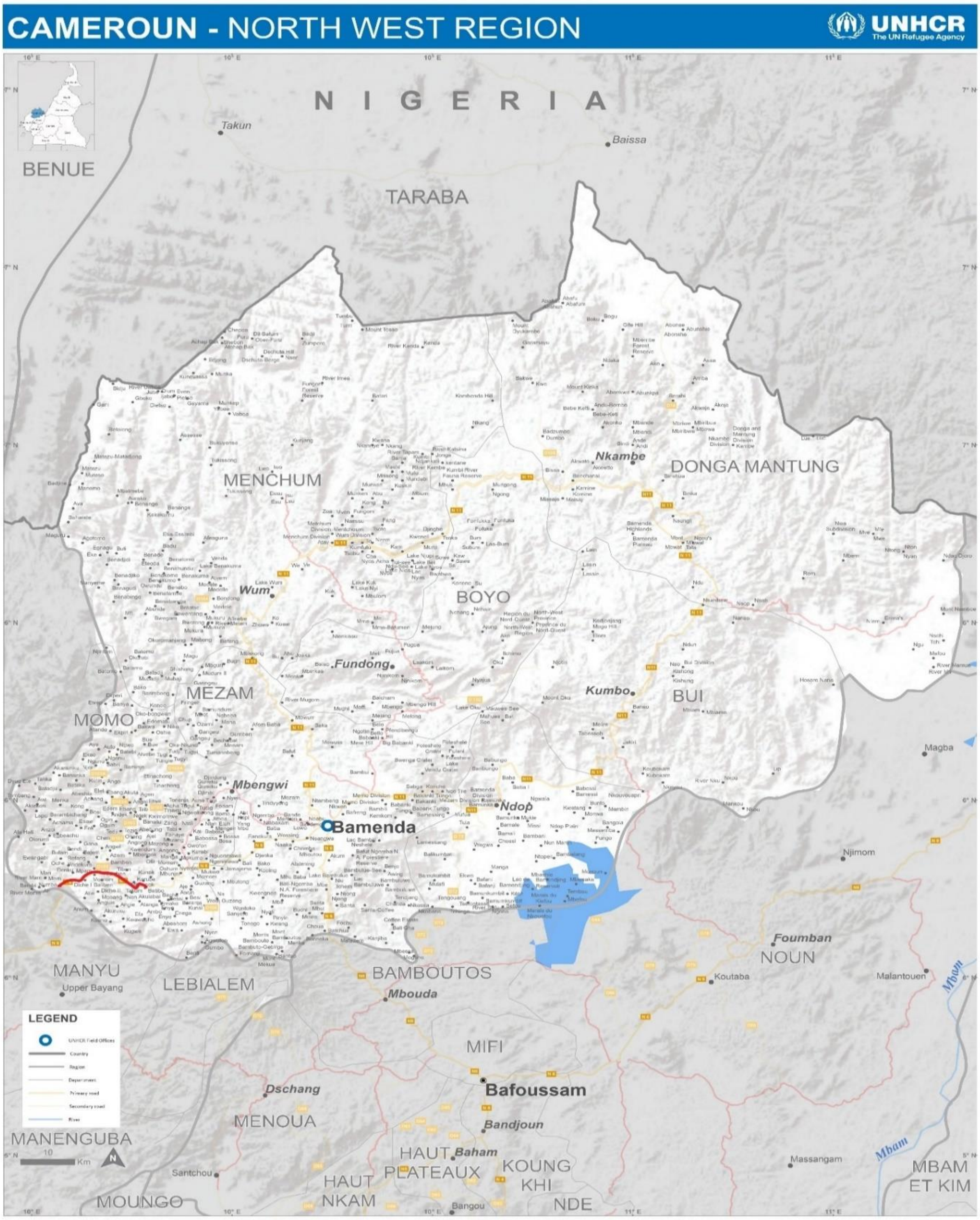

Figure 3.1. Administrative map of the North West region of Cameroon (UNHCR Cameroon).

3.1.2. Relief

The road section belongs to the high plateau zone of Cameroon. It passes through an area with a hilly to mountainous topography presenting numerous cliffs and interfluves.

Altitudes in the Batibo-Numba area range between 100 and 150 m on the plains, and 500 and 1300 m in the mountainous areas. Three great geomorphological units around the study area are shown in Figure 3.2. The low zone (250 – 800 m) is vast and slightly corrugated and marked by a gentler relief, the median zone (800 – 1300 m) corresponds to a higher projecting plateau and is marked by the juxtaposition of the morphological characteristics suitable for the low and the high-altitude zone. The high-altitude zone (1300 m and more) occupies the summit of the study area.

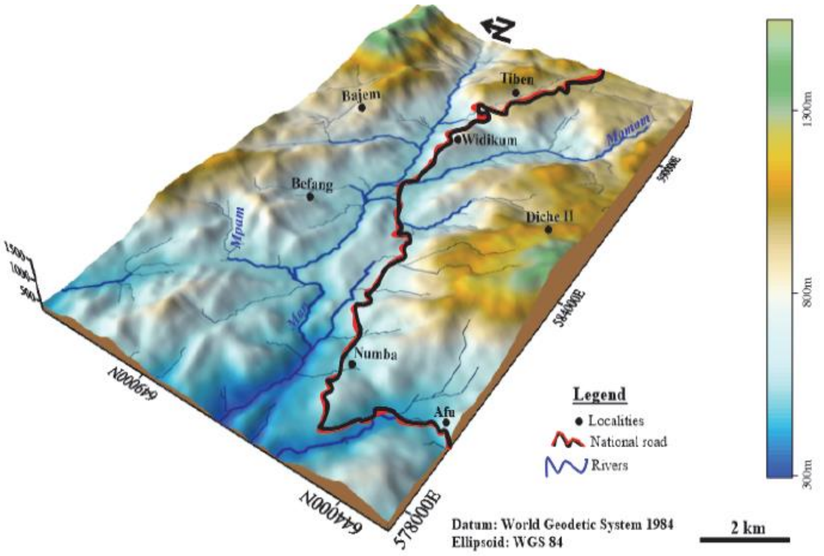

Figure 3.2. Main geomorphological units within the study region (Rodrigue et al. 2018)

3.1.3. Climate

The Batibo-Numba road area is under the influence of the Cameroon montane climate, with relatively consistent temperatures throughout the year, though the area experiences somewhat cooler temperatures in July and August. The region typically has warm climatic conditions with temperatures varying in the relief zones. In the low zone, temperatures are high throughout the year with a monthly average of 20ºC and an annual range of 4.6ºC. The perceived humidity varies extremely with seasonal changes. Like every other region within the Equatorial climatic zone, this area has two seasons: the rainy and dry seasons. The road section lies in a tropical forest zone with average annual rainfall values above 2000mm (about 2300mm). The rainy season covers from mid-March to mid-November and the dry season from mid-November to mid-March. The major rainy period runs from August to October.

3.1.4. Soils

The different soils present in the region are mostly lateritic soils resulting from volcanic products such as old basalts and faded granites. Other soil types found in the region include red soils and black or loam soils. The dominant soil here is the red soil. The black soil which is also found in great quantities is also very fertile and promotes the growth of crops. Rodrigue et al. (2018) stated that the soils in this area that result from weathering activities are characterized by poor mechanical properties and high plasticity making them very susceptible to creep.

3.1.5. Vegetation

The main phytogeographical units of the Batibo-Numba area belongs to the Afro-montane zone (African Development Fund, 2007). The characteristic features of this zone include markedly degraded facies of the montane forests found notably in areas with high human concentration, particularly around Batibo and Widikum. The region is also characterized by degraded facies of evergreen forests which are notably found along the roads and agglomerations.

3.1.6. Hydrography

The road under study crosses the Batibo municipality which is a water catchment area with a dendritic hydrographical network. Also found in the area are a few boreholes which are not functional. The volume of water in these areas increases during the rainy season but despite all this, the water table is very deep and as a result, during the dry season, most streams dry up completely. The few wells and boreholes also empty out during the dry season.

3.1.7. Demography

The average population density is 184 inhabitants per km² in the Batibo subdivision, 60 inhabitants per km² in Widikum Menka subdivision and in Numba, the estimated population as of 2005 is 1,384 inhabitants.

3.1.8. Economic activities

Economic activities in the region promote basically the primary sector which is centered around agriculture. This sector is very important for the local economy and social development as it absorbs a significant proportion of the labor force in the population. Agricultural activities in the Numba-Batibo area include mostly stockbreeding which is practiced but limited to small-scale livestock and farming of crops such as cocoyam and coffee. Palm wine tapping in the Batibo village (also referred to as Cameroon's palm wine capital) is a very common activity. There are also stone quarries and sandpits in the area which serve as a source of employment and an income-generating source to the population.

3.2. Data presentation

The results of geotechnical tests and the geometrical information collected will be presented in this part.

3.2.1. Geotechnical data

Presented herein is the information on the essential geotechnical parameters which had been obtained from the various laboratory and field investigations. The landslide-affected road sections are shown in Table 3.2. These portions were identified during a survey conducted along the road. The DPH tests were carried out at the foot and head of each slope in the affected road portions. The tests were stopped at depths where boulders and pebbles were encountered. The water table was not reached during the investigations. Presented in Table 3.1 are the values of the resistance to dynamic penetration with the depths reached at the various sections. Rpmin refers to the minimum value and Rpmax to the highest value recorded at a particular depth. The results obtained through laboratory tests and correlations with field exploration data for the mechanical and physical characteristics of the soil in the zone Pk19 are presented in Table 3.3.

Table 3.1. Results of the DPH tests

Survey reference	Depth (m)	Rp_{min}- Rp_{max} (daN/cm^2)	σ_{adm} (daN/cm^2)	Observations
Pk12	0.00	30-54	2.0	Whitish clayey sand
	0.80	27-100	3.7	Reddish clayey sand
	2.10	62-192	7.6	Rocky alterations with pebbles
	4.20	173-	11.5	
Pk16	0.00	24-30	1.9	Reddish clayey sand
	0.40	27-84	2.9	Reddish clayey sand
	1.80	62-161	5.5	Blackish clayey sand plus pebbles
	2.80	135-199	9.4	Blackish clayey sand plus basaltic blocks
	4.00	173-	11.6	
Pk19	0.00	16-48	1.2	Whitish clayey sand
	1.20	27-67	2.2	Whitish clayey sand
	2.30	47-160	4.9	Rocky alterations plus pebbles
	3.40	116-209	9.3	Rocky alterations
	4.40	172-	11.6	
Pk20	0.00	22-61	2.0	Reddish clayey sand
	1.80	20-64	1.7	Blackish clayey sand
	2.0	54-128	5.2	Rocky alterations
	4.60	100-172	9.8	Rocky alterations
	5.60	172-	11.5	

Table 3.2. Landslide-affected sections of the road and their references

From PK	To PK	Report Reference
12 + 0.160	12 + 0.180	PK12
16 + 0.290	16 + 0.435	PK16

19 + 0.360	19 + 0.500	PK19
19 + 0.960	20 + 0.030	PK20

Table 3.3. Soil mechanical characteristics and physical features for the slope at PK19

Nature of material	Blackish clayey sand plus pebbles	Whitish clayey sand	Reddish clayey sand	Rocky alterations
Natural water content (%)	15.7	20,0	18,4	13.8
Dry unit weight (kN/m^3)	16.46	14,03	13,83	16.43
Unit weight of solids (kN/m^3)	26.16	26,46	26,39	25.52
Internal cohesion (kPa)	11.1	0,0	4,9	4,000
Angle of internal friction (^{o})	34.4	9,65	10,76	35.37

3.2.2. Geometrical data

The main geometrical data collected were the geometry of the slope before and after the construction of the retaining wall for the landslide stabilization. The geometry of the slope before the wall construction as shown in Figure 3.3 is quite complex. The slope was reshaped to obtain a more regular geometry as illustrated in Figure 3.4. A superposition of both geometries can be seen in Figure 3.5 showing the approximated gradient of the slope.

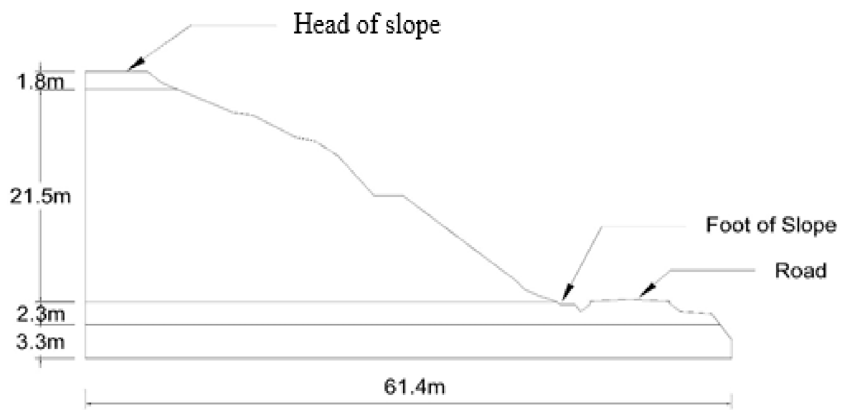

Figure 3.3. Slope geometry without retaining wall

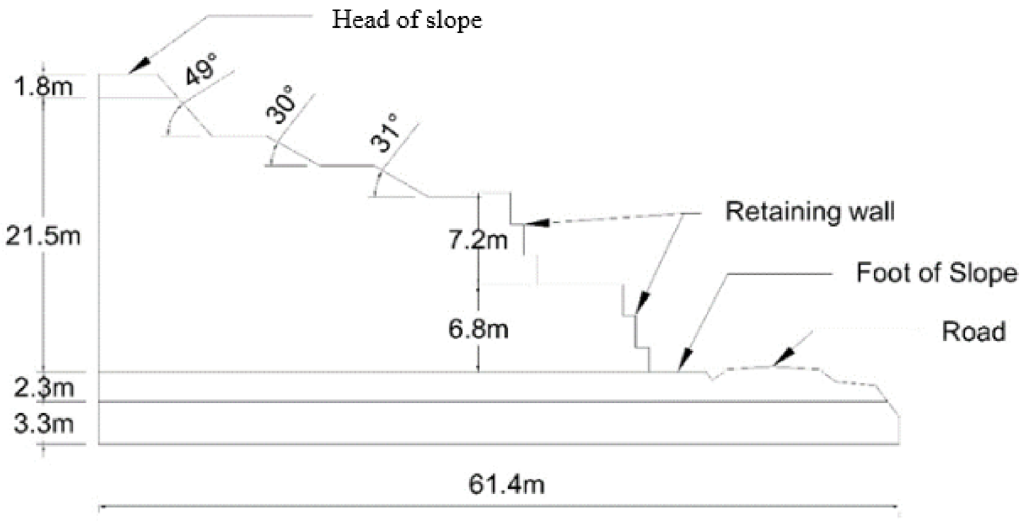

Figure 3.4. Geometry of the slope showing slope modification and retaining walls

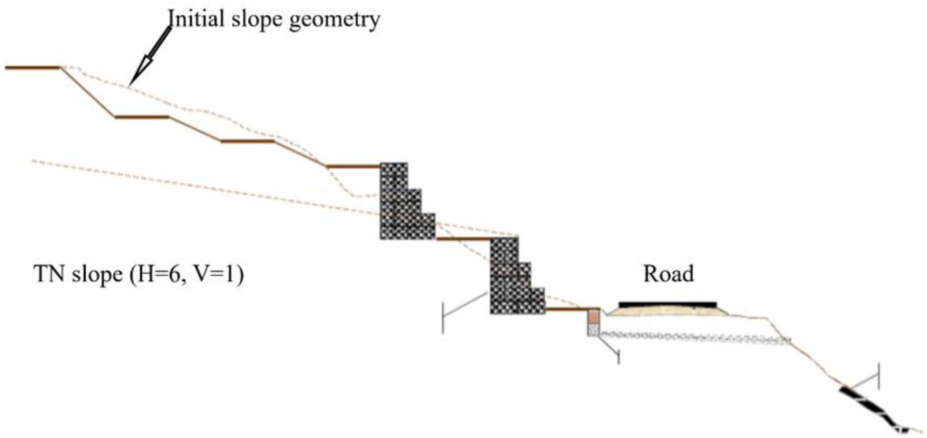

Figure 3.5. Illustration of the slope showing initial geometry and geometry with the retaining wall

3.3. Numerical models and results

The slope stability analysis of the investigated area was performed on the idealized cross-sections of the soil profiles according to the topography and geotechnical properties obtained from the various kilometric points.

3.3.1. Soil parameters

After a thorough analysis of the results of the site investigations, the parameters given in Table 3.4 represent the parameters of the slope that were inputted in SLOPE/W for stability analyses. The soil parameters considered were those of PK19 which has a retaining wall already put in place for the slope stabilization. Performing the analysis on the slope of PK19 will help in the prediction of the cause of the recurrence of landslides in the zone even after the construction of the retaining wall.

Table 3.4. Soil input parameters for the numerical model of the slope

Soil	Dry unit weight (kN/m^3)	Cohesion (kPa)	Angle of internal friction (°)
Reddish clayey sand	13,83	4.9	10.76
Blackish clayey sand plus pebbles	16.46	11.1	34.4
Whitish clayey sand	14.03	0.0	9.65
Rocky alterations	16.43	4000	35.37

3.3.2. Slope Geometry

The geometry of the slope analyzed with SLOPE/W is shown in Figure 3.6 where the slope was represented using the four distinct layers with the soil properties listed in Table 3.4.

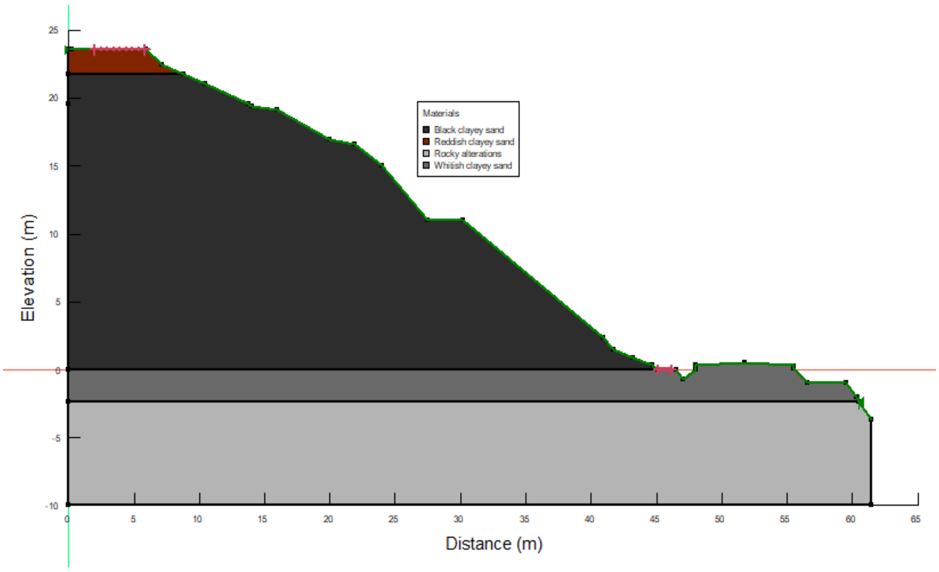

Figure 3.6. Slope geometry on SLOPE/W

3.3.3. Numerical analyses and results

Presented in this part are the models of the cases listed in section 2.3 and the results of the various numerical analyses.

3.3.3.1. Free slope without reinforcement

The slope in its original state without the application of any remedial measure was first modeled and the critical slip surface was identified.

a. Model

The model of the natural slope (without modification or reinforcement) was defined with the entry and exit ranges as indicated in Figure 3.7. The water level was gotten from a back analysis of the slope to find the conditions of the slope just before failure (at a FoS equal to unity). The analysis was performed for the M-P method. The Spencer method was used for the verification of the computed FoS.

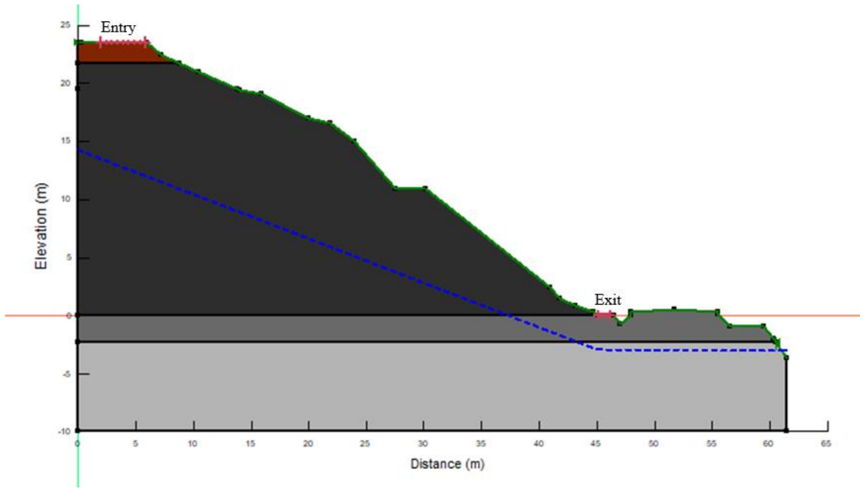

Figure 3.7. Model of the slope without any remediation showing the entry and exit ranges

b. Results and interpretation

Figure 3.8 shows the overall slip surfaces of the slope in its initial state. The resulting critical FoS of the unreinforced slope for both M-P and Spencer methods is shown in Table 3.5. Different analyses were performed along various parts of the slope and the lower part of the slope (Figure 3.9) showed an even more critical FoS (Table 3.6).

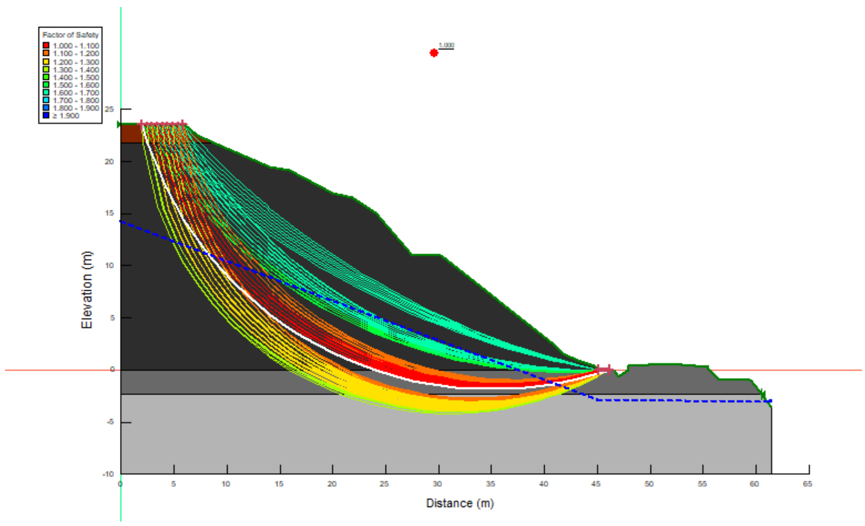

Figure 3.8. Unreinforced slope model showing the overall CSS and FoS

Table 3.5. Overall FoS of the unreinforced slope

Method	**Critical FoS**
M-P	1.000
Spencer	1.006

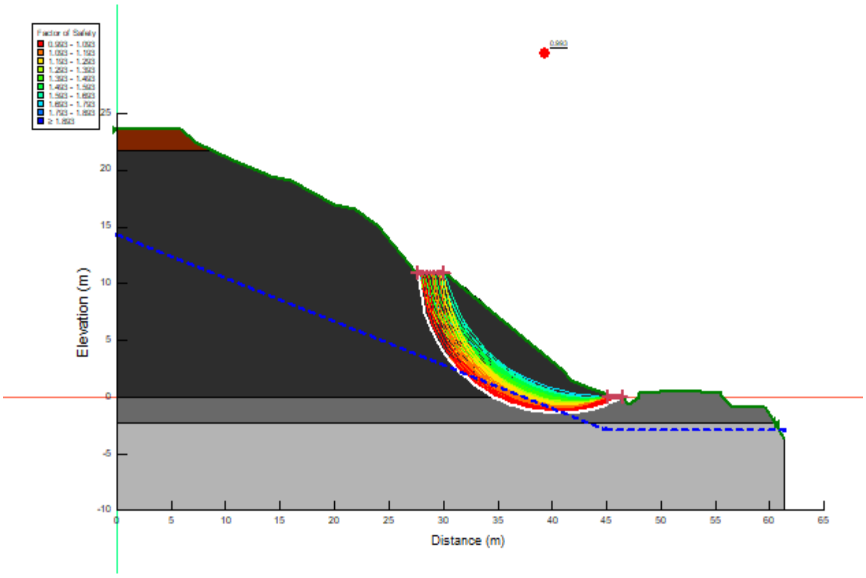

Figure 3.9. Analysis of the lower half of the unreinforced slope showing the CSS

Table 3.6. Critical FoS of the unreinforced slope

Method	**Critical FoS**
M-P	0.994
Spencer	0.994

The overall critical safety factor was found to have a value of 1.0 which is an indication of marginal stability. An observation of the slip surfaces on the lower half of the slope which is closer to the road shows an even more critical failure surface with a safety factor of 0.994. It is worth taking into account the influence of the groundwater level on the stability of the slope. For this reason, a sensitivity analysis was performed in order to demonstrate the change in the FoS with rising and falling water levels.

c. Sensitivity analysis

Figure 3.10 shows the variations in the water level that were made The FoS was checked for a 1m interval of rise and fall in water levels (from the initial water level) with increments of 0.1m. From Figure 3.11, it can be seen that the FoS corresponding to a 0.1m rise is equal to

0.995 and for a 0.1m drop in water level, the FoS is 1.005. A conclusion can therefore be drawn from here that a slight increase in the water level will render the slope unstable.

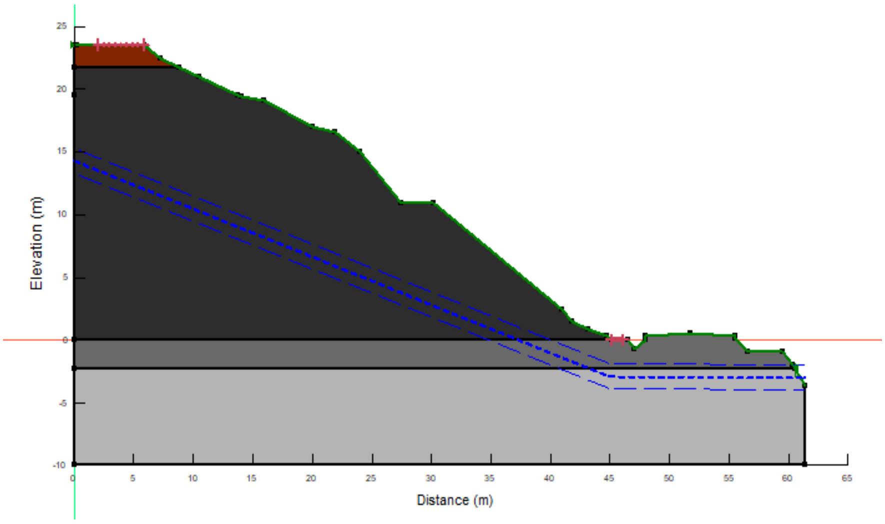

Figure 3.10. Model of the slope showing the variations in the groundwater

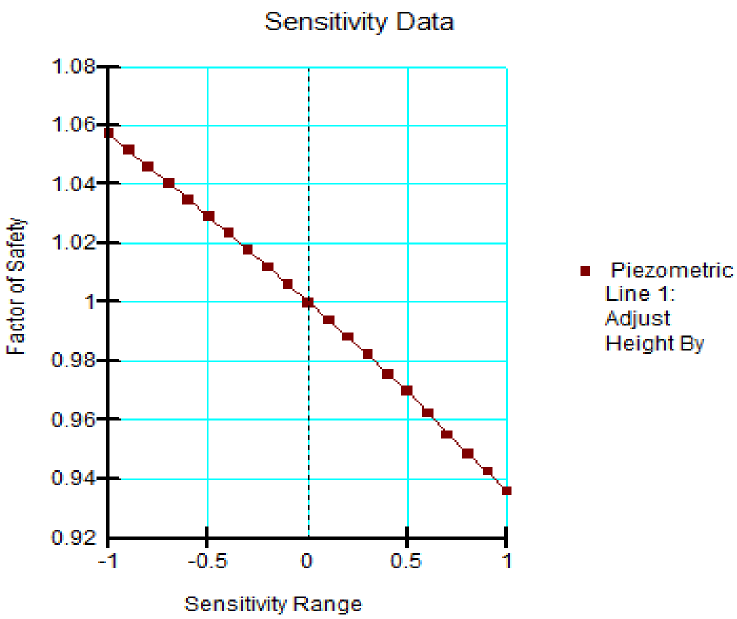

Figure 3.11. Plot of sensitivity data

3.3.3.2. Slope with Retaining wall

Two retaining walls were constructed to protect the roadway from intrusion by slope materials from subsequent slides but the solution failed to serve its purpose. This part investigates the slope supported by the walls.

a. Model of the slope with retaining wall

The model of the initially implemented solution is seen in Figure 3.12 showing the slope modification which was done before constructing the retaining walls. The retaining walls considered were gabion walls with heights of 7.22 m and 6.85 m upslope and downslope respectively with wall embedment depth of 0.6m. The properties of the fill materials considered were crushed stone with a unit weight of 17kN/m^3 and friction angle of 35°. The Mohr-coulomb strength model was also assigned to the fill material.

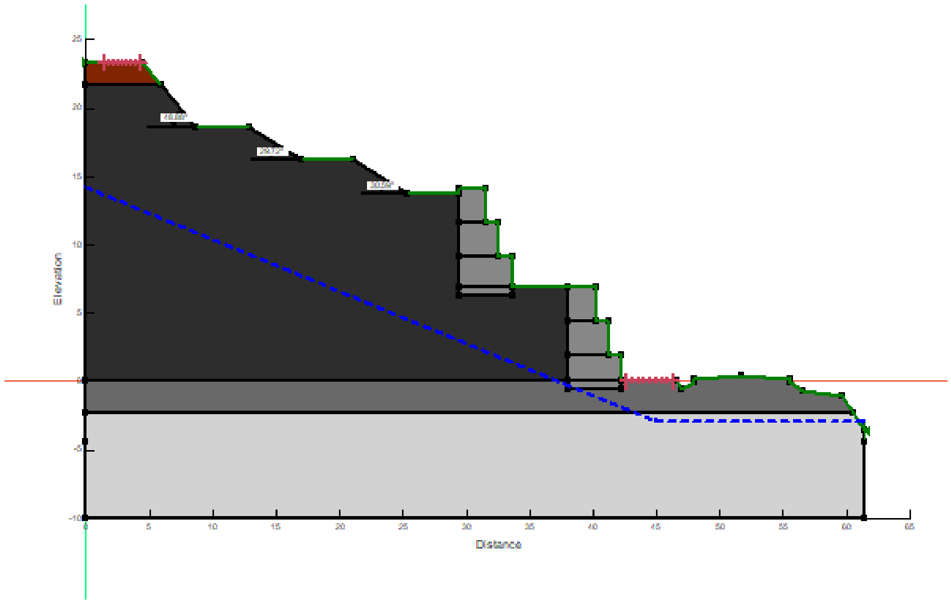

Figure 3.12. Illustration of the slope modification with the retaining walls

b. Results

After running analyses, the critical sliding surfaces obtained for both the entire slope and the external global stability of the walls are shown in Figure 3.13 and Figure 3.14 respectively. The factors of safety for both cases are presented in Table 3.7 and Table 3.8.

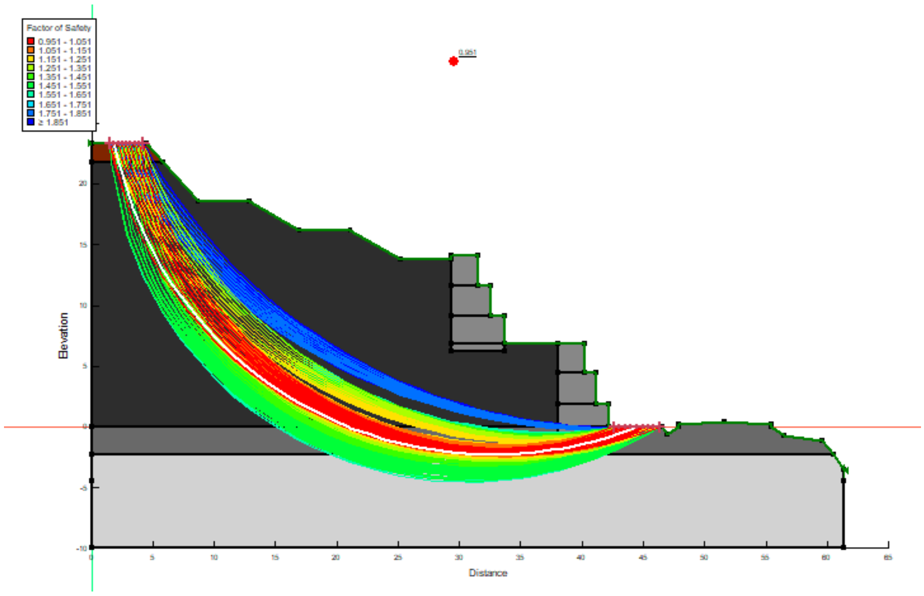

Figure 3.13. analysis of the slope with retaining wall showing the most CSS

Table 3.7. FOS for the CSS

Method	Critical FoS
M-P	0.951
Spencer	0.956

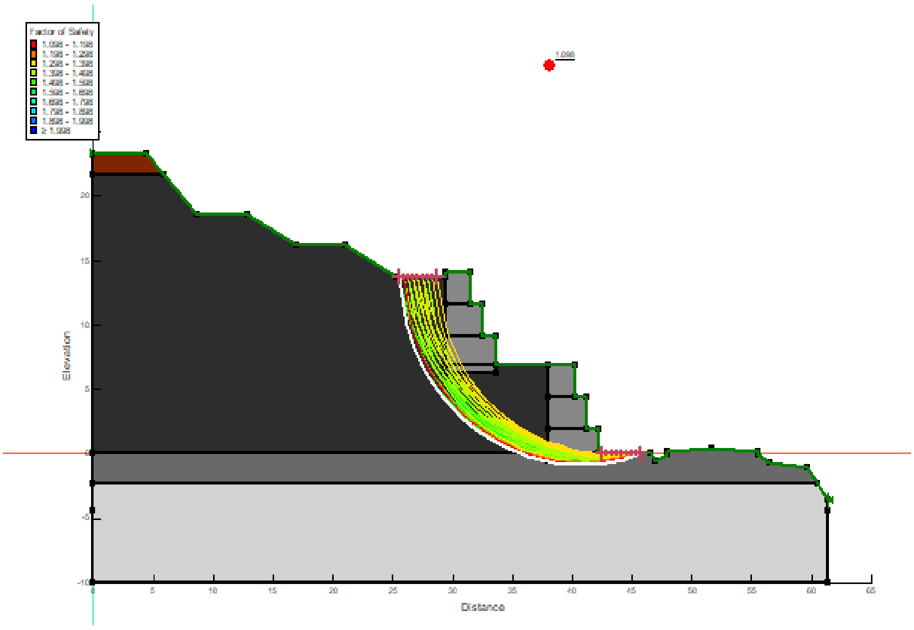

Figure 3.14. Analysis of slope with retaining wall

Table 3.8. FOS for the CSS of the lower half of the slope

Method	Critical FoS
M-P	1.098
Spencer	1.100

Figure 3.15 shows the free body diagram and force polygon for the most critical slice. The mobilized base shear force has a value of 61.518 kN.

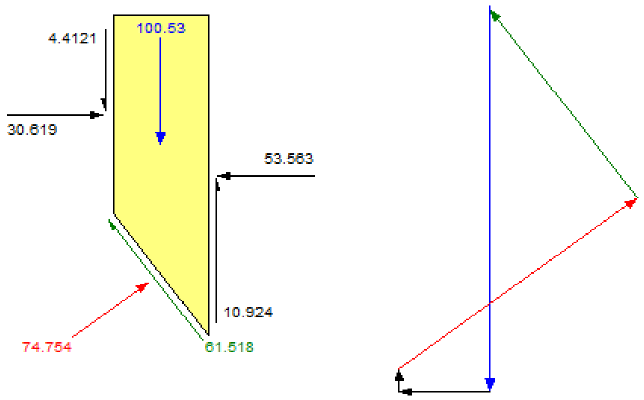

Figure 3.15. Free body diagram and force polygon for Slice 11

c. Interpretation

The lower half of the slope supported by the retaining walls has a safety factor of 1.098. For the entire slope, the overall FoS changed from 1.00 to 0.951. But these values of 1.098 and 0.951 are less than the recommended minimum FoS of 1.3 for retaining walls given in section 2.4. This FoS value indicates that the lower half of the slope is still very close to failure with the retaining wall and thus a slight increase in the groundwater will lead to a probable slope failure. On the other hand, the entire slope is unstable. It is therefore needful to design a complementary technique to make the slope more stable.

d. Overall stability checks

Before the design of a solution to complement the wall, it is essential to check which stability conditions are verified in order to better position the reinforcements. Table 3.9 and Table 3.10 give the hypotheses and computations made for both the upslope and downslope retaining walls respectively to verify their external stability.

Table 3.9. External stability verifications for the upslope wall

Hypothesis	
➢ Wall embedment depth of 0.6m ➢ Homogeneous soil behind the retaining wall ➢ Fill material with a unit weight of 17kN/m^3 and angle of friction 35$^{o.}$ ➢ No settlement of the fill after construction. ➢ Soil-wall friction is half the internal friction of the soil.	
Soil properties	
➢ Unit weight	16.56 kN/m^3
➢ Friction angle	34.4^o
➢ Acceptable bearing capacity, σ_{adm}	0.55 kN/m^2
Gabion wall	
➢ h1	2.9 m
➢ h2	2.43 m
➢ h3	2.49 m
➢ Wall height H	7.82 m
➢ b1	4.27 m
➢ b2	3.2 m
➢ b3	2.2 m
➢ Unit weight of fill material	17 kN/m^3
➢ Internal friction angle of fill material	35^o
➢ Wall weight, W	521.12kN/m
➢ X_G	2.52
➢ Y_G	1.30
➢ Wall base slope, α	0^o
➢ Ground surface inclination, β	0^o
Overturning verification	
➢ Ka	0.33
➢ Sa,h	161.06 kN/m
➢ Sa,v	98.77 kN/m
➢ Overturning moment, M_o	461.82 kNm/m
➢ Resistant moment, M_r	1451.75 kNm/m

➢ FoS of overturning ➢ Interpretation	3.14 > 1 Verified!
Sliding verification	
➢ Coefficient of active earth pressure ➢ Sa,h ➢ Sa,v ➢ Total Horizontal forces, Hd ➢ Total vertical forces, Nd ➢ Resistance to sliding, Rd,slid ➢ FoS of sliding ➢ Interpretation	0.26 106.35 kN/m 77.27 kN/m 143.57 kN/m 540.14 kN/m 392.43 kN/m 2.73 Verified!
Bearing capacity verification	
➢ Total vertical forces, Nd ➢ Summation bearing pressure on the base ➢ FoS ➢ Interpretation	692.68 kN/m 0.26 MPa 2.09 Verified!

Table 3.10. External stability verifications for the downslope retaining wall

Hypothesis	
➢ Wall embedment depth of 0.6m ➢ Homogeneous soil behind the retaining wall ➢ Fill material with a unit weight of 17kN/m^3 and angle of friction 35^o ➢ No settlement of the fill after construction. Soil-wall friction is half the internal friction of the soil.	
Soil properties	
➢ Unit weight ➢ Friction angle ➢ Acceptable bearing capacity, σ_{adm}	14.03 kN/m^3 9.65^o 0.22 kN/m^2
Gabion wall	
➢ h1 ➢ h2 ➢ h3	2.51 m 2.54 m 2.4 m 7.45 m

➢ Wall height H ➢ b1 ➢ b2 ➢ b3 ➢ Unit weight of fill material ➢ Internal friction angle of fill material ➢ Wall weight, W ➢ X_G ➢ Y_G ➢ Wall base slope, α	4.15 m 3.12 m 2.14 m 17 kN/m^3 0^o 399.11 kN/m 2.47 m 1.22 m 0^o
Overturning verification	
➢ Ka ➢ Sa,h ➢ Sa,v ➢ Overturning moment, M_o ➢ Resistant moment, M_r ➢ FoS of overturning ➢ Interpretation	0.76 288.07 kN/m 49.77 kN/m 786.90 kNm/m 1113.98 kNm/m 1.42 > 1 Verified!
Sliding verification	
➢ Coefficient of active earth pressure ➢ Sa,h ➢ Sa,v ➢ Total Horizontal forces, Hd ➢ Total vertical forces, Nd ➢ Resistance to sliding, Rd,slid ➢ FoS of sliding ➢ Interpretation	0.71 273.63 kN/m 46.53 kN/m 369.4 kN/m 461.92 kN/m 78.54 kN/m 0.21 Not verified!
Bearing capacity verification	
➢ Total vertical forces, Nd ➢ Summation bearing pressure on the base ➢ FoS ➢ Interpretation	601.62 kN/m 0.217 MPa 1.01 Verified!

The results in Table 3.9 and Table 3.10 show that all except the sliding condition for the downslope wall were satisfied for both walls. The complementary solution will have to take into account the stability of the wall too.

3.3.3.3. Complementary solution

From the results of the previous analyses, it is obvious that the slope is unstable even with the implemented solution. The technique designed to complement the retaining walls was soil nailing. This solution has the potential to provide a cost and time-saving alternative because the retaining wall will not need to be removed therefore significant volumes of excavation and backfill can be avoided. Also, the soil nailing will serve as a method to stabilize the landslide as well as support the downslope wall from eventual sliding failures. The technique comprises of sub-horizontally drilled and grouted soil nails through the retaining walls into the ground. The heads of the nails can be fastened in the retaining wall given that it is made of stone, it can serve as a bearing face.

a. Parameters for the numerical model

Table 3.11 presents the parameters used to define the soil nails in SLOPE/W. The values on the table were progressively selected and computed from the preliminary design specifications of the FHWA soil nailing systems based on the standard nail dimensions and were changed until the slope was seen to have a satisfactory FoS value. The nail standards are ASTM A615 Grade 80. The recommended properties for the cement grout are shown in Table 3.12.

Table 3.11. Soil nail parameters for the design

Layout	
Nail inclination:	20°
Nail lengths:	16 m
Vertical spacing SV:	2 m
Horizontal spacing SH:	1.2 m
Number of nails in the vertical direction:	3
Tendon capacity	
Bar diameter:	45 mm
Yield resistance, fyk:	827.37 MPa
Tensile resistance of tendon, Rt	1,377 kN
Determination of Pull-out resistance	
Drill hole diameter:	100 mm
Pull-out resistance per unit length rpo	31.4 kN/m
Bond strength, qu:	100 kPa

Table 3.12. Recommended grout properties (FHWA-NHI, 2015)

Minimum compressive strength at 28days	Between 21 to 28 MPa
Water/cement ratio	0.45
Consistency	Free from lumps and undispersed cement
Bleeding	Not to exceed 4% of the initial value. All bleed water shall be reabsorbed after 24hours
Volume change after 24 hours	Within the range 0% to +5%

b. Results and interpretation

The various slip surfaces of the reinforced slope with retaining walls are shown in Figure 3.16. The top row of nails is positioned at 1m below the top of the retaining walls. As seen in Table 3.13, the overall slip surface has a FoS value of 1.308 which corresponds to a stable soil-nailed slope as designated by FHWA and has been quite improved with the application of the soil nails to complement the wall.

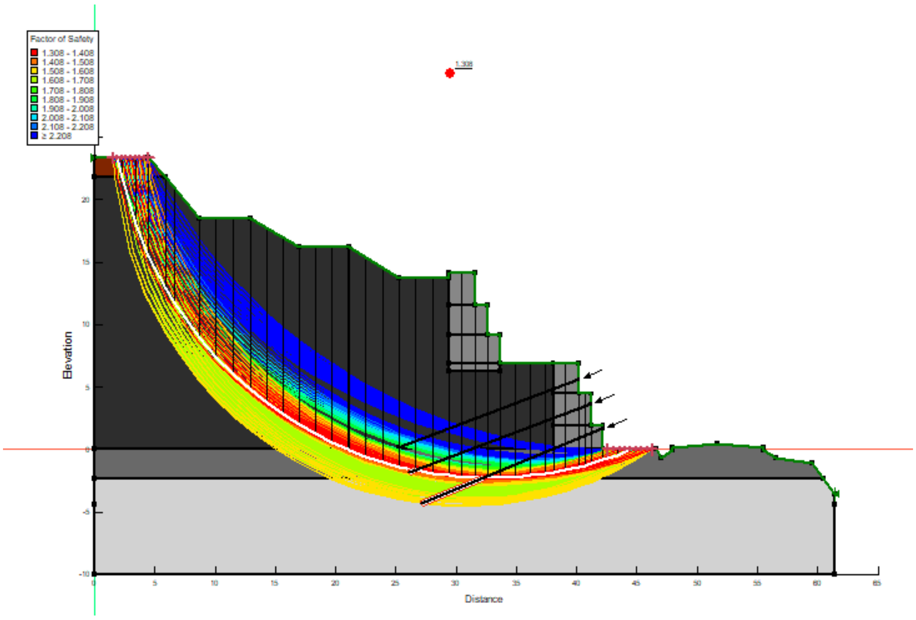

Figure 3.16. Analysis showing possible slip surfaces in the remediated slope

Table 3.13. Critical FoS for the analysis

Method	Critical FoS
M-P	1.308
Spencer	1.318

Figure 3.17 illustrates the force polygon for the forces in a slice whose base intersects with a soil nail. The force acting on the slice base due to the nail has a value of 15.078 kN.

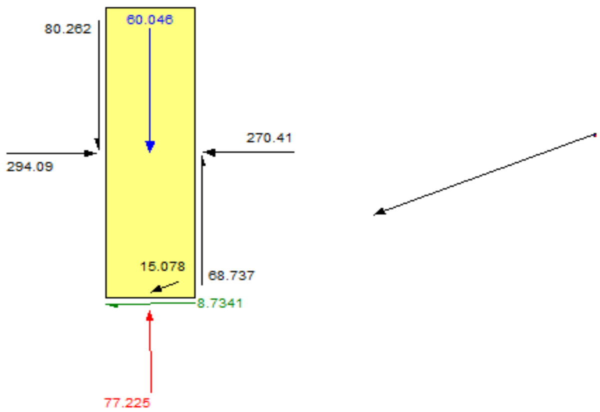

Figure 3.17. Slice showing soil nail force

3.3.3.4. Alternative stabilization technique

The details of the design and analysis of the proposed alternative solutions will be presented in this section beginning with the analysis of the unsupported slope.

a. Unsupported slope

Before the slope reinforcement, analyses were conducted on the unreinforced slope. The slope geometry considered is the same as the modified slope without the retaining wall.

i. Results

Figure 3.18 shows the slip surfaces on the excavated slope without any reinforcement or support. The factors of safety for the two LEMs used in this study to analyze this case are reported in Table 3.14. An analysis was also performed to verify the stability of the vertical excavations. The slip surfaces are shown in Figure 3.19 and the FoS for both LEMs are shown in Table 3.15.

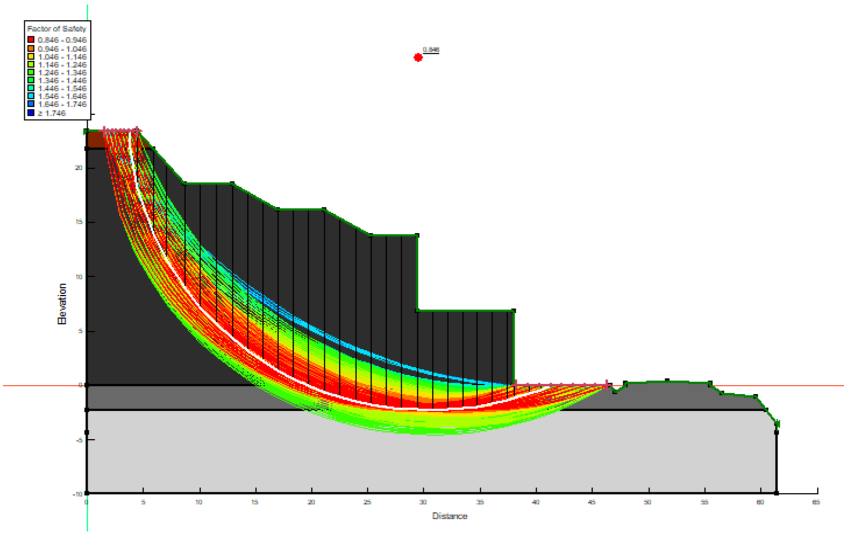

Figure 3.18. Analysis of the unsupported excavated slope

Table 3.14. Critical FoS for the analysis

Method	Critical FoS
M-P	0.846
Spencer	0.865

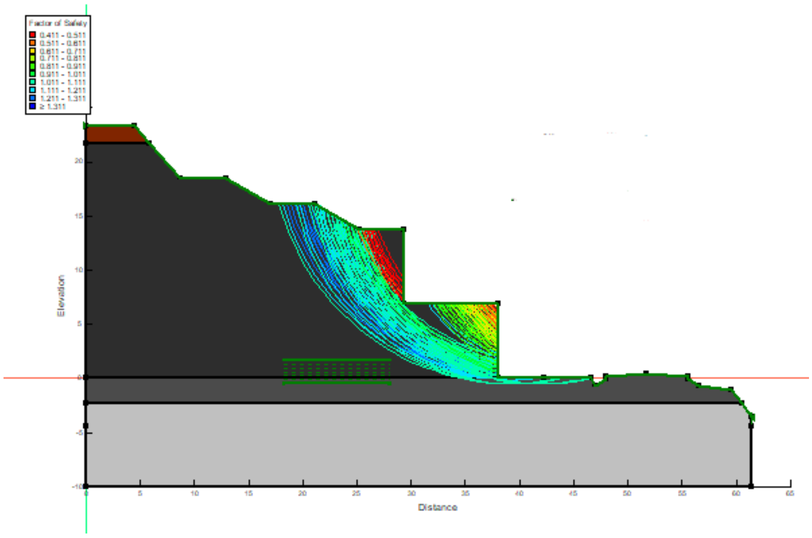

Figure 3.19. Analysis of the vertical excavated portions of the slope

Table 3.15. Critical FoS for the analysis

Method	Critical FoS
M-P	0.411
Spencer	0.411

ii. Interpretation

The FoS of the unsupported excavated slope $Fs_{initial}$ is 0.846 indicating instability with total resisting and driving forces having values 484.23 kN and 572.73 kN respectively. The vertical excavated portions show instability with a very low FoS of 0.411, thus the alternative solutions will have to consider that a support will be needed to prevent soil collapses at these locations.

b. Slope with micropiles

For the alternative solution, a micropile slope stabilizing system was designed. The grout properties are the same as those for the soil nails.

i. Micropile parameters and model

The solution consists of a system of two rows of FHWA micropiles of type A1 (gravity grout reinforced with steel reinforcing bar). The micropiles are of lengths 17 m and 10 m with a reinforcing bar whose outside diameter of cross-section is 43 mm and nominal yield stress of 520 MPa. The micropiles serve as both a retaining and slope stabilizing system. The spacing of 0.76 m was assumed to be small enough to prevent plastic flow between two adjacent piles on the same row. The micropile characteristics and layout are given in Table 3.16. The model of the solution is shown in Figure 3.20.

Table 3.16. Micropile design parameters

Assumptions	
➢ The micropile spacing considered is small enough to prevent plastic flow of soil between two adjacent piles on the same row	
➢ The required resisting force is the same across the entire slope	
Calculation of the required micropile resistance	
➢ FoS of the unsupported excavated slope, $F_{s\ (initial)}$	0.846
➢ Resisting forces	484.23 kN
➢ Driving forces	572.73 kN
➢ Target FoS	1.3
➢ Required micropile shear resistance, Frm-p	260.32 kN
Layout	
➢ Micropile diameter	230 mm
➢ Spacing (center to center):	0.76 m
➢ Upslope micropile length	17 m
➢ Downslope micropile length	10 m
➢ Direction:	90^{o}
Bar parameters	
➢ Steel reinforcing bar diameter	43 mm
➢ nominal yield stress fyk:	520 MPa
➢ Yield Strength of reinforcing bar, Rt:	751 kN
Shear resistance of the micropile	
➢ Shear resistance, SR:	340.4 kN

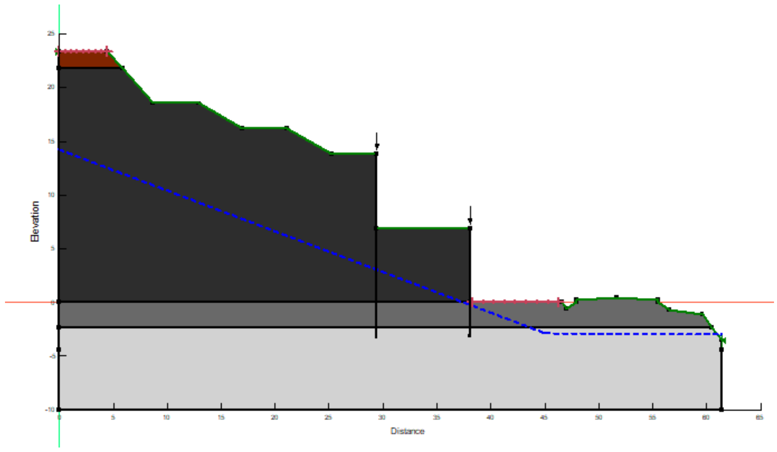

Figure 3.20. Slope with two rows of micropiles

ii. Results

After running the analysis, the critical slip surface obtained is shown in Figure 3.21. The overall safety factor of the slope increases to a value of 1.400 (Table 3.17) which is greater than the target FoS of 1.3 and therefore satisfactory.

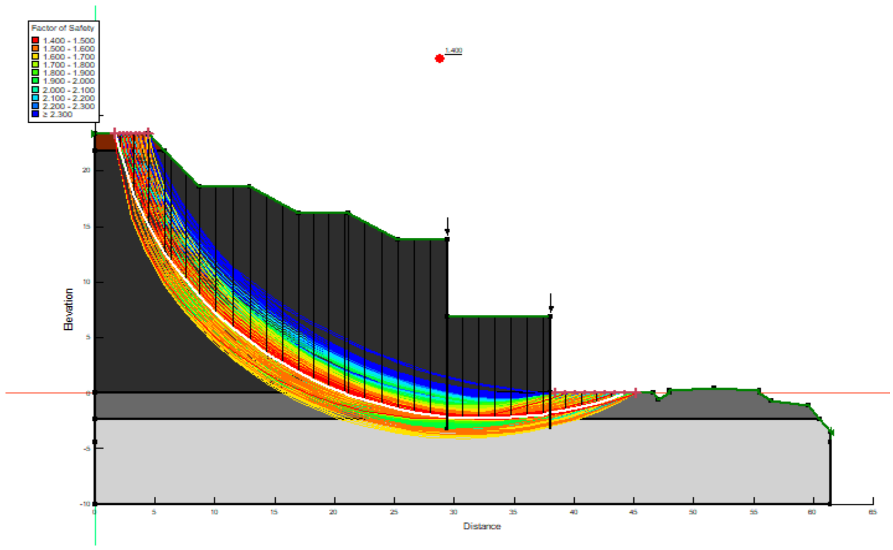

Figure 3.21. Analysis of the slope stabilized using two rows of micropiles

Table 3.17. Critical FoS for the analysis

Method	Critical FoS
M-P	1.400
Spencer	1.420

The mobilized micropile forces can be seen in Figure 3.22 oriented at the same angle as the bottom of the slice with a value of 136.52 kN implying that the available micropile shear resistance produced by the reinforcement is 191.13 kN.

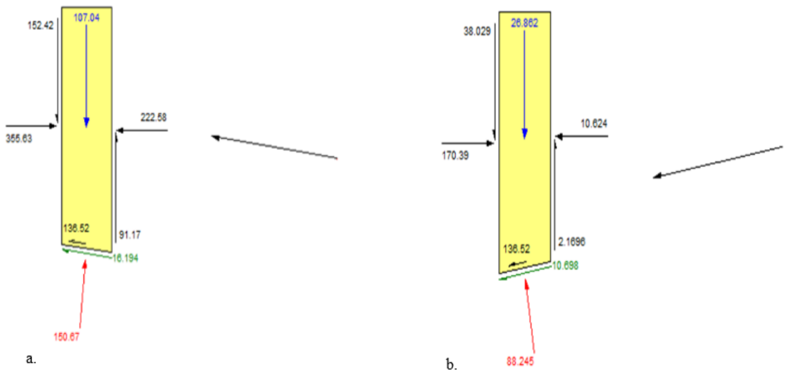

Figure 3.22. Slice at the position of a. upslope micropile; b. downslope micropile

For the minimum required length of the micropile below the slip surface, the upslope micropile was considered given that it supports greater loads. The length of the micropile above the slip surface is 15.87m implying the length below the slip surface has a value of 1.13m. Table 3.18 summarizes the calculation of the minimum required micropile length L_{below} which resulted in a value of 0.47m. L_{below} is less than 1.13m implying the length below the slip surface for the upslope micropile is sufficient to resist the axial loads.

Table 3.18. calculation of the minimum required micropile length (L_{below})

➢ Uniformly distributed load P, ➢ Angle between the slip surface and the micropile θ, ➢ Axial force, P' ➢ f_{all}	16.40 kN/m 82.63° 33.33kN 520kPa
➢ minimum required micropile length, L_{below}	**0.47m**

c. Anchored shotcrete

An anchorage of the slope with a shotcrete wall facing for shoring of the vertical excavation and also to serve as a bearing face for the anchor plate was considered as an alternative technique.

i. Model and anchor parameters

The model of this technique was represented using the parameters in Table 3.19 considering the anchor reinforcement loads integrated in SLOPE/W. The reinforcing elements considered are grade 150 prestressed bars with anchor unbonded length of 4m. The model of the solution is represented in Figure 3.23. The design specifications are given in this part.

Table 3.19. Ground anchor parameters

Assumptions	
➢ The internal friction angle of the sliding surface is equal to the internal friction angle of the soil.	
Layout of anchors	
➢ Horizontal spacing between adjacent anchors in a row, B:	1.5 m
➢ Number of anchors in the vertical direction, N:	4
➢ angle of inclination of the anchors, α:	30^o
Required preventive force	
➢ Slope of the slip surface, θ:	27.81^o
➢ Internal friction angle of the slip surface, ϕ:	9.5^o
➢ FoS of the slope before anchorage, Fs(initial):	0.846
➢ Target factor of safety Fs:	1.3
➢ Tangential force attributable to gravity, T:	747.27 kN
➢ Required preventive force, P:	508.64 kN/m
➢ Straining effect (tensile resistance) P1:	270.96 kN/m
➢ Clamping effect (shear resistance) P2:	430.45 kN/m
Design anchor force (pullout resistance)	
➢ Anchor pullout resistance Td:	281.74 kN
Anchor steel material	
➢ Nominal diameter:	26 mm
➢ Nominal cross-sectional area:	548 mm^2
➢ Ultimate stress, fpu:	1035 MPa
➢ Tensile capacity:	568 kN
➢ Prestressing force:	454 kN
➢ Anchor diameter, D:	100 mm
Anchor bond length	
➢ Allowable adhesive stress between the soil and grout, τab:	100 kPa
➢ Bond length:	12 m
Shotcrete facing	
➢ Thickness:	150 mm
➢ Compressive strength, f'c (FHWA):	21 MPa

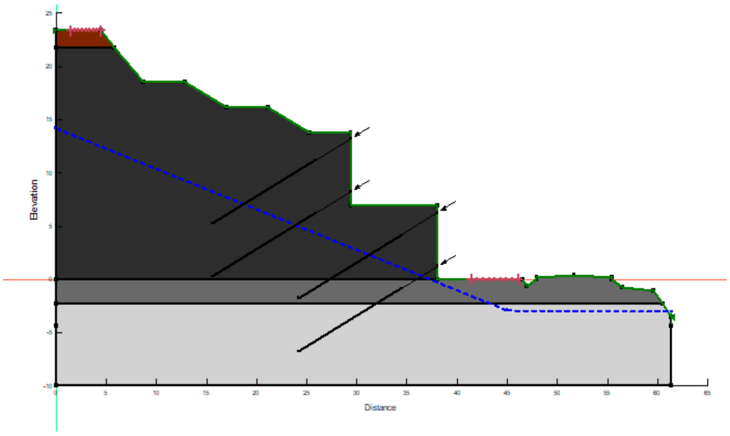

Figure 3.23. Model of slope reinforced with ground anchors

ii. Results

The resulting critical slip surface is shown in Figure 3.24. The FoS for the anchored lateral support yielded a value of 1.329 (Table 3.20). This is an acceptable safety factor according to FHWA minimum FoS for external stability of anchored excavations (between 1.3 to 1.5).

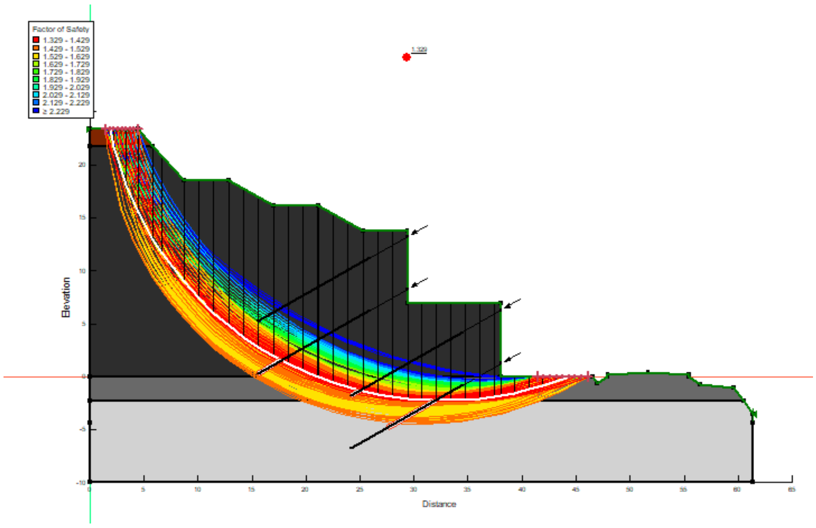

Figure 3.24. Analysis showing the critical slip surface of the slope reinforced with ground anchors

Table 3.20. Critical FoS for the analysis

Method	Critical FoS
M-P	1.329
Spencer	1.338

3.4. Comparison of the alternatives proposed for the landslide stabilization

Using the criteria given in chapter two, the proposed remediation techniques will be compared in this part to see which is the optimal solution.

3.4.1. With respect to FoS

Table 3.21 gives a summary of the FoS obtained from the analysis with the different stabilization techniques. The assessment of the safety factors shows that the micropiles offer greater safety than the other techniques.

Table 3.21. Techniques and their respective FoS

Technique	FoS
Retaining wall plus soil nails	1.308
Micropiles	1.400
Ground anchors	1.329

3.4.2. With respect to the cost of materials

The materials were estimated for a linear meter of the slope. As seen from the numerical models, the excavated slope was considered for modelling the stabilization techniques. For the soil nailing technique, designated in section 3.2 is the nail spacing which is 1.2m horizontal spacing and the number of rows used (3 rows of 16m long nails). The nails are 45mm in diameter with drillhole diameter of 100mm. For the alternative stabilization technique using micropiles, 2 rows of 230mm micropiles of lengths 10m and 17m, spaced at 0.76m are required. The reinforcement bars have a diameter of 43mm. The anchored shotcrete technique consists of 4 rows of ground anchors spaced horizontally at 1.5m and vertically at 5m for each pair on the excavated slope with a thickness of 150mm for the concrete facing. The estimate of the total quantity and cost of the techniques needed per meter of the span of the affected slope sections is presented in Table 3.22.

Table 3.22. Quantity and cost estimate for the stabilization techniques

Excavation				
Material	Unit	Quantity per linear meter of slope	Unit price (FCFA)	Total cost (FCFA)
Earth	m^3	40.9	3585	146,627
Retaining wall plus soil nails				
Material	Unit	Quantity per linear meter of slope	Unit price (FCFA)	Total cost (FCFA)
Steel bars	m	80.16	20, 878	1,670,240
Grout	m^3	0.06	217,175	13,135
Crushed stone	m^3	49.11	13,100	643,341
Total cost (FCFA)			**2,473,343**	
Micropiles				
Material	Unit	Quantity per linear meter of slope	Unit price (FCFA)	Total cost per linear meter of slope (FCFA)
Steel bars	m	44.28	14,075	623,241
Grout	m^3	1.195	217,175	259,542
Total cost (FCFA)			**1,029,410**	
Anchored shotcrete				
Material	U	Quantity per linear meter of slope	Unit price (FCFA)	Total cost per linear meter of slope (FCFA)
Tendon	m	86	20,878	1,781,589
Grout	m^3	0.27	217,175	59,767
Shotcrete	m^3	2.06	175,000	360,500
Total cost (FCFA)			**2,348,483**	

From Table 3.22, it can be concluded that the use of micropiles to stabilize the slope is more economical. However, the estimates of transportation costs, labor and equipment also have to be taken into account.

Conclusion

This chapter had the objective of presenting the results obtained after applying the methodology described in chapter 2. The first part consisted of the general presentation of the site through a description of the location, climate, relief, soils, vegetation, hydrography, demography and the economic activities of the population in the study area. Next, the geotechnical and geometrical data collected were presented. A comprehensive analysis of the data collected was done to refine the assemblage of the parameters essential for the stability analysis. In addition, the numerical models and results obtained were presented. Finally, a comparison of the techniques used to stabilize the landslide was done using a FoS and cost criteria in order to make a choice of the most suitable solution for the landslide. The micropile technique was seen to be the optimal solution giving a relatively higher FoS than the other techniques and also costing less thus making it the most suitable choice for the landslide stabilization.

GENERAL CONCLUSION

The general objective of this work was to propose solutions to the recurrent landslides along the Batibo-Numba road section of the NR6 in Cameroon and identify the possible causes of these recurrences despite the construction of retaining walls for the landslide remediation. In order to achieve this objective, stabilization techniques were proposed and modelled using the numerical software tool SLOPE/W. The first chapter of this work gave a general idea about landslides beginning with the development of a few concepts describing soils and their characteristics, the major features associated with landslides, and the causes and effects of landslides. In addition, the various methods for assessing slope stability were presented and concluding the chapter was with the description of different slope stabilization techniques. Chapter two detailed the methodology adopted comprising of site recognition, data acquisition, the design of the proposed landslide stabilization techniques, a description of the numerical modeling procedure and a presentation of the criteria for selecting a choice stabilization technique. Lastly, the results obtained were presented in Chapter 3 of this work. The geotechnical parameters used for the stability analysis were gotten from the laboratory and field test results. Four sections of the road (PK 12, PK16, PK19 and PK20) were identified to have been affected by landslide occurrences. Among these, the strength parameters of the soil at PK19 were used for the numerical analyses which were conducted on SLOPE/W. The numerical analyses were performed on an idealized cross-section of the slope at PK19 and the results obtained were evaluated in order to assess the stability of the slope under study. When the slope was shown to be unstable, remedial solutions were proposed and analyses were conducted in order to verify the stability of the slope under the action of these reinforcement loads. The solutions proposed include soil nailing to complement the retaining wall, a micropile system and an anchored shotcrete support of the excavated slope. To end with, a comprehensive estimate of the cost of the materials needed to stabilize a linear meter of the slope span was done. A choice of the most suitable technique on the basis of FoS and cost was made. The micropile slope support was seen to be the most suitable technique giving a safety factor of 1.4 with a material cost estimate of 1,029,410 FCFA. The anchored shotcrete supported slope gave a FoS of 1.329 with an estimate of 2,348,483 FCFA for the cost while the soil nailed-retaining wall composite gave a FoS of 1.308 with a cost estimate of 2,473,343 FCFA, thus making it the most expensive in terms of materials cost and the least efficient amongst all the proposed techniques.

From the numerical analyses, the major causes of the recurrence of landslides along the road-side slope can be identified:

- Initially, the entire slope was not studied and the retaining wall was used to stabilize the slope locally. From the safety factor calculation, the margin of stability is not so wide after the application of the retaining wall. Also, from the analysis of the overall stability of the entire slope, the slip surfaces pass below the retaining wall which implies the wall which is not good for the system's efficiency in providing support to the slope. This is the reason why it failed and the landslides became recurrent.
- From the sensitivity analysis conducted, it is possible that with a 10cm rise in the water levels perhaps during the rainy season, the slope can be rendered unstable thus causing slope failures.

The major difficulties encountered during this study include limited geotechnical data on the slope at the various kilometric points which made it difficult to obtain an accurate model of the slope. Also, the current crisis in the region made it difficult to conduct a site visit for better observation and further geotechnical site exploration.

This research applied a numerical modeling tool to conduct stability analyses and perform slope reinforcement. Based on the results obtained from the numerical analyses, it can be concluded that the slope can be effectively stabilized using the proposed solutions. However, drainage measures should be incorporated with the techniques in order to reduce the effects of rising water levels on the stability of the slope. Also, there is a need to carry out more field investigations for a better understanding of the soil movements at the site.

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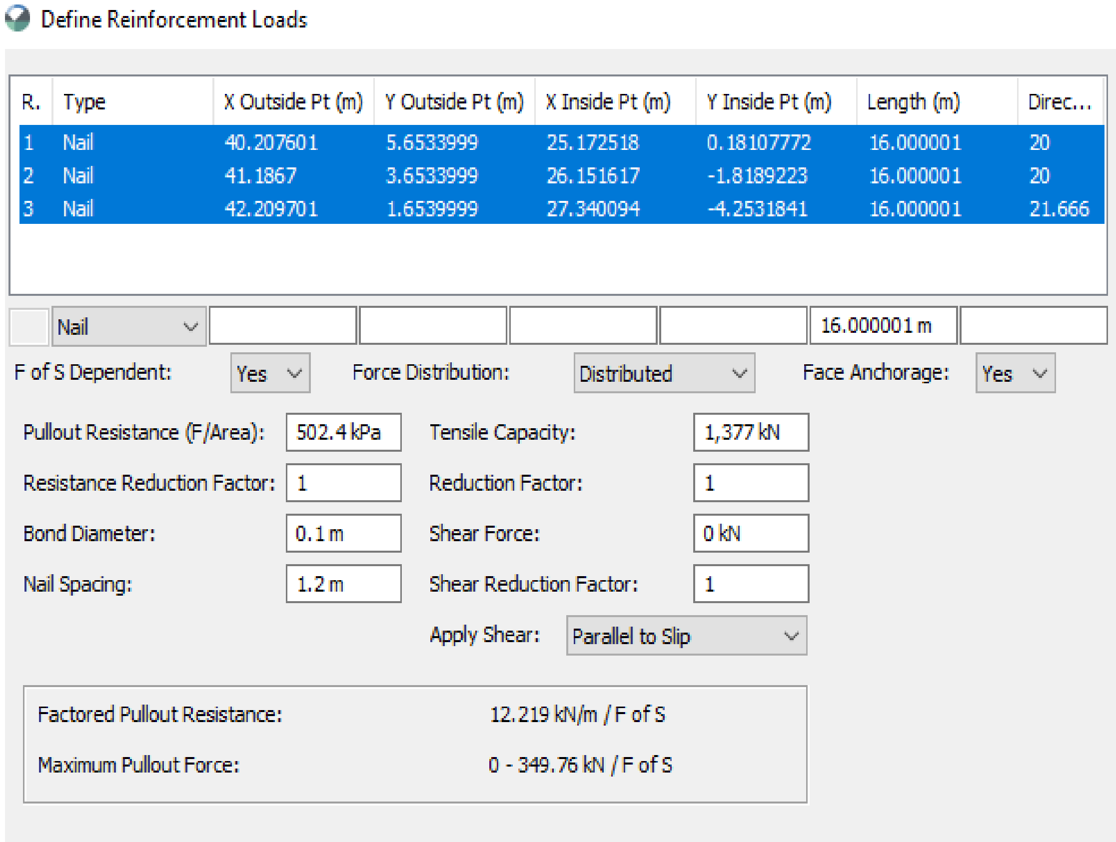
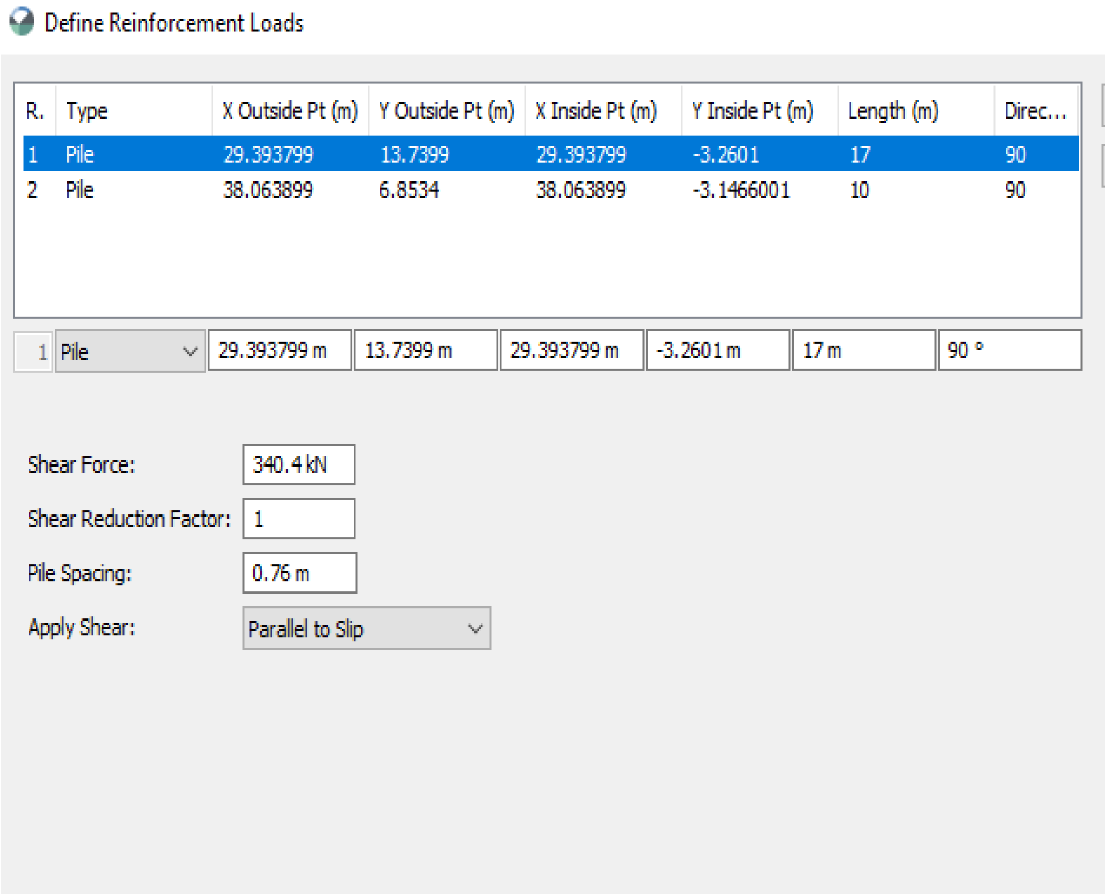
APPENDICES

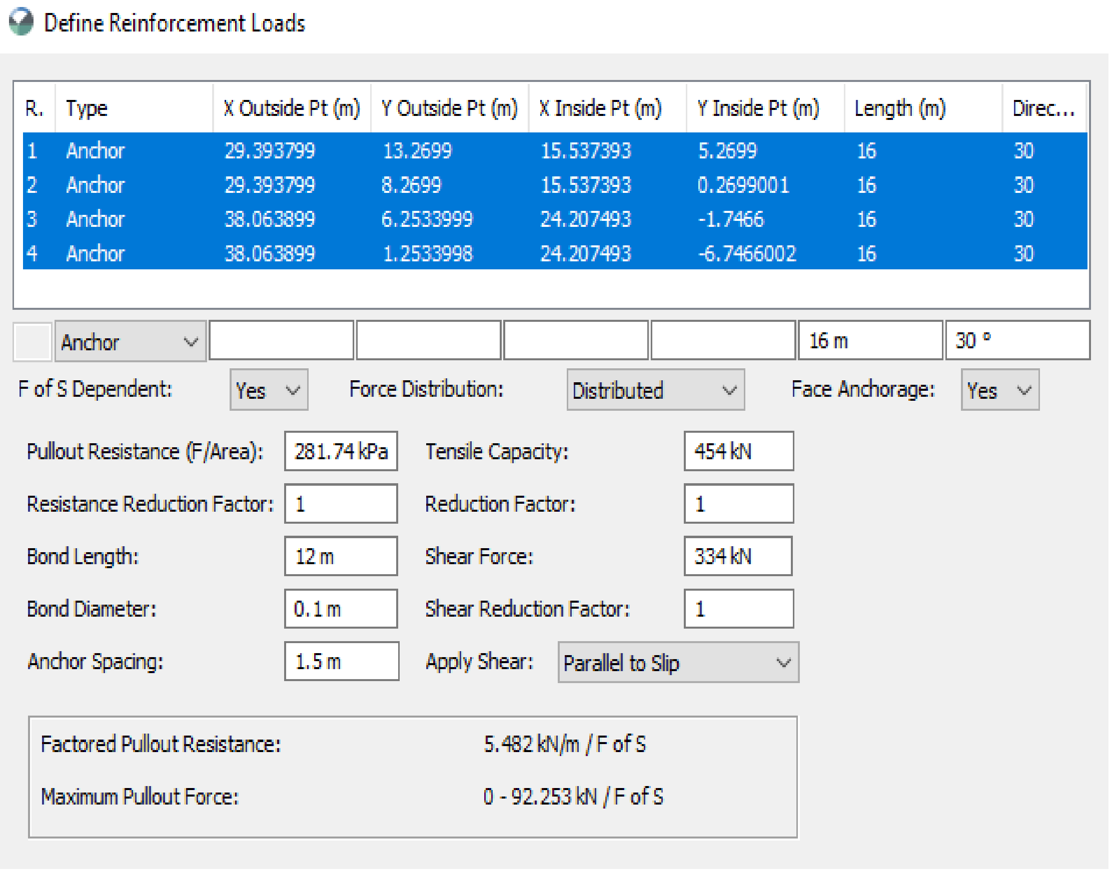
Appendix A: Soil nail parameter specification on SLOPE/W



Appendix B: Micropile definition on SLOPE/W



Appendix C: Anchor definition on SLOPE/W



Define Reinforcement Loads

R.	Type	X Outside Pt (m)	Y Outside Pt (m)	X Inside Pt (m)	Y Inside Pt (m)	Length (m)	Direc...
1	Anchor	29.393799	13.2699	15.537393	5.2699	16	30
2	Anchor	29.393799	8.2699	15.537393	0.2699001	16	30
3	Anchor	38.063899	6.2533999	24.207493	-1.7466	16	30
4	Anchor	38.063899	1.2533998	24.207493	-6.7466002	16	30

Anchor | 16 m | 30 °

F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes

Pullout Resistance (F/Area): 281.74 kPa
Tensile Capacity: 454 kN
Resistance Reduction Factor: 1
Reduction Factor: 1
Bond Length: 12 m
Shear Force: 334 kN
Bond Diameter: 0.1 m
Shear Reduction Factor: 1
Anchor Spacing: 1.5 m
Apply Shear: Parallel to Slip

Factored Pullout Resistance: 5.482 kN/m / F of S
Maximum Pullout Force: 0 - 92.253 kN / F of S