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M.Sc. THESIS

RESEARCH AND EVALUATION OF LANDSLIDE POTENTIAL ON SLOPE OF AKCABURGAZ LOCALITY AT NORTH-WEST SECTION OF ISTANBUL

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FOREWORD

I have to express my sincere appreciation to my principal supervisor, Assist. Prof. Dr. Erdal Emre ÇEÇEN. Although he has always busy, he did not refrain his support, guidance and insight throughout the research.

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February 2019 Burak SEVİŞ

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbol	Explanation
b	: Width of slice
c	: Cohesion
C_c	: Compression index
C_r	: Recompression index
c'	: Effective stress strength parameters
d	: Diameter of Plate
e	: Void ratio (dimensionless)
E	: Young's modulus (kN/m², mN/m²)
E_{50} , E_{50}^{ref}	: Secant elastic modulus at 50% peak strength and 50% secant modulus at reference pressure, P^{ref}
E_{oed} , E_{oed}^{ref}	: Oedometer modulus and oedometer modulus at reference pressure, P^{ref}
E_{ur} , E_{ur}^{ref}	: Unloading/reloading modulus and unloading/reloading modulus at reference pressure, P^{ref}
$E_{ur,oed}$, $E_{ur,oed}^{ref}$	: Unloading/reloading oedmeter modulus and unloading/reloading modulus at reference pressure, P^{ref}
FOS	: Factor of Safety
F_c , F_φ	: The factor of safety for effective stresses
G	: Shear modulus
G_s	: Specific gravity
h	: Average height of slice
I_I	: Liquidity index
I_p	: Plasticity index
k	: Modulus number
K	: Bulk modulus
K_v	: Vertical seismic coeeficient
K_h	: Horizontal seismic coeeficient
K_0	: Coefficient of earth pressure at rest
K_0^{nc}	: Coefficient of earth pressure at rest for normally consolidation, soil
M	: Slope of critical state line

$\boldsymbol{N}$	: Stability ratio
$\boldsymbol{N'}$	: Effective normal force
$\boldsymbol{N_p}$	: Pressuremeter constant
$\boldsymbol{P^{ref}}$	: Reference pressure
$\boldsymbol{S_a}$	: Available strength
$\boldsymbol{S_m}$	: Mobilized strength
$\boldsymbol{s_u}$	: Undrained shear strength of the soil (kN/ m²)
$\boldsymbol{u}$	: Water pressure (kN/ m²)
$\boldsymbol{U_a}$	: Pore water force
$\boldsymbol{U_\beta}$	: Surface water force
$\boldsymbol{V_s}$	: Shear wave velocity (m/s)
Q	: External surcharge
$\boldsymbol{W}$	: Weight of slice
$\boldsymbol{w, w_n}$	: Water content (%)
$\boldsymbol{w_l}$	: Liquid limit (%)
$\boldsymbol{w_p}$	: Plastic limit (%)
$\boldsymbol{Z_L}$	: Left interslice force
$\boldsymbol{Z_R}$	: Right interslice force
$\boldsymbol{\delta}$	: Inclination of surcharge
$\boldsymbol{\varepsilon}$	: Uniform Radius ground loss (mm)
$\boldsymbol{\varepsilon_0}$	: Equivalent ground loss (mm)
$\boldsymbol{\varepsilon_s}$	: Shear strain (dimensionless)
$\emptyset$	: Angle of internal friction (degree)
$\boldsymbol{\phi'}$	: Effective stress angle of frction degree
$\boldsymbol{\theta_L}$	: Left interslice force angle
$\boldsymbol{\theta_R}$	: Right interslice force angle
$\boldsymbol{\gamma}$	: Unit weight of soil (kN/m³)
$\boldsymbol{K^*}$	: Modified swelling index (dimensionless)
$\boldsymbol{\lambda^*}$	: Modified compression index (dimensionless)
$\boldsymbol{v}$	: Poisson’s ratio (dimensionless)
$\boldsymbol{\rho}$	: Soil density (Mg/m³)
α	: Inclination of slice base
β	: Inclination of slice top
$\boldsymbol{\sigma_1', \sigma_3'}$	: Major, minor effective principal stress (kN/ m²)

$\boldsymbol{\sigma_h, \sigma_h{}'}$	: Total, effective horizontal stress (kN/ m²)
$\boldsymbol{\sigma_v, \sigma_v{}'}$	: Total, effective vertical stress (kN/ m²)
$\boldsymbol{\tau}$	: Shear stress (kN/ m²)
$\boldsymbol{\tau_{req}}$	: Required shear strength
$\boldsymbol{\tau_f}$	: Shear stress at failure (kN/ m²)
$\boldsymbol{\psi}$	: Dilatancy angle (degree)

Abbreviation	**Explanation**
CD	: Consolidated Drained
CU	: Consolidated Undrained
DS	: Direct Shear
EDM	: Electronic Distance Meter
FEM	: Finite Element Method
GP	: Poorly-graded Gravels
GPS	: Global Positioning System
GW	: Well-graded Gravels
HS	: Hardening Soils
IAEG	: International Association for Engineering Geology
InSAR	: Interferometric Synthetic Aperture Radar
MC	: Mohr Coloumb
NAS	: National Academy of Science
OC	: Over Consolidated
OTDR	: Optical Time Domain Reflectometry
RC	: Reinforced Concrete
SP	: Poorly-graded Sands
SW	: Well-graded Sands
TS	: Turkish Standarts
UU	: Unconsolidated Undrained
WLI	: World Lanslide Inventory

ÖZET

YÜKSEK LİSANS TEZİ

İSTANBUL KUZEY-BATI KESİMİ AKÇABURGAZ MEVKİİ YAMAÇLARININ HEYELAN POTANSİYELİNİN ARAŞTIRILMASI VE DEĞERLENDİRİLMESİ

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İnşaat ve Altyapı Mühendisliği Anabilim Dalı

Danışman : Dr. Öğr. Üyesi Erdal Emre ÇEÇEN

Heyelanlar hala dünyanın bir çok bölgesi için önemli bir afet problemidir. Aşırı yağışlar, yüzey ve yeraltı suları, deprem ve diğer titreşimler, eğimin fazla olması, zeminin killi olması ve tabakaların eğim yönünde uzaması gibi durumlar heyelan oluşumunu tetiklemektedir. Heyelanların olumsuz etkilerini azaltmak veya ortadan kaldırabilmek için bunların izlenmesi ve mekanizmalarının çözülmesi oldukça önemlidir. Bu çalışmada İstanbul Akçaburgaz semtinde bulunan heyelan potansiyelli bölgede gerçekleştirilen zemin hareketleri izleme çalışmaları ve laboratuar deneylerinden elde edilen sonuçlar kullanılarak yapılan tahminlerden bahsedilmiştir. Bu çalışmanın amacı, sayısal analiz yöntemlerini kullanarak sondaj yapılan bölgedeki heyelan potansiyelinin risk derecesini görebilmektir. Uzun süreli, kısa süreli ve depremli durum gibi üç farklı duruma göre zemin hareketlerini ölçebilmek için sonlu elemanlar metodunun pekleşen zemin modeli ve limit denge yöntemleri ile bilgisayarlı modelleme analizleri yapılmıştır. Son olarak zemin hareketlerini izlemek için yerleştirilen inklonometrelerden alınan okumalar ile analizlerin deplasman sonuçları karşılaştırılmasının yanı sıra hesaplanan Güvenlik Katsayısı TS8853 (Yamaç ve Şevlerin Dengesi ve Hesap metodları) Türk standartına göre kontrol edildi.

Şubat 2019, 90 sayfa.

Anahtar kelimeler: Heyelan, İnklonometre, Yamaç ve Şevlerin Dengesi, Sonlu Elemanlar Yöntemi, Limit Denge Yöntemi.

SUMMARY

M.Sc. THESIS

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Landslides is still a disaster problem for many regions of the world. Some situations trigger landslide formation, such as; Excessive precipitations, Surface water and groundwaters, earthquake and other vibrations, dramatically more inclination, clayley soil, Extending the inclined direction of layer. Monitoring of landslides and solving their mechanisms are important in order to reduce or eliminate their negative effects. In this study, studies of monitoring soil movements realized in a region having landslide potential located in Istanbul-Akcaburgaz vicinity are referred with the estimates made using the results obtained from lab experiments. The aim of this study is to estimate the degree of landslides risk in regions made drilling by using a numerical analysis methods. Computerized modellings were analysed to estimate the soil movements in three different situations as; long term, short term and with an earthquake, by means of hardening soil model of FEM and limit equilibrium methods. Finally, displacement results of analysis and readings received from inclonometre placed for monitoring movements were compared, as well as evaluated Factor of Safety was cheched according to the Turkish standard TS 8853 (Slope Stability and Evaluation methods).

.February 2019, 90 pages.

Keywords: Landslide, Inclonometre, Slope Stability, Limit Equilibrium Method, Finite Element Method.

1. INTRODUCTION

1. 1. BACKGROUND

Recently, the urbanization has been started to tend to hillside on the border of the city located in a potentially landslide regions due to population growth. To achieve this aim, the geotechnical engineers started to focus on exploration and monitoring studies in unfavorable areas. To constitute habitats in safe, soil movements should be under controlled to complete this process successfully. Hence the most important subject is that should be taken precautions against to landslide by used to the possible movement estimates during construction stages or even after construction. The fact remains that must be predicted how to effect buildings which are existing nearby the excavation. Since the movement of the ground depends on factors such as excessive precipitations, surface water and groundwaters, earthquake and other vibrations, dramatically more inclination, clayley soils.

Landslide is an exogenous geological process that can take place almost everywhere. There are many hills on the earth surface, some of which are stable and the others are or going to be unstable. Deformation of an unstable slope may be dynamic, which has the most hazardous after effects, though the quasistatic displacement of slopes can also be dangerous for local infrastructure. The investigation of landslide processes is a topical task either in construction or operation of various objects. Systematic monitoring of sliding hillsides allows timely forecasting and preventing failure of slopes, and keeping out from accidents and, thus, injuries and heavy expenditure (Zakharov et al., 2014).

Geological structure variety gives rise to many kinds of landsliding, e.g., shear sliding (shearing, shearing–consequent); outsqueezing sliding; viscoplastic sliding (flow, slipout, mudflow); hydrodynamic exportation (suffusion, hydrodynamic flowoff) (Zakharov et al., 2014).

Landslides can be classified based on the criterion of the potential sliding surface. This surface either exists before sliding starts, or it forms directly at the moment of sliding. In the latter case, the sliding surface geometry and location are unknown beforehand and should be determined while assessing stability of a slope considering its stress–strain state. Nearly all current stability estimation methods lack such procedure, which lowers their reliability (Zakharov et al., 2014).

As a rule, landslides do not start suddenly but develop in time. A sign of landsliding may be cracking of the ground surface and the associated events (road breakage, displacement and failure of surface objects, etc.). Landslides reach maximum rate (up to few tens of kilometers per hour) in a certain time and then decelerate. Owing to gradual beginning of landsliding, it is possible to fix the process start and predict the process development. At present, depending on characteristics of a particular slope, there are many practical methods to determine landsliding, most of which are based on geodesy techniques (Zakharov et al., 2014). This study focuses on predicting the mass movemet by using a numerical solution with finite element method.

Nowadays, the use of numerical methods to analyze slope stability is becoming wider spread. Nevertheless, any estimation related to this is strongly hinge on the model applied for modelling the soil characteristic. "A landsliding process may be divided into three phases: a slow preliminary phase gradually changes into a dynamic phase and then, to a quasistatic phase. In the dynamic phase, the actual sliding surface, either existing or newly formed, shows itself, and downward displacement takes place over this surface. During the dynamic phase, landsliding can cover area hundreds of meters in size. The characteristics of the interface may change greatly, as a consequence of which displacement velocity goes to zero. The quasistatic landsliding phase is associated with viscoplastic displacement of the landslide body, that is much slower (up to a few meters a year)" (Zakharov et al., 2014).

1. 2. PURPOSE OF STUDY

Landslide which occur slope terrains can be considered as a major issue for the geotechnical engineers. For that ignoring this estimation could lead to sever results that could buildings and roads etc. surrounding it. This study focuses on estimating the magnitude of the ground movement that is generated by in three different phases (long term, short term, with earthquake). A finite element model is used assess the slope stability and the level of landslide risks is according to Factor of Safety based on TS8853.

1. 3. SCOPE OF STUDY

Clayey soils can be considered as the most critical type of soil which is more vulnerable to produce sliding by via earthquake and other vibrations. The best way to avoid damage of landslide is analyse their mechanisms and monitoring soil movements. This research depends on comparison between measures taken from 2 different inclinometer placed as SK-1 and SK-2, and analysis of modelling. Nevertheless, to estimate the soil movements, used software programs called PLAXIS 8.2 and SLIDE 5.0 with using different parameters obtained by specific laboratory experiments.

2. LITERATURE REVIEW

2.1. GENERAL

Shifting urban settlements to landslide-bearing areas to reduce the intensity of urban centers is causing landslides that are devastating. Particularly, in some recent studies have emphasized that that losses of life are intensified in less developed countries. It means that settlements are built in areas that are not suitable for construction in the cause of financial interests and there is unfortunately little funds less than required for understanding the risks of hazards relevant to landslides. Moreover, In many developing countries where the transportation route is always on critical areas, have been building the roads and bridges in landslide prone areas regardless of its geological conditions due to huge cost of construction. Providing that this priority model continue to implementation in urban hillside areas, population who lives at those areas, be under constant disaster threats..

The first slope stability analyse was established by (Namdar, 2010). There are several varied factors for structure of stable slope and capacity of soils. Assessment of mixed soil technique is referred by the unclosed conclusion of exploration, analysis of computerized modeling, decreasing project expenditure, the probability of utilization proximate local material, accurate comprehension of soil characteristics and making a solution problem of geotechnical. When the force of soil cohesion reach a half of cohesion strength, the ultimate pore water pressure resulted. Only it could appear emprical methods at pre-analysis of any construction with slope.

Long and short term slope stability was handled by (Martirosyan and Proshin, 2002) in Vorob’ev Hills in Moscow. This study inferred from rheologic testing of soil samples in deformity and circular-shear devices to gather quantitative prediction of proportion of creep displacements. Furthermore one more evaluation of the stress strain condition of slope belonged a different model of a creeping slope, is submitted by the finite element method. A few zones the proportion of creep displacements attains to 1.5 cm/year, which corresponds to results of field observations.

The best way of managing and understanding slope instability phenomena is taking convenient landslide risk palliation measures. Decision makers and authorities who are liable for sectional landuse planning are in a sustained requirement for plans that may be ventured by landslides and indicate the zones. A typically landslide sensitivity map is prepared to represent for reveal this aim. According to referred study, landslide sensibility is the possibility of the positional happening of familiar failures of slope failures when paid regard of geoenvironmental situations (Guzzetti et al. , 2005). A territory have a potential to be effected by slope movements, that is admissible area where landslides are probably to happen in the future.

Thoroughout the last ten years, the problems of landslides risk and susceptibility have been come up according to the scale of analysis as well as the purpose of research. The basics for landslides risk analysis were established by Varnes in (1984). In his study, he elucidated how it is probable to specify fields where a potential for landslide exists by using the uniformitarian principles, which expressed that the previous and existing experiences ease the understanding of slope failures in the future. These could be more presumably to become under the same circumstances in consideration of familiar state of instability. Overall informations of exploration in the issue of landslides susceptibility can be found in the studies of Fell et al., (2008), Van Westen et al., (2006), Guzzetti et al., (1999), Carrara et al., (1999), Dai et al., (2002), Aleotti and Chowdhury (1999) and Soeters and van Westen (1996).

2.2. CLASSIFICATION OF MASS MOVEMENTS

Fully completed slope movement processes are classified and defined according to characteristics that are also some rating related to evasion, recognition, correction or control. The classification contains exceedingly slow distributed movements of both soil and rock (described as creep in many classifications). Day by day increasingly known toppling failures or overturning are included in the classification. Caution is also given to monitoring movements caused by thawing and freezing. Figured among the properties that have been used as criteria for classification and identification are the level of degradation of the displaced mass, the speed of movement, type of material, type of movement, the geometry of the failure area and the age of occurrence, connection or absence of the correlation of slide geometry to the geological structure, degree of improving, geographic location of type patterns, and state of activity. An argumentation is stated about of sliding slope movements with factors that conduce to enhanced or reduced stress and strength of shear. (Varnes, D J 1978)

2.2.1. Major Classifications

Sharpe in (1938) aggregated the form of landslides which are priorly addressed in many of the literature, into an individual and universal classification layout. Also it was the initial book about landslides in America (Fig. 2.1). He investigated kind of materials to utilize the secondary stage of the H_2O (from water to ice and on the contrary) for understanding type of movement and velocity. Towards the Second World War, Sharpe could not find opportunity to advance further on landslides (Shroder, 1998 a), but his pioneering theories handled later by other researchers.

Varnes in (1958) began his investigation about landslides based on seminal works of Sharpe (1938). First, he expressed the two type of material (engineering soils and rock) and three types of earth movement (falls, flows and slides). He also discriminated between planar and rotational slides alongside variety of saturated and unsaturated soils or deformation of the material (Shroder et al., 2005).

This classification was particularly accepted the reliable as well as its usefulness and plausible character. Because National Academy of Science (NAS) had published to contribute the improvement of the nation’s new Highway System (Varnes, 1958). Nonetheless, it had a some kind of troubles, such as; its deliberating lateral spreads as a type of motion, undesignation of speed and flow of rock fragments (Shroder et al., 2005).

In his classification updated in 1978 (Figure 2.2), Varnes answered to propositions of Shroder (1968, 1971, Dr. David J. Varnes, U. S. Geological Survey, 1975) and revised his designation of engineering soils to include earth and debris, which were freshly described rate of fine to coarse clastic sediments. Also, well known term "landslide" was forestalled and instead of this "slope failure" term has called by him. Because this term was closer than its connotation of an incentive mechanism. Furthermore, they divide lateral spreads apart from included topples although they include joined rock flow or deep shrinkage to count in the complicated category, fragment of moved rock flows to the complicated category, and combine the recently acquainted sagging failure.

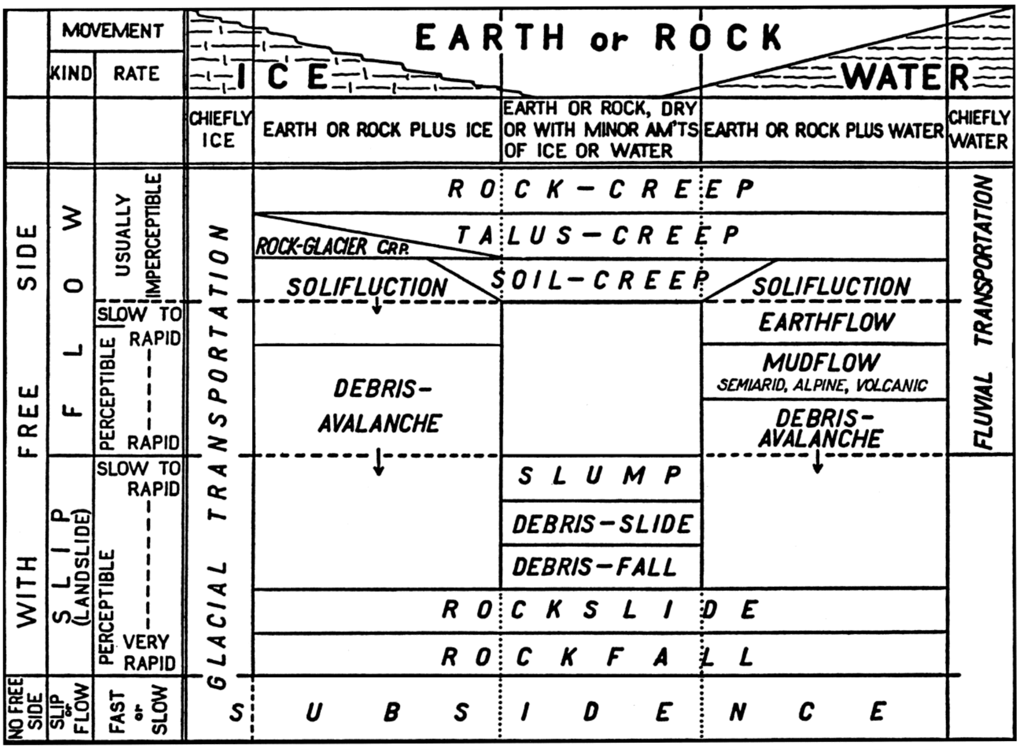


Figure 2.1: Classification of landslides and related phenomena on the primary basis of type of material and type of movement (Sharpe, 1938)

TYPE OF MOVEMENT			TYPE OF MATERIAL		
			BEDROCK	ENGINEERING SOILS	
				Predominantly coarse	Predominantly fine
FALLS			Rock fall	Debris fall	Earth fall
TOPPLES			Rock topple	Debris topple	Earth topple
SLIDES	ROTATIONAL	FEW UNITS	Rock slump	Debris slump	Earth slump
	TRANSLATIONAL		Rock back slide	Debris back slide	Earth back slide
		MANY UNITS	Rock slide	Debris slide	Earth slide
LATERAL SPREADS			Rock spread	Debris spread	Earth spread
FLOWS			Rock flow (deep creep)	Debris flow (soil creep)	Earth flow
COMPLEX	Combination of two or more principal types of movement				

Figure 2.2: Abbreviated and updated classification version of Varnes (1978). The expanded pictoral version includes illustrations of each type.

2.2.2. Terminology For Type Of Material

Three material types are utilizied by most classifications: earth, rock and debris. Debris has the remainder <2 mm and % 20 to % 80 of fragments >2 mm in size. Also debris includes other material that can become added into landslides, such as vehicles, buildings and vegetation. Rock is described as rock material or bedrock with at least % 80 of fragments > 2 mm in size. Earth is a material equal with % 80 or more of fragments <2 mm in size (Shroder, 1971). Although, the initial term soil is still utilized instead of earth, maybe externalizing a weakly approach to such a material terminology (Dikau et al.,1996).

2.2.3. Terminology For Types Of Movement

The kinds of movement defined are based initially on the study of Cruden and Varnes, (1996) and Dikau et al., (1996). Some convergency between european and american systems of nomenclature is fixed, although they are not completely harmonious. The main kinds of mass motion have accepted such as; flows, spreads, slides, topples and falls (Figure 2.3). Not included are periglacial phenomena, talus, subsidence or creep (Shroder et al., 2005).

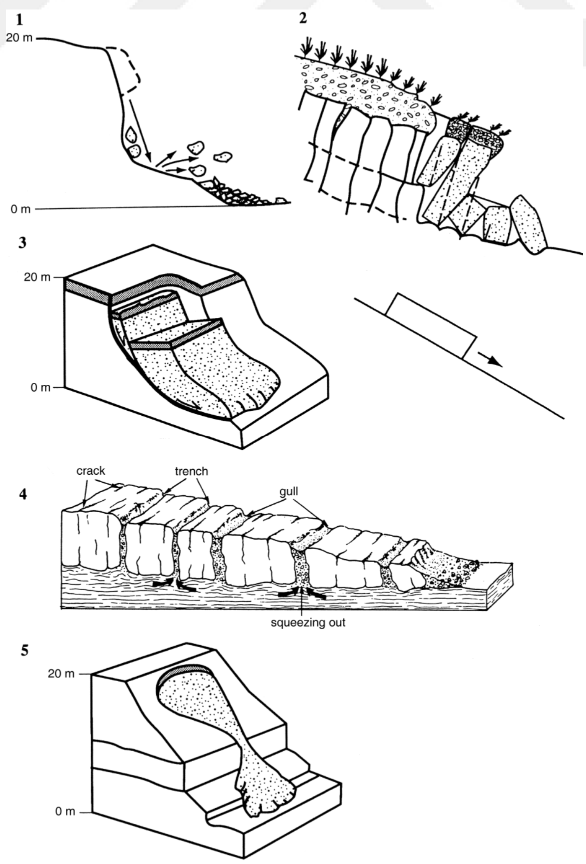


Figure 2.3: The five basic types of movement (Dikau et al.,1996): 1- Fall, 2- Topple, 3- Slide (translational block glide and rotational block slump), 4- Spread and 5- Flow.

Motions consist in a few main directions. The simplest of them are slides or near perpendicular falls. Prior slope failures move vertically downward, this means that what is called the main scarp steep. Beneath a perpendicular fall, constituent of gravity and non-uniform slopes can induce laminar or more linear movement or sliding or bouncing. Translational, planar, laminar or linear movement become in a approximately smooth line from higher to lower altitude, such as; block glides or slides over a shear or slip (rupture) surface. Rotational motions can be either backward or forward and it is also feasible for the speed up of moving masses to carry them up slope for some interval (Shroder et al., 2005).

Mainly reason of mass movement has long been referred as moisture in the literature (Shroder, 1971). According to Varnes (1978), pore pressures and pore water do not indicate the visible wetness of a landslide, conversly: (1) too wet, where the soil material contains sufficient water to flow like a liquid at low altitude, (2) wet, which assist substantial system of stable water, has fluid water infer from or includes sufficient water therefore the soil is partially liquid, (3) humid; the material which includes some water without pore water, is a plastic solid, and (4) dry; the material having no visible humidity (Shroder et al., 2005).

Spreads are moves laterally over a plastic substrate, wet, unstable and settled or deforming mass. Spreads where sediment expands or upper cohesive rock and cause from the flow of the more tender material. In addition liquefaction and expulsion trigger it. The top cohesive materials can liquefy, crumple, rotate, flow, transform and disperse themselves to generate a failure that is obviously comlex, due to be lacking experiences of the stage is not enough to categorize all of the mechanical system. Nevertheless, lateral spreads are common, significant and hazardous sufficient to assure specific caution (Shroder et al., 2005).

The phenomenon of flowing movements have been hard to absorb due to its complex rheology as well as varied acceleration of movement and reaching the high value of water content. That varied motion composing flow in granular masses (debris and earth) can be inter-granular without shear surfaces. When the differential movements occur in among close spaces, all or some distributed shear surfaces can fix in sediments without leaving any proof and can be short lived. Appearance of plastic behaviour (folding, bending, etc.) can encountered in bedrock, because of the flow may be exceptionally distributed and slow amongst a lot of microfractures and other large and small fractures. Therefore, viscous fluid can be imitated by the internal velocity distributions. In the case of slow-motion, long term observing of individual particles can be required for definition of flowing mass flowing and nature or differential flow what more linear slides (Shroder et al., 2005).

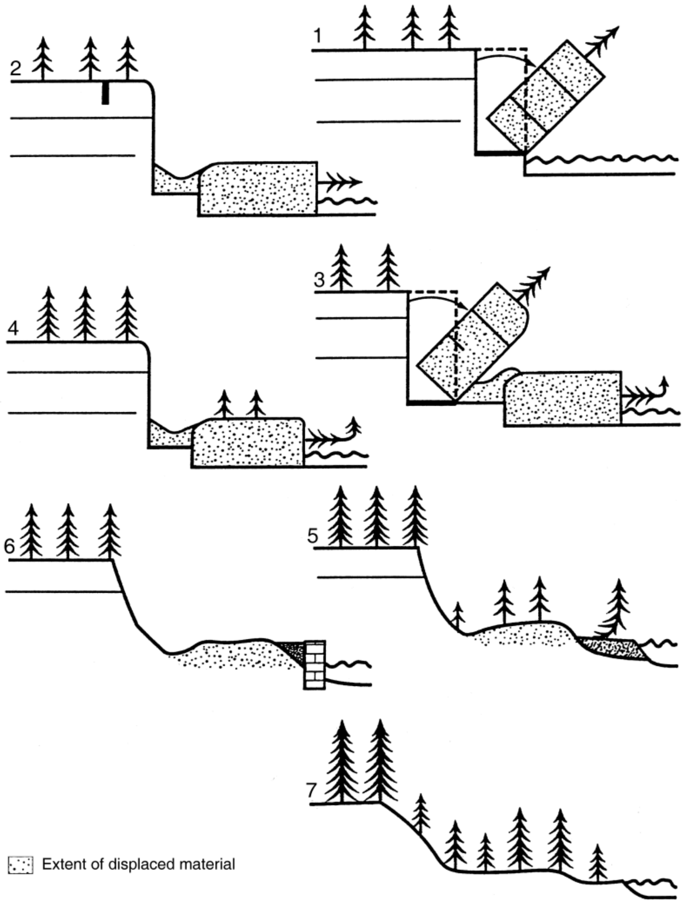


Figure 2.4: States of activity of slope (after Cruden and Varnes, 1996; Dikau et al., 1996).

1- Active; Block topple at the bottom of slope due to Erosion. 2- Suspended; Local cracking of head Although it has been inactive, had mobilised in recent past. 3- Reactivated; Recent action hassle to previous material which is displaced. 4- Inactive; No motion in recent past and can be divided into a following condition, lethargic, where mass starts to recuperate tree cover so it can reactivate features. 5- Abandoned; the state of slope having inactivated failure effected by natural reasons anymore. 6- Stabilized; deactivated failure of slope protected by regenerative prevention. 7- Relict; deactivated failure of slope improved under varied situation and no existing tree covered.

The last of 20th centuries, the basic kinds of motion which is referred in many schemes of classification in literature, utilized by other researchers. There are also some parameters taking in account so that proportion of motion, as well as the condition, way of the mass, and distribution. Ratio of velocity or classes or movement have been dealed recurrently in last century. Seven separate classes of activity (Figure 2.4), distributions of activity (Figure 2.5) and velocity established by Cruden and Varnes in (1996). Later they also added five kinds of activity (Figure 2.6; Dikau et al., 1996, Cruden and Varnes, 1996).

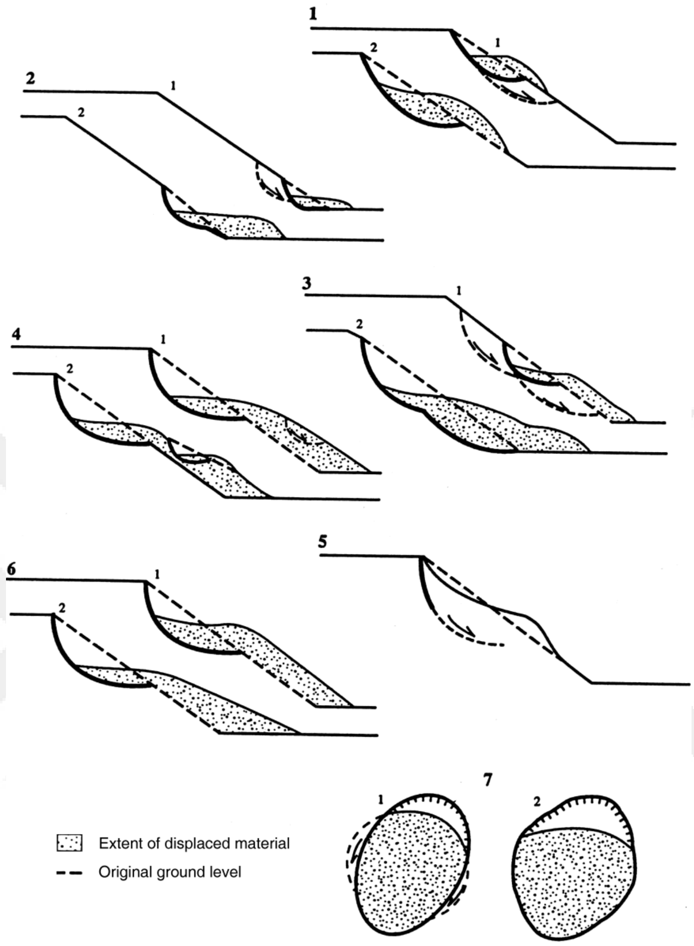


Figure 2.5: Sections and maps of landslides showing different distributions of activity (after Cruden and Varnes, 1996; Dikau et al., 1996).

1- Advancing; Means to extent the surface of rupture in motion way. 2- Retrogressive; Means movement of failure is opposite to expanding way of rupture surface. 3- Enlarging; Means surface of rupture expanding in multi ways. 4- Diminishing; Means declininig of volume. 5- Confined; Means scarp happens without rupture at the bottom. 6- Moving; Means motion but no visible alteration on surface of rupture. 7 Widening; Means to extent the surface rupture to single or both sides.

Such complex landslides as; flow of sandy-clayey soils and running of sandy soils or translational or rotational slide compose multi kind of the same main movement. Consequently, multi failure types appear, prevalently flow and slump. Lateral spreads are basically complicated. Failures of slope are also complicated and proper definition can infer from solution of mechanism. For instance, blocks of topple glide and blocks of backward rotating slump mostly improve slope downwards to flows can ensue while blocks turn over ahead to move further slope downwards by translational motion. Rock falls may sometimes occur high speed, long expire zone of fluidal granular inflow (Hungr and Evans, 2004).

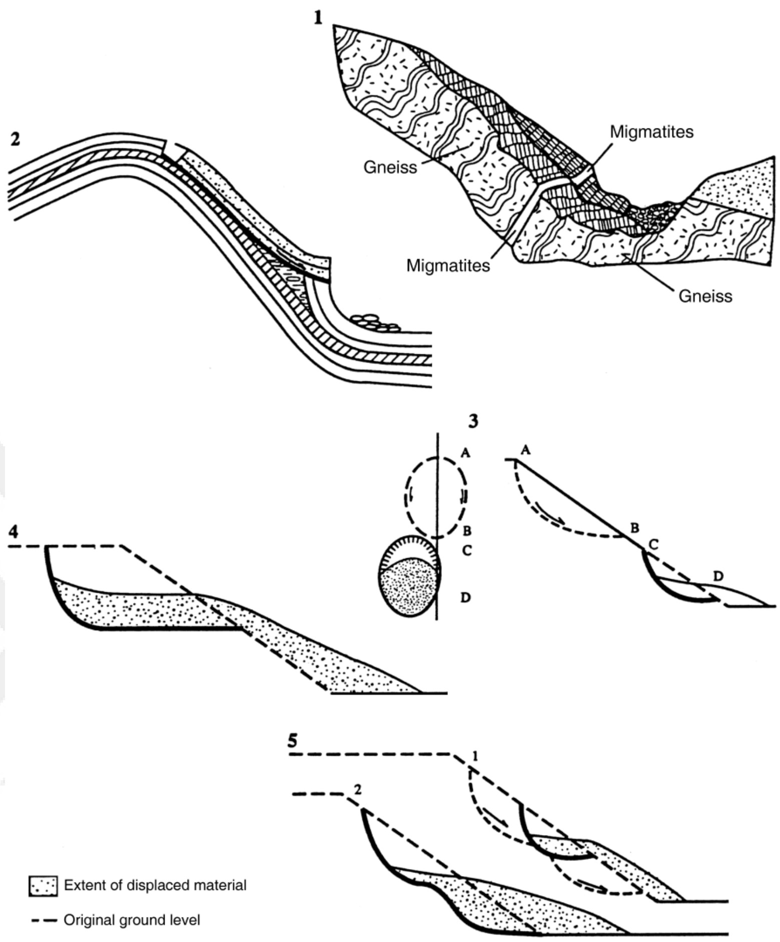


Figure 2.6: Styles of slope-failure activity (Dikau et al., 1996, after Cruden and Varnes, 1996).

1- Complex; Means, multi types of motion in respectively (for instance; indicates topple after that sliding). 2- Composite; Means, double kinds of synchronous motion in dissimilar sections (for instance; indicates topples and slides). 3- Successive; Means, roughly same kind of failure without displaced materials and multi sharing surfaces of rupture. 4- Single; Means, material is displaced in one direction. 5- Multiple; Means recurrently repeated improvment of same kind of motion.

Both Dikau et al., (1996) and Cruden and Varnes (1996) have submitted the last considering of the IAEG Commission about terminology on Landslides (1990), dimensions and descriptions of landslides (Table 2.1; Figures 2.7, 2.8, and 2.9). In addition, UNESCO and the International Geotechnical Societies on World Landslide Inventory (WP/WLI, 1993 b) have improved multi-lingual glossary of landslide in thirteen languages (Dikau et al., 1996).

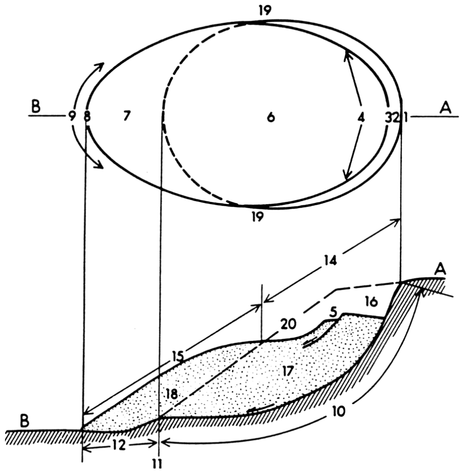


Figure 2.7: Cross-sectional diagram and map of slope-failure features with enumerated definitions in Table 2.1 (after Cruden and Varnes, 1996).

Table 2.1: Nomenclature and Definitions of Slope-Failure Features.

Number	Name	Definition
1	Crown	Practically undisplaced material adjacent to the highest parts of main scarp
2	Main Scarp	Steep surface on undisturbed ground at upper edge of landslide caused by movement of displaced material (13, stippled area) away from undisturbed ground; visible part of surface of rupture (10).
3	Top	Highest point of contact between displaced material (13) and main scarp
4	Head	Upper parts of landslide along contact between displaced material and main scarp (2)
5	Minor Scarp	Steep surface on displaced material of landslide produced by differential movements within displaced material
6	Main Body	Part of displaced material of landslide that overlies surface of rupture between main scarp (2) and toe of surface rupture
7	Foot	Portion of landslide that has moved beyond toe of surface of rupture (11) and overlies original ground surface
8	Tip	Point on toe (9) farthest from top (3) of landslide
9	Toe	Lower, usually curved margin of displaced material of a landslide, most distant from main scarp (2)
10	Surface of Rupture	Surface that forms (or that has formed) lower boundary of displaced material (13) below original ground surface (20); mechanical idealization of surface of rupture is called slip surface
11	Toe of Surface of rupture	Intersection (usually buried) between lower part of surface of rupture (10) of a landslide and original ground surface (20)
12	Surface of	Part of original ground surface (20) now overlain by foot (7)
13	Displaced material	Material displaced from its original position on slope by movement in landslide; forms both depleted mass (17) and accumulation (18)
14	Zone of Depletion	Area of landslide within which displaced material (13) lies below original ground surface (20)
15	Zone of Accumulation	Area of landslide within which displaced material lies above original ground surface (20)
16	Depletion	Volume bounded by main scarp (2), depleted mass (17), and original ground surface (20)
17	Depleted mass	Volume of displaced material that overlies surface of rupture (10) but underlies original ground surface (20)
18	Accumulation	Volume of displaced material (13) that lies above original ground surface (20)
19	Flank	Undisplaced material adjacent to sides of surface of rupture; compass directions are preferable in describing flanks, but if left and right are
20	Original Ground	Surface of slope that existed before landslide took place

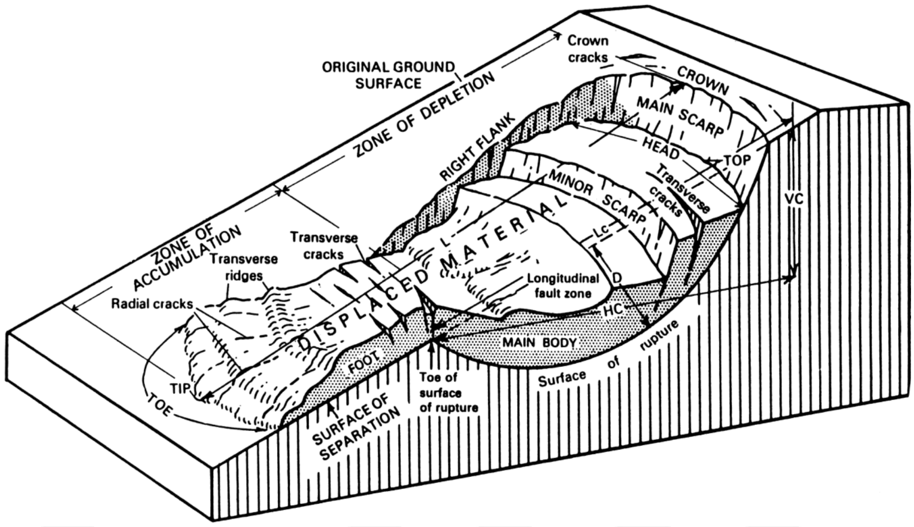


Figure 2.8: Block diagram of idealized complex earth slide-earth flow (after Varnes, 1978; Cruden and Varnes, 1996).

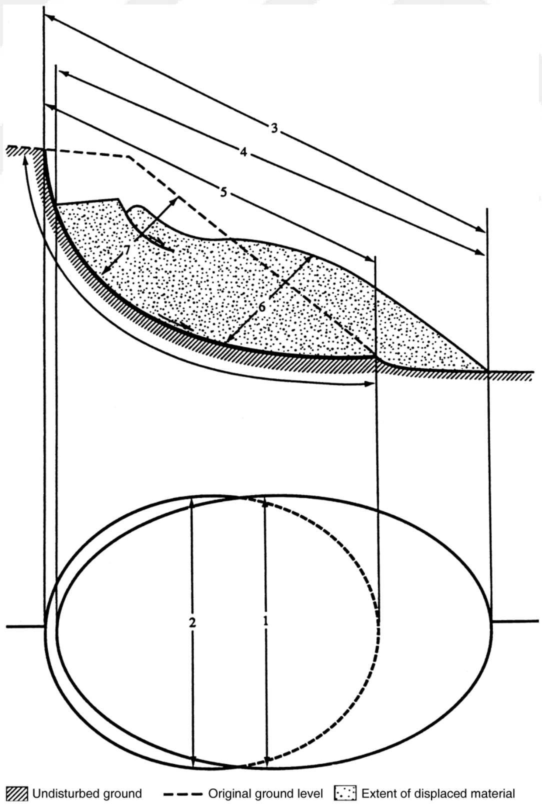


Figure 2.9: Dimensions of slope failures (after IAEG, 1990; Dikau et al., 1996).

1- Means, ultimate wideness of ousted mass is vertical to length. 2- Means, ultimate wideness surface of rupture between sides of landslide vertical to length. 3- Means, whole length from diadem to end. 4- Means, length of ousted mass. 5- Means, length of ruptures surface. 6- Means, deep of ousted mass. 7- Means, depth of ruptures surface.

2.2.4. Slope Failure Types

Field investigation and in-situ test are mostly necessary for categorizing a failure of slope. A significant effort is given to understanding of what type of material and evidence of initial movement and defining the morphology of the settled deposits. Geotechnical datas are mostly acquired by observation of active failures or drilling (Shroder et al., 2005).

2.2.4.1. Rock, Debris and Earth Falls

Near vertical or vertical motion of material from a steep slope or scarp, is called falls. Falling materials move mainly through the aerial from its origin to its deposits, even though it may shot the scarp at two or more spaces along its direction and may rolling several interval at the ground and disperse. Some falls rarely appear, apart from the clench mechanism of particles that lets clayey-sandy soils to halt in perpendicular walls, very high scarp is not mostly formed of earth and debris (Shroder et al., 2005).

2.2.4.2. Rock, Debris and Earth Slides

There are many diversity for slides. It can be consisted of only a several or many pieces, also be rotational or translational. So that failures sustain connection with multi comparatively shear surfaces or well defined slip. Slump blocks are slides which move by rearward turn; these blocks that act by translational or linear movement are sliding mass. The grade of disrupted mass pending displacement is frequently a task of disharmony of the slipping surfaces as well as the interval the block movements. When the slide of block disrupt into numerous singular parts, the name of slide is utilized with just the earth modifiers, debris and rock (Shroder et al., 2005).

2.2.4.3. Lateral Spreads of Rock, Debris and Earth

Comprehensive failures which includes slide and flow pieces are mainly flowing whereas these are assumed an isolated types due to being very much disruptive (Varnes, 1978). Movement of turbulent flowing is involved by reason of having mostly remolds or liquefies in the subgrade, and compatible blocks can be carried throughout in the flows (Shroder et al., 2005).

2.2.4.4. Rock, Debris and Earth Flows

Degenerated or the motion of extremely deformed materials are involved, mostly relevant the turbulent flowing of shear dispensed along the block in slope failures. Morphology of the surface has significant role for discriminate between a flowing and sliding in case of monitoring movement of the failure. Although some flowings are leisurely and act slightingly just 1 m in a year, generally many of these flowings are quick (Shroder et al., 2005).

2.3. FACTORS AFFECTING TO SLOPE FAILURES

Decreasing shear strengths of the soil masses mostly cause the slope failures. Factors which mostly cause an increment in the shear stresses in slope materials are listed in (Table 2.2). Subsequent list in (Table 2.3) indicates the factors which mostly cause lessen in the shear strength in slopes (Abramson et al., 2002).

Table 2.2: Factors That Cause Increases in Shear Stresses in Slopes (Highway Research Board 1978).

(1) Removal of support
- A. Erosion
  1. By streams and rivers
  2. By glaciers
  3. By action of waves or marine currents
  4. By successive wetting and drying (e.g., winds, freezing)
- B. Natural slope movements (e.g., falls, slides, settlements)
- C. Human activity
  1. Cuts and excavations
  2. Removal of retaining walls or sheet piles
  3. Drawdown of bodies of water (e.g., lakes, lagoons)

(2) Overloading
- A. By natural causes
  1. Weight of precipitation (e.g., rains, snow)
  2. Accumulation of materials because of past landslides
- B. By human activity
  1. Construction of fill
  2. Buildings and other overloads at the crest
  3. Water leakage in culverts, water pipes, and sewers

(3) Transitory effects (e.g., earthquakes)

(4) Removal of underlying materials that provided support
- A. By rivers or seas
- B. By weathering
- C. By underground erosion due to seepage (piping), solvent agents, etc.
- D. By human activity (excavation or mining)
- E. By loss of strength of the underlying material

(5) Increase in lateral pressure
- A. By water in cracks and fissures
- B. By freezing of the water in the cracks
- C. By expansion of clays

Weathered bedrock and residual soil can be debilitated by preexisting discontinuities such as soil dikes, relict joints, sheared zones, cleavages, foliations, bedding surfaces and faults. Joints of relict usually lose strength in case of residual soils are saturated. Slopes of weathered rock and residual soils can primarily have poor slickensides or seams or dikes. Slope stability of rocks is more vulnerable than slope stability of soils with regards to foliation, cleavage, bedding surface and fault (Abramson et al., 2002).

Table 2.3: Factors That Cause Reduced Shear Strength in Slopes (Highway Research Board 1978).

(1) Factors inherent in the nature of the materials
A. Composition
B. Structure
C. Secondary or inherited structures
D. Stratification
(2) Changes caused by weathering and physiochemical activity
A. Wetting and drying processes
B. Hydration
C. Removal of cementing agents
(3) Effect of pore pressures
(4) Changes in structure
A. Stress release
B. Structural degradation

2.4. GEOLOGICAL FEATURES ASSOCIATED WITH SLOPES

Comprehension the geology of an area is remarkable significance in tackling troubles relevant with development of slopes. Regional details about geology, such as; 1- Geometry of the subsurface, 2- Soil characteristics, 3- Groundwater (sometimes called the three rules of slope stability), have a notable effect on the efficiency of singular slopes. Therefore slope stability evaluation is an interdisciplinary effort needed notion and information from rock&soil engineering geology and mechanics (Abramson et al., 2002).

Terzaghi and Peck (1967), Terzaghi (1950) and other researchers have emphasized how important that geology with regard to slope stability. Main geological properties related to slope stability are 1- Soil & Rock fabric, 2- Geological structures, 3- Discontinuities, 4- Groundwater, 5- Ground stresses, 6- Weathering, 7- Preexisting landslide activities, 8- Clay mineralogy, and 9- Seismic effects (Abramson et al., 2002).

2.4.1. Soil/Rock Fabric

Soil&rock have a specific fabric belong to themselves. Mineral fabrics of some rocks can be adequately improved so that effects their engineering properties and characteristics. For instances of this fabrics such as; schists, shales, laminated clays and slates, which may cause spectacular anisotropy of their deformation and strength characteristics. Sometimes, the rot of the mineral fabric could outcome in fulfill loss of strength upon failure (i.e, quick clays) (Abramson et al., 2002).

2.4.2. Geological Structures

Geomorphology of the slope forming materials are a dominant property in the slope behavior. For instance, there is a direct relevance between possible instability in sedimentary rocks, and the attitude, thickness and succession of beds. Such forms take a substantial role in comprehension slope improving of colluvial deposits, talus and residual soils. Another primary and secondary structural discontinuities, such as joints, folds and faults, must also be attentively investigated and sketched. It is necessary to acquaint properties such as hydro geological influences, fault zones, previous failure surfaces, slim beds and a sequence of strong and weak beds in order to estimate slope stability completely (Abramson et al., 2002).

2.4.3. Discontinuities

Providing that a perpendicular slope of hard rock has no discontinuities, it can be stable with an altitude of hundreds mile. The significance water pressures of discontinuities and water seepage forces inside a rocks must not be unnoticed (Terzaghi, 1960). Assuming absence of seepage forces, the conformance of slopes as regards prominent discontinuities would effect the probable consistent motions. Figures 2.10 indicates potential correlation between slope stability and the discontinuities as ardued by Terzaghi, (1962). The angle of friction throughout discontinuities depends on some factors just as roughness, waiveness, weathered seams, infills, and continuity of joints (Abramson et al., 2002).

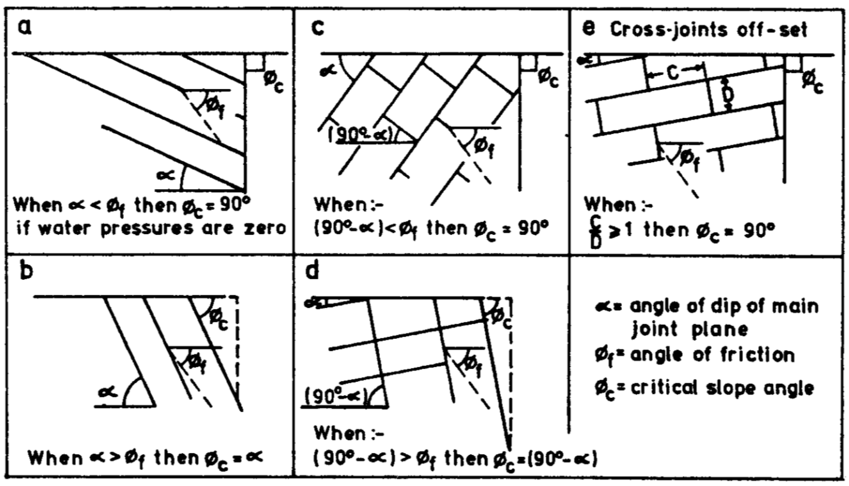


Figure 2.10: Effect of structure on rock slope stability. (Terzaghi, 1962).

2.4.4. Groundwater Conditions

The groundwater is assumed that is one of the most significant parameters of slope stability problems. Information about groundwater situation is priority for the design and analysis of slopes. This part defines the groundwater flow in rocks/soils and the methods whereby the effect of rainfall on groundwater conditions can be evaluated (Abramson et al., 2002).

Briefly, groundwater can affect the strength of slope establishing materials by 1- Softening of stiff fissured shales and clays, 2- Decline of obvious cohesion due to capillary forces (soil suction) upon saturation, 3- Rise in porewater pressures and subsequent reduce in shear strength, and 4- Hydrothermal and Chemical alteration and solution (Abramson et al., 2002).

2.4.4.1. Movement of Groundwater

Flowing of the groundwater is caused by the energy which is originated from gravity. Power of the gravity absorbs water downwards to ground water table; it flows from here through to a point of discharge of a lake or stream. As surface water requires a slope to flow, so there must be a slope in order to flow of groundwater (Abramson et al., 2002).

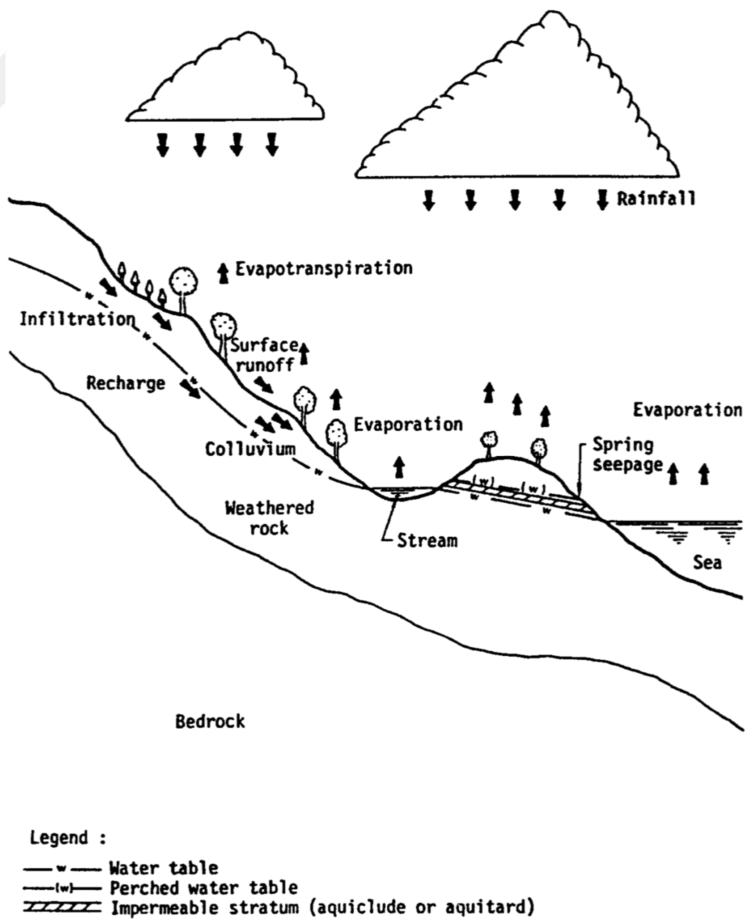


Figure 2.11: Simplified representation of the hydrological cycle. (Geotechnical Control Office, 1984).

2.4.4.2. Groundwater Levels

The major sources of groundwater origination are melting snow and rainfall. Some water moves over the surfaces surface runoff, while some water infiltrates into the ground and percolates downwards to the saturated zone at depth. Groundwater where is in the saturated zone acts against lakes, seas and rivers, where it evaporates and returns to the land as clouds of water vapor, which precipitates as snow and rain. This circulation of water is frequently called as the hydrological cycle, as indicated in Figure 2.11 (Abramson et al., 2002).

2.4.4.3. Zones

Just as said previously, snowmelt and rainfall on the surface will able to infiltrate into the subsurface materials, which are segregated to saturated or unsaturated zones. The unsaturated zone is constantly located between the surface and the actual groundwater table with voids relatively filled with water. This zone is known as the region of aeration, and it enlarges from the ground down through the major root region (Figure 2.12). Its thickness differs with the vegetation and soil types. Inside this region, some spaces between granules are filled partially with air and partially with water. Nonetheless, the saturated zone is inside the major groundwater form with voids entirely filled with water. Groundwater flows which is determinant on the stability of slopes, have varied type and improve in the unsaturated and saturated zones (Abramson et al., 2002).

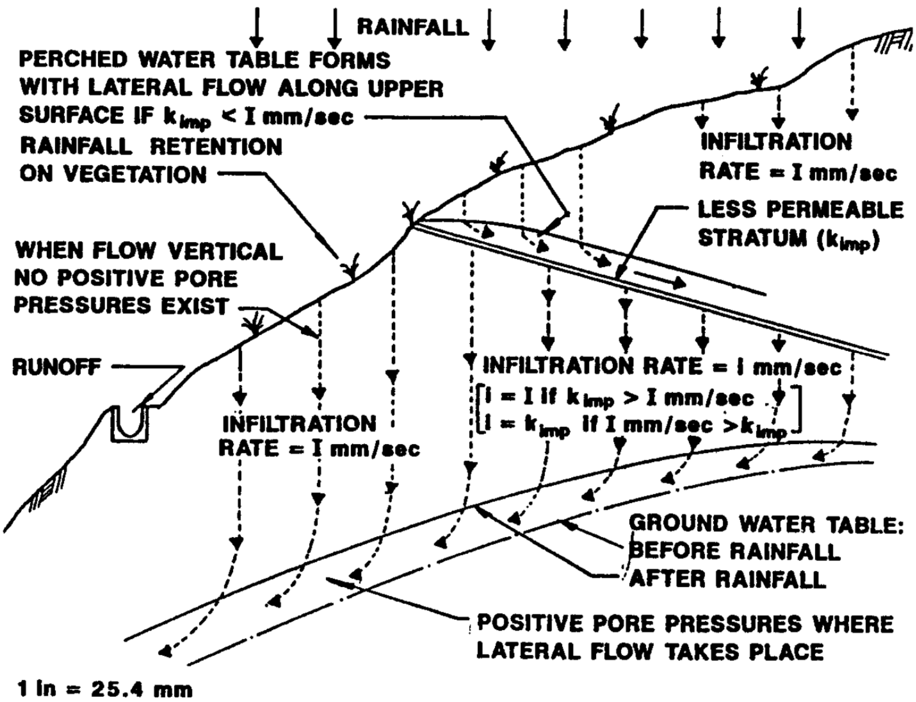


Figure 2.12: Modes of groundwater flow.

2.4.4.4. Infiltration

While the infiltrated groundwater from rainfall and snowmelt, was being decline, can encounter a material which has lower permeability. In case of, if the proportion of infiltration is more than the permeability of lower zone flowing would be inhibited. Consequently, occuring a perched water table is expected on the surface of the impermeable zone (Figure 2.12). The infiltration proportion will be decreased to the amount of the permeability of the zone, below the impermeable zone (Abramson et al., 2002).

2.4.4.5. Runoff induced by Rainfall and Snowmelt

Runoff is rate of snowmelt or rainfall that flows from a watershed region into seas, lakes, or streams. Surface runoff from a watershed region depends on some of the following factors:

- Nature of the subsurface soils and situation of the surface
- Space of cultivation or vegetation and nature
- Length of the slopes and steepness
- Field and shape of the watershed area
- Rainfall intensity (Abramson et al., 2002).

2.4.4.6. Floods

When rainfalls at a proportion that overlaps the infiltration capability of the ground, flood happens, concluding in deposits of water. Deposits of water which is in surrounding of a slope decrease the existing strength of the slope since cumulative water finally infiltrates into the slope, and enhance pore pressures on probable sliding surfaces (Abramson et al., 2002).

2.4.4.7. Pore Pressures

The soils are completely saturated under the groundwater table, and in positive values by virtue of the pore pressure is over atmospheric pressure. Unsaturated soils which are over the groundwater table, and in negative values of soil suction due to the pore pressure is under atmospheric pressure. Any alteration of shear strength of the soil causes enormous influences on the slope stability in case of the pore pressures change. The variance of pore pressures in any areas effect slope stability in dissimilar ways. A shematic diagram of variance in pore pressure according to each zones is shown in Figure 2.13 with as a result of rainfall. (Abramson et al., 2002).

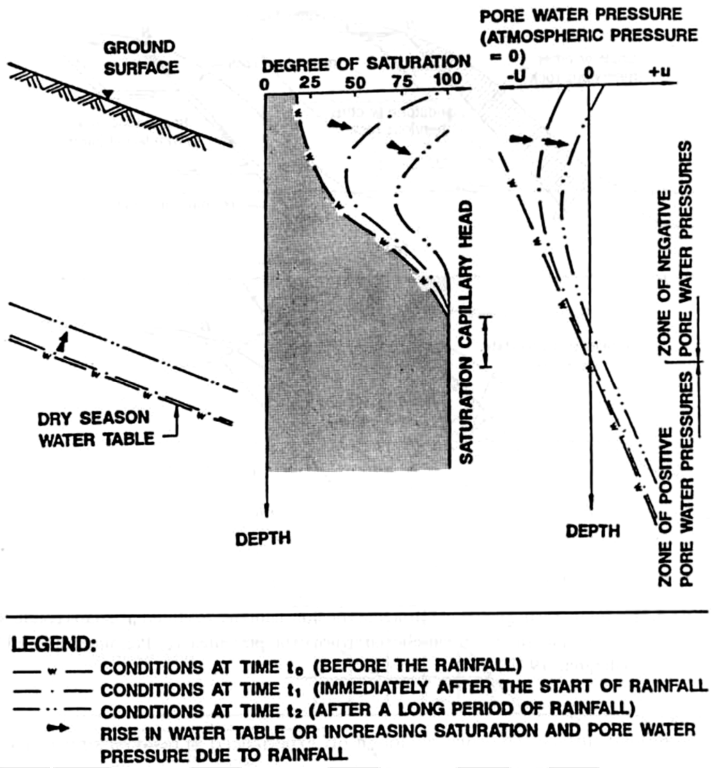


Figure 2.13: Typical changes in water table: degree of saturation (s) and pore water pressure (u) are due to rainfall. (Geotechnical Control Office, 1984).

2.4.4.8. Reduction in Shear Strength

The frictonal shear strength will be decreased as long as saturation of a soil. The reason is that, "the effective stress principle" namely, the pore pressure requires decline of the buoyant in normal force for frictional shear strength. In addition soil saturation can demolish evident cohesion and capillarity on the component of the cohesive soil, or can decrease the dry strength of the cohesive soil (Abramson et al., 2002).

2.4.5. Ground Stresses

All materials constituent of slope are subjected to initial stress as a consequence of weathering, erosion, tectonic activity, gravitational loading and others. Such processings produce the stress, and are formed automatically remaining their place after the stimulus that generated them has been removed. Therefore, it is known as "residual stresses" (Abramson et al., 2002).

Big lateral stresses have taking an important part in launch landslides in clay&shales and overconsolidated clays. There are some studies in literature about significance of the clay&shales and overconsolidated clays in landslides. Researchers have mainly focused at length of slide.

2.4.6. Weathering

Two kinds of weathering in literature; first is the chemical weathering by virtue of chemical variations, second is the mechanical weathering as a consequence of erosion by streams and rivers, freeze&thaw cycles, temperature changes and wind. The proportion of chemical weathering scales from a several days to a lot of years and may effect both of them (long and short term stability of slopes) (Blyth and de Freitas, 1984). Another respects, mechanical weathering can take a lot of years without any contrary impact on slopes (Abramson et al., 2002).

2.4.7. Preexisting Landslide Activities

Information of local geology is too spectacular in comprehension both ancient and recent landslide activities. Slope composing materials in this zones are generally launch and comprise of changing percents by weight of gravels, clays, sands, silts, boulders and cobbles. The materials are frequently remolded and take up a lot of humidty as a consequence of remolding. Shear strengths frequently attain their residual values (Abramson et al., 2002).

2.4.8. Clay Mineralogy

If the soils having fine grain mix with clay mineral, that will display plasticity, in case of encountering to water. The clay minerals in terms of chemistry, are silicates of aliminium and calcium. Groundwater is severely sucked by clayey minerals. The behaviour and structure of sucked water in clay minerals are varied from those of common water. Also it takes a significant part in soil behavior (Abramson et al., 2002).

The most prevalent clay minerals are montmorillonite, kaolinite and illite. Of these minerals, montmorillonite has the most onerous in regardings of slope stability, as a result of supreme swelling potential. By reason of hydrothermal and chemical alteration, weathering or nonargillaceous and argillaceous rocks. So, clay minerals have a substantial impact on the behavior of rock mass (Abramson et al., 2002).

2.4.9. Seismic Effects

A lot of minor and major landslides have occurred during earthquakes in the past. Geological features, whether major or minor, have a substantial impact to stability of slope pending earthquakes. Earthquakes conclude in a descent of shear strength and an increment of shear stresses by increment of pore pressures. Liquefaction of little saturated silt and sand lenses inside a slope can conclude in progressive failures of materials that may be partially insensible to seismic distortion (Abramson et al., 2002).

2.4.9.1. Earthquake induced Landslides

Loose and saturated sands are especially vulnerable for liquefaction pending earthquakes, primary to flow slipping or precarious foundation situation for superjacent residuals of slope. According to cases in history, well compacted fills built over poor foundation are more inclined to violent slump or fully failures than are those established on strong foundation during earthquakes (Abramson et al., 2002).

These failures can be classified by lateral spreading of ground of the fills in case of the motion is smaller violent, and the sliding cause to serious longitudinal cracking in the fills (Abramson et al., 2002).

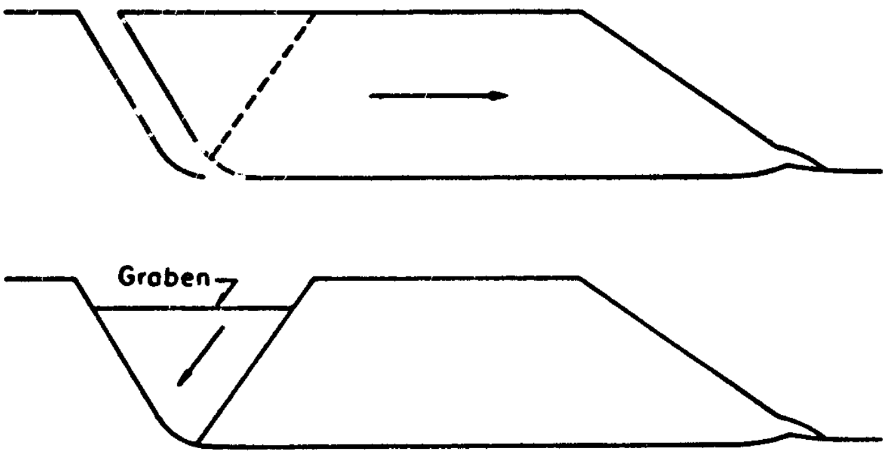


Figure 2.14: Mechanism of graben formation due to sliding on horizontal layer (Seed, 1970a).

The eartquakes may contribute to induce an overlying sloping soil mass to slide laterally along the liquefied layer in slim lenses of loose saturated sands and silts. As indicated in Figure 2.14 a zone of soil at the back end of the sliding mass sinks into the vacant space formed as the mass translates, resulting in a depressed zone known as graben. Main slide motions may also become in clay residuals during eartquakes. Though, clay residuals frequently include sand lenses, and liquefaction of these lenses may well contribute substantially to the slide improvement in these cases (Abramson et al., 2002).

2.5. SEGREGATIONS FOR ANALYSIS OF SLOPES

The degradation of the delicate balance of the slopes by humans or natural soil formations is the biggest problem in terms of slope stability. Moreover, the growing demand in these regions is lead by the required to figure out stabilization and analytical ways, research devices to find a way slope stability problems (Abramson et al., 2002).

A comprehension of soil properties, hydrology and geology is initial factor to implementation of slope stability principles appropriately. Analyzes have to be based on a form that realistically symbolizes applied loads, soil behavior and underground surface conditions. Acceptable risk or safety factors in the literature should be taken as a reference to assess the results of the analysis (Abramson et al., 2002).

In many executions, the initial aim of slope stability analysis is to contribute to the economic and safe design of spoil heaps, landfills, earth dams, embankments, and excavations (Abramson et al., 2002).

The purpose of slope stability analyses is:

- To investigate the affect of seismic loadings on embankments and slopes.
- To allow the re design of failure of slopes and the plan and design of remedial and preclusive predictions, in which required.
- To analyze landslide and to figure out mechanisms of failure in slope and the impact to surrounding factors.
- To assess the probability of landslides including natural or existing engineered slopes.
- To interpret the stability of slopes under long term and short term (mostly in a construction stage) conditions.
- To figure out the improving and form of natural slopes and the process liable for different natural properties (Abramson et al., 2002).

2.5.1. Natural Slopes

Valley and ridges in which may be inclined to slope stability problems are intersected by many projects. Although, natural slopes have been stable for a lot of years, that may be influenced by some variation in weathering, stress changes, loss of strength, groundwater flows, seismicity and topography. Normally, these failures are not figured out well due to lack of investigation until the failure makes it necessary (Abramson et al., 2002).

If previous slip surfaces are on a natural slope, it facilitates to predict and understand the behavior of this slope. Generally, tectonic activities or previous landslides constitute these slip surfaces. Moreover, the slip surfaces may also be derived from involved valley rebound, glacial phenomena and glacial shove just as nonuniform swelling of clay&shales and clays and solifluction. The shear strength is usually very low along the sliding surfaces due to the previous motion shear resistance and gradually falls to residual values (Abramson et al., 2002).

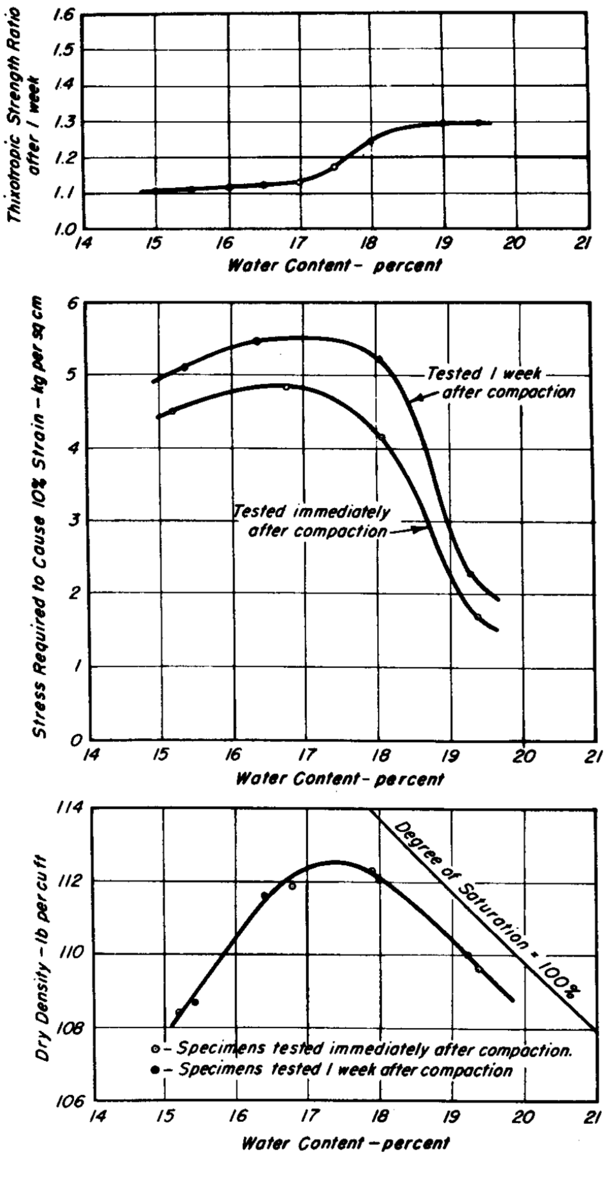


Figure 2.15: Strength of compacted clay versus moisture content. (From Seed and Chan, 1959).

2.5.2. Cohesionless Fills

Such a soils remain permeable when compacted, besides that usually consist relatively gravels and clean sands. These soils are displayed SW, SP, GW, GP and boundary groups of anyone of them with reference to soil groups of Unified Classification System (Abramson et al., 2002).

Compacted cohesionless soils are not effected substantially by water content in a compaction stage, because they are relatively permeable. The proctor curve is often circular. Because it is depicted in the dry density against optimum moisture content graph. Therefore, their compactness is frequently interpretated based on their relative density, as acquaints by Terzaghi (1925).

2.5.3. Cohesive Fills

The soil mass is rendered relatively impervious in case of appropriately compressed cohesive soils that include adequate amount of clay and silt molecules. Contrary compressed cohesionless soils, whose physical features are generally developed by compaction to the ultimate dry unit density, the physical features of cohesive soils are not necessarily developed by compaction to an ultimate unit density. For instance, Figure 2.15 shows that the strength of compacted silty clay reduces with boost casting water content (Seed and Chan, 1959).

2.6. EVALUATION OF SLOPE STABILITY

After the geometry of slope and situation of subsoil have been inferred from exploration, the slopes stability may be identified using either a computer analysis or assumed chart solutions. There are some computer programs which can analyse slope stability. Mostly these are based upon the limit equilibrium approachment for a multi dimensional model (Abramson et al., 2002).

Actually, there are more complex programs. For example; boundary or finite element methods are also viable and enable the engineers to carry out two or three dimensional slope assessment. On the other hands these analyses need a relatively full modelling of the subsoils and their constituent factors defined by a qualified programs of laboratory experiment (Abramson et al., 2002).

Namely, be started with disputation on the singularity of failure mechanisms and a brief literature review. Afterward, factor of safety methods is indicated how it is derived and modified and to highlight the significance of modeling assumptions. Then a slice methods approach is presented followed by an instance that explores the importance of interslice forces on calculated factors of safety. The suggested method of slices procedure applies force equilibrium on the global and local levels and comptes the corresponding global safety factors based on moment equilibrium. It supplies progressive failure by regarding a constituent relation between the relative motion among slices and the stresses (Abramson et al., 2002).

2.6.1. Factor of Safety Concepts

One of the substantial thing for designing of slopes is a comprehension of the act of the (FOS) “factor of safety”. That insert into the analyze, such as stratigraphy, pore pressure distribution and strength parameters may be informed about characteristic of model by means of taking into account a well recognized function of the factor of safety. Generally, as inversely correlated, the inferior capacity of the site exploration, the higher desired FOS should be, in the case that if the analyzer has scarce practice with the material in inquiry. Other mission of the (FOS) is that composes an empirical tools via stability and deformation performances are limited to tolerable quantity. Hence, the selection of the FOS is thoroughly impressed by the cumulative experimentation with a specific soil mass. The actual magnitude of the FOS used in design will change with performance requirements and material type due to the limit of risk that can be taken is also majorly impressed by experimentation (Abramson et al., 2002).

In this case the FOS is supposed to be fixed for whole failure surface. For instance, at point A in the upper slope shown in Figure 2.16, this average FOS will be given by the proporiton of existent to required shear strength (Abramson et al., 2002).

If τ_{req} is the required shear strength, then

$$\tau_{req} = \frac{S_u}{F} \qquad \text{for total stresses} \qquad \text{(Eq. 2.1)}$$

$$\tau_{req} = \frac{c'}{F_c} + \frac{\sigma' \tan\phi}{F_\phi} \qquad \text{for effective stresses} \qquad \text{(Eq. 2.2)}$$

Where S_u = the total stress strength

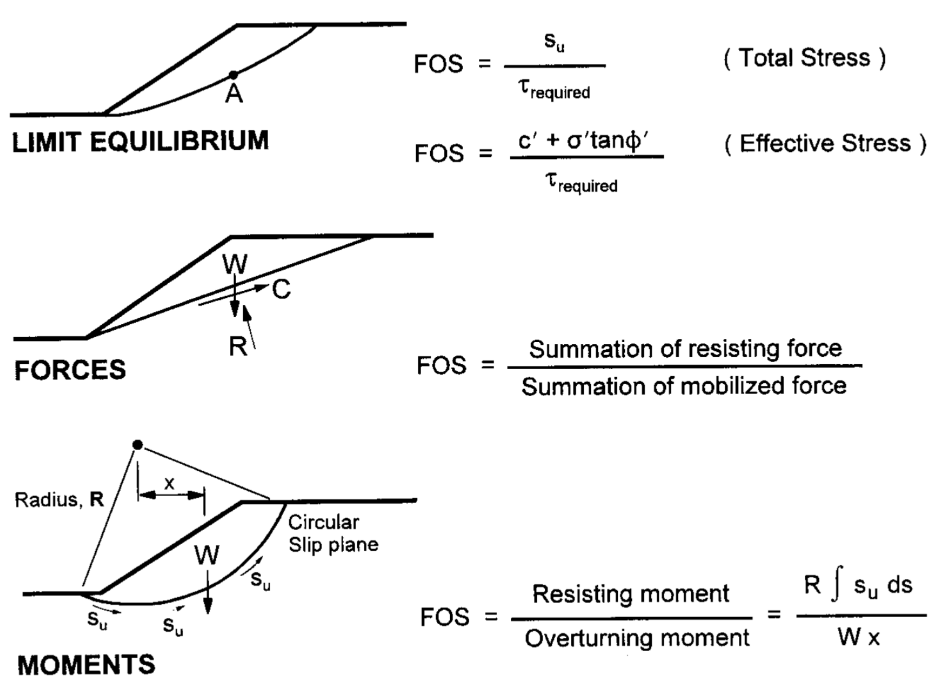


Figure 2.16: Various Definitions of factor of safety (FOS).

c' *and* ϕ' = effective stresss strength parameters

F = the factor of safety for total stresses

F_c *and* F_ϕ = the factor of safety for effective stresses (Abramson et al., 2002).

2.6.2. Method of Slices

The methods argued previously are not dependent on distribution of normal effective stress throughout surface of the failures. Even tough, if the actuated force of a c-ϕ soil is computed, the distribution of normal effective stress throughout surface of the failures have to be acknowledged. So that is ordinarily analysed by dividing the masses of the slope failures into small several slices and curative each one as an individual slab (Abramson et al., 2002).

Sliding masses are divided into "n" less slices for slope stability analysis in all limit equilibrium methods, as shown in Figure 2.17. All slices are effected by a common system of force, as indicated in Figure 2.18. The thrust line displayed in the figure hitchs the joints of implementation of the intersliced forces, Zi. The place of thrust line can be supposed, as within the meticulous method (Janbu, 1954a, 1954b, 1973), or its place can be determined utilize a meticulous method of analyze omission the location of the interslice force by reason of full equilibrium is not gratified for the mass of failure (Abramson et al., 2002).

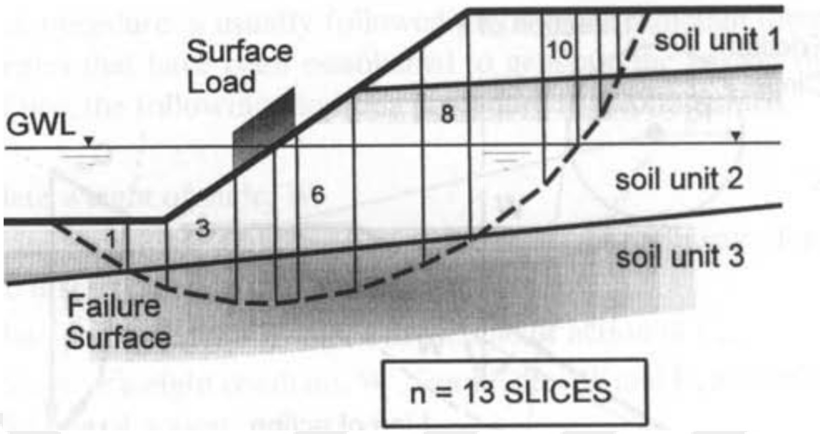
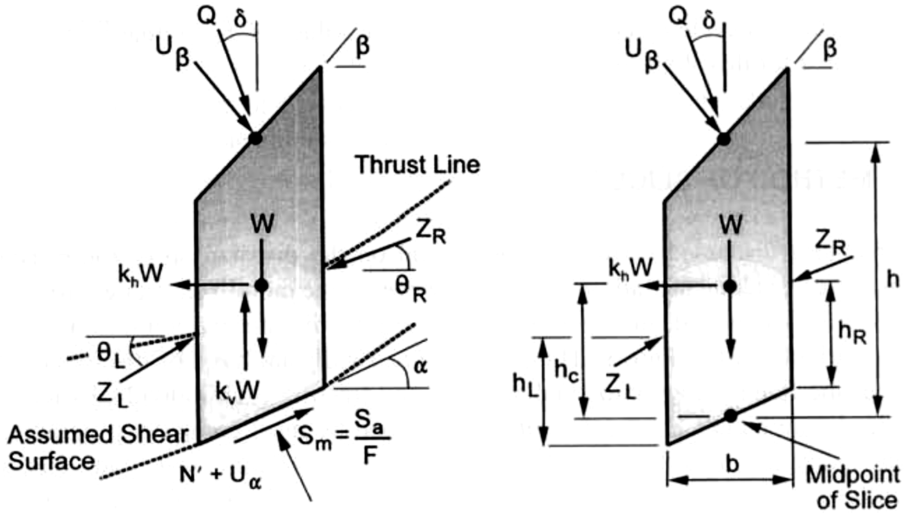


Figure 2.17: Division of potential sliding mass into slices.



F = factor of safety
S_a = available strength
= C + N′ tanϕ
S_m = mobilized strength
U_α = pore water force
U_β = surface water force
W = weight of slice
N′ = effective normal force
Q = external surcharge
k_v = vertical seismic coefficient
k_h = horiz. seismic coefficient

Z_L = left interslice force
Z_R = right interslice force
θ_L = left interslice force angle
θ_R = right interslice force angle
h_L = height to force Z_L
h_R = height to force Z_R
α = inclination of slice base
β = inclination of slice top
δ = inclination of surcharge
b = width of slice
h = average height of slice
h_c = height to centroid of slice

Figure 2.18: Forces acting on a typical slice.

The situations of static equilibrium and common methods of analysis are listed in Table 2.4. These make it satisfy defining the FOS. The guesses derived by whole mentioned techniques, to provide the trouble specific, are also shortly given in this study.

Table 2.4: Static Equilibrium Conditions Satisfied by Limit Equilibrium Methods.

Method	Force Equilibrium *x*	Force Equilibrium *y*	Moment Equilibrium
Ordinary method of slices (OMS)	No	No	Yes
Bishop's simplified	Yes	No	Yes
Janbu's simplified	Yes	Yes	No
Lowe and Karafiath's	Yes	Yes	No
Corps of Engineers	Yes	Yes	No
Spencer's	Yes	Yes	Yes
Bishop's rigorous	Yes	Yes	Yes
Janbu's generalized	Yes	Yes	No
Sarma's	Yes	Yes	Yes
Morgenstern–Price	Yes	Yes	Yes

2.6.3. The Finite Element Method (FEM)

Engineers can quickly evaluate the slope stability by means of the limit equilibrium methods. Tough, these operations are the same whether the analyze count an existing natural slope, slopes after recent excavation or slope of newly constructed embankment. The stress inside these slopes are violently impressed by K0, the rate of lateral to perpendicular effective normal stress, however traditional limit equilibrium principles disregard this significant characteristics (Chowdhury, 1981). Natively, their stability is substantially influenced due to presence of varied stress distributions within these slopes (Abramson et al., 2002).

Most insufficiencies which are spontaneous in the limit equilibrium methods are averted by the finite element method (FEM). This method was first recognized to geotechnical engineering by Clough and Woodward (1967), nevertheless its utilization has been scarced by the summary presented by Duncan (1996). For typical cases, the FEM can incorporate behaviour of topography to simulate the stress history of the soils inside the slope. The data can be aggregated for recent embankment designs by means of lab experiments. For natural slopes and excavations, the constituent modelling may only be improved on the foundation of upper standart in-situ tests which are additionally contributed by land investigations (Abramson et al., 2002).

The Finite element methods fundamentally divide the soil continuity into separated unities finite elements (see Fig. 2.19). Those elements are connected each other at predetermined boundaries of the continuity and at their nodes. Geotechnical engineer basically utilizes the displacement method derived from FEM formulation, and nowadays conclude in the amount of stresses, strains and displacements at the nodal points. In addition, performing two and three dimensional finite element analysis of embankments and slopes, is possible by means of many available computer programs (Abramson et al., 2002).

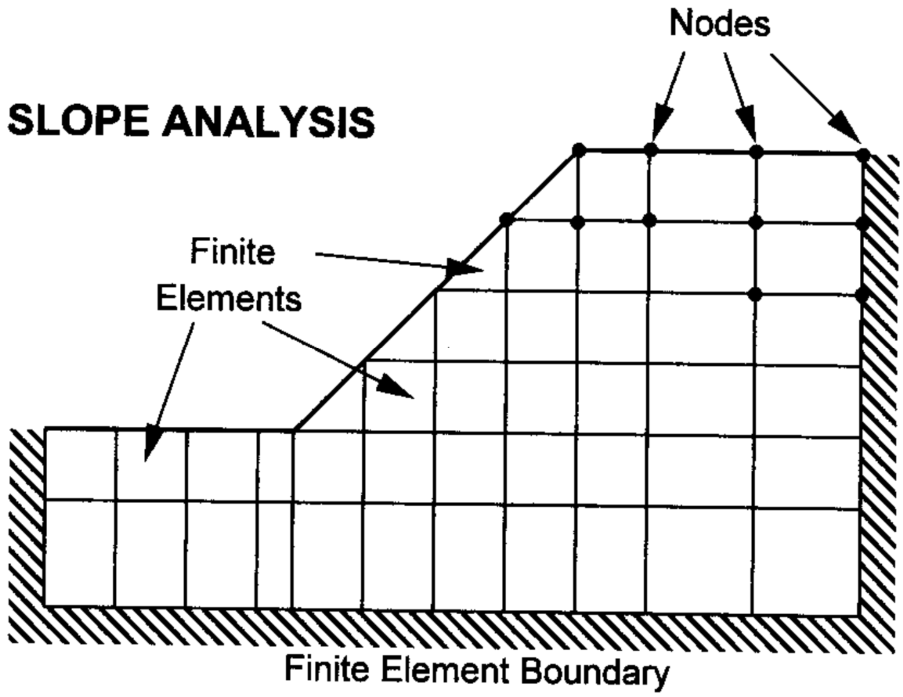


Figure 2.19: Definitions of terms used for finite elements method (FEM).

3. METHODOLOGY AND ANALYSES

In this study, two critical points in Akcaburgaz vicinity located at ridges of watersheds of the Istanbul-Buyukcekmece Lake are investigated in terms of landslide potential. The methodology for the this study comprised three steps: (1) to obtain geotechnical parameters of land via performed laboratory tests to undisturbed samples (2) to monitor soil movements via placed inclinometer tubes. (3) to estimate the degree of risk potential of landslides 3 different situations as a long term, short term and with earthquake by using a commercial software program PLAXIS and SLIDE.

3.1. LABORATORY TESTING AND INTERPRETATION

Limit equlibrium methods utilized for assessment the slope stability need correct and dependable prediction of the shear strength of the materials in slope. Though, parameters of the shear strength are violently effected by a lot of comprehensive situations, including the soil composition, drainage, overconsolidation rate, loading ratio and in-situ state of stress. The objective of this part is to provide us with a suitable understanding of shear strength concepts such that appropriate laboratory tests were performed to identify the physical, and mechanical, and engineering features of soils in this context, we performed; (Abramson et al., 2002).

- Fall cone and Casagrande tests for define liquid limit,
- Plastic limit tests,
- Grain size distribution tests,
- Consolidation tests,
- Shear box tests,
- Triaxial compression tests,

3.1.1. Shear Strength of Soils

That can able to be carried out expressive slope stability analyses depends on the correct identification of representative soils shear strength of the slope materials. Even tough some strength surveying of in-situ are able to be satisfying in case of under suitable circumstances, lab estimations of strength are by far the most widespread for fine grained soils that can be illustrated faithfully. Though, the estimation of shear strength identified from lab experiments are dependent on many parameters, especially the kind of soils, the experiment methods, the size of experiments specimens, and quality of the experiment specimens. These experiments will usually identify the curves of stress strain for the anticipated soil situations, hence it allows the engineer to choose convenient strain harmonious strength estimations (Abramson et al., 2002).

Strength testing requires the proper testing processing and equipment, preparation of representative soil specimens, and the attentive choice. Occasionaly, the direct shear experiment (DS) can be choice for its ease of use in case of drainage situations are reliable (Abramson et al., 2002).

The typical necessity for improving a testing program is based upon the sort of loading situations anticipated and whether the subsoils attitude follows undrained or drained behavior. Figure 3.1 shows a simplified approachment for choosing proper experiments to identify the Mohr&coulomb parameters (ø and c) (Abramson et al., 2002).

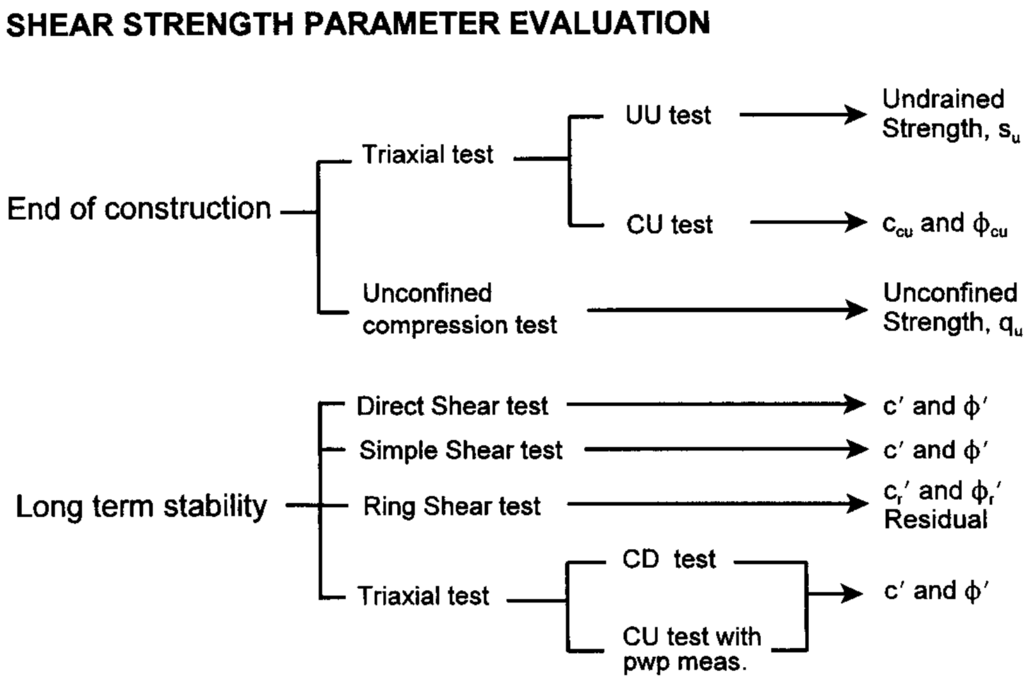


Figure 3.1: Selection of laboratory testing procedures (Abramson et al., 2002).

3.1.1.1. Residual Strength

Decline to an inferior residual shear strength is dedicated to the layer like form to clay minerals. Those minerals have a inclanation to arrange each other in parallel to a shear surface. Hence the shear strength throughout this realigned region might be significantly lower than the strength of the adjacent undistrubed material. The strength of this realigned materials is acknowledged as the residual strength (Abramson et al., 2002).

3.1.2. The Triaxial Tests

The experiment for identifying the shear strength of soils. (Saada and Townsend, 1981). Figure 3.2 indicates a photograph of triaxial frame with a triaxial chamber. The faster unconsolidated undrained experiment (UU) can be utilized for undrained strengths in case of well-quality specimens are viable. (UU) experiments which the specimen in the pressure cell is subjected to a detaining pressure with hindering the specimen to consolidate. Drainage is restricted during implementation of the axial load for the undrained experiments. (Abramson et al., 2002).

The outcomes of triaxial experiments can be plotted either as a series of points or Mohr circles a symbolizing the ultimate shear stresses on the mohr circles. Graphical instances of experiment data aggragated during a triaxial experiment are indicated in Figures 3.3 and 3.4 (Abramson et al., 2002).

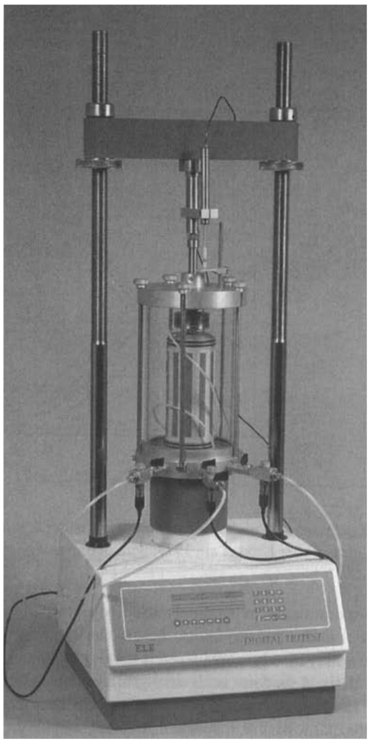


Figure 3.2: A triaxial loading frame capable of stress path testing (Courtesy ELE Engineering, Inc.).

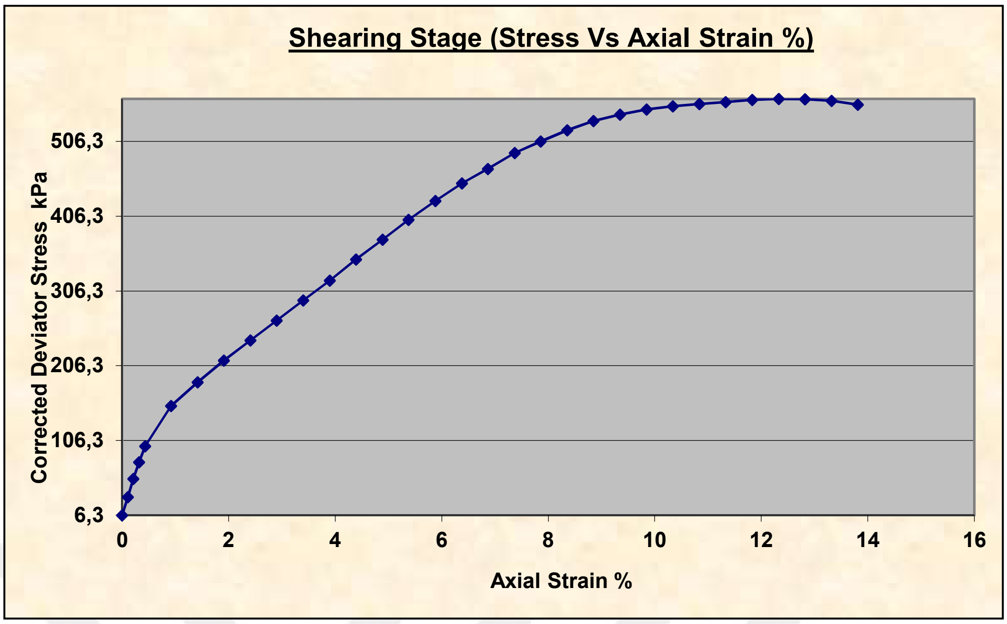


Figure 3.3: Examples of shearing stage of sample depth 2,50 m of Sk-2.

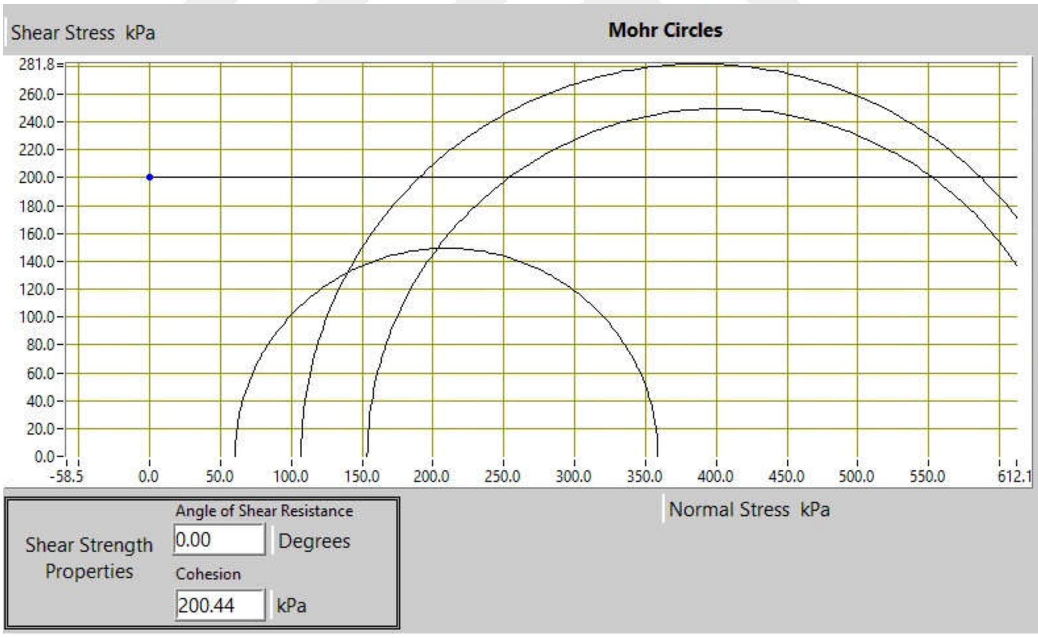


Figure 3.4: Examples Mohr circles of sample depth 2,50 m of Sk-2.

3.1.3. Direct Shear Tests

The shear strength of a soil specimen is determined by means of the test of direct shear (DS) which is one of the earliest and easiest experiments. Although there is no knowledge about the stress situation inside the specimen, this test can be performed confidingly to identify the residual shear strength (Saada and Townsend, 1981).

The DS box experiments are satiably performed on specimens however obtain in less results owing to taking long times. Figure 3.5 indicates a schematic diagram of the direct shear box, and figure 3.6 indicates a photograp of a commercial direct shear testing device.

The specimen is entrenched between the two porous stones to ease drainage, and then the normal load is applied to the specimen with to hang weights to arm, or with a hydraulic piston. The shear force is obtained from the piston laterally moved thru by a configurable automatic impulse system. The lateral displacement is reading value by the shear force and a lateral dial, and by dial of load and a test ring (Abramson et al., 2002).

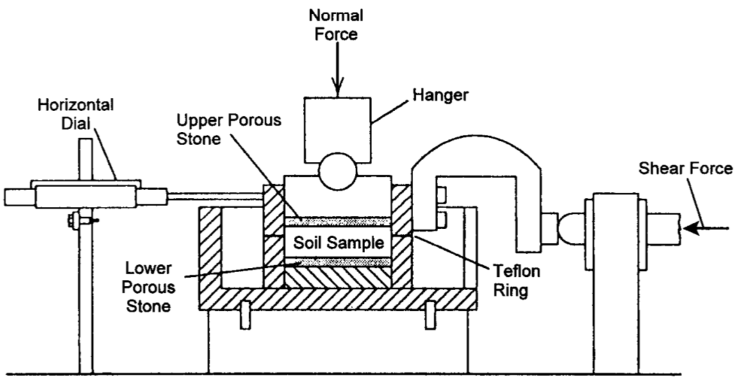


Figure 3.5: Schematic diagram of a direct shear test box assembly (Abramson et al., 2002).

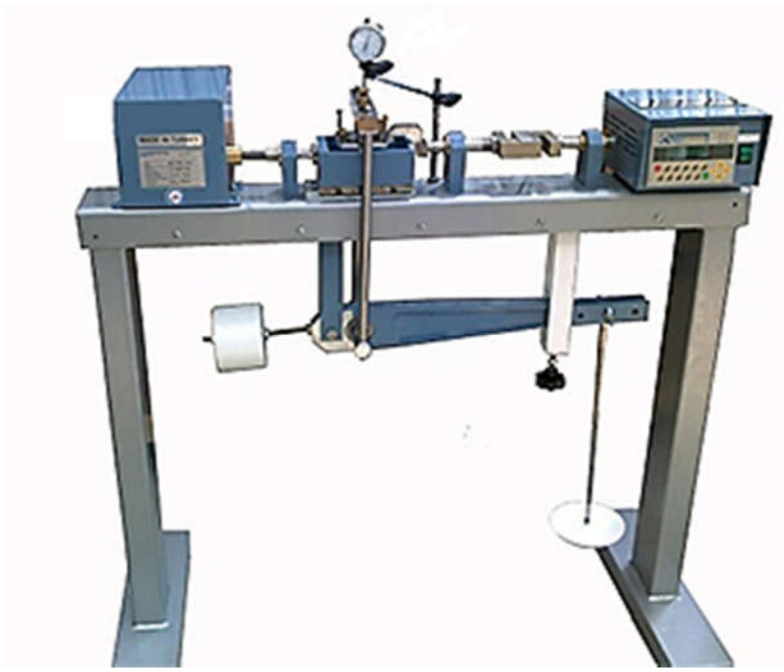


Figure 3.6: A direct shear testing device (Courtesy ELE Engineering, Inc.).

This experiment which examines the shear attitude of soils significantly varying with regards their residual and peak strengths and the shear displacement necessary to attain the residual values (Chen X. P. & Liu D., 2013).

Residual strength is estimated with using the shear box. Providing that dry soils hydrate, samples were rendered fully saturated. The process of shearing and consolidation were carried out in a water sink to conserve fully saturated situations. Afterward continued slowly loading consolidation to prohibit soil loss due to the apparatus, and whole specimens were subjected to a shear experiment. Consequently, shearing was performed at a proportion of 0.0250mm/min, and consistent situation residual strength estimated (Paolo and Marco, 2008).

The outcomes of the shear experiment are usually plotted as normal stress versus shear stress, thus from which effective angle and the effective cohesion can be acquired as indicated in Figure 3.7. Further, alteration of lateral displacement and shear stress may occasionally be plotted as well indicated in figure 3.8 (Abramson et al., 2002).

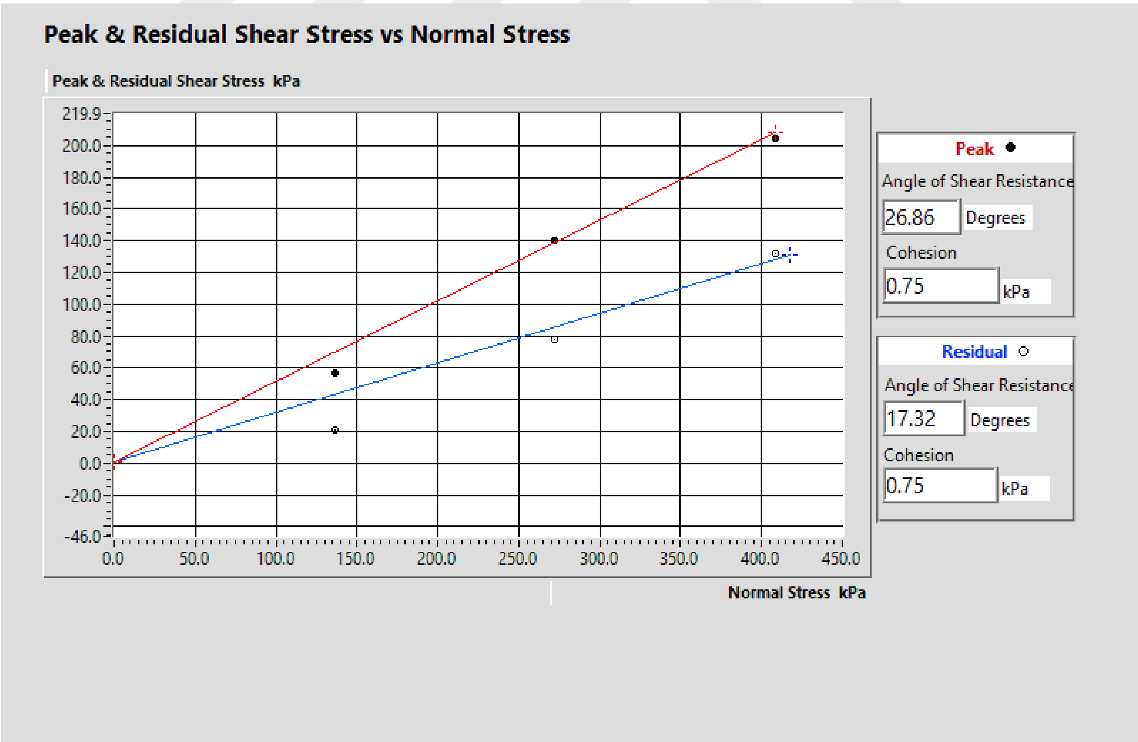


Figure 3.7: Typical direct shear results performed at three different normal loads.

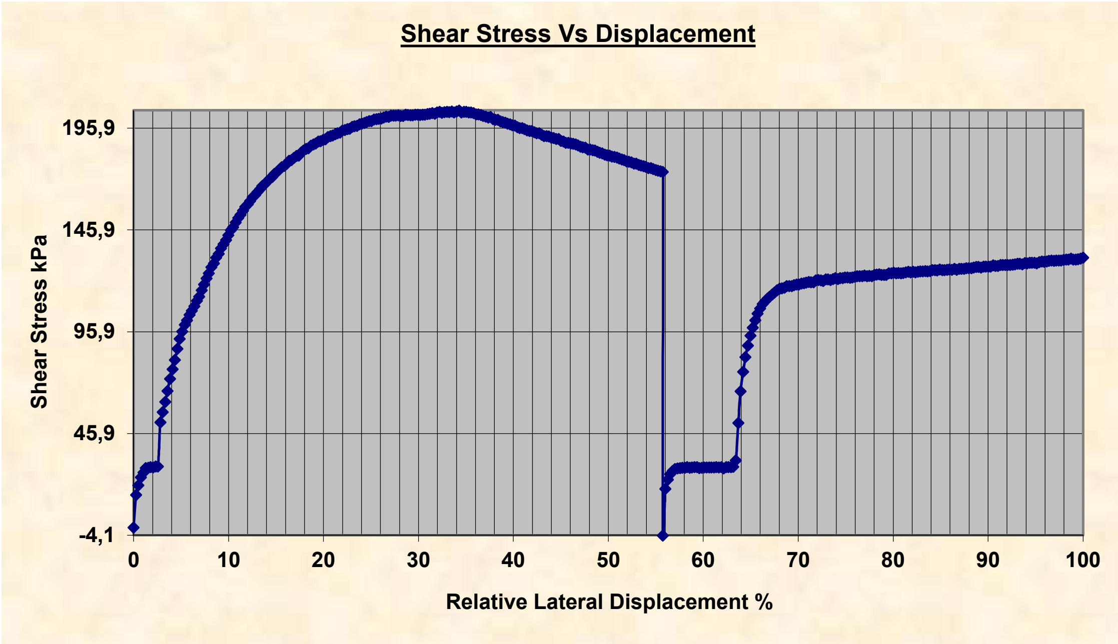


Figure 3.8: Examples of results of Shear stress vs displacement under certain load.

3.1.4. Consolidation Tests

Consolidation experiments are utilized to identify the compressibility(C_r and C_c), coefficient of consolidation (**c**$_v$), and preconsolidation pressures (σ'_{um}) for fine grained soils (Abramson et al., 2002).

Datas obtained from consolidation experiments are utilized to improve the stress history of the subsoils, and possibly to determine undrained shear strengths. Hence it is vital that the geotechnical engineer choose and average values for the expected stress scala (Transportation Research Board, 1975).

The Consolidation experiments are carried out too many specimens. Figure 3.9 indicates a photograp of a commercial consolidation testing device.

The outcomes of the consolidation experiment are frequently plotted as void rate versus applied pressure as indicated in figure 3.10.

Figure 3.9: A Consolidation testing device (Courtesy ELE Engineering, Inc.).

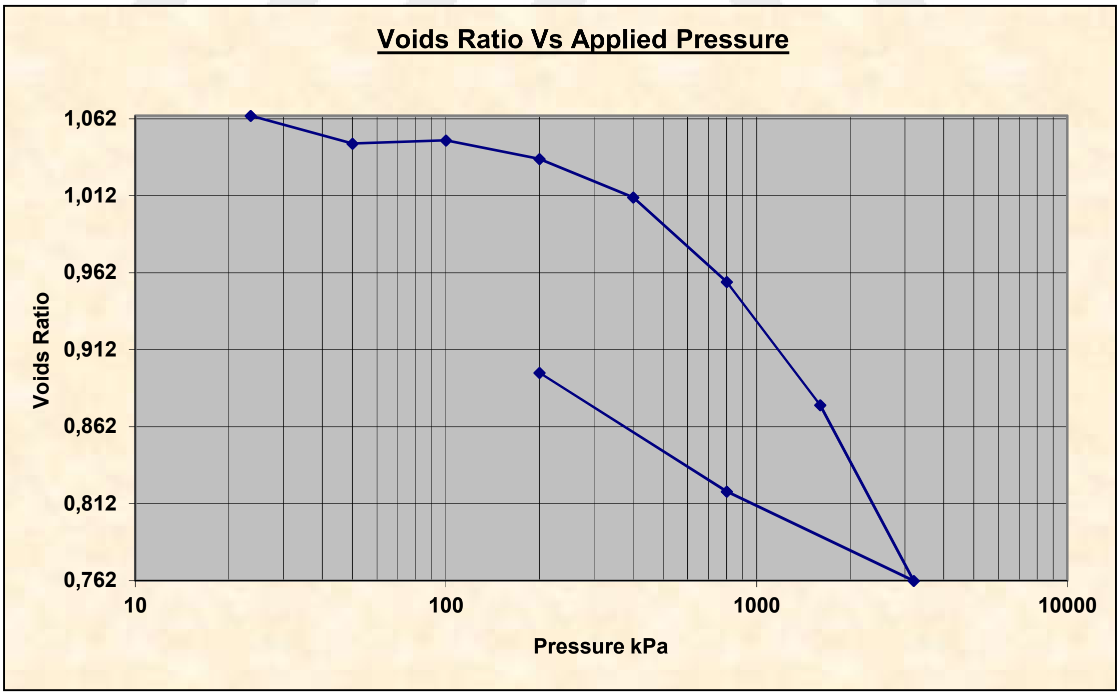


Figure 3.10: Examples of results of consolidation test under loading and unloading stages.

3.2. FIELD INVESTIGATIONS

In scope of the study, some field studies was carried out under limited circumstances in order to take results which are close to real situations. Firstly, two critical points selected in campus area, then conducted drilling up to 30 meters. Thus undisturbed samples collected via borehole. These boreholes are SK-1 and SK-2. Furthermore, inclinometer tubes placed up to 24 meters in borehole and fixed it with water cement mixture in order to provides us readings from real movement in soils as well as datas may be used in comparison with modelling results. In meantime, GPS measurements were taken to draw section of land for computer modelling.

3.2.1. Monitoring of land by Inclinometer

The long term monitoring of slowly flowing landslides which move smaller than 18mm in a year, are characterized by (Varnes 1978). This application can be carried out utilizing modern or traditional styles. Modern styles contain Optical Time Domain Reflectometry (OTDR), Interferometric Synthetic Aperture Radar (InSAR), Electronic Distance Meter (EDM) and Global Positioning System (GPS), (Gilli et al., 2000, Petley et al., 2005, Cappa et al., 2006). Conventioanl styles contain steel tape extensometers, wire or geodetic surveying, and rod dilatometers (Hartvich and Mentlík 2010, Hartvich et al., 2007).

Lateral deformation of the ground can be monitored by the inclinometer devices. The slope motions effect the part of the perpendicular casing in the borehole in case of motion throughout the landslide slipping surface. Hence, magnitude, depth and ratio of these movements may be defined by comparison the preliminary investigation datas with the following measures (Stark and Choi, 2008). In this study, placed inclinometer with a thickness of 5mm and an internal diameter of 7cm was planned and produced, as indicated in Figure 3.11. The readings were taken at 1m intervals of the borehole. Initial readings taken in 23.02.2016 and last readings taken in 13.05.16 (see table 3.1).

Table 3.1: Maximum displacements taken from inclinometer in 3 months.

ink. No	Maximum deformation (perpendicular to the direction of driling) mm
SK-1	-1,00
SK-2	2,40

The outcomes of the inclinometer readings are also graphs of inclonemeter were attached in appendix A.

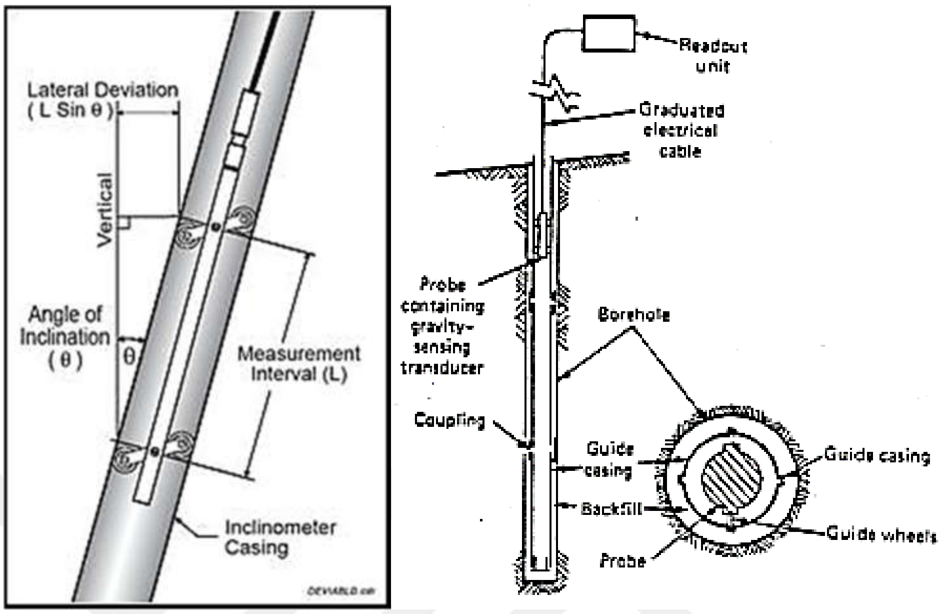


Figure 3.11: Inclinometer tubes and working mechanisms of reading devices.

Inclinometer can be located in slopes (see fig. 3.12) and in fields that are sensible to landslide and can allow to determine the sequence;

- Locate shear zones
- Measurement of movements
- Determine the speed of movement
- İdentify if shearing is occuring in plan or circular form.

Figure 3.12: Field studies when placing the inclonometer tubes.

3.2.2. GPS Measurements of Section of Lands

The GPS (Global Positioning System) is a space based navigation system which supplied position anyplace on the earth and time knowledge irrespective of how the weather is, where there is an unobstructed line of sight at least four or more satellites (Science Reference Service).

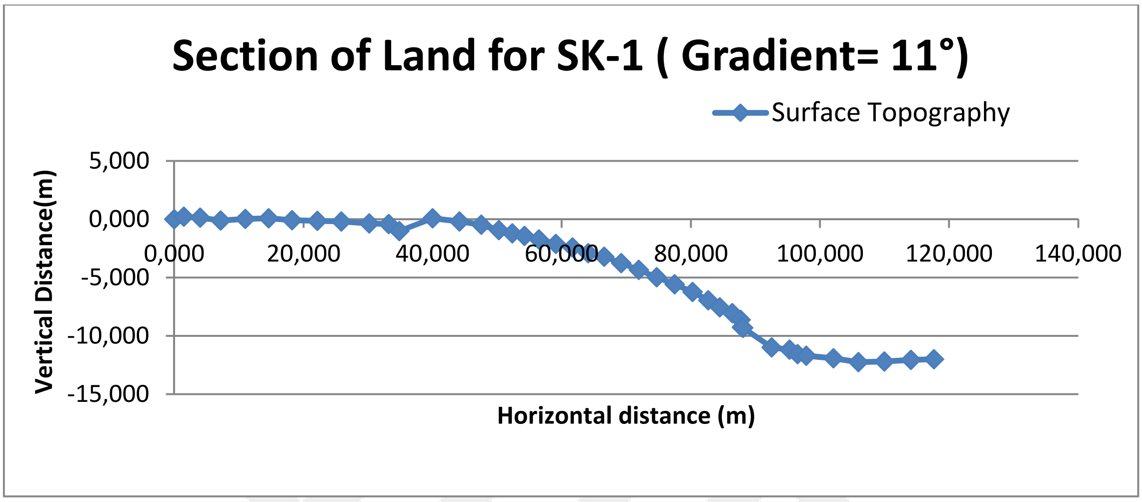


Figure 3.13: Graphs of slope section of SK-1 depends on GPS device.

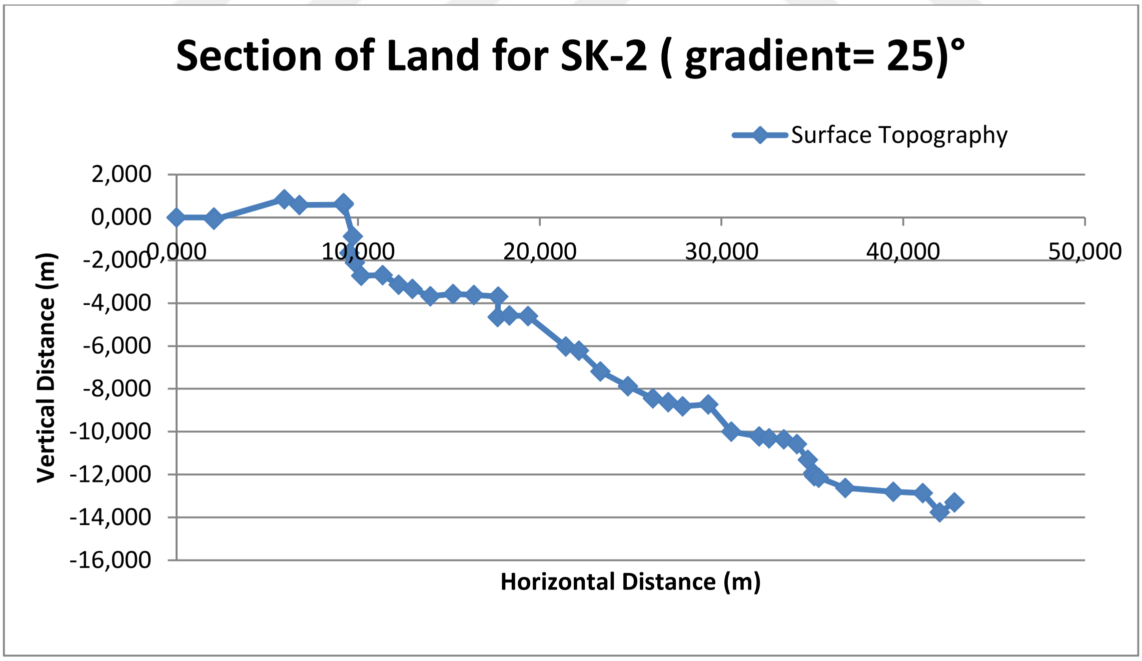


Figure 3.14: Graphs of Slope Section of SK-2 depends on GPS Device.

In this study, gps measurements were taken from slope in order to draw section of land. (see figure 3.13 and 3.14).

3.3. SOIL PROPERTIES AND GEOLOGY

The clay soil was modelled within this research in two types which are; dark yellow clay at the top and dark green clay at the bottom. According to derived results from laboratory test (see table 3.2). Both of them may be almost stiff and probably over consolidated. On the other hand, hard to say that field is best place for civilization. Because there are more inclination slopes and the climate of region is ardous. So some cases were searched which type of condition lead to unfavorable in these soils. For instance, many trials conducted as if land subjected to excessive precipitation or affected by seismic force.

Table 3.2: Basic properties existing dark yellow and dark green clays in Land.

Properties	Dark yellow clay	Dark Green clay
Natural water content, $w\%$	40	35
Plasticity index, I_p	40	40
Natural Unit Weight, γ (kN/m³)	17,80	17,50
Saturated Unit Weight, γ_{sat} (kN/m³)	19,00	19,00
Clay, Silt and Sand contents (%)	98	96

3.4. ANALYSIS OF SLOPE STABILITY

The detailed slope stability analyzes have been performed in the study areas (SK-1 and SK-2). The computerized modeling the section of zones which have landslide potential are analysed with software programs SLIDE and PLAXIS.

3.4.1. Limit Equilibrium Modeling with Slide

SLIDE analysis the stability of slipping surfaces utlizing perpendicular slice limit equilibrium methods. This techniques can be performed to place the critical slipping surface for a plotted slope. The minimum safety factors values obtained analysis based on Bishop and Janbu principles. Four different conditions are modeled depending on way failure direction.

3.4.1.1. Effective Stress Analysis (long term)

The effective shear strength parameters are thought to represent the behavior of the cohesive soils in the best way. Thus, c' and $\emptyset'$ values infered from shear box test, used in analyzed separately as peak and residual. The results obtained from tests are given in tables 3.3. In meantime, there is an existing building load assumed as 70 kPa and the stone wall load is below the slope assumed as 2 kPa (see Figure 3.15). Shows the section of SK-2 modelling. Furthermore one more analysis carried out with groundwater shown in figure 3.16 assumed that soil is fully saturated due to excessive precipitations.

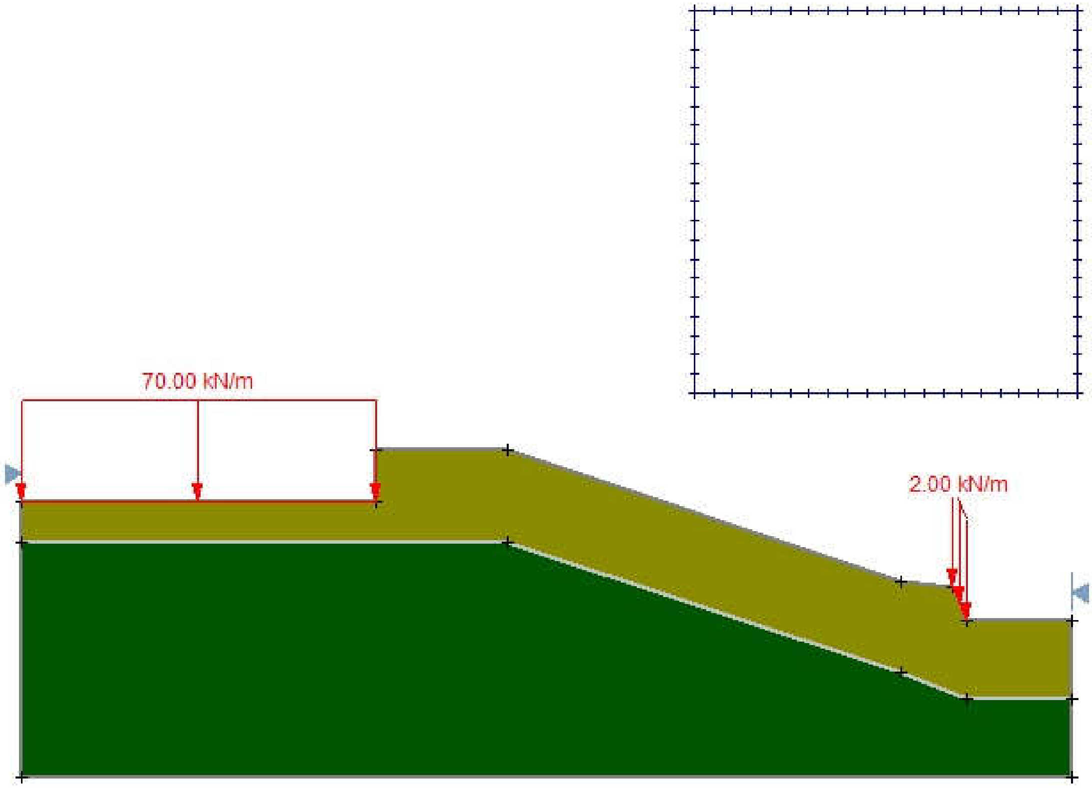


Figure 3.15: Sk-2 Effective Stress Analysis modelling without groundwater.

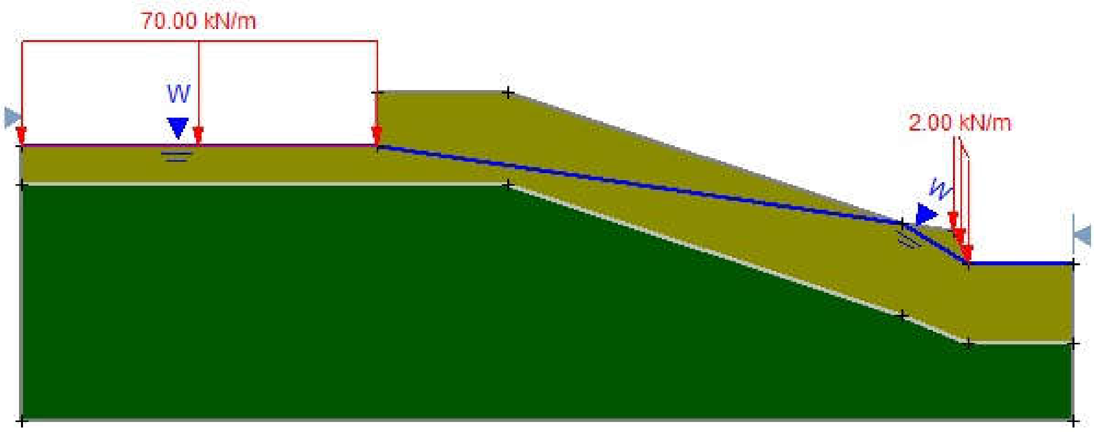


Figure 3.16: Sk-2 Effective Stress Analysis modelling with groundwater.

Table 3.3: Parameters obtained from shear box tests.

Depth (m)	Peak Values		Residual Values		Soil Model
SK-2	c' kPa	$\emptyset'$	c' kPa	$\emptyset'$	
2,70	30	16	-	-	Mohr-Coulomb
4,50	65	22	-	-	Mohr-Coulomb
7,20	0,75	27	0,75	17	Mohr-Coulomb

3.4.1.2. Total Stress Analysis (short term)

Using with undrained parameters for fine grained soil, drained parameters for coarse grained soils are suitable in dynamic analyses. (Day, 2002 “Geotechnical Engineering Handbook”). The maximum seismic coefficient assumed as 0,2 in situation with eartquake. Thus, c and ø=0 values infered from undrained triaxial tests. The average c values used in analyzed for each layer.

Results obtained from tests are given in Table 3.4. For SK-2, existing building load assumed as 70 kPa and the stone wall load assumed as 2 kPa. As for Sk-1,the adjacent building loads assumed such as; 30 kPa,10 kPa and 30 kPa respectively. The total stress analysed by computerized modelling in varied conditions. These models indicates eartquake effect for SK-2 shown in Fig 3.17 and without groundwater for SK-2 shown in Figure 3.18, as for SK-1 in case eartquake and groundwater effect shown in Figure 3.19 and without groundwater condition shown in figure 3.20.

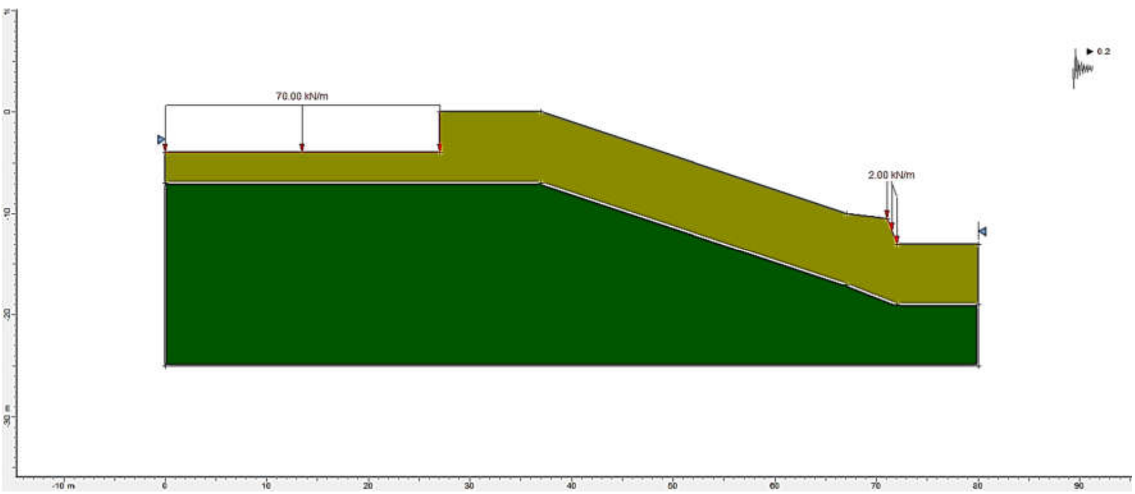


Figure 3.17: Sk-2 Total Stress Analysis modelling with eartquake effect.

Table 3.4: Parameters Obtained from Undrained Triaxial Pressure Tests.

Borehole	Depth (m)	c (kPa)	Soil type	Soil Model
SK-2	1,80	165	Dark yellow Clay	Mohr-Coloumb
SK-2	2,50	200	Dark yellow Clay	Mohr-Coloumb
SK-2	4,50	394	Dark yellow Clay	Mohr-Coloumb
SK-2	5,50	226	Dark yellow Clay	Mohr-Coloumb
SK-2	5,60	111	Dark yellow Clay	Mohr-Coloumb
SK-2	7,00	315	Dark green Clay	Mohr-Coloumb
SK-2	7,40	169	Dark green Clay	Mohr-Coloumb
SK-2	9,40	178	Dark green Clay	Mohr-Coloumb
SK-2	13,80	160	Dark green Clay	Mohr-Coloumb
SK-2	18,50	397	Dark green Clay	Mohr-Coloumb
SK-2	19,00	229	Dark green Clay	Mohr-Coloumb
SK-2	24,50	292	Dark green Clay	Mohr-Coloumb
SK-1	3,00	225	Dark yellow Clay	Mohr-Coloumb
SK-1	8,00	293	Dark green Clay	Mohr-Coloumb
SK-1	13,90	325	Dark green Clay	Mohr-Coloumb
SK-1	20,70	311	Dark green Clay	Mohr-Coloumb

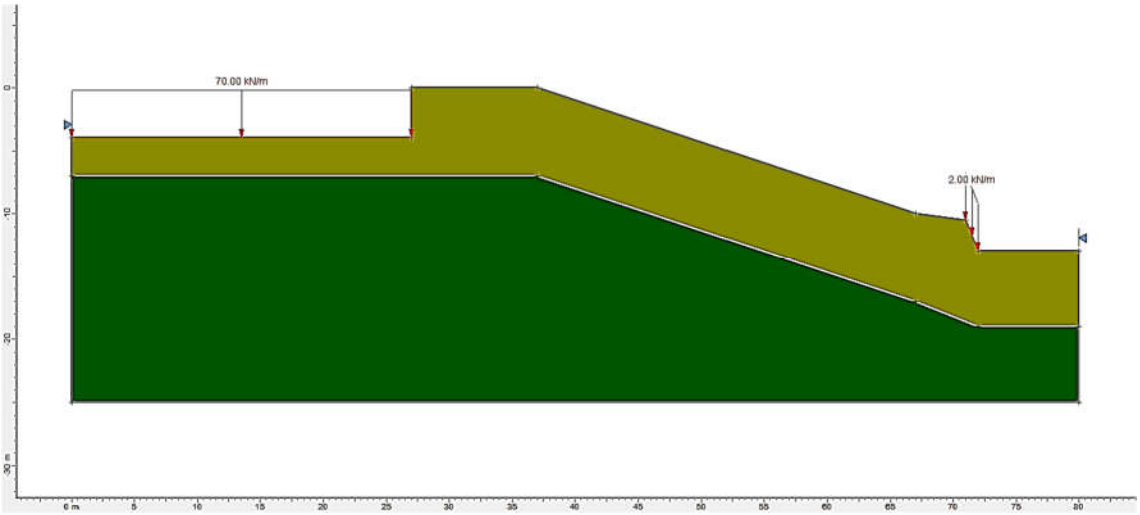


Figure 3.18: Sk-2 Total Stress Analysis modelling without groundwater.

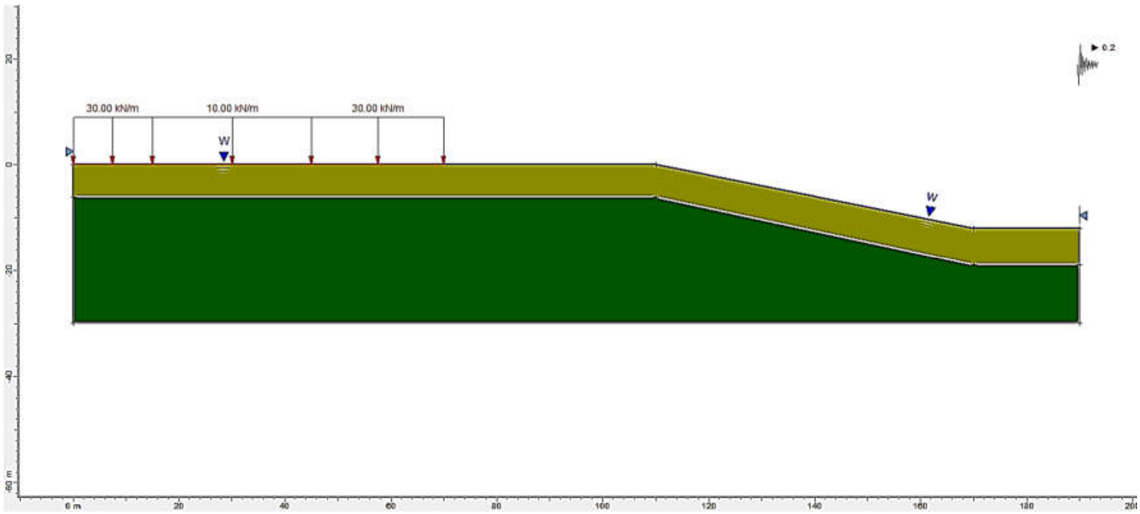


Figure 3.19: Sk-1 Total Stress Analysis modelling with groundwater and earthquake effect.

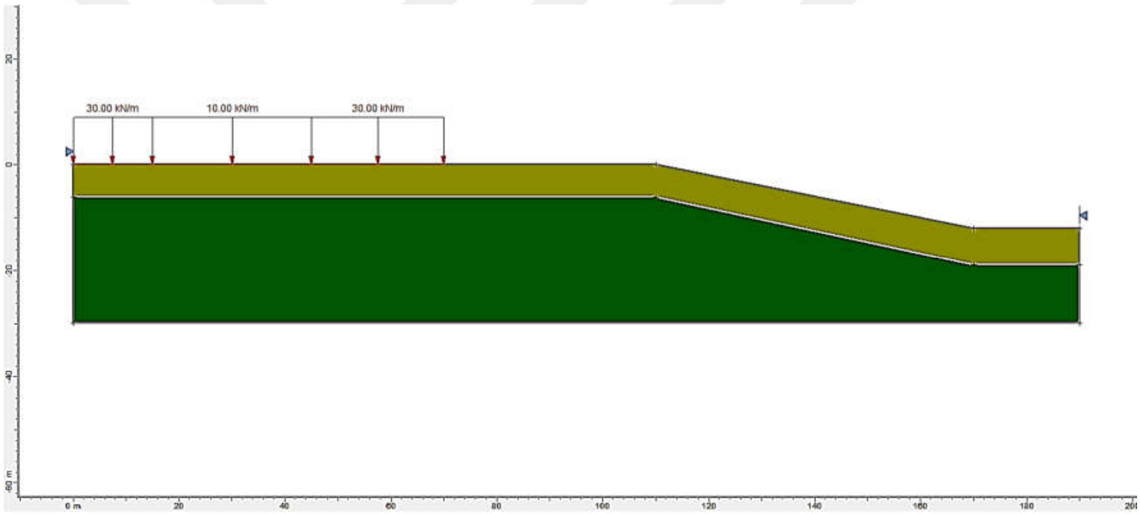


Figure 3.20: Sk-1 Total Stress Analysis modelling without groundwater.

3.4.2. Finite Element Modeling With Plaxis

The undrained soil is modeled by depending on plane-strain condition. For predicting the safety factor hardening soil (HS) models that were used in this study. Also (HS) model is used in order to simulate soil behavior to estimate the displacements. This model considers an advanced elasto-plastic soil model and it can be distinguished from (MC) model by its approach to stiffness. Moreover, by following this model, it’s possible to model the soil with more accuracy by depending on three different input stiffness on the contrary to (MC) model that focuses on one stiffness. These three different stiffness moduli are:

- E_{50}^{ref} : Secant stiffness in standard drained triaxial test, kN/ m².

- E_{ur}^{ref} : Unloading/ reloading stiffness, default ($E_{ur}^{ref} \approx 3 E_{50}^{ref}$), kN/ m².
- E_{oed}^{ref} : Tangent stiffness for primary oedometer loading, kN/ m².

As a result this model gives more convenient results which are closer to the real behavior of the soil.

In slope stability analyses are perfomed with finite elements method. c' and $\emptyset'$ which are strength parameter of soil, reduced in order to derive factor of safety, until it reaches collapse value. The evaluated safety factor is used to the obtained these reduced parameters.

This method based on Mohr-Coloumb failure criterion. The superiority is according to present slip circle methods; slipping is no obligation to take place in a circular plane.

3.4.2.1. Input phase of SK-1

The first step to proceed in the analysis in Plaxis program is to define the boundary conditions which are called as mesh generation in the program. Here the dimension of the mesh used is approximately 200 m x 30 m representing x-axis and y-axis coordinates respectively, and the elements in the mesh which being used here was triangular shape with 15-nodes. According to plaxis manual method B is used in this (HS) model analysis. Soil behavior is undrained and c and ø=0 values assumed for SK-1. Figure 3.21 and 3.22 shows the mesh boundary condition for slopes. Hardening soil parameters used in this study is shown in Table 3.5 and 3.7.

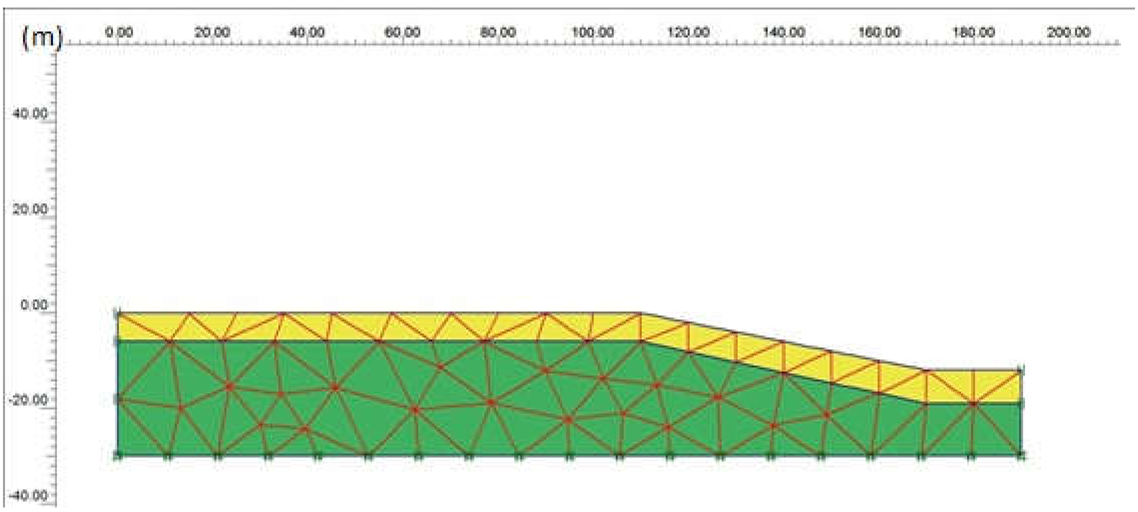


Figure 3.21: The Mesh Boundary conditions of SK-1.

Table 3.5: HS parameters for SK-1.

Parameters	Dark yellow Clay	Dark green Clay
E_{oed}^{ref}(kN/m²)	21460	32000
E_{50}^{ref} (kN/m²)	10000	16860
E_{ur}^{ref} (kN/ m²)	39100	67450
v_u	0.2	0.2
c '(kN/ m²)	70	0.75
Ø'	20	27

3.4.2.2. Calculation phase of SK-1

After finishing the first step of the program procedure that covers the soil properties and the mesh boundary condition, calculation process is followed. The initial stresses are generated by depending on total M weight = 0, in the calculation phase. Second step in this phase deals with load which comes from the soil weight. Significantly nearby building load was not taken into account due to the soil assumed as fully compressed in time.

Table 3.6: Calculation phases of SK-1 and SK-2.

Phase	Phase number	Phase start	Calculation type	Loading input
Initial phase	0	0	-	-
Gravity	1	0	Plastic	Total multipliers
existing field	2	1	Plastic	Staged construction
No Gwt analysis	3	2	Phi/c reduction	Incremental multipliers
Gwt on surface	4	3	Plastic	Staged construction

The next calculation step is no ground water analysis. Benefitions of this method were mentioned before in plaxis model section. After this step, the minimum safety factor can be derived for section. As for last step one more plastic analysis were conducted but in this step groundwater level was assumed on surface. Therefore there is an opportunity to realize displacements difference between gwt on surface or not. Table 3.6 shows the features of calculation phases.

3.4.2.3. Input phase of SK-2

The aim of following the input phase is to create the geotechnical, for creating the geometry condition, lines, points, and plate has been used. Also appropriate fixities of the profile is performed. Table 3.8 shows input parameters of RC members.

Table 3.7: HS parameters for SK-2.

Parameters	Dark yellow Clay	Dark Green Clay
E_{oed}^{ref}(kN/m²)	21920	12000
E_{50}^{ref} (kN/m²)	10000	7500
E_{ur}^{ref} (kN/ m²)	40000	22000
v_u	0.2	0.2
c '(kN/ m²)	70	0,75
Ø'	20	27

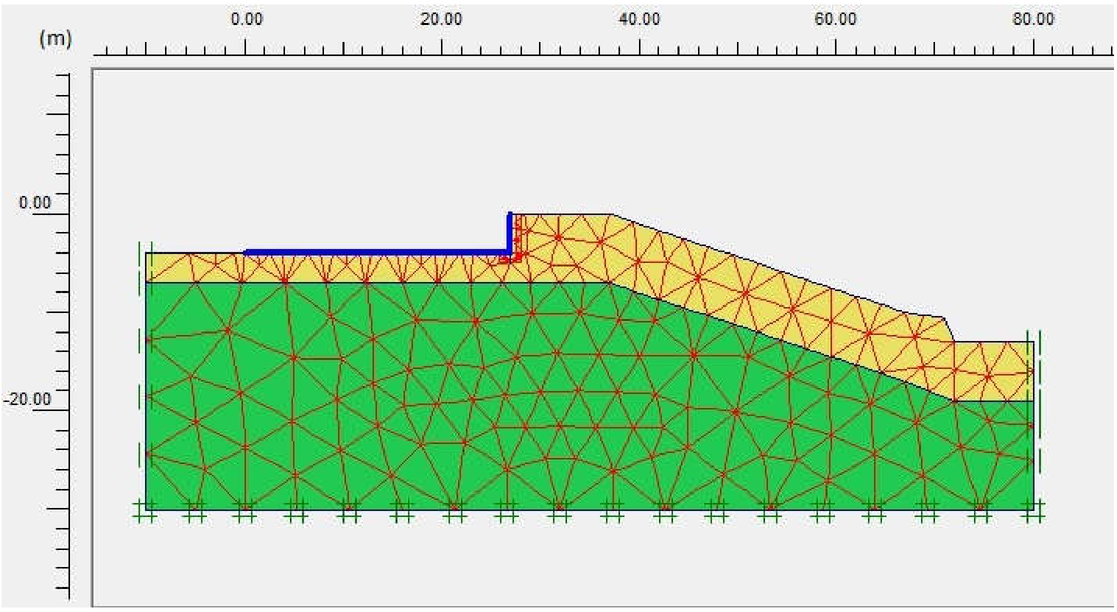


Figure 3.22: Meshed and Boundary conditions of SK-2.

Table 3.8: Input parameters – Plate.

Parameters	Plates
Elastic modulus of concrete, E_c (mN/ m²)	25
Axial stiffness, EA (mN/m)	15000
Flexural rigidity, EI (mN m²/m)	313
Weight, w (kN/ m²)	2.0
Poisson ratio, v	0.2
Final lining, d (m)	0.5

3.4.2.4. Calculation phase of SK-2

In Plaxis program the calculation phase is a main part of the whole simulation process which is performed after the analysis of the model formation. Same stages were used in SK-1 calculation is compatible for this analysis.

4. RESULTS AND DISCUSSION

4.1. SLIDE INTERPRET

Soil condition in the field were examined with different approaches. The infered safety factors in all of calculations were higher than proposal value of TS 8853 Slope Stability and Evaluation Methods. The residual strength condition of soils which is only transpired under large deformations is most unfavorable approach. FS is for stability SK-2 in analysis using residual strength parameters; FS(residual)=1.56(Janbu Simplified)>1.20 (TS 8853), shown in Figure 4.9. FS(residual)= 1.72(Bishop simplified)>1.20 (TS 8853), shown in Figure 4.8. As for that, in analysis using the peak effective parameters; FS(peak)=2.60>1.20 (TS 8853), shown in Figure 4.6.

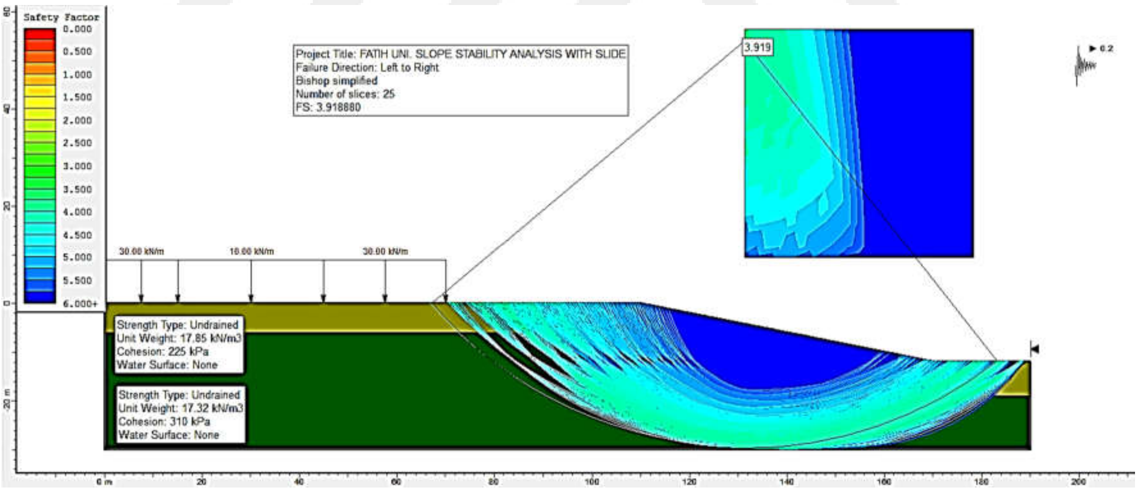


Figure 4.1: Total stress analysis of SK-1 with earthquake effect.

In the performed analysis which assumed total stress conditions, FS is for stability of SK-1, FS(tsa)=8.61>1.80 (TS 8853), shown in Figure 4.2. FS is for stability of SK-2, FS(tsa)=5,73>1.80 (TS 8853), shown in Figure 4.5. In the active earthquake analysis when the maximum ground acceleration is assumed as 0.2g. FS is for stability of SK-1, FS(eq)= 3.92>1.20 (TS 8853), shown in Figure 4.3. Although there is no encounterd groundwater in this study, assumed that soil is fully saturated due to excessive precipitations, shown in Figure 4.1. FS is for stability of SK-2, FS(eq)= 3.37>1.20 (TS 8853), shown in Figure 4.4.

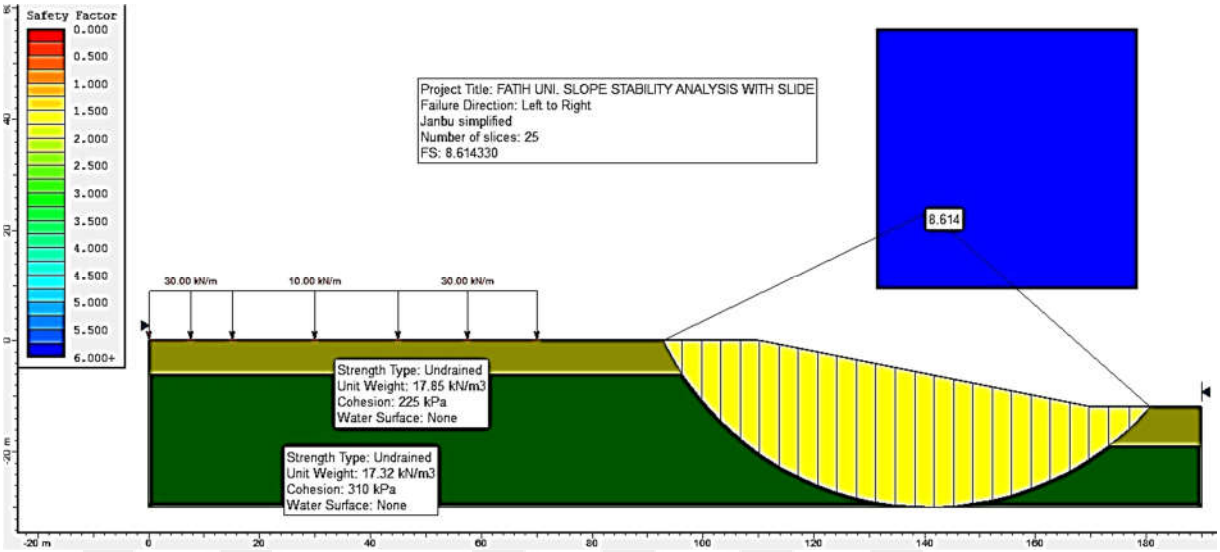


Figure 4.2: Results of slope stability analysis of SK-1 (Total stress - Janbu simplified).

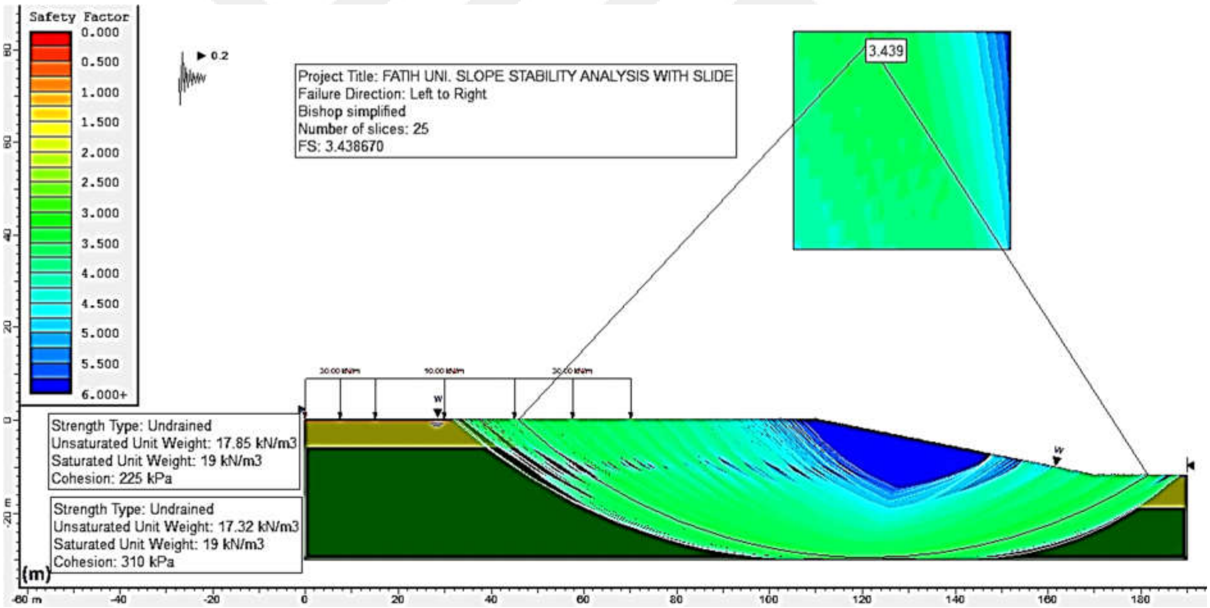


Figure 4.3: Total stress analysis of SK-1 with earthquake effect (soil is assumed saturated).

To see the critical points in the whole section, effective normal stress and shear strength & distance graph were drawn by plotting grapher of SLIDE shown in Figure 4.7 and 4.10.

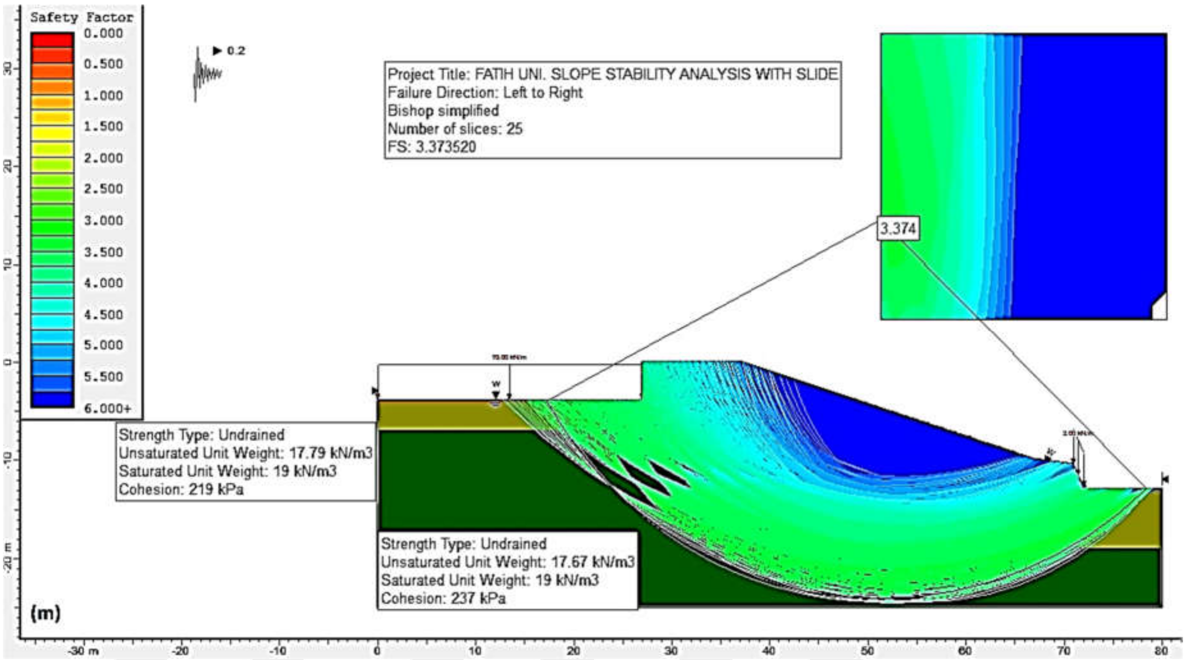


Figure 4.4: Total stress analysis of SK-2 with earthquake effect.

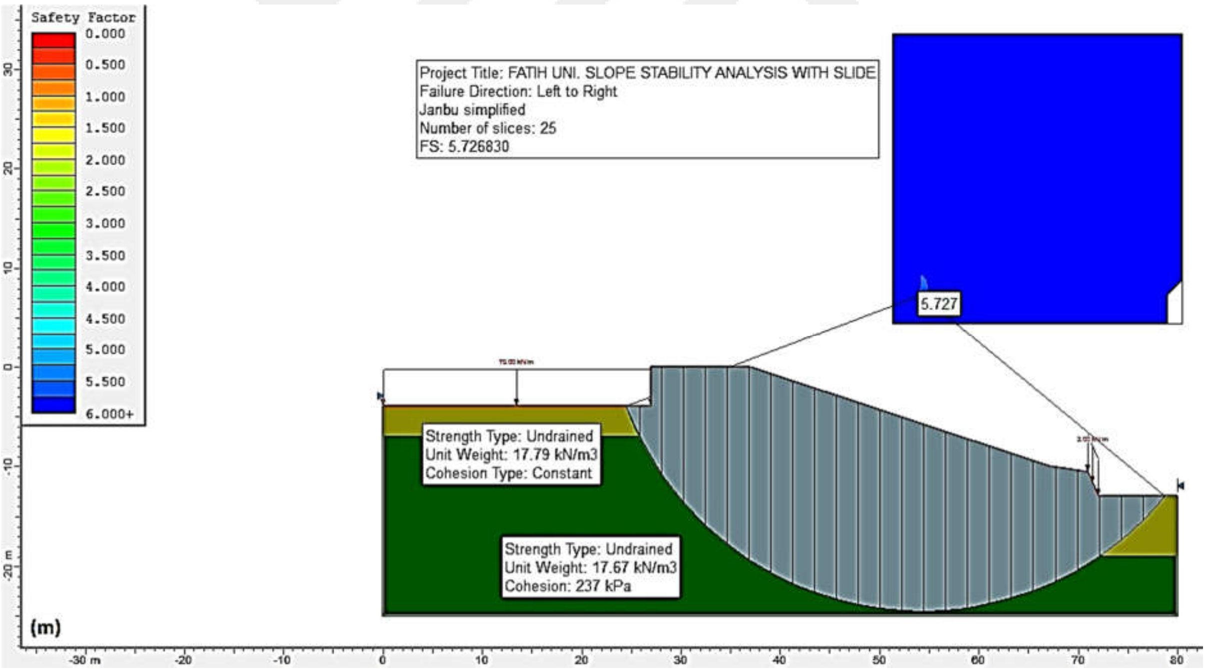


Figure 4.5: Results of slope stability analysis of SK-2 (Total stress - Janbu simplified).

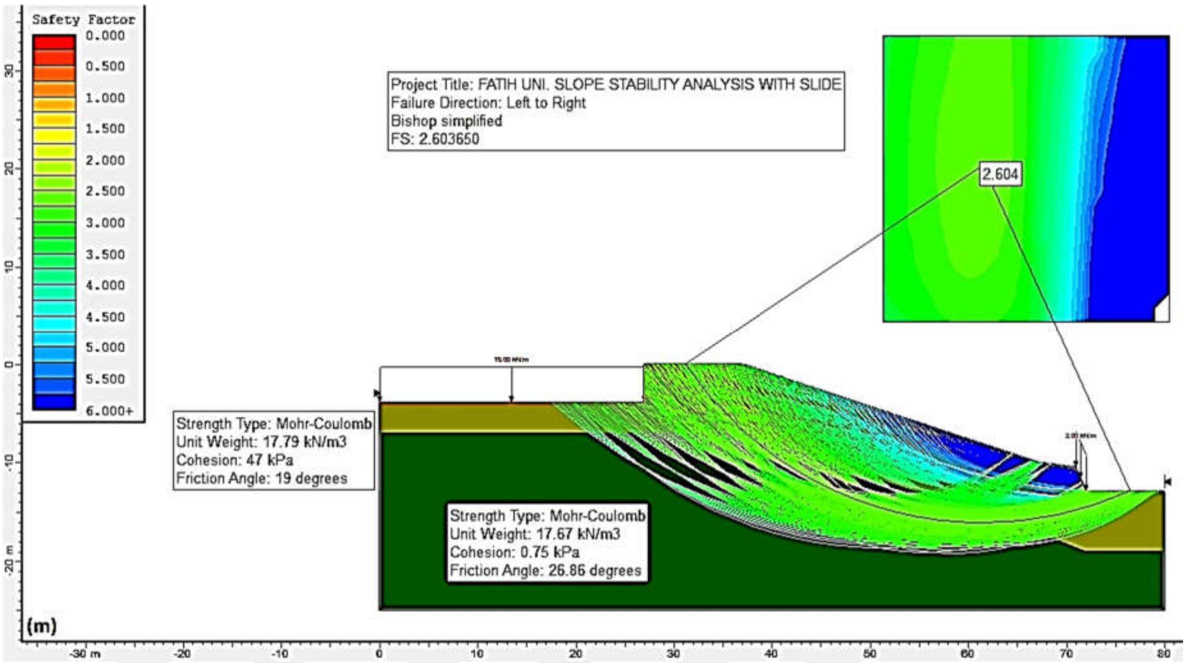


Figure 4.6: Results of slope stability analysis of SK-2 (peak strength - Bishop simplified).

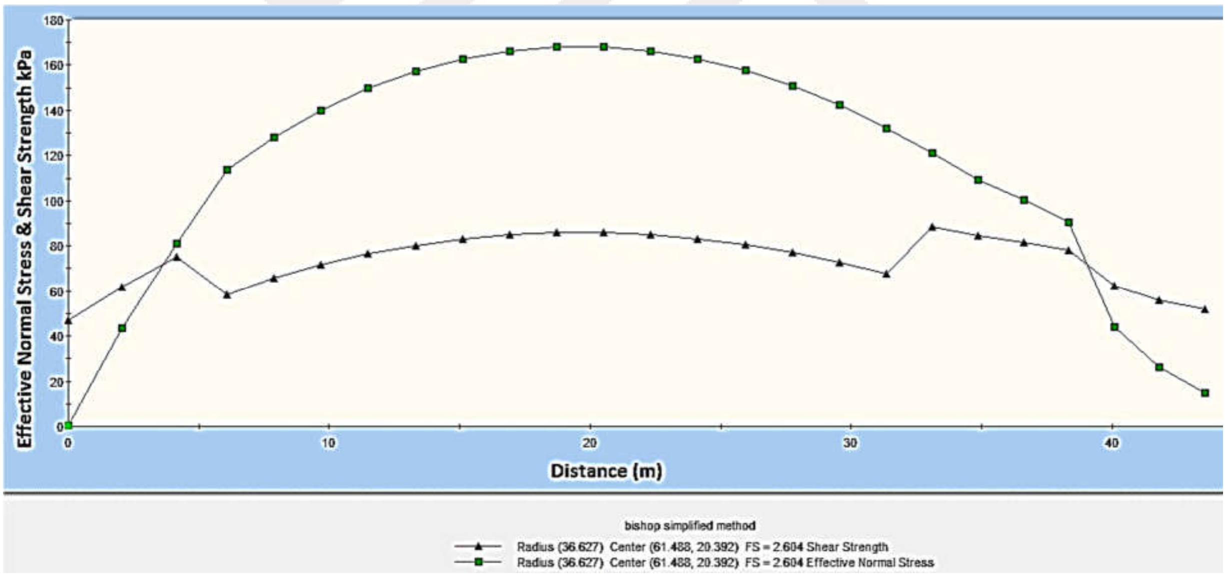


Figure 4.7: Results of slope stability analysis of SK-1 (effective normal stress and shear strength & distance graph - Bishop simplified-peak).

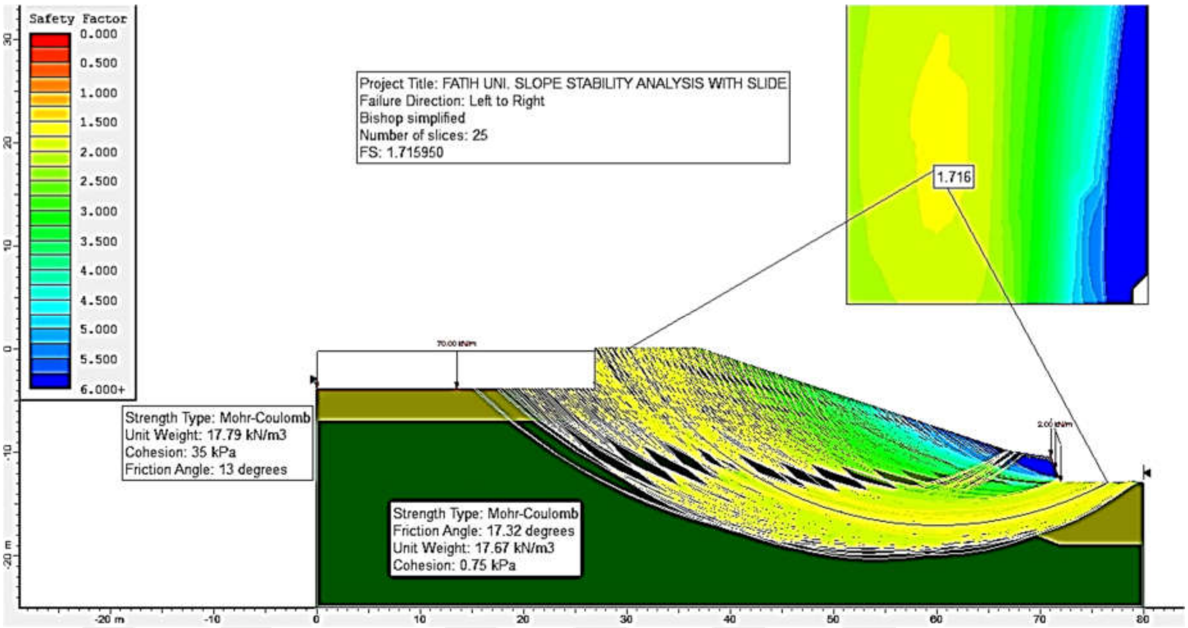


Figure 4.8: Results of slope stability analysis of SK-2 (Residual strength - Bishop simplified).

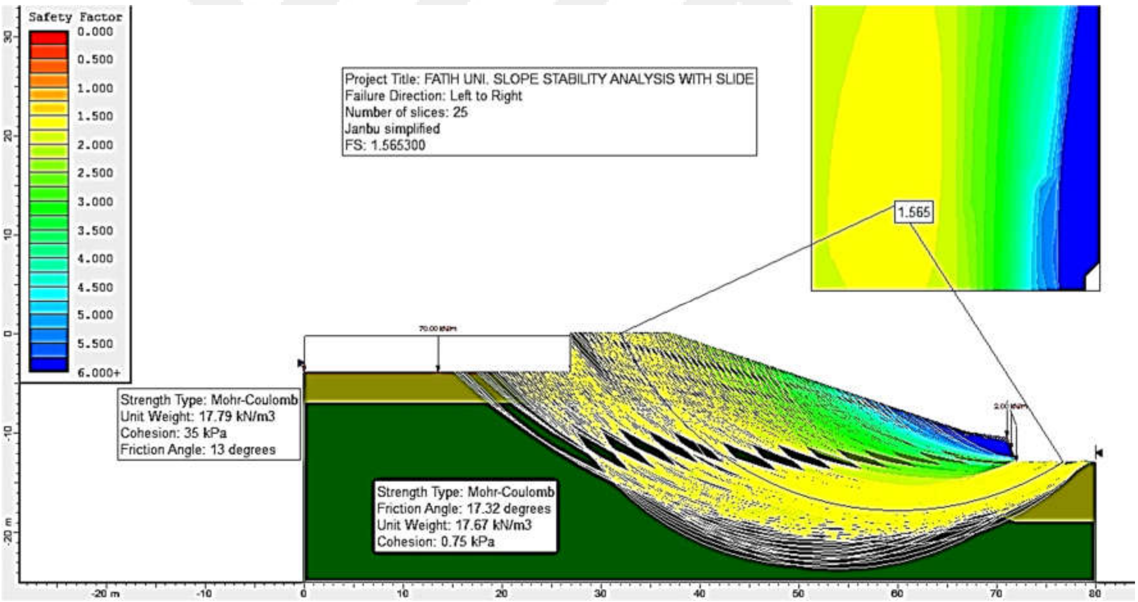


Figure 4.9: Results of slope stability analysis of SK-2 (Residual strength - janbu simplified).

In the lights of the analysis, it can easily be said that the slope has a significant stability.

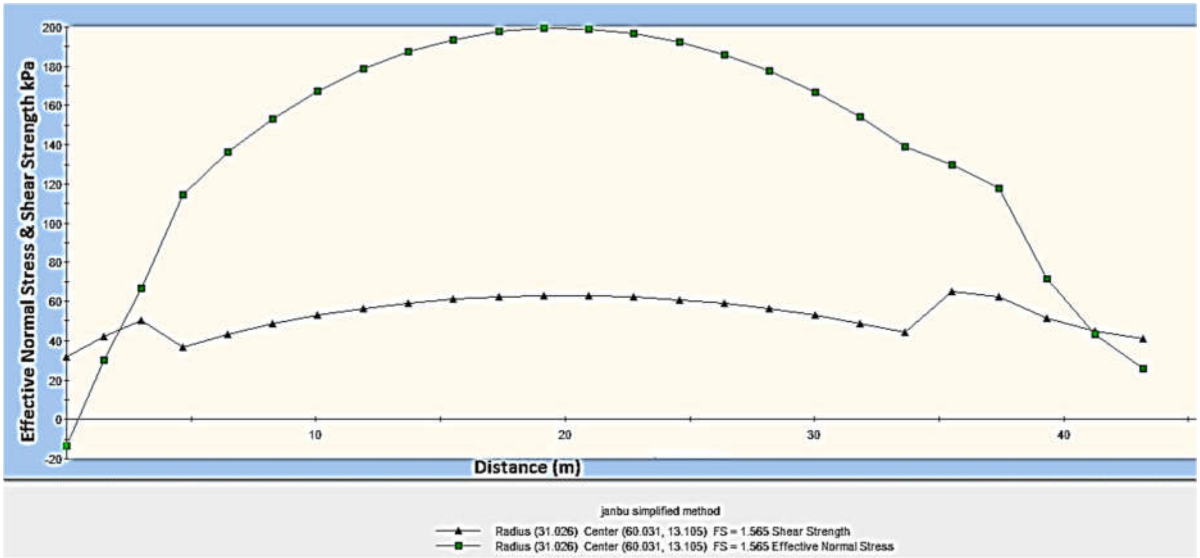


Figure 4.10: Results of slope stability analysis of SK-2 (effective normal stress and shear strength & distance graph - Janbu simplified-residual).

4.2. PLAXIS OUTPUTS AND RESULTS

4.2.1. Model Statement

SK-1 and SK-2 Models that is used in the Finite Element analyses is hardening soil model which considers more realistic behavior of the soil because it deals with three different kind of stiffness. Anaylsis were performed by PLAXIS in terms of effective stresses. For each profile different effective strength parameters were used as mentioned before in order to be able to specify the critical slip surfaces of slope at the same time to get the total incremental displacements that occurs in short term and long term.

4.2.2. Output Phases of SK-1 and SK-2

Among a lot of different output options which can be easily taken from Plaxis program is the stability of the slope. This study focuses on safety factor and the total incremental displacement that occur in time. Particularly, many trials are conducted to see horizontal displacement when soils are fully saturated (see Figure 4.15 and 4.16). A longitudinal sections of the profile zone has been taken in this research to be able to get ultimate horizontal displacements in order to compare displacements readings taken from inclonemeters. (see Figure 4.12). At the same time, A-A sections are shown in Figure 4.11 and 4.16 the SK-1 and SK-2 place respectively.

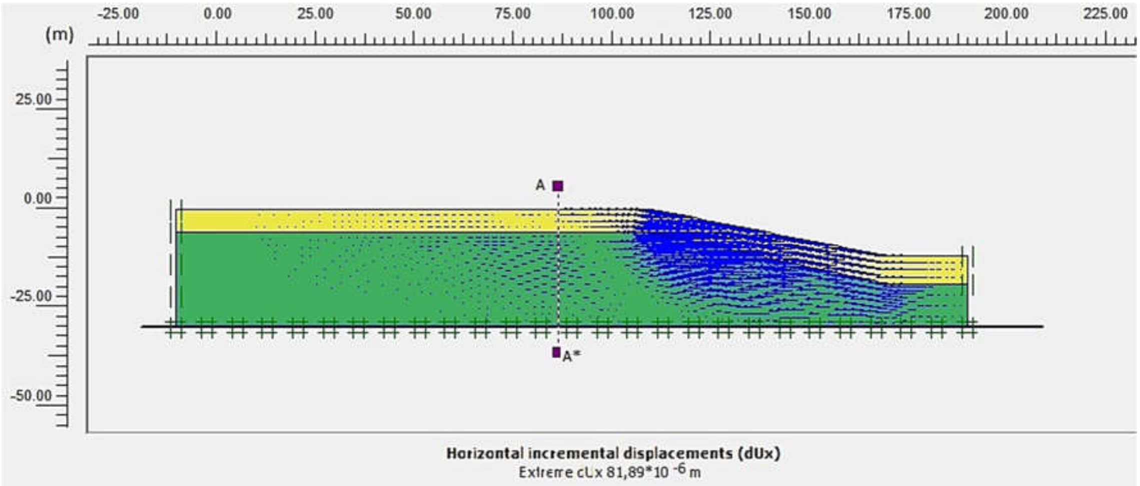


Figure 4.11: Results of slope stability analysis of SK-1 (Horizontal incremental displacements).

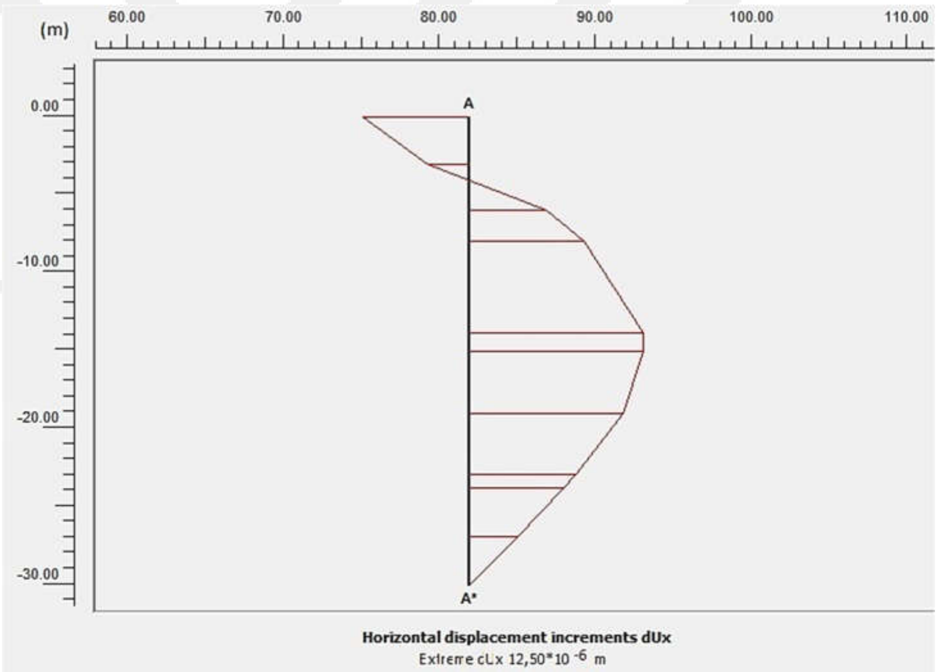


Figure 4.12: Results of slope stability analysis os SK-1 (Longitidunal section of SK-1).

Supplemental displacements are generated during a phi/c reduction computation. The total displacements do not have a physical meaning, however the increasing displacements in the failure and final step give a signal of the probably failure mechanisms. The concluding graphs appear a well impression of the failure mechanism of SK-1 and SK-2 (see Figure 4.13 and 4.18 respectively). The magnitude of the displacement increases is not related.

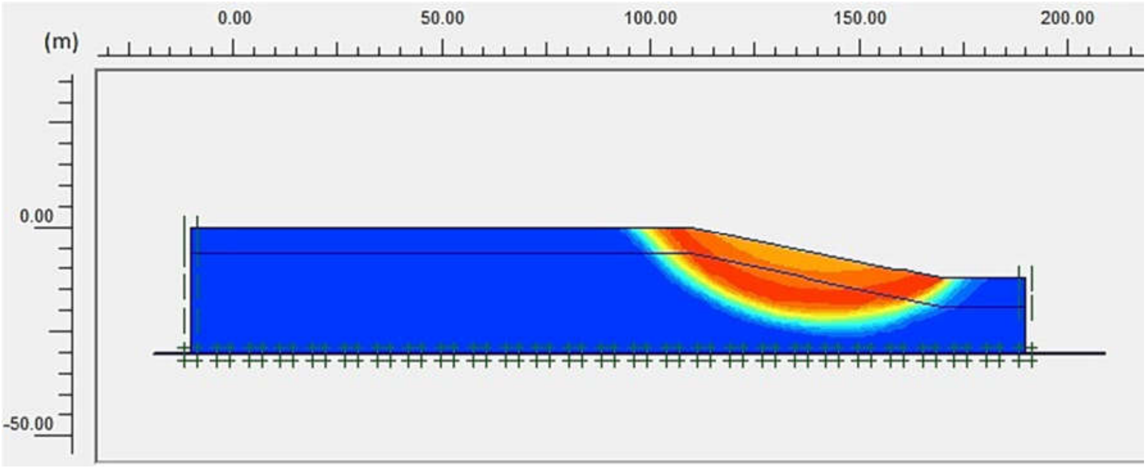


Figure 4.13: Shadings of the total displacement increments indicating the most applicable failure mechanism of the SK-1 in the final stage.

The safety factor can reliably be evaluated by means of plotting a curve where Σ Msf the parameter is sketched counter the displacements of a definite node. Even though the displacements are not convenient, they show whether or not a failure mechanisms have improved (see figure 4.14 and 4.19).

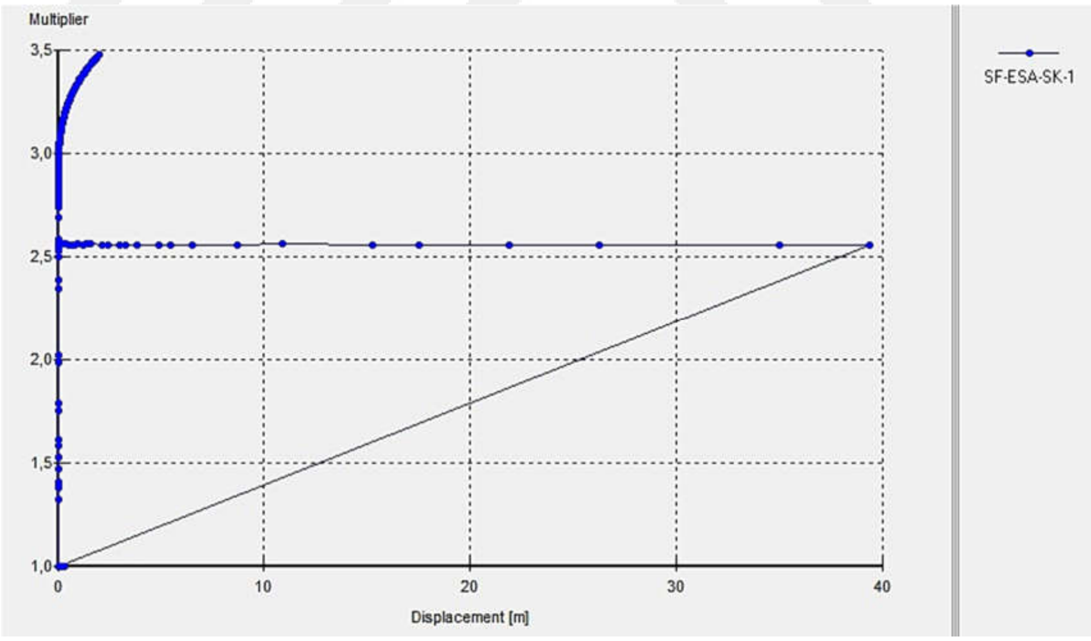


Figure 4.14: Evaluation of safety factor of SK-1.

Figure 4.17 shows the ultimate displacement under existing circumstances for A-A longitudinal section of SK-2.

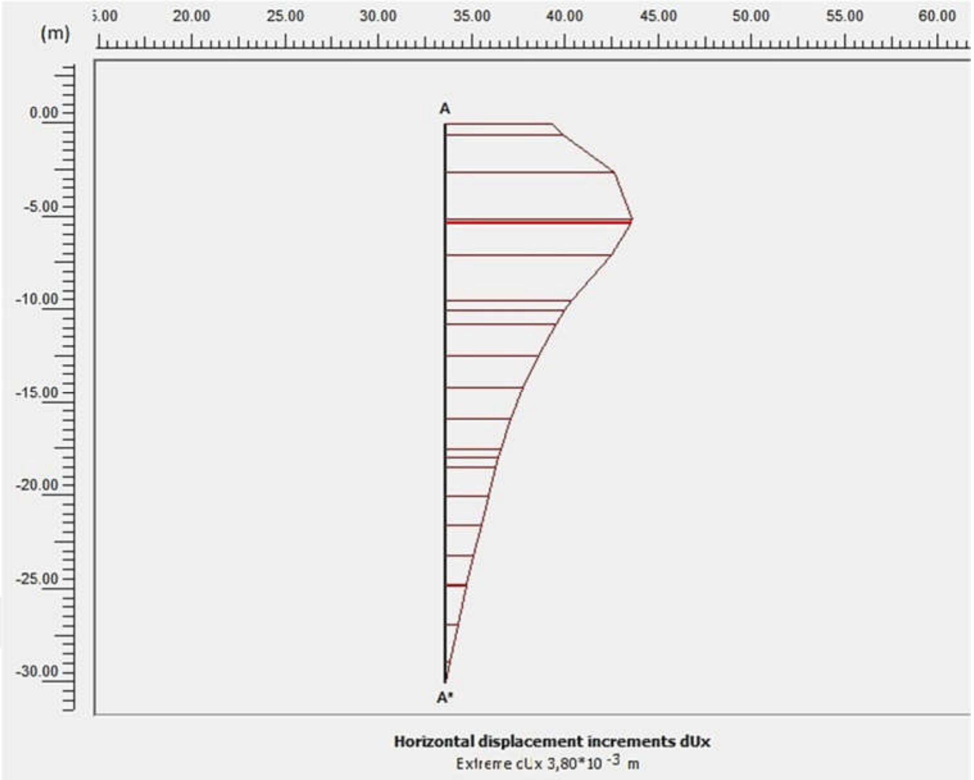


Figure 4.15: Results of slope stability analysis of SK-2 (Horizontal displacements of longitidunal section with GWT).

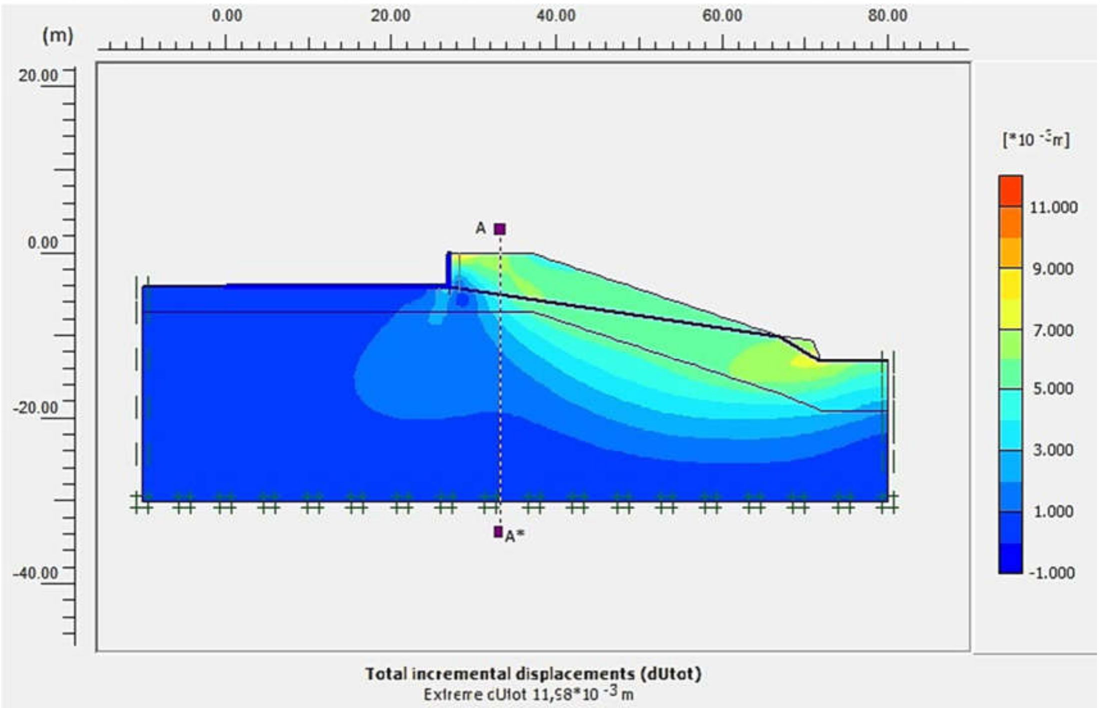


Figure 4.16: Results of slope stability analysis SK-2 (displacements of slope).

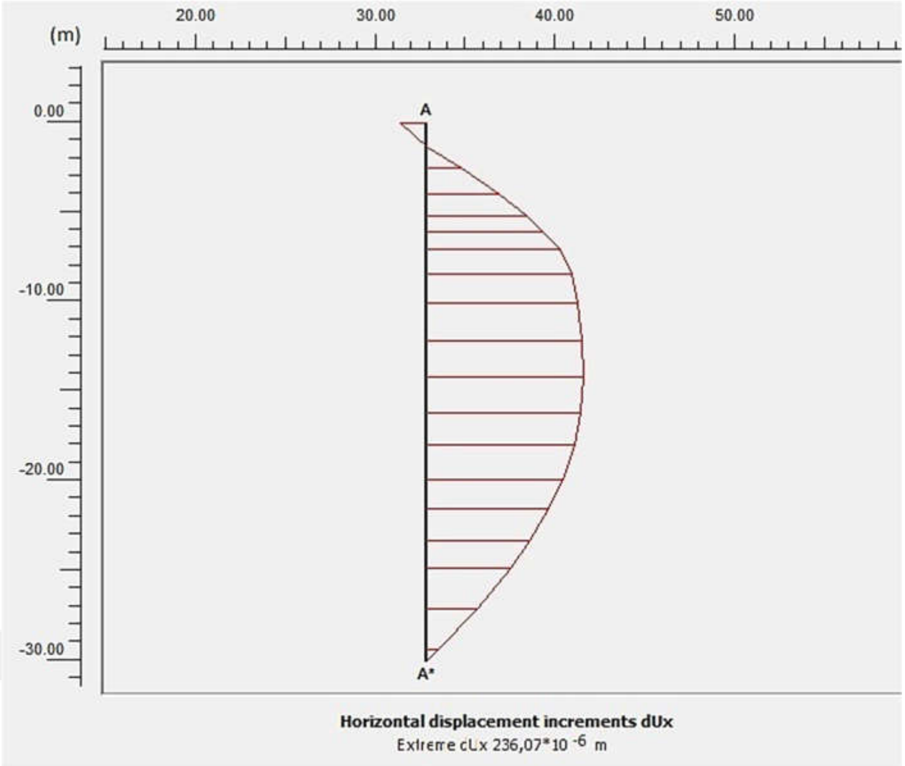


Figure 4.17: Results of slope stability analysis SK-2 (Horizontal displacements of longitidunal section- no GWT).

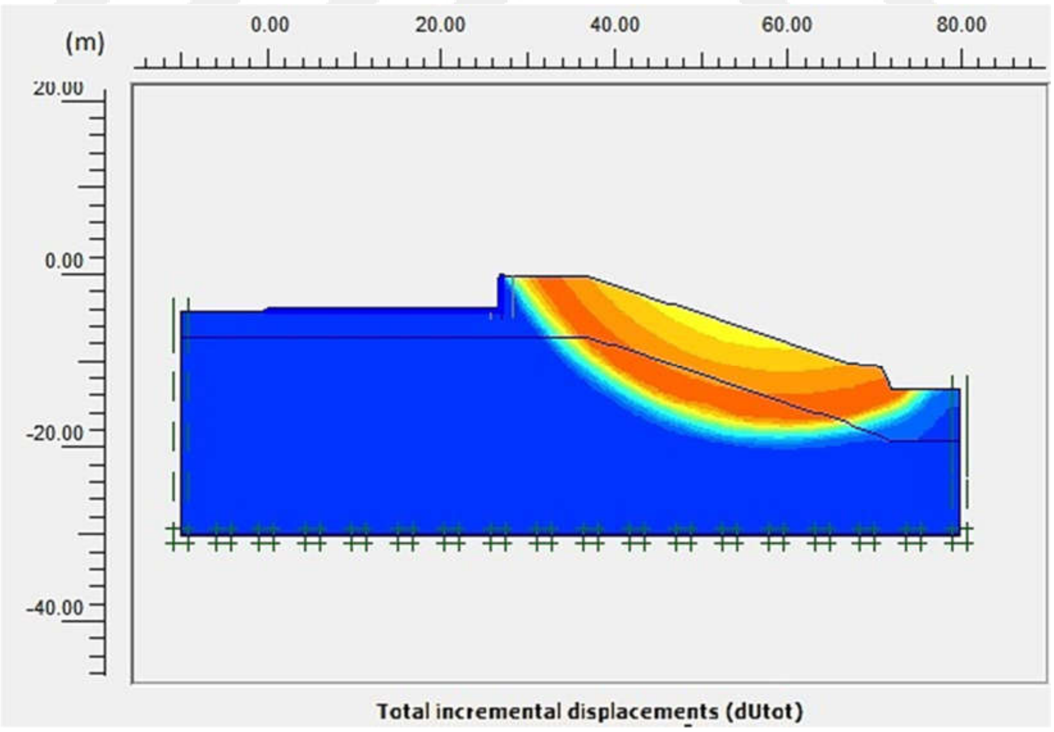


Figure 4.18: Shadings of the total displacement increments indicating the most applicable failure mechanism of the SK-2 in the final stage.

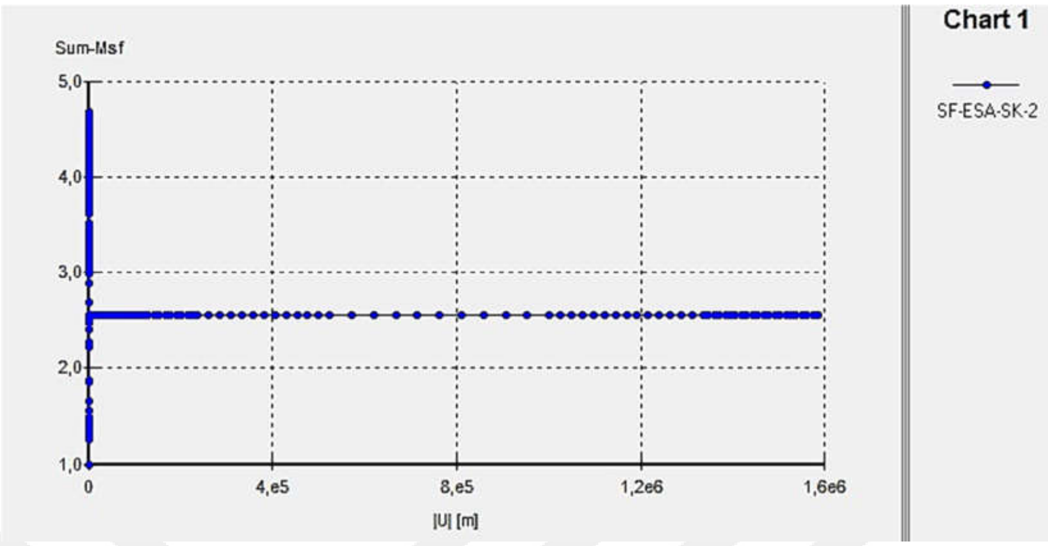


Figure 4.19: Evaluation of safety factor of SK-2.

It is also found that for all the cases that are examined, the slope stability is present under all circumstances taken into account.

4.2.3. Comparison Between Analysis And Inclinometer Readings

Displacements according to analysis were obtained from mentioned above analysis methods by computerized modelling with FEM and Limit equilibrium. Opposite results were also obtained from field investigations (i.e. inclonemeter measurements). Thus good alternative is registrated that can be compared real and theorical outcomes about investigation of landslides by means of this study. Table 4.1 shows the analysis results and readings taken from inclinometer tubes.

Table 4.1: Displacement discussion between analysis and real situation.

	SK-1	SK-2
Minimum Safety factor	2,50	1,56
Maximum displacement (mm)	0,08	12,00
Maximum horizontal displacement for A-A longitudinal sections ((mm)	0,01	3,80
Max. Inclinometer Readings (mm)	1,00	2,40

Slopes have stability in terms of both numerical analysis methods. The ultimate displacement results of plaxis analyze are close to real situation for SK-2. Also, slope movements may able to be reached 12mm according to analysis. It is quite sensible results when compare the inclinometer readings. On the other hand, maximum displacement of readings higher than the ultimate displacements of analysis for SK-1. Although this is the suspicious result, movements are generally negligible under natural circumstances.

5. CONCLUSION AND RECOMMENDATIONS

The movements that occur on the slope caused by unstability problem and other effects are considered as an important issue for humanity. A small amount of slope movement is not unexpected for steep slopes or unfavorable geology condition. Therefore; the best way is understanding their mechanism and handle it all of aspects. In the scope of this study, empirical and numerical investigations were tried to support on relationship between reason and result. The parameters that were obtained from different methods helped the researcher to understand the effect on the slopes and how the soil behaves against it.

Various trials have conducted with different parameter, in these cases give an awareness about how the clayey soil behave failure. While these cases were searched in a few software programs for both models it showed that modellings used with effective strength parameter give more compatible results. Consequently, it can be probable to evaluate the strength parameters of the slide surface owing to simulation of surveyed displacements.

Further research is required on clayley soils in field in order to get accuracy behavior. In this study, selected lands investigated different methods. Consequently, there is no big difference between numerical and empirical approaches. Moreover, some cases such as; precipitation or earthquake etc. taken into account. Eventually, providing that uncontrolled excavation is not performed in the area or soil is not subjected excessive loading, the slope stability is present.

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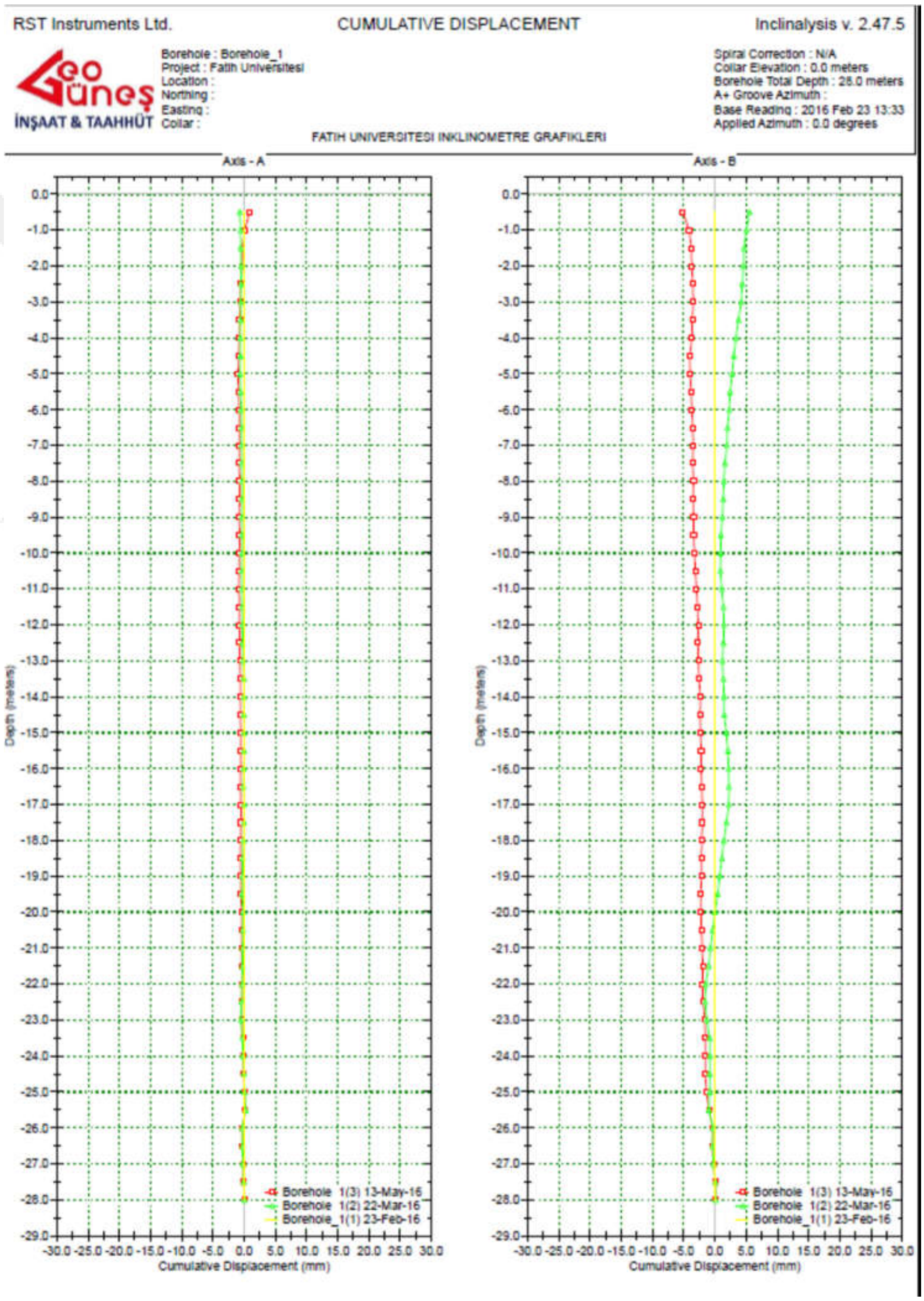
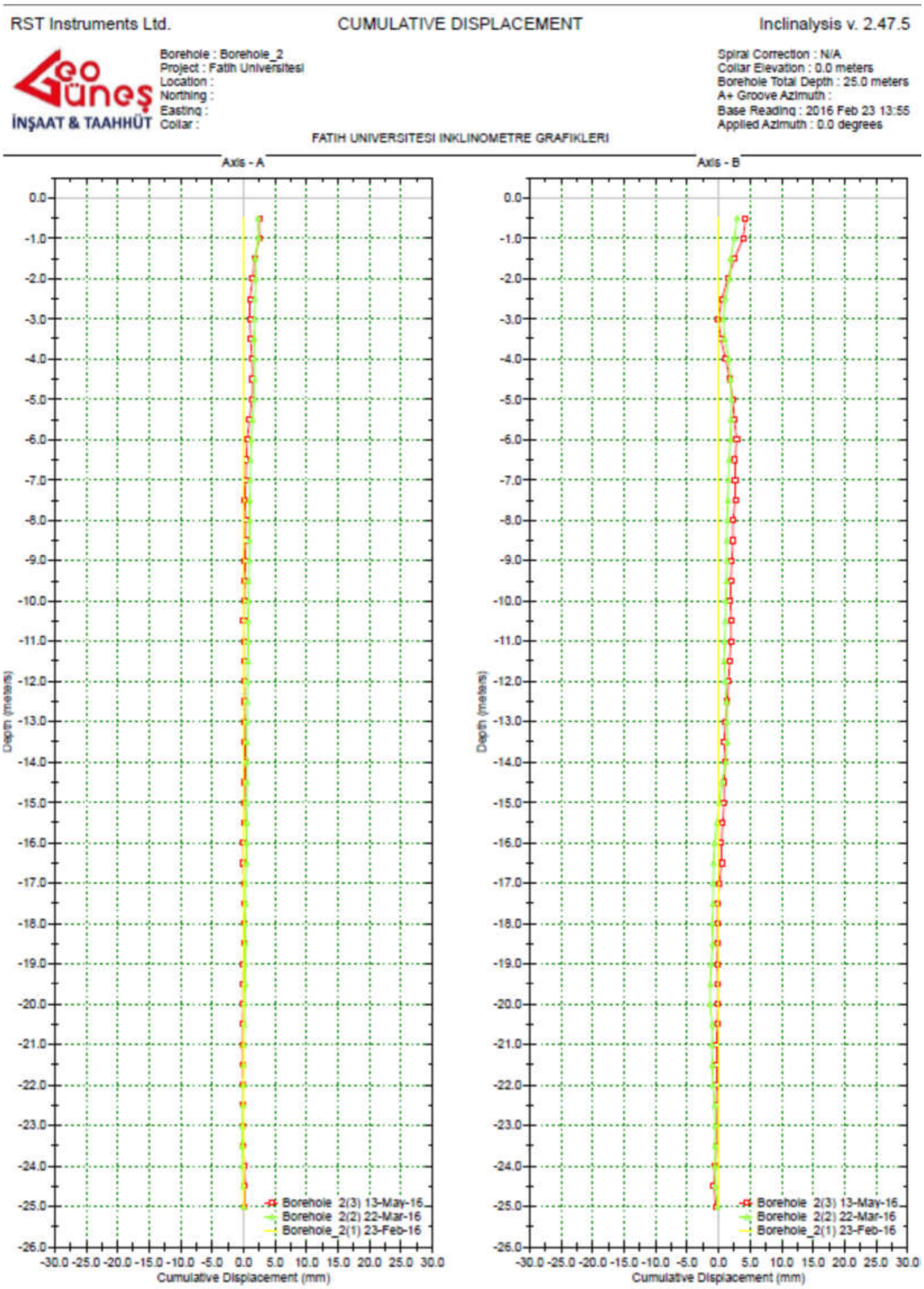
APPENDICIES

APPENDIX 1: Graphs of Inclonometer for SK-1.



APPENDIX 2: Graphs of Inclonometer for SK-2**.**



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