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DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONNEMENTAL ENGINEERING

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MODELLING THE TRIGGER OF A LANDSLIDE DUE TO SUCTION DISSIPATION: CASE OF A SLOPE IN PELOPONNESE (GREECE)

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

Curriculum: Geotechnical Engineering

Presented by

YOGNO YAMBA Hyacinthe

Student number: **15TP21026**

Supervised by

Prof. Simonetta COLA

Academic year: 2019/2020

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DEDICATION

To my lovely parents,

Mr. YAMBA Josephe and **Mrs. YAMBA Lucie**.

ACKNOWLEDGEMENTS

We may first thank **GOD OUR LORD,** the most merciful, for the renewed breath of life, for the health, the will and determination to begin and carry out this work.

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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation
USCS	Unified Soil Classification System
USDA	United States Department Of Agriculture Soil Classification System
FS	Factor of Safety
LEM	Limited Equilibrium Method
SWCC	Soil Water Characteristic Curve
SRM	Strength Reduction Method
GLE	General Limit Equilibrium
ϕ'	Effective friction angle
ϕ^b	Angle defining the increase in strength due to suction
θ_r	Residual volumetric water content
θ_w	Saturated volumetric water content
σ'	Effective stress
σ_v	Vertical total stress
τ_f	Shear strength
u_a	Pore air pressure
u_w	Pore water pressure
γ_w	Unit weight of water
c'	Effective cohesion
S	Saturation degree
ω	Water content
e	Void ratio
n	Soil porosity
ρ	Bulk density

ABSTRACT

The main objective of this thesis was to model the trigger of the landslide of a slope at Peloponnese that occurred in February 2003 with numerical simulations, exploring the effect of suction decrease, in turn affecting the soil strength. Once the instability behaviour is understood, different risk mitigation measures can be recommended to stabilise the slope with the help of numerical modelling. To achieve this main objective, a preliminary description of the generalities on soils, landslide, and landslide stabilisation techniques has been analyse. Documentary research that led to the collection of data for the landslide event that occurred in February 2003 are used for the deterministic back analysis of the slope in Peloponnese (two-dimension numerical model). The deterministic two-dimensional numerical modelling was used to perform safety analysis with SEEP/W (to access the soil pore pressure distribution with the finite element method) and SLOPE/W (to access the slope's safety factor with the limit equilibrium method) from GeoStudio 2012 software. From the stability back analysis, the residual angle value (17.2°) obtained for the weak soil layers was compared and found reasonable to the published data (between 16° and 20°). The back analysis before the landslide showed a higher safety factor for the slip geometry in 2003 (FS=1.17) compared to the slip geometry in 2001 (FS=1.00) simulating the precarious slope conditions in the year of 2001. The increase in the water table had a destabilising effect on the Peloponnese slope, which represents the final trigger due to suction dissipation (FS=1.00) in February 2003. Eventually, at the end of the different stability analyses or simulations on the problem submitted, risk mitigation measures that could have prevented the occurrence of the landslide in 2003 were suggested. As risk mitigation measures the coupling of horizontal drains or piles gave respectively a safety factor of 1.42 and 1.41. The results obtained in this work were convincing indicators of the effectiveness in the numerical modelling of a landslide on site, proposing adequate risk mitigation measures that are certainly recommended for similar future projects worldwide.

Keywords: Landslide; Numerical modelling; Unsaturated soil; Soil strength; Risk mitigation measures.

RESUME

L’objectif principal de ce mémoire était de modéliser le déclenchement du glissement de terrain d’une pente à Péloponnèse survenu en février 2003 avec des simulations numériques, en explorant l'effet de la diminution de la succion du sol, à son tour affectant la résistance du sol. Une fois le comportement d’instabilité compris, différentes mesures d’atténuation des risques peuvent être recommandées pour stabiliser la pente à l’aide de la modélisation numérique. Pour atteindre cet objectif principal, une description préliminaire des généralités sur les sols, les glissements de terrain et les techniques de stabilisation des pentes a été analysée. Les recherches documentaires qui ont conduit à la collecte de données pour l’évènement de glissement de terrain qui s’est produit en février 2003 sont utilisées pour la modélisation numérique de l'analyse rétrospective de la pente de Péloponnèse (modèle numérique bidimensionnel). La modélisation numérique bidimensionnelle est utilisée pour effectuer une analyse de sécurité avec SEEP/W (pour accéder à la distribution de la pression interstitielle du sol par la méthode des éléments finis) et SLOPE/W (pour accéder au facteur de sécurité de la pente par la méthode de l'équilibre limite) à partir du logiciel GeoStudio 2012. De l'analyse rétrospective, la valeur de l'angle résiduel (17,2°) obtenue pour les couches de sol faibles a été comparée et jugée raisonnable par rapport aux données publiées (entre 16° et 20°). L'analyse rétrospective avant le glissement de terrain a montré un facteur de sécurité plus élevé pour la géométrie de glissement en 2003 (FS=1,17) par rapport à la géométrie de glissement en 2001 (FS=1,00) simulant les conditions de pente précaire en 2001. L'augmentation de la nappe phréatique a eu un effet déstabilisant sur la pente de Péloponnèse, qui représente le déclenchement final dû à la diminution de la pression d’aspiration (FS=1,00) en février 2003. Finalement, à la fin des différentes analyses ou simulations de stabilité sur le problème soumis, des mesures d'atténuation des risques qui auraient pu empêcher l'apparition du glissement de terrain en 2003 ont été suggérées. Comme mesures d'atténuation des risques, le couplage des drains horizontaux avec des clous de sol ou pieux a donné respectivement un facteur de sécurité de 1,42 et 1,41. Les résultats obtenus dans ce travail ont été des indicateurs convaincants sur l'efficacité dans la modélisation numérique d'un glissement de terrain sur site, proposant des mesures adéquates d'atténuation des risques qui sont certainement recommandées pour des projets futurs similaires dans le monde entier.

Mots-clés : Glissement de terrain ; Modélisation numérique ; Sol non saturé; Résistance du sol; Mesures d'atténuation des risques.

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GENERAL INTRODUCTION

Natural disasters are aggressive events capable of causing damage to the physical and social space where they occur not only at the time of their occurrence but also in the long term. These consequences affect the safety of people and property, having a major impact on society and infrastructures. Earthquakes, volcanoes, floods, landslides, storms and tsunamis are the main types of natural disasters. Slope movements include one or more of the mechanisms of flow, fall, topple, slide, creep and spread. They can occur on natural or artificial slopes. With climate change, landslides have become the most recurring natural phenomena in recent decades. Landslides can be caused by earthquakes, surface freezing and thawing, ice melt, the collapse of groundwater reservoirs, and volcanic eruptions. But torrential rains most commonly activate landslides. Landslides occur worldwide and are responsible for thousands of deaths and injuries.

Despite the considerable progress in the understanding of soil behaviour, it is very difficult to predict when or where a landslide may happen. Nevertheless, it is recognized that rainfall-induced landslides are caused by significant changes in pore water pressures, which in turn affect the effective stress regime. A large number of soil slopes in nature are in various saturated conditions, with the phreatic line located at some depth below the ground. The increase in pore water pressure in unsaturated residual soils leads to a reduction in matric suction and thus a decrease in shear strength. This reduction in shear strength causes instabilities in such soils. The shear strength of the problematic layer of soil may be estimated through a back analysis in correlation with laboratory test results. This said safety analysis with numerical simulations will be carried out considering a slope in Peloponnese whose movement was triggered by a prolonged very wet season.

The main objective of this thesis is to model the trigger of the Tsakona landslide event that occurred in February 2003 with numerical simulations, exploring the effect of suction decrease on the soil shear strength and suggesting risk mitigation measures that could have prevented landslide occurrence. The residual friction angle conditions are firstly explored. Then, the effect of suction decrease is simulated as a consequence of the prolonged rainfall that occurred. A preliminary literature study concerning a suitable constitutive model able to take into account the soil-suction dependence will be analysed, to be further adopted in the numerical analysis. The software GeoStudio will be used to conduct the numerical simulations. The SLOPE/W module will be used for the limit equilibrium method. The SEEP/W module will be used to reproduce the pore water pressure condition of the soil through the finite element method. Lastly, simulation of landslide stabilisation interventions will also be carried out by assessing the safety factor improvement with different combined techniques.

To achieve the objective, the present case study is divided into three chapters. The first chapter provides a review of landslides starting with soils, shear strength governing slope stability, landslide, and landslide stabilisation techniques. The second chapter shows the methodology followed to meet the expectations of this research. The third chapter presents the study area, data collected and results of the methodology applied to the case of the slope in Peloponnese (Tsakona landslide).

CHAPTER 1. LITERATURE REVIEW

Introduction

Slopes are widespread in nature due to geological causes or human activities. Those slopes are subjected to several agents that tend to change their topography, leading to a possible failure mechanism known as a landslide. A thorough study of landslides requires knowledge of several concepts that will allow us to describe them. This chapter gives a general overview of soils thorough its definitions, composition, formation, properties, typology and classification. Furthermore, shear strength governing slope stability will be discussed through the explanation peak, fully softened and residual shear strength. Unsaturated shear strength will take into account the additional strength offered by soil suction. The process of progressive shear failure will also be discussed. Moreover, landslide's definition, classification and triggering factors are also described. Eventually, landslide stabilisation measures on slopes will be analysed.

1.1.Soils

Soil can be perceived in several ways depending on the field of study, pedology, soil geology, soil biology, agrology, botany, geochemistry, ecology, and geotechnics. With the evolution of time, different scientists have given many different definitions to the soil to show the evolution of the modern concept of soil.

1.1.1. Definitions

From a geotechnical point of view, the materials making up the earth's crust can be divided into two main categories: rocks and soils. Rocks (silica, feldspar limestone,) are hard materials that can only be broken up with consistent mechanical effort. As reported by Johnson & DeGraff (1988), the soil consists of the mass of solid particles produced by the physical and/or chemical disintegration of bedrock found in various thicknesses mantling the ground surface.

Conforming to Dokuchaev (1893), soil means the outer horizons of rocks naturally modified by the mutual influence of water, air, and living and dead organisms which is an independent and varying natural body.

From the perspective of civil engineering, the soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void space between the particles containing water and/or air (Herrmann & Bucksch 2014). Weak cementation can be due to carbonates or oxides precipitated between the particles or due to organic matter. Residual soil is the material resulting from the in situ weathering of rocks remaining at the same location of origin with little or no movement of individual soil particles. If the products are transported and deposited in a different location they constitute a transported soil, the agents of transportation being gravity, wind, water, and glaciers. During transportation, the size and shape of particles can undergo change and the particles can be sorted into size ranges.

1.1.2. Composition of soils

In geotechnical engineering, soils are considered three-phase material composed of; rock or mineral particles, water, and air. The soil's void is the spaces in between mineral particles, that contain the water and air as shown in Figure 1.1. There is an infinite variation of composition like; gravel and sand fractions (coarse particles or granular particles), silt or clay fractions (fine particles or cohesive) present alone or mixed with organic materials.

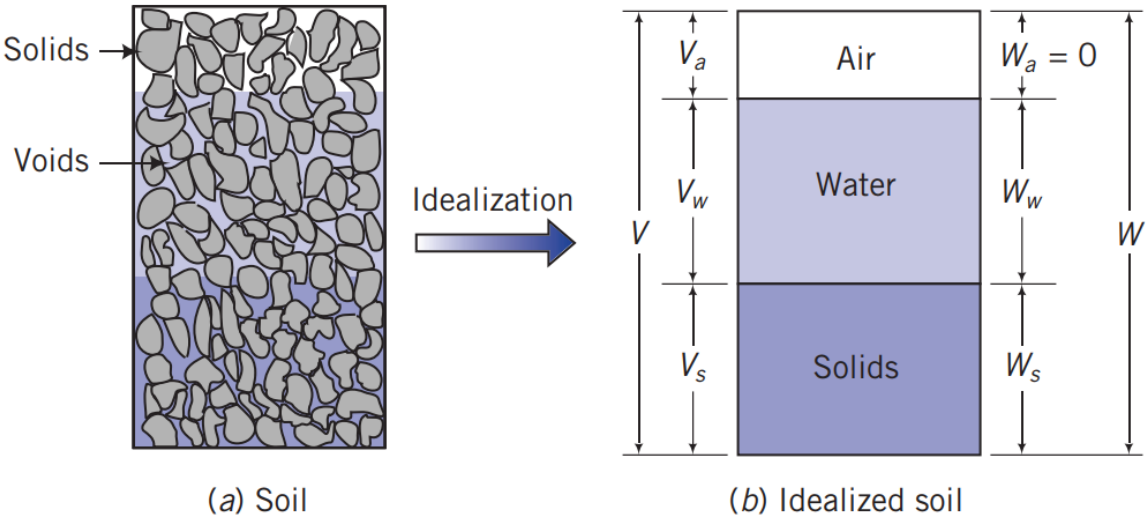


Figure 1.1. Phase diagram of soil (Budhu, 2015)

1.1.3. Formation of soils

Soils are formed either by physical disintegration (mechanical weathering) or chemical decomposition of rocks which interact with micro-organisms that decide the nature, colour, and chemical properties of the soil. Soil formation is affected by different factors and processes.

1.1.3.1. Factors influencing soil formation

The principal factors which influence soil formation are the parent material, climate, organisms, topography, and time.

Soil parent material is the material that soil develops from, the rock that has decomposed in place, or material that has been deposited by wind, water, or ice. Soil texture is greatly influenced by parent material which in turn affects the movement of water and nutrients. Parent material influence on soil formation process and properties tends to decrease with time as it is altered.

Precipitation and temperature are the two most important climatic factors that affect physical, chemical, and biological weathering. Temperature directly affects the speed of chemical reactions. Precipitation governs water movement in the soil. The activity of soil organisms plays a significant role in soil formation by accumulating organic matter and cycling nutrients.

Organisms such as bacteria and gophers can speed up or slow down soil formation. Bacteria and other microorganisms can facilitate chemical reactions or discharge organic substances to improve water infiltration in the soil. Organisms like gophers slow down soil formation by digging and mixing soil materials and soil horizons that have formed.

Topography is generally described in terms of slope, elevation which determines the effect of heat, wind, ice, and water on the parent rock. Steep regions with no greenery will be more affected by wind, whereas regions with heavy rainfall have moist soil and the mineral accumulation will also be more.

The accumulation period is another important factor determining soil properties. In a soil's life, different stages of temporal evolution can be distinguished: neo-formation, youth, maturity, senility. This evolution is quite slow, but its duration varies from one type of soil to another. From time to time, the soil profile changes, and all the other factors become time dependent.

1.1.3.2. Process of soil formation

The basic source for soil formation is the rock. Rock formation though its genesis (magmatic, sedimentary, metamorphic and paragenesis) ends up in the process of soil formation. The transition process from the rock to the soil takes place in four steps which include:

- The alteration of the parent rock is initiated by climatic conditions.
- The formation of the organic base from the death and renewal of plants leaving organic compounds while mineral fragmentation continues.
- The formation of humus layers layer by the decomposing agents, where organic and mineral components are closely intertwined.
- The repetition of the cycle describes the activities of the previous steps and continues simultaneously contributing to the powering of all the processes that will ultimately make it possible to obtain a fertile and deep soil.

1.1.4. Properties of soils

All soils contain mineral particles, organic matter, water, and air. The combination of these determines the soil's properties. Physical, mechanical and seepage properties of soil is discussed in the next subsection.

1.1.4.1. Physical properties

Physical properties play an important role in determining soil's suitability for geotechnical engineering and environmental uses. The next subsection will discuss on soil texture, soil structure, soil colour, void ratio and soil porosity, bulk density, water content and finally degree of saturation.

a. Soil texture

Soil is made up of different-sized particles. Soil texture corresponds to the size of the particle that makes up the soil and depends on the proportion of sand, silt, clay-sized particles, and organic matter in the soil. Sand particles are the largest and clay particles the smallest. Soils are made up of different combinations of sand, silt, and clay particles. Soils that are a mixture of sand, silt, and clay are called loams.

b. Soil Structure

Soil structure describes the way the sand, silt, and clay particles are clumped together. Organic matter and soil organisms like earthworms and bacteria influence soil structure. Good quality soils are friable and have fine aggregates so the soil breaks up easily if you squeeze it. Poor soil structure has coarse, very firm clods or no structure at all.

c. Soil colour

Soil colour is influenced primarily by soil mineralogy and organic matter. Soil colours range from black to red to white. Soils high in iron are deep orange-brown to yellowish-brown. Soils that are high in organic matter are dark brown or black. Colour can also tell us how a soil "behaves" – a soil that drains well is brightly coloured and one that is often wet and soggy will have a mottled pattern of greys, reds, and yellows.

d. Void ratio (e) and soil porosity (n)

The void ratio is the ratio of the volume of void space, (V_v) to the volume of solids, (V_s) given by equation (1.1).

$$e = \frac{V_v}{V_s} \tag{1.1}$$

The porosity is the ratio of the volume of voids, (V_v) to the total volume of the soil (V) given by equation (1.2).

$$n = \frac{V_v}{V} \tag{1.2}$$

The void ratio and the porosity are inter3-related by equations (1.3) and (1.4)

$$e = \frac{n}{1-n} \tag{1.3}$$

$$n = \frac{e}{1+e} \tag{1.4}$$

Soil porosity also refers to the pores within the soil influencing the movement of air and water, which is very useful for civil engineers to guide in soil construction works. This ability of soil to allow water to pass through it is called permeability.

e. Bulk density (ρ)

The bulk density of a soil is the ratio of the dry weight, (M) to the total volume (V) given by equation (1.5).

$$\rho = \frac{M}{V} \tag{1.5}$$

Bulk density is also defined as the dry weight of soil per unit volume of soil. This is the weight of the oven-dry soil with its natural structural arrangement. It is determined by dividing the weight of the oven-dried soil by the soil volume measured. Bulk density considers both the solids and the pore space; rather than on a particle basis. The variation in bulk density is due largely to the difference in total pore space. High and low bulk densities have a great influence on the engineering properties of soils such as during compaction.

f. Water content (ω)

The water content ω(%) of a soil is the ratio of the mass of water, (M_w) to the mass of solid grains or dry soil, (M_s). It is expressed by equation (1.6).

$$\omega(\%) = \frac{M_w}{M_s} * 100 \tag{1.6}$$

The determination of water content is most probably the most frequently performed test in soil mechanics. Indeed, the value of the water content is required in most tests measuring the physical, mechanical, or hydraulic properties of soils, either because it is necessary for the processing of the results, or because it allows to better situate the measured property in the natural context of the soil in phase. Several methods are used to measure water content in the laboratory. For an accurate measurement, the water content is determined by drying the soil in a drying oven, a hot plate, or even a microwave oven, for a quick measurement. The soil sample is weighed before and after drying with the difference in weights representing the mass of water.

g. Degree of saturation

The degree of saturation denotes the actual relationship between the weight of moisture existing in a space and the weight that would exist if the space were saturated, given by equation (1.7).

$$\text{Degree of saturation } (\%) = \frac{SH_{actual}}{SH_{saturated}} * 100 \qquad (1.7)$$

Saturation, (S) is the percentage of water that occupies the pore spaces present in soil. With the water volume (V_w) and voids volume (V_v), saturation is given by equation (1.8).

$$S(\%) = \frac{V_w}{V_v} * 100 \qquad (1.8)$$

Generally, the soil has three phases; soil solid, water and air. If the pore or void space in the soil is fully occupied with the water, then it is fully saturated and the degree of saturation is 100%. If the voids space in the soil is partially occupied by water, it is said to be partially saturated. A completely dry soil has a saturation of 0%.

1.1.4.2. Mechanical properties

The mechanical properties of soil are those that will characterise the response of soil to the application of different load actions. Among the properties which evaluate the response of a soil, it can be distinguished, the characteristics at failure, and the shear strength of soil, discussed in the next subsection.

a. The characteristics at failure

The characteristics at failure of a soil corresponds to the combination of the most unfavourable load actions to which the soil can withstand without failure. This principally corresponds to the cohesion and friction angle.

Cohesion is the measure of the resistance due to intermolecular forces. This corresponds to the shear strength at a zero-shear stress resistance. Cohesion between soil particles comes from three major sources cementation, electrostatic and electromagnetic attraction, and primary valence bonding and adhesion. The adhesion due to cohesion is observed in fine-grained soil (clay) and partially saturated sand having a zero value in dry or saturated sand and normally consolidated clay. Examples of cohesive soils include sandy clay, silty clay, clayey silt, and organic clays.

The angle of friction is a measure of the ability of a unit of soil to withstand a shear stress. This is also called the angle of shearing resistance. It is the angle measured between the normal force and resultant force, that is attained when failure just occurs in response to a shearing stress.

b. Shear strength of soil

The safety of any geotechnical structure is dependent on the strength of the soil, which endangers lives and causes economic damage if soil failure occurs. The term ‘‘strength of a soil’’ is defined as the ability of the soil to resist imposed forces and normally refers to the shearing strength or shear strength.

The shear strength of soil is the maximum internal shear resistance to applied shearing forces. It is the level of shear stresses a material can resist without fracture. Shear stresses are forces applied tangentially along the face of the soil. The shear strength is a function of the normal stress and is measured in the laboratory from the direct shear and triaxial tests.

The basic principles in the description of strength properties are the failure criterion and the effective stress principle. The Mohr-Coulomb criterion is widely used to define failure in geotechnical applications. The Mohr-Coulomb criterion assumes that failure is controlled by the maximum shear stress and that this failure shear stress depends on the normal stress. This can be represented by plotting Mohr's circle for states of stress at failure in terms of the maximum and minimum principal stresses. The Mohr-Coulomb failure line is the best straight line that touches these Mohr's circles (Figure 1.2). The Mohr-Coulomb shear strength equation of a saturated soil presented by Terzaghi (1936) is given by equation (1.9).

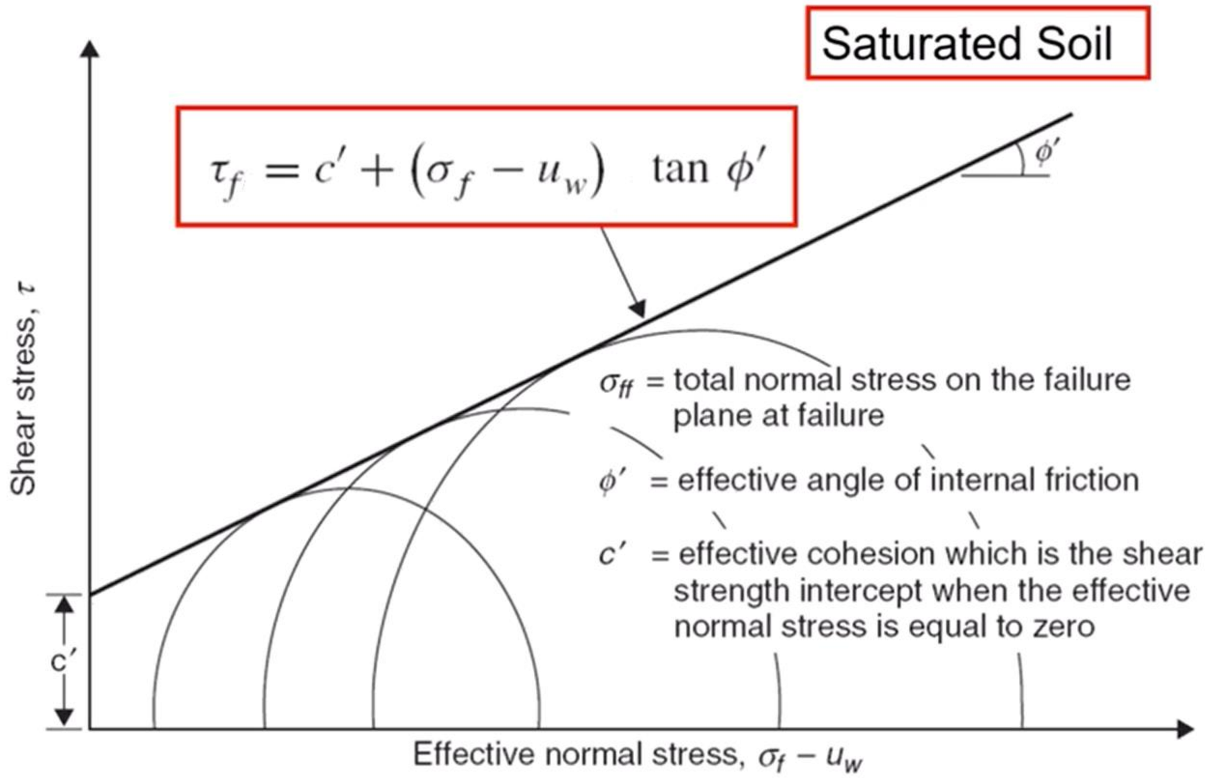


Figure 1.2. Mohr-Coulomb failure envelope for saturated soil. (Fredlund et al., 1978)

$$\tau_f = c' + (\sigma_f - u_w) \tan \phi' \qquad (1.9)$$

Where, τ_f is the shear stress on failure at failure, c' is the effective cohesion, $(\sigma_f - u_w)$ is the effective stress on failure plane at failure, u_w is the pore water pressure on the failure plane at failure, and $\phi'$is the effective angle of friction.

The Mohr-Coulomb criterion, unlike the Drucker-Prager criterion, assumes that failure is independent of the value of the intermediate principal stress. The failure of typical geotechnical materials generally includes some small dependence on the intermediate principal stress, but the Mohr-Coulomb model is generally considered to be sufficiently accurate for most applications. This failure model has vertices in the deviatoric stress plane (Figure 1.3).

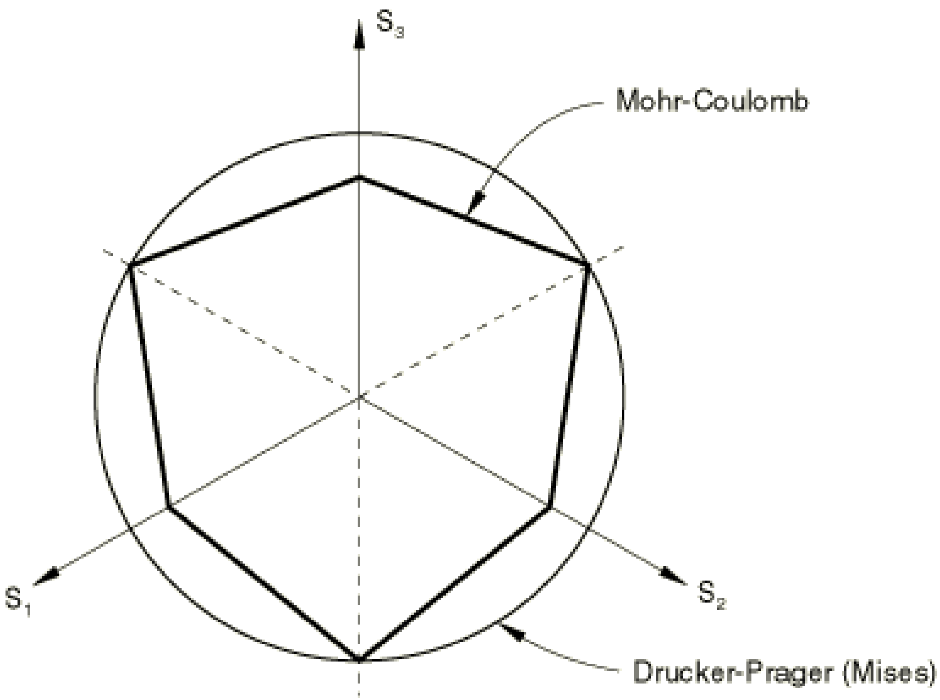


Figure 1.3. Mohr-Coulomb and Drucker-Prager models in the deviatoric plane (Pan et al., 2011)

1.1.4.3. Seepage properties

With the existence of interconnected voids, soils are permeable and allow water flow from points of high energy to points of low energy. Permeability is the measure of soil's ability to permit water to flow through its pores or voids. In geotechnical engineering, knowledge of permeability properties of soil is necessary for estimating groundwater seepage amount and analysing the stability of slopes or earth retaining structures subject to seepage forces. Bernoulli's equation for an incompressible steady flow of fluid has total energy as the sum of three contributions (potential, pressure, and kinetic energy) as shown in equation (1.10). Dividing equation (1.10) by the unit weight γ_w, gives equation (1.11). Seepage velocity in soils being small makes the velocity negligible giving the expression in equation (1.12) to be the piezometric head. Where $\frac{u}{\gamma_w}$ is the pressure head, $\frac{v^2}{2g}$ is the velocity head, H is the total hydraulic head, and z is the position above a selected datum.

$$E = \rho_w g z + u + \frac{\rho_w v^2}{2} \quad (1.10)$$

$$H = z + \frac{u}{\gamma_w} + \frac{v^2}{2g} \quad (1.11)$$

$$H = z + \frac{u}{\gamma_w} \quad (1.12)$$

The flow of water through soils is expressed by Darcy's empirical law which states that the velocity of flow (v) is directly proportional to the hydraulic gradient (i). This law is expressed in equation (1.13). Where k is the coefficient of proportionality known as the coefficient of hydraulic conductivity or permeability. Equation (1.13) can be expanded to obtain the rate of flow through an area of soil (A). The equation for the rate of flow (Q) is given by equation (1.14).

$$v = ki \quad (1.13)$$

$$Q = kiA \quad (1.14)$$

The coefficient of permeability is generally assumed to be a constant when analysing flow through saturated soil. However, the coefficient of permeability for an unsaturated soil can vary widely depending on the stress state (or degree of saturation) of the soil. Annex 8. indicates the hydraulic conductivity of different soils from the Unified Soil Classification System.

1.1.5. Typology of soils

Gravel, sand silts, and clays are used to identify specific textures in soils which will be referred to as soil types. Sand and gravel are grouped as coarse-grained soils meanwhile clays and silts are fine-grained soils. The coarseness of soils is determined by knowing the distribution of particle sizes, which is a primary means of classifying coarse-grained soils. Some soil description and soil types in usage are listed as follows:

- Alluvial soils which is fine sediments that have been eroded from rock and transported by water that has settled on riverbeds.
- Colluvium soil which is found at the base of mountains that have been eroded by the combination of water and gravity.
- Gypsum clays which is calcium sulphate formed under heat and pressure from sediments in ocean brine.
- Lacustrine soils which is this is mostly silts and clays deposited in glacial lakes waters.

- Lateritic soils which is this is residual soils that are cemented with iron oxides and are found in tropical regions.
- Loam which is a mixture of sand, silt and clay that may contain organic material.
- Loess which is a wind-blown, uniform fine-grained soil.
- Mud which is clay and silt mixed with water into a viscous fluid.

1.1.6. Classification of soils

A soil classification system represents a language of communication among engineers. It provides a systematic method of categorizing soils according to their probable engineering behaviour. The most common of these systems are AASHTO, USCS, and USDA. Each system has a distinct method and nomenclature to identify and classify soils.

1.1.6.1. AASHTO Classification System

The AASHTO system was developed specifically for highway construction and is still widely used for that purpose. With practice and experience, a reasonably accurate field classification can be determined. However, it is necessary to run sieve analyses and plasticity determinations to precisely classify a soil with this method. Table 1.1 presents the basic AASHTO soil classification system.

Table 1.1. AASHTO soil classification system (Wisconsin Department of Transportation, 2017)

General Classification	Granular Materials 35 percent or less of total sample passing No. 200 (75 µm)								Silt-Clay Materials More than 35 percent of total sample passing No. 200 (75 µm)						
Group Classification	A-1		A-3 [1]		A-2				A-4		A-5	A-6		A-7	
	A-1-a	A-1-b	A-3	A-3a	A-2-4	A-2-5	A-2-6	A-2-7	A-4a	A-4b		A-6a	A-6b	A-7-5	A-7-6
Sieve analysis, percent passing:						*				**	*			*	
No. 10 (2 mm)	50 max														
No. 40 (425 µm)	30 max	50 max	51 min	[2]					[3]	[4]					
No. 200 (75 µm)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	35 max	36 min	50 min	36 min	36 min		36 min	
Characteristics of fraction passing No. 40															
Liquid limit	—	—	Non-	—	40 max	41 min	40 max	41 min	40 max		41 min	40 max		41 min	
Plasticity index	6 max	6 max	Plastic	6 max	10 max	10 max	11 min	11 min	10 max		10 max	11 – 15	16 min	≤LL-30	>LL-30
Group Index	0						4 max		8 max		12 max	10 max	16 max	20 max	
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Sand	Silty or clayey gravel and sand				Silty soils			Clayey soils			
General rating as subgrade	Excellent to good								Good to fair						

Notes

With the test data available, the classification of a soil is found by proceeding from left to right on the chart. The first classification that the test data fits is the correct classification.

* A-2-5 is not allowed under 703.16.B. A-5 and A-7-5 is not allowed under 703.16.A. See "Natural Soil and Natural Granular Soils" (203.02.H) in this manual

** A-4b is not allowed in the top 3 feet (1.0 m) of the embankment under 203.03.A.

[1] The placing of A-3 before A-2 is necessary in the "left to right" process, and does not indicate superiority of A-3 over A-2.

[2] A-3a must contain a minimum 50 percent combined coarse and find sand sizes (passing No. 10 but retained on No. 200, between 2 mm and 75 µm).

[3] A-4a must contain less than 50 percent silt size material (between 75 µm and 5 µm).

[4] A-4b must contain 50 percent or more silt size material (between 75 µm and 5 µm).

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

1.1.6.2. Unified Soil Classification System

The unified soil classification system is based on the engineering properties of soil and it is most appropriate for earthwork construction. The USCS has been through several transitions since it was developed. Upon recognizing a USCS symbol of a classification group, one can immediately deduce the approximate permeability, shear strength, and volume change potential of soil and how it may be affected by water, frost, and other physical conditions. It can also be used in estimating excavation and compaction characteristics, potential dewatering situations, and workability.Table 1.2 represents the basic Unified soil classification system.

Table 1.2. Unified soil classification system (Wisconsin Department of Transportation, 2017)

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

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

1.1.6.3. USDA Classification System

The USDA system was developed for agricultural purposes. It has some engineering applications in that it provides a relatively easy method for the general field classification of soils. However, "loamy", while descriptive, is not an engineering term and should be avoided when discussing the engineering properties of a soil. Figure 1.4 presents the basic USDA soil classification system.

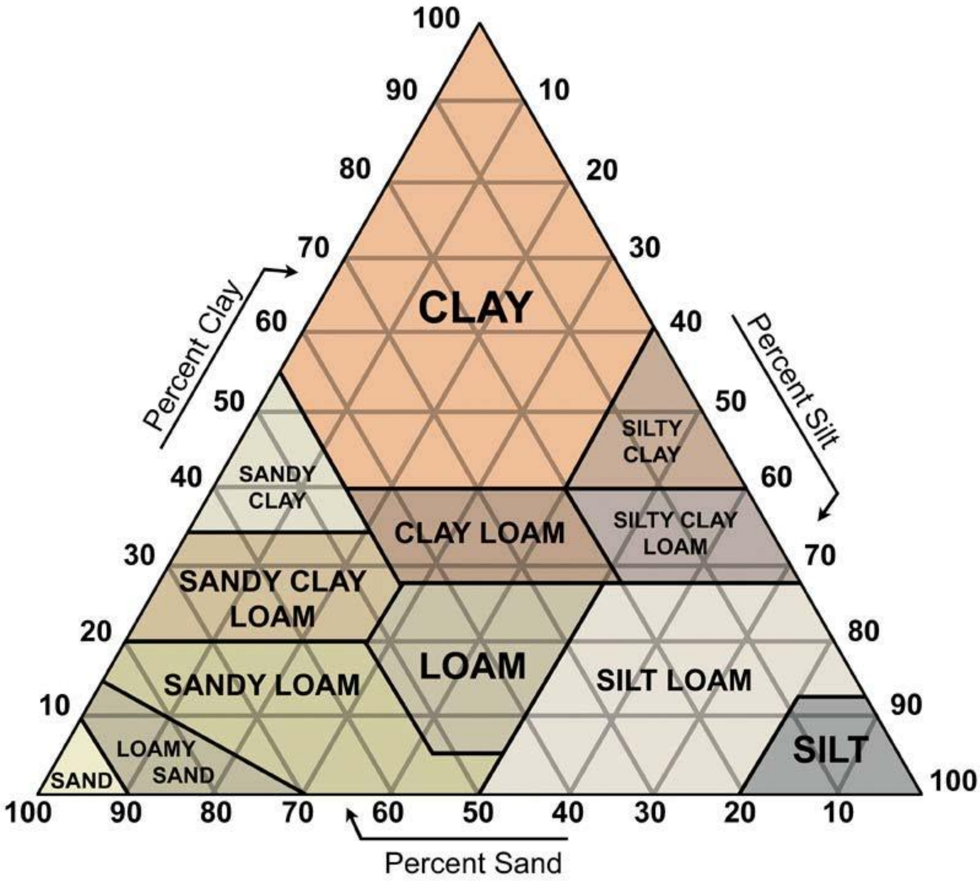


Figure 1.4. USDA Soil Classification System (Wisconsin Department of Transportation, 2017)

1.2.Shear strength governing slope stability

There are basically three representations of shear strength associated with strength failure envelopes in soil that have traditionally been used in slope stability analyses which include the peak shear strength, the residual shear strength and the fully softened shear strength (Duncan et al. 2011). Unsaturated shear strength that accounts for the contribution from soil suction will be discussed with progressive shear failure.

1.2.1. Peak shear strength

The peak shear strength is also called “intact strength” and corresponds to the maximum strength a soil can reach for a given initial moisture content and dry density. It essentially represents an upper bound shear strength relative to existing soil conditions in the field. The peak shear strength is time-dependent and occurs before the fully softened condition is reached. Mesri & Shahien (2003) showed that there can be a wide variation in the shear strength under the same effective normal stress, as the soil may have experienced different degrees of softening during its history. Existing shear strength can be obtained by conducting triaxial tests on undisturbed samples. For compacted soils, the peak shear strength can be obtained by running triaxial or direct shear tests that recreate the moisture content and compaction (i.e., dry density) used on-site.

1.2.2. Fully softened shear strength

The soil softens as a result of external factors, such as weather. The infiltration and loss of water due to wetting/drying is responsible for the swelling/shrinking of the soil and these variations of volume soften the soil. According to Skempton (1970) the phenomenon of softening leads to a decrease in the initial as-compacted shear strength (i.e., the peak shear strength) to the fully softened shear strength. The fully softened shear strength is the reduced strength where the over-consolidated clay has passed the peak strength and starts to shear at constant volume. Skempton (1970) said that the fully softened shear strength corresponds to a new failure envelope, which seems to be similar to the failure envelope for normally consolidated clays. Mesri & Shahien (2003) suggested that the fully softened shear strength should be used for most of first time slope failures; however, in some cases part of the slip surface of these slopes at failure may already be at residual conditions resulting from rather small displacements.

1.2.3. Residual shear strength

Residual strength is the constant strength of clay at a large shear displacement. In the drained triaxial test, most clays would eventually show a decrease in shear strength with increasing strain after the peak strength has been reached. However, in the triaxial test, there is a limit to the strain which can be applied to the specimen. The most satisfactory method of investigating the shear strength of clays at large strains is by means of the ring shear apparatus, an annular direct shear apparatus according to Bishop et al. (1971). The annular specimen as shown in Figure 1.5 (a) is sheared, under a given normal stress, on a horizontal plane by the rotation of one half of the apparatus relative to the other; there is no restriction to the magnitude of shear displacement between the two halves of the specimen. The rate of rotation must be slow enough to ensure that the specimen remains in a drained condition. Shear stress, which is calculated from the applied torque, is plotted against shear displacement as shown in Figure 1.5 (b). The shear strength falls below the peak value and the clay in a narrow zone adjacent to the failure plane will soften and reach the critical state. However, because of the non-uniform strain in the specimen, the exact point on the curve corresponding to the critical state is uncertain. With continuing shear displacement the shear strength continues to decrease, below the critical-state value, and eventually reaches a residual value at a relatively large displacement. The residual angle can also be obtained using the direct shear apparatus.

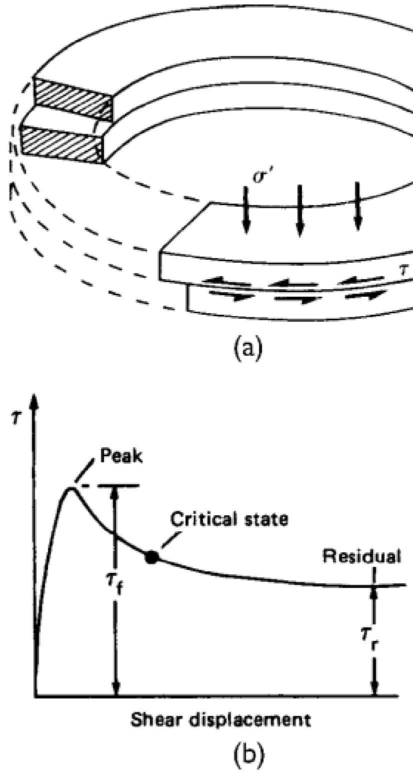


Figure 1.5. Sketch of the ring shear test functioning and identification of peak, critical and residual strengths on $\tau - \delta$ plane (Bishop et al., 1971)

The results from a series of tests, under a range of values of normal stress, enable the failure envelope for both peak and residual strength to be obtained, the residual strength parameters in terms of effective stress being denoted C_r' and ϕ_r'. Residual strength data for a large range of soils have been published by Lupini et al. (1981), which indicate that the value of C_r' can be taken to be zero. Thus, the residual strength can be expressed by equation (1.15).

$$\tau_r = \sigma_f' \tan \phi_r' \tag{1.15}$$

1.2.4. Unsaturated shear strength

Unsaturated soil shear strength is an important parameter in soil mechanics. The three common engineering application areas are; bearing capacity, lateral earth pressures and slope stability. Embankment and cut slopes are often unsaturated before wetting events leading up to failure. While it may be reasonable to evaluate long-term stability by assuming saturated conditions, to accurately determine the existing stability of a slope or the evolution of stability over time, it is necessary to consider the shear strength of the soil under unsaturated conditions. Additionally, it is possible slopes may fail when a significant portion of the failure surface is in an unsaturated condition.

The shear strength theory of unsaturated soil can be viewed as an extension of the Mohr-Coulomb Theory (1914). The shear strength of unsaturated soil is controlled by two independent stress state variables and the soil property is associated with each of the state variables as shown in Figure 1.6. The selected stress state variables are first, $(\sigma_f - u_a)$ which represents the net normal stress on failure, and second, $(u_a - u_w)$ which is the matric suction on the failure plane. A simple bilinear model presented by Fredlund & Rahardjio (1993) for the shear strength of unsaturated soil is expressed by equation (1.16).

$$\tau_f = c' + (\sigma_f - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \qquad (1.16)$$

Where τ_f is the shear stress on failure plane, c' is the effective cohesion, ϕ' is the effective angle of friction, u_a is the pore-air pressure at failure, u_w is the pore-water pressure at failure, and ϕ^b is an angle (soil property) defining the increase in strength due to the pore water pressure (matric suction). The expression $(u_a - u_w)\tan\phi^b$, represents the suction strength having a linear trend. Soil suction or negative water pressures have the effect of adding strength to soil. In the same way that positive pore-water pressures decrease the effective stress and thereby decrease the strength, negative pore-water pressures increase the effective stress and in turn increase the strength.

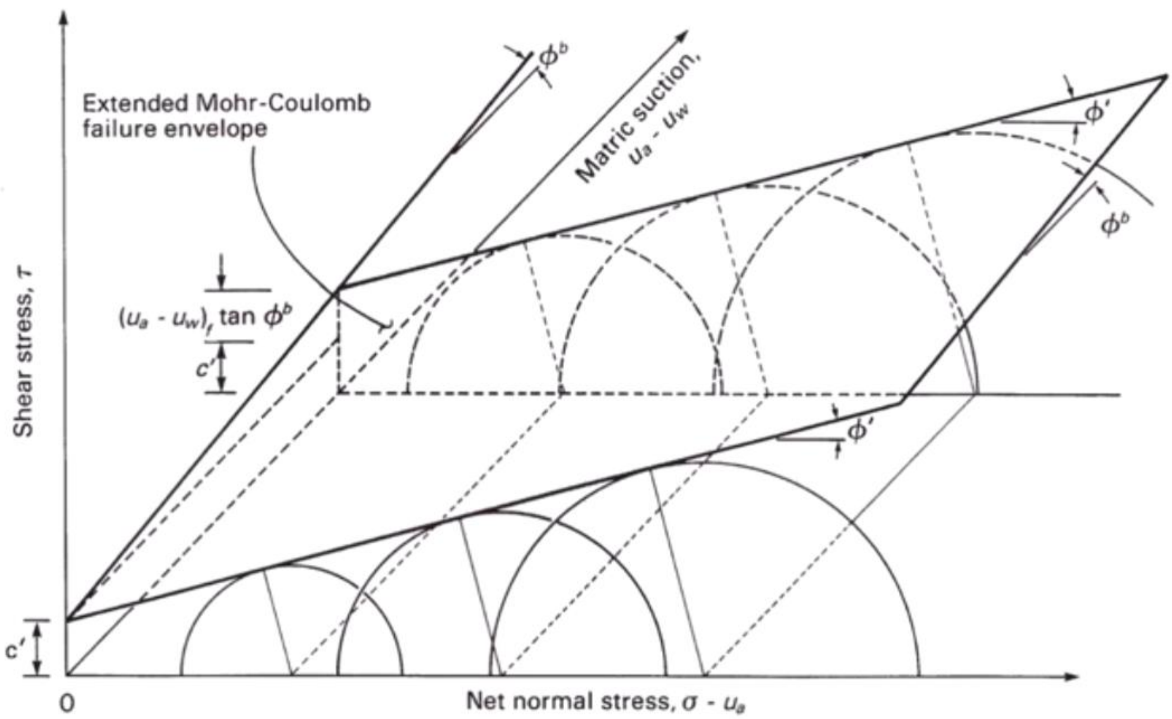


Figure 1.6. Extended Mohr-Colomb failure envelope for unsaturated soils (Fredlund et al., 1978)

The angle ϕ^b is a material property which can be suitably calibrated, whereby for practical purposes ϕ^b can be taken to be about $\frac{1}{2}\phi'$. In the capillary zone where the soil is saturated, but the pore-water pressure is under tension, ϕ^b is equal to the friction angle ϕ'. As the soil desaturates, ϕ^b decreases. The decrease in ϕ^b is a reflection of the fact that the negative pore-water pressure acts over a smaller area. More specifically, ϕ^b is related to the soil-water characteristic curve which is also known as the volumetric water content function

Considerable research has been done to better quantify the unsaturated shear strength of soil using the soil-water characteristic curve and the effective shear strength parameters (c' and ϕ'). As a better alternative to the use of ϕ^b to model the increase of shear strength due to soil suction, the following estimation equation (1.17), proposed by Vanapalli et. al. (1996) can be used.

$$\tau_f = c' + (\sigma_f - u_a)\tan\phi' + (u_a - u_w)\left[\left(\frac{\theta_w-\theta_r}{\theta_s-\theta_r}\right)\tan\phi'\right] \tag{1.17}$$

Where θ_w is the volumetric water content, θ_s is the saturated volumetric water content, θ_r is the residual volumetric water, and $\theta_n = \left(\frac{\theta_w-\theta_r}{\theta_s-\theta_r}\right)$ defined as the normalised water content. The expression $(u_a - u_w)\left[\left(\frac{\theta_w-\theta_r}{\theta_s-\theta_r}\right)\tan\phi'\right]$, represents the suction strength having a non-linear trend. It is observed that that the main difference between the equations proposed by Fredlund and Vanapelli is that the shear strength contribution due to suction can be more accurately estimated using the soil-water characteristic curve that has been determined taking into account the influence of the stress state and the initial water content conditions according to Vanapalli et al. (1999). According to Vanapalli & Fredlund (2000), a comparison graph between the shear strength as a function of soil suction for a Madrid clay sand as shown in Figure 1.7.

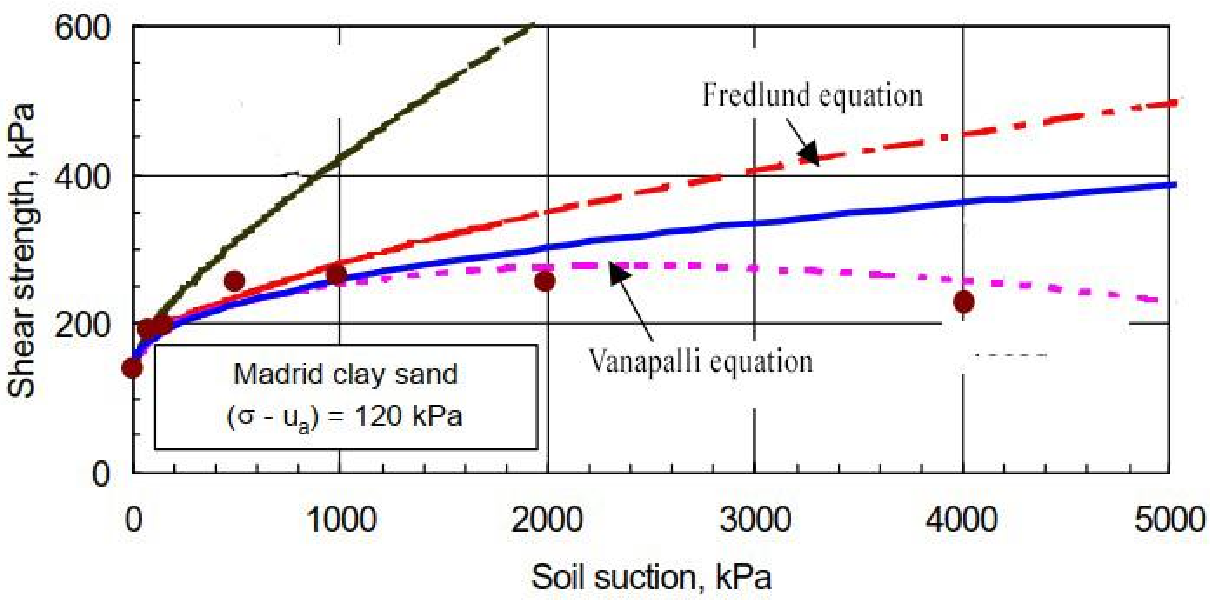


Figure 1.7. Comparison of shear strength as a function of soil suction for a Madrid clay sand (Vanapalli & Fredlund, 2000),

The passage or seepage of water through the interconnecting voids of soil represents the soil permeability. A decrease in effective stress can occur when water enters surface cracks and infiltrates unsaturated soil due to rainfall or any other factor, causing an increase in pore water pressure. This is attributed to the decrease of matric suction and results in a loss of shear strength. When this phenomenon happens in a slope, the driving forces may become greater than the resisting forces along some critical failure surface and a failure occurs. This phenomenon is influenced by the nature and structure of the soil. Studies conducted by Rahardjo et al. (2007) have emphasized the important role of permeability in high plasticity soils. In these soils where the permeability is very low, the infiltration of water is much slower so they are less sensitive to short rainfall events. However, the rate of recovery for the factor of safety is also slower for slopes composed of low permeability soils. As a result, long and/or repeated rainfall events may be necessary to fail such slopes. The authors further concluded that slopes with high permeability usually fail as a consequence of a rising water table while the low permeability soil would likely fail due to a reduction in matric suction of the soil located above the water table.

1.2.5. Progressive shear failure

Progressive shear failure refers to the progressive transfer of shear stress along a developing shear surface with increasing shear strain as shown in Figure 1.8. The direction of the major and minor principal stresses changes along the length of the slip surface. At different parts of the slip surface, the soil may be below the peak strength, at the peak strength, or pass the peak strength, depending on the amount of shear deformation that has occurred at each location along the slip surface.

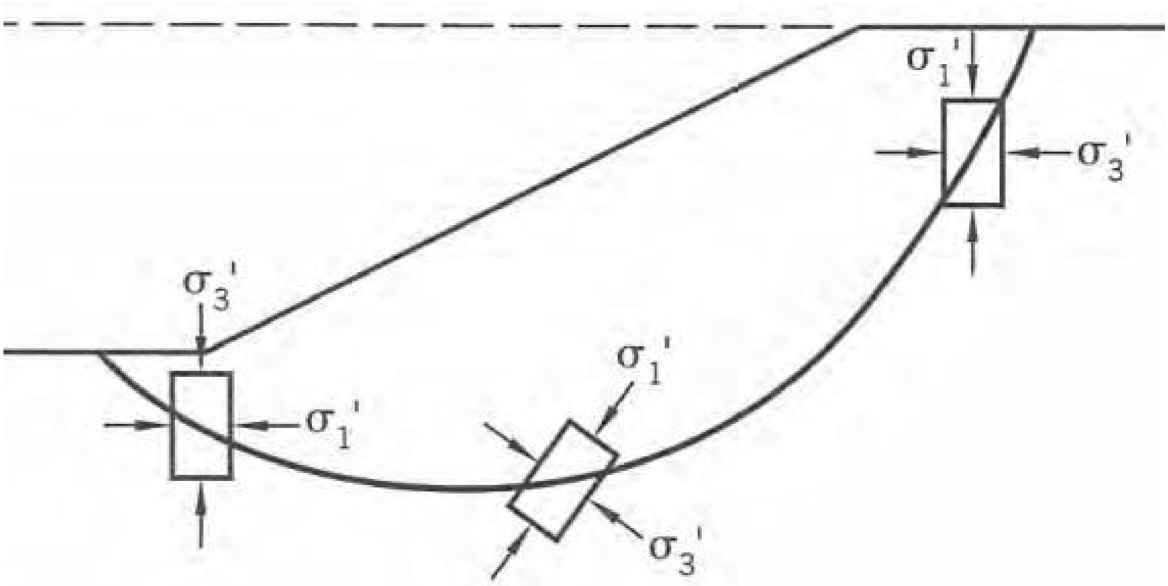


Figure 1.8. Rotation of the principal stresses along a potential circular arc slip surface in a cut slope (Cornforth, 2005)

When a cut is first made into a stiff over-consolidated clay, shear movements occur to mobilize sufficient strength to provide stability. High negative pore pressures temporarily provide high strengths, limiting the amount of shear movement needed to mobilize the required resistance for stability. As pore pressures in the slope slowly rise and strains continue, some areas of the potential slip surface cross their peak resistance and become weaker with increasing strain. This loss of resistance is transferred to adjacent areas that are still able to mobilise increased resistance with increasing strain (have not yet reached the peak resistance). Shear failure progresses along the potential slip surface and may result in failure of the entire block if the total resistance along the slip surface becomes less than the total shear stresses induced by gravity.

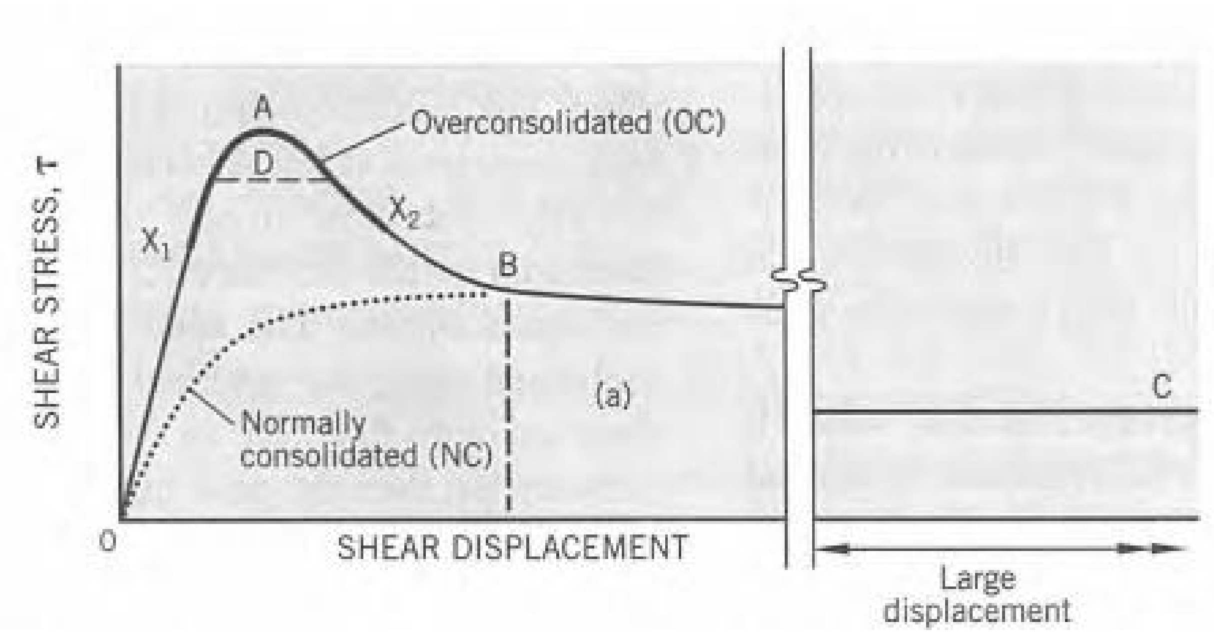


Figure 1.9. Shear stress-shear displacement curves (Conforth, 2005)

Variable stain rates within a slope ensure that the peak resistance of the clay cannot be mobilized simultaneously at all parts of the slip surface. Therefore, some reduction in the peak strength Mohr envelope is needed in slope stability and back analysis of landslides. The reduction should be greatest where the clay has a very sharp peak at low shear strain. The concept of variable strength along the slip surface is shown by the bold line X_1 to X_2 around the peak strength in Figure 1.9. The total shearing resistance that can be mobilised along the slip surface is a combination of shear strengths that have an average value of D. The value of D is always less than the peak strength A and is always more than the fully softened strength B.

1.3.Landslides

This section will analyse landslides from its definition, classification, movement types and triggering factors.

1.3.1. Definition

Cruden (1991) has suggested a simple definition o landslide: "the movement of a mass of rock, debris, or earth downslope''.

Varnes (1958) has suggested a definition of a landslide as a downward and outward movement of slope forming materials composed of natural rock, soils, artificial fills, or combinations of these materials. The dimension and geometry of a landslide have been described by Varnes, (1978) using the cutaway drawing shown in Figure 1.10.

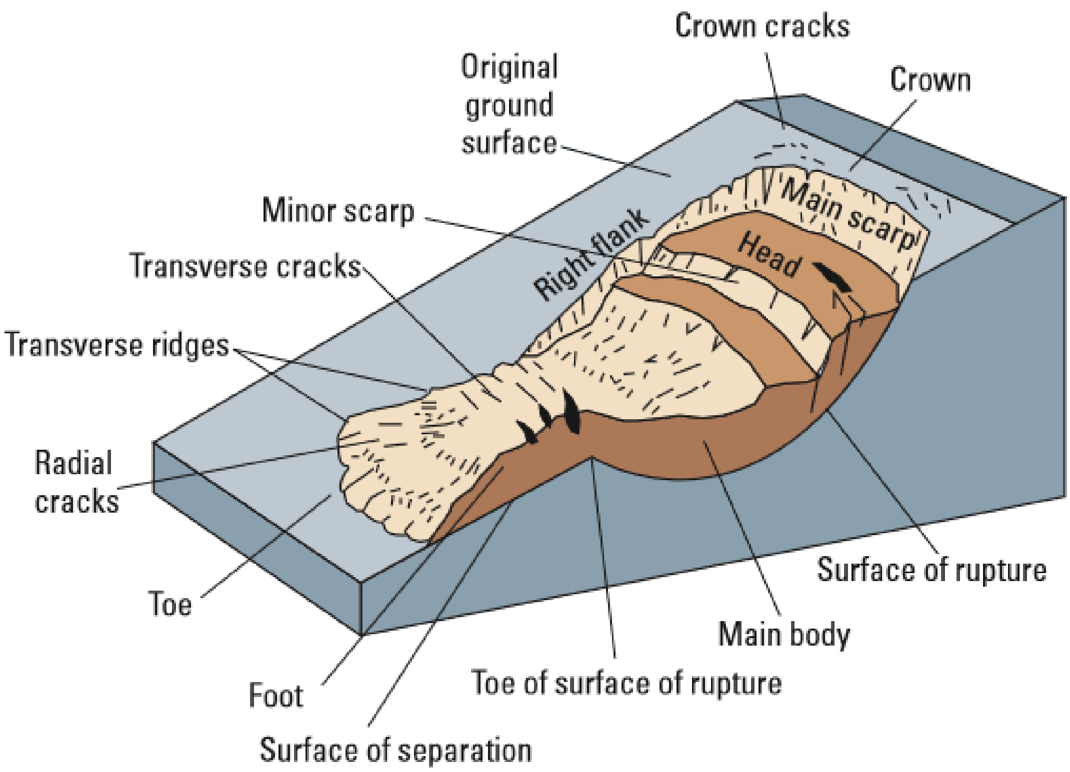


Figure 1.10. Terminology describing landslide features (Varnes, 1978)

1.3.2. Landslide classification

The most widely used landslide classification scheme, proposed by Varnes (1958, 1978) and updated by Hutchinson (1988), followed by Cruden & Varnes (1996), divides slope movements according to three source material types (rock, debris, earth), and six movement types (fall, topple, slide, spread, flow, complex). This classification was taken over by Hungr et al. (2001) and other publications. However, this definition of landslide-forming materials is compatible neither with the geological terminology of materials distinguished by origin nor with geotechnical classifications based on mechanical properties.

Hungr et al. (2014), aimed to introduce modifications to the Varnes classification to reflect recent advances in the understanding of landslide phenomena and the materials and mechanisms involved, added “slope deformation” as a sixth movement type. Moreover, they distinguish rock (which contains pure water in a form of ice or snow) and soil as materials to provide compatibility with accepted geotechnical and geological terminology. From that, Table 1.3 could be set up.

Table 1.3. Updating of the Varnes classification (Hungr et al. 2014)

Type of movement	Type of material	
	Rock	Soil
Fall	Rock/ice fall	Boulder/debris/silt fall
Topple	Rock block topple	Gravel/sand/silt topple
	Rock flexural topple	
Slide	Rock rotational slide	Clay/silt rotational slide
	Rock planar slide	Clay/silt planar slide
	Rock wedge slide	Gravel/sand/debris slide
	Rock compound slide	Clay/silt compound slide
	Rock irregular slide	
Spread	Rock slope spread	Sand/silt liquefaction spread
		Sensitive clay spread
Flow	Rock/ice avalanche	Sand/silt/debris dry flow
		Sand/silt/debris flow slide
		Sensitive clay flow slide
		Debris flow
		Mudflow
		Debris flood
		Debris avalanche
		Earthflow
		Peat flow
Slope deformation	Mountain slope deformation	Soil slope deformation
	Rock slope deformation	Soil creep
		solifluction

It can be observed that from Table 1.3, the soil is composed of clay, mud, silt, sand, gravel, boulders, debris, and peat. The term “debris” in their publication indicates a mixture of sand, gravel, cobbles, and boulders, often with varying proportions of silt and clay. The word was “mud” was presented as a material with a sufficient silt and clay content to produce plasticity (cohesiveness) with high moisture content. Typically, a material with Plasticity Index greater than 5 % and a Liquidity Index during motion greater than 0.5 (i.e., they are in or close to a liquid state). Although the term "earth" has been used several times, here, it has a new definition related to the concept of "earthflow" which means a cohesive, plastic, clayey soil, often mixed and remoulded, with a Liquidity Index below 0.5.

Another important aspect that characterizes a landslide is the velocity as shown in Table 1.4.

Table 1.4. Landslide's velocity scale (Cruden & Varnes, 1996)

Velocity Scale	Description	Velocity(mm/s)	Typical Velocity
7	Extremely rapid	5.10^{3}	5 m/s
6	Very rapid	5.10^{1}	3 m/min
5	Rapid	5.10^{-1}	1.8 m/h
4	Moderate	5.10^{-3}	13 m/month
3	Slow	5.10^{-5}	1.6 m/year
2	Very slow	5.10^{-7}	161 mm/year
1	Extremely slow		

1.3.3. Movement types

Landslides can be categorised according to the movement type as falls, topples, slides (rotational and translational), spreads, flows, and slope deformation.

1.3.3.1. Falls

A fall is the detachment of soil or rock from a steep or vertical slope on which little or no shear displacement has occurred (Figure 1.11). The material subsequently descends mainly by falling bouncing or rolling. The rolling velocity depends on the impact energy and the constitution of the slope. Generally, the velocity of such movement is in the range of very rapid to extremely rapid. This abrupt movement can take place in clusters. In any case, the dynamic interaction between the most mobile moving fragments is still present. Even if fragments can break during impact, it can be said that fragment deformation is unimportant. Usually, this kind of event includes a limited quantity of volume.

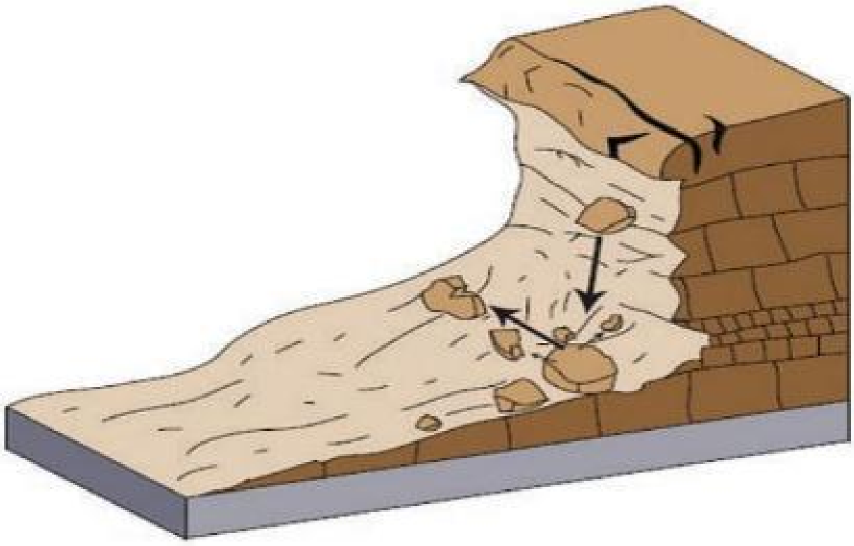


Figure 1.11: Schematic of fall (Highland & Bobrowsky, 2008)

1.3.3.2. Topples

A topple is recognized as the forward rotation out of a slope of a mass of soil or rock around a point or axis below the center of gravity of the displaced mass by Cruden & Varnes (1996). Topples (Figure 1.12) can consist of rock, debris (coarse material), or earth materials (fine-grained material). The movement velocity can vary from extremely slow to extremely rapid, sometimes accelerating throughout the movement depending on the distance of travel (Highland & Bobrowsky, 2008). This type of movement is known to occur globally, often prevalent in columnar-jointed volcanic terrain, as well as along stream and river courses where the banks are steep.

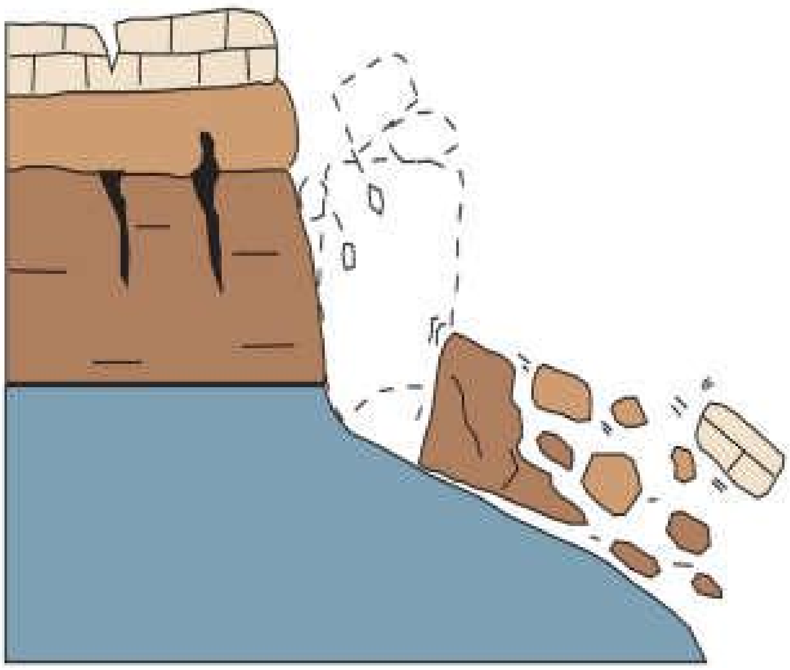


Figure 1.12.: Schematic of Topple (Cruden & Varnes, 1996)

1.3.3.3. Slides

Slides are often recognized as the downslope displacement of a soil or rock mass which occurs prevalently on surfaces of rupture or on the relatively narrow band (shear band) where the shear strain is intense. The movement does not initially occur simultaneously over the whole of what eventually becomes the slip surface; the volume of displacing material enlarges from an area of local failure. A slide can be rotational or translational.

Rotational slides occur most frequently inhomogeneous materials. They are the most common landslide occurring in “fill” materials which are materials used in artificial embankments. The movement rate range is from extremely slow to moderately rapid a rotational slide has parallel curved planes of movement as shown in Figure 1.13.

The mass in a translational landslide moves out, or down and outward, along a relatively planar surface with little rotational movement or backward tilting. This type of slide may progress over considerable distances if the surface of rupture is sufficiently inclined, in contrast to rotational slides, which tend to restore the slope equilibrium. The material in the slide may range from loose, unconsolidated soils to extensive slabs of rock, or both. Translational slides commonly fail along geologic discontinuities such as faults, joints, bedding surfaces, or the contact between rock and soil. Translational slides (Figure 1.13) are the most common worldwide and can occur in any environment. Movement rate ranges from slow to extremely rapid. With increased velocity, the landslide mass of translational failures may disintegrate.

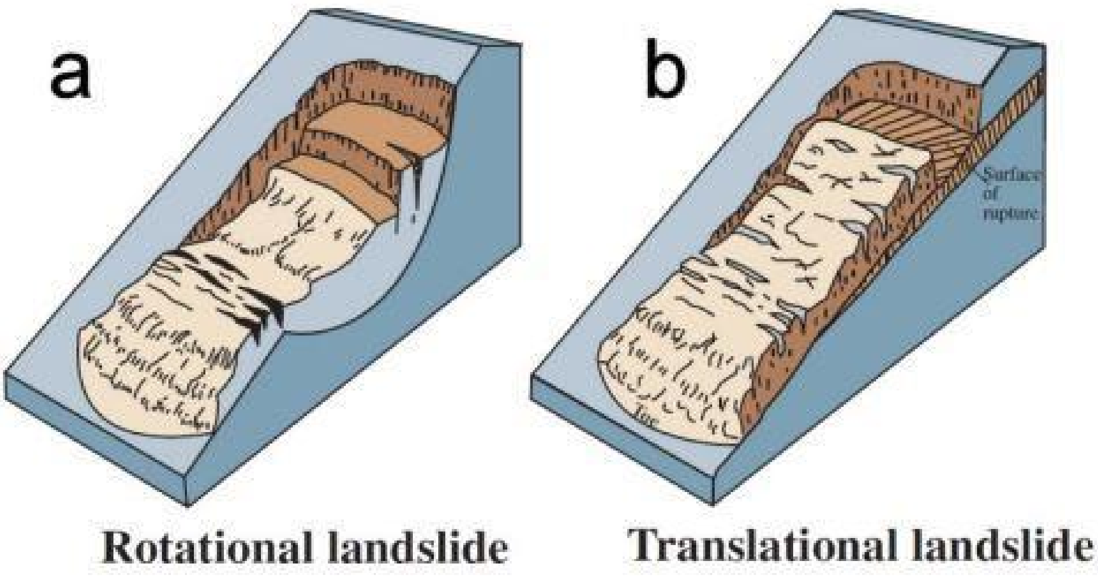


Figure 1.13. Schemes of slides (Highland & Bobrowsky, 2008)

1.3.3.4. Spread

Spread concerns a movement of a mass of hard soil or rock due to the extrusion and lateral displacement of a lower layer with less strength. In this case, there is not an evident sliding surface, but the lower layer behaves as a viscous material. This is shown in Figure 1.14. The causes are the liquefaction of saturated granular soil or the loss of cohesion in cohesive hard soil. Due to that, the upper part, more rigid, breaks itself into several blocks that seem to float above the lower consistent material. Lateral spreads usually occur on very gentle slopes or essentially flat terrain.

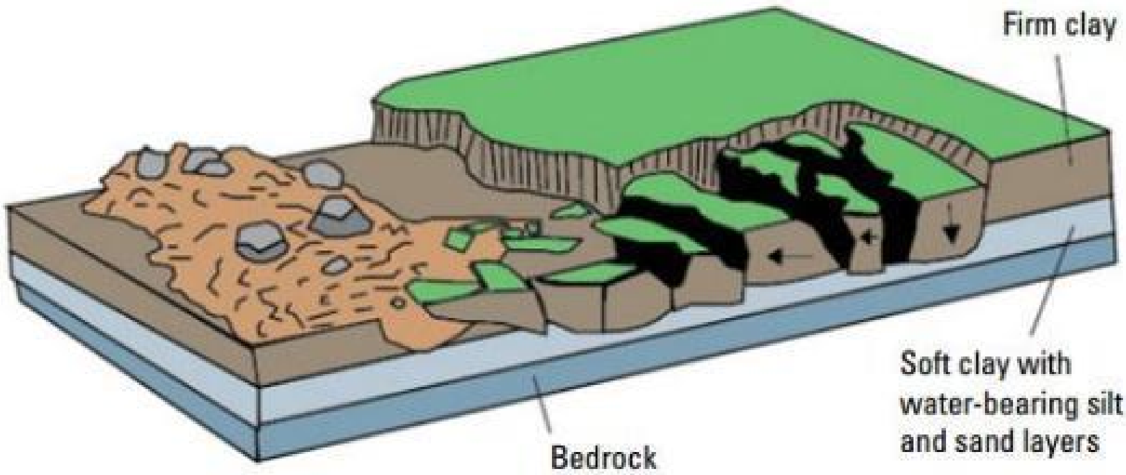


Figure 1.14. Scheme of lateral spread (Highland & Bobrowsky, 2008)

1.3.3.5. Flows

A flow is related to the detachment of a mass of soil or rock with a subsequent movement similar to that of a viscous fluid (Figure 1.15). There are several sliding surfaces and the mass does not exhibit uniform displacement, but the displacement rate varies inside the mass. Generally, the mass can cover along a path before stopping and it is possible to distinguish 3 zones: the depletion basin, the intermedium channel, and arresting tail.

The flow-like landslides can be subdivided into various groups, depending on the moved material, the velocity of the flow, and the saturation of the soil. The material is commonly inconsistent and the flow evolves depending on the topography of the site.

The rock/ice avalanches are extremely rapid, massive, the flow-like motion of fragment rock, starting from a rockfall or a large rockslide. When the debris is dry or at least not liquefied, we can identify a dry sand/silt/gravel/debris flow. Practically, the flow can be rapid or slow, and there is not excess pore pressure.

The sand/silt/debris flow-slides are very or extremely rapid flows of sorted or unsorted saturated granular material on moderate slopes, involving excess pore-pressure or liquefaction of material originating from the landslide source. The material may range from loose sand to loose debris (fill or mine waste), loess, and silt. Usually originates as multiple retrogressive failures. May occur subaerially, or underwater. If the material involved in an extremely rapid flow is mainly composed of liquefied sensitive clay, the sensitive clay flow-slides take place.

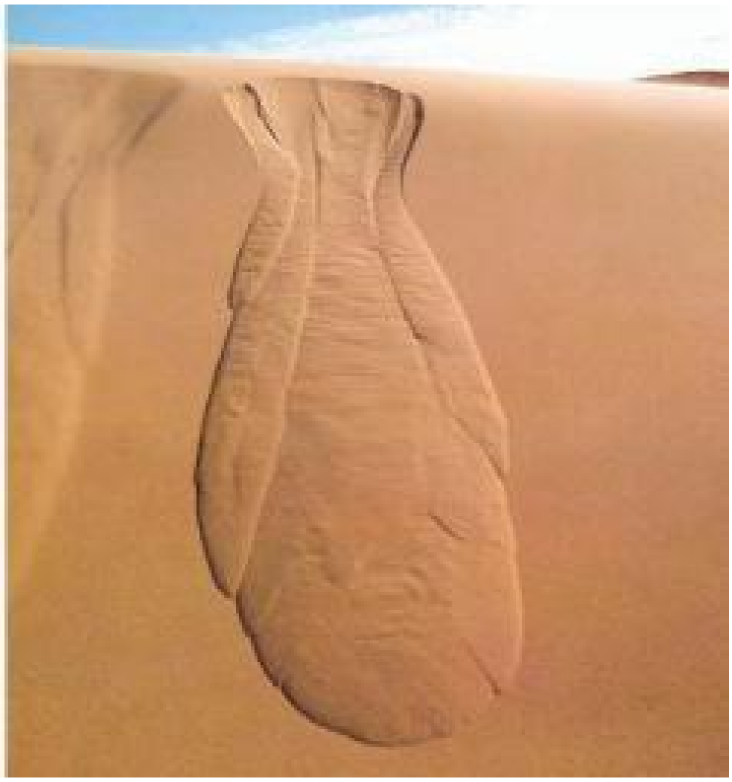


Figure 1.15. Dry sand flow on the lee slope of a dune, Naib desert (Hungr et al., 2014)

Generally, a debris flow is a very or extremely rapid surging flow of saturated debris in a steep channel. Commonly, it is characterized by the strong entrainment of material and water from the flow path.

The classification continues identifying the mudflow as a very or extremely rapid surging flow of saturated plastic soil in a steep channel. Even in this case, the entrainment of material and water from the flow path is still common.

When there is a rapid flow of water, heavily charged with debris takes place, we can identify a debris flood. The event is strongly dominated by the liquid matrix. Continuing speaking about debris, the debris avalanche is a very or extremely rapid shallow flow of partially or fully saturated debris on a steep slope. The main difference compared to the debris flow is the fact that it takes place without confinement in an established channel.

The earthflow is rapid or slower, intermittent flow-like movement of plastic, clayey soil, facilitated by a combination of sliding along multiple discrete shear surfaces, and internal shear strains. Finally, the peat flows are rapid flows of liquefied peat, caused by an undrained failure.

1.3.3.6. Slope deformation

Slow velocities of deformation characterize the last group of landslides. Five types of landslides are included in this last family whose mountain slope deformation, rock slope deformation, soil slope deformation, soil creep, solifluction.

The mountain slope deformation can be identified as large-scale gravitational deformation of steep, high mountain slopes, manifested by scarps, benches, cracks, trenches, and bulges, but lacking a fully defined rupture surface. Often, extremely slow or unmeasurable movement rates.

Deep-seated slow to extremely slow deformation of valley or hill slopes is called rock slope deformation. The movement is very slow. If the event includes cohesive soil, instead of rock, soil slope deformation can be stated.

The soil creep is an extremely slow movement of surficial soil layers on a slope, typically less than 1m deep, because of climate-driven cyclical volume changes (wetting and drying, frost heave). The soil creep is recognized by small soil ripples or ridges as illustrated in Figure 1.16. The solifluction is a very slow but intensive shallow soil creep involving the active layer in Alpine or polar permafrost.

Figure 1.16. Creep in an area near east Sussex, United Kingdom (Highland & Bobrowsky, 2008)

1.3.4. Landslide triggering factors

For a landslide occurrence, the physicomechanical parameters of the soil together with initial and boundary conditions play a great role in slope instabilities whereby there is a decrease in strength, increase in driving forces and increase in slope susceptibility which could cause the landslide occurrence. The landslide triggering factors include internal (related to the reduction in frictional force or shear strength of the soil and rock material properties), external and human factors. A landslide has generally more causes, but one of them is the most relevant, even if it may not be that which has induced the final failure. Possible causes include:

- Exceptional rains and consequent water infiltration.
- Reactivation of quiescent landslides.
- Strength reduction due to weathering.
- Seismic forces.
- Presence of artesian aquifer or localised springs.
- Pipes for irrigation and water adduction.
- Seepages in earth embankment (levees, dams, or other slopes).
- Embankment instability on soft soils.
- External erosion of river embankments.
- Excavation, loads on a slope, construction with too steep slopes.

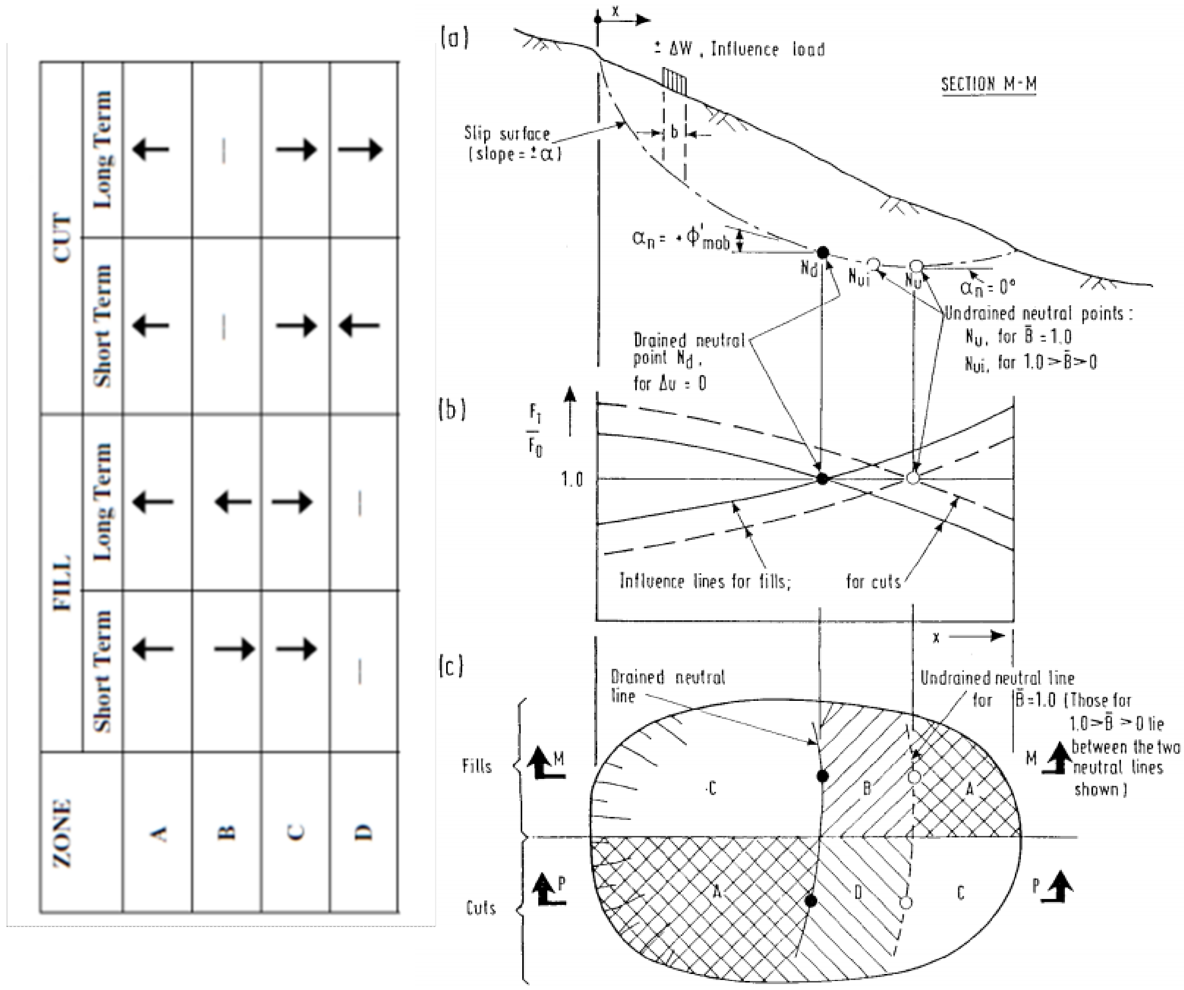
1.4.Landslide stabilisation techniques

Landslide stabilisation encompasses different strategies used in reducing the risk of landslides by increasing the factor of safety of an unstable slope. Any stabilization methods have the function to reduce either the driving forces or increase the resisting forces. However, the proposed technique must be directly related to the cause of instability and has to be a compromise between a suitable factor of safety; the cost (as cheap as possible) and the working conditions on the site. Landslide stabilisation techniques may include slope profile modification, erosion control, drainages and the use of retaining structures.

1.4.1. Modification of slope profile

The variation of stability and driving forces of a slope angle can be done by the addition of extra material or by excavation. The choice depends on the characteristic of the site (increasing the length of the slope or not). If the modification rises to a constant slope, it can be applied to limited landslides and will be useful only for the upper layer of soil. It won't give an effect in the case of deep landslides. The efficiency of a corrective cut or fill is controlled by its location, weight, shape, and characteristics of the actual or potential landslide to be treated. To assist the design, Hutchinson (1977, 1984) proposed the ''neutral line'' concept to evaluate the relative merits of performing cuts and/or fills at different locations in the slope as shown in Figure 1.17. Technics to modifying the slope profile include:

- Removal of soil from the head of a slide.
- Reducing the height of the slope.
- Backfilling with lightweight material.
- Benching.



ZONE	FILL Short Term	FILL Long Term	CUT Short Term	CUT Long Term
A	↑	↑	↑	↑
B	↓	↑	—	—
C	↓	↓	↓	↓
D	—	—	↑	↓

Figure 1.17. Influence lines and neutral points (a, b) and neutral lines (c) for cuts and fills on a typical landslide (a).After Hutchinson (1977, 1984)

1.4.2. Erosion control

Another technique for slope stability includes erosion control which consists of covering the external surface with biomaterial, synthetic material, or using vegetation. The use of surface vegetation protects the most external layer of ground from rain and wind which is referred to as ‘‘bioengineering’’ or ‘‘biotechnical slope protection’’. Depending on the soil type the stabilisation design is based on the different root types (size and strength) planted on the slope.

1.4.3. Drainages

This section will discuss treat drainages by giving a brief definition of the method entering into surface and subsurface drainage systems.

1.4.3.1. Definition

Drainage is often a crucial remedial measure due to the important role played by pore water pressure in reducing shear strength. Drainage includes all operations that aim to reduce the pore pressure with an increase in the effective stress and the shear strength of the soil. According to Turner & Schuster, (1996) because of drainage's ability to be highly efficient in terms of design and construction costs, and success at stabilising a wide variety of slope stability cases, drainage of both surface and subsurface water is the most widely used slope stabilisation method.

From the basic slope stability theory and analyses, the existence of water within a potential slide mass generally always reduces stability. There are several reasons for this. First, as potential instabilities are fundamentally gravity-driven, the added weight provided by water in the soil mass adds to the driving forces of the potential sliding mass. Second, any positive pore water pressures generated at the potential slide surface will reduce effective stresses, thereby reducing resisting frictional strength. Third, any seepage forces produced by water flow in the direction of the potential slide will reduce slope stability by adding to driving forces. Drainage of water will also reduce the risk of surface and internal erosion (piping). Therefore, by dewatering and/or redirecting water so that it does not reach the potential slide surface, pore water pressures, seepage forces, and added water weight may are reduced. Drainage technics that apply to slope stabilisation are discussed in the next part.

1.4.3.2. Surface drainage

Surface drainage is the removal of excess water from the surface of the land. Surface drainage requires minimal engineering while providing an effective means of aiding in slope stabilisation. Proper collection and redirection of surface water will ensure that runoff will not erode the surface soils or infiltrate a slope, thereby avoiding additional seepage forces and added water to the slope mass. Surface drainage systems are often simply concrete-lined channels (ditches) or corrugated steel pipes strategically placed at the head of slopes or at berms to divert water that tends to collect, as shown in Figure 1.18. Another option is the installation of one or more shallow interceptor drains placed at strategic locations on a slope to catch and discharge both surface and near-surface water, to maintain any groundwater at a controlled depth within the slope.

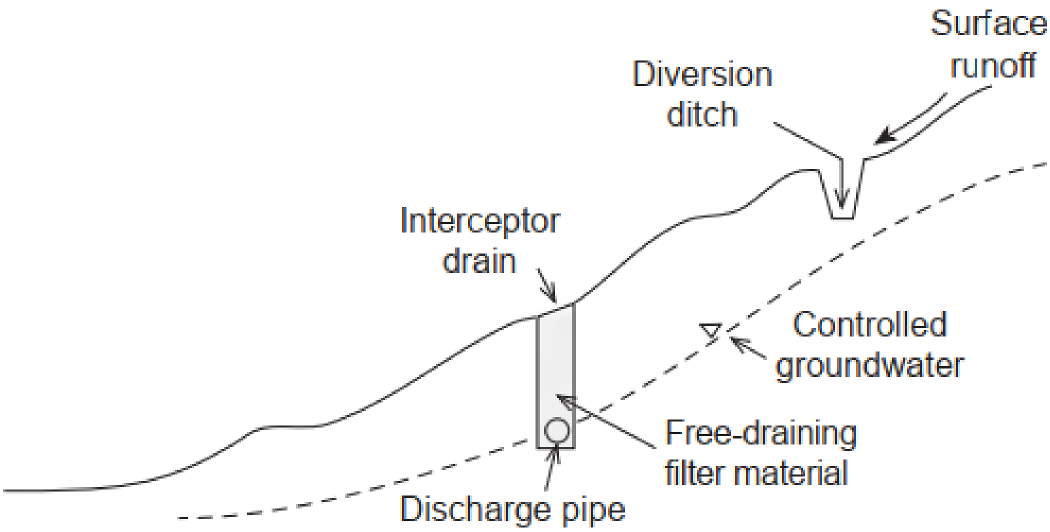


Figure 1.18. Example of surface drainage methods(Nicholson, 2014)

Surface drainage considers maximum inflow volumes, which can be calculated using parameters such as:

- Area and shape of the catchment basin.
- Rainfall intensity.
- Duration of inflows.
- Infiltration coefficient of the surface and subsurface soils (based on ground cover and vegetation).

1.4.3.3. Subsurface drainage

Subsurface drainage is the removal of water from the root zone of plants and deep below the ground to permit better control of the water table. Its intervention includes the use of deep open drains or buried pipe drains as shown in Figure 1.19. Some subsurface drainage methods employed for slope stabilization include:

- Drainage blankets
- Trench drains or cut-off drains
- Horizontal drains
- Relief drains
- Drainage galleries and tunnels
- Vacuum dewatering, siphoning, and electroosmosis.

All the subsurface drainage systems drain by gravity, but pumps may occasionally be employed to assist in removing groundwater.

Figure 1.19. Schematic of drainpipes(Highland & Bobrowsky, 2008)

1.4.4. Retaining structures

Retaining structures include all operations that aim to stabilise landslides, by increasing the external forces that improve slope stability with an increase in the overall soil strength. These methods do not deal with the causes of movements but aim to reduce or stop the deformations by increasing the soil resistance. Retaining structures built to stabilise slopes include retaining walls, anchors, soil nails and piles.

1.4.4.1. Retaining walls

Retaining walls are generally used for small landslides, modification of slope profile at the toe, toe protection from erosion, retains in excavation. Among the retaining walls, we can distinguish them into gravity walls, embedded walls, and reinforced earth walls.

a. Gravity walls

A gravity wall uses its self-weight and the strength of the ground itself to maintain equilibrium. Lateral sliding of the wall is prevented largely by friction between its base and the founding soil. Typically made of masonry or mass concrete. Because of their large mass, they need reasonable foundation soil and are not generally efficient for retaining great heights of material. Gabion walls are the most used example of gravity walls.

A gabion consists of a box made of metal or plastic mesh that is filled in situ with coarse granular material such as crushed rock or as shown in Figure 1.20. The major advantages of the system are:

- Flexible
- Good at absorbing impact energy and often used as rockfall barriers
- Simple to maintain and repair if damaged
- Easy to reuse and recycle
- Cheap

Figure 1.20. Example of a Gabion wall (Bizimana & Sönmez, 2015)

b. Embedded walls

Embedded walls are those that prevent lateral movement partly or wholly by embedding the base of the wall in the ground, normally to a significant depth below the excavation level. Additional support can be provided to the upper part of the wall by propping, or by anchoring into the natural ground on the retained side of the wall. Sheet pile walls are the example of embedded walls.

c. Reinforced earth walls

Here, the soil is reinforced by metal strips, plastic strips, grids, soil nails to allow the outer face to stand at relatively steep slopes and provide internal stability. To design reinforced earth, the strength of the system earth material is considered, but also the maximum deformations with the use of several materials varying in resistance and durability.

1.4.4.2. Anchors

Anchors are threaded steel bars or cables that are inserted into the rock via drilled holes and bonded to the rock mass by cement grout. The anchors may be fully grouted and un-tensioned or anchored at the end and tensioned. They act by applying tensile loads to the face of a slope or by modifying the normal and shear forces acting on the sliding surface.

The purpose of the ground anchor is to generate a force across a structure, either to compensate for an uplift force or compress the foundation on the ground. It must mobilise a volume of ground with sufficient weight to offset the required force. The bond length is designed to transmit the forces to the ground, and the free length is defined according to the required volume of the ground. The prestressing force plays a vitally important role in reducing or preventing vertical movement. In the case of repeated forces, it eliminates the risks of fatigue on the bonding. Anchorage subjected to pre-tensioning is classified as active anchorage. Passive anchorage, not subjected to pre-tensioning, can be used both to nail single unstable blocks and to reinforce large portions of the rock. The disadvantage of the active anchor is that; it will need to be activated each time because after a while it loses its pre-stressed behaviour. Whereas the passive anchor won't need it, for it works when the motion of soil wants to occur.

Anchorage can also be used as pre-reinforcement elements on a scarp to limit hillside decompression associated with cutting. Elements of a composite anchor are shown in Figure 1.21. Inside the anchorage, the resistant elements are the tendons that must be accurately protected from corrosion.

When the anchorage acts over a short length it is defined as a bolt, which is not structurally connected to the free length, made up of an element resistant to traction. The anchorage device may be connected to the ground by:

- Chemical means: in this case, polyester resin cartridges are placed in perforation to fill the ring space around the end part of the bolt.
- Mechanical expansion: the anchorage is composed of steel wedges driven into the sides of the hole.
- Concreting: the anchorage is achieved by concreting the whole mental bar.

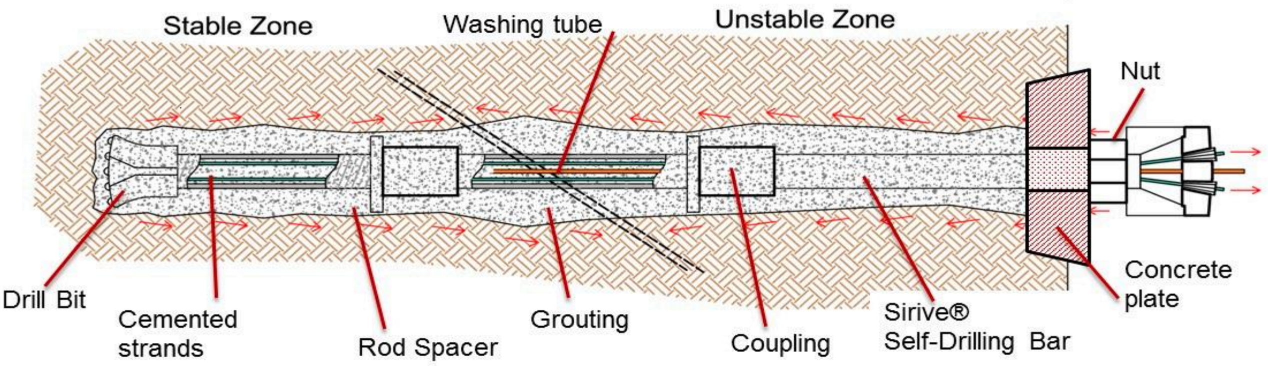


Figure 1.21. Elements of the composite anchors. (Bisson, 2015)

1.4.4.3. Soil nailing

It is a reinforcement technique in which closely spaced parallel steel bars are installed into the face of a slope or vertical cut to improve stability. Figure 1.22 is an example of a soil nail wall.

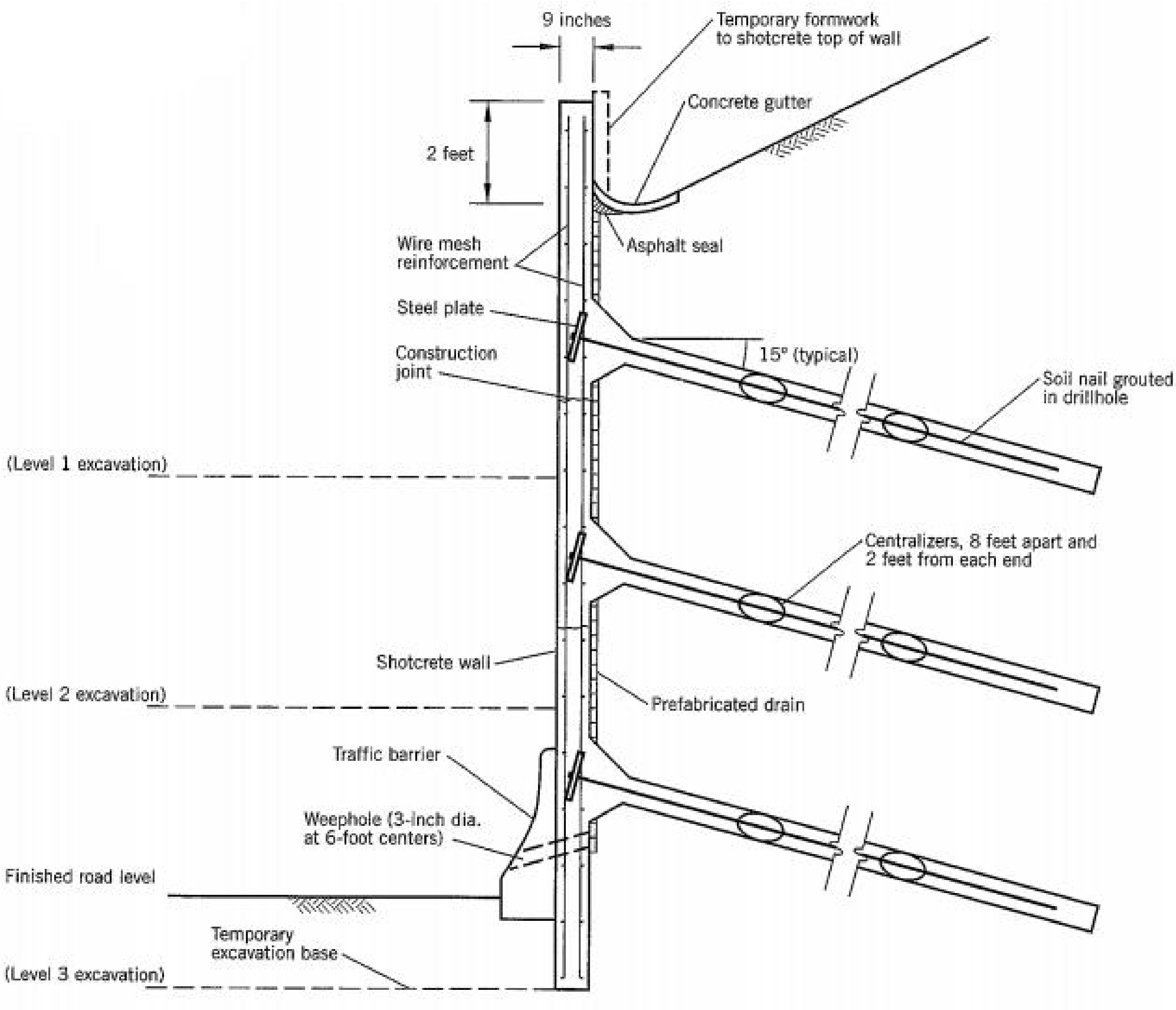


Figure 1.22. Example of soil nail wall details (Cornforth, 2005).

Soil nails develop their reinforcing action through soil-nail interaction due to the ground deformation which results in the development of tensile force. The major part of resistance comes from the development of axial force which is a tensile force; conventionally, shear and bending have been assumed to provide little contribution in providing resistance. The effect of soil nail acts through: increasing the normal force on the shear plane and hence increase the shear resistance along slip plane in friction soil; reduce the driving force along slip plane both in friction and cohesive soils; providing pull-out resistance and act over the entire length. Soil nailing is applied generally on residual soils (weathered rocks), hard cohesive soils, natural bonding dense sand (with apparent cohesion greater than 5 kPa), and soils over the water table.

1.4.4.4. Pile reinforcement

According to Cai & Ugai (2000), piles are an effective way to stabilise slopes with deep sip surface. The interaction among the stabilizing piles is very complex and depends on the soil and pile properties and the level of soil-induced driving force as shown in Figure 1.23.

Piles used to stabilize slopes will be subject to lateral forces developed from the movement of the surrounding soil. De Beer & Wallays (1970) and Ito et al. (1979) suggested that piles subjected to lateral forces can be divided into “passive piles” and “active piles”. According to Ito et al. (1979), the passive piles case is more complex than the active one due to the interaction between the piles and the soil that generates the lateral forces. The general design approach for stabilizing piles follows a procedure presented by Viggiani (1981) in which three main steps are followed. First, evaluating the needed shear force to increase the safety of factor of the slope. Second, evaluating the maximum shear force provided by each pile. Third, selecting the number of pile and the optimum location.

The first step is conducted by performing a stability analysis with a target factor of safety. From the difference between the actual factor of safety (unreinforced) and the target value, the stabilizing force required can then be found. The second step is addressed by performing a lateral response analysis. In this regard, several empirical and numerical methods can be used. These methods can be categorized as:

- Pressure-based methods
- Displacement-based methods.
- Continuum methods

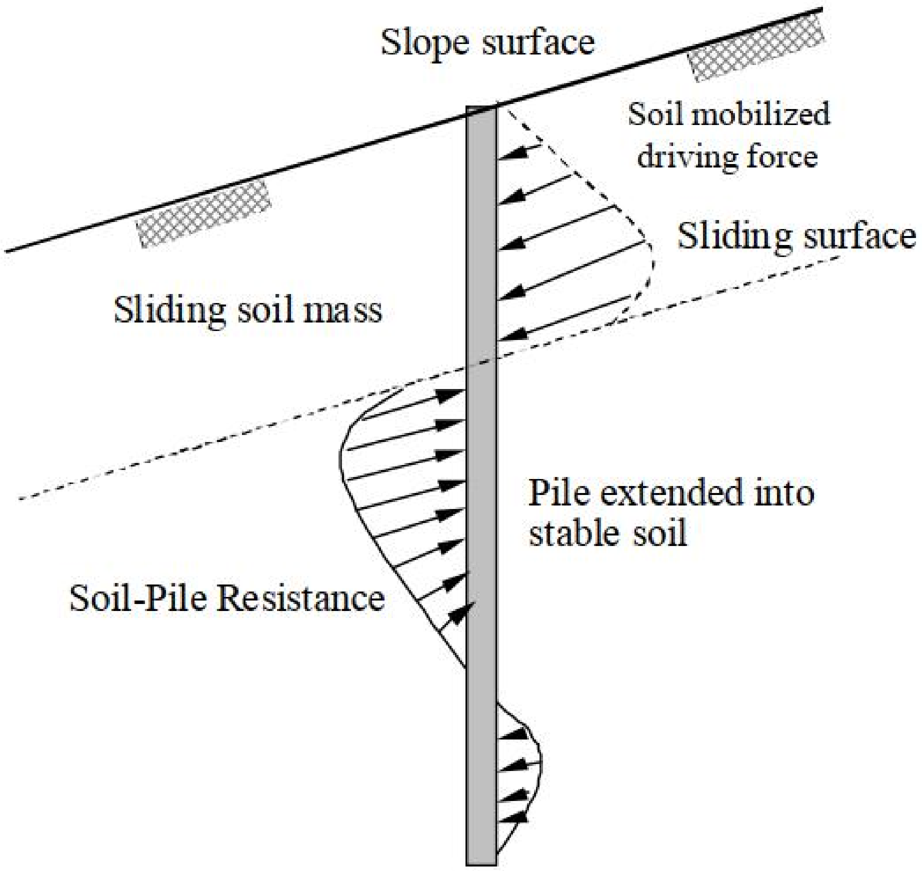


Figure 1.23. Driving force induced by the sliding soil mass above the sliding surface. (Ito et al. 1979)

The design procedure for pile reinforced slopes for distributed load approach can be summarised as follows:

- Determining if the slope requires reinforcement by performing an unreinforced analysis and comparing the factor of safety to a target value.
- Determining the required loads for stabilization that will reach the target factor of safety. This must be determined by performing the stability analysis after placing the pile in the proposed position. From the analysis, the location of the critical surface should be also determined.
- Analysing the pile response with a computer program.
- Developing *p-y* curves for the soil above and below the sliding mass (modifiers should be applied for active landslide and closely-spaced shafts).
- Applying a distributed load to the pile according to NAVFAC (1986) and Reese et al. (2004). The distributed load must be applied from the ground surface to the sliding depth.
- Determining the pile responses to the applied loads and compare the maximum bending moment and shear to the nominal values of the pile to verify structural integrity.

Conclusion

In this chapter soil definitions, composition, formation, properties, typology and classification. The concept of shear strength governing slope stability was discussed for a better explanation of residual shear strength during progressive shear failure. Slope failure causing the downward and outward movement of soil or rock is called a landslide. Landslide occurs when the factor of safety of a slope is equal to or less than 1 caused by the mechanism of several triggering factors. For an unstable slope, it will be necessary to perform a slope stabilisation technique. The next chapter will present the case study and methodology of work for the numerical modelling in the case study.

CHAPTER 2. METHODOLOGY

Introduction

The previous chapter introduced an overview of soils, shear strength governing slope stability, landslide, and landslide stabilisation techniques. This chapter will focus on the description of the methodology of work. The methodology is the part of the study that establishes the research procedure after definition of the problem, so as to achieve the set objectives. It is partitioned into different sections, the first being a site recognition with the landslide location and sequence of events done by documentary research. This is followed by data collection that will enable the modelling and analysis of the slope. Thereafter, this chapter will focus on the description of the design procedures and the governing equations used in numerical modelling which are intended to be used for the stability assessment, with and without reinforcement. The software used shall be GeoStudio 2012. GeoStudio 2012 comprises a suite of products, which provides a single platform for analysing a wide range of geotechnical and geoscience problems; however, the results of these analyses are strongly dependent on the assumptions made and the understanding of the working principles of the software, so care is always recommended when adopting numerical solutions. Finally, this chapter will present the proposed landslide stabilisation methods for this particular case study.

2.1. Site recognition

Recognition of the site was done through research from available documents to know the geographic location, geology, climate, relief and hydrogeology.

2.2. Data acquisition

This section is about how information was collected the case study through documentary research. The data collected were monitoring, geometric and geotechnical data.

2.2.1. Monitoring data

The collection of monitoring data of the case study is done through documentary research. According to Belokas & Dounias (2016), a comprehensive series of instrumentation sets was carried out aiming at monitoring ground movements, structural movements, and water pressure. The monitoring parameters of the slope enabled to carry out the numerical modelling giving the position of the slip surface for a series of events, position of the phreatic line, and the amount of soil movement for the slope deformation.

2.2.2. Geometric data

Through documentary research, the geometric data were obtained from the field investigation presented by Belokas & Dounias, (2016). These geometrical properties give information on the topographic profile, the position of the boreholes, plan view with landslide limits, and potential interpretation of the slip surfaces surface in 2001 and 2003.

2.2.3. Geotechnical data

Through documentary research, the geotechnical data was based on exploratory boreholes and corresponding laboratory tests on selected samples from the two geotechnical investigations, one in 2000-2001 before the major event and one in 2003-2005 after the major event according to Dounias et al. (2006) and Belkonas et al. (2013). From Belokas & Dounias, (2016), boreholes series were performed and monitored during 2001-2003 to obtain the soil stratigraphy, strength parameters, and variation of the groundwater table all along the slope.

2.3. Numerical modelling

Site investigations and soil laboratory testing was intended to compile the soil parameters which are used as input data for slope modelling. The goal of modelling is to simulate as real as possible the in-situ conditions of the slope, understand the combination of material properties and pore pressure distribution immediately before the landslides, assess the effect of increasing water table caused by excessive precipitation and simulate the interventions proposed for stabilisation. The numerical modelling is done following stability analyses, limit equilibrium method and performing numerical simulations.

2.3.1. Stability analyses

Slope stability analyses are performed to assess the safety factor of a particular slope in a given geologic and physical conditions. For a slope to be stable the resisting forces in the slope must be sufficiently greater than the forces causing the failure from Duncan & Wright (2005). Stability analysis can be used for the following:

- To assess the safety of a structure in terms of its stability.
- To locate the critical failure surface and to know it shape of failure.
- To understand and numerically evaluate the sensitivity of stability to its geologic parameters and climatic conditions.
- To assess the movement of the slope.
- To assess remedial measures and improvements in their design.

To perform a slope stability analysis the geometry of the slope, external and internal loading, soil stratigraphy and strength parameters, with the location of the groundwater table along the slope must be defined. In the current state of practice, there are several stability analysis methods available. However, the scope of this report is limited to a discussion on the limit equilibrium method using the software GeoStudio 2012.

2.3.2. Limit equilibrium method

The Limit Equilibrium Method (LEM) of analysis is a well-established method and widely used by geotechnical engineers. This method provides an assessment of the stability of the slope in terms of its safety factor. For determining the factor of safety of a particular slope the primary requirement is the strength properties of the soil material involved and does not consider its stress-strain behaviour. The limit equilibrium method provides only an estimate of the stability of a slope but doesn't provide any information about the magnitude of movement of the slope. The Limit Equilibrium Method is based on the following hypothesis from Nash (1987).

- The perfectly rigid (soil is not deformed until the ultimate condition is attained with the shear strength being constant and independent of the accumulated deformation).
- Prior knowledge of the circular or no circular slip surface is required.
- The mass of soil in potential slip is divided into rigid and limited slice blocks.
- The resistance of the soil is fully mobilized along the entire slip surface.
- Uses plain strain analysis with the failure criterion of Mohr-Coulomb used.

In limit equilibrium techniques, the objective is to find the point or condition where all of the driving or de-stabilizing force is equal to the resisting or stabilizing forces. This condition is determined by satisfying all equations of statics. That is, the summation of forces in the horizontal ($\sum F_x = 0$) and vertical ($\sum F_y = 0$) directions must equal zero and the summation of moments ($\sum M_0 = 0$) about any point must equal zero. The static limit equilibrium methods have two different approaches:

- Single Free Body Procedures.
- Method of Slices.

In the Single Free body procedures the entire mass of the soil is considered to be in equilibrium and a single free body diagram is assumed for the entire mass. The infinite slope method, Swedish slip circle method and logarithmic spiral method are some of the examples of these methods. But this method imposes challenges in calculations when used in the case of a non-circular or a wedged slip surface. As most of the landslides observed in the real world fail with a non-circular failure surface, this method was not popularly used.

The method of slices was adopted by many geotechnical engineers to overcome this disadvantage. The method of slices is appropriate to solve both circular and non-circular slip surfaces separately by Duncan & Wright (2005).In this method, the entire slip surface is divided into several vertical slices and equilibrium equations are applied to each slice. This method is illustrated in Figure 2.1. The figure represents a circular slip surface which is subdivided into slices and also a single slice (named as i^{th} slice in Figure 2.1) is shown with forces acting on it. For illustration purposes, the slice forces used in Bishop's Simplified method are shown. The forces on the slices vary from one method to another method. In Figure 2.1, W_i represents the weight of the i^{th} slice, S_i is the shear force at the base of the i^{th} slice, a_i is the moment arm, α_i is the inclination of the base of the slice. In the single slice figure, N represents the normal force acting in the base of the slice, E_i and E_{i+1} represent the forces acting on the sides of the slices (shear stresses in between the slices are neglected in Simplified Bishop's method). So, using these forces, resisting and driving moments are calculated and their ratio gives the factor of safety value.

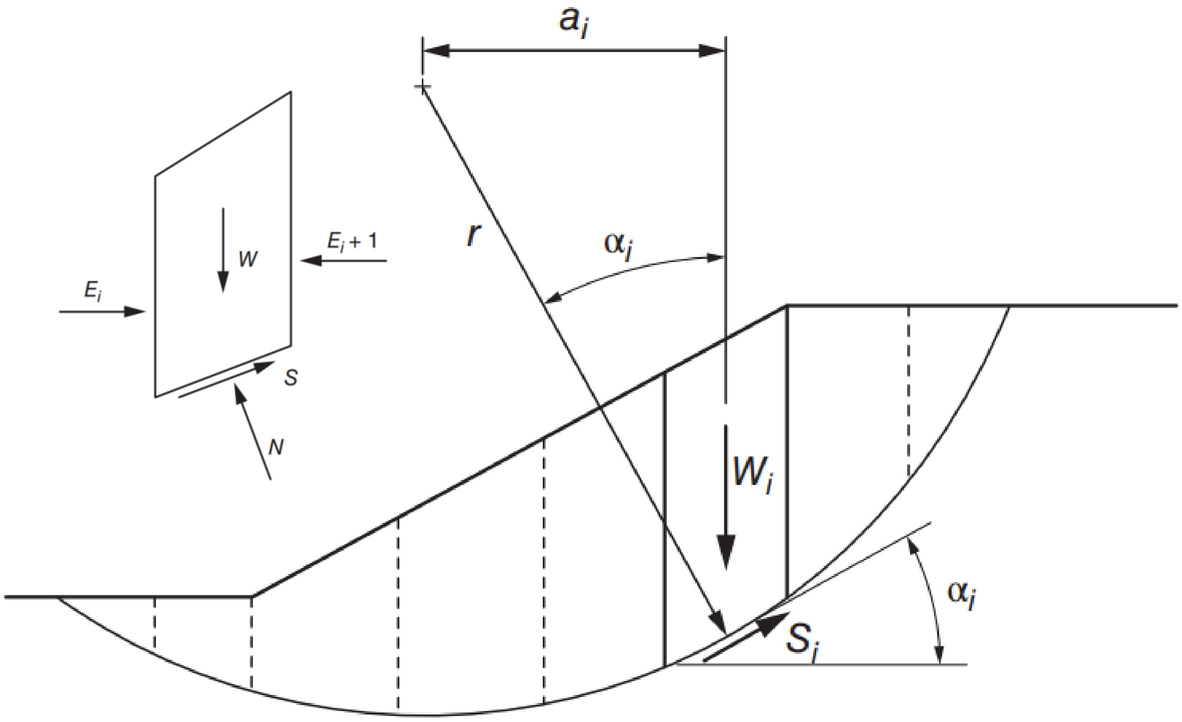


Figure 2.1. Typical representation of a circular slip surface subdivided into vertical slices and forces acting on it (Duncan & Wright, 2005).

A general limit equilibrium (GLE) formulation was developed by Fredlund for unsaturated soil at the University of Saskatchewan in the 1970s According to Fredlund & Krahn (1977) and Fredlund et al. (1981) this formulation encompasses the key elements of all the limit equilibrium methods, based on two factors of safety equations, and allows for a range of interslice shear normal force assumptions. Equation (2.1) gives the factor of safety with respect to moment equilibrium (F_m), while equation (2.2) gives the factor of safety with respect to horizontal force equilibrium (F_f).

$$F_m = \frac{\sum\left(c'\beta R + \left(N - u\beta\frac{\tan\phi^b}{\tan\phi'}\right)R\tan\phi'\right)}{\sum Wx - \sum Nf \pm \sum Dd} \quad (2.1)$$

$$F_f = \frac{\sum\left(c'\beta\cos\alpha + \left(N - u\beta\frac{\tan\phi^b}{\tan\phi'}\right)\tan\phi'\cos\alpha\right)}{\sum N\sin\alpha - \sum D\sin\omega} \quad (2.2)$$

The GLE factor of safety equation with respect to moment equilibrium is given by equation 2.1 while the factor of safety equation with respect to horizontal force equilibrium is given by equation 2.2 with the terms in the equations being:

c' = Effective cohesion

ϕ' = Effective angle of friction

u = Pore-water pressure

N = Slice base normal force

W = Slice weight

D = Concentrated point load

ϕ^b= angle defining the increase in strength due to the pore water pressure (matric suction)

$\beta, R, x, f, d, \omega$ =Geometric parameters

α =Inclination

Some of the popular methods which follow the procedure of slices are the Ordinary Method of Slices (Swedish method of slices or Fellenius method (1927), Bishop's Simplified procedure (Bishop 1955), Janbu's method (1973), Spencer's Method (1967), Morgenstern and Price Method (1965). The accuracy of the computational methods available is based on the extent to which it can satisfy the equilibrium conditions and its assumption on the inclination of side forces on each slice. According to Duncan & Wright (2005), the accuracy of the methods is described in Table 2.1.

The maximum variation in the factor of safety values obtained by using the various limit equilibrium methods taking into account their limitations is+6% by Duncan & Wright (2005). As Morgenstern–Price and Spencer's method satisfy both the moment and force equilibrium they are considered to be the most accurate methods by Duncan & Wright (2005). Also, it is observed that both the methods result in an identical factor of safety values suggested by Duncan & Wright (2005) and Fredlund & Krahn (1977). Based on the accuracy for each method discussed, Morgenstern-Price is used for conducting the back analysis of the Tsakona landslide in 2D in the further sections.

Table 2.1. Summary of 2D Limit Equilibrium methods for Slope stability analysis (Duncan & Wright, 2005)

Method	Accuracy and Limitations
Ordinary method of slices (Fellenius 1927)	• Gives a very low Factor of safety value in case of effective stress analyses for flat slopes with high pore water pressures. • Accurate only when $\emptyset = 0$ analyses • Accurate in case of total stress analyses with circular slip surfaces.
Modified Swedish method (Corps of Engineers 1970)	• Applicable for all types of slip surfaces • Factor of safety values are generally higher than the other methods which satisfy all the conditions of equilibrium.
Bishop's modified method (Bishop 1955)	• Accurate only when circular slip surfaces are involved. • Factor of safety values differ 3% to 5% from the Ordinary method of slices.
Janbu's simplified method (Janbu 1968)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface • Results in a lower factor safety values than other methods satisfying all equilibrium equations.
Spencer's method (Spencer 1967)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface
Morgenstern and Price method (Morgenstern and Price 1965)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface

2.3.3. Performing the numerical simulation

The numerical simulation is done with the use of the software GeoStudio 2012, which comprises a suite of modules, which are SLOPE/W, SEEP/W, SIGMA/W, QUAKE/W, CTRAN/W, TEMP/W, AIR/W, and VADOSE/W. The presence of many modules within GeoStudio provides a single platform for analysing a wide range of geotechnical and geoscience problems. For the present study, SEEP/W is used to simulate the real physical process of pore-water pressure distribution in the model meanwhile SLOPE/W is used to perform the limit equilibrium analysis.

2.3.3.1. Simulation with SEEP/W

SEEP/W is a finite element software product for modelling groundwater flow in porous media that can simulate the real physical process of water flowing through a particular region. For the present study, analysis results obtained using the SEEP/W module can be directly used in terms of pore-water pressure and suction distributions for SLOPE/W. The workflow of the SEEP/W code in GeoStudio is described in the following section.

a. Geometry

Finite element numerical methods are based on the concept of subdividing a continuum into small elements, describing the behaviour or actions of the individual pieces and then reconnecting all the pieces to represent the behaviour of the continuum as a whole. SEEP/W uses the concept of regions to define geometry. Regions defines the slope geometry and stratigraphy which can be drawn directly in GeoStudio or imported as a ‘‘.dxf ’’or ‘‘.dwg ’’ file. Since the geometry of the case study was complex, it is drawn in AutoCAD for importation in GeoStudio. Regions were connected by points to form a continuum. Material properties together with boundary conditions can be applied to each region also modifying the mesh size where necessary.

b. Meshing

Meshing is an aspect of finite element modelling together with material properties and boundary conditions. Meshing involves defining geometry, distance, area, and volume. In GeoStudio, the mesh is generated automatically. The size of the elements can be altered at a global level for the entire mesh, within any one or more regions, or along a line or around a point. The approximate global element size was is set to 20m and the element size set to 5m as shown in Figure 2.2 using the triangular meshing shape.

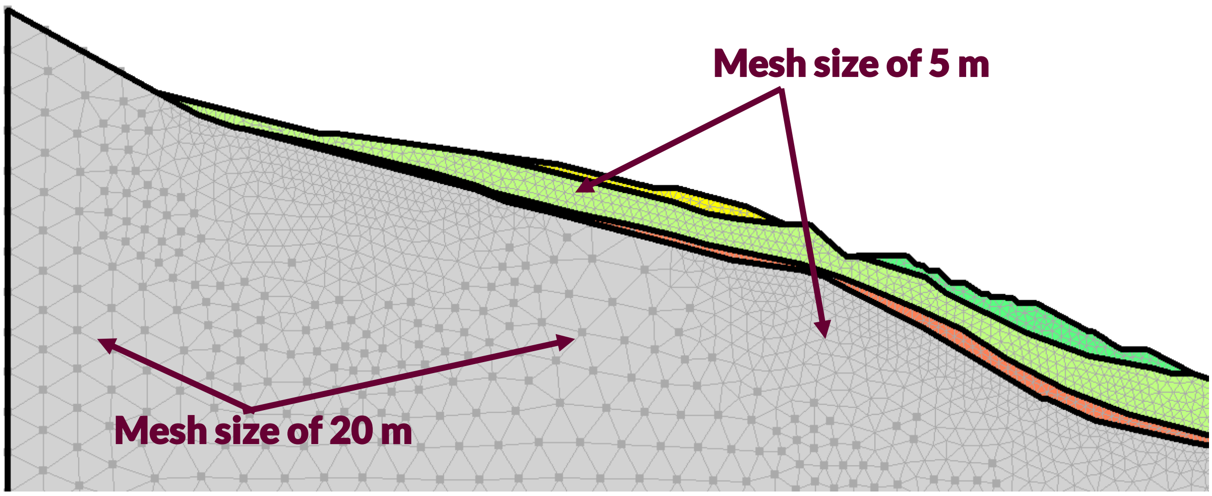


Figure 2.2. Slope geometry and mesh in GeoStudio 2012

c. Material models

Well-defined soil properties can be critical to obtain an efficient solution of the finite element equations. There are four different material models to choose, when using SEEP/W:

- None
- Saturated/Unsaturated model
- Saturated only model
- Interface model

The ‘‘Saturated Only’’ soil model will be used in the SEEP/W analyses for all soil properties with the average permeabilities assigned to the respective soil layers following the documentary research done to obtain the geotechnical data in section 2.2.3. The interface model will be used in the simulation of horizontal drain stabilisation.

d. Boundary conditions

The solution of the FEM equations is constrained by boundary conditions specified across the domain. The boundary conditions generally include ‘‘Pressure’’ and ‘‘Flux’’ conditions. Pressure boundary conditions will be used to come out with an approximate piezometric line as compared to the one proposed by Belokas & Dounias (2016).

e. Analysis Types

There are two fundamental types of finite element seepage analyses, steady-state and transient state analysis. The steady-state analysis will be used over the transient state because it describes a situation where the state of the model is steady and not changing in time. The steady-state analysis is suitable to represent a particular slope condition failure during the numerical analysis. The model will reach a solved set of pressure and flow conditions for the given set of unique boundary conditions applied to it.

2.3.3.2. Simulation with SLOPE/W

This software works on a limit equilibrium framework and includes methods such as the Morgenstern-Price method. The main advantage of this program is its ability to be coupled with other programs such as SEEP/W. The stability of reinforced slopes can be assessed using this software. Reinforcements such as soil nails and piles can be designed. The workflow of the SLOPE/W code in GeoStudio is described in the following section.

a. Geometry

Coupling SLOPE/W to the parent SEEP/W analyses incorporates the same slope geometry as the one initially imported as a ‘‘.dxf ’’or ‘‘.dwg ’’ file.

b. Slice Discretisation

SLOPE/W uses a variable slice width approach to discretize the sliding mass. This approach is used to ensure that a unique soil type is present at the bottom of the sliding mass, to prevent ground surface break at the top of a slice, and to prevent the phreatic line from cutting through the base of a slice. The sliding mass is divided from the point where the slip surface enters the ground surface into several slices determined by the user (default value is 30) as shown in Figure 2.3.

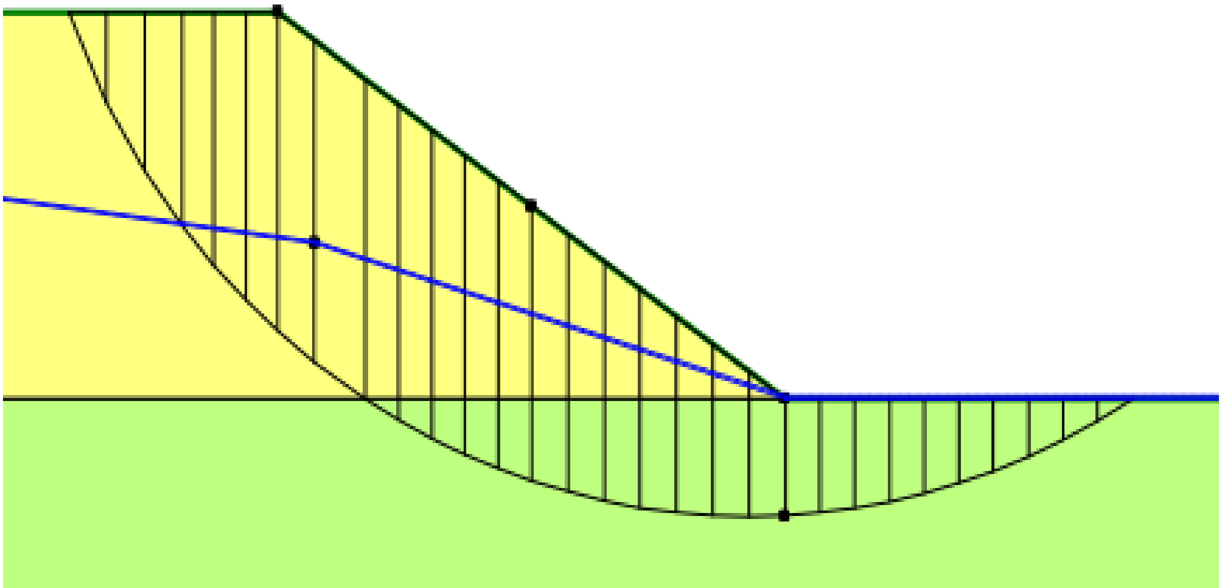


Figure 2.3. Section in a slice discretization process (SLOPE/W 2012)

c. Slice Discretisation

The ground surface line is a necessary feature for controlling what happens at the actual ground surface. Its definition will be done so that the slip surface must enter and exit along this line.

d. Slip surface

Determining the position of the critical slip surface with the lowest factor of safety remains one of the key issues in a stability analysis. As is well known, finding the critical slip surface involves a trial procedure. A possible slip surface is created and the associated factor of safety is computed. This is repeated for many possible slip surfaces and, in the end, the trial slip surface with the lowest factor of safety is deemed the governing or critical slip surface. There are multiple methods available for defining the shape and positions of trial slip surfaces which include:

- Fully specified slip surfaces.
- Entry and exit specifications.

- Grid and radius for circular slips with single and multiple radius points.
- Block specified slip surfaces.

The ‘‘fully specified slip’’ surfaces with the ‘‘entry and exit’’ specifications are used during the stability analyses in the case study (Tsakona landslide) because of the specified slip surfaces obtain from the monitoring data.

e. Material strength

There are many different ways of describing the strength of geotechnical materials (soil or rock) in a stability analysis in SLOPE/W. The Mohr-Coulomb model will be used for the case study due to the available data obtained and its extension to give entry parameters for the additional suction strength of the soil.

f. Pore water pressure

The most realistic position of the water table and pore water conditions are obtained using the SEEP/W module and coupled as parent analysis to the SLOPE/W analysis for stability determination.

g. Safety Factor calculation

There are several methods to calculate the safety factor using the limit equilibrium method. The Morgenstern-Price method is used for this study because it takes into consideration moment and force equilibrium together with interslice normal and shear in its calculation.

2.4. Design of landslide stabilisation techniques

Landslide stabilisation or mitigation works on a slope take into account technical aspects such as the landslide morphology in relation to its accessibility, safety of workers; pre-existing structures and infrastructure that may be affected directly or indirectly; phase and rate of the movement at the time of implementation; capital and operating cost including maintenance; and finally environmental constraints. According to Cornforth (2005), the calculated factor of safety has to be set at high enough levels to overcome analysis errors. It is in this sense that he suggested a value ranging from 1.2 to 1.5 as the safety factor (Annex 6). Chapter 1 has developed several technics used in slope stability. For this research, horizontal drains are coupled either with soil nails (Sirive® special composite self-drilling bars) or piles as an intervention.

2.4.1. Horizontal drain design

Dewatering a slope is a fundamentally simple concept for improving stability. Drainage of both surface and subsurface water is the most widely used slope stabilisation method according to Turner & Schuster, 1996 because of its ability to be highly efficient in terms of design and construction cost, and success at stabilizing a wide variety of slope stability cases.

Horizontal drains are commonly installed to draw down water levels in slopes to add stability. They do this by unweighting the slope material and reducing unwanted excess pore pressures where the drainage water is discharged at the face of a slope into a suitable drain outlet. Horizontal drain is the best alternative when the depth (distance into a slope) to desired dewatering is great. Due to their relatively rapid installation speed, horizontal drains are often a good choice for stabilising active or incipient slides. Many excellent case studies have shown the economic and practical solution to slope stability problems as stated by Black et al. (2009); Machan & Black (2012) and Rodriguez et al. (1988).

The design practice of horizontal drains for slope stabilization has mostly relied on empiricism and judgment, resulting in various successes according to Lau & Kenney (1984) and Pathmanathan (2009). The horizontal drain design model of the present case study of the Tsakona landslide was done according to Black et al. (2009) who described a test program where horizontal drains were installed (with moderate difficulty) to stabilize an approximately 30 m deep landslide in western New York State corresponding to the same slip depth of the case study. The design criteria for the horizontal drainage setup are as follows:

- A minimum permeability of 300 m/day (3.47e-03 m/s), provided by the place-in drilled pipes.
- Installation of small diameter perforated pipes, typically 5-6 cm (2-2.5 inch) in diameter, wrapped with geosynthetic filters in larger diameter drilled holes from Abramson et al. (2002).
- Horizontal drains are installed at small inclines of 5-10° to the horizontal so they can drain by gravity.
- Drains are installed in a fan-shaped arrays with radial spacings of 5-7° and placed at multiple elevation series of increment distance,15-20m to the vertical.
- Drain length varied from 100 m to 180 m to be some few meters close to the bedrock and ensure that water is drawn down and away from any potential slide mass.

2.4.2. Soil nail design (Sirive® special composite self-drilling bars)

Soil nailing uses steel bars to strengthen the ground. The bars are inserted with grout by using either a pre-drilled hole or a self-drilled hole. The self-drilling technic was the one suggested because it is considered a timesaver. Satisfactory performances rely on mobilising tensile resistance in the nails, which in turn requires a significant bond between soil and nail which grouting ensures.

In landslide soil nails the axial stresses that can develop are very high as large volumes of soil are moved, for this reason, Sirive® floating bars can be manufactured using a new type of bar called Sirive® Special Composite Bars. This technology involves the coupling of a traditional bar and strands by inserting one or more strands into the bar cavity and then cementing them in place using a special cement injection (Figure 1.21). The purpose of the harmonic steel strands is to improve the mechanical characteristics of the soil nail.

According to Barison (2020), experimental tests have shown that the coupling of the bars to the strands allows the realisation of soil nails with high breaking loads while maintaining ductility. Furthermore, the cracks in the external concrete bulb are very low, thus increasing the durability of the soil nail against corrosion. An analysis of production and installation costs has shown that, for the same breaking load, a saving of 45% can be made compared to a traditional self-drilling bar. Other strengths of the composite bars, in addition to lower costs, are the simplicity of transporting the materials, the speed of execution, and the continuity given by the strand to the complete reinforcement. This last point is particularly important when the perforations are very long. The strand helps to improve the coupling between successive bars, which would otherwise only be guaranteed by the jointing sleeves between the bars. The anchorage is completed at the end by a reinforced concrete floating plate with a truncated cone shape (Annex 5), connected to the bar by a special steel header. The geometry of the plate was designed in such a way as to generate a greater volume of influence on the ground, improving the surface stability of the slope.

Soil nail with Sirive® S90 Special self-drilling composite bars with 8 strands were used for the stabilisation intervention with the bar characteristics shown in the datasheet in Annex 9.

2.4.3. Pile Stabilisation Design:

Piles are generally used in the stabilisation of deep slip failure surfaces. Piles used to stabilise slopes will be subject to lateral forces developed from the movement of the surrounding soil. As explained in section 1.4.4.4., the general design of piles follows a procedure presented by Viggiani (1981) in which three main steps are followed. First, evaluating the needed shear force to increase the safety factor of the slope using equation (2.3). Second, evaluating the maximum shear force provided by each pile. Third, selecting the number of piles and the optimum location using the data in Annex 7.

$$\Delta F_R = FS * \sum F_D - \sum F_R \tag{2.3}$$

Where F_R represent the resisting forces, F_D the activating forces, FS the factor of safety, and ΔF_R the shear resisting force or limit resistance of pile.

Conclusion

This chapter presented the methodology of work. This followed documentary research for site recognition (geographic location, geology, climate, relief, and hydrology). The investigations made provided the monitoring data which gave the ground movements, structural movements and water pressure for the sequence in the occurrence of the landslide. The geometrical data provided the shape and size of the slope where the landslide occurred. The geotechnical data gave the stratigraphy, physical and hydromechanical characteristics of the soil. Slope stabilisation was carried out using the limit equilibrium method considering the current state of the soil in 2001 and the effect of excessive rainfall in 2003. The software GeoStudio 2012 was used, emphasising modules SEEP/W for the simulation of the flow of water in the soil and the module SLOPE/W for the stability analysis. Presentation of site, modelling parameters, and interpretation of the numerical simulation analyses results will be presented in chapter 3.

CHAPTER 3. PRESENTATION AND INTERPRETATION OF RESULTS

Introduction

Instability-related issues causing landslides in engineered, as well as natural slopes, are common challenges to both researchers and professionals. In this chapter, after the site presentation of the Tsakona landslide taking into consideration the geographic location, geology, climate, relief, and hydrogeology, the modelling parameters will be presented. The previous methodology will be applied to reproduce numerically the landslide events that lead to the major soil mass movement in February 2003. The conception of soil suction will be considered in the analyses. The design of landslide stabilisation techniques that could have prevented the landslide occurrence will be modelled and simulate too.

3.1. Presentation of the site

The study area is presented though its geographic location, geology, climate, relief, hydrogeology and history of the Tsakona landslide.

3.1.1. Geographic location

The Tsakona landslide occurred in Peloponnese (Greece) as shown in Figure 3.1.

Figure 3.1. Location of the Tsakona Landslide in Peloponnese (Pitichinaccio, 2008)

Greece is a country found in South-Eastern Europe in the southern part of the Balkans Peninsula, Bordering the Mediterranean Sea. At the southern tip of the Greece mainland (21,549.6 square kilometers), there is a large peninsula called Peloponnese, where a major catastrophic landslide occurred in February 2003. The affected area is located in the prefecture of Arcadia, where a fertile plateau is surrounded by mountains covered with lush vegetation. The landslide occurred on the Paradesia-Tsakona part of a Motorway that connects Tripoli (capital of Arcadia) with Kalamata (capital of Messinia and second biggest port of Peloponnese) as shown in Figure 3.1.

3.1.2. Geology

The geology of Greece is highly structurally complex due to its position at the junction between the European and African tectonic plates. Some of the oldest rocks in Greece are from the Paleozoic and are usually metamorphosed with no fossils. The Tsakona landslide area is situated on an overthrust part of the Pindos unit, in the Gavrovo-Tripolis zone that defines the geological settings of the area. Information on the geological section of landslide is presented by Sotiropoulos et al (2004) is shown in Figure 3.2.

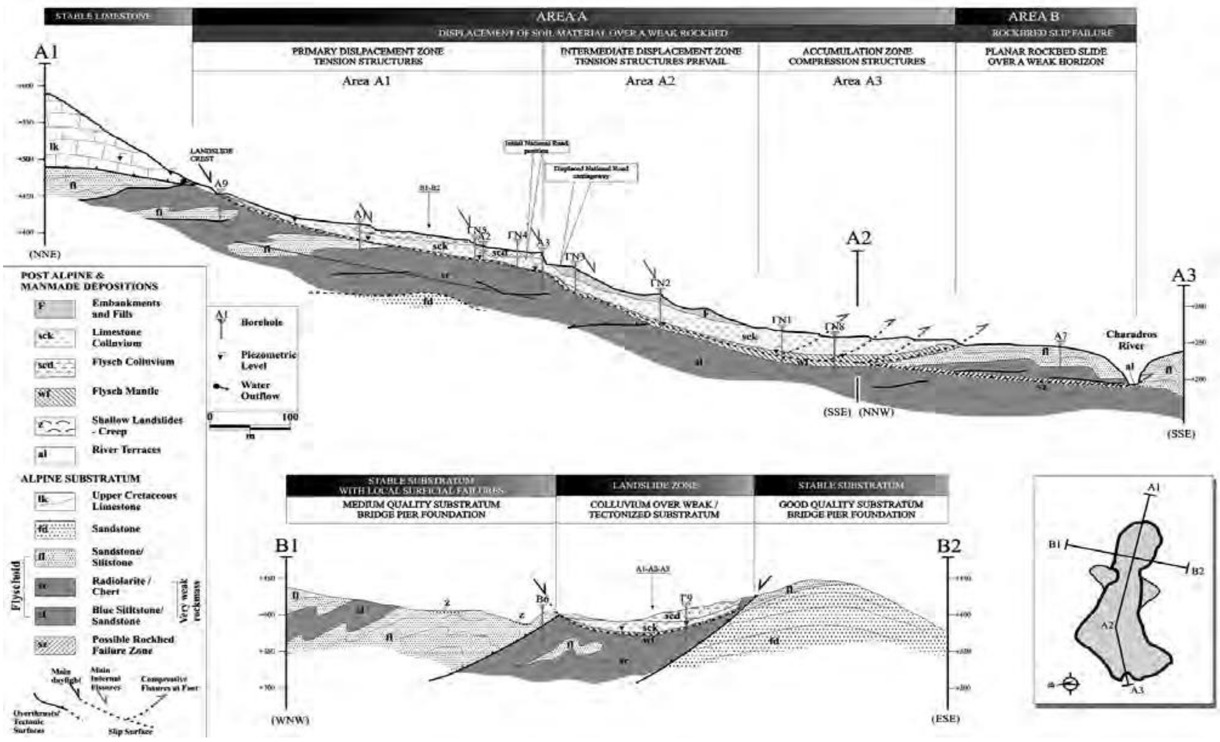


Figure 3.2. Geological section of the landslide. (Belokas & Dounias, 2016)

The zone is composed of an alpine substratum together with post-alpine with manmade depositions:

- Alpine tectonized Flysch of Upper Jurassic-Lower Cretaceous age with the two subunits being sandstone and flysch.

- Alpine Upper cretaceous Limestone with clay-marl interlayers in the deeper horizons and quartzite interlayers located above the landslide scarp.
- Weathered flysch (flysch mantle), a weak layer that comes from the weathering of the in situ underlying bedrock.
- Post alpine geological deposit (colluvium deposit) composed of siltstone and claystone flysch colluvium being clayey with limited presence of sand and gravel.
- Manmade deposits (embankments and fills), created during the construction of the motorway coming predominantly from the excavated flysch, with a variable grading, clayey or sandy deposits with gravel and cobbles mostly of sandstone origin.

3.1.3. Climate

The climate in Greece is predominantly Mediterranean. The Greek mainland is extremely mountainous and has a remarkable range of micro-climates and local variations. Peloponnese is one of the warmest regions in Greece with an average daily temperature of 23 ºC. From the Köppen-Geiger climate classification, the climate of central-western Peloponnese is Mediterranean, dry-summer subtropical characterised by a strong winter to summer rainfall contrast. The extreme rainfall events are mostly associated with cyclones as humid Mediterranean air is advected (transferred) against the slope of the mountain ridges like the one of the Tsakona landslide area. From the year 2000 to 2003 (period of the landslide occurrence), in the broader area of interest, the average monthly winter rainfall is more than 5 times greater than the corresponding summer rainfall (1957-1999 online data from the Hellenic National Meteorological Service). The monthly cumulative rainfall in January and February 2003 was up to 200% of the average rainfall for the period 1961-1990 according to the National Observatory of Athens (2003a and 2003b) monthly meteorological bulletins. Direct comparisons of the rainfall with the landslide evolution are not possible because precipitation measurements were not available in the landslide or nearby area (like surface movements, pore water pressure, soil profile movements).

3.1.4. Relief

Peloponnese has a mountainous interior and deeply indented coasts. Mount Taygetus in the south is the highest mountain in the Peloponnese, at 2,407 meters. Other important mountains include Cyllene in the northeast, Aroania in the north, Erymanthos and Panachaikon in the northwest, Mainalon in the center, and Parnon in the southeast.

3.1.5. Hydrogeology

Greece is dependent on groundwater resources for its water supply. The main aquifers are within carbonate rocks (karstic aquifers) and coarse-grained Neogene and Quaternary deposits (porous aquifers). At the landslide area, the highly permeable upper cretaceous limestone is the main source of groundwater within the colluvium and the substratum. The stable substratum in the landslide area shows a lower permeability except for the sandstone and chert horizons that influence the piezometric regime. Springs do develop at the limestone-flysch contact above the unstable area. The surface runoff from the springs and the rainfall infiltrates primarily within the high permeability limestone colluvium horizons and the man-made fills. The piezometric level developed a few meters above the low permeability stable substratum, under the colluvium (the weathered flysch), where most of the landslide movement occurs. The piezometer recordings during the observation period revealed a small seasonal fluctuation of this piezometric level.

3.2. Modelling parameters

This includes the monitoring, geometric and geotechnical parameters of the Tsakona slide to perform the suitable numerical modelling.

3.2.1. Monitoring parameters

A complex movement of a deep translational slide accompanied by a surficial soil flowing movement caused one of the largest motorway landslides in Greece in February 2003. Geotechnical investigations in 2000-2001, before this catastrophic event, had already revealed a soil slope deformation, of about 680 m length and 2.500.000 m^3 volume, moving at a slow rate. The moving materials were mainly surficial deposits (including embankments and fills), while the deep movements were developing mainly within a weathered flysch above the flysch bedrock, at depths varying from about 20 m to 35 m.

Although the displacement rate was low, acceleration could not be avoided in this environment, modified by a combination of geological processes and manmade interventions. In the Tsakona case, a prolonged very wet season most probably provided the final trigger of an already precarious stability condition. This major event occurred in February 2003 and expanded the limits of the landslide down to the riverbed, locally blocking the river flow. Notably, about 200 m of the carriageway slipped for approximately 100 m in plan and 40 m vertically. At this stage, the landslide material amounted to 1050 m length and 6.000.000 m^3 volume. Movements were negligible after the end of the major event. The landslide was destructive, totally altering the landscape, as cracks and ruptures of various widths and heights developed, streams were diverted, scattered ponds were created and agricultural roads and warehouses were destroyed. The corresponding failure is one of the largest highway landslides in Greece, being a complex translation of a deep slide and a flow of surficial material. The materials moved were colluvium, weathered mantle and recent embankment and fill materials. The deep slide developed mainly around the interface of the colluvium with the flysch bedrock.

3.2.2. Geometric parameters

For a two dimensional (2D) analysis, the geologic section A1, A2 and A3 shown in Annex 1 and Figure 3.2 is considered. This section is considered to be representative of the overall landslide in 2D and ranges from the southern Charadros river to the crest above the national road.

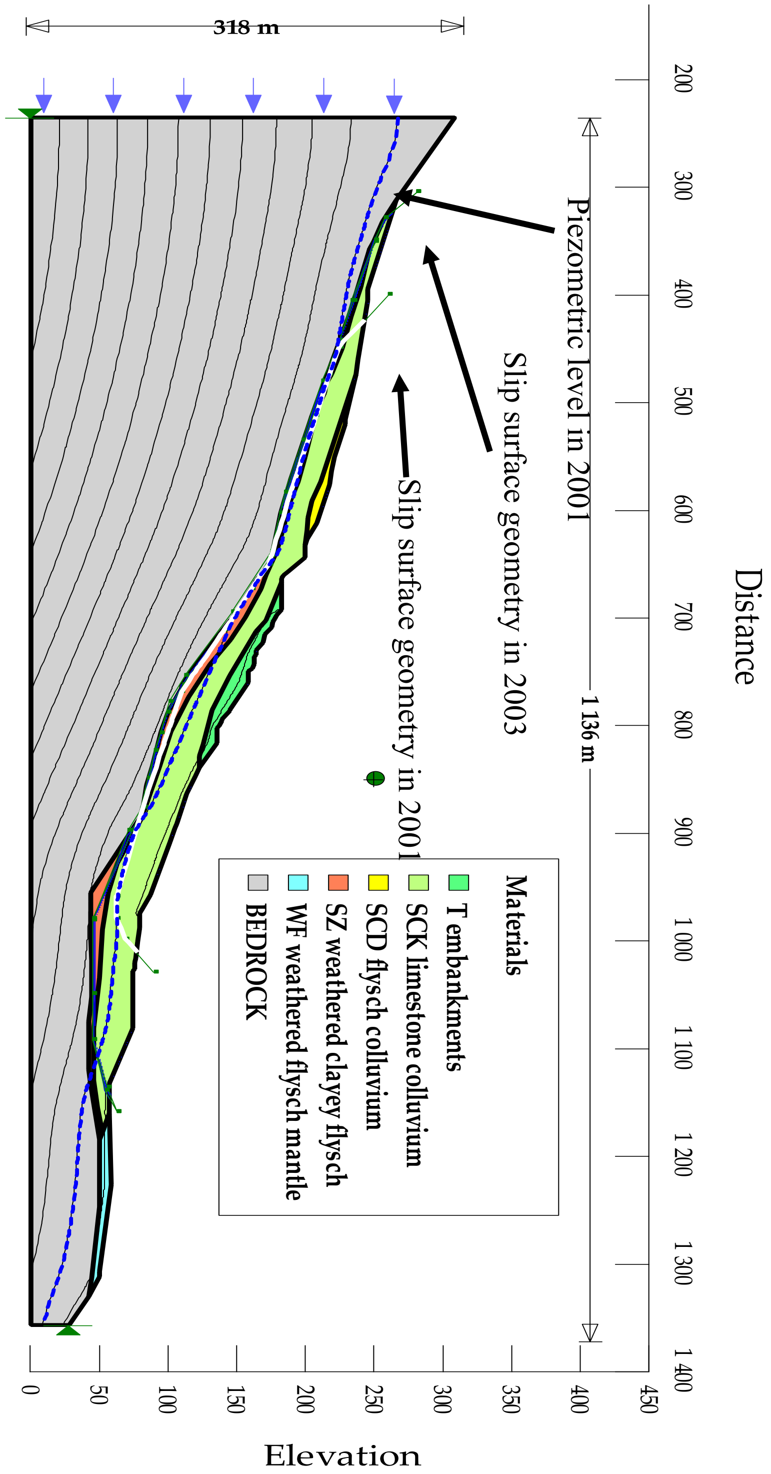


Figure 3.3: GeoStudio model of the Tsakona landslide cross-section (SEEP/W 2012)

From Annex 1 and Annex 2 the landslide had the following geometrical characteristics:

- Width on the road axis: 200m
- Maximum width (below road axis): 370m
- Crest to toe length: 680m
- Maximum depth: 32m
- Maximum depth on the road edge: 28m
- Volume (approximately): 2.500.000 m^3
- Average displacement rate: about 20 to 25 cm/year.

These observations lead to the conclusion of a slow rate active landslide (velocity class 1) according to Cruden & Varnes (1996) classification and soil slope deformation according to Hungr et al. (2014) in which a different pore pressure regime could lead to a more destructive activation. Figure 3.3 shows the section of the Tsakona landslide cross-section, drawn in the software AutoCAD and imported to GeoStudio.

3.2.3. Geotechnical parameters

The soil characterisation is obtained on exploratory boreholes and corresponding laboratory tests. Two geotechnical investigations were carried out. One in 2000-2001 before the major event and another in 2003-2005 after the major events. Boreholes series ΓN were performed and monitored revealing the following depth soil formations as shown in Annex 2.

- Embankments and fills (manmade deposits): These materials have been placed with a maximum horizontal/vertical slope angle of 3/2 that is considered stable. These materials are characterised according to USCS as clayey sands with gravel to clayey gravel with sand.
- Limestone and flysch colluvium: According to USCS limestone colluvium is characterised as clayey sand with gravel to clayey gravel with sand while flysch colluvium is characterised as low plasticity clay.
- Weathered flysch mantle: It is the in-situ surficial weathered flysch, found mainly on top of the bedrock outcrop close to the Charadros River (north of the stream). This material is characterised as clayey gravel with sand.
- Weathered clayey flysch (flysch mantle): This is the zone where most of the shear slide occurred. According to USCS this material is characterised as clayey sand with gravel.
- Bedrock: This is the substratum which is the flysch, an inhomogeneous material. It is made of sandstone/siltstone flysch as shown in Annex 2.

From the documentary research and several trials and error LEM analysis carried out to represent the Tsakona landslide, the optimal geotechnical parameters in Table 3.1 were used, being following Belokas & Dounias (2016) and the Swiss Standard SN 670 010b (2013).

Table 3.1. Summary of geologic parameters. According to Belokas and Dounias (2016) together with the Swiss Standard SN 670 010b (2013).

Symbol	Soil Layer	Classification according to USCS	Symbol	Permeability (m/sec)	Permeability (m/day)	Unit weight (kN/m^3)	Cohesion (kPa)	Phi, ϕ (°)	Phi, ϕb (°)
T	Embankment and fills	Clayey sand with gravel to clayey gravel with sand	(SC-GC)	5.5e-07	4.752e-02	20.5	15	28	14
SCD	Flysch colluvium	Low plasticity clay	(CL)	5.0e-09	4.320e-04	19	4	27	12
SCK	Limestone	Clayey sand with gravel to clayey gravel with sand	(SC-GC)	5.5e-07	4.752e-02	20	20	30	15
SZ	Weathered clayey Flysch	Clayey sand with gravel	(SC) locally SM	5.5e-06	4.752e-01	22	22	30	-
WF	Weathered flysch mantle	Clayey gravel with sand	(GC)	5.0e-07	4.320e-02	20	20	28	17
	Bedrock	-	-	1.0e-10	8.640e-06	-	-	-	

Residual strength measurements on soil samples obtained through the reversal direct shear technique gave a range for the residual angle of friction, ϕ_r =16° to 20°. Moreover, samples of this material gave Atterberg Limits of about PL=15% and LL=35%. The residual strength values determined for the weathered clayey flysch are close to the low end of the known published values, empirical equations, and correlations for the measured Atterberg

Limits (Annex 3). However, the inhomogeneity along the slip surface and the large extent of the landslide makes it unsafe to solely rely on the laboratory determined residual shear strength for the interpretation of the slope stability. Therefore, the back analysis technique can also be used for an estimation of the average mobilized residual angle of shearing resistance.

3.3. Numerical simulation analyses

This section will discuss on the numerical simulations done using the limit equilibrium method for the slope back analysis and simulation of the stabilisation techniques.

3.3.1. Limit equilibrium stability back analysis before the major event of 2003

This section will describe the numerical simulation, result, and interpretation of the limit equilibrium stability back analysis before the major event using the slip surface geometry in 2001.

3.3.1.1. Simulation of the landslide event in 2001

Due to the considerable displacements developed along the deep slip surface, the mobilized strength is expected to correspond to the residual one (behaviour of the slide is governed by residual strength properties). Residual properties can be obtained either by conducting the traditional back analysis or by laboratory testing. According to Bromhead and Dixon (1986), the reliability of the value obtained from the back analysis is proportional to the confidence with which the pore water pressure and the location of the slip surface in a slope are known. Conforming to Belokas and Dounias (2016), because of the extensive availability of instrumentation data in the Tsakona Landslide case study, the defined slip surface and the groundwater elevations reported are considered to be reliable.

In this study, a back analysis is performed to determine the residual friction angle (ϕ_r) along the slip surface of the event in 2001. The location of the slip surface and piezometric level is shown in Figure 3.3 and Annex 2. The hydromechanical parameters obtained from documentary research based on the USCS showed that the stability analysis gave a FS greater than one. This means that the first event in 2001 occurred when the mechanical properties have already undergone a reduction. Therefore, the mechanical properties (friction angle and cohesion) were reduced to obtained a combination that gave a FS of unity using the literature slip surface of 2001.

The module SEEP/w in GeoStudio using steady-state analysis simulate the physical flow of water through the slope by defining pressure heads (-15 m, -10 m, and -25 m) below the ground surface as boundary conditions set up under steady-state analysis as shown in Figure 3.4. The boundary pressure was chosen to have a phreatic line being similar to the published shape by Belokas & Dounias (2016). The idealized section made in GeoStudio (Figure 3.3) is compliant with the realist section of Annex 2. The SEEP/W analysis is made parent analysis to the SLOPE/W model, performed in the Morgenstern-Price methods with all soil layers are modelled using Mohr-Coulomb's model and the ''fully specified'' slip surface defined.

3.3.1.2.Interpretation of the results

The seepage through the slope brings about the development of two regions; a saturated zone found below the phreatic surface and the unsaturated zone found above the phreatic surface. The pore pressure development in the unsaturated zone is due to capillarity giving rise to a negative pore water pressure (suction). Figure 3.4 presents the pore water variation across the Peloponnese slope in steady-state conditions. Pore water pressure is calculated using equation (3.1) in SEEP/W.

$$u = \gamma_w * z \qquad (3.1)$$

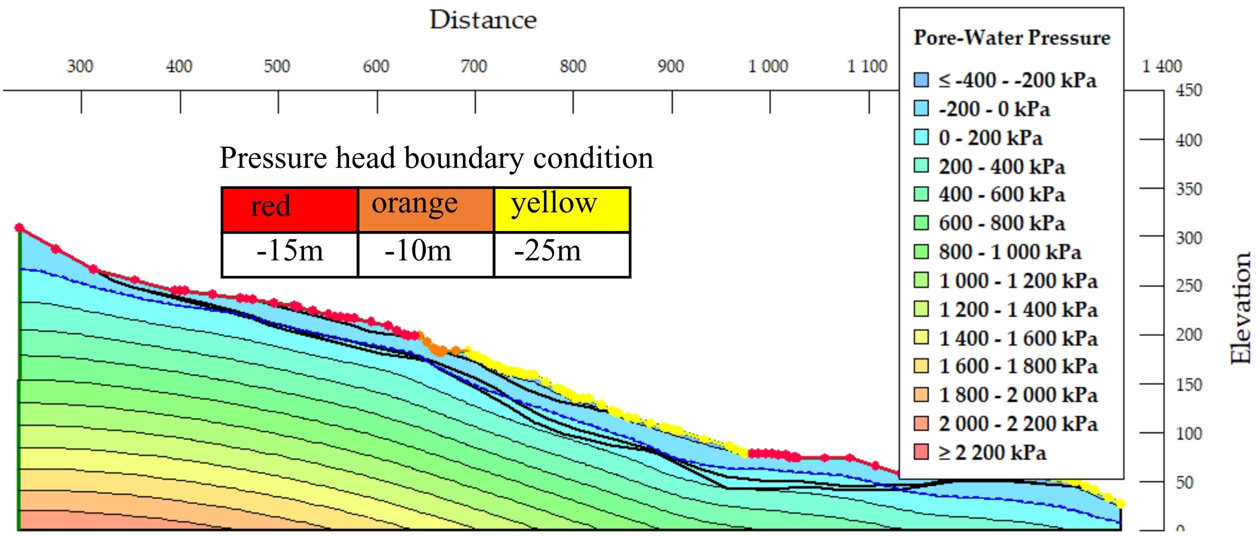


Figure 3.4. Pore water variation of the Peloponnese slope in 2001 (SEEP/W 2012)

Figure 3.5 represents the limit equilibrium back analysis of the slope cross-section for the slip geometry in 2001. The green colour shaded area represents the sliding surface area. The residual angle obtained by this analysis was 17,2° for both the weathered clayey flysch (SZ) and limestone colluvium (SCK). This residual value is in good agreement with the published data (between 16° and 20°) based on Lupini et al. (1981) with Stark and Eid (1994). The factor of safety obtained was equal to 1.00 for the slip surface in 2001 and equal to 1.17 for the slip surface in 2003. The first event that occurred in 2001 was related to residual angle conditions (reduction in friction angle). The LEM analysis done in SLOPE/W doesn't allow the visualisation of displacements and velocities of the unstable slope included slow rate displacements (velocity class 1) according to Cruden & Varnes (1996) classification and soil slope deformation according to Hungr et al. (2014).

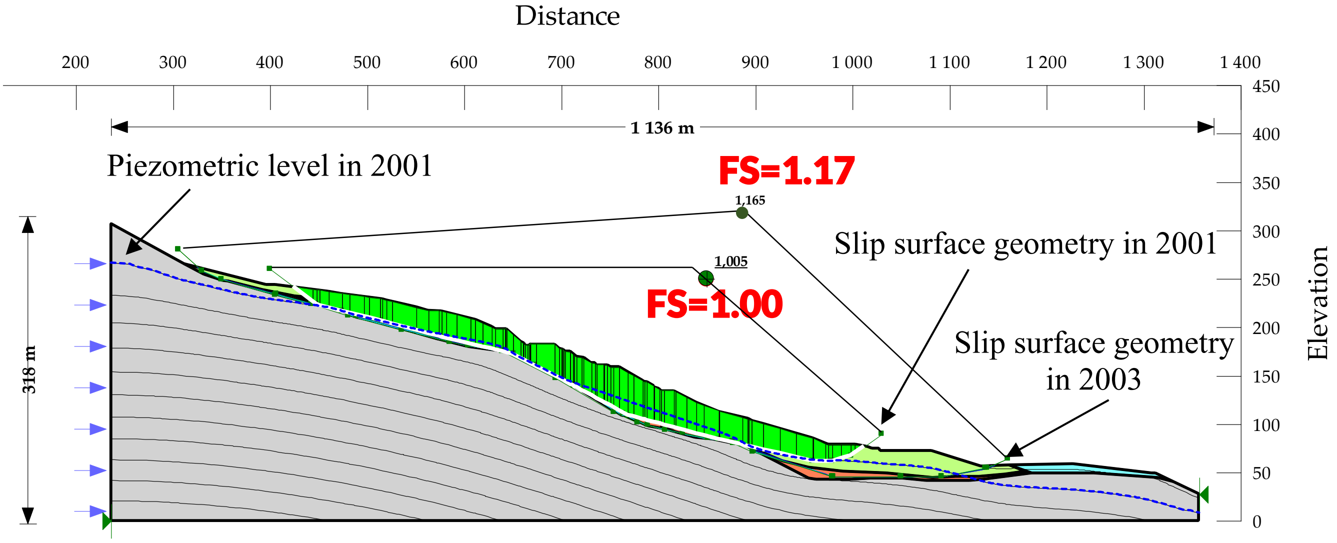


Figure 3.5. Back analysis for the slip geometry of 2001 (SLOPE/W 2012)

The factor of safety obtained along the slip geometry of 2001 with respect to force and moment equilibrium is illustrated by Figure 3.6 converging towards the value of 1.

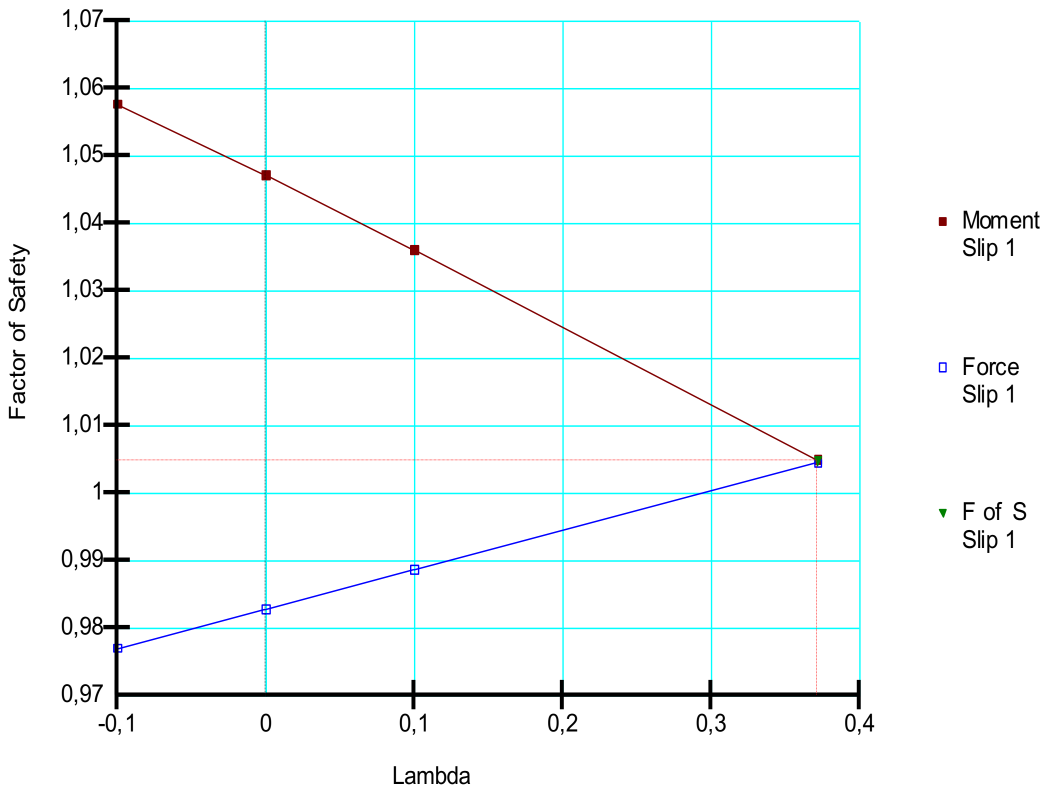


Figure 3.6. Factor of Safety vs. Lamda for the slip geometry of 2001. (GeoStudio 2012)

3.3.2. Limit equilibrium stability back analysis for the major event in 2003

This section will describe the numerical simulation, result, and interpretation of the limit equilibrium stability back analysis for the major event in 2003.

3.3.2.1. Simulation of the landslide event in 2003

For this new stability analysis, the SLOPE/W parameters were kept constant as in section 3.3.1. but the pressure head boundary condition in SEEP/W was changed to -10 m; -7 m, and -20 m, to simulate the raise of the piezometric surface. The only way to simulate the increase in water level is the change in pressure boundary condition because according to Belokas & Dounias (2016), precipitation measurements were not available for the slope or nearby area.

3.3.2.2. Interpretation of the results

Raising the phreatic level by modifying the pressure boundary conditions give the new pore water variation of the Peloponnese slope in 2003 as shown in Figure 3.7.

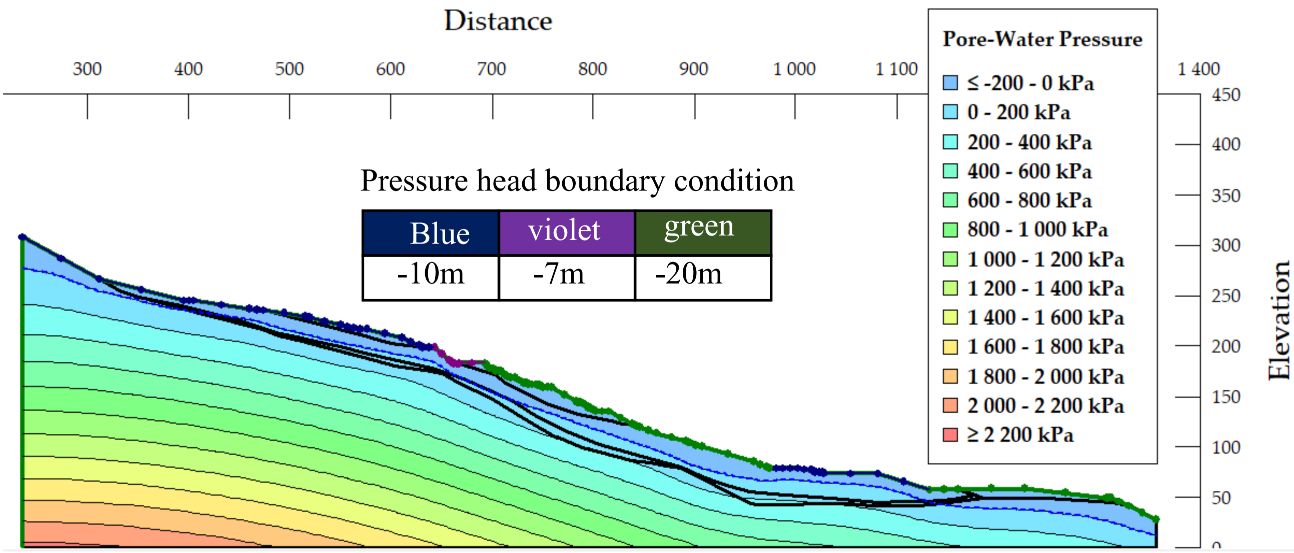


Figure 3.7. Pore water variation of the Peloponnese slope in 2003 (SEEP/W 2012)

The new factor of safety for the slip geometry of 2003 was equal to 1.00 as shown in Figure 3.8. From the previous analysis, it is realised that the factor of safety of the second slip surface in 2003 was 1.17. The slope's relative stability between the two events of 2001 and 2003 is explained by the additional suction strength in the unsaturated soils. Conforming to Belokas & Dounias (2016), the prolonged unusually very wet season in 2003 provided the final trigger on the already precarious stability condition. The combination of material residuals properties and the raise of the piezometric surface were used in the back analysis simulates the landslide event in 2003. According to Belokas et al. (2013), the landslide event of 2003 included rapid and big displacements (velocity class 5) according to Cruden & Varnes (1996) classification and translational slide according to Hungr et al. (2014).

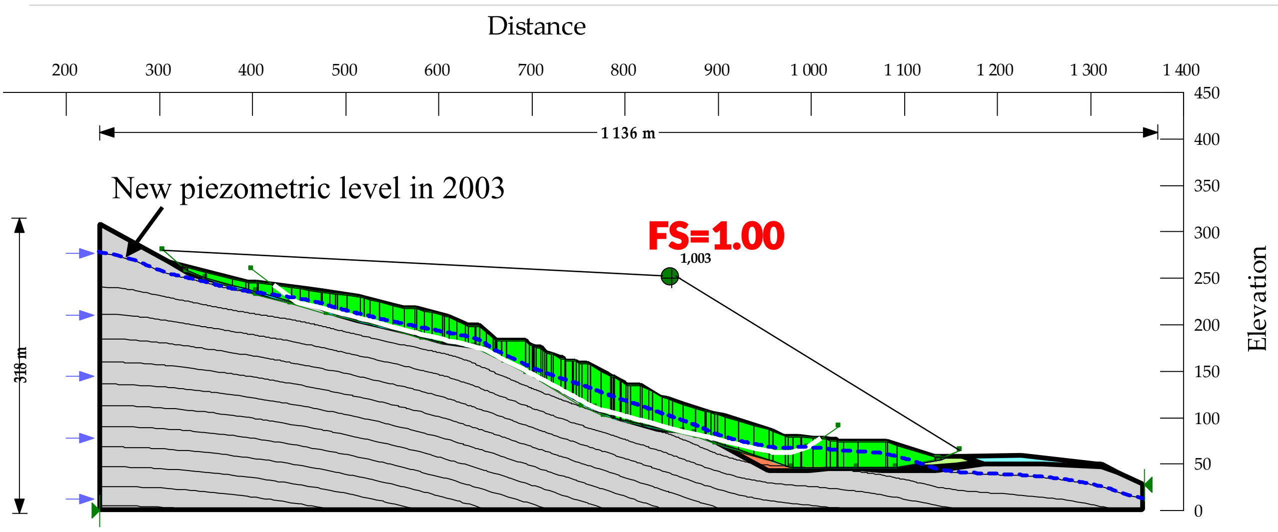


Figure 3.8. Back analysis for the Tsakona landslides event of 2003 (SLOPE/W 2012)

The factor of safety obtained along the slip geometry of 2003 with respect to force and moment equilibrium is illustrated by Figure 3.9 converging towards the value of 1. The new surface geometry after the landslide with the landslide limit down the riverbed are shown in Annex 1 and Annex 2, giving a good visibility of the magnitude of the event.

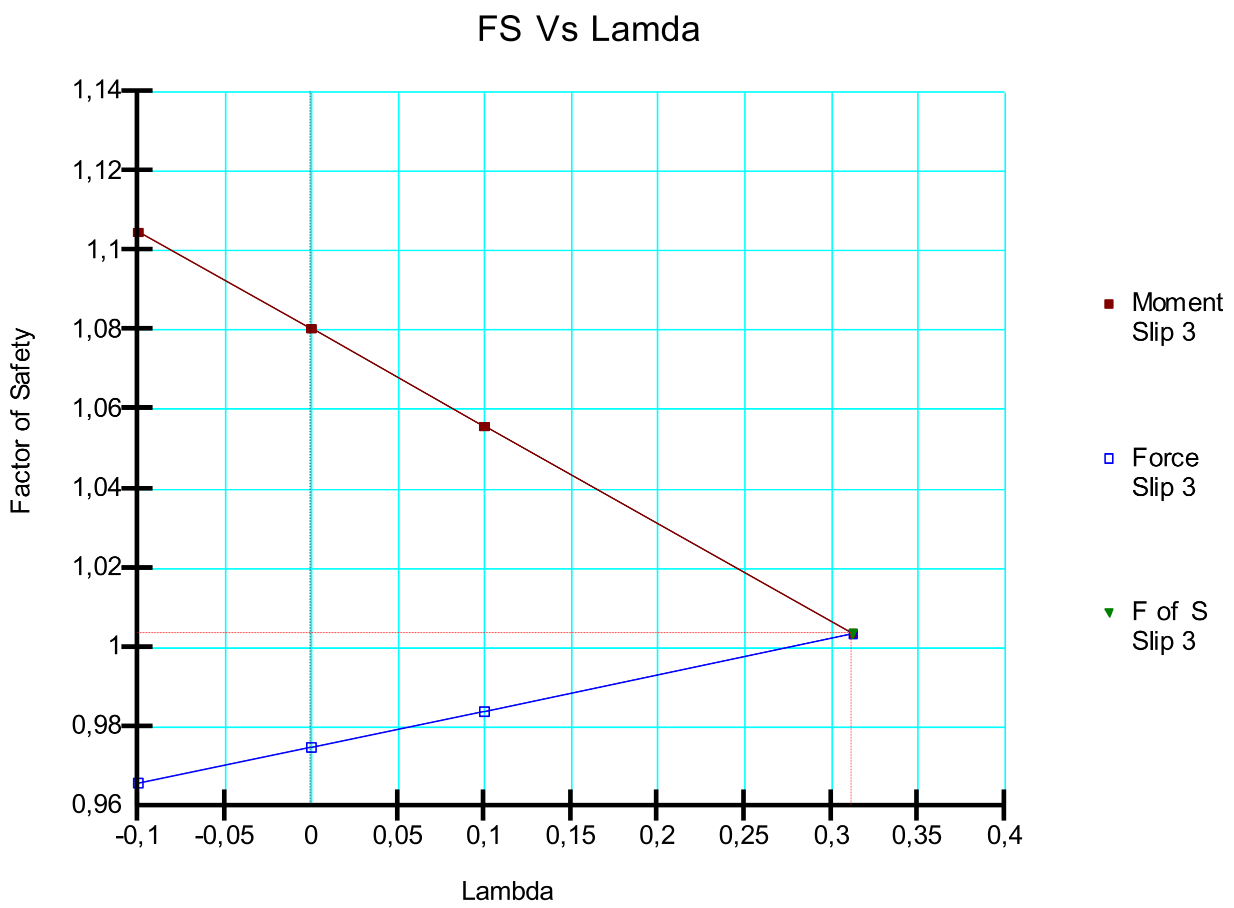


Figure 3.9. Factor of Safety vs. Lamda for the slip geometry of 2003 (GeoStudio 20121)

3.3.3. Simulation of landslide stabilisation interventions

Landslide stabilisation interventions on the Tsakona slope have the purpose of improving stability. The slope safety factor presented in the previous section analysis is not enough to avoid slope motion. So, thinking about the remedial works that could have been performed to prevent the large landslide of February 2003, it has been suggested; soil nailing and pile stabilisation are both coupled with a drainage system (horizontal drains).

3.3.3.1.Horizontal drain stabilisation simulation

Horizontal drains are simulated in SEEP/W with the use of a fairly horizontal line to which the material model property, ‘‘Interface’’ and a minimum permeability of 300 m/days (3.47e-03 m/s) is assigned as described in section 2.4.1. The SEEP/W pressure boundary conditions and SLOPE/W parameters are kept constant as described in section 3.3.2. The design (section 2.4.1.) of the horizontal drains was done according to Black et al. (2009). Horizontal drains are installed at small inclines of 5-10° to the horizontal so they can drain by gravity. Drains are installed in a fan-shaped arrays with radial spacings of 5-7° and placed at multiple elevation series of increment distance,15-20 m to the vertical.

3

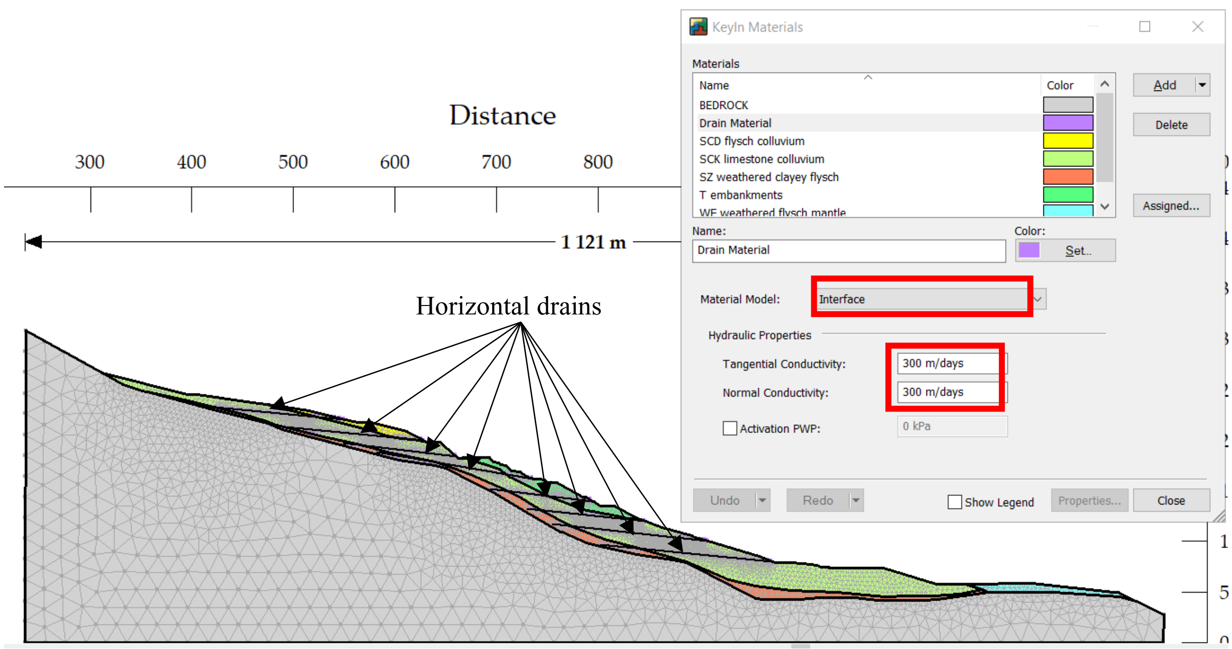


Figure 3.10. Horizontal drain design and input data (SEEP/W 2012)

The Horizontal drains are installed as shown in Figure 3.10 in a series of eight elevations of varying lengths from 100 m to 180 m (Table 3.2).

Table 3.2: Lengths of Horizontal drains for the Tsakona slope cross section.

Series number	1	2	3	4	5	6	7	8
Length(m)	130	138	112	157	102	118	154	172

The horizontal drain arrangement gives a new configuration to the pore water distribution with SEEP/W as shown in Figure 3.11. Horizontal drains make it possible to raise the factor of safety to 1.22 for the slip geometry in 2003 as shown in Figure 3.12. The FS equal to 1.14 shows that the drainage system is present to maintain the water table as low as possible over time but that only doesn't satisfy the precarious stability condition of the slope in 2001. Hence, need for a reinforcement technique to further increase the factor of safety of the slope.

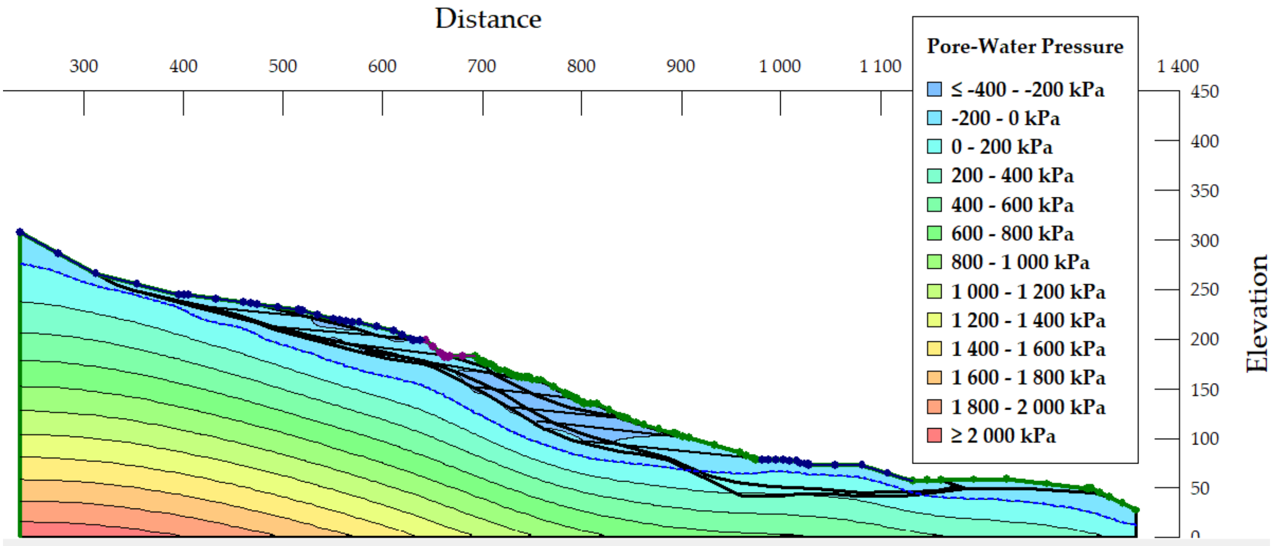


Figure 3.11. Pore water variation of the Peloponnese slope in 2003 (SEEP/W 2012)

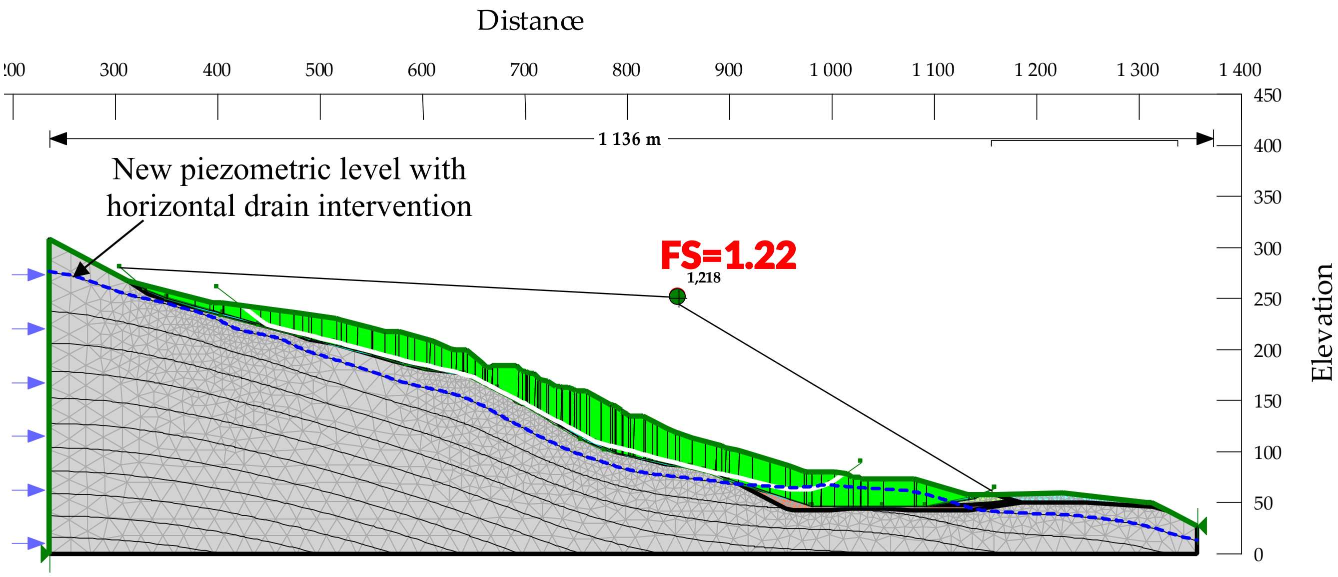
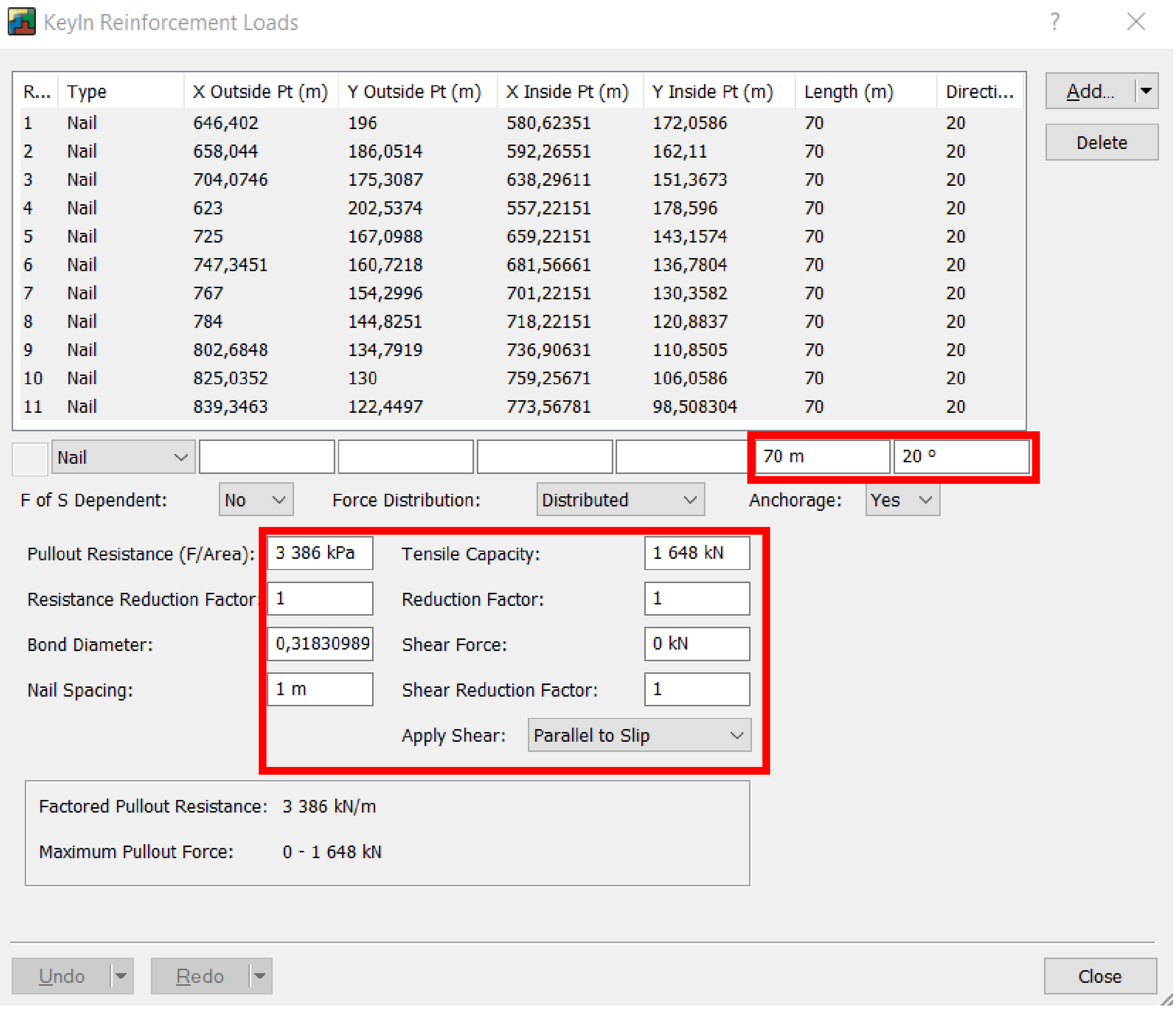


Figure 3.12. Stability analysis with horizontal drains (SLOPE/W 2012)

3.3.3.2.Soil nail stabilisation simulation (Sirive® special composite self-drilling bars)

To the stabilisation method proposed in section 3.3.3.1., soil nails could be coupled to it with the design methodology described in section 2.4.2. The SEEP/W pressure boundary conditions and SLOPE/W parameters are kept constant as described in section 3.3.2. For this analysis both the ''Fully Specified'' slip surface and ''Entry and Exit'' were used. Soil nail with Sirive® S90 Special self-drilling composite bars with 8 strands were displaced on the slope geometry according to section 2.4.2 as shown in Figure 3.14. The Input data assigned to each bar of soil nail is shown in Figure 3.13.

The stabilisation consisted of distributing the bars along the slope close to the National Road into eleven (11) rows of anchors with inclination 20°, 1 m spacing, and 70 m length. This choice is done to reach the stable bedrock, provide a satisfactory safety factor using the installation recommendation from Dalla Gassa s.r.l. company.



R...	Type	X Outside Pt (m)	Y Outside Pt (m)	X Inside Pt (m)	Y Inside Pt (m)	Length (m)	Directi...
1	Nail	646,402	196	580,62351	172,0586	70	20
2	Nail	658,044	186,0514	592,26551	162,11	70	20
3	Nail	704,0746	175,3087	638,29611	151,3673	70	20
4	Nail	623	202,5374	557,22151	178,596	70	20
5	Nail	725	167,0988	659,22151	143,1574	70	20
6	Nail	747,3451	160,7218	681,56661	136,7804	70	20
7	Nail	767	154,2996	701,22151	130,3582	70	20
8	Nail	784	144,8251	718,22151	120,8837	70	20
9	Nail	802,6848	134,7919	736,90631	110,8505	70	20
10	Nail	825,0352	130	759,25671	106,0586	70	20
11	Nail	839,3463	122,4497	773,56781	98,508304	70	20

Figure 3.13. Input parameters for soil nailing simulation (SLOPE/W 2012)

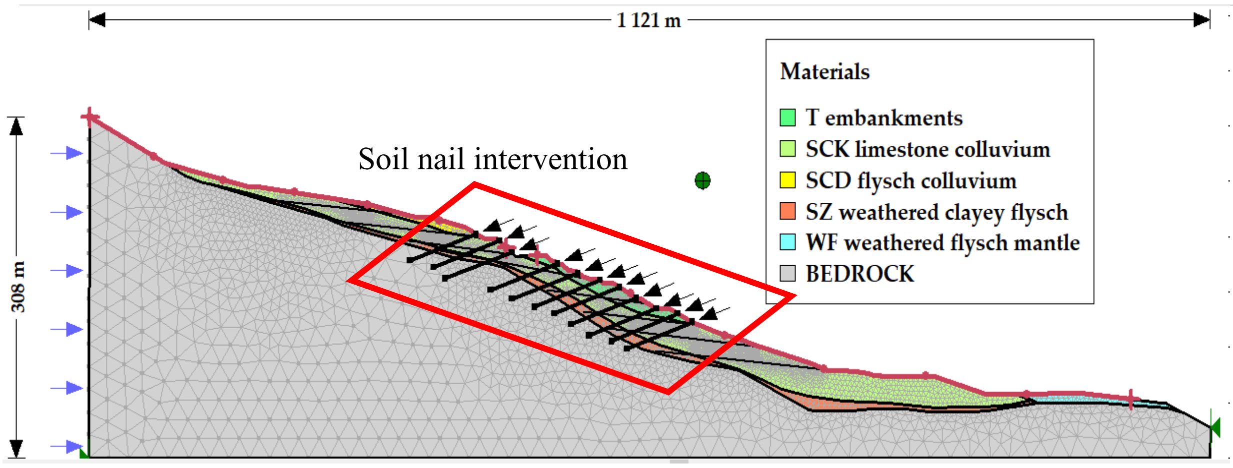


Figure 3.14. Soil nail design (SLOPE/W 2012)

In SLOPE/W, using the "Fully Specified" slip surface the soil nails arrangement makes it possible to raise the factor of safety to 1.42 for the slip geometry in 2003 as shown in Figure 3.15. The "Entry and Exit" specification was used to simulate any other slip surface that could be formed differently from the slip geometry known in 2001 and 2003. The results of this analysis gave a critical factor of safety equal to 1.33 as shown in Figure 3.16. According to Cornforth (2005), the various safety factors obtained for different conditions with soil nailing are satisfactory since they lie within the range of 1.20 to 1.50 (Annex 6).

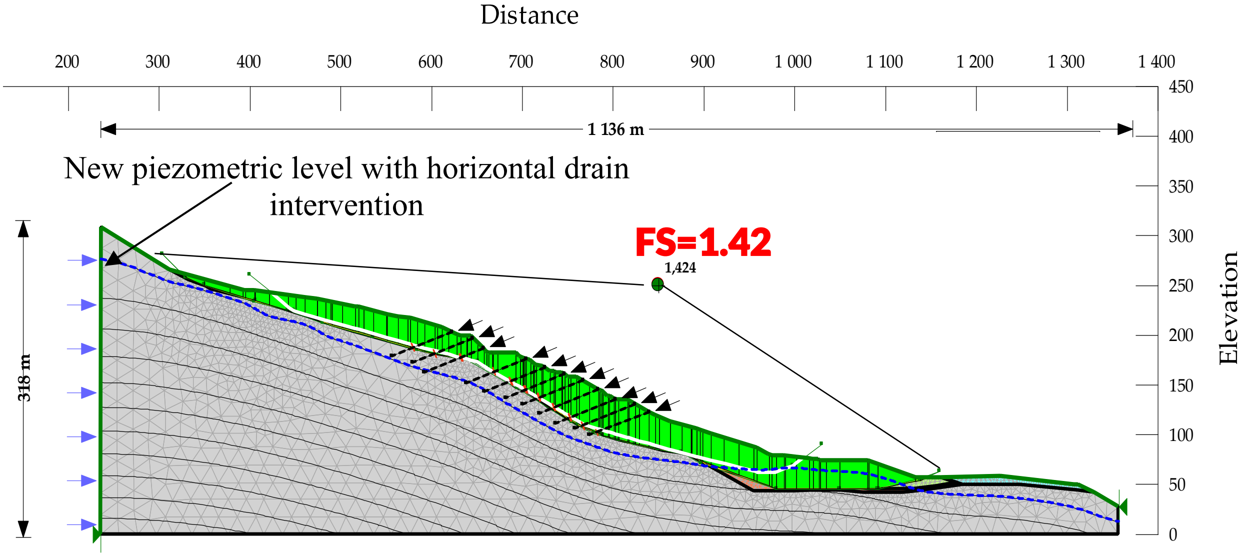


Figure 3.15. Stability analysis with soil nail using the "Fully Specified" slip surface (SLOPE/W 2012)

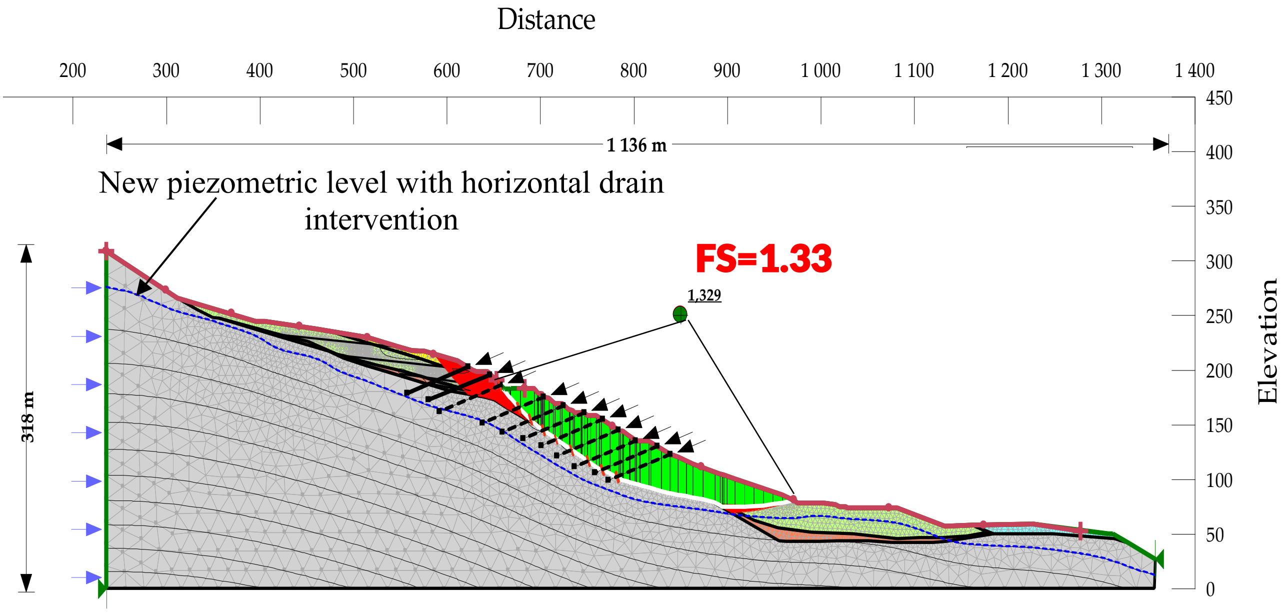


Figure 3.16. Stability analysis with soil nail using the "Entry and Exit" specification (SLOPE/W 2012)

3.3.3.3.Pile stabilisation simulation

Pile was modelled in SLOPE/W by entering the calculated limit resistance (shear resistance) of the pile; spacing between piles and length of the pile. The pile location is below the National Road as shown in Figure 3.18. The SEEP/W pressure boundary conditions and SLOPE/W parameters are kept constant as described in section 3.3.3.1. For this analysis both the ‘‘Fully Specified’’ slip surface and ‘‘Entry and Exit’’ were used. From the result report obtained in the analysis in section 3.3.3.1, the values of the resisting forces (F_R) and activating forces (F_D) were available. Using equation (2.15) and fixing the factor of safety (FS) as 1.5 we have the limit resistance of pile (ΔF_R) to be:

$\Delta F_R = FS * \sum F_D - \sum F_R$, with $\sum F_D = 99\,521.493$ kN,; $\sum F_R = 121\,251.15$ kN

$\Delta F_R = 28\,031.09 \text{ kN} \approx 30\,000 \text{ kN}$

For the case study, a single pile row is used of 2m spacing placed with an optimum depth of 60m since our slip depth is approximately 30m. According to Poulos (1995), the embedment depth has been suggested to be 0,5 the ratio sliding depth/ pile length which means at least half of the pile must be placed in the stable zone. The parameters used to model the pile in GeoStudio are shown in Figure 3.17.

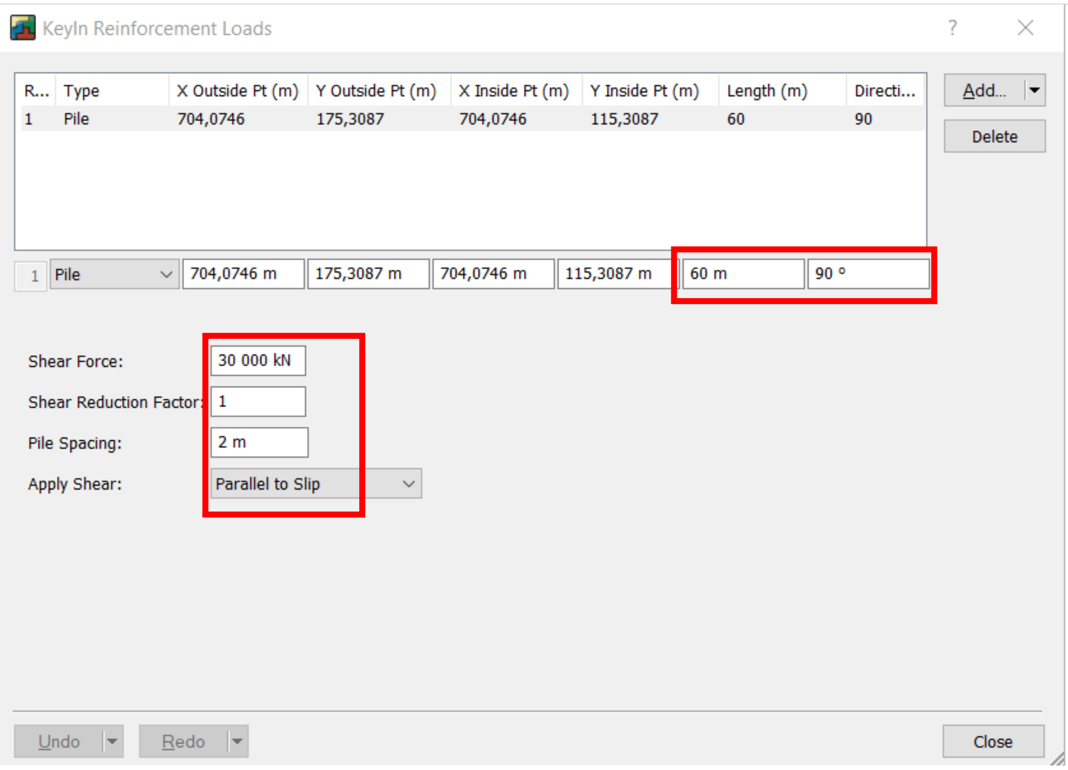


Figure 3.17. Input parameters for pile stabilisation (SLOPE/W 2012)

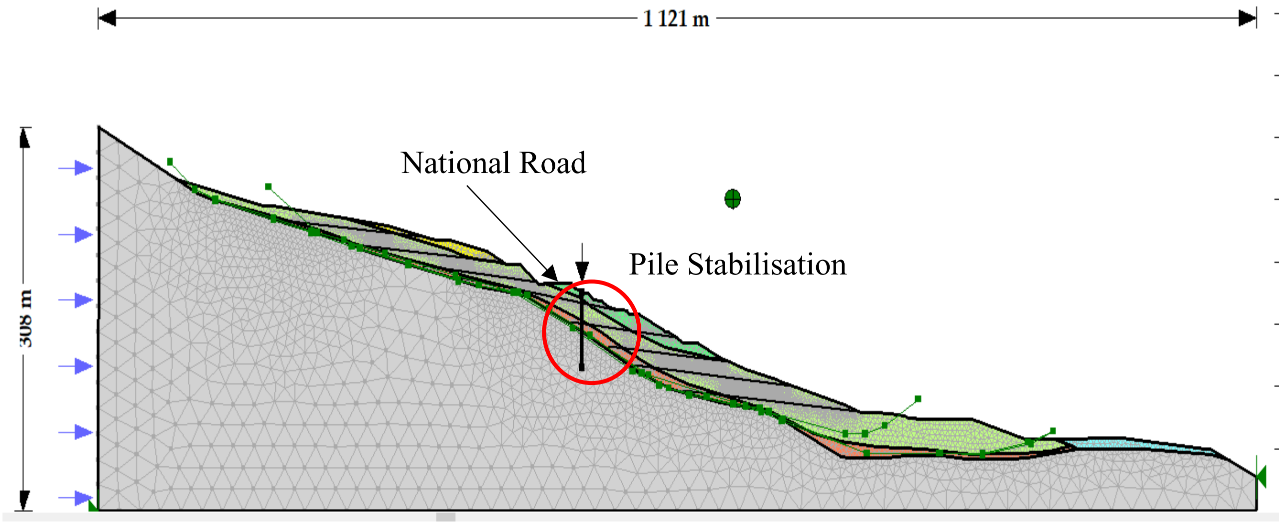


Figure 3.18. Pile stabilisation below the National Road (SLOPE/W 2012)

In SLOPE/W, using the "Fully Specified" slip surface the pile arrangement makes it possible to raise the factor of safety to 1.41 for the slip geometry in 2003 as shown in Figure 3.19. The "Entry and Exit" specification was used to simulate any other slip surface that could be formed differently from the slip geometry known in 2001 and 2003. The results of this analysis gave a critical factor of safety equal to 1.33 as shown in Figure 3.20. According to Cornforth (2005), the various safety factors obtained for different pile intervention analyses are satisfactory since they lie within the range of 1.20 to 1.50 (Annex 6).

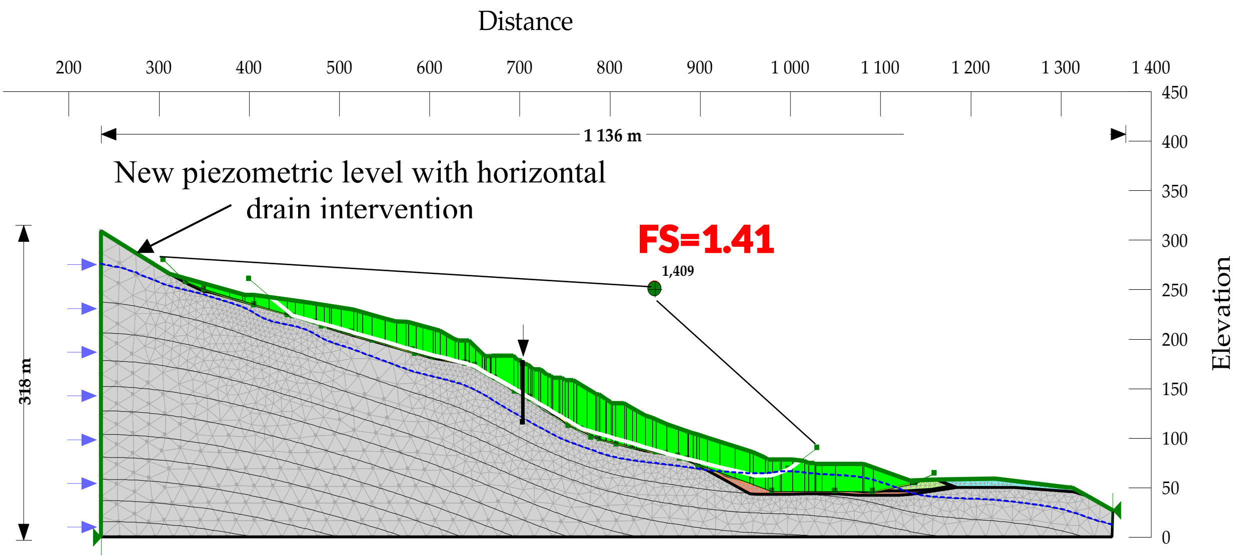


Figure 3.19. Analysis of pile stabilisation using the "Fully Specified" slip surface (SLOPE/W 2012)

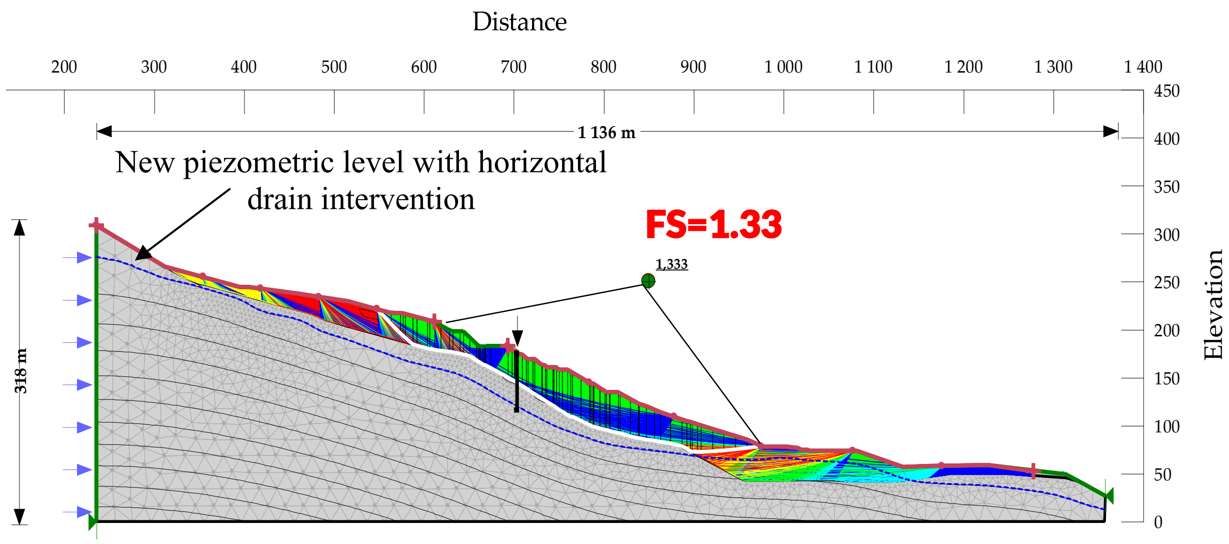


Figure 3.20. Analysis pile stabilisation using the "Entry and Exit" specification (SLOPE/W 2012)

Conclusion

In this chapter, the triggering factor that caused the Tsakona landslide in 2003 was modelled with numerical simulations and assessing a suitable constitutive model with the available data obtained through documentary research. The software used was GeoStudio, constituting the modules SEEP/W and SLOPE/W. After the application of the methodology, the soil parametric conditions that cause the landslide event were obtained. As an intervention for stabilisation, a drainage system (horizontal drain) was coupled either with soil nails or piles. The SLOPE/W simulation stabilities provided satisfactory safety factor results when either soil nails or piles were used, falling between 1.20 and 1.50 according to Cornforth (2005).

GENERAL CONCLUSION

The main objective of this thesis was to model the trigger of the the Tsakona landslide event that occurred in February 2003 with numerical simulations, exploring the effect of suction decrease on the soil shear strength and suggesting risk mitigation measures that could have prevented landslide occurrence.

To achieve this objective, firstly documentary research was made to bring out the generalities on soils, landslide and stabilisation techniques. Secondly, based on the data collected from the landslide event that occurred in February 2003, and a precedent event in 2001, numerical simulation to conduct the stability back analysis of the Tsakona landslide was carried out. The modelling was done with the software GeoStudio 2012 using the modules SEEP/W and SLOPE/W. The physical flow of water was simulated by SEEP/W through the finite element method meanwhile SLOPE/W access the safety factor of the slope with the limit equilibrium method. Finally, simulations of the risk mitigation measures (horizontal drains coupled with soil nails or horizontal drains coupled with piles for stabilisation) were done. The following conclusions were deduced from the analysis results:

- The extension of the Mohr-Coulomb Theory using Fredlund's model was the suitable constitutive model used to take into account the soil-suction dependence during the analyses.
- The residual angle value (17.2°) of both the weathered clayey flysch and limestone colluvium, obtained from the back analysis was in good agreement with the published data (between 16° and 20°) based on Lupini et al. (1981) with Stark & Eid (1994).
- The simple back analysis for the event in 2001 to obtain FS=1, shows that those mechanical properties of soil were already degraded during this event and not only for the second event in 2003.
- The back analysis before the landslide showed a higher safety factor for the slip geometry in 2003 (FS=1.17) compared to the slip geometry in 2001 (FS=1.00). This represents the precarious slope conditions in the year 2001.
- The increase in the water table had a destabilising effect on the Peloponnese slope, which made the safety factor to equal to 1.00 for the second slip surface geometry (2003) in the back analysis for the major event in 2003.
- To prevent the landslide in February 2003, horizontal drains coupled with soils nails (Sirive® Special composite self-drilling bars) offered a safety factor of 1.42. Also, horizontal drains coupled with piles offered a safety factor of 1.41. Horizontal drains must be present to maintain the water table as low as possible and reduce suction dissipation

meanwhile the reinforcement (soil nail or pile) is present to reduce displacement and velocity of the soil movement.
- Following the Cornforth (2005) recommendations, the various safety factors obtained for horizontal drain, soil nail and pile intervention analyses are satisfactory since they lie within the range of 1.20 to 1.50.

These numerical simulations studied, are vast in modern geotechnical engineering. It was necessary to limit the field of work. However, this work could not be, without imperfections, due to either failure to carry out certain tests or failure to take into consideration certain factors. To improve this work, the following perspective could be followed:

- Laboratory and field studies can be pursued to collect extra data on the rainfall intensity of the region and observations of the past earth movements on a larger scale than in the area immediately affected, with proper geotechnical investigations and monitoring to minimize uncertainties.
- Performing a detailed back analysis using a 2D finite element method (Strength Reduction Method) with other commercial software such as MIDAS GTS NX, and PLAXIS for a comparative study with the values reported in this thesis to understand the effects of suction dissipation on slope stability. Then eventually using 3D FEM to better understand the event depending on the availability of data.
- Evaluating the feasibility of the application of each proposed landslide mitigation technology with a comparative analysis (including the cost estimate) in the specific slope of Peloponnese.
- Using the stability results reported in this thesis to contribute a comprehensive database for the back-propagation neural network method (Artificial Neural Network model), combined with the fuzzy set theory for evaluating the stability/susceptibility of slopes according to Ni et al. (1996). Results of several slope sections of the same Peloponnese slope will be adequate for the training of the Artificial Neural Network.

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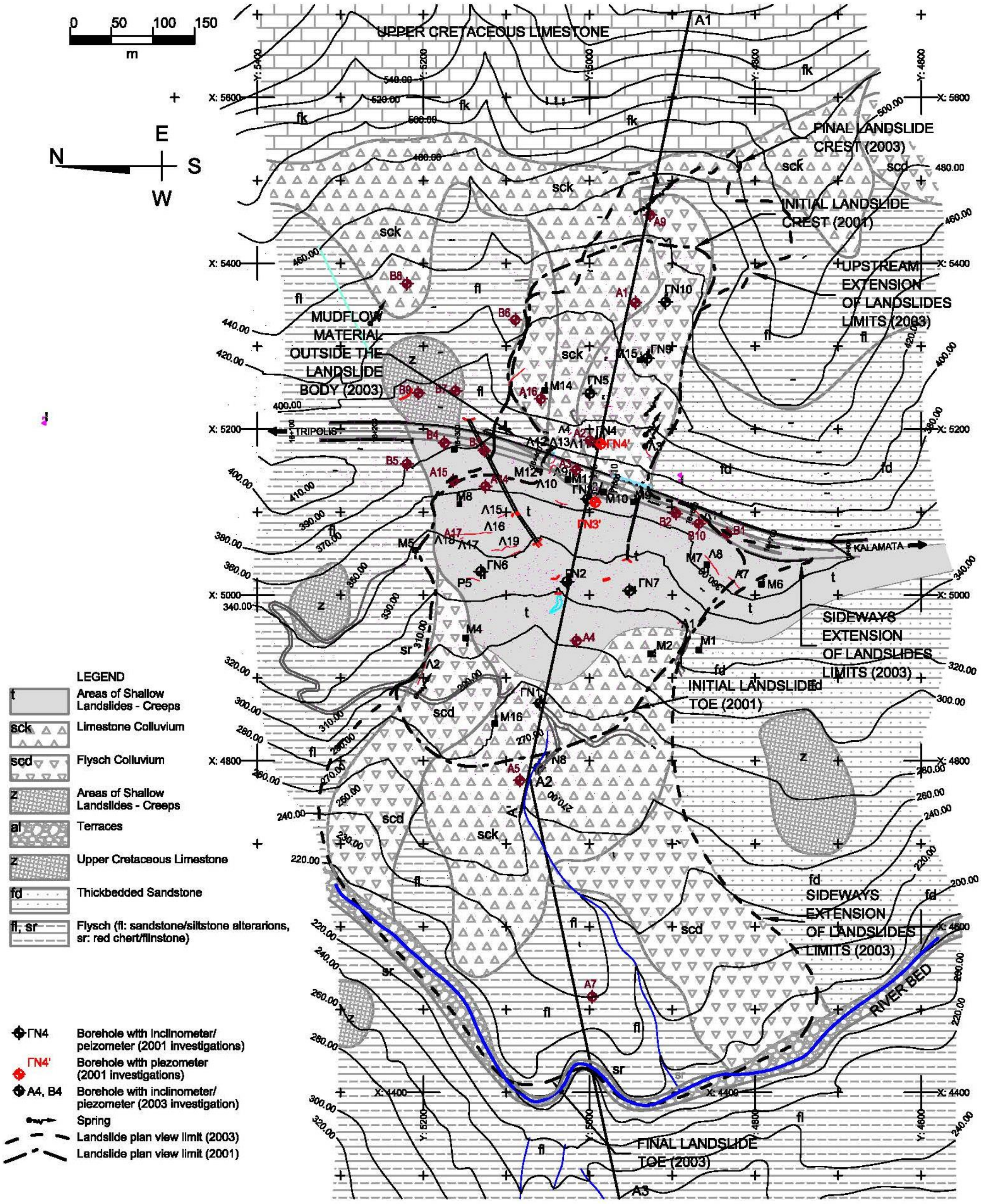
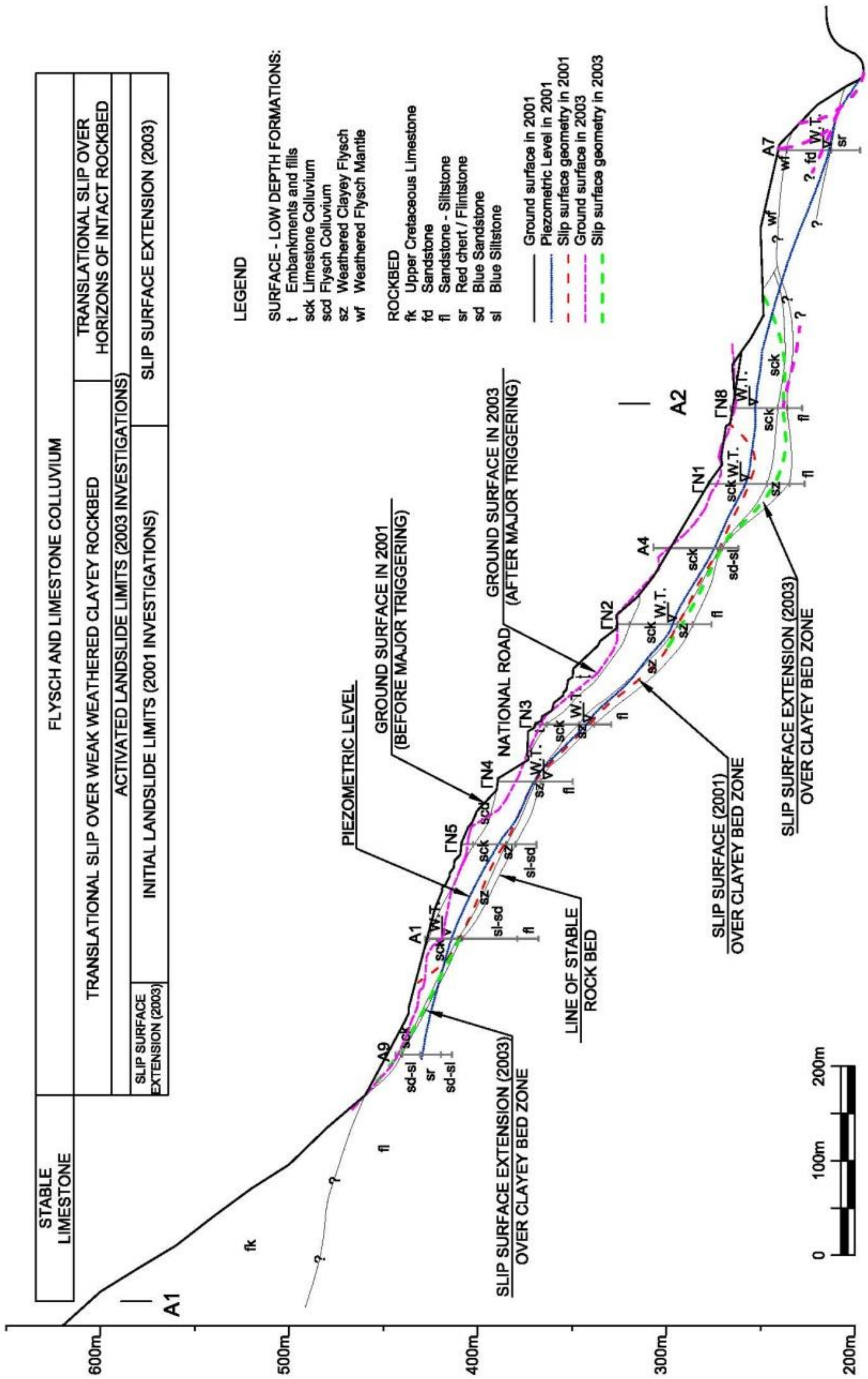
ANNEXES

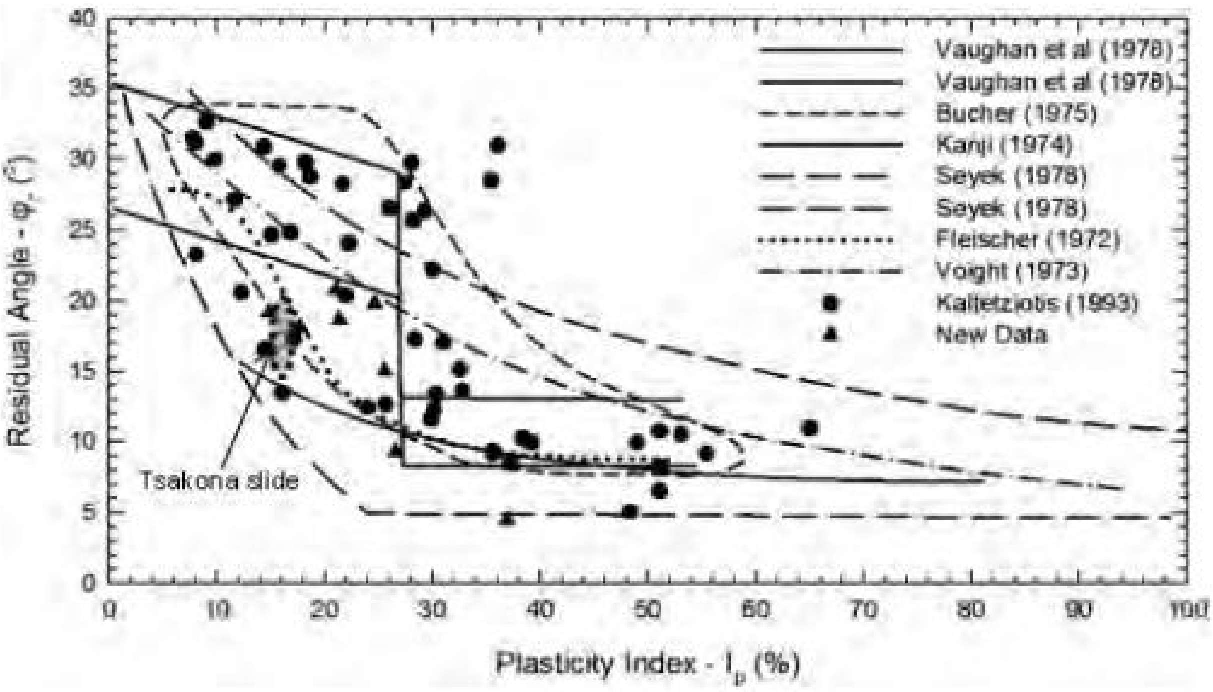
Annex 1: General plan view with surface displacements and landslide limit in 2001 and 2003. Modified from Belokas et al. (2013).

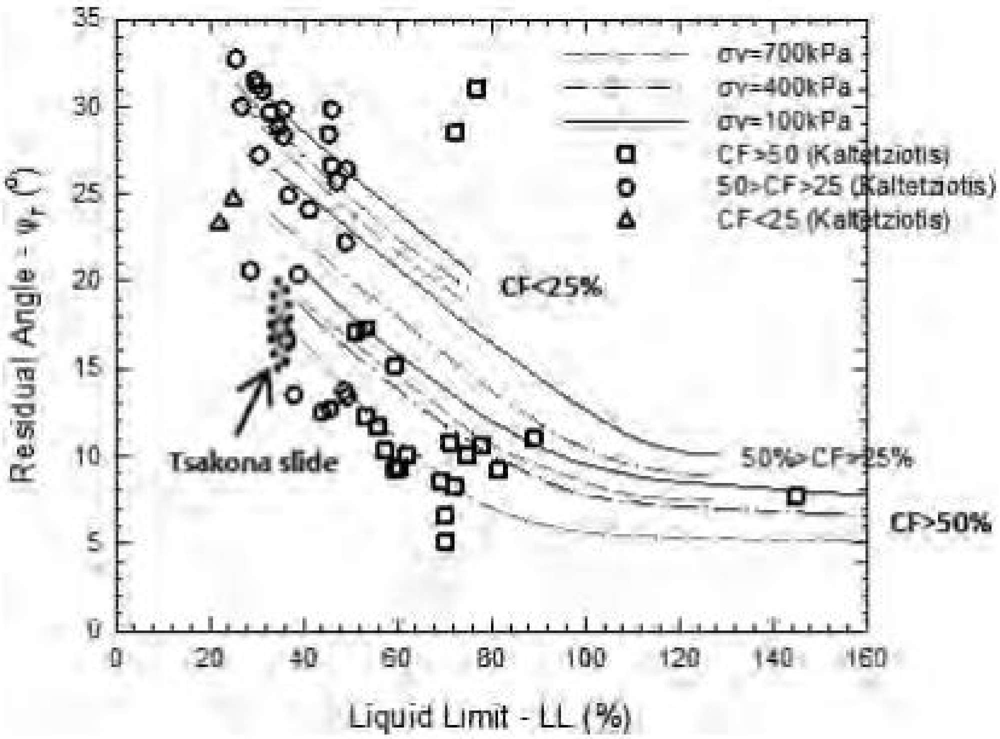


Annex 2: Cross section showing the two part slip mechanism in 2001 and 2003 along the assumed landslide axis. According to Dounias et al. (2006) and Belokas et al. (2013).



Annex 3: Comparison of Tsakona landslide residual strength with previously published data based on: a) Lupini et al. (1981) and b) Stark & Eid (1994).





Annex 4: Remediation Methods for slope Stability Problems (Turner & Schuster, 1996).

Category	Procedures	Limitations
Do nothing	"No action"	Non effective if long term catastrophic failure Applicable to slow moving or stable accelerating slopes
Avoid the problem	Relocate project	Should be studied during planning Cost impact regarding location selected No cost effective if design is complete
Maintenance	Activities such as removing materials, closing areas, etc.	Limited to slow moving landslides Will be temporary then it will need to be re-done
Monitoring	Inclinometers, surface survey points, etc.	Limited to slow moving landslides
Decrease driving forces	Change the grade	Could affect sections of adjacent roadways Not always available physical space for change in geometry
	Drain surface	Will only correct surface infiltration or seepage due to the surface infiltration
	Drain subsurface	Will depends on permeability of the sliding mass
	Reduce weight	Requires right-a-way and lightweight materials Produce excavation waste that need to be handle
Increase resisting forces	Use buttress or counterweight fills; toe berms	Could not be effective in deep-seated landslides Requires right-a-way Could require a firm foundation
	Use structural system	Could not handle large deformations Should penetrate well below sliding surface
	Install anchors	Requires a firm foundation to resist the shear forces of by the anchors tension
	Drain subsurface	Will depend on permeability of the sliding mass
	Use reinforced backfill	Requires durability of reinforcement
	Install in situ reinforcement	Requires long-term durability of nails, anchors, piles, micropiles
	Use biochemical stabilization	Limited to the height of the slope Affected by climatic conditions
	Treat chemically	Long-term effectiveness is still in evaluation Could be affected and affect the environment
	Use electrosmosis	Requires maintenance and constant direct current Could be expensive
	Use thermal stabilization	Require expensive and carefully designed system Could be expensive

Annex 5: Truncated conical floating plate for a Sirive® Special Composite self-drilling bars. Picture from Giovanni B. (2020).

Annex 6: Suggested Guidlines for Factor of Safety in Landslide studies to level of information and Landslide size. From Cornforth (2005)

Landslide Size	Minimum Study*		Normal Study*	
	Borings	Calculated Minimum F	Borings	Calculated Minimum F
Very small	1**	1.50	1	1.50
Small	1	1.50	2	1.35
Medium	2	1.40	4	1.25
Large	3	1.30	6	1.20
Very large	4	1.20	8	1.15

*see text
**occasionally zero, depending on circumstances

Annex 7: Suggested Pile Optimum Location

Method/Paper	Pile location	Model characterization
Ito et al. (1979)	Middle	Pressure based method/ Uncoupled
Poulos (1995)	Middle	Displacement based/ Uncoupled
Hassiotis et al. (1997)	Upper Top	Finite Difference Method/Pressure based/ Uncoupled
Lee et al. (1995)	Toe and Crest	Boundary Element/ Displacement based/ Uncoupled
Cai and Ugai (2000)	Middle/B and Top/UB	3D Shear Reduction Finite Element Method/Coupled analysis
Ausilio et al. (2001)	Toe	Kinematic approach limit analysis/ Uncoupled
Jeong et al. (2003)	Middle/Toe	Displacement based and Strength Reduction Finite Element Method/ Uncoupled and Coupled
Nian et al. (2008)	Toe	ABAQUS Finite Element Method/ Uncoupled
Chen and Martin (2002)	-	Finite difference program FLAC /Coupled
Pradel et al. (2010)	-	Strength Reduction Method/Coupled

Annex 8: Engineering properties of Unified Soil Classes. Part 631 National Engineering Handbook (2012).

Typical names	Requirements for seepage control						Unified soil classes
	Shear strength	Compress-iblity	Workability as construction material	Permeability			
				When compacted	K cm/s	K ft/d	
Well-graded gravels, gravel-sand mixtures, little or no fines	Excellent	Negligible	Excellent	Pervious	$K > 10^{-2}$	$K > 30$	**GW**
Well-graded gravels, gravel-sand mixtures, little or no fines	Good	Negligible	Good	Very pervious	$K > 10^{-2}$	$K > 30$	**GP**
Silty gravels, gravel-sand-silt mixtures	Good to fair	Negligible	Good	Semi-pervious to impervious	$K = 10^{-3}$ to 10^{-6}	$K = 3$ to 3×10^{-3}	**GM**
Clayey gravels, gravel-sand-clay mixtures	Good	Very low	Good	Impervious	$K = 10^{-3}$ to 10^{-6}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	**GC**
Well-graded sands, gravelly sands, little or no fines	Excellent	Negligible	Excellent	Pervious	$K > 10^{-3}$	$K > 3$	**SW**
Poorly graded sands, gravelly sands, little or no fines	Good	Very low	Fair	Pervious	$K > 10^{-3}$	$K > 3$	**SP**
Silty sands, sands silt mixtures	Good to fair	Low	Fair	Semi-pervious to impervious	$K = 10^{-3}$ to 10^{-6}	$K = 3$ to 3×10^{-3}	**SM**
Clayey sands, sand-silt mixtures	Good to fair	Low	Good	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	**SC**
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	Fair	Medium to high	Fair	Semi-pervious to impervious	$K = 10^{-3}$ to 10^{-6}	$K = 3$ to 3×10^{-3}	**ML**
Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, and lean clays	Fair	Medium	Good to fair	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	**CL**
Organic silts and organic silty clays of low plasticity	Poor	Medium	Fair	Semi-pervious to impervious	$K = 10^{-4}$ to 10^{-6}	$K = 3 \times 10^{-1}$ to 3×10^{-3}	**OL**
Inorganic silts, micaceous or diatomaceous fine sandy or silty soil, elastic silts	Fair to poor	High	Poor	Semi-pervious to impervious	$K = 10^{-4}$ to 10^{-6}	$K = 3 \times 10^{-1}$ to 3×10^{-3}	**MH**
Inorganic clays of high plasticity, fat clays	Poor	High very high	Poor	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	**CH**
Organic clays of medium to high plasticity, organic silts	Poor	High	Poor	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	**OH**
Peat and other highly organic soils	Not suitable for construction						**PT**

Annex 9: Sirive® Special Composite Bars datasheet.

SIRIVE®
self-drilling anchor
Barre cave autoperforanti qualificate DM 17/01/2018

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Prodotto Made in Italy

BARRE AUTOPERFORANTI COMPOSITE SIRIVE® SPECIAL

SIRIVE® SPECIAL COMPOSITE SELF-DRILLING BARS

Configurazione passiva / Passive configuration

Denominazione barra *Designation*	Barra autoperforante *Self-drilling bar*		Trefoli *Tendons*			Barra composita *Composite bar*		
	Snerv. (A=2,3‰) *Proof* [kN]	Rottura (A>5%) *Ultimate* [kN]	0,5" 12,5 mm n.	0,6" 15,2 mm n.	0,7" 17,8 mm n.	Snerv. *Proof* [kN]	Rottura *Ultimate* [kN]	Precarica *Max preload* [kN]
R32LL Special 1 tr. 0,6"	198	241		1		262	488	-
R38L Special 1 tr. 0,6"	274	333		1		338	575	-
R38L Special 1 tr. 0,7"	274	333			1	361	668	-
R38 Special 1 tr. 0,5"	368	448	1			411	596	-
R51 Special 1 tr. 0,6"	538	655		1		602	878	-
R51 Special 1 tr. 0,7"	538	655			1	625	971	-
S60 Special 3 tr. 0,6"	722	879		3		914	1610	-
S76 Special 4 tr. 0,6"	952	1159		4		1208	2134	-
S76 Special 5 tr. 0,6"	952	1159		5		1272	2394	-
S76 Special 6 tr. 0,6"	952	1159		6		1336	2654	-
S76 Special 7 tr. 0,6"	952	1159		7		1400	2914	-
S90 Special 8 tr. 0,6"	1136	1383		8		1648	3386	-
S90 Special 10 tr. 0,6"	1136	1383		10		1775	3906	-

Nota: resistenze minime garantite
Note: minimum guaranteed strengths