ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

SLOPE SAFETY ANALYSIS USING FINITE ELEMENT METHODS

M.Sc. THESIS

Behram KARAPAPAK

Department of Civil Engineering

Soil Mechanics And Geotechnical Engineering Programme

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Thesis Advisor: Prof. Dr. Ayfer ERKEN

İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

SONLU ELEMANLAR YÖNTEMİYLE ŞEV STABİLİTESİ ANALİZİ

YÜKSEK LİSANS TEZİ

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Zemin Mekaniği ve Geoteknik Mühendisliği Programı

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To my family,

FOREWORD

I thank to my advisor Prof. Dr. Ayfer ERKEN for helping my thesis and in my university life.

May 2019 Behram KARAPAPAK

(Civil Engineer)

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ABBREVATIONS

FHWA **:** Federal Highway Administration
FOS **:** Factor Of Safety
LEM **:** Limit Equilibrium Method
FEM **:** Finite Element Method

SYMBOLS

t	**:** Time
α	**:** Angle of deviation from the direction of the principal stresses
ρ	**:** Density
K	**:** Impermeability
E	**:** Elasticity Module
s	: Existing shear stress
τ	: Shear Strength Required
$\mathbf{C_a}$	: Adhesion
$\mathbf{N_i}$	: Vertical reaction
$\mathbf{U_i}$	: Pore pressure of water
$\mathbf{\Delta l_i}$	: The arc length at the I section

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SLOPE SAFETY ANALYSES BY USING FINITE ELEMENT METHODS

SUMMARY

Landslide is a natural earth transforming phenomenon which nowadays in many countries is known as one of the destructive natural disasters. Unfortunately, due to the lack of adequate consideration to the slope stability in the studies and construction of development projects, every year particularly after a heavy precipitation or an earthquake, we are witnessing a large number of unpleasant incidents originating from landslides in many countries. Landslides direct and indirect damages along with their allocated annul study and stabilization costs impose an enormous sum to the country economy. This implies that we have to pay to a serious attention to the landslides problems in this paper, after review of the different mechanisms of earth layers sliding, the possible causes for the slope instability has been studied and investigated. During this study, the slope stability analysis has been performed based on the information obtained from field inspections and investigations, geological surveys, manual and mechanical borings and laboratory and field experimented, based on the results obtained, the effects of different factors on the slope instability has been investigated and discussed.

After the landslide occurred due to the movement of mass volume of materials at the surface of the earth, tensions and strains in the region Groundwater levels, piezo metric pressure, and other geotechnical issues are subject to change, which causes secondary problems such as creep, seating, subsidence, as well as shrinkage or contraction, and so on. We will explain each one.

In this project we try to investigate that increasing the water table how effect the factor of safety and analyze the Slope in Static condition by the usage of Moher-Columb and Fi-C reduction method in Plaxis 2D. The analyze result shown that in dry condition the value of factor of safety is 1.1 that is so critical and totally lower than the acceptable value in regulation of FHWA (FOS=1.25) and also by increasing the water surface, Factor Of safety decreasing lower than the minimum of value (FOS=1), and the value of 0.95 for Static analyze which means it is unstable and can cause undeniable damage to building in sounding and neighborhood.

Then we implied three different improvement system to soil model. We found the best results of FOS for soil nailing system in dry and wet conditions.

SONLU ELEMANLAR YÖNTEMİYLE ŞEV STABİLİTESİ ANALİZİ

ÖZET

Gelişen toplum nüfusu ve inşaat mühendisliğindeki yenilikler, yapıların daha fonksiyonel hale gelmesini sağlamıştır. Toprak kayması zemindeki doğal olaylardan biridir. Çağımızda çoğu ülkede tahrip edici doğal afetlerden biri olarak değerlendirilmektedir. Genelde geoteknik yapılar her ülkenin en önemli ve altyapısal konularından biridir. Maliyeti çok yüksek ve tahrip durumunda büyük mal ve can

Kayıplarına yol açmaktadır. Şev stabilitesinde yeterli zemin etüdü yapılması çok önemlidir. Maalesef çoğu ülkede şevlerin yetersiz stabilite incelemesi sonucu, deprem ve yağmur sonrası zeminlerde kayma oluşup ve ciddi mal ve can kayıplarına yol açmaktadır. Toprak kaymaları, dolaylı ve doğrudan, ülkelerin ekonomilerini olumsuz yönde etkilemektedir. Şevlerin kayma incelemesi ve onların stabilizasyonuna gereken maliyetler de ekonomik olarak önem arz etmektedir. Kitlesel zemin hareketleri sonucunda zemin yüzeysel olarak kaymaya başlar. Bu kayma neticesinde yer altı su seviyesi, piezometrik basınç ve zemindeki geoteknik parametreler değişime maruz kalmaktadır. Bu da sünme, oturma, yanal kayma, büzülme ve sıkışmalara sebep olmaktadır.

Belirli bir eğimde ve dengedeki kaya veya zemin kütlesi duraylılığının (stabilitesinin) çeşitli etkilerle bozularak hareket etmesini tetikleyici faktörler doğal nedenler (Kuvvetli yer sarsıntıları, iklim etkilerine bağlı olarak zemin-kayaç yapısında oluşan ayrışma, ysuya doygun olmayan zeminde yağış, yağışın etkisiyle zemin dayanımının azalması, suya doygun haldeki zeminde pozitif boşluk suyu basıncının artması, zemin dayanımının azalması) ve yapay nedenlerdir. Bilinçsiz mühendislik kazıları ve dolguları yapılması, insan yapısı titreşimler)dir.

Zeminlerin kaymasını önlemek için çağımızda birçok yöntem geliştirilmiştir. Ancak bu yöntemleri uygulamak için birçok bakış açısından değerlendirmek gerekir. Bu etkenlerden bazılarına değinmek gerekirse, zemin cinsi, topoğrafik koşullar, ekonomik imkânlar, iklimsel çevre koşulları ve proje özellikleri gözönüne alınabilir.

Şevlerin duraylılığını iyileştirmek için farklı yöntemler geliştirilmiştir. Bunlardan şevin uzunluğunu arttırmak, şevlerde kademeler oluşturmak, diyafram duvarları, Zemin çivileme, ankrajlara değinebiliriz.

Şev stabilitesi analizinde birçok yöntem geliştirilmiştir. Bunların bir kısmı klasik ve deneysel verilere dayalı yöntemlerdir. İkinci kısım teknolojinin geoteknik mühendisliği dalına kattığı nümerik ve bilgisayar modellemesi yöntemleridir. Klasik yöntemlerden bahsedersek, Bishop, Morgenstern ve Janbu'ya değinebiliriz. Numerik yöntemlerde Sonlu Elemanlar (FEM) ve Sonlu Farklar (FDM) a değinebiliriz. Bunlara artı olarak çağımızda hızla gelişen yapay zekâ sistemlerine dayalı yapay sinir ağları (Artificial Neural Network) da kullanılmaya başlamıştır. Ancak bazı yöntemlerin, özellikle son zamanlarda geliştirilen yöntemlerin, yeterli tecrübe edinilmemesinden dolayı çoğu mühendislik şirketleri tarafından tercih edilmemektedir. Çağımızda bilgisayar ortamında birçok şev stabilitesi analizi gerçekleştiren program geliştirilmiştir. Bu programlardan FLAC, GeoSlope, Nina, Geo5, Geostudio ve Plaxis'e değinebiliriz. Bu programların içerisinde en gelişmiş olanları Plaxis ve FLAC olarak değerlendirebiliriz. Bu çalışmada Hollanda Delft Üniversitesi'nde geliştirilmiş olan Plaxis programını kullandık. Bu program Sonlu Elemanlar Yöntemine dayalı çalışmaktadır.

Araştırmamızda şev stabilitesini dikkate aldık. Doğada kaymış olan bir şevin geoteknik değerlerini alarak bilgisayar programında sanal model oluşturduk.

Araştırmamızdaki vaka Japonya'daki depremde kaymış olan bir şevdir. Bu vakayı analiz yaptığımızda kuru durumda stabil bulunmasına ragmen, doygun durumundaki analiz sonucu, şevin stabil olmadığını gösterdi. O nedenle şevin değişik yöntemlerle iyileştirilmesine başvuruldu. Bu sistemden elde edilen güvenlik sayıları, efektif gerilmeler, ve deplasman değerleri elde edilerek değerlendirildi.

Bu araştırmada yer altı su seviyesinin şevin kayma potansiyelini nasıl etkilediğini gözlemledik. Bu incelemeyi statik durumda, Mohr-Columb modelini kullanarak, C-Phi Reduction yöntemiyle yaptık. Plaxis 2D programında modelimizi oluşturarak iki ıslak ve kuru durumunda kaymış olan şevimizi ele aldık. Araştırmamızda kaymış şevin kuru güvenlik sayısı 1,1 elde edildi. Bu değer FHWA(G.S=1,25) değerinden küçük olduğu için şevin stabil olmaması öngörüldü. Nitekim ıslak durumu analizi yaptığımızda güvenlik sayısı değeri 0,95 olarak elde edildi.

Daha sonra şevi üç farklı yöntemle iyileştirdik. Bu yöntemler, kazık, mini kazık ve zemin çivisi olarak ele alındı. Her yöntem için iki farklı kuru ve ıslak durum analizleri yapıldı. Bu analizlerin sonucu elde edilen efektif gerilmeler, deplasmanlar ve güvenlik sayıları tek tek birbirleriyle kıyaslandı. Bu araştırma konusunda zemin çivisi en iyi güvenlik sayılarını, hem ıslak hem de kuru durumlarında, ortaya koydu.

1. INTRODUCTION

Slope stability analysis are very important in civil engineering for many years. There are many analysis methods concerning slope stability problems. We compared a slope's stability safety factor by using finite element methods.

1.1 Purpose of Thesis

One of the important issues of civil engineering in urban areas is the stabilization of the vertical and sloping walls of the earth. Many research articles and papers had been published that either were related to slope stability or involved slope stability analysis subjects.

In construction areas, instability may result due to rainfall, increase in groundwater table and change in stress conditions. Similarly, natural slopes that have been stable for many years may suddenly fail due to changes in geometry, external forces and loss of shear strength [1]. Thielen et al. (2005) say, "The combination of intense rainfalls, steep topography and soil conditions are critical". Nepal has been facing challenges of large number of water-induced disasters such as landslides or slope failures mainly along the Highways [2]. Likewise, Tayler & Burns (2005) say, "Earthquakes are the greatest danger to the long-term stability of slopes in earthquake active zones" [3]. In addition, the long-term stability is also associated with the weathering and chemical influences that may decrease the shear strength and create tension cracks. In such circumstances, the evaluation of slope stability conditions becomes a primary concern everywhere.

Good understanding of analytical methods investigative tools and stabilisation measures are the engineering solutions to slope instability problems, [1]. According to Nash (1987), a quantitative assessment of the safety factor is important when decisions are made [4]. Likewise, Chowdhury (1978) says, "The primary aim of slope stability analyses is to contribute to the safe and economic design of excavation, embankment and earth dams"[5]. Development activities may face great challenges due to unstable grounds. Similarly, the slope failure may interrupt the established imperative services like traffic movement, drinking water supply, power production and similar infrastructures. In this way, to save human lives, reduce property damages and provide continuous services is the main motivation of stability analyses. Therefore, the most suitable and reliable stability analysis methods have great scope and thus, they are increasingly demanding. The chosen method should be able to identify the existing safety conditions and suggest for technically feasible and economically viable solutions.

The most common methods for analaysing are the limit equilibrium methods of slices, boundary element methods [6], finite element methods [7], and neural network methods [8].

In engineering, one of the powerful computational tool is Finite element method. By using computational tools without the need to simplify the problem, this method gains its power from the ability to simulate physical behaviors. To obtain more reliable and accurate results, complex engineering problems need finite element methods. Nowadays, new proposed analysis methods, in engineering, can be verified using finite element method as a benchmark. The finite element method can be used to study the stability of slopes using a failure definition.

In finite element slope stability analysis, the use of the strength reduction method with advanced soil models is similar to the Mohr–Coulomb model since stress-dependent stiffness behavior and hardening effects are eliminated [9]. Stiffness modulus is stress-dependent and changes based on step-size computation increments, in advanced constitutive soil models. When strength reduction method is used during computations stiffness modulus from the previous step is used as a constant stiffness modulus [9], and as a result, the advanced soil model acts like the Mohr–Coulomb model where a constant stiffness modulus is used, as well

2. LANDSLIDE RELATED PHENOMENA

After the landslide occurred due to the movement of mass volume of materials at the surface of the earth, tensions and strains in the region Groundwater levels, piezo metric pressure, and other geotechnical issues are subject to change, which causes secondary problems such as creep, seating, subsidence, as well as shrinkage or contraction, and so on. We will explain each one.

2.1 Settlement

Settlement is a phenomenon that gradually arises in this case and in some cases suddenly arises. This Settlement has caused cracks and gaps in the earth and has an impact on the pattern of underground and surface currents, changes in the quality of groundwater, changes in the shape of the earth's surface, flooding of the area and the advancement of seawater towards the land in the coastal areas. These changes cause serious damage to urban, industrial and agricultural areas.

After a landslide, a large amount of material is displaced, resulting in overpressure, compressive stresses on the soil layers, and compaction. Soil compaction due to compression deformation and displacement of particles in the soil is due to the drift of air and water from soil cavities and other factors. In a particular soil, one or more of the above factors may be involved. Since the compression of the soil causes the structure of the structure to be placed on it, from the point of view of engineering, they are referred to this soil summit. In general, soil compartments are divided into two groups:

- An immediate settlement caused by changes in the shape of dry, wet soils and saturated soils in the first installment without any changes for water present.
- A consolidation settlement caused by a change in saturated soil volume due to the displacement of water in the cavities.

The main difference between the elastic settlement and the instantaneous settlement is in their duration of occurrence, so that the instantaneous settlement occurs immediately after loading, but the consolidation settlement depends on the soil (coarse or fine grain).

2.2 Subsidence

According to the US Department of Geology, the phenomenon of land subsidence includes downfall or downstream landing, which may have a slight horizontal displacement vector. Motion is not limited in terms of severity and extent of encountered areas and subsidence can occur due to natural geological phenomena such as dissolution, waterlogging and aggregate deposits, smooth movements of the shell, and the removal of lava from solid earth, landslide, or Human activities such as mines, groundwater and oil extraction are created.

According to the UNESCO definition, subsidence is the collapse or land leveling that occurs for a variety of reasons on a large scale. Typically, this term is referred to as the upward downward movement of the ground, which can be associated with a small horizontal portrait. This definition does not include phenomena such as landslides, because their movement has a significant horizontal vector, and does not include landings in manual soils that have a different mechanism.

In general, the subsidence occurs in places where material on the ground can move toward the cavities inside the earth. The phenomenon of subsidence can arise in various forms, some of which are referred to below:

1. Tectonic subsidence
2. Volcanic subsidence
3. The subsidence due to the cave roofs (Mineral subsidence consolidation of soft sediments resulting from the extraction of fluids)

2.2.1 Tectonic subsidence

Vertical gravity movements along normal-fault zones cause the surface to fall in the vast area. The tectonic state occurs gradually or suddenly, which in the latter case is accompanied by severe earthquakes. This process leads to the formation of a lake and managed will grow up during faults.

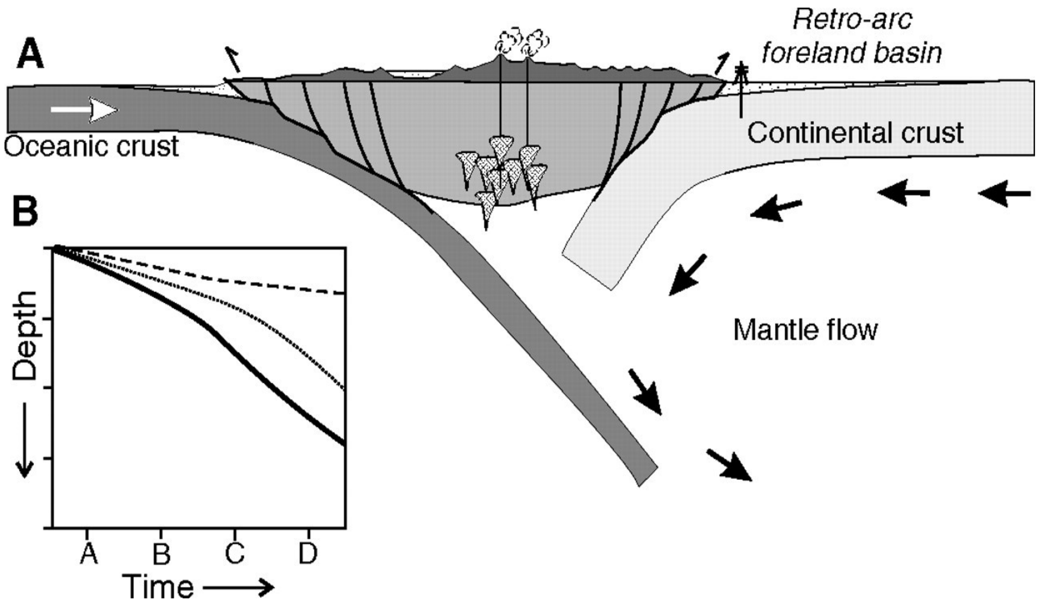

Figure 2.1: Vertical gravity movements along normal-fault zones. [9]

2.2.2 Volcanic subsidence

The displacement of a large amount of material during the vulcanization process causes a change in surface topography and collapse of the surface of the earth. In addition, vacant spaces created at the crater of the volcano may collapse later, causing the earth's surface to collapse around the volcano. A different mode of subsidence in the volcanic areas is the fall of the ceiling of the lava tunnels. The formation of lava tunnels is such that when magma moves on the surface of the earth, the upper part that is in contact with the air is cooled faster and forms a duct or a tunnel. Which moves from within a magma. As magma decreases, an empty space forms between the upper and lower layers of the magma, which is known as the lava tunnel. Subsequent factors may be due to various factors such as loading, earthquake and water level changes the underground ceiling of the lava tunnel collapsed and collapsed at the surface of the earth.

2.2.3 The subsidence due to the cave roofs

The movement of water in rock masses which is susceptible to dissolution, such as gypsum, salt, and lime, forms liquidation spaces. Although the cave can exist in different geological conditions and rocks, the caves in nature are mainly created in calcareous stones. The cave is also rarely seen in the travertine of tin, salt and gypsum to areas where there are dissolution caves, called karstic.

The word karst is derived from the geographical name of the region in the northwest of Yugoslavia near the Italian border. The native area of the region has used the term "karst" and “Kareso “since the last few centuries. Both indigenous roots And European are from the word (KAR).

2.3 The Phenomenon of Karstication

The name of Karst is taken from a region in Yugoslavia where the rocky areas of the region are mainly from limestone susceptible to cavitation and the formation of a cave. In this way, the term "Karst" is synonymous with the corrosion of carbonate and dolomite rocks by water. Gradually, with the development of research in this field, the term "Karstic" is used today for sulfate stones (gypsum and anhydrite), peridotite rocks, sandstone and conglomerate. For example, carbonate earthenware rocks into the Iran's larder dam and non-carbonated earthen rocks point to Mequinenza in Spain, Krematsa in Greece, and Yate in New Caledonia.

2.3.1 Processes of karsticization

Generally, corrosion or stone dissolution properties in water are much higher in carbonate rocks than in sulfate rocks. Dolomite rocks usually show greater resistance to water dissolution. The amount of dissolution of carbonate rocks in pure water has changed in different conditions of its chemical composition, but on average, the degree of saturation of dissolution of limestone in pure water is between 350-400 mg/l. With increasing impurities in pure water, longer time is required for the dissolution of limestone. Along faults and crushed areas Karsticization occurs faster. As the phenomenon extends along faults, lineaments and any tectonic factors. The annual amount of annual precipitation and the dissolution of carbon dioxide in rainwater also play an important role in the development of karst. The more depressions are, the more serious the karmic process becomes. The cold climate accelerates the phenomenon of calcareous karst. On a global scale, the dissolution of lime and gypsum by natural waters has created widespread and widespread Karst forms that have created very different geological conditions for civil engineers. The resulting caves will be a threat to the foundations, which is considerably larger than the thickness of the caves.

In the first step, Karsticity, dissolution, and morphology of cavernous forms develop in rocks that have qualities such as fractures and single-axis compressive strength in a healthy state of 200-300 MPa. Carbonate deposits offshore usually have no karst morphology. Gypsum karsts are similar to those of calcareous karsts, with the difference that there is a better spread of keratinous shear and shear tubing, but they do not have adult tufts and cones. Salt stones dissolve quickly and have a special morphology.

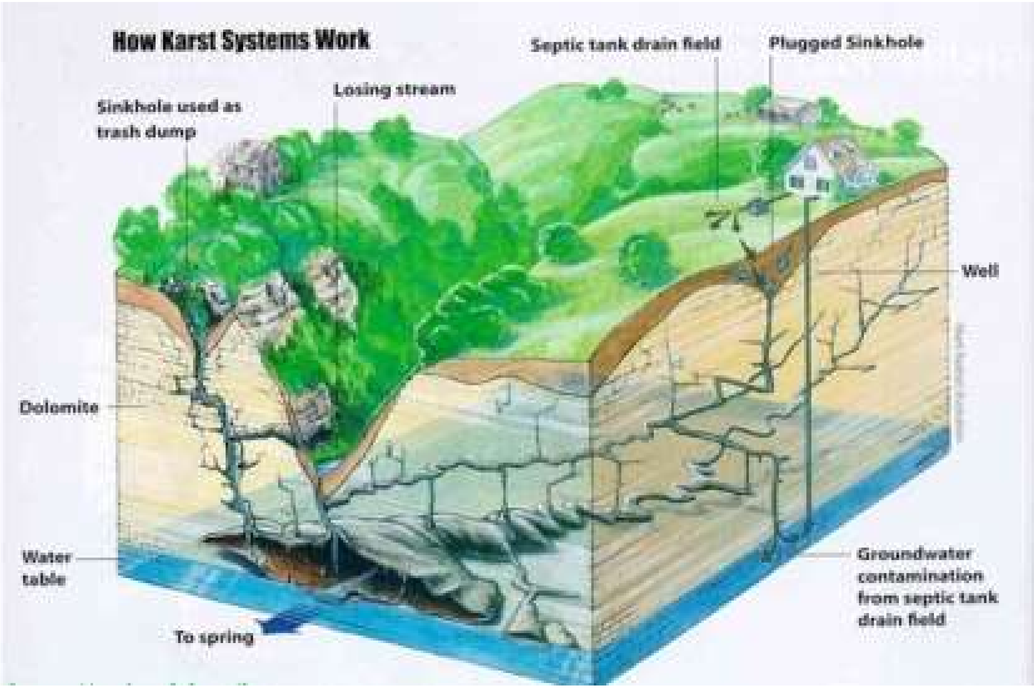

Figure 2.2: Suspension caused by the roof of the caves [9]

2.4 Creep

The creep is a time dependent motion that refers to a mixture of soil or rock and soil along downward slope of 0.3 m in 5 years of the bounce of the Warner (1972) (Sharia Ja'fari, 1996; Assistance to Asylum, 1373; Dadashi Arani, 2003). Creeps are created by the force of material (weight) and leakage water (Memarian, 2007). Therefore, the creep is a slow flow of soil or rock that does not move along the sliding surface and its displacement in the vertical direction. Creep only exists on the surface layer of the earth.

There are many factors that help to creep soils in rocks, but their nature is less known. Old landslide areas are especially susceptible to creep.Terzaghi (1950) divides the creep into two types of seasons and continuous creep. Seasonal creep occurs due to changes in temperature and humidity in the rock or soil within the surface layer. The depth of the surface layer affected by seasonal creep is often equal to or less than the depth of seasonal variation in temperature or seasonal humidity. Seasonal blemishes often occur in silty or clay soils, and its background is caused by freezing and melting ice. Due to this operation, water pressure in the soil increases in spring and thus decreases shear strength and causes the soil to move downwards. The movements generated by seasonal creep are generally less than a few millimeters a year, but effects such as trunks of trees, curved guarded fences or walls, hedges and torn beams, or with small waves and small bumps of the soil They create.

Creep is created continuously by weighted forces and occurs under the surface of the zone of seasonal changes. Bayroumi (1998) has pointed out that continuous creep originally occurs in shale. In general, it can be said that the creep rate in the solidified clay is very low and continuous creep occurs in shale and relatively in thick zones.

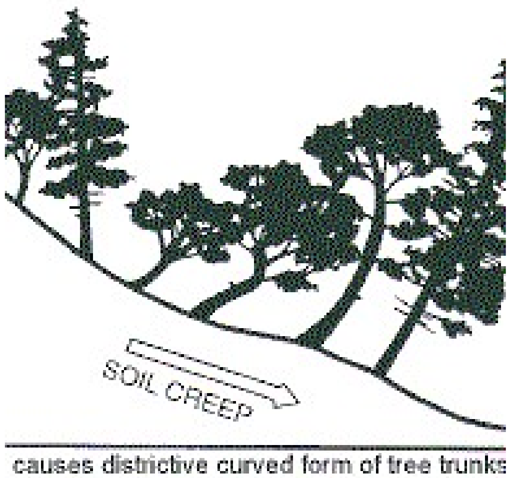

Figure 2.3: Changes caused by creep[9}

2.5 The Effect of Natural Factors on Landslides

2.5.1 Landslides and water

Saturation or dampening of slopes is a primary cause of landslides. This can occur because of a heavy rain, melting of snow, altering groundwater levels, and changing water levels along coastlines, dams and waterways, lakeside shores, reservoirs, canals and rivers.

The landslide and flood are closely linked because they are dependent on rainfall, water flow, and saturation of the soil. In addition, creep and mudflows usually occur in small channels and are often mistaken with flood; in fact, these two events often occur together in the same area.

Landslides can cause flood by creating sloping barriers that create water channels and stores plenty of water behind them. By breaking the barrier, the flood goes downhill. Solid landslides can also accumulate and increase the volume and density of natural water flow, or by closing channels, causing flood conditions or local erosion. Landslides can also cause the flow of flood from reservoirs of dams or reduce the capacity of reservoirs to store water.

2.5.2 Landslides and seismic activity

Many mountainous areas susceptible to landslides have a history of minimal earthquake rates. Earthquakes in sloping and landslide areas are likely to Increases probability of landslides. Trembling the ground lonely can cause landslide, or creating a shake will cause the expansion of the soil, increase the high permeability of the water, and create landslides. For example, the Great Alaska earthquake caused massive landslides and other ruptures in the land in 1964, which was the cause of many financial losses during the earthquake. Iran, considered active in seismicity, has caused many landslides in relation to earthquakes, the example of which is the largest landslide of the world's history in the Ancient of Kebirkouh, the so-called Seymareh.

2.5.3 Landslides and volcanic activity

Landslides created by volcanic activity are one of the most destructive types of landslides. Volcanic lavas can melt the snow quickly and create a plenty of rock, dirt, ash and water. This high-speed flood moves down the slopes of the volcano down and destroys everything in its path. These volcanic currents (Lahara) reach distant distances, and since they leave the volcanoes, they destroy all the buildings to the volcanic land near the volcano. For example, the eruption of the St. Helen Mountain in Washington in 1980 created a large landslide on the northern edge of the volcano, the largest landslide in the United States.

2.6 Introducing several important landslides

The subject of landslide, like other phenomena such as earthquakes, volcanoes, has attracted the attention of humans, because slipping is one of the threats to life and property. Because of landslides, many damage to forests and their growth, agricultural land, power lines and gas lines, mines, engineering structures and buildings, even from economical point of view create big problems. Accordingly, landslide, as one of the most important natural disasters of the current decade, has been introduced by the United Nations. Thus, recognizing the factors, characteristics and conditions for the creation and development of landslides, it is possible to achieve ways to prevent the risks and damage caused by their expansion. The best possible practice in this regard is to provide a map of landslide hazard zonation at national, regional and local scale so that they can identify vulnerable areas and prevent many damage before any incident occurs.

Below we introduce some important landslides:

1) The largest prehistoric landslide in Iran is about 10,000 BC with an area of 20 square kilometers in the Kabir Kouh region located at 47.65 degrees east and 33 degrees north.[10]
2) Between 1991 and 1992, more than 9,000 landslides were recorded in Czechoslovakia. [11]
3) In Japan, only an average of 2000 slopes annually occur along Japan's national railroad.[12]
4) The landslide on August 25, 1933 in Matsu China, which destroyed a village and killed 2,500 people.
5) The landslide of July 5, 1938, Japan, in which landslides occurred successively and destroyed 130,000, homes in the Kansai region.
6) Slipping the rock on Aug. 16, 1945 in Peru, where a rock fall resulted in the destruction of a 100-meter high dam and a huge flood at the bottom of the dam.
7) The landslide and avalanche in Los Angeles during the winter of 1951-51 caused a loss of $ 7.5 million.
8) The landslide in Norway, which occurred in 1892, blocked the entrance to the lake and created a lake with an area of 53.2 square kilometers, killed a hundred and eleven person.
9) The April 1983 landslide in Utah, the United States, damaged nearly $ 300 million to the pier of the city.
10) Landslide July 10, 1949 which occurred because of an earthquake, which is one of the biggest landslides. .[12]
11) Due to the occurrence of a stone slip in 1912 in Northern Italy, the village was blocked and a lake with a depth of more than 50 meters was created. The

water collected behind this natural dam over 20 years and over 60 people were killed in the incident.[13]

12) During the June 13, 2009 earthquake of Iran, more than 200 people were killed due to earthquake-induced landslides in the whole region, and many roads were damaged due to slipping and many roads in the villages were blocked due to landslide.
13) The latest landslide occurred in Afghanistan in 2014, which killed more than 350 people and left more than 2,500 homeless people missing more than 2,000 families. Experts estimated the height of this slip between 10m and 30 m estimated. [14]

Considering the above-mentioned cases, there are many financial and psychological losses due to landslide occurrence. The study of this important phenomenon and the identification of the causes and reasons for its occurrence is of secondary importance and the necessity of landslide hazard zonation and management to properly address this phenomenon and to act appropriately to avoid danger in areas susceptible to landslide or risk reduction and landslide control.

3. THEORETICAL FOUNDATIONS (METHODS OF ANALYSIS AND SUSTAINABILITY OF SLOPES)

3.1 Introduction

Regarding the stability of slopes, important issues related to buildings, highways, canals, embankments, and the design and construction of dams are proposed. Because of a slip or failure of a slope, the intended performance for that slope will be lost.

The discussion of the stability of the gates was first proposed by Columb (1776) regarding the analysis of the retaining walls, considering the shear strength of several soil types with internal adhesion and friction. [15]

Since then, in slope stability analysis significant progress has been made, and methods such as limit equilibrium method, fractal analysis, and numerical methods are presented, including the new methods of slope stability analysis. The purpose of the analysis of a slope, investigating the gradient status in terms of potential instability, determining the level or probable failure levels, and the safety coefficient of failure.

By definition, the factor of safety is the ratio of the force of the drive along the surface or the levels of failure to the force against it. When the drive force of the overload weight is greater than the resilient force, the shear strength of a surface, safety factor of that surface will occur to less than one value and slip. The shape of the level of failure depending on the homogeneity or the heterogeneity of the material mass may be regular or irregular. For example, if a stone trench is one or more joints, the joints and fractures in the rock mass and the location of their collisions or concentrations will be a decisive determinant of stability. If the materials forming the trench are homogeneous, the fracture surface forms part of a sphere it is an ellipsoid, because the sphere or ellipse has a smaller area of mass than other geometric volumes. If the material of the structural layers of the slope is heterogeneous and affects the levels of unconformity of the structure of the rock, the fracture surface takes an irregular shape.

3.2 Methods for Stability Analysis of Slopes

Most of these methods are:

a) Limit Equilibrium method (LEM)

b) Finite Element method (FEM)

c) Numerical method

3.2.1 Limit equilibrium stability analysis method

This method is one of the most widely used methods based on the Mohr-Coulomb rupture. Limit equilibrium stability analysis methods have long been used to analyze the stability of natural and abnormal domains. The source of these methods is established on the balance of force or of the moment of inertia or both. In these methods, in general, if the unstable agent increases on the stabilizing factor, the target mass becomes unstable, hence the discussion of resistant and stimulating forces acting on a slip surface.

In total, it can be said that the strength of the forces depends on the geotechnical parameters and the amount of load perpendicular to the slip surface and the amount of destructive forces depends on the amount of parallel load of the slip surface.

Rupture can have structural control (For example, a slip on the surface of the joint in the rock masses) or lacking structural control (For example, landslide in homogeneous soils).

Given that the rupture in the slopes is mainly of a kind of structural-non-controlling nature, this section discusses most of this.

In an uncontrolled method, a number of hypothetical slip surfaces are considered and the safety coefficient of each of them is obtained. Finally, the level with the lowest reliability coefficient is determined, whereas if the safety coefficient of this level is lower than the permitted safe9ty coefficient and the mass above this is in unstable level.

The method of equilibrium without structural control is divided into two general categories:

1- Charting methods

2- Analytical methods (Linear methods, nonlinear methods

The factor of safety in nonlinear methods is obtained using trial and error and in linear methods acquired direct. Based on the Moher – Coulomb rupture theory, it is assumed that, along hypothetical rupture level, the equation $S = C + tan\emptyset$ and to gain the shear stress along the slip:

$$\tau = {}^{S}/_{FS} \qquad 2\text{-}1$$

$$s = C_a + \sigma tan\emptyset_a \qquad 2\text{-}2$$

$$\tau = C_r + \sigma tan\emptyset_r \qquad 2\text{-}3$$

s: Existing shear stress

τ: Shear Strength Required

C_a: Adhesion

To control the stability of a slope, first, using one of the above mention methods and applying a critical slip surface, minimizes factor of safety. After obtaining the factor of safety, the minimum slope is constant, if this value is larger than 1.5 it does not need to be fixed. Otherwise, by usage of slope stabilization method, factor of safety had been increased.

3.2.2 Limit analysis method

Limit Analysis is a method in which using some cases up and down ramps sustainability issues will be thoroughly investigate. In this method, the lower limit solution in relation to the stress field is such that the soil mass at any point under the load is in equilibrium and the upper limit solution is related to the speed field and the displacement applied. In simple terms, under load in the lower limit state when the lower limit converges with the load of failure, the failure will occur or will be imminent. Limit Analysis method uses an ideal yielding layer, which is a fit between tension and strain, and it, is assumed that the soil is a completely dense material and follows the adherent flow rule. Soil plasticity theory will indicate the state of tensions as$F(\sigma'_{ii})$ so $F(\sigma'_{ii})$ represents the yield level and $F(\sigma'_{ii})$ represents the tension tensor.

The flow rule and the strain rate are related, assuming that the yield level F matches the potential level of G, the plastic rate will be$\varepsilon_{ij}^{p} = \lambda \frac{\partial G}{\partial \sigma'_{ij}} = \lambda \frac{\partial F}{\partial \sigma'_{ij}}$, and when plastic deformation occurs, its value will be positive. The flow rule and the strain rate are related, assuming that the yield level F matches the potential level of G, the plastic rate will be$\varepsilon_{ij}^{p} = \lambda \frac{\partial G}{\partial \sigma'_{ij}} = \lambda \frac{\partial F}{\partial \sigma'_{ij}}$, and when plastic deformation occurs, its value will be positive.

Table 3.1: Differential Equilibrium Methods in Slope Analysis

Method	Surface of rupture	Assumptions	Reference
Koolman's Infinite Slope	straight line	A constant angle with the Wide Range and Snake on toe slope failure plane	(Teylor,1948) (Culman,1866)
Wedge	straight line	Block Slider with Side Force	(Moregenstern,1968) Hoke and) (Bray,1977
The beams	Circular arc	The lateral forces are equal on each side	(Fellnius,1936)
Friction Circle	Circular arc	An effective vector on the tangent rupture arc on a circle in the radius $Rsin\emptyset$	(Teylor,1948)
Bishop	Circular arc	The whole forces affect the side faces of the beam	(Bishop,1955)
Corrective Bishop	Circular arc	The system of forces affects the side faces of the beam	(Bishop,1955)
Spencer	Irregular	The forces are drawn .between a parallel beams	(Spencer,1967,1981)
Moregenstern Price	Irregular	Based on the total power of the binoculars	Moregenstern, 1965) (Price &
Janbu	Irregular	The forces are considered .between the beams	(Janbu,1954,1973)

Table 3.2: General methods for stability analysis of slopes and geological conditions

Geological conditions of the region	General methods of analysis
Non-sticky sand, shallow soils, hard-clay clay and marine shale in weathered areas	Infinite slope (Depth of rupture is low relative to its length)
Slider blocks, rock or soil, sloping layers, faulting materials or rubbing, sticky soil, firm and firm, hard to very steep up to gradient slope	Limited slope: Simple wedge (single plate rupture level)
Slab blocks in rock mass, rocks with close joints, sedimentary rocks of the layered layer, natural layers of soils and zoned soils, soil on the slopes on soils	Slope limited: Wedges, blocks
Soils and homogeneous slopes and isotropic	Limited slope: circular arc (the total slider mass is analyzed as a free object.
Thick layer of soils, soft shale or soft seas, soft to hard clay loose soils, natural layers of soil or dams, dirt on the soft ground	Limited slope: Circular arc (slab method)

3.2.3 Stability analysis by numerical method

Numerical methods are an approximate method of solving engineering problems, which help them to solve the problem at specific points in an approximation and then use that solution at any desired point. In solving practical engineering problems, since the governing equations are the boundary condition of the environment, and the behavioral relationships of the complex environment, therefore, numerical methods are inevitable and, in many cases, provide more accurate answers to complex problems than other methods.

The basis of numerical methods is established on the transformation of an environment with an infinite degree of freedom into an environment with a number of degrees of freedom in a certain number of environmental points. By analyzing the effect of force and loading in these points and determining the degree of their deformation, they can be used to transform the other points by means of the interpolation. The position, number, and relevance of the above points are determined by the environmental element. Each element represents a small part of the environment that has geometric characteristics, instruments and materials of its own. Nodes interconnect elements.

In each element, the borders do not have an impression; of course, it is possible to consider nodes on the sides to increase the accuracy of the nodes. In all numerical methods, different elements exist with different geometries, and a particular model can have several different elements, their choice being to the type of problem and the purpose of modeling depends.

The most important numerical methods used to model geotechnical environments (rock and soil) are: the method of boundary components, the method of separate components and the finite difference method. Combined (hybrid) methods have also been used and various applications have been written based on it. For example, the Flac, Plaxis computer programs are written based on the combination of finite difference methods and finite element method with the boundary component method. [16]

4. GENERAL SLOPE BEHAVİOR WITH LIMITED HEIGHT

In some cases, the soil adhesion coefficient is so large that the critical depth of a large number is obtained.

One of the first signs of an impending defeat of slopes is usually a protrusion or crack development on the slope toe crust. Slip (defeat), downstream or outburst, and the movement of soil masses to a new equilibrium, during which the slipped mass is broken into smaller blocks. As shown in Figure 4.1, the sliding surface in the homogeneous mass is often circular.

In some cases, the surface of the failure and the shape of the slipped mass depends on the weak layer inside the soil, as shown in Figure 4.1a and Figure 4.1b, on a slope with a limited height that requires curved surface defects or a combination of several straight lines is used.

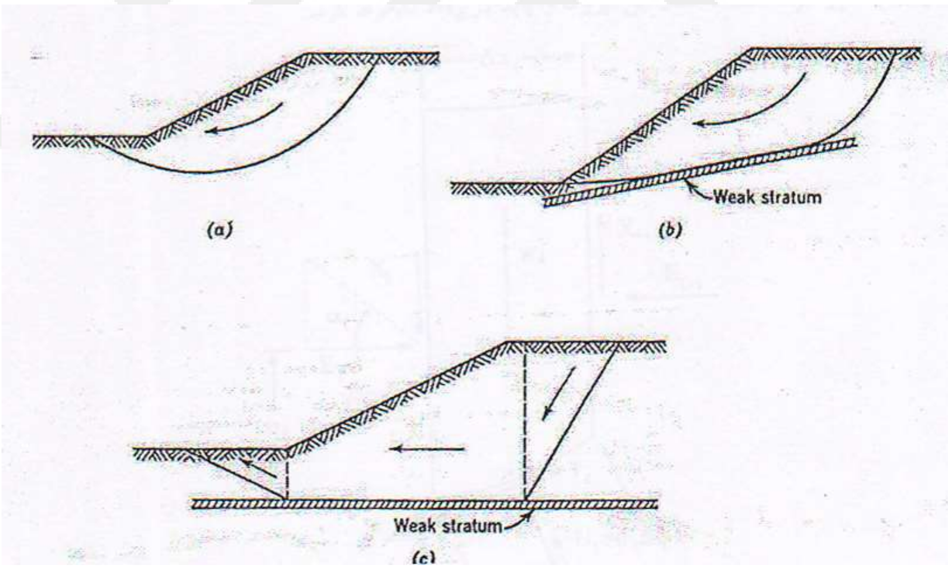

Figure 4.1 Types of circular slider surface (a) non-circular (b) and slip block (c) [17]

.4.1 The General Form of Equations in Extreme Equilibrium Methods

In all these methods, the confidence coefficient in terms of the moment around the center of the slip arc (in homogeneous masses) is defined as follows:

$$FS=\frac{M_r}{M_d} \tag{4-1}$$

As shown in Figure 4.2, the momentary arm for each piece is equal to $rsin\theta$ so it can be written:

$$M_d=r\sum_{i=1}^{i=n} w_i sin\theta_i \tag{4-2}$$

Where r is the radius of the deflection arc (slip), N is the number of parts and w_i shown in the Figure 4.2.

Resistant moment is defined as:

$$M_r = r.\left(\overline{c}l + tan\overline{\varnothing}\sum_{i=1}^{n} N_i\right) \tag{4-3}$$

$$FS = \frac{\left(\overline{c}l + tan\phi\sum_{i=1}^{n} N_i\right)}{\sum_{i=1}^{i=n} w_i sin\theta_i} \tag{4-4}$$

Where in $\bar{C}$ ، $\overline{\emptyset}$، l and N_i respectively, the average value of adhesion and friction layer of the arc length, arc length, vertical reaction at this point I and l is the number of pieces.

Due to the fact that the water pressure drop passes through the center of the arc, M_d does not play any role.

If any external force other than gravity to be entered on the charge slip (like the weight of the building that is located on a slope) M_d moment of these forces should be taken into account.

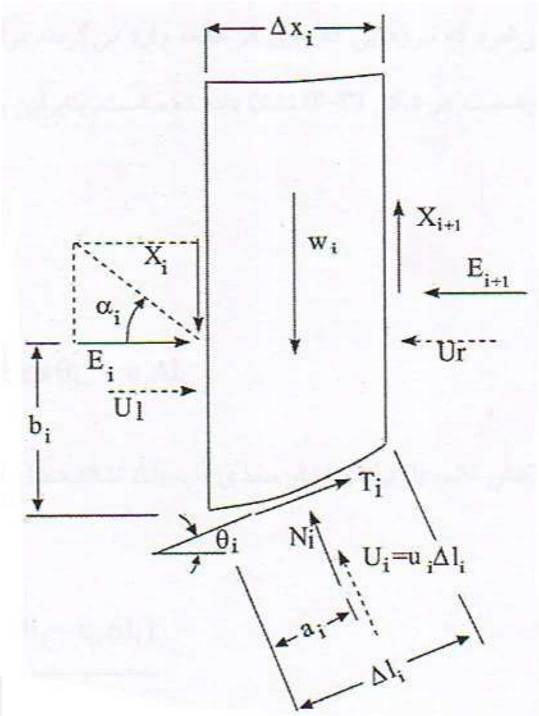

Figure 4.2 The system of forces imposed on a piece [17]

4.2 A Typical Method of Parts

In the parts method, the slip surface is taken as an arc of the circle and the soil mass is located above the surface.

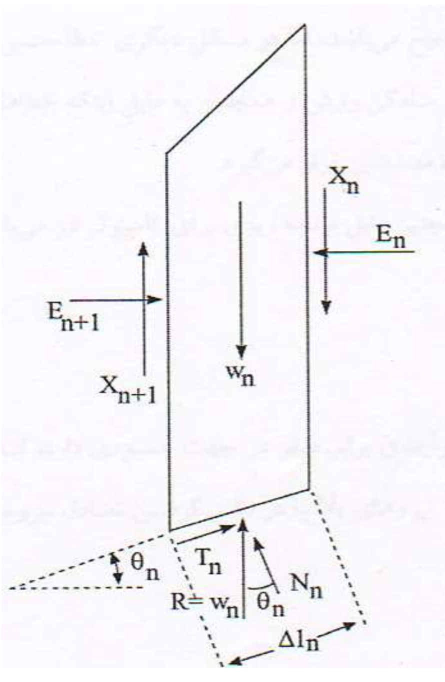

Figure 4.3 The forces are assumed in the typical method of parts[17]

The slip is divided into several vertical parts, the forces applied to each piece are then applied to the equilibrium equations of forces, and the unknown forces are calculated. Finally, the overall confidence coefficient of the hypothetical slip surface is obtained. In this method, it is assumed that the forces that enter on each piece have a zero margin in the direction of perpendicular to the slip arch.

$$N_i + U_i = w_i \cos\theta_i \tag{4-5}$$

$$N_i = w_i \cos\theta_i - U_i = w_i \cos\theta_i - u_i n l_i \tag{4-6}$$

In the above function, $N_i$represents the vertical reaction, $U_i$indicates the pore pressure of water, and Δl_i represents the arc length at the I section. Then we will have:

$$FS = \frac{\bar{c}l + tan\bar{\varnothing}\sum_{i=1}^{n}\left(w_i cos\theta_i - u_i \text{�} l_i\right)}{\sum_{i=1}^{i=n} w_i sin\theta_i} \tag{4-7}$$

Considering the number of assumptions made for the forces of the two sides of the n-1 pieces, while the total n-2 is missing. Therefore, the system parts are over defined and there is practically no possibility of static equilibrium. Hence, the reliability coefficients obtained from this method usually fall below the lower limit. In some cases, the confidence coefficient obtained from this method is only 10% 15% below the correct answer level, but in other issues, the error is even more than 60% arrives.

In spite of these errors, the method is widely used by engineers because of the simplicity of the method as well as the fact that the errors are in order. In this way, the calculations can be done manually and can be programmed for the computer.

4.2.1 Bishop's method

In this way, it is assumed that the forces applied to the edges of each piece is zero in the vertical direction, in which case this method will be simple Bishop's method. In this method, forces $N_i$is obtained by taking into account the balance of forces [23]

$$N = \frac{w_i - u_i x_i - \left(\frac{1}{FS}\right) \times \bar{c} \times x_i tan\theta_i}{cos\theta_i + \left[tan\theta_i \times tan\bar{\varnothing}\right] / FS} \tag{4-8}$$

$$FS = \frac{r \times \sum_{i=1}^{i=n} \left[\bar{c} X_i + \left(w_i - u_i X_i \right) tan\bar{\varnothing} \right] \Big/ M_i(\theta)}{\sum_{i=1}^{i=n} \left[r w_i sin\theta_i + a w_i \left(r cos\theta_i - h_1 \big/ 2 \right) \right]} \qquad)\text{-}9(4$$

$$M_i(\theta) = cos\theta_i \left(1 + \frac{tan\theta_i tan\bar{\varnothing}}{FS} \right) \qquad (4\text{-}10)$$

Since F.S. exists on both sides of the equation, a trial and error method is used to solve the problem. The convergence is usually achieved quickly.

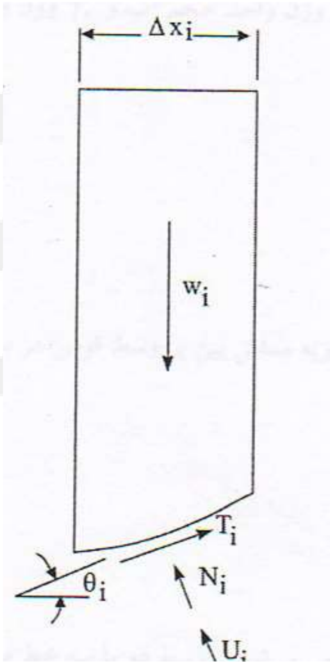

Figure 4.4: The forces assumed to be simplified in the Bishop's method[17]

It should be noted that later, the modified Bishop method, in which the resulting forces on the edges of each piece were included in the calculations, were invented, which is more accurate than the simplified method. [18]

4.2.2 Fellenius method

One of the simplest methods to study the stability of slopes is the Fellenius method, in this method considers a sliding surface with a circular cross-section or curve with different curves, and the section of sliding part is divided into several vertical bars, and by neglecting the interlacing forces are calculated as a coefficient of reliability relative to the momentum of the force and the stimulus on the slip surface. The corresponding modalities will be as follows:

$$FS = \frac{\sum_{i=1}^{i=n} C'L + \sum_{i=1}^{i=n} (N - U) tan\varnothing'}{\sum_{i=1}^{i=n} T} \quad (4\text{-}11)$$

In this relation, N, T and U, respectively, the weight component of each slab along the normal slope, the tangent component of the weight of each slab and the force of the water pressure along the normal along the slip surface, as well as $C'$and $\emptyset'$are the soil resistance parameters in each bar and L is the representative of length in each slab. If (h) is the total height of each beam, $h_w$is the piezometric height of water, $\gamma_w$is the weight of the unit volume of water, and $\gamma_t$is the unit weight of the volume of soil in each slab.

$$N = \gamma_t hbcos\alpha \quad (4\text{-}12)$$

$$T = \gamma_t hbsin\alpha \quad (4\text{-}13)$$

$$U = \gamma_w h_w l \quad (4\text{-}14)$$

In the above relations, α is the tangent angle between, in the middle of the arc of each beam along the horizon and b is the horizontal width of each beam. [19]

4.2.3 Wedge method

In some cases, the actual slip surface can be approximated by two or three straight lines, this situation especially occurs when there is a weak layer inside or under the gradient, while the whole slope is located on a solid line. Figure 4.5 illustrates a situation in which the use of straight lines imposes a fairly good approximation. Usually, a very accurate and satisfactory estimate of the slope confidence coefficient can be obtained using the wedge method.

In this method, the slip mass is divided into two or three wedges. Shear strength parameters along each side of the sliding surface in terms of adequate strength and a confidence factor of safety F.S which is expressed in all fields.[20]

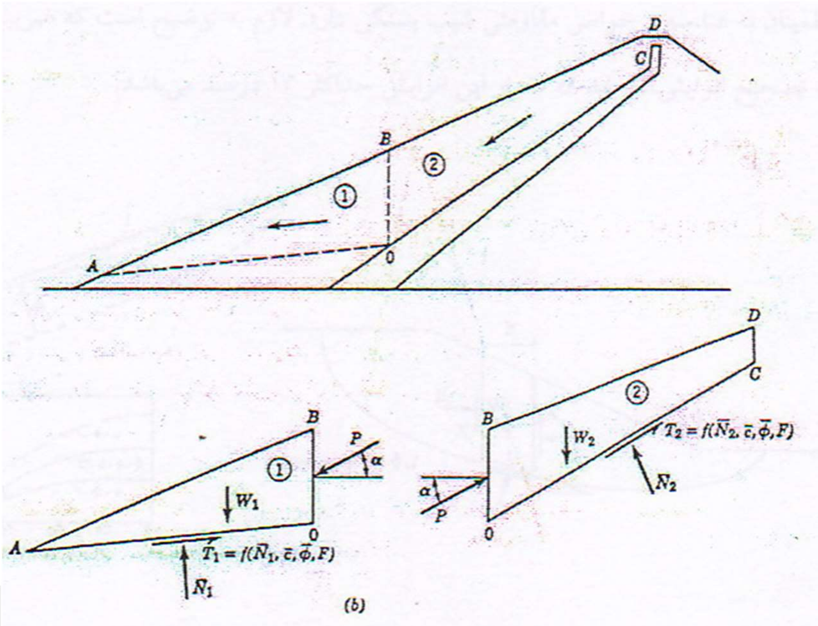

Figure 4.5: A wedge of stability analysis.[17]

4.2.4 The Janobu method

Janbou and colleagues (1965) presented a method for determining the slope confidence coefficient, in which the sliding surface is non-circular (Figure 4.6),and the shear forces are assumed zero.

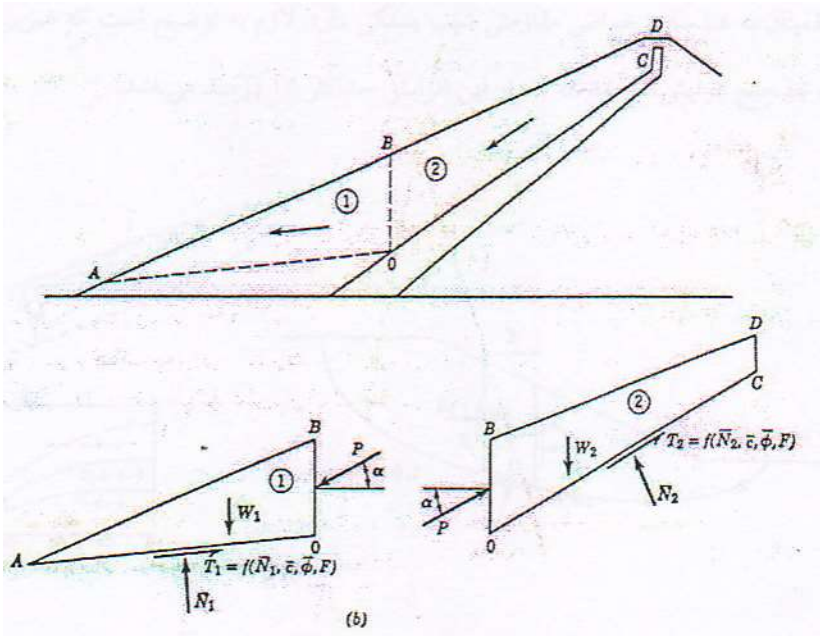

Figure 4.6: A wedge of stability analysis.[17]

To determine the problem, we assume a certain amount on the slope so that the confidence coefficient can be calculated. So each piece is made to vertical forces that the rule would be the same as Bishop. Thus, the expression for the total vertical forces on the rule of each piece will be the same as the Bishop's method. To apply the shear forces between the parts, a correction coefficient is used for the coefficient of confidence. Accordingly, the corrected coefficient of reliability is defined as follows. [27]

$$FS = f_0 \times F_0 \tag{4-15}$$

$$F_0 = \frac{\sum\left[cb + \left(W + \Delta X - ub\right)tan\varnothing\right]}{\sum wtan\alpha} \times \frac{sec^2\alpha}{1 + \left(tan\alpha \times tan\varnothing\right)/Fs} \tag{4-16}$$

The correction factor can be obtained from the curve of Figure 4.7. It is worth noting that the coefficient of confidence increases with the correction coefficient, which is the maximum increase of 13%. [28]

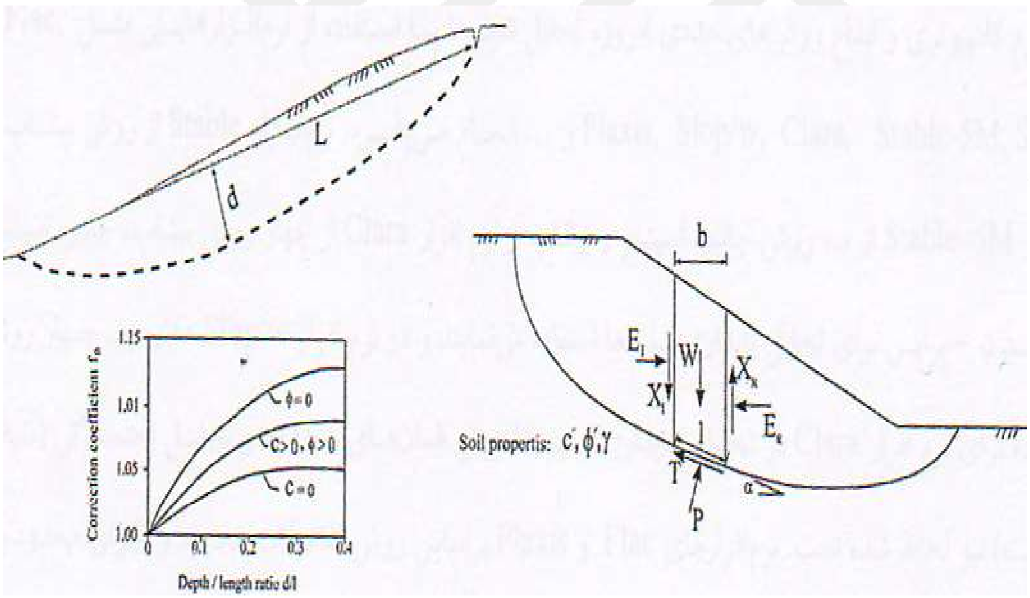

Figure 4.7: The slip surface and forces on the element in the junbou method and the coefficients of correction curves.[17]

4.3 Comparison of Different Methods of Stability Analysis of Slopes

The accuracy of a gradient analysis generally depends on the accuracy of the geometry considered for that gradient, as well as groundwater conditions, and the accuracy of the soil properties (C, ∅), and ultimately the accuracy of choosing the method of analysis.

The confidence factor of FS determined from anchoring equilibrium is less sensitive to component forces, this suggests that methods such as the improved bishop modification obtained from anchor equilibrium can be modified from other methods, such as the modified junction, which balances forces they are much more accurate. According to comparative studies, the following equations can be used for finite balance methods:

I. Methods that provide for all equilibrium conditions (such as Janobu, Spencer, Merckenstone, and Price) are within 15% of the exact results.

II. The Bishop method, which only maintains an anchor equilibrium, usually gives precise results, unless the slip surface has a slope in the slope.

III. Other methods for analyzing the stability of slopes that are not able to apply all equilibrium conditions (such as the conventional method of parts, force balancing methods) may yield very poor results.

5. STABILIZATION

In terms of slope stability, stabilization can be divided into two groups:

If the space around the slope is limited and closed, a simple and economical way to protect the walls, to run it steeply or to increase the angle of incline to a certain extent. Thus, there is no need to use a bracing system, but if the conditions of the place are such that the execution of the walls is not uncontrolled, then there is no choice but to use the guardian structure. [21]

Sometimes in loose areas, such as parking lots and basements, the condition of the ground and building is such that the gradient cannot be met for them. In such cases, considering the width of the pipeline, the excavation wall will be dug in a stage. The width of the recess must be such that the workers can easily work on the wall and the stage. In lands with natural moisture, it is possible to digging up to 1 meter for sand, 1/25 m for cleared sand, 1.5 m for clay and 2 m for very dense soil.

5.1 Increase of The Slope Length

The absence of a temporary storage system inside the main land that reduces the area of the land or the maintenance of the main structure, as well as high execution speed and low cost, are the advantages of this method. This method requires open space around the earth, therefore, in areas with limited work space this method cannot be used.

The maximum slope of the canal or dewatered area with a height less than 6.1m (20ft) can be determined directly on the type of soil in the area. [22]

Figure 5.1 Excavation in the form of constant slope and stem slope[23]

5.2 Benching

There are two methods for benching, which are simple and multiple, in each of them, based on the type of soil, the vertical aspect ratio determines the horizontal side of the step. Based on a general rule, the height of the lowest step from the bottom of the channel should not exceed 1.2m (4ft).

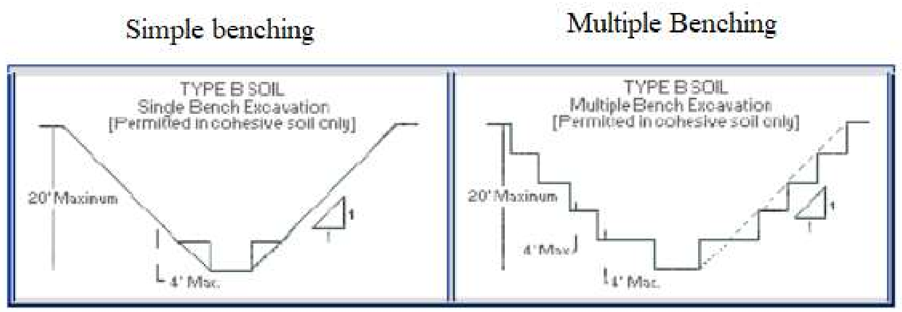

Figure 5.2 Allowed slope depending on the type of soil in the stepped excavation.[23]

5.3 Types of Stabilization Methods

- ✓ Braced wall using wale struts
- ✓ Soldier beam& lagging
- ✓ Braced sheet pile
- ✓ Bored pile walls
- ✓ Diaphragm walls-Slurry wall
- ✓ Soil nailing
- ✓ Anchorage
- ✓ Tie back
- ✓ Micro-pile
- ✓ Truss – Raker

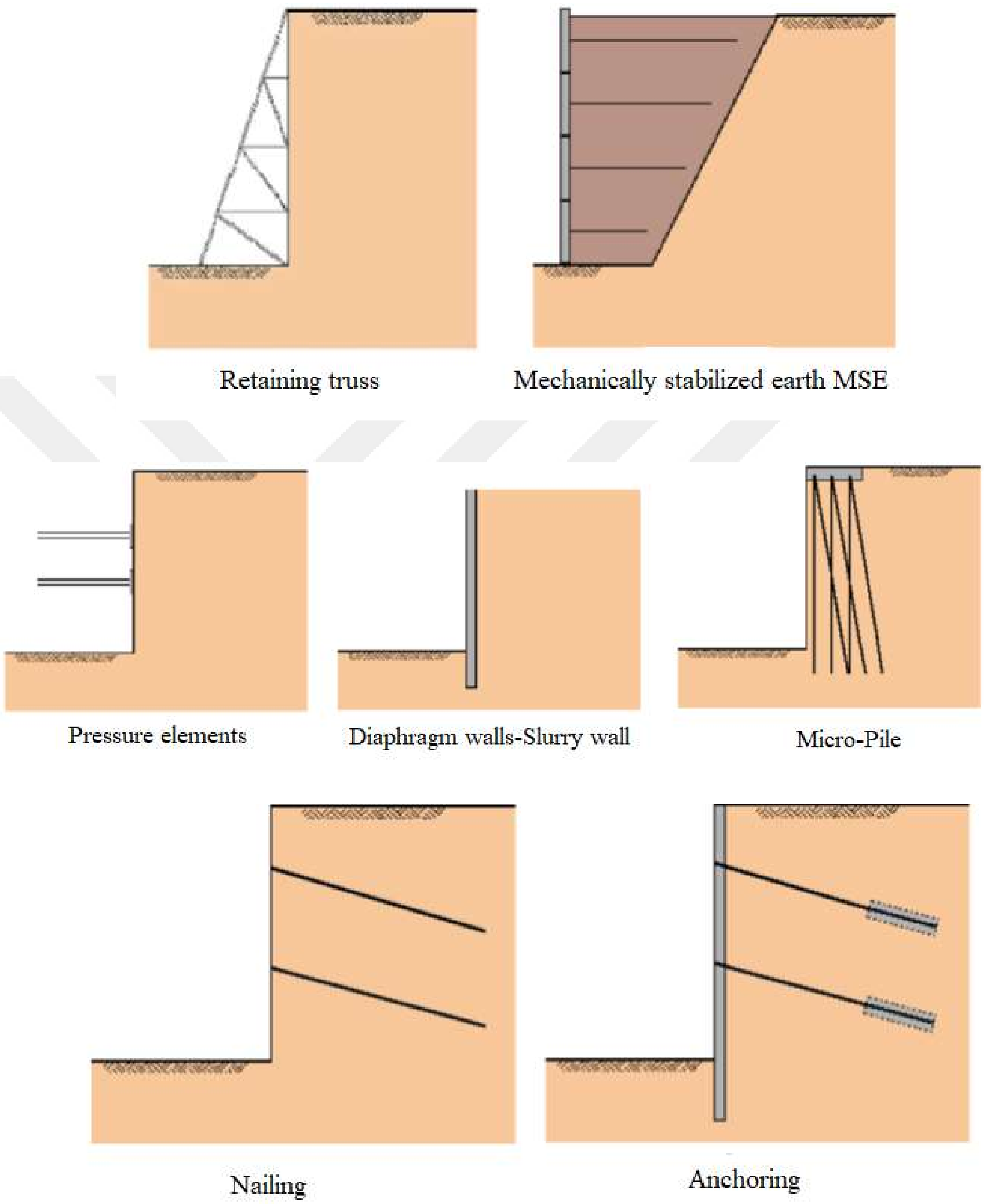

Figure 5.3 Stabilization methods [23]

5.3.1 Braced wall using wale struts

This simple method is used to maintain and protect the walls of the excavation and to prevent the displacement of lateral locations in narrow gauges in urban environments. The disadvantages of this method are a considerable loss of workspace and a limitation in the use of the required machinery and equipment, as well as increasing the risk of collision with the elements and jeopardizing them.

Figure 5.4: Braced Wall

5.3.2 Soldier beam& lagging

This method is used as a common method for temporary hailing in urban areas. In this method, ordinary steel profiles are used in the form of continuous columns that are immersed in the soil, which will run to the depth of the bottom. The distance between the elements is between 2 and 4 meters, so that the space between them can be filled with wooden larches (Chinese larvae). In this method, tensile barriers are used to protect the side of the hollow and the connection between the columns is done by the guards and welding welds.

Figure 5.5 Soldier beam& lagging

5.3.3 Braced sheet pile

In this method, sheet piles are pneumatically driven by means of vibration into the soil and connected to each other with a variety of connections between each other and forming a continuous wall.

The advantages of this method are the convenience of knocking, installing and removing metal plates, and the materials can be reused in other projects. Also, in this method, more horizontal and lesser elements are needed. Therefore, there are fewer restrictions on the occupation of the interior space. One of the disadvantages of this method is the dependence on the installation of metal shields that create limitations in urban environments due to the existence of urban infrastructure and the shaking and noise caused by knocking shields. It is also difficult to knock shields in rocky soils or very dense soils, and it can be restricted in areas with these conditions.

Figure 5.6 Braced sheet pile

5.3.4 Bored pile walls

One of the common methods for the stability and protection of the walls with various conditions, including hard and soft soil, is the use of lateral supporting pile and, in some cases, in addition to the role of lateral protection, also plays the role of water proofing, and always in case of vertical load also tolerates lateral supporting pile in the following cases are considered as the preferred option for protection systems:

- In cases where there is no possibility of a steel shield (knocking and installation), or the hardness and density of the earth is too much of a shield, it can be difficult to handle.

- In a situation where due to the presence of groundwater and high surface water it is necessary to seal the wall.
- In cases where it is not possible to create tensile barriers in the adjacent buildings due to excavation or in the intersection with the infrastructure of the city and underground tunnels.
- When there is a possibility to use the groove protection system as part of the main structure and the load.

Figure 5.7 Bored pile walls

5.3.5 Diaphragm walls-Slurry wall

One of the other ways to protect the wall is to install a diaphragm wall or a wall of slurry. In this method, firstly, grub machines are used to fit the drilling conditions of the part of the wall. At the same time, drilling is performed to ensure the stability of the walls of the wall and prevent local bursts of bentonite slurry.

Figure 5.8 Diaphragm walls-Slurry wall

Diaphragm walls-Slurry wall steps of running:

- Drilling the wall area step by step with special devices (hydrophones or grubs)
- Simultaneous filling of bentonite drilling site
- Reinforcement rack broker
- Concreting wall

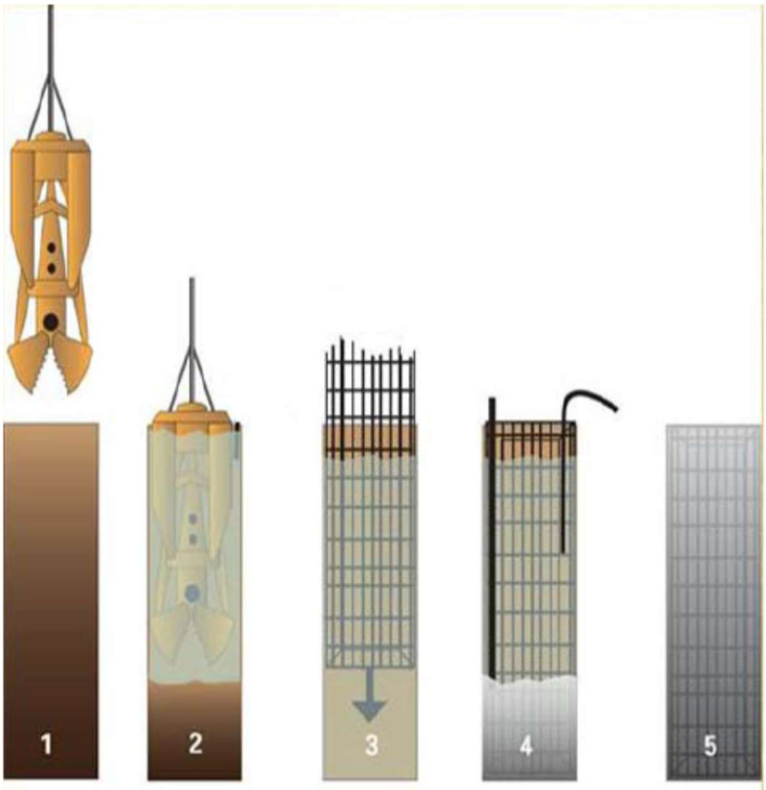

Figure 5.9 Diaphragm walls-Slurry wall steps of running [23]

5.3.6 Soil nailing

This method has begun for about three decades and is still used as a technique for the stability of trenches and high resilience protection. The theory of using the nailing method is based on the arming and solidification of soil mass by sewing the soil mass by steel tensile fasteners at close proximity to each other. The use of this method increases the shear strength of the soil mass, restricts and controls the change of soil locations due to increased shear strength at the slip surface due to increase of vertical force and reduces the slip strength at the level of rupture and slip. It should be noted that all levels of drilled trenches to be arranged by nailing must first be protected using mesh and shatterite mesh and then the nailing system is applied to them. Nailing tensile restraint is usually made of steel reinforcements with a diameter of 20 to 40 mm and with a yield limit of 420-500 nm per square millimeter, which is inserted into a drilled chisel with a diameter of 76 to 150 mm and is circulated inside the chamber. The distances between the tensile barriers are about 1 to 2 meters, and their length is about 70 to 100 percent of the height of the hole, and the minimum slope is about 15 degrees horizontally. It should be noted that the shatkrite treatment on drilled trenches does not have a structural role, but it can be used to ensure the interim stability of the soil between the barriers.

Figure 5.10: Soil nailing execution.

5.3.6.1 Steps to implement the nailing system

1) Excavation
2) Drilling nail holes
3) Installing and grouting nails
4) Construction of temporary shotcrete facing
5) Construction of subsequent level
6) Place final facing on permanent wall

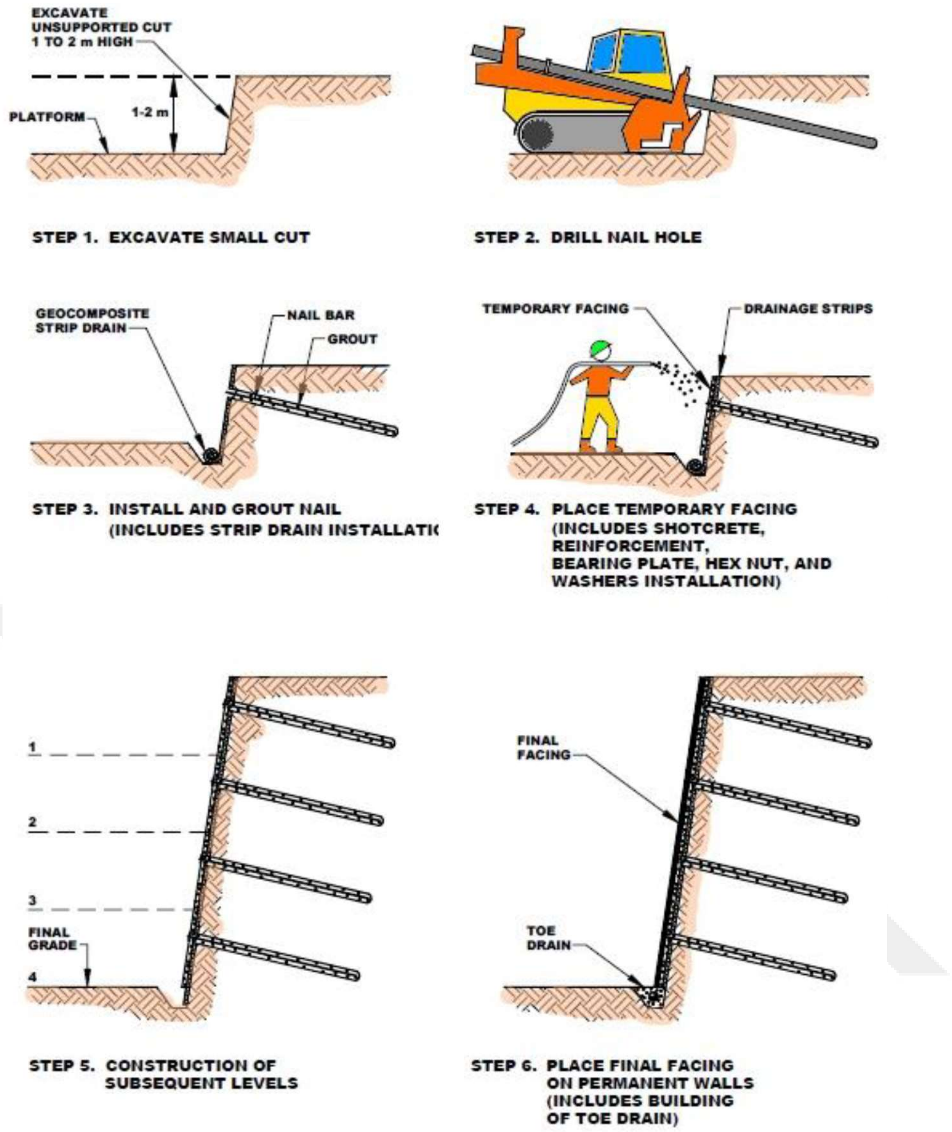

Figure 5.11: Steps to implement the Nailing System.[23]

5.3.7 Anchorage

In this way, the wells are excavated at certain intervals on the edge of the land to be excavated. The depth of the wells is equal to the depth of the well, as well as an additional amount for the concrete piles of the bottom end of the wells. Inside the wells I form profiles of shape or h shape are about 30% lower than the bottom of the bottom and at the end of the profiles are tentacles. Concrete the lower end of the candle previously fitted with the armature so the profiles are held in the pile and then the step-by-step sampling starts from top to bottom. In order to prevent it from falling, the drill holes in the hollow or horizontal hollow are 10 to 15 cm in diameter, and within them, the reinforcement is 5 to 10 m in length and the concrete is injected.

Prefabricated concrete panels are placed between vertical profiles and on the other hand, we restrict the hinges to the armature.

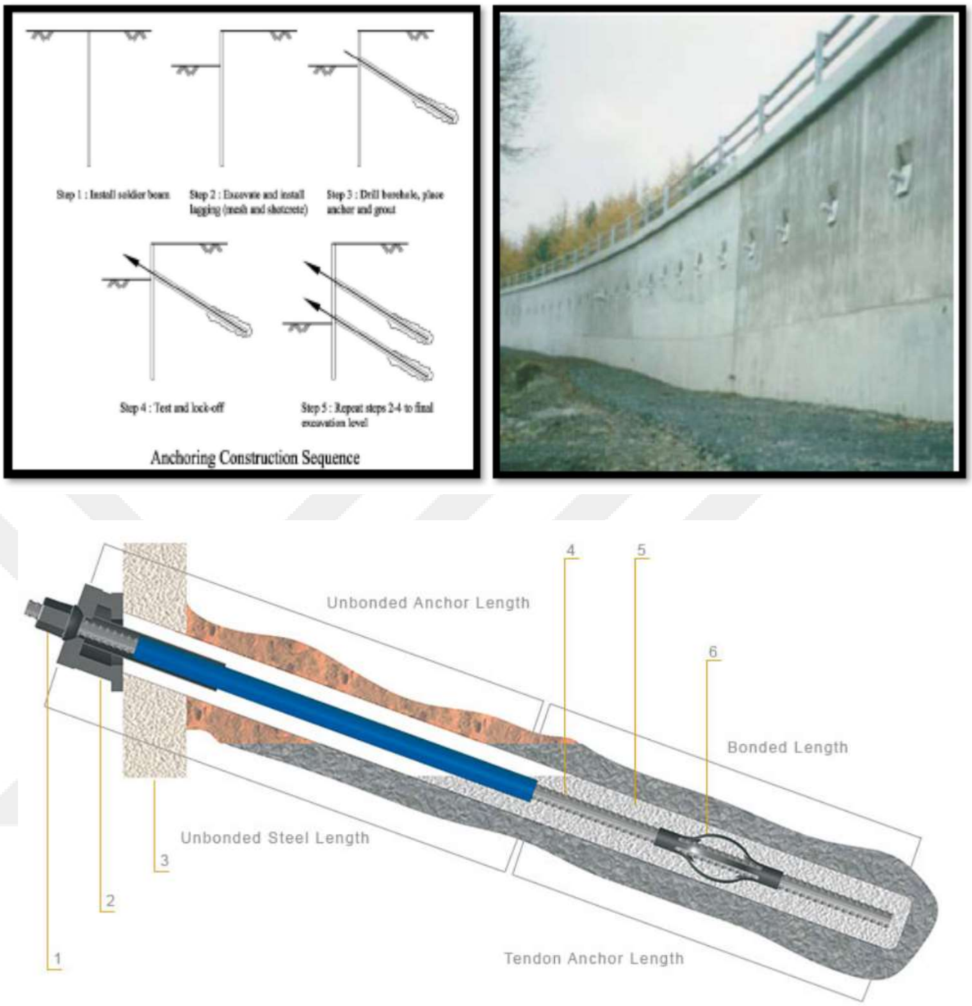

Figure 5.12 Anchorageing construction sequence and processes[23]

5.3.8 Tie back

It is very similar to the anchoring method and runs through the following steps:

a) Step-by-stage drilling operation from the top down to the bottom (2 to 3 meters)

b) Drill holes in the hollow wall

c) Placing pre-stress cables in the well

d) Concrete injection at the bottom of the well

e) Draw and seal the cables at the level of the wall

f) Concrete injection throughout the well

g) Releasing cables after concrete hardening

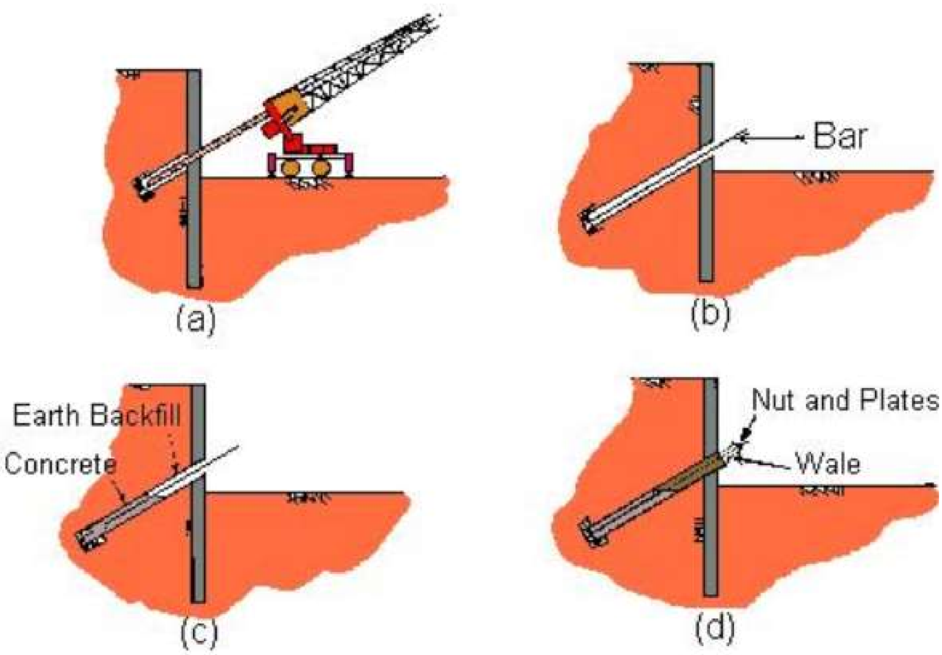

Figure 5.13 Tie back system construction stages[23]

5.3.9 Micro-pile

In a situation where micro-pylons are used to strengthen and improve the foundation bed, the micro-fiber technical calculation is similar to that of conventional candles. These calculations are based on three sections of the structural design, geotechnical design and punch cutting control. On the one hand, the microfiber has a weapon element including a thick steel frame and an armature armor, the ability to transfer and load the load to lower resistant layers, and also the control of the seating because of the high hardness of the steel and the depth of the soil, and on the other hand, due to the injection of cement slurry Mechanical properties of the soil, such as hardness, compressibility, bearing capacity, friction coefficient and adhesion, etc., are improved.

Therefore, micro-pile has superiority in comparison with other methods such as drilling and injection, soil stabilization with cement or lime, dynamic density, etc. due to combined performance (using barrier elements and soil remediation). It should also be noted that the micro-pile knocking operation causes the kinetic and vibrational energy transfer to the soil mass, which will result in the consolidation and density of the soil mass around the micro-pile. This is especially evident in granular soils. Also, in the event of the presence of submerged subsurface layers, it is difficult or impossible to knock large diameter piles, the small diameter of the micro-pile can be the best solution.

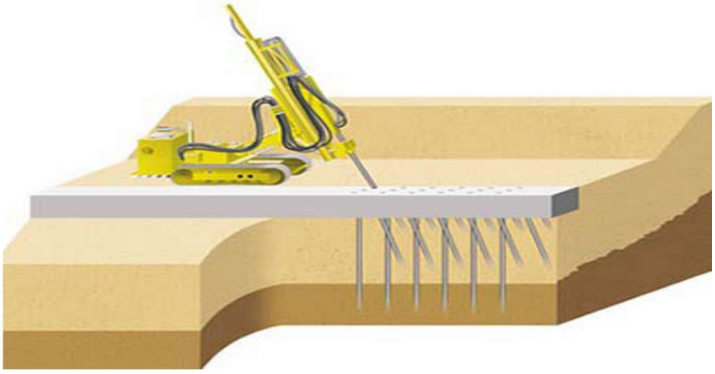

Figure 5.14: Micro pile execution [23]

5.3.10 Truss – Raker

This method is one of the most appropriate and most common methods for the implementation of Guardian structures in urban areas. To carry out this type of guardian structure, we first dig a well in the place of the vertical joints of the truss, which are adjacent to the hollow wall. Then, we place the inside of the candle in a candle and place the vertical member inside the candle, and then we will attach the candle. After the hardening of the bottom concrete, the vertebrae will be placed inside the candle. Then we enclose the enclosed soil between the horizontal and vertical trusses along the wall along the steps, and gradually installing the truss members gradually until the truss is completed.

Figure 5.15 Truss-rocker implementation

6. MODELLING IN PLAXIS

6.1 Introduction

In this chapter, the methods employed by Plaxis 2D software, assumptions, methods, and other related items will be discussed.

In the beginning, we introduce the Plaxis 2D software used in this study, [24], then we fully describe numerical modeling with data and applied assumptions. In the next section, how to extract the results and parameters discussed in the study will be explained.

Plaxis 2D is a finite element program that is used to investigate and analyze offshore foundation or tunnels or obtaining factor of safety for slopes. Unlike its not so sophisticated appearance, the program creates complex, finite elemental models with advanced output, with simple inputs and simple graphical compositions. The design of this software is such that the user will be able to work with it after learning a bit of software.

With the help of this software, it is possible to model and stage seedlings with loading conditions and different boundary conditions using 6-node and 15-node triangular elements. The first edition of this software was designed to analyze the soil dams constructed on soft soils in the lower reaches of the Netherlands and commissioned by the country's water resources management at Delft University of Technology in 1987, and then in 1993, its capabilities were developed by the The Center for Civil Engineering Research and Codes has also been endorsed.

6.2 Models and General Aspects of Them

The first step in examining each project is its geometric modeling. The geometric model means a form of a real 3D problem that is defined through work planes and boreholes. The geometric model should include soil layers, structural elements and loading, and then analyze the results by drawing phases in phases.

6.2.1 Points

Specify the start and end points of the lines. Also, the points can be used to fix the position of springs and points of forces and to clean the locally positioned mesh elements.

6.2.2 Lines

Lines are used to describe the physical limits of geometry, structural alignment lines, and discontinuities in geometry such as walls or pylons or excavation areas.

6.2.3 Clusters

Clusters are areas that are entirely enclosed by lines. Plaxis automatically detects clusters based on input geometric lines. Within a cluster, all properties are homogeneous. Clusters can be considered as a homogeneous part of a structure with a soil layer. The cluster-related actions are related to all elements within the cluster.

After creating a two-dimensional geometric model at the work level, based on the combination of clusters and lines in a geometric model, the two-dimensional elemental mesh of six nodes can be generated automatically.

6.2.4 Elements

During the mesh construction, the elements consist of six triangular nodes within the work surface created by the two-dimensional mesh production.

6.2.5 Stress Points

Unlike displacements, tensions and strains are calculated by the Gaussian integral method. A 15-node triangular element consists of six stress points. It is possible to select the points of interest in knowing the tension in them in order to plot the tension diagram.

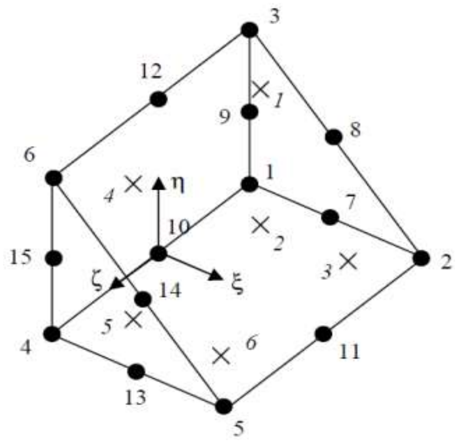

Figure 6.1: Distribution of nodes (•) and stress points (x) in a 15-node triangular element [24]

The first step in examining each project is its geometric modeling. The geometric model means a form of a real two-dimensional problem.The geometric model should include soil layers, structural elements and Then, by plotting the steps in the phase separation section, the results are obtained.

6.3 Entering the specification of earth materials

After clicking on the  icon, the screening screen for soil materials opens. In drop down menu, we choose Set Type and then Soil and Interfaces.Then select the New option and enter the soil profile. Specifications are entered in the 3 sections of the Interfaces, Parameters, General.

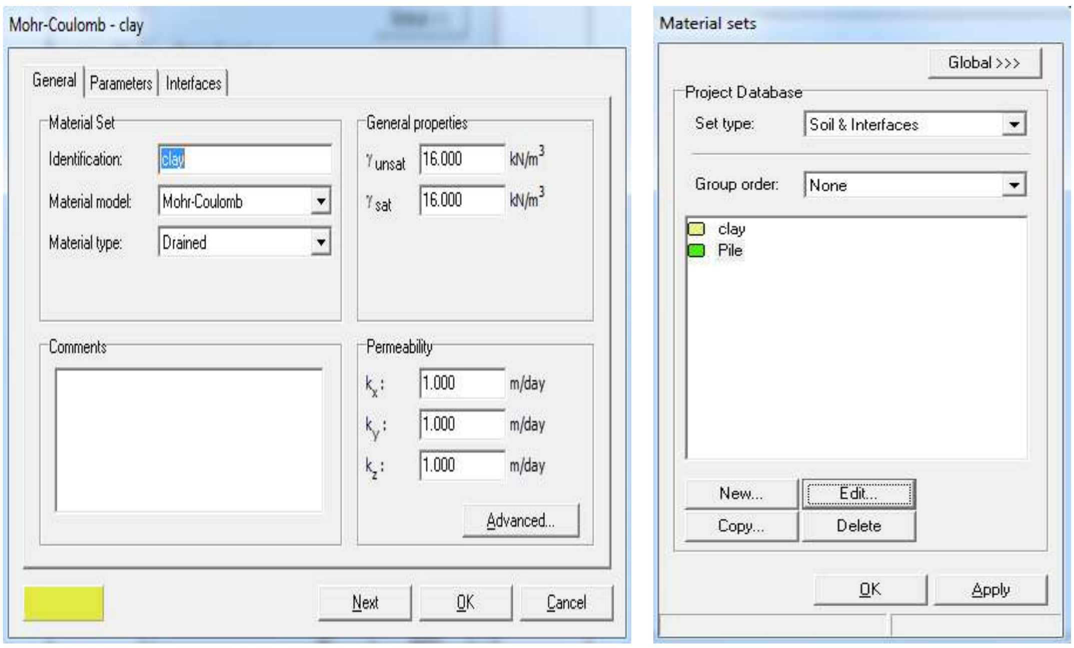

Figure 6.2: Introduction of soil materials.

The third part defines interfaces. In modeling, there is a very thin sheet of soil between soil and soil that is neither soil nor soil. In fact, for the proper modeling of the interaction between the soil and the coefficient structure in Plaxis software, it is defined by the makers. The numbers that software makers have suggested for this coefficient is according to the shape (Figure 6.2)

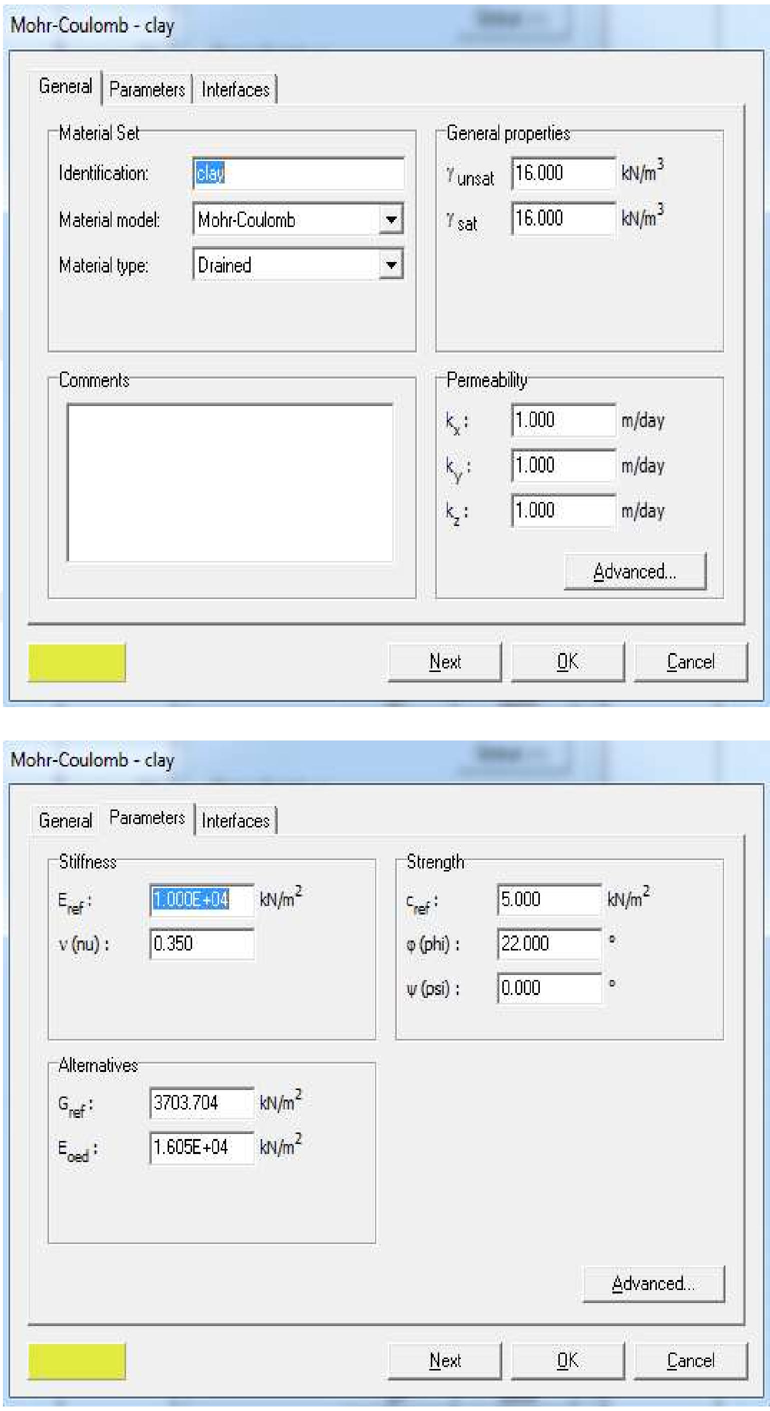

Figure 6.3: General and Parameters menu

Suggestions for R_{inter}:

- Interaction sand/steel = R_{inter} ˜ 0.6 – 0.7
- Interaction clay/steel = R_{inter} ˜ 0.5
- Interaction sand/concrete = R_{inter} ˜ 1.0 – 0.8
- Interaction clay/concrete = R_{inter} ˜ 1.0 – 0.7
- Interaction soil/geogrid = R_{inter} ˜ 1.0
  (interface may not be required)
- Interaction soil/geotextile = R_{inter} ˜ 0.9 – 0.5
  (foil, textile)

Figure 6.4: Soil-Structural Interaction Values

6.4 Mesh

By choosing the mesh option and its grade, the model will be meshed and the meshes will be finer at the critical points.

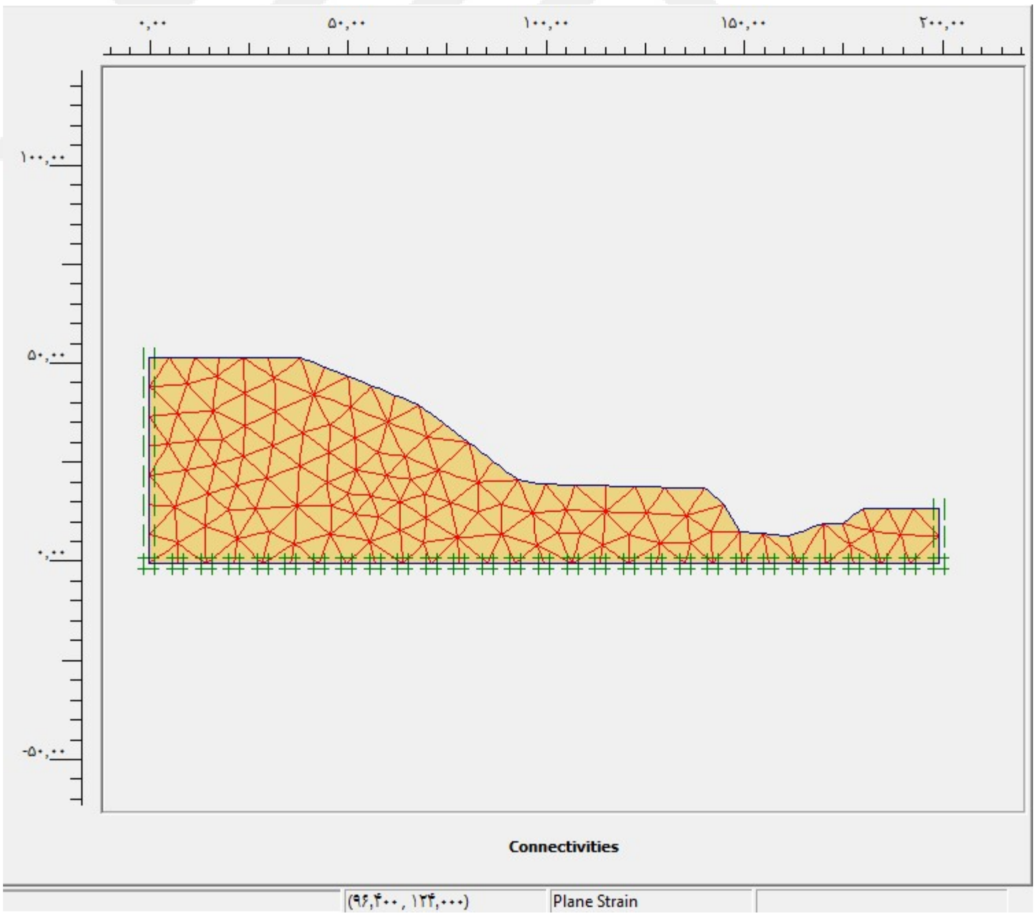

Figure 6.5: Mesh

6.5 Loading

Plaxis starts the load from zero and loads it to the final load entered by the user. Plaxis applies the load as a M-Stage and imports from 0 to 1.

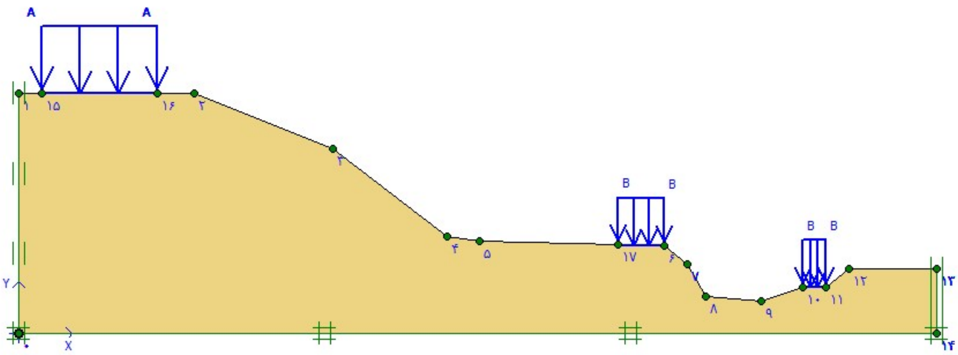

Figure 6.6: Loading

6.6 Calculations

After the modeling, we enter the computational stage. Start the calculation program from the initial phase and continue to the end. This step starts by clicking the Calculation icon.

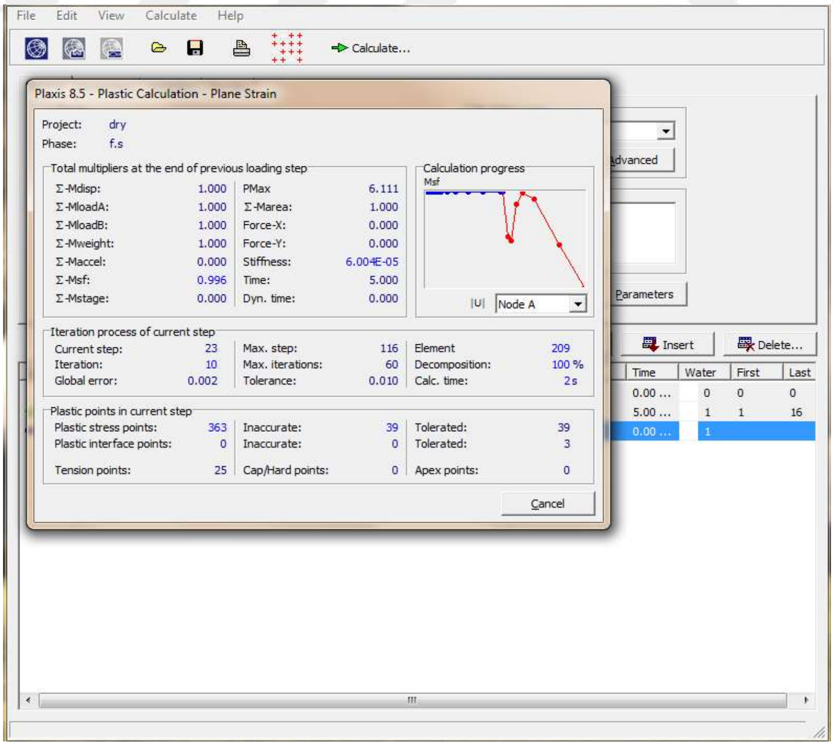

Figure 6.7: Calculation page

6.7 Output

After finishing the model analysis, outputs can be viewed with the help of the Output program. In this section we can see various data, including types of tensions, strains and displacements in different directions.

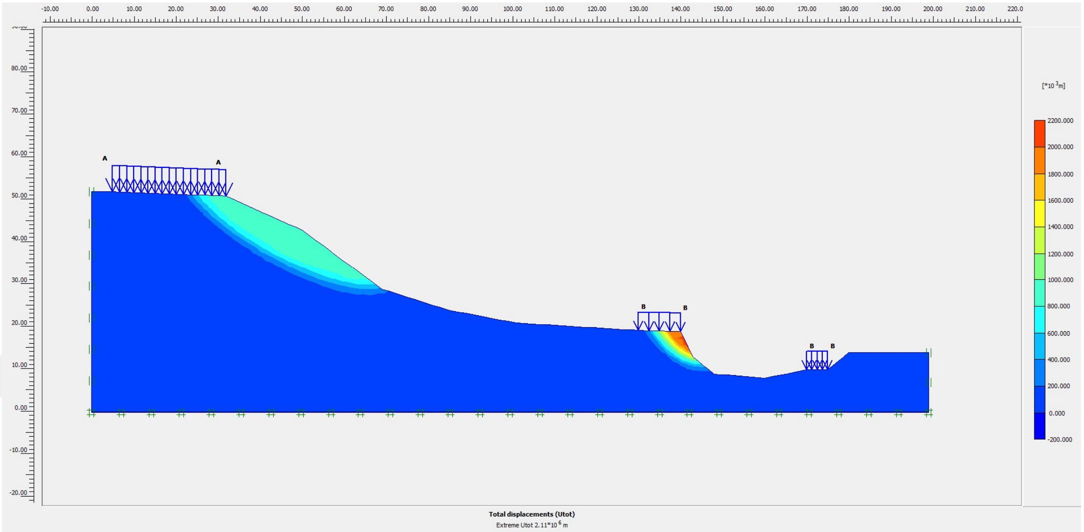

Figure 6.8: Example of Outputs

7. CASE STUDY

7.1 The NIKAWA Landslide

One of the largest disasters from the landslide was the Nikawa landslide, which destroyed 11 houses and buried 34 persons. The landslide volume was 110,000-120,000m^3. All of the people buried by the landslide mass were note rescued. Photograph JGS, 1996 shows an aerial view of the source area of Nikawa landslide taken by k.Sassa on 21 January 1995. Then plan of the slope before the landslide and landslide area are showing in Figure 7.1. Locations of boring and excavations pits for observation and sampling points are plotted. The white arrow in Figure 7.2shows standing ground water on secondary moved debris. The period was very dry. The level of the ground water table was re-confirmed later by monitoring the level in the borings. Trees, buildings are shown only schematically. The slope was about 20° but the energy line connecting the top of the source area for the landslide and the top of deposit landslide mass show and 11.4° degrees. [25]

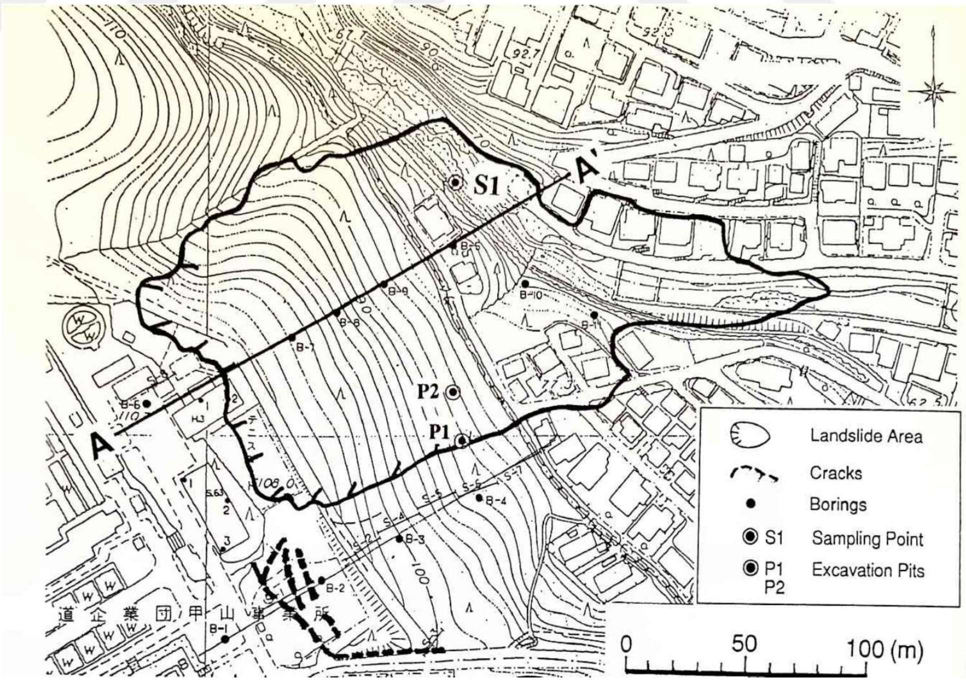

Figure 7.1 Plan of slope before Nikawa landslide and the outline of landslide area [25]

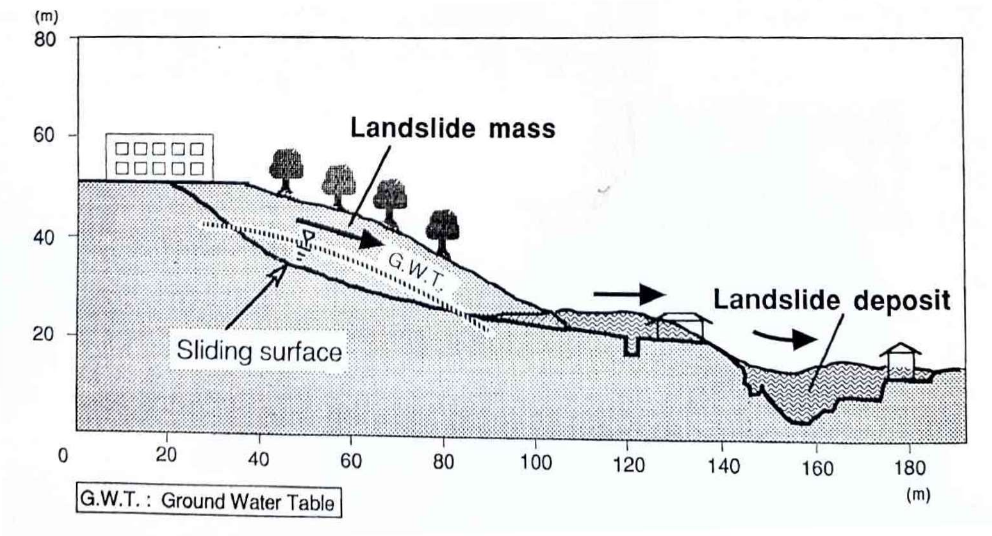

Figure 7.2: The central section A-A' of the Nikawa landslide [25]

The geotechnical specifications of the materials and soil used for the models made in this thesis are extracted from the article 'Some experience in numerical modelling of unsaturated slope instabilities, which is written by Josef Josifovski and Stanislav Lennart the following table. It is worth noting that modeling is based on Mohr Coulomb's method for determining the maximum displacement and obtaining the factor of safety by the usage of fi-c reduction in plaxis 2D.

Table 7.1: Soil material parameters

Properties	$\gamma(KN/m^3)$	φ(degree)	C(Kpa)	ύ	E(Kpa)
Sandy-clay	20.14	23	12	0.32	10000

Table 7.2: Hydraulic data

Properties	K_{sat} (m/sn^2)	θ_s	θ_r	φ	n
Sandy-clay	1E-6	10.45	1.1	0.355	4.17

For this case study we considered two different conditions of analysis as:

1- Dry condition
2- Wet condition

For both conditions, FOS values are obtained.

7.2 Dry Condition Analyses

In this section, modeling for two dry models and another in saturated soil state using software Plaxis 2D, And the results of the sessions and tensions were taken in the loading phase and then in the next phase and with the method used phi-C reduction factor of safety FOS obtained. Modeling, results, meeting values and tensions and confidence coefficients are as follows.

The initial model and defined boundary conditions are shown in Figure 7.3.

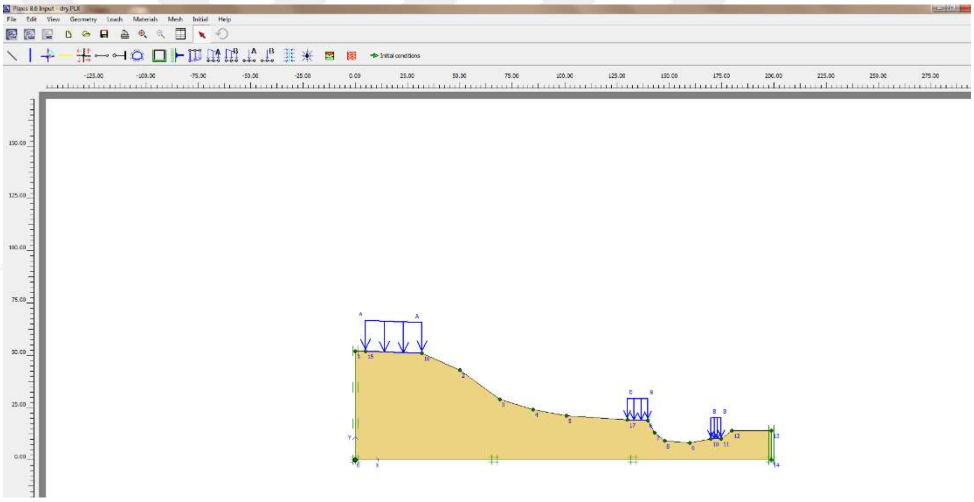

Figure 7.3 Primary model with loading

In view of the dryness of the environment, we define the level of alignment below the model. (Figure 7.4)

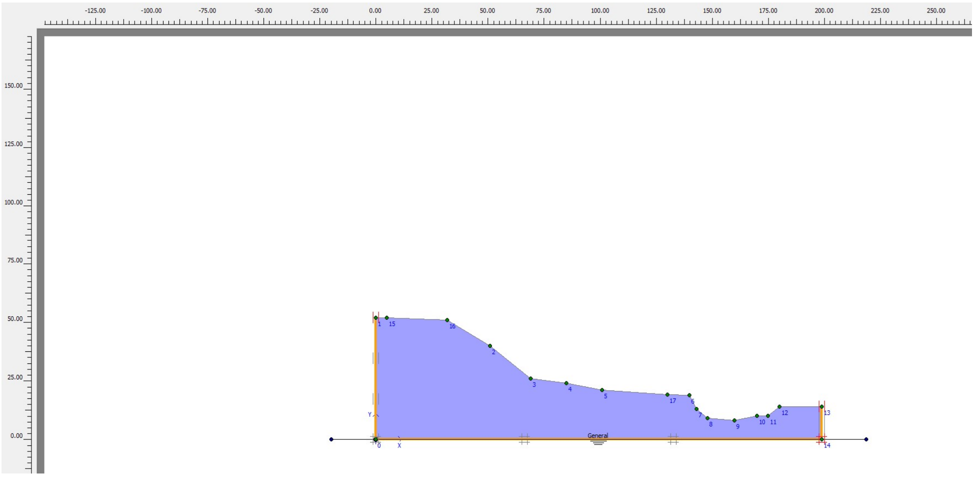

Figure 7.4 Determine water level in dry state

Apply coefficient K_0 by the usage of plaxis 2D initial calculation as shown in Figure 7.5.

Phase and run calculations to determine the summation and coefficient of confidence.

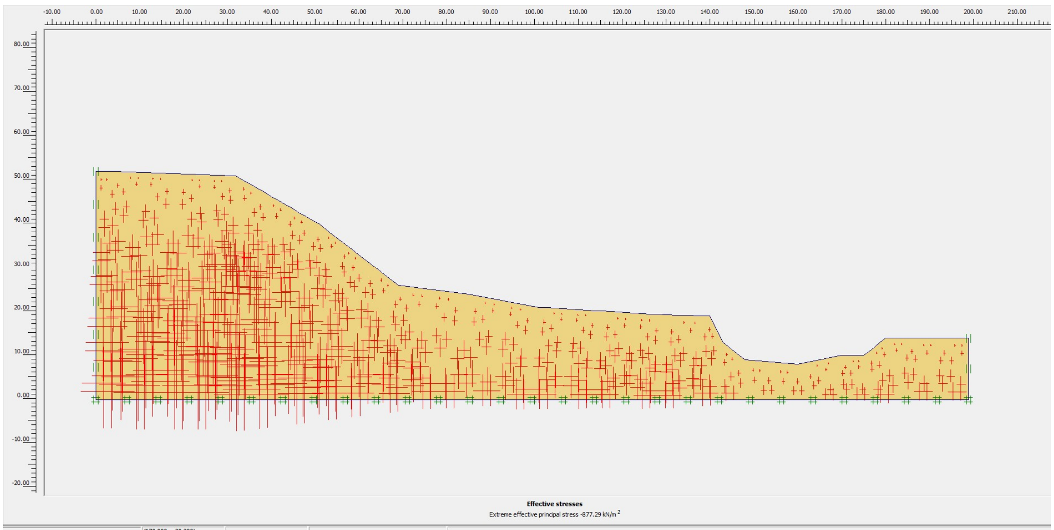

Figure 7.5: Effective stress in dry condition. Total effective stress is 400 kN/m^2..

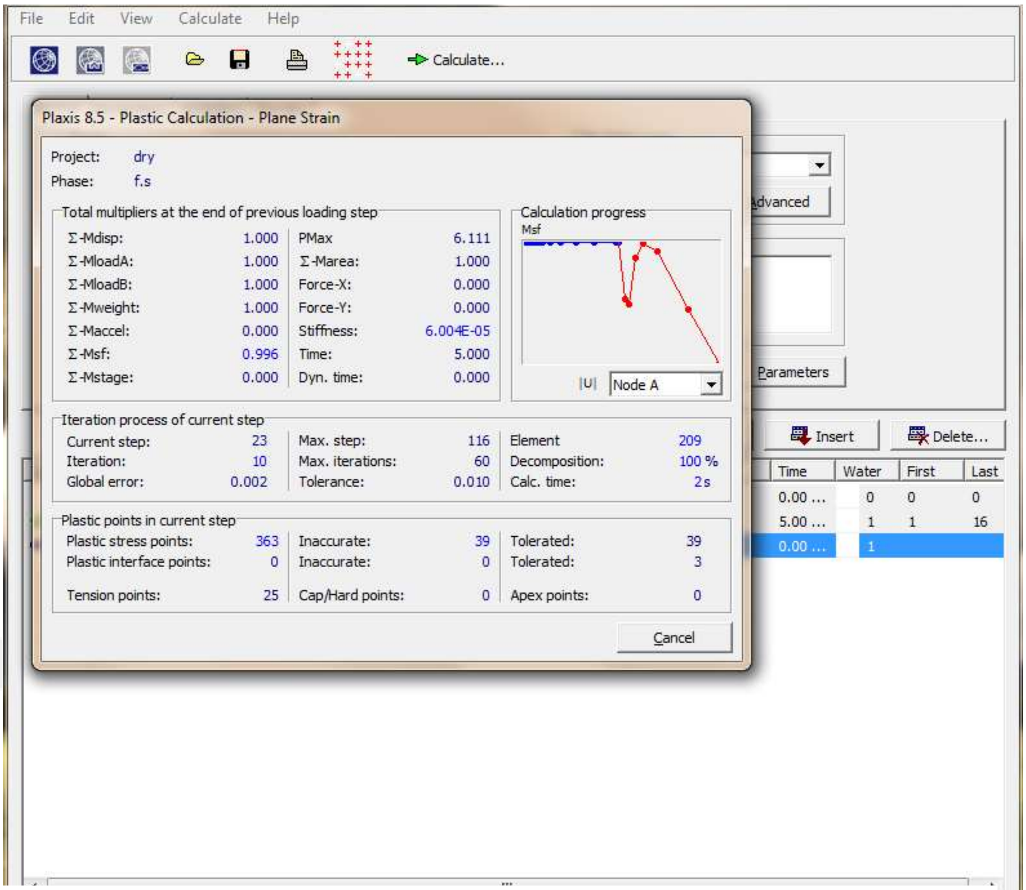

Figure 7.6: Phasing and run process

By examining the outputs of the software, the amount of displacement in the direction of x and the summation and the values of the created values under the load of the upper structures are obtained as follows.

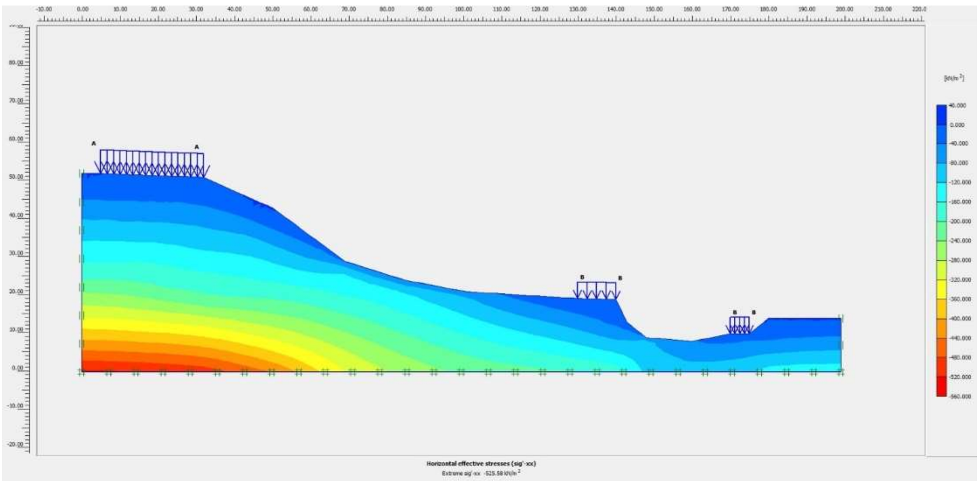

Figure 7.7: Horizontal effective stress

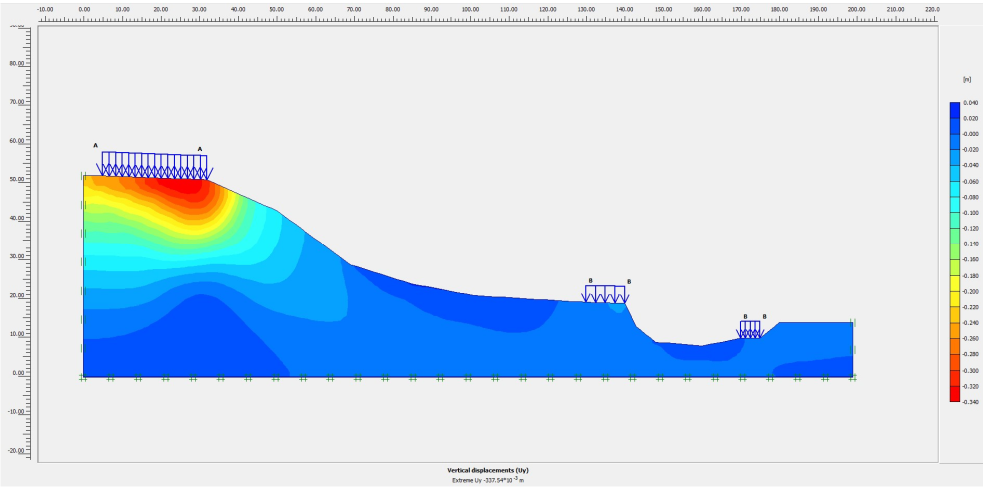

Figure 7.8: Vertical displacement.

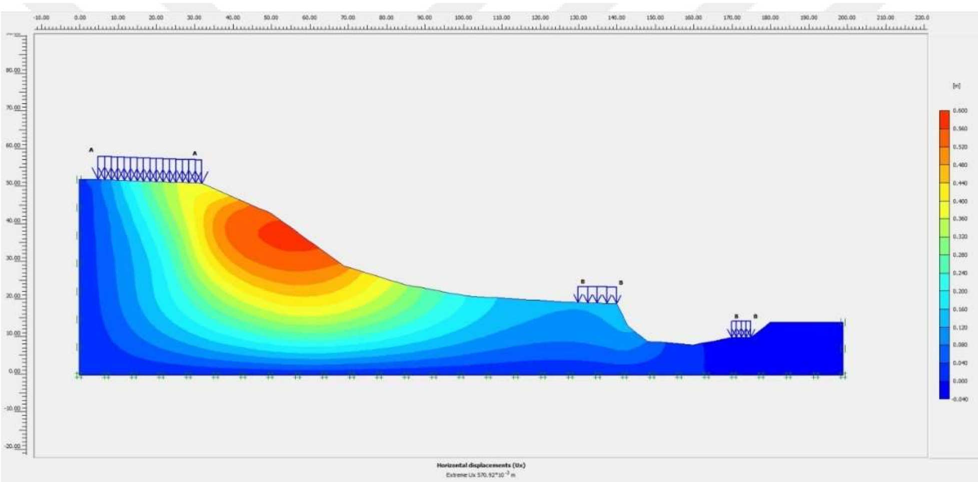

Figure 7.8: Horizontal displacement.

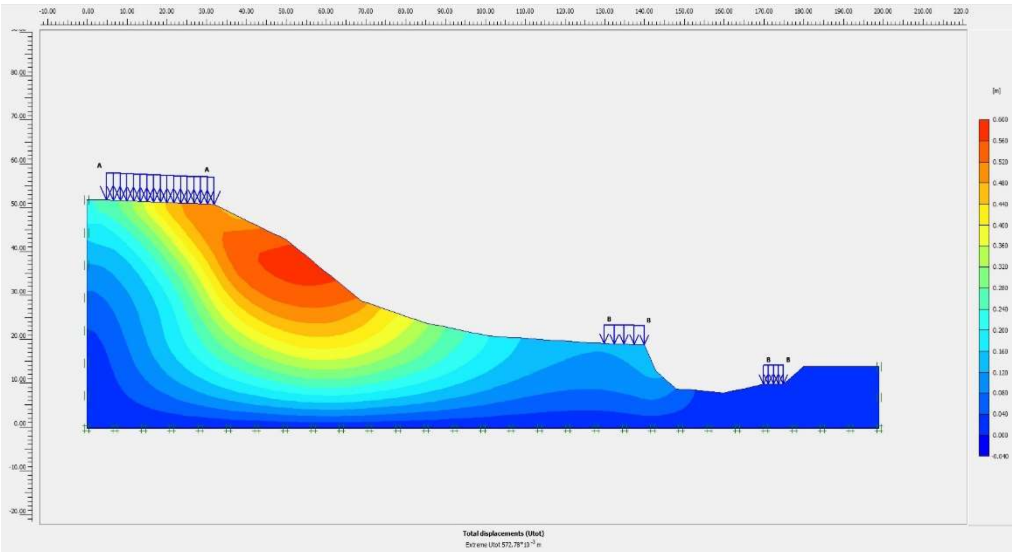

Figure 7.9: Total displacements.

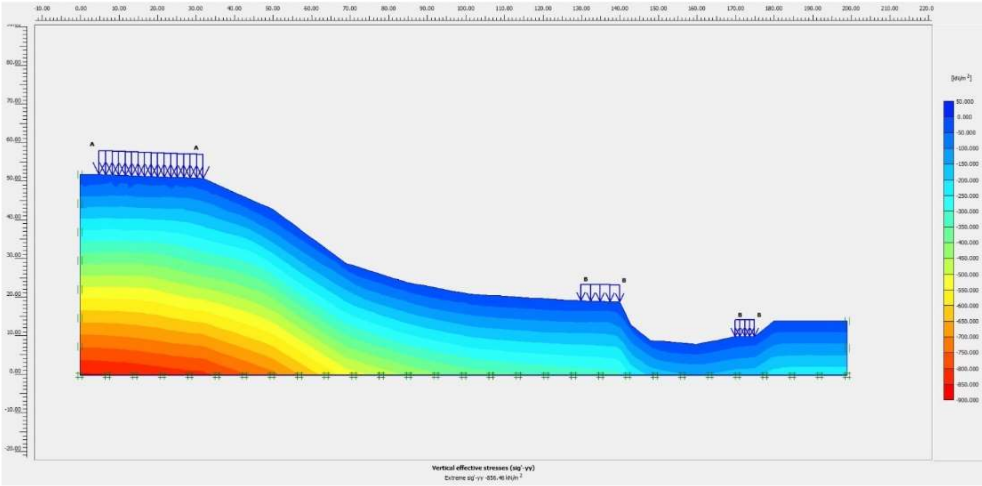

Figure 7.10: Vertical effective stress

As it can be seen, in a dry state, the factor of safety (FOS) obtained from the slip potential is about 1.1, which indicates the numerical stability of the relative, but by examining the regulationn of FHWA the minimum value of the factor of safety is equal to the static state (min factor of safety)=1.25 Which indicates the likelihood of a future slip under the atmospheric or seismic conditions in the area.

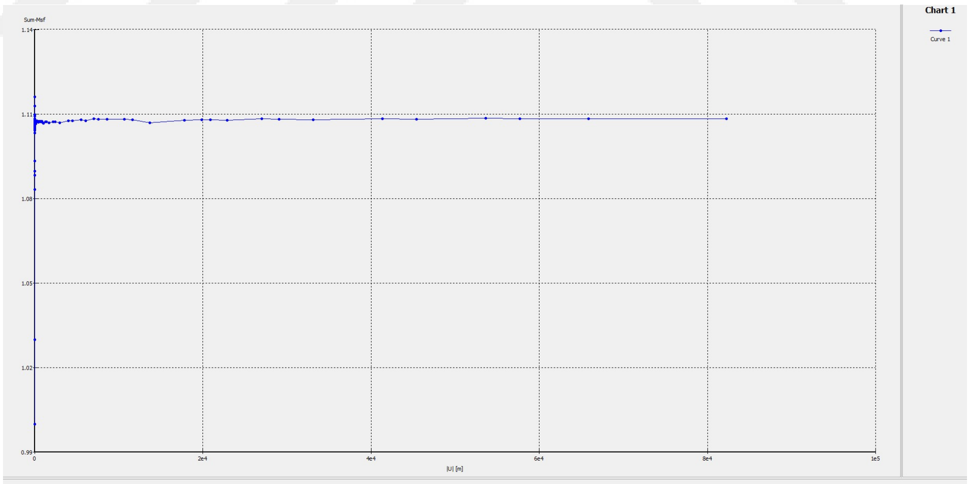

Figure 7.11: Value of factor of safety.

7.3 Wet Condition Analyses

The initial model and boundary conditions are in saturation mode as shown in Figure 7.12.

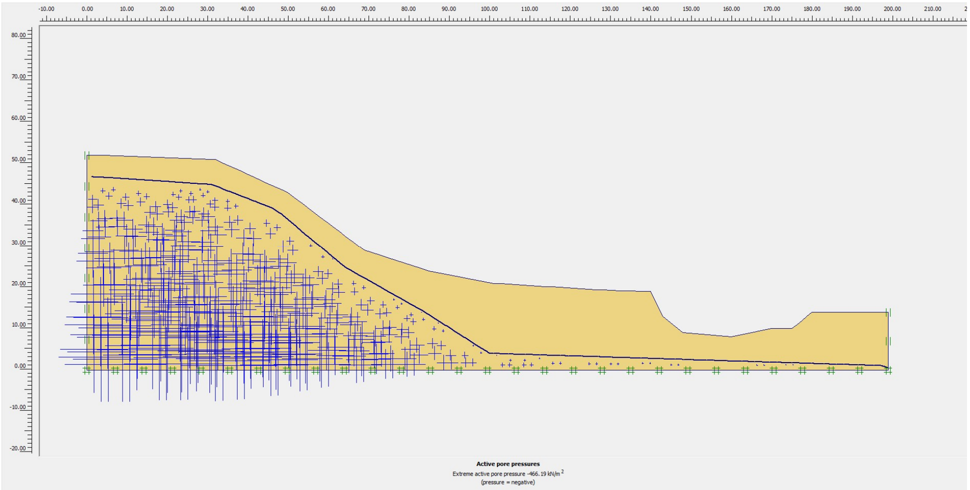

Figure 7.12: Primary model in saturation mode. Total effective stress is 400 kN/m^2.

The changed shape of the environment is derived from the following figure:

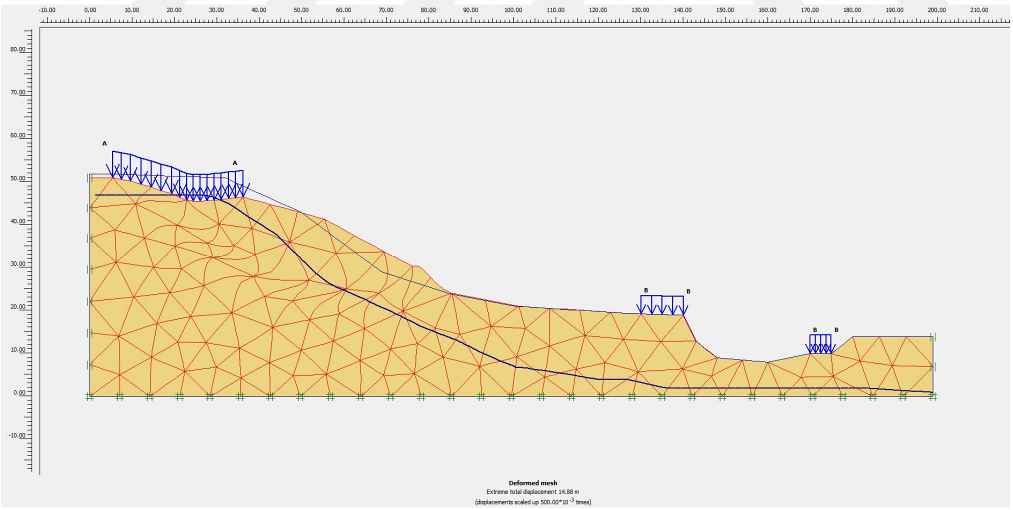

Figure 7.13: Deformed mesh.

By examining the outputs of the software, the amount of displacement in the direction of x and the summation and the values of the created values under the load of the upper structures are obtained as follows.

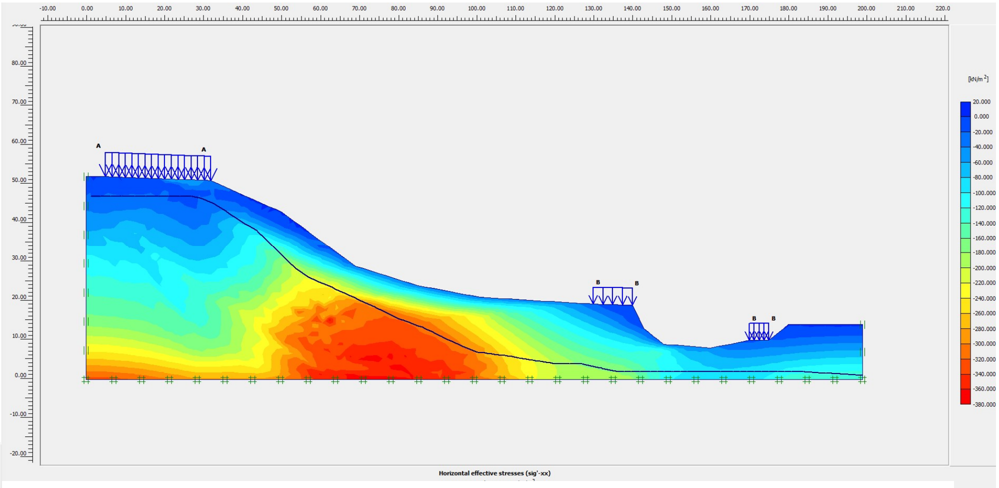

Figure 7.14: Horizontal effective stress.

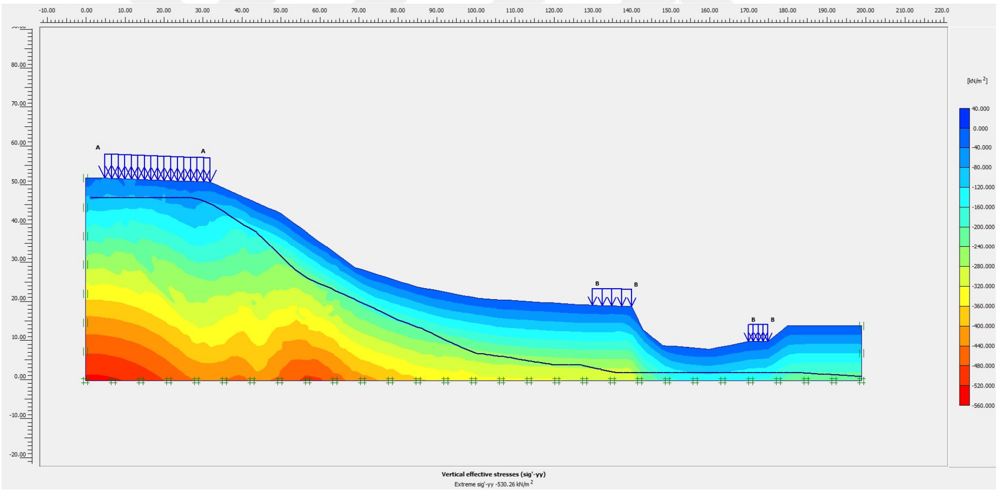

Figure 7.15: Vertical effective stress.

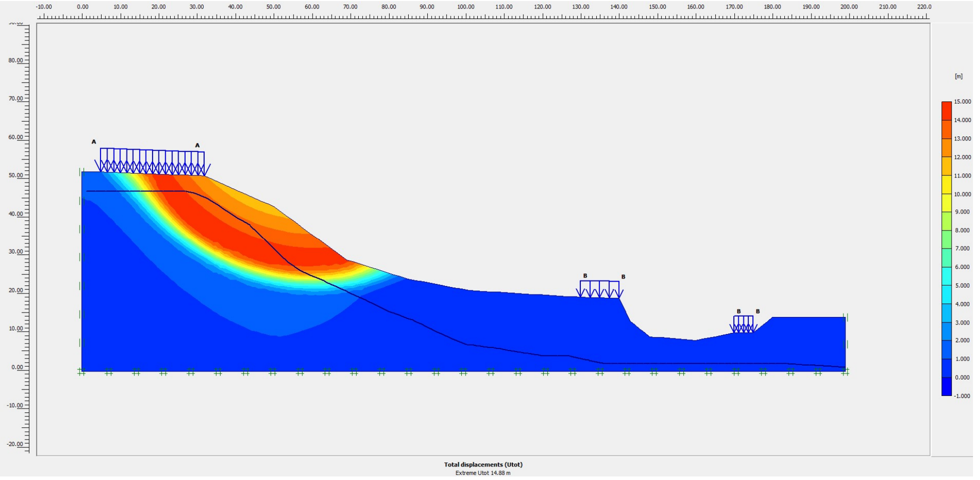

Figure 7.16: total displacements

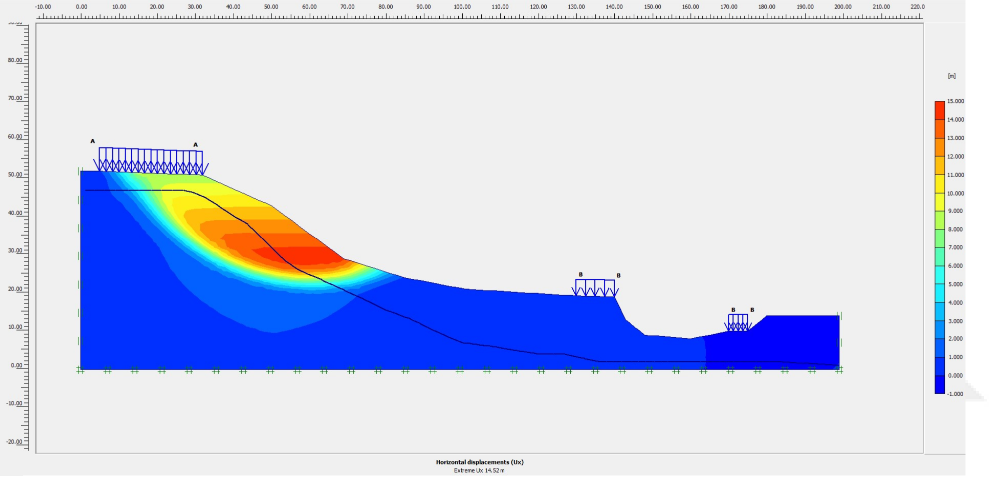

Figure 7.17: Horizontal displacement.

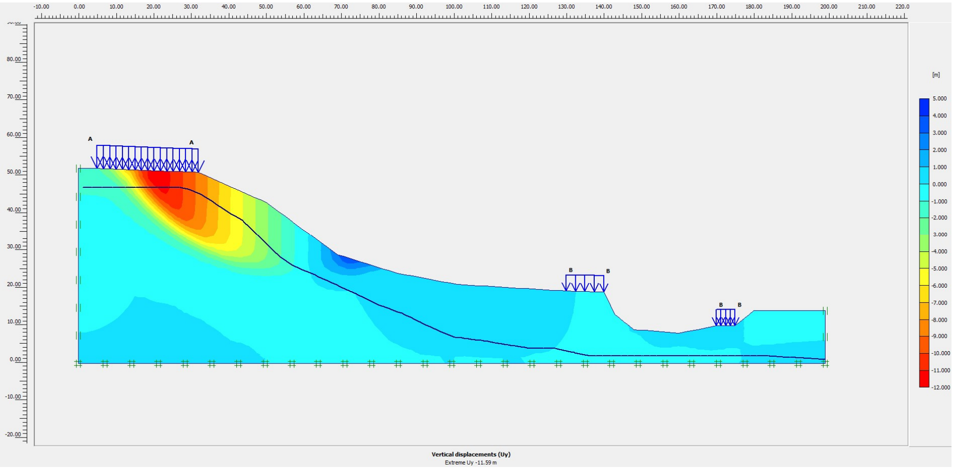

Figure .7 19: Vertical displacements.

As can be seen, in the saturation state, factor of safety obtained from the slip potential is about 0.95, which is numerically unstable, indicating a slip in saturation state.

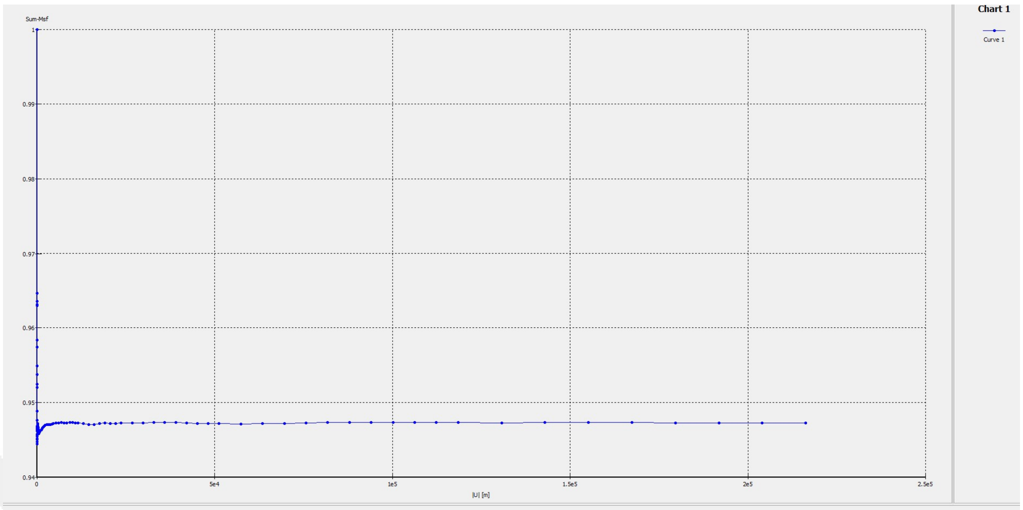

Figure 7.18: Factor of safety in saturation condition. FOS value is 1.11

7.4 Analysis of Displacements

The following results are visible by carefully examining the software outputs of wet and dry movements. The maximum displacement was 57.28 cm in dry state, its position was in the middle of the slope, but with increasing water level, the maximum displacement increased significantly to 14.88 meters and its location below the structure is located above the slip potential. The reason for this is the increase in water levels and the reduction of effective stress between soil grains and the reduction of friction between grains.

8. COMPARING THREE DIFFERENT STABILIZATION METHODS OF THE CASE STUDY

In this section we applied three different sabilization methods within two different soil saturation conditions as dry and wet. FOS values are obtained and compared from safety and economy point of views.

Structural elements are assignd according to stress distribution counter. These elements are connecting high stress layers to low stress layers.

8.1 Stabilization with Locating Pile

Since the slip potential is unstable, a structural element such as a pile is used to stabilize and increase the Factor of safety. The specification of the constructed pile is described in the following table and its distances are 15 meters apart.

Table 8.1 Physical properties of pile[1]

Pile	Unit weight (kN/m3)	Elastic modulus (MPa)	Poisson's ratio
C25 concrete	24	2.9e4	0.2

Table 8.2 Dimensional properties of pile system

Pile dimension(m)	Vertical distance(m)	Pile length(m)
1,5	1	15

8.1.1 Analising Piled System In Dry Condition

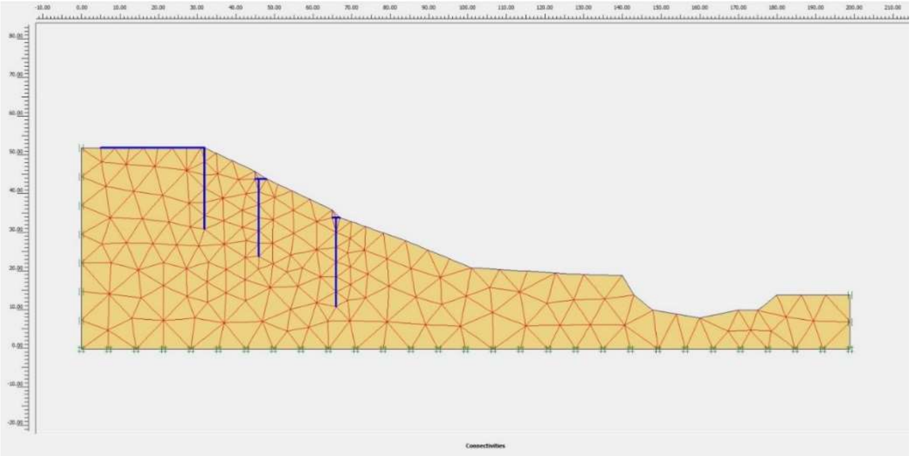

Figure 8.1: Mesh condition in the presence of piles in dry condition.

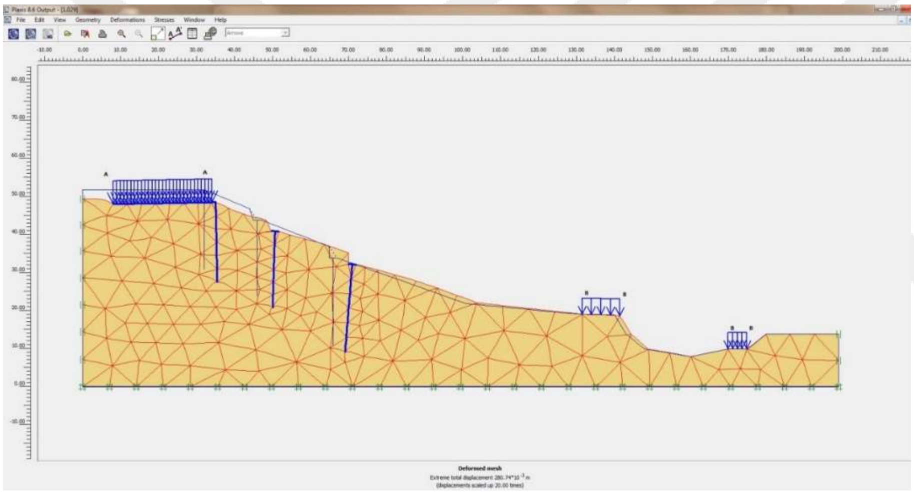

Figure 8.2 Deformed mesh in the presence of piles (scaled up 20 times) in dry condition.

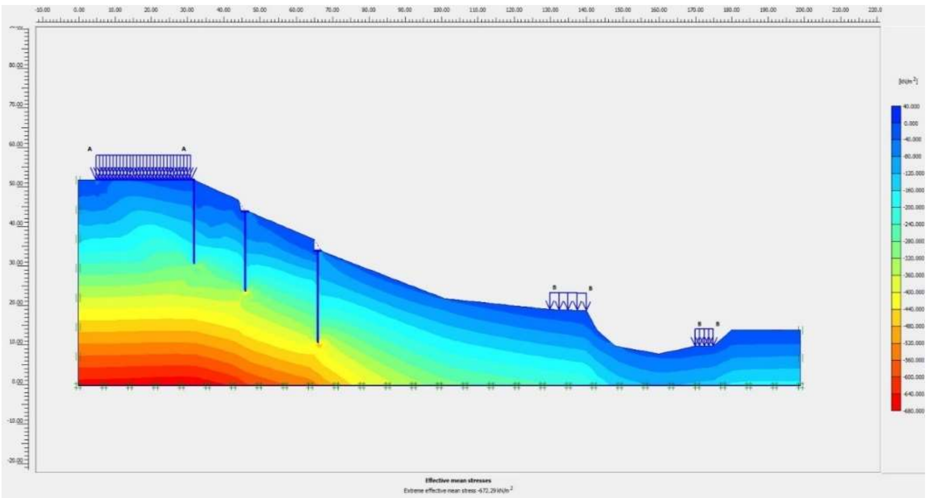

Figure 8.3: Effective stress in the presence of piles in dry condition. Total effective stress is 516kN/m^2..

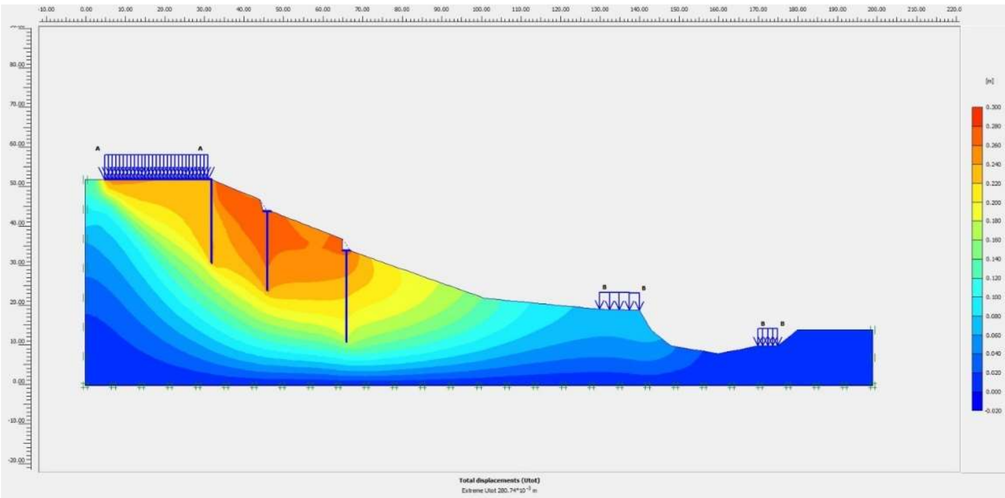

Figure 8.4: Total displacement in the presence of piles in dry condition. Total displacement is 0.281 m.

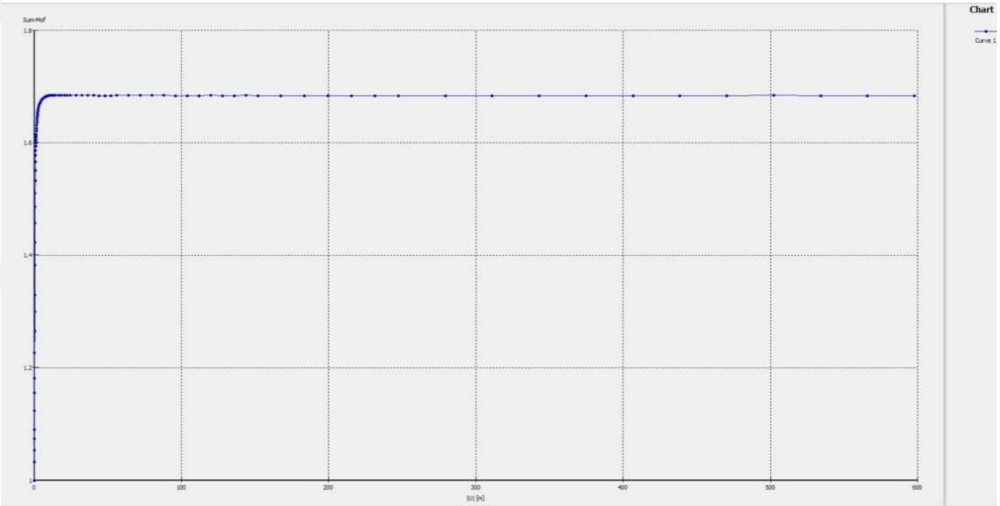

Figure 8.5: Diagram of factor of safety in presence of piles in dry condition. FOS value is 1.68.

As seen value of factor of safety in dry condition increases from 1.1 to 1.67.

8.1.2 Analising piled system in wet condition

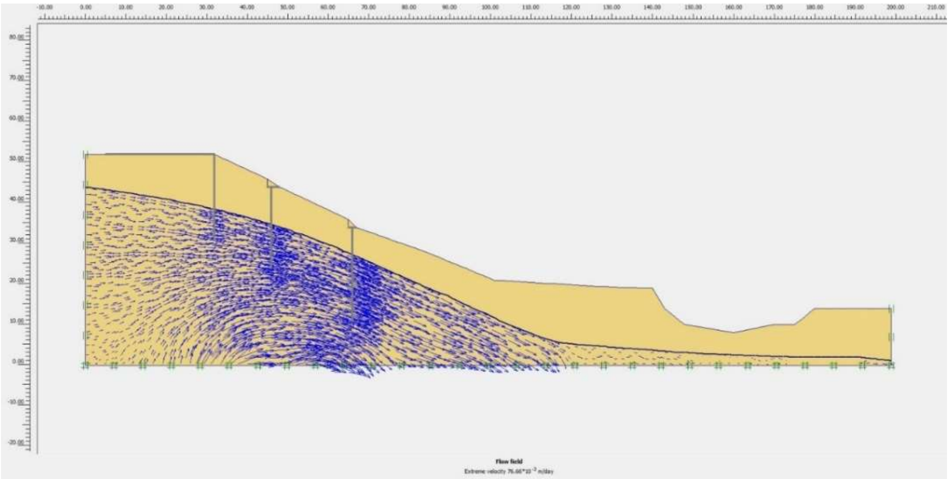

Figure 8.6 Flow field in presence of piles in wet condition

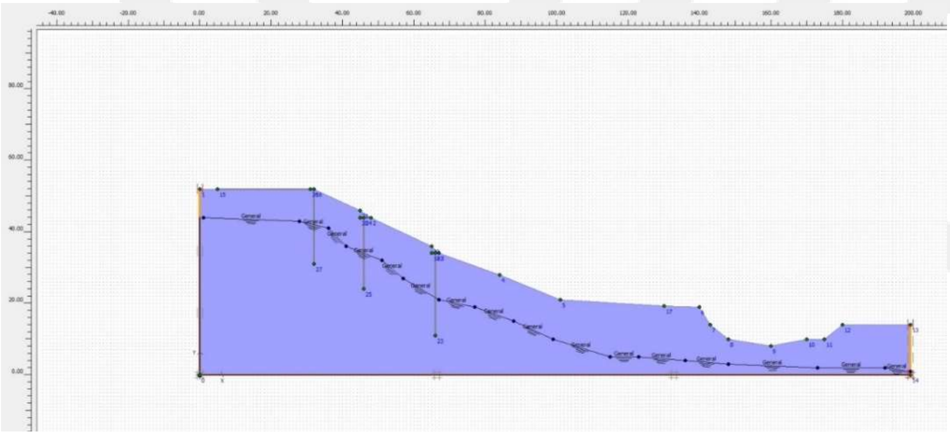

Figure 8.7: Water surface in presence of piles in wet condition.

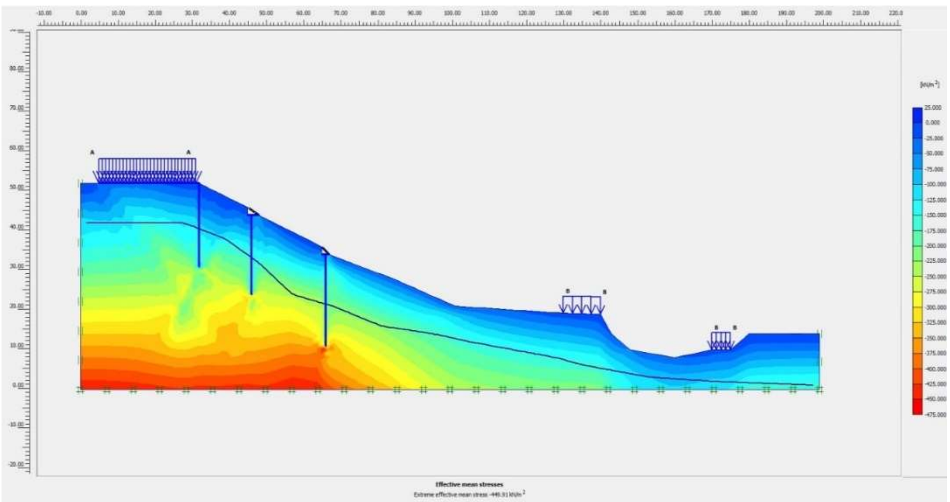

Figure 8.8 Effective stress in presence of piles in wet condition. Total effective stress is 500 kN/m^2..

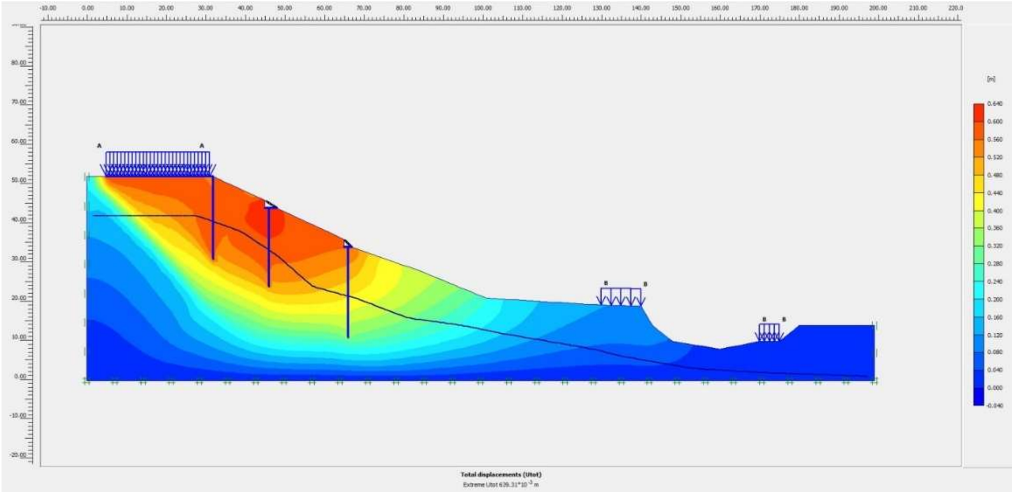

Figure 8.9: Total displacement in presence of piles in wet condition. Total displacement is 0.639 m.

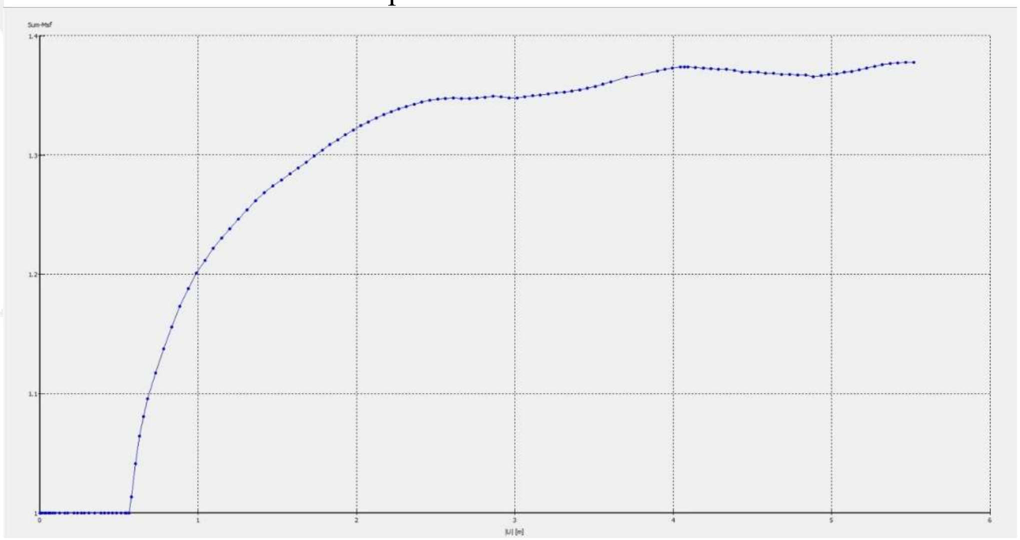

Figure 8.10: Diagram of factor of safety in presence of piles in wet condition. FOS value is 1.37.

As seen value of factor of safety in dry condition increases from 0.9 to 1.37 which is acceptable.

8.2 Stabilization With Locating Micro-pile

8.2.1 Micro-pile in dry condition

Micro-piles are used to stabilize and increase the Factor of safety and compare with other elemans. The specification of the constructed mcro-pile is described in the following table and its distances are 7 meters apart.

Table 8.3 Dimensional properties of micro pile system

Pile dimension(m)	Vertical distance(m)	Pile length(m)
0,5	1	15

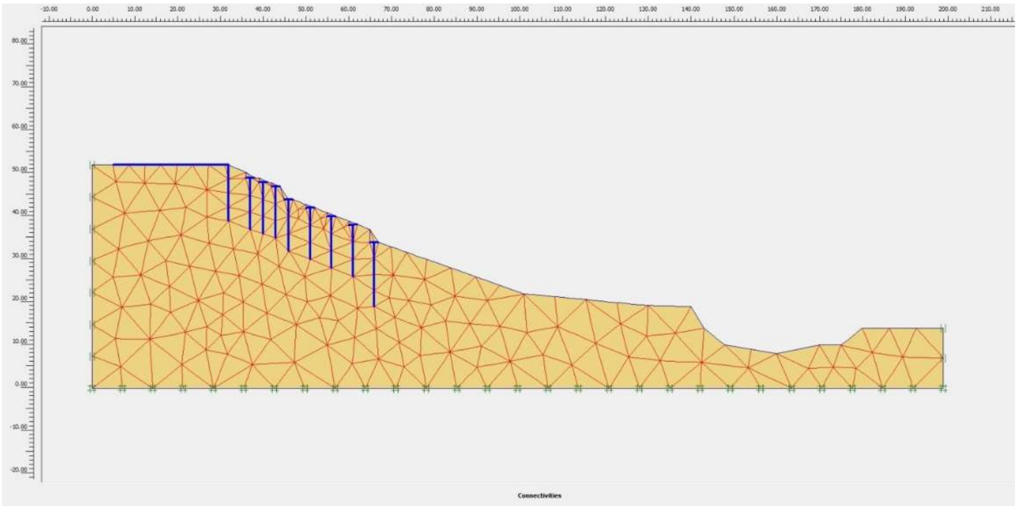

Figure 8.11: Mesh condition in the presence of Micro-piles in dry condition

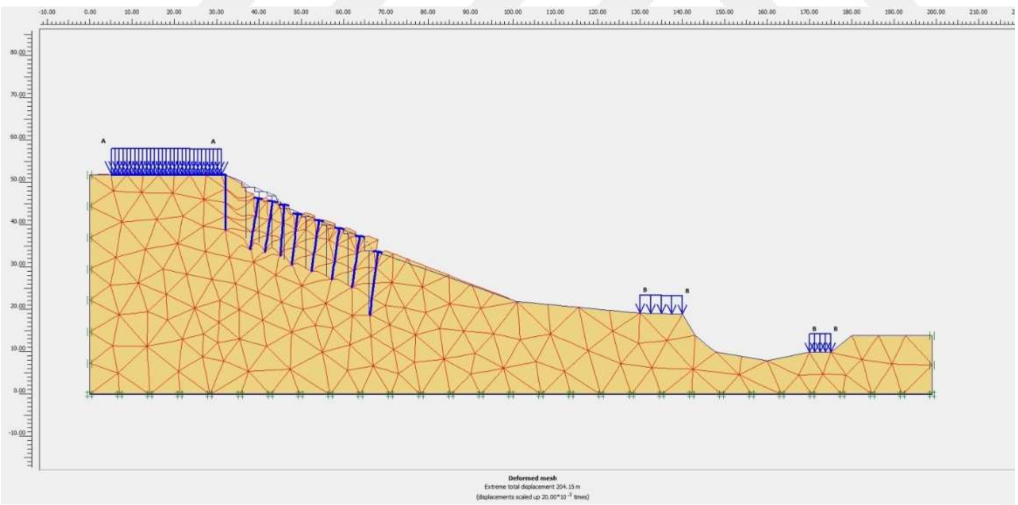

Figure 8.12: Deformed mesh in the presence of Micro-piles(scaled up 20 times) in dry condition

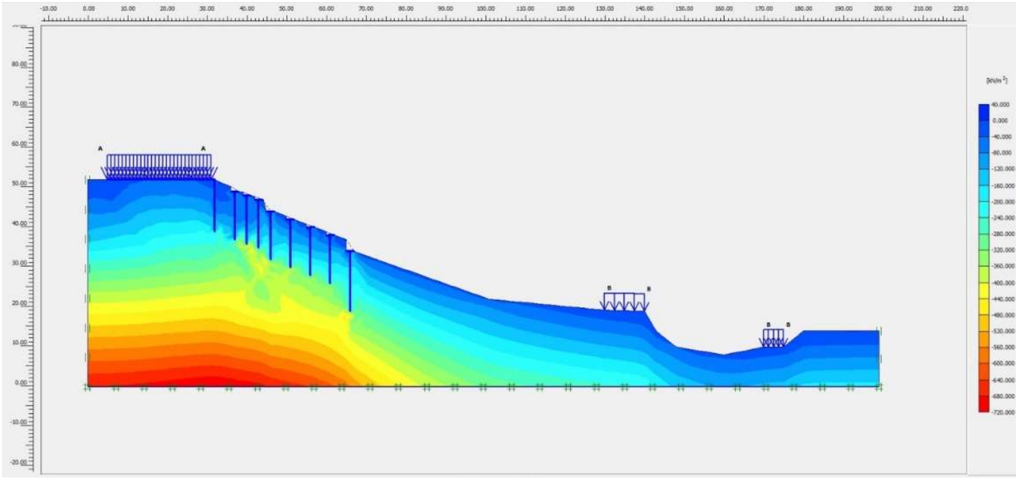

Figure 8.13: Effective stress in the presence of Micro-piles in dry condition. Total effective stress is 718 kN/m^2..

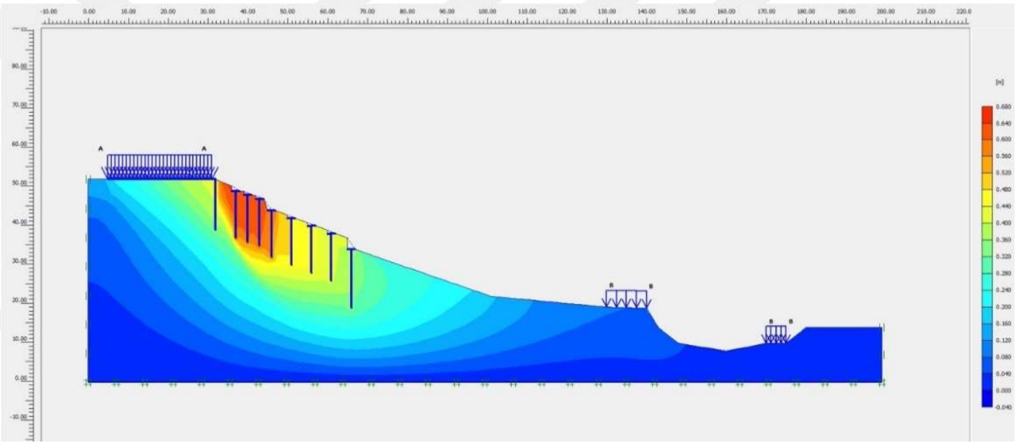

Figure 8.14: Total displacement in the presence of Micro-piles in dry condition. Max. displacement is 0.667 m .

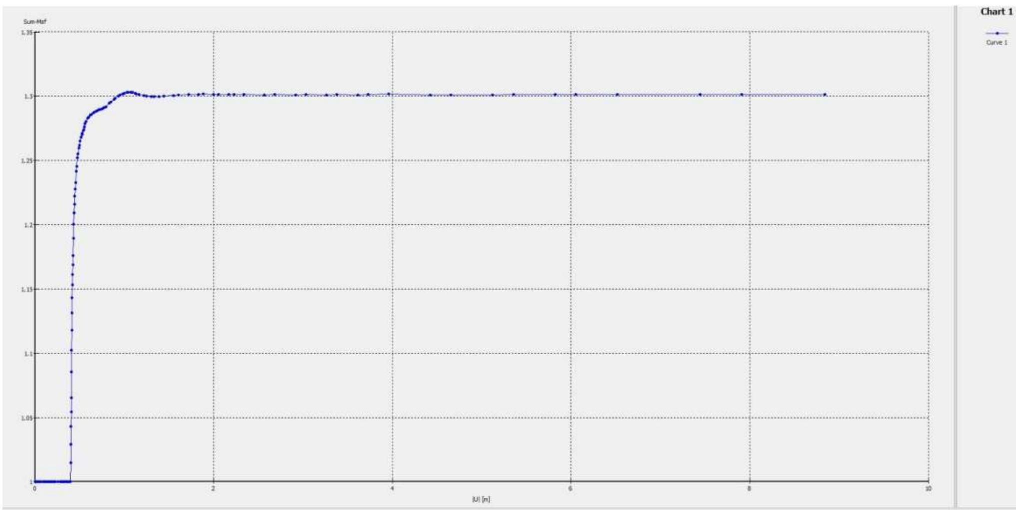

Figure 8.15: Diagram of factor of safety in presence of Micro-piles dry condition. Value of FOS is 1.31.

Value of factor of safety in dry condition in presence of Micro-pile increases from 1.1 to 1.31 which is acceptable.

8.2.2 Micro-Pile In wet condition

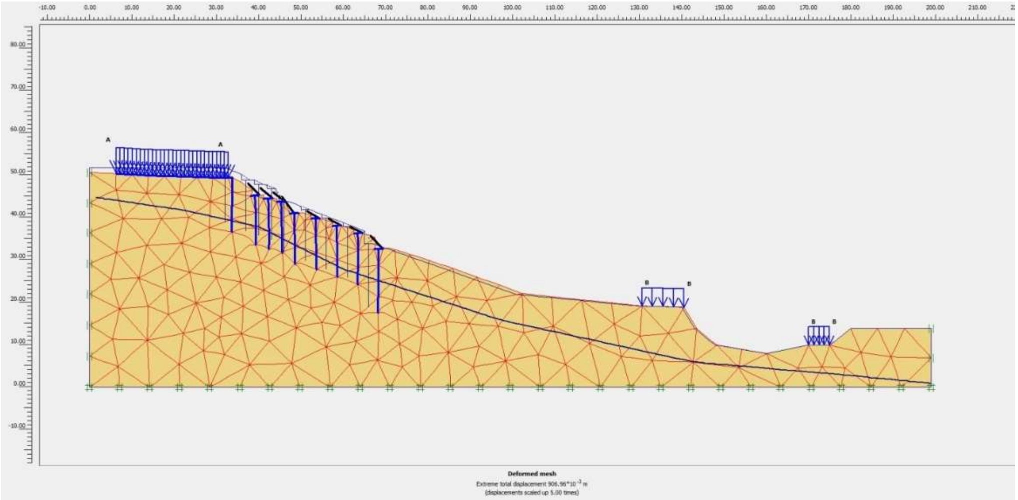

Figure 8.16: Deformed mesh in the presence of Micro-piles (scaled up 5 times) in wet condition

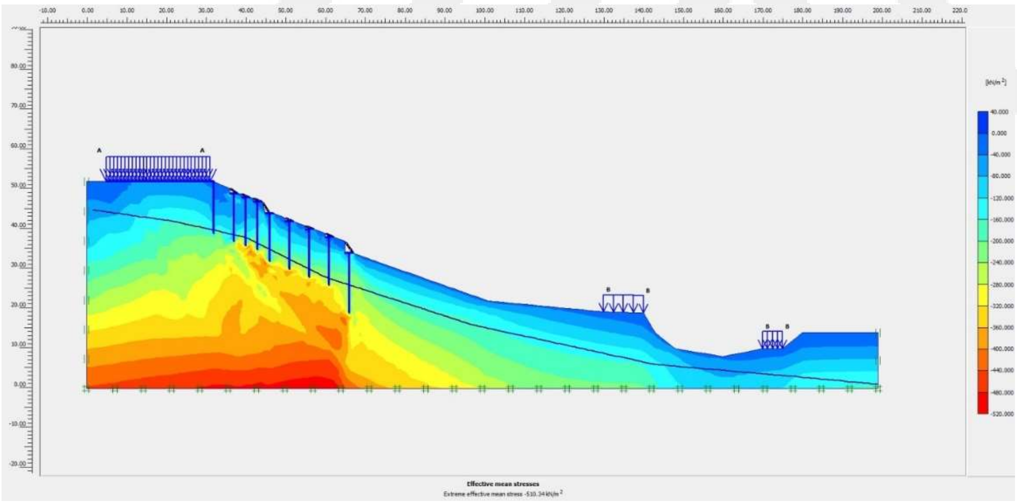

Figure 8.17: Effective stress in the presence of Micro-piles in wet condition. Total effective stress is 510 kN/m^2..

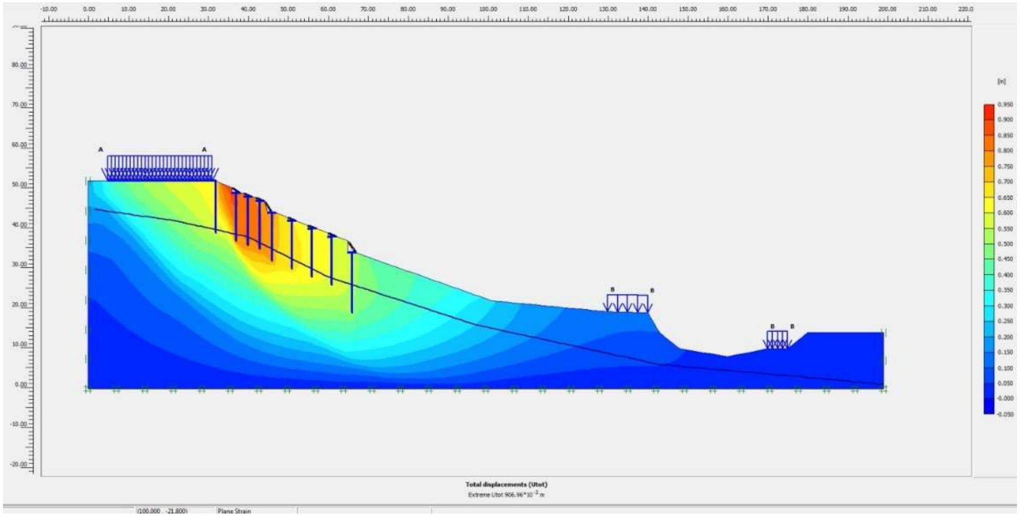

Figure 8.18: Total displacement in the presence of Micro-piles in wet condition. Max. displacement is 0.906 m.

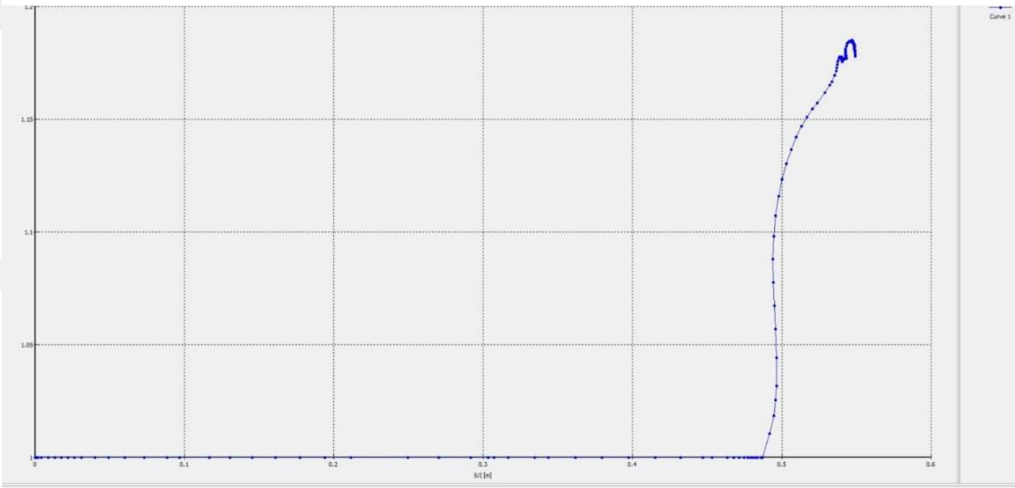

Figure 8.19 : Diagram of factor of safety in presence of Micro-piles in wet condition. FOS value is 1.17

Value of factor of safety in dry condition in presence of Micro-pile increases from 0.9 to 1.17 .

8.3 Stabilization With Soil Nailing

8.3.1 Soil nailing in dry condition

Other structural element that can help to improvement of soil condition is nail. The specification of the Nail is described in the following table and its distances are 5 meters apart.

Table 8.2: Properties of Nail [2]

Nail	
Density (kg/m^2)	7860
Elastic modulus (Gpa)	200
Poisson's ratio	0,3
Yield strength (Mpa)	275
Shotcrete	
Unit weight (kN/m^3)	24
Elastic modulus (Gpa)	16
Poisson's ratio	0,2

Table 8.3 Dimensional properties of nails system

Dimension(m)	Vertical distance(m)	Nail length(m)
0,2	1	2

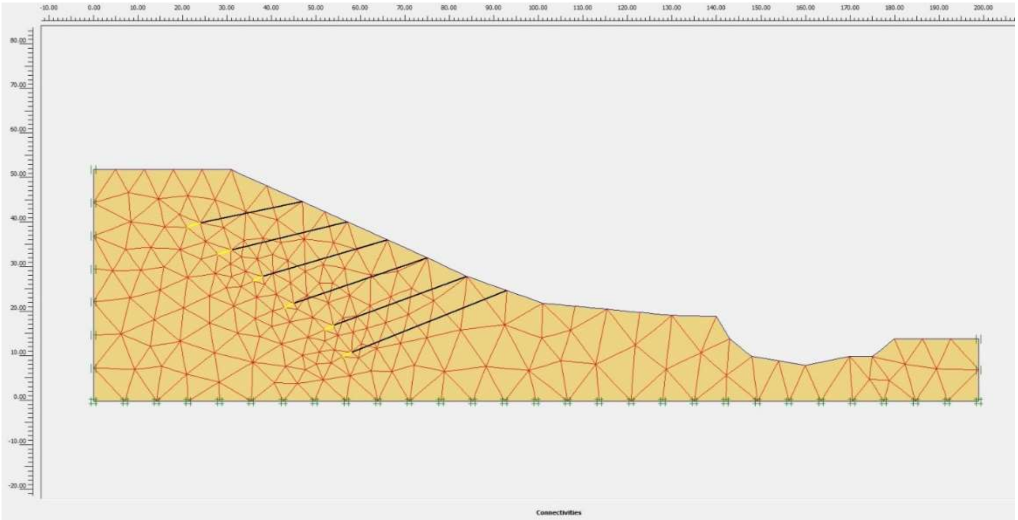

Figure 8.20: Mesh in the presence of Nail in dry condition.

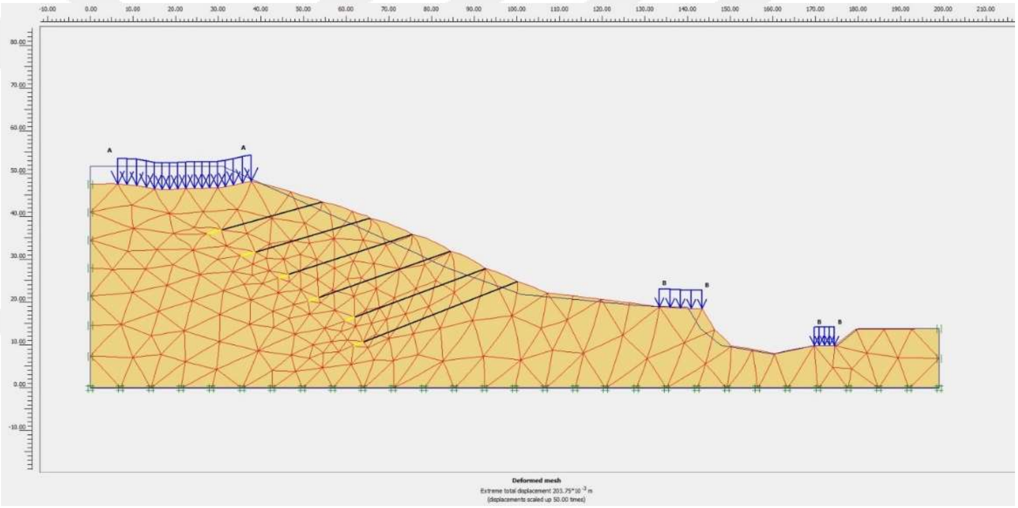

Figure 8.21: Deformed mesh in the presence of Nail in dry condition.

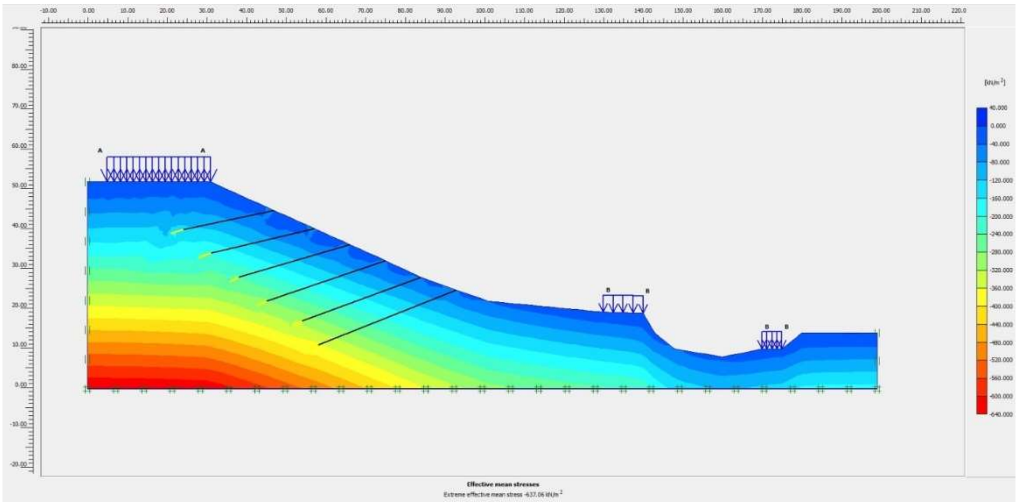

Figure 8.22: Effective stress in the presence of Nail in dry condition. Total effective stress is 637 kN/m^2..

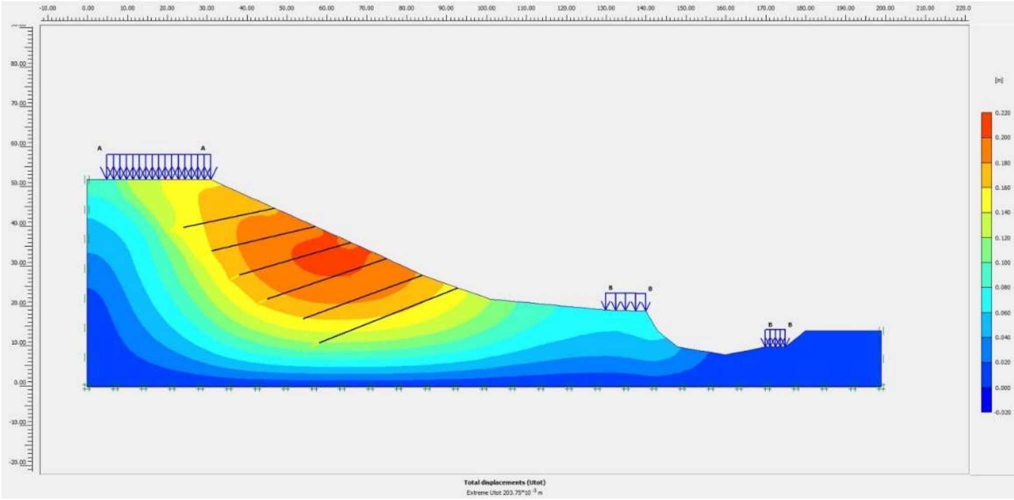

Figure 8.23: Total displacement in the presence of Nail in dry condition. Max. displacement is 0. 203 m

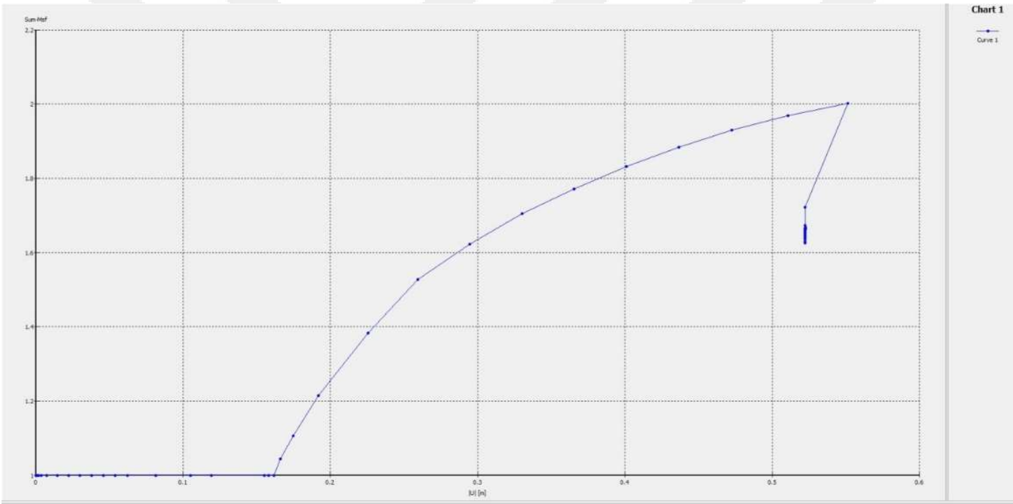

Figure 8.24: Diagram of factor of safety in presence of Nail in dry condition. FOS value is 1.66.

Value of factor of safety in dry condition in presence of Nail increases from 1.1 to 1.66 which is acceptable and In comparison to other structural elements it is the most beneficial one.

8.3.2 Soil nailing in wet condition

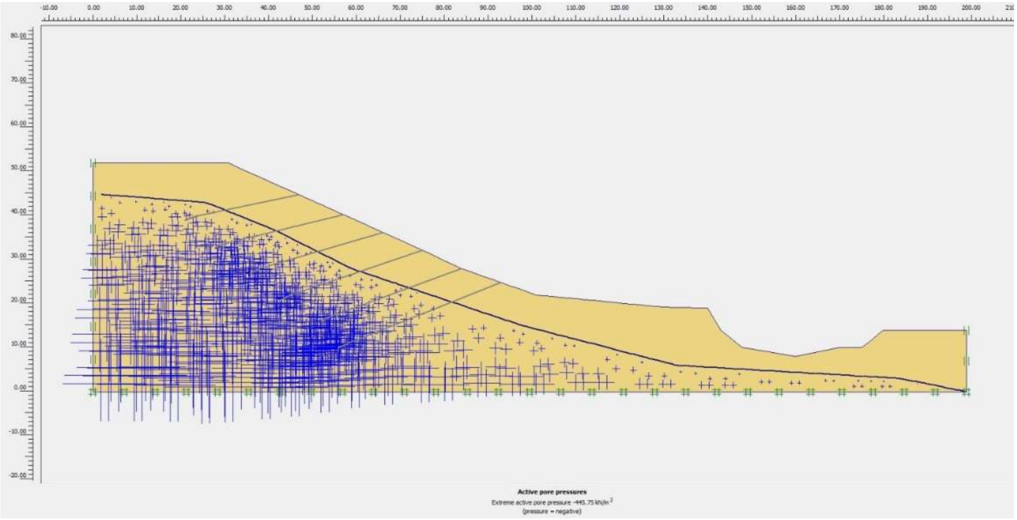

Figure 8.25: Flow field in presence of Nail in wet condition.

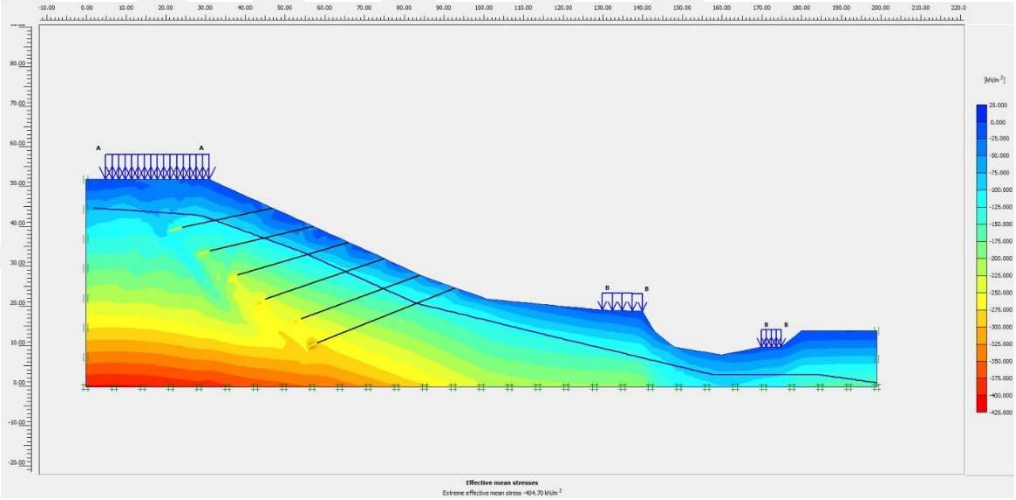

Figure 8.26: Effective stress in the presence of Nail in wet condition. Total effective stress is 404 kN/m^2..

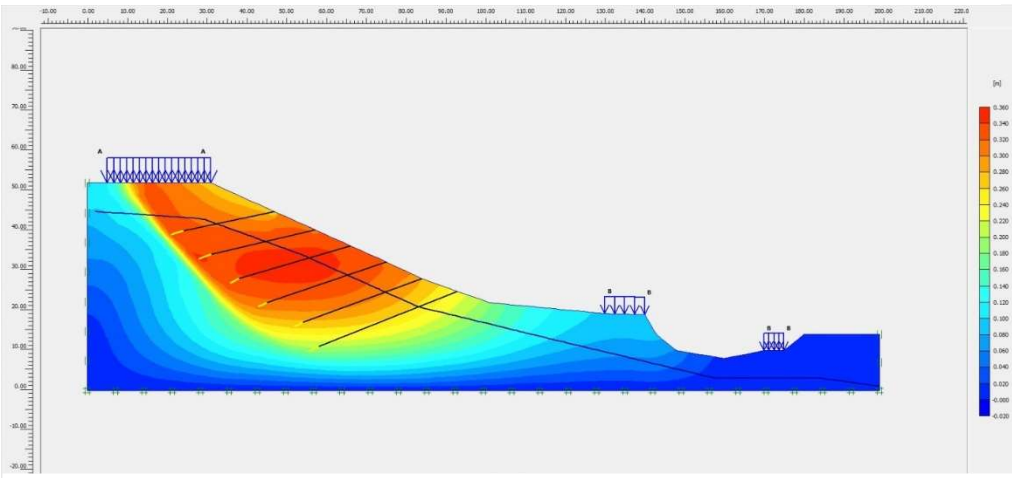

Figure 8.27: Total displacement in the presence of Nail in wet condition. Max. displacement is 0.346 m.

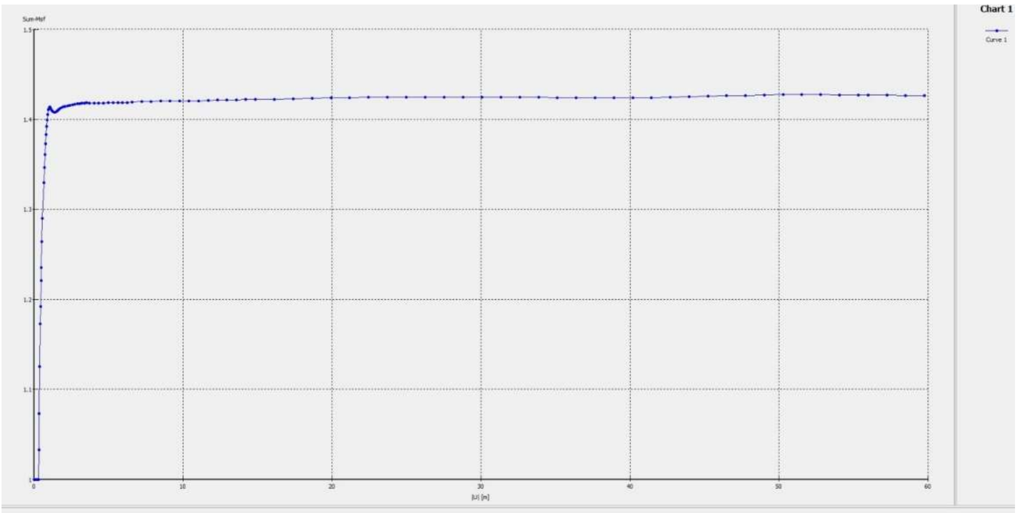

Figure 8.28: Diagram of factor of safety in presence of Nail in wet condition. FOS value is 1.42.

By the usage of nailing in wet condition value of factor of safety increase from 0.9 to the 1.42.

8.4 Comparisons of Outputs

8.4.1 Comparison of factors of safety

The comparation result for different conditions are given in graphs below.

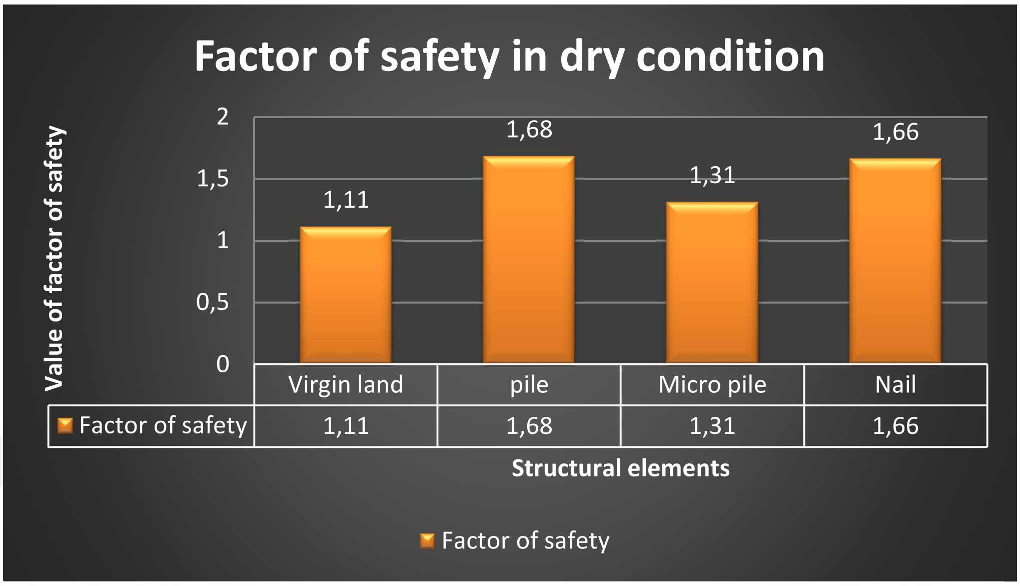

Figure 8.29: Comparison of the factor of safety in different modes of dry condition.

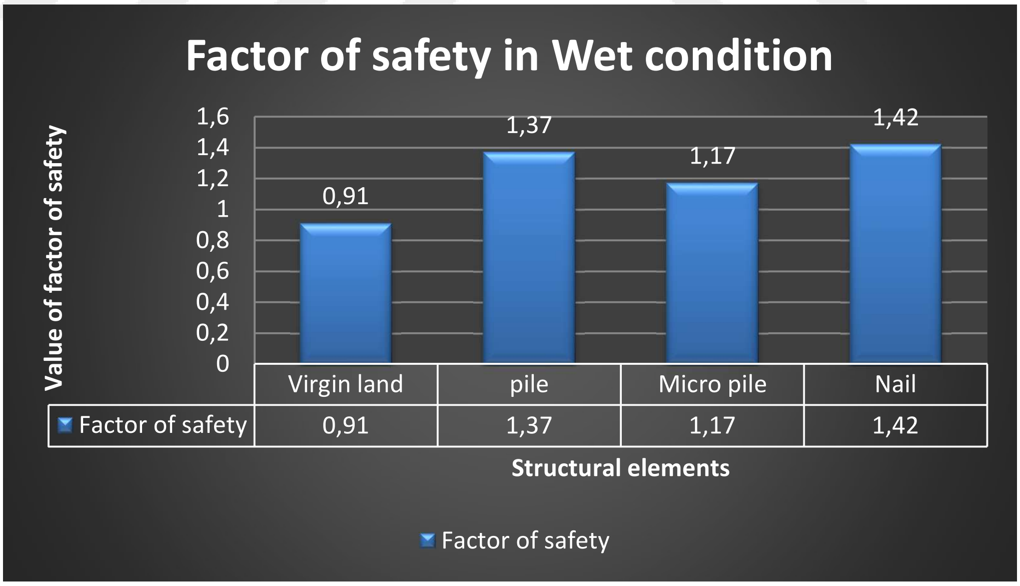

Figure 8.30: Comparison of the factor of safety in different modes of wet condition.

As we can see from charts, FOS value of nailing is reliable than other systems.

8.4.2 Comparison of displacements

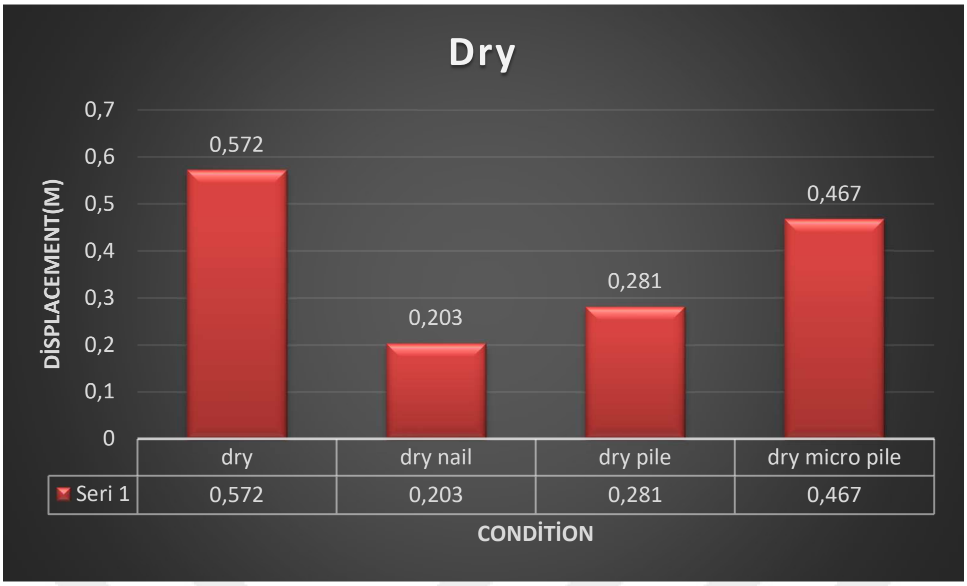

Figure 8.31: Comparison of total displacements in different modes of dry condition.

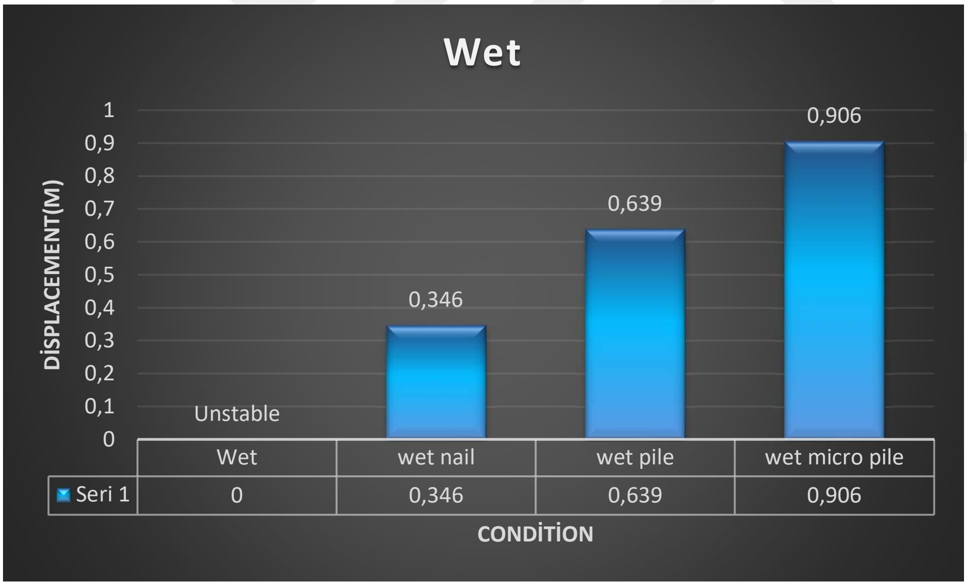

Figure 8.32: Comparison of total displacements in different modes of dry condition.

As it can be seen from charts, the total displacement value within nail system are less than other systems.

8.4.3 Comparison of effective stresses

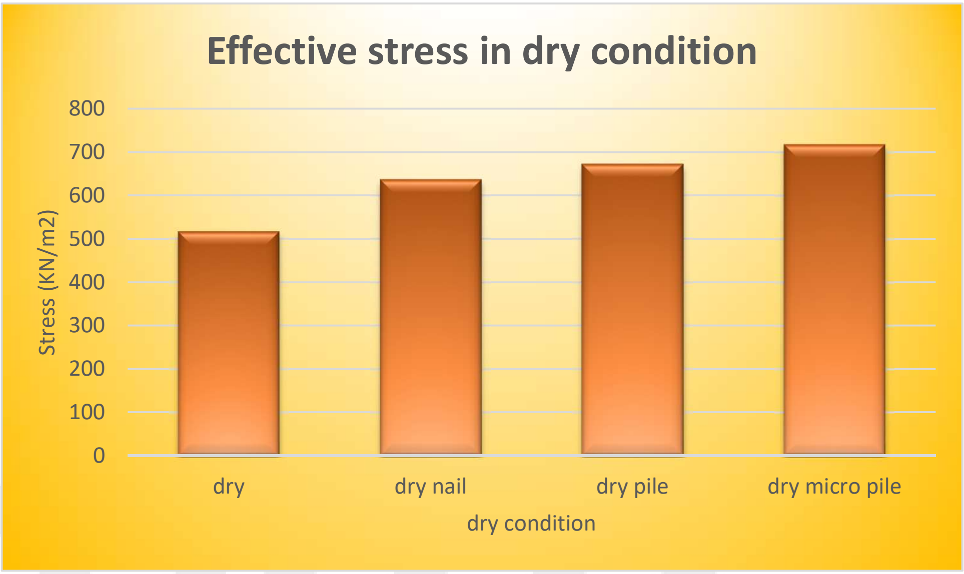

Figure 8.33 Effective stresses in dry condition

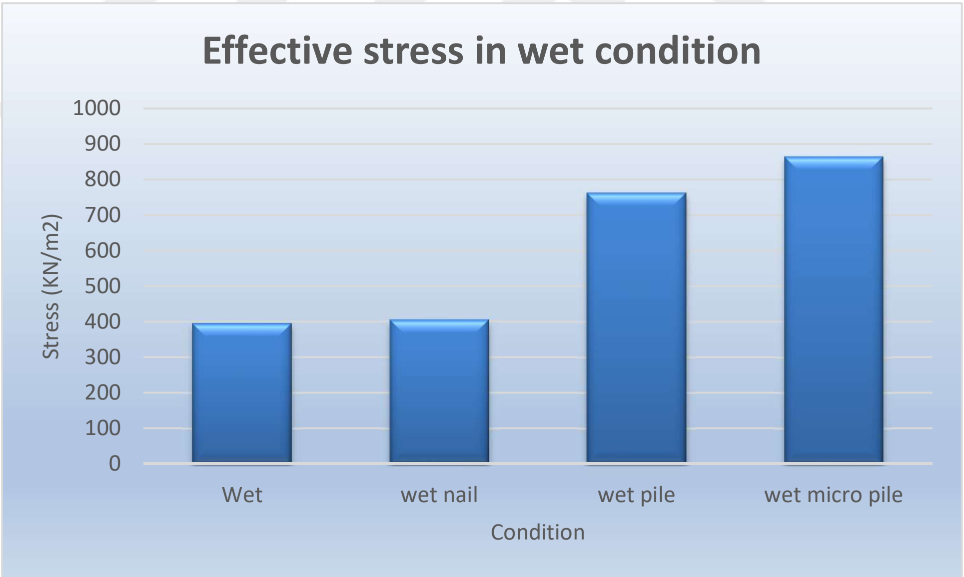

Figure 8.34 Effective stresses in wet condition

According to charts above, effective stresses of nailing system are more reliable than other systems.

9. CONCLUSIONS

9.1 Natural Soil

According to the results obtained from modeling and contour, the following general results were obtained.

1. In the dry state, despite Factor of safety with the value of 1.1, this FOS was below the permissible limit of 1.25 in the regulation FHWA, it could be expected that geotechnical changes in the site would occur, such as underground water rise, excavation and dynamic loads the slope become unbalanced.
2. Regarding the location of the building and the residential area, it is recommended that the soil be stabilized.
3. In the case where the groundwater level has been applied in the model, the Factor of safety has dropped from 1.1 to 0.95 this indicating an imbalance in the slope potential in the presence of water.
4. It is best to use conventional drainage methods to lower the risk and increase the coefficient of confidence, and to maintain the water level.

9.2 Improved Soil

After modeling the slip surface without structural elements and in two states of dryness and existence of groundwater level, we simulated the sliding surface with structural elements. At first, the piles, then the microplate and finally the nailing's were modeled and the results the following is obtained:

1. In all cases, using structural elements, factor of safety in dry state was higher than that associated with groundwater level.

3- In the dry state, the maximum factor of safety is related to the modeling with the Pile.

4- In the presence of underground water level, the highest factor of safety is related to Nailing modeling.

5- Value of factor of safety in 2 modeling with pile and nailing in both dry conditions and the presence of groundwater are close together and there is a significant difference with the modeling with micro-pile.

6- Considering the economics of the nailing model with respect to the pile and the closeness of its factor of safety to modeling with piles, modeling with Nailing seems to be the most appropriate method of improvement.

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