

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

FAILURE OF SLOPES DURING 2010 ELAZIĞ EARTHQUAKE

M.Sc. THESIS

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Department of Civil Engineering

Soil Mechanics and Geotechnical Engineering Programme

Thesis Advisor: Prof. Dr. Ayfer ERKEN

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To my family,

FOREWORD

This master of thesis has been prepared for submission to Istanbul Technical University , Civil Engineering Department, Soil Mechanics and Geotechnical Engineering Programme.

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ABBREVIATIONS

AES	: Average Element Size
CH	: High plasticity Clay
CPT	: Cone Penetration Test
EAF	: East Anatolian Fault
FAS	: Fourier Amplitude Spectra
FEM	: Finite Element Method
FOS	: Factor of Safety
GC	: Clayey Gravel
GM	: Silty Gravel
HS	: Hardening Soil
HS_{small}	: Hardening Soil with small strains
LE	: Linear Elastic
MC	: Mohr Coloumb
ML	: Local Magnitude
MMC	: Modified Cam Clay
MW	: Moment Magnitude
NAF	: North Anatolian Fault
PGA	: Peak Ground Acceleration
RD	: Relative Density
SPT	: Standard Penetration Test
SC	: Clayey Sand
SM	: Silty Sand
SS	: Soft Soil
SSC	: Soft Soil Creep

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FAILURE OF SLOPES DURING 2010 ELAZIĞ EARTHQUAKE

SUMMARY

Recently, the population of the world and also the tendency to live in urban areas are growing in a rapid way so, there is an ever greater need to build more civil structures such as high-rise buildings, bridges, dams, ports, towers, and other facilities. Sometimes these structures should be built near or on slopes for any reason like architecture or need for existence of a particular building on or near slope. One of the most essential components of any structure is the foundation that has the primary role of transferring the loads produced by the weight of structure to the underlying foundation soil. When structures are built on slopes or near them the stability of slope and also stability of the structure itself, need to be considered. The behavior of foundations on slope and also evaluation of the behavior of the super structure are another issues that have to be taken into account. Generally, the aim of the thesis is to evaluate the behavior of slopes and foundations and superstructures on or near slopes during earthquake. Turkey is located in a region which has a great potential of earthquake. There are a lot of slopes in this region which always have the contingency of being instable by effect of ground motion. Elazığ earthquake and the region that was affected by this ground motion have been selected as the case study in this thesis.

The main objective of this thesis is to study 2010 Elazığ earthquake and see how it has affected the slopes at this zone. One of the most noteworthy earthquake incident in Turkey is 1999 Kocaeli earthquake. The ground motion record of this earthquake is used in the analyses to study the effect of a stronger earthquake in the Elazığ region.

The geology of the damaged area has been discussed and the incident of Elazığ earthquake is explained. The models have been created based on the real soil data, available from the affected area. The soil model parameters have been obtained from available soil data and some of them are calculated based on estimations by using correlations proposed by wellknown researchers.

PLAXIS 2012 finite element program and GeoStudio 2012 software have been used for performing dynamic and static analysis.

Several analysis have been performed to evaluate different conditions in the area, like existence of a building on slope or the effect of ground water table in the affected zone.

Then the results have been compared to each other in order to help us have a better understanding of the area.

ELAZIĞ DEPREMİ SIRASINDA ŞEVLERİN GÖÇMESİ

ÖZET

Son zamanlarda dünyanın nüfusun artması ve ayrıca kentsel alanlarda yaşama eğiliminin hızlıca artması nedeniyle daha fazla yapı (yüksek yapılar, köprüler, barajlar, limanlar vs.) inşasına her zamankinden fazla ihtiyaç vardır. Bazen bu yapılar mimari ya diğer sebeplerle şevlere ya da şevlerin yakınına inşa edilebilir. Yapının taşıyıcı sisteminin en önemli parçalarından olan temeller yapının yükünü temelin altındaki zemine aktarır. Yapılar şevlere ya da şevlerin yakınına inşa edildiğinde şevin ve ayrıca yapının stabilitesinin göz önüne alınması gerekir. Şevlerdeki temellerin davranışı ve ayrıca yapının davranışı hesaba katılmalıdır. Yapılan bu tezin amacı deprem sırasında şev üzerinde veya yakınında yer alan temel davranışını ve üzerinde yer alan yapının davranışını değerlendirmektir. Türkiye bulunduğu yer itibari ile büyük deprem potansiyeli olan bir bölgededir ve ayrıca birçok şev deprem nedeniyle dengesiz hale gelebilir. Elazığ şehri ve şehrin yer aldığı bölge Elazığ Depremi sırasında hasar görmüştür ve bu deprem vaka analizi için seçilmiştir.

Şevler ve şev üzerinde yer alan yapıların araştırılması her dönem geoteknik mühendisliğinde ilgi çekici bir konu olmuştur. Son zamanlarda, örneğin binalar gibi birçok yapı, şevlere ve dağlık arazilere inşa edilmektedir. Bu yüzden şev üzerinde yer alan yapıların ayrıntılı bir şekilde incelenmesi ve deprem sırasında nasıl davrandıklarının araştırılması gerekmektedir.

Bu tez kapsamında, Elazığ Depremi incelenmiştir. Çok sayıda toprak kayması ve şevlerde yer alan yapıların göçmesi bu konuyu önemli bir hale getirmektedir. Etkilenen bölgede yer alan şevlerin davranışına depremin etkisini araştırmak için Plaxis 2D sonlu eleman programı kullanılmıştır. Son zamanlarda, sonlu eleman yöntemi şevlerin davranışını incelemek için en etkili ve en güvenilir yöntemlerden biri olmaktadır.

Analizleri gerçekleştirmek için sonlu eleman programından bulunan bünye modelleri incelenmiş ve dinamik analize en uygun olan Hardening Soil with small strains modeli seçilmiştir.

Bünye modeli için gereken uygun zemin parametrelerini belirlemek amacıyla literatür taraması yapılmıştır ve model parametreleri ile parametrelerin elde edilmesinde kullanılan yöntemler incelenmiştir.

Zemine ait model parametreleri depremden etkilenmiş bölgeden alınan numuneler üzerinde yapılan elek analizi deneylerine dayanan yöntemler ile belirlenmiş ve bu yöntemler yapılan bu çalışma kapsamında açıklanmıştır.

İlk olarak, 510 metre genişliğe ve 180 metre yüksekliğe sahip olan bir geometrik model kurulmuştur. Şevin eğimi 25 derece olarak seçilmiş ve bu eğim bölgede yer alan şevlerin eğimini genel olarak temsil ettiği kabul edilmektedir. Depremden etkilenen bölgede yer alan şevlerin eğimi 10 ila 35 derece arasında değişmektedir. Bu yüzden depremden etkilenen bölgedeki zeminin davranışını değerlendirmek için bahsedilen geometriye sahip tipik bir şev seçilmiştir. Depremden etkilenen bölgede killi kum (SC), siltli kum (SM) ve yüksek plastisiteli kil (CH) olmak üzere üç tip zemin bulunmaktadır. Bahsedilen üç tip zemin çeşidinin tümü Elazığ depremi için analiz edilmiştir ve deplasman ile şeve ait güvenlik sayıları hesaplanmıştır ve birbirleriyle kıyaslanmıştır. Diğer analiz serisinde ise bina yüklerini temsil eden yayılı yük altında şevin davranışı incelenmiştir. Son olarak iki farklı tipteki yapının davranışı 1999 Kocaeli Depremi ve 2010 Elazığ Depremi için analiz edilmiştir. 1999 Kocaeli Depremi, Türkiye’de gerçekleşen kuvvetli yer hareketlerinde biridir ve bu tez kapsamında farklı deprem etkisini kıyaslamak amacıyla seçilmiştir. Deprem etkisinin incelendiği analiz serisinde kritik zemin koşullarını temsil etmesi için kumlu zemin ve killi zemin olmak üzere iki tip zemin seçilmiştir. Analizlerde kullanılan zemin tipi, gevşek ve orta sıkı zemin özelliklerine sahiptir. Analiz edilen yapılar 2 katlı ve 5 katlı yapılar olarak modellenmiştir ve bölgede bulunan taşra tipi yapılardan daha dayanıklıdır. Kocaeli ve Elazığ Depremlerine ait iki kayıt modellere uygulanmıştır ve elde edilen sonuçlar birbirleriyle kıyaslanmıştır.

Analiz sonuçlarına göre 15 ila 35 cm arasında deplasman bulunmuştur ve sonuçlar arazide ölçülen gerçek değer olan 20 cm ile kıyaslandığında kabul edilebilir mertebededir. Analiz sonucunda elde edilen deformasyonlar sahada gözlemlenen deformasyonlar ile benzerlik ve uyum göstermektedir. Şevin güvenlik sayısı 1.1 olarak hesaplanmıştır ve şevin deprem sırasında göçmelere karşı güvensiz olabileceği görülmüştür.

Alınan sonuçların doğruluğunu kontrol etmek için Geostudio bilgisayar programı kullanılmış ve analizler tekrarlanmıştır. Alınan sonuçlar diğer sonuçlarla uyumlu olup ve modellemenin doğru olduğu tespit edilmiştir.

Şevin farklı noktasına uygulanan bina yükü altındaki davranışının incelendiği analiz sonunda şevde meydana gelebilecek kaymaların temeli göçme tehlikesine açık hale getirdiği olduğu görülmüştür. Killi tabakada meydana gelen deformasyonlar ve yer değiştirmeler daha fazla miktardadır ve bu bölgeye yapılacak binalar inşa edilmeden daha detaylı incelemelerin yapılması gerekmektedir.

Şev üzerinde yer alan yapı davranışının incelendiği analiz serisinde ise yapının şev üzerindeki yerinin ve yapının temeli altındaki tabakaların önemi görülmüştür. Şevin eğimi ile yapılarda meydana gelen deformasyon doğrudan ilişkilidir. Elazığ kaydının uygulandığı analizlerde düşey deplasman 30 derecelik şev açısı için 1 ila 10 cm arasında değişirken 15 derecelik şev açısında ise 1 cm’ye kadar düşmektedir. Depremden etkilenen bölgede bulunan yapı modellenen yapıya göre daha zayıftır ve bu bölgede yer alan kırsal tek katlı yapılarda meydana gelecek deformasyonun miktar olarak daha fazla olabileceği göz önüne alınmalıdır.

Aynı analizler 1999 Kocaeli Depremi için yapıldığında ise daha fazla deformasyon meydana gelmiştir ve killi tabakalarda 2 metreye varan deformasyonlar bulunmuştur. Bu sonuçlar bölgenin yer alan zemin tabakaları nedeniyle tehlikeye açık olduğunu

göstermektedir. Bu sebeplerden ötürü bu bölgeye yapı malzemesine ve temel zeminine de bağlı olarak 2 ya da 3 katlı yapıların yapılması uygundur. Ağır yapı yükü verecek olan yapılar inşa edilmemelidir. Şevin göçme riskini azaltmak için şeve bina inşasından kaçınılmalıdır.

Kocaeli kaydı için yapılan analizler sonucunda deprem sırasında tabakalarda oluşan aşırı boşluk suyu basınçlarının Elazığ vakası için çok önemli bir etkiye sahip olmadığı görülmüştür. Yapıların şev üzerinde bulunduğu noktanın önemi bu analizlerde anlaşılmıştır. Ayrıca şev göçmesine yatkın olan bölgeler, ağır binalar inşa edilmeden detay önce detaylı bir biçimde incelenmelidir.

1. INTRODUCTION

Recently, the population of the world and also the tendency to live in urban areas are growing in a rapid way so, there is an ever greater need to build more civil structures such as high-rise buildings, bridges, dams, ports, towers, and other facilities. Sometimes these structures should be built near or on slopes for any reason like architecture or other reasons. One of the most essential components of any structure is the foundation that has the primary role of transferring the loads produced by the weight of structure to the underlying foundation soil. When structures are built on slopes or near them the stability of slope and also stability of the structure itself, need to be considered. The behavior of foundations on slope and also the super structure are another issues that have to be taken into account. Generally, the aim of the study is to evaluate the behavior of foundation and superstructure on or near slopes during earthquake .Turkey is located in a region which has a great potential of earthquake and also lots of slopes which would be instable by earthquakes. The Elazığ earthquake and the region that was affected by this earthquake has been selected as case study.

1.1 Problem Statement

Sometimes we build structures on slopes or near them instead of building them on level ground due to various reasons. For example land limitation issues, such as in retaining walls and bridges abutments, to avoid floods or architectural purposes. One of the issues that is extremely affected by existing of slopes is the bearing capacity of the soil under the foundation of buildings. It becomes more important when we are facing the situation which earthquakes happen. Design of foundation under these conditions is complex, and different parameters are involved. In Turkey also, a lot of structures have been built or being built on slopes or near them and seismic stability of slopes is an important problem in this region. Evaluating the behavior of foundations, buildings and also the stability of them are very important. Having considered all mentioned above, it is very important to study the behavior of

foundations on slopes or near them, during earthquakes. In this thesis the case study of Elazığ earthquake has been chosen as the main subject of study. The Kocaeli earthquake also is another case and it will be discussed.

1.2 Purpose of Thesis

The main objective of this study is to evaluate the behavior of foundations and buildings on or near slopes during Elazığ earthquake as a case study for Turkey. Studying this earthquake and landslides caused due to ground motion and the consequences of this earthquake which were failure of slopes and destruction of buildings located on them, are another purposes of the study. Comparison with another earthquakes like Kocaeli earthquake is another aim of the thesis. Comparison between different types of buildings and proposing the appropriate type of them, is another purpose of the study.

1.3 Scope of Thesis

This thesis includes six chapters. In the first chapter definition of problem and the purpose of thesis are given. In the second chapter, an overview of landslide and slope stability is given and different aspects of it are discussed. In the third chapter, the Elazığ earthquake is explained and ground motion characteristics and geology of affected area are discussed. In the fourth chapter, the finite element method and the methods for derivation of parameters are explained. In the fifth chapter, results of analysis are explained and discussed. Finally, in the sixth chapter conclusions and recommendations are presented.

2. OVERVIEW OF SLOPE STABILITY AND LANDSLIDE

2.1 Introduction

In this chapter an overview of slope stability and landslides is given. Different types of landslides and causes of them are discussed. The definition of slope movement and physical aspects of landslides are presented. Different types of landslide movements and methods of stabilization and the main causes of landslides are discussed. Types of failure surfaces and methods of stability analysis are briefly discussed. The action of water in soil layers is another issue that is presented in this chapter.

2.2 Definition of Landslide

A landslide can be defined as the movement of rock, detritus, or soils caused by the action of gravity. To distinguish landslides from other forms of gravity mass flows, we require in the definition that the bulk of the moving material should have density at least 10% greater than the density of water. Most landslides are very small. Every year mountain roads need removal of blocks fallen from the flanks (De Blasio, 2011). Larger collapses may affect local watercourses and influence the activity of local communities; greater slides may provoke disasters and change the geomorphologic setting of several square kilometers of land. Some landslides evolve very slowly, and special instruments may be necessary to become aware that they are in fact moving. Others may travel faster than 100 km/h. And some start with a creeping, imperceptible movement, to suddenly accelerate and degenerate in a catastrophic debris avalanche. Some landslides travel similar to a fluid, resembling the flow of water. Others are akin to granular flows. Many landslides come to a halt without affecting vast areas beyond the immediate surroundings; others plunge into the sea and cause damage hundreds of kilometers away (De Blasio, 2011).

A landslide is a geological phenomenon which includes a wide range of ground movement, such as rock falls, deep failure of slopes and shallow debris flows, which

can occur in offshore, coastal and onshore environments. Although the action of gravity is the primary driving force for a landslide to occur, there are other contributing factors affecting the original slope stability. Typically, pre conditional factors build up specific sub surface conditions that make the area/slope prone to failure, whereas the actual landslide often requires a trigger before being released. Landslides, especially in soils, are gravitational downward and outward movements of soil masses that may be set off by the liquefying effect of earthquakes. Typically, soils containing minerals with swelling capacity, such as clay, clayey silt, and marl, are frequently fissured and the action of pore-water pressure is especially harmful to stability. Landslides develop in unstratified, homogeneous soils or are induced by certain types of stratification, e.g., interposed layers of sand or clay. Other causes are excessive load pressures at the head of a slope or undercuttings at the foot and also earthquakes which has an important role in slope instability and causes significant landslides. In some cases the soil structure is gradually weakened by chemical or physical processes. As far as landslides are concerned, Japan represents an extreme case. In view of its 7,000 landslides, landslide stabilization has become a national concern. Other countries may be faced by occasional occurrences, a slide here and a slide there that usually can be handled by local authorities, but Japan must meet this menace by all available means unless the country wants to resign itself to severe damages occurring with increasing frequency. It is well known that only about 20% of Japan can be inhabited and cultivated; the remaining 80% are densely forested mountains. These figures must be seen in the context of a population of about 115 million. Understandably, every single square meter of soil must be used for the traditional cultivation of rice. But the areas that are especially fertile owing to ample presence of water, are the same ones where water, which is such a blessing for rice crops, has a disastrous effect upon soil stability, generating landslides by the well-known mechanisms. The Japanese tried in vain to cope with the landslides by primitive methods, such as surface drainage, but lately the situation has become even more critical. Water storage reservoirs for agricultural purposes and power plants, construction of motorways, and irrigation and drainage of large areas by modern methods may cause water movements that trigger slides in areas where none occurred before (Veder, 1981).

2.3 Slope Movement

The stability analysis of slopes is one of the most important issues in civil engineering problems. Stability analysis is used in the construction of different structures such as highways, railroads, towers, and other civil structures. There are other kinds of problems that the stability analysis must be taken into account such as surface mining, refuse disposal, and earth dams and also many other human activities involving building construction and excavations. Failures of slopes due to human activities are caused mostly by movements within the fills which are created by mankind, in the natural slope, or a combination of both (Huang, 1983). These movement phenomena are usually studied from two different aspects. Geologists and engineers have different ideas about the way that we study the slopes. The geologists consider the moving phenomena as a natural process and study the cause of their origin, their courses, and the resulting surface forms. On the other hand, engineers mostly try to evaluate the safety of construction based on the principles of soil mechanics and develop methods for a reliable assessment of the stability of slopes, as well as the controlling and corrective measures needed (Huang, 1983). The evaluation of the stability of slopes must be done having a vast field of information about the geological structure of the area, the detailed composition and orientation of strata, and the geomorphological history of the land surface if we want to solve this stability issue using the methods of soil mechanics (Huang, 1983). Geologists can help the engineers by helping engineers obtain a more vivid picture of the real conditions and character of movement process by checking comparing their own information with the results of the analyses that are done based on soil mechanics by engineers. The most useful results of stability studies can be obtained if both of the mentioned aspects are considered while tackling stability problems. There are lots of things that have to be taken in mind for classification of slope failures. For example, they can be categorized according to the types of failures, the sort of materials that the slope is made of, the age of the slope prone region, or the stage of development. A special report published by the Transportation Research Board (Schuster and Krizek, 1978) is one of the most reliable references on landslides or slope failures. According to this report, the form of slope movements is divided into five main groups: falls, topples, slides, spreads, and flows. A sixth group is also defined as complex slope movements, which includes combination of two or more of the above

five types. The kind of materials is divided into two classes: rock and soil. Soil is further divided into debris and earth. Table 2.1 shows the classification of slope movements. Recognizing the form of slope movements is important because they dictate the method of stability analysis and the remedial measures to be employed. They are described as follow:

- In falls, a mass of any size is detached from a steep slope or cliff, along a surface on which little or no shear displacement takes place, and descends mostly through the air by free fall, leaping, bounding, or rolling. Movements are very rapid and may or may not be preceded by minor movements leading to progressive separation of the mass from its source (Huang, 1983).
- In topples, one or more units of mass rotate forward about some pivot point, below or low in the unit, under the action of gravity and forces exerted by adjacent unit or by fluids in cracks. In fact, it is tilting without collapse (Huang, 1983).
- In slides, the movement consists of shear strain and displacement along one or several surfaces that are visible or may reasonably be inferred, or within a relatively narrow zone. The movement may be progressive; that is, shear failure may not initially occur simultaneously over what eventually becomes a defined surface of rupture, but rather it may propagate from an area of local failure. This displaced mass may slide beyond the original surface of rupture onto what had been the original ground surface, which then becomes a surface of separation. Slides are divided into rotational slides and translational slides. This distinction is important because it affects the methods of analysis and control (Huang, 1983).
- In spreads, the dominant mode of movement is lateral extension accommodated by shear or tensile fractures. Movements may involve fracturing and extension of coherent material, either bedrock or soil, owing to liquefaction or plastic flow of subjacent material. The coherent upper units may subside, translate, rotate, or disintegrate, or they may liquefy and flow. The mechanism of failure can involve elements not only of rotation and translation but also of flows; hence some lateral spreading failures may be

regarded as complex. The sudden spreading of clay slopes was discussed by Terzaghi and Peck (1967), (Huang, 1983).

Table 2.1 : Classification of slope movements (Varnes, 1987).

Type of Movement			TYPE OF MATERIAL		
			Bedrock	Engineering Soils	
				Predominantly Coarse	Predominantly Fine
Falls			Rock fall	Debris fall	Earth Fall
Topples			Rock topple	Debris topple	Earth topple
Slides	Rotational	Few units	Rock slump	Debris slump	Earth slump
	Translational		Rock block slide	Debris block slide	Earth block slide
			Many units	Rock slide	Debris slide
	Lateral spreads			Rock spread	Debris spread
Flows			Rock flow (deep creep)	Debris flow (soil creep)	Earth flow (soil creep)
Complex			combination of two or more principal types of movement		

Many examples of slope movement cannot be classed as falls, topples, slides, or spreads. In unconsolidated materials, these generally take the form of fairly obvious flows, either fast or slow, wet or dry. In bedrock, the movements are extremely slow and distributed among many closely spaced, noninterconnected fractures that result in folding, bending, or bulging. According to age, slope movements are divided into contemporary, dormant, and fossil movements. Contemporary movements are generally active and relatively easily recognizable by their configuration, because the surface forms produced by the mass movements are expressive and not affected by rain wash and erosion. Dormant movements are usually covered by vegetation or disturbed by erosion so that the traces of their last movements are not easily discernible. However, the causes of their origin still remain and the movement may be renewed. Fossil movements generally developed in the Pleistocene or earlier periods, under different morphological and climatic conditions, and cannot repeat themselves at present (Huang, 1983).

Slope movements can be divided into initial, advanced, and exhausted movements, according to their stage of movement. At the initial stage, the first signs of the disturbance of equilibrium appear and cracks in the upper part of the slope develop. In the advanced stage, the loosened mass is propelled into motion and slides

downslope. In the exhausted stage, the accumulation of slide mass creates temporary equilibrium conditions. Chowdhury (1980) classified slides according to their causes:

(1) landslides arising from exceptional causes such as earthquake, exceptional precipitation, severe flooding, accelerated erosion from wave action, and liquefaction; (2) ordinary landslides, or landslides resulting from known or usual causes which can be explained by traditional theories; and (3) landslides which occur without any apparent cause (Huang, 1983).

2.4 Landslides as a Geological Hazard

The most important reason that makes studying landslides so crucial, is the prevention and reduction of disasters and risks that would be caused by a landslide. Most landslides happen in a small range and do not have a considerable damage. However, large landslides may be extremely calamitous. Figure 2.1 shows some of the consequences of earthquake. In 1786, a strong earthquake in the province of Sichuan in China released a landslide that dammed a local river for 10 days. As many as 100,000 people were drowned when the dam failed inundating an area 1,400 km downstream. The Vaiont tragedy (Northern Italy, 1963) was the result of water spilling over a dam, thrust by the body of a large landslide that invaded the artificial reservoir. Submarine landslides may generate devastating tsunamis. Accounting for both the small but frequent and for the large and rarer catastrophic landslides, it has been calculated that during one average year, landslides kill about 5–7 people in Norway, 18 in Italy, 25–50 in USA, 186 in Nepal, 170 in Japan, and 140–150 in China (Sidle and Ochiai, 2006). Fortunately, this is not an extremely heavy death toll compared to other natural disasters or car accidents. However, the negative consequences of landslides are not limited to loss of life, but include the destruction of houses and infrastructures loss of productivity in the area affected, unpredictable changes in the local watercourse, and reduction of arable or habitable land. Estimated costs of landslide damage (including both the direct costs caused by destruction, and indirect costs due to long- time effects in the local economy) are about 4 billion USD in Japan, 70 million USD in Canada, 2.6–5 million USD in Italy (Sidle and Ochiai, 2006).

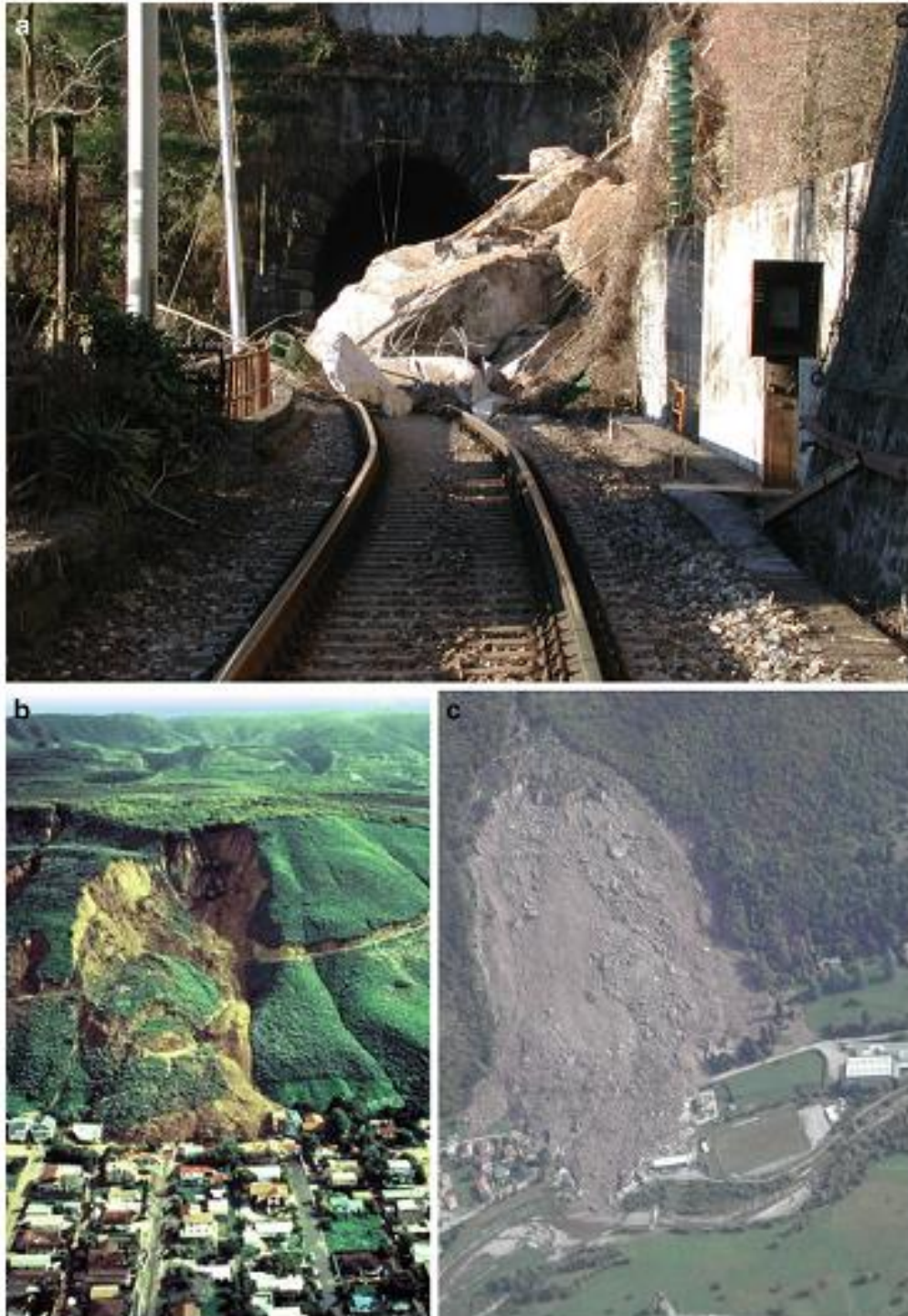


Figure 2.1 : Top: In addition to the death toll, landslides often interrupt railways and roads and devastate infrastructures. (a) A boulder has interrupted the railway in Maccagno (Varese, Italy). It is part of a larger landslide occurred in 2004 in Varenna (Northern Italy). (b) The village of La Conchita in California is frequently affected by killing landslides. (Photograph USGS of public domain.) Bottom (c) A small landslide in Cortenova (northern Italy). (Photograph courtesy of G. B. Crosta.)

2.5 Physical Aspects of Landslides

Numerous researchers have tried to treat the dynamics of landslides using the methods of mechanics and physics. Landslides are in fact a complex phenomenon, and for understanding their behavior we need to understand a few basic laws of mechanics. Their complex and unpredictable behavior is mostly because of the lack of knowledge on the physical conditions and materials characteristics, rather than to a lack of knowledge for the basic laws. The motion of landslides can be described in terms of laws of friction, cohesion, and gravity (Fig. 2.2a). Below are some of the reasons that make the physical description of landslides favorable (De Blasio, 2011).

- The need to quantify the observable facts typical of exact sciences urges the geologist to go beyond the mere description of the phenomenon. It compels to think in terms of physical laws, broadening the perspective of the geological phenomenon. If on the one hand this is not always possible with geological problems, on the other hand the process of thinking quantitatively is often rewarding in terms of gaining a much deeper insight into observable facts (De Blasio, 2011).
- Secondly, as people continue to move to hazardous zones, the practical need to predict future run-out, damage, energy, and path of landslides or other gravity mass flows will increase. Good predictive models may result in saving human life and properties. The writing of the equations of motion of a landslide is a prerequisite to any computer modeling (De Blasio, 2011).
- Part of the physics and mathematics needed to describe landslides belong to the standard curriculum of a physicist and engineer: fluid mechanics, elasticity theory. Others, like the mechanics of granular media and rheology, are often skipped in regular courses (Fig. 2.2b). In any case, the knowledge acquired by studying the physical aspects of gravity mass flows can be applied to other areas of physics, applied mathematics, and earth sciences. The study of landslides thus gives the possibility to tackle interesting problems in other disciplines (De Blasio, 2011).

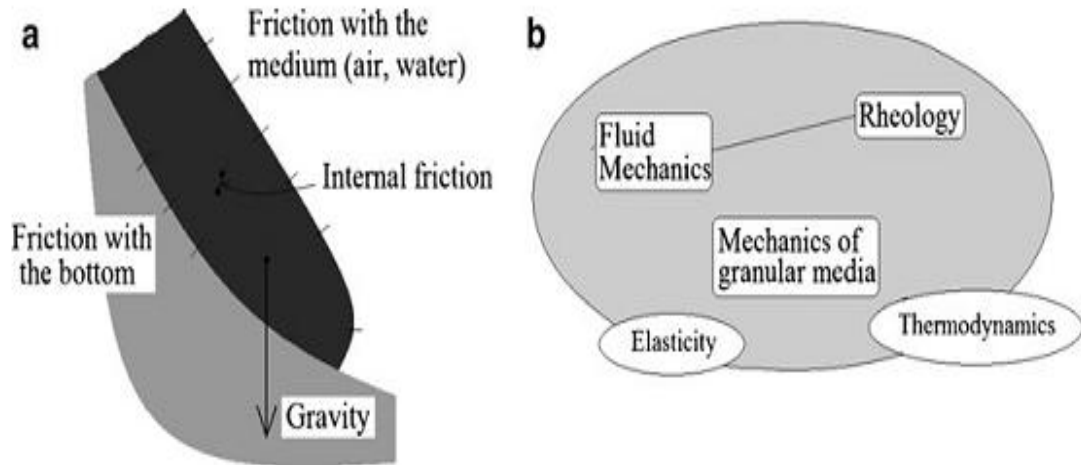


Figure 2.2 : (a) Very simplified scheme of the forces acting on a moving landslide. (b) Physics and mechanics needed for a physical approach to landslides (De Blasio, 2011).

Figure 2.3 indicates an idealized block diagram of a landslide. The failed material starts from a zone of depletion and deposits in the accumulation zone. The crown of the landslide identifies the region adjacent to the highest parts of the failed mass. The scarp is the steep rupture surface between the failed body and the terrain. Several minor scarps due to internal shearing may also punctuate the main landslide body. The surface of rupture identifies the interface at the base of the landslide where the material has slid. In this example the surface appears curved while in other cases it may be planar or complex. The foot is the material deposited in the accumulation zone, beyond the surface of rupture. The landslide deposit ends with a toe, which is the line (usually bent) between the accumulated material and the untouched terrain. The tip is the point of the land slide deposit farthest from the crown. Some landslides exhibit transverse tension cracks in the region of the foot closest to the scarp. Closer to the toe, transverse ridges are sometimes formed. The right and left flanks of the landslide are identified by standing with shoulders to the crown (De Blasio, 2011).

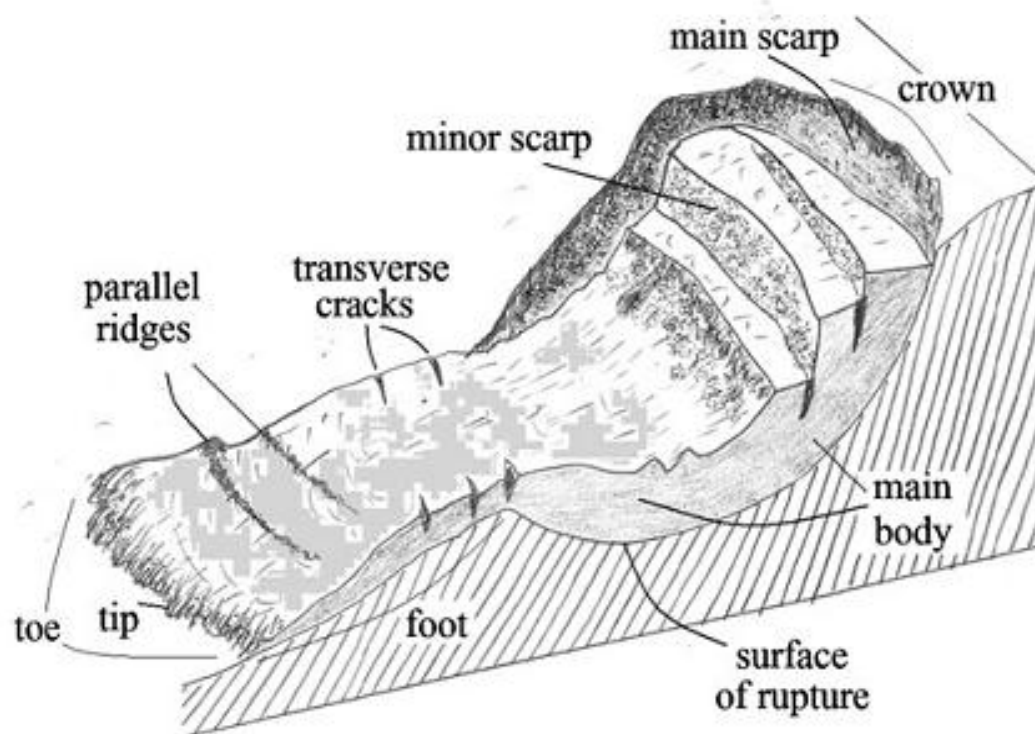


Figure 2.3 : Main geometrical elements of a landslide (Varnes, 1978).

2.6 Description of the Seven Types of Movements

There are different systems of landslide classification that have been proposed by numerous researchers, such as the system by Varnes (1978) and Hutchinson (1988). Various authors have contributed to the EPOCH classification system, which stems from the Hutchinson system (1998). In the EPOCH system, there are seven classes and three material types, for a total of 21 possibilities. These are listed in Table 2.2 (De Blasio, 2011).

Table 2.2 : Classification of landslides adopted from the EPOCH system (Dikau et al, 1996).

Movement	Material		
	Rock	Debris	Soil or earth
Fall	Rockfall	Debris fall	Soil fall
Topple	Rock topple	Debris topple	Soil topple
Translational slide	Rock slide	Debris slide	Soil slide
Rotational slide or "slump"	Rock slump	Debris slump	Soil slump
Lateral spreading	Lateral rock spreading	Lateral debris spreading	Lateral soil spreading
Flow	Rock flow or Sackung	Debris flow	Soil flow
Complex			

In boldface the most common types

2.6.1 Fall

A fall is the movement of material from a stiff headwall or cliff. It generally involves limited volumes of material, most usually rock. The material falls en Fig. 2.2 moving freely in the gravity field. The contact with the terrain occurs especially in the last part of the path, where the material becomes frequently shattered (De Blasio, 2011), (Fig 2.4a).

2.6.2 Topple

A topple is the rotation of a vertical slab about a pivoting point located at the base (Fig. 2.4b). Topple is typical of compact vertical slabs (usually but not exclusively rock) lying on soft, unconsolidated terrain. The movement may be extremely slow for long periods, culminating with a catastrophic fall of the slab (De Blasio, 2011).

2.6.3 Translational slide

A slide is defined as the movement of material along a shear surface. For a translational slide, this surface is planar (Fig. 2.4c). The identity of the shear surface is somehow preserved and distinguishes a slide from a flow (De Blasio, 2011).

2.6.4 Rotational slide

In a rotational slide, the detachment surface is roughly circular, spoon-like. In contrast with translational slides, where the planar surface often originates from a weakness zone, the circular shape of a rotational slide is created by the failure itself and derives from the geometrical distribution of the shear stress (De Blasio, 2011), (Fig. 2.4d).

2.6.5 Flow

According to Dikau et al. (1996), a flow is a landslide in which the individual particles travel separately within a moving mass. They involve whatever material is available to them and may therefore be highly fractured rock, clastic debris in a fine matrix or a simple, usually fine, grain size. Flow in its physical sense is defined as the continuous, irreversible deformation of a material that occurs in response to applied stress. A flow is thus characterized by a fluid-like movement, in which the information on the detachment surface has been lost. A slide may evolve into a flow

if the energy and/or the run-out are sufficient to rework completely the material (Fig. 2.4e).

2.6.6 Lateral spreading

It consists of a lateral movement of rock or soil, often of large extension (Fig. 2.4f). In the case of rock spreading, the rate is often slow (from one tenth of mm to 10 cm per year) and is generally caused by deep-seated viscoplastic material underneath the rocky slabs. Soil spreads, like the ones involving quick clays, can move extremely fast, with speed in the range of several meters per second (De Blasio, 2011).

2.6.7 Complex

It is a generic name used when a landslide changes behavior during the movement. In addition to the rock avalanche (rock slab turned into a granular flow) the EPOCH system considers the flow slide as a member of this class (Dikau et al. 1996). A flow slide consists of a portion of soil losing cohesion during the flow, to the point of becoming a completely fluidized mass (De Blasio, 2011).

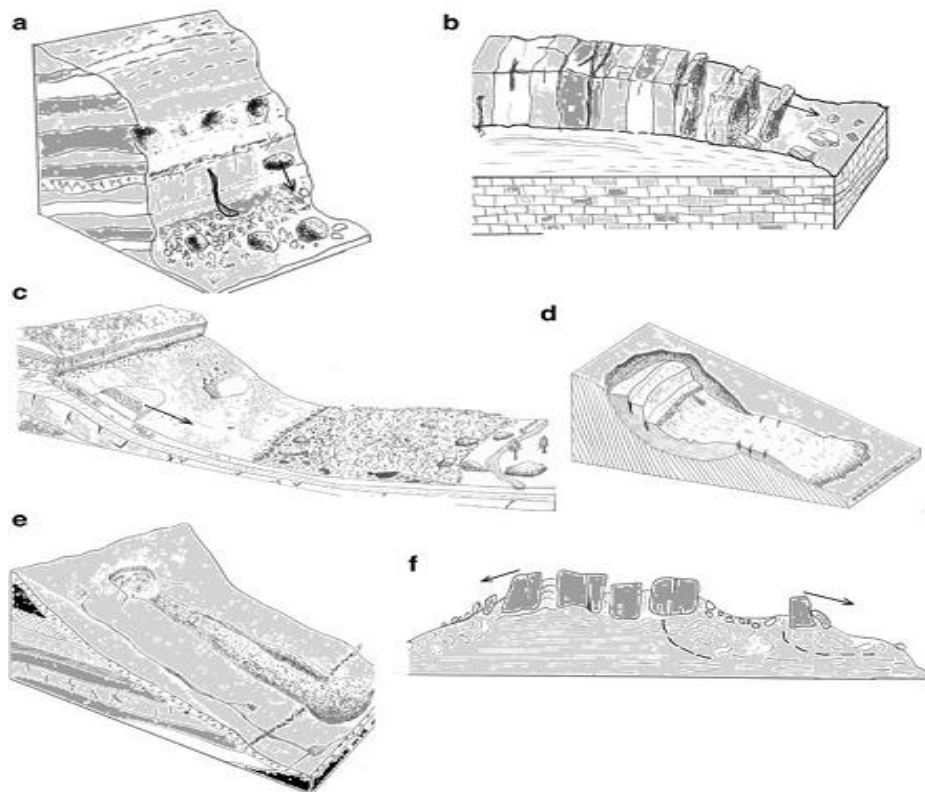


Figure 2.4 : Six landslide types: (a) fall, (b) topple, (c) translational slide, (d) rotational slide, (e) flow, and (f) lateral spreading (Some drawings inspired from Dikau et al. 1996).

2.7 A Physical Classification of Gravity Mass Flows

2.7.1 Rheological flows (Cohesive)

A mudflow is a kind of landslide where large amount of clay-rich soil mix with water and runs downslope, often canalized along river beds. Such kind of flow represents the epitome of a rheological flow. A rheological flow is ideally portrayed as a fluid with complex properties. Its physical description will be based on fluid mechanics, with rheological properties specified in terms of a nonNewtonian fluid. The basic building blocks of a rheological flow are the fine particles (especially clay or silt) that when mixed with water confer a cohesive behavior to the flow. Rheological flows in nature will normally include sand, gravel, and boulders, which can possibly give a frictional component to the rheological behavior. Depending on composition, rheological flows may be called debris flows (if the composition has certain variability in clast size), mudflows (if mostly composed by fines as in the previous example), or hyper concentrated flows (if the water is present in higher degree, resulting in a Newtonian fluid behavior like in a flooding river). Rheological flows affect particular sedimentary environment such as moraines, loess, pyroclastic deposits, river banks, or subaqueous environments dominated by clastic deposition. Water greatly affects the mechanical behavior of debris flows, altering the rheology and diminishing the effective shear strength and viscosity (Fig. 2.5), (De Blasio, 2011).



Figure 2.5 : An example of the product of a rheological flow: lahar deposits of the Cotopaxi volcano, Ecuador (Photograph courtesy of Roberto Scandone and Lisetta Giacomelli).

2.7.2 Granular flows (Frictional)

Granular flows are rapid movement of granular material where friction, described by the classical laws of Amontons-Coulomb, plays an important role in the dynamics (Fig. 2.6). A small-scale analog of a granular flow is the superficial movement of sand inside a jar gently tilted around the axis. Sand avalanches on a large scale, however, are rare. In nature, large-scale granular flows normally develop from the collapse of an initially unbroken rock slab. High-energy impact, internal deformation and crushing determine a rapid disintegration of the rocky mass (De Blasio, 2011).



Figure 2.6 : The hummocky deposit of a small rock avalanche in the Elbortz Mountains, Iran (De Blasio, 2011).

2.7.3 Rock falls and topples

A rock fall is characterized by the fall of blocks in isolation in the gravity field, in which the phase of free fall represents a significant part of the path (Fig. 2.7). The rock fall is essentially described by a rigid body motion, but the impact with the terrain may be a much complicated problem. The block may bounce against hard rock, and then collide and roll on soft ground. The soil deformation involves complex interactions between a cohesive and the frictional component of the soil. Usually the process is described in terms of empirical coefficients of restitution and rolling friction, but this level of description neglects the complexity of block-terrain

interaction. However, the rigid body dynamics coupled with coefficients of restitution to describe the impact is in a first approximation considered satisfactory to describe the process and to provide valuable information for mitigation measures in rock fall prone areas. In this class we include topple failures, where sub-vertical slabs revolve at high speed around a pivoting point at the base, much like a book resting on an edge and pushed at the top. Upon impact with the ground, toppling slabs may partly disintegrate, without much horizontal movement (De Blasio, 2011).



Figure 2.7 : An example of rock fall. The large bluish boulder in the torrent is a porphyrite block fallen from the side of a steep gorge. Val di Scalve (Italy), (De Blasio, 2011).

2.7.4 Slow landslides and creep

Certain slides move very slowly and often their presence is revealed only by clefts and fallen blocks. The behavior of slow landslides is often poorly known. Slow slides may respond to the presence of clay layers underneath the rock formation, while others may be associated to fractures along a weak plane. They are often affected by karst phenomena or freeze thaw activity.

Fig. 2.8 shows a broader class of gravity mass flows. For brevity, not all kinds of gravity mass flows are cited (De Blasio, 2011).

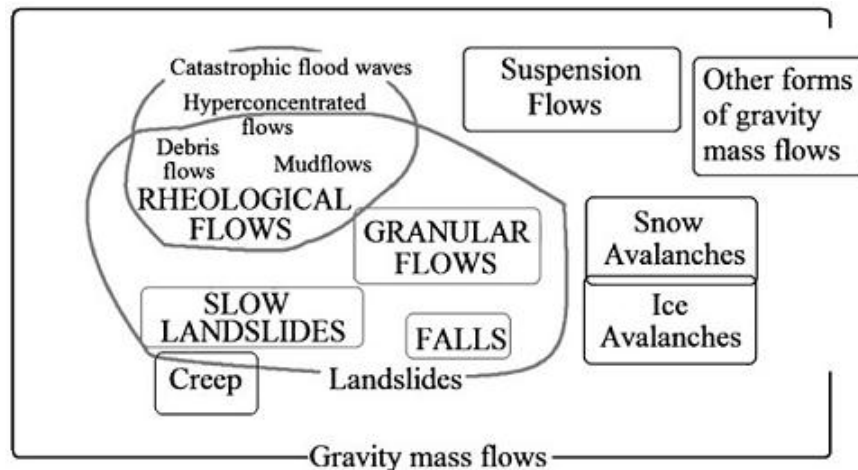


Figure 2.8 : Landslides as members of the broader class of gravity mass flows (De Blasio, 2011).

2.8 Characteristic Types of Slides and Alternatives for Their Stabilization

Geotechnical engineers know that soil which is a two-phase or three-phase system, is a complex material and stability of soil is related to the solid components, water or air. An experienced engineer, will first investigate the effect of water when confronted with a landslide, which may be present as pore water or ground water, or be mixed mechanically with the soil particles, causing incipient or already-progressing soil movement. Soils are very rarely truly homogeneous or, to use another word, "unstratified." Also, chemically homogeneous soils, especially if they are over consolidated, show fissures and cracks that represent weak spots in its structure, and consequently cannot be considered unstratified: But the majority of landslides occur in stratified soils (Vedernikov, 1981). These are:

- A) Loose, water-bearing sand: At-Mixtures of gravel, sand, and silt, loosely deposited on stiff clay.
- B) Soft, fissured clays.
- C) Stiff, fissured clays.
- D) Clays with extended planar or pocket-shaped embedments of sand or silt.

Band C may be found in extended layers, whereby the lower layer has different chemical properties from those of the upper one; frequently, the upper layer originates from the decomposition of the lower one. A water film may be present between the clay layers. The engineer designing the stabilization measures faces a great challenge. Based on observations of the ground surface and the mostly rather

meager results from soil explorations in boreholes and trenches, he must be able to take into account the almost infinite variety of combinations of soil properties and hydraulic conditions. In addition, the designing engineer must be aware of the Characteristic Types of Slides and Stabilization Alternatives construction problems of the various stabilization methods and be able to evaluate these properly. Lastly, he must submit estimates regarding duration and costs of construction work, and here it is advantageous if he can draw on personal experience in general, but also on a knowledge of local conditions (veder, 1981).

2.9 Main Causes of Landslides

The causes of landslides are mostly defined as what triggers the landslide. The trigger is the single event that finally initiated the landslide. Thus, causes combine to make a slope vulnerable to failure, and the trigger finally initiates the movement. There are numerous reasons that can lead to a landslide. Most of the time causes of landslides are considered to be factors that made the slope vulnerable to failure, that predispose the slope to becoming unstable. Landslides can have many causes but can only have one trigger as shown in the next figure. Usually, it is relatively easy to determine the trigger after the landslide has occurred.

2.9.1 Geological causes

It is very important to study the causes of landslides from a geological point of view. The soil materials, existing water in the layers and alot of other issues can be counted. Geological causes are listed below:

- Weathered materials
- Sheared materials
- Jointed or fissured materials
- Adversely orientated discontinuities
- Permeability contrasts
- Material contrasts
- Rainfall and snow fall
- Earthquakes

2.9.2 Morphological causes

The formation of the terrain and its inclination play an important part in the stability of a slide area. By experience it may be stated that slopes with a 2:3 inclination (vertical to horizontal) generally are stable. As a rule, it is human beings who disturb the equilibrium of slopes by cutting through them and constructing embankments and bridges for transportation routes. But landslides may also occur without human action, from erosion (undercutting at the toe by unregulated rivers) or from debris deposits (overloading the head of the slope). The morphological causes are listed below.

- Slope angle
- Uplift
- Rebound
- Fluvial erosion
- Wave erosion
- Glacial erosion
- Erosion of lateral margins
- Subterranean erosion
- Slope loading
- Vegetation change
- Erosion

2.9.3 Physical causes

Physical causes are another reasons of landslides. These causes can be categorized as follows:

- Intense rainfall
- Rapid snow melt
- Prolonged precipitation
- Rapid drawdown

- Volcanic eruption
- Ground water changes
- Soil pore water pressure
- Surface runoff
- Seismic activity
- Soil erosion

2.10 Earthquake induced landslide

Earthquakes may trigger not only devastating landslides but also rock falls because of their acceleration effect. Cotecchia and Melidoro (1974) describe how a series of landslides was triggered by an earthquake in Calabria, which in turn dammed up rivers until they became lakes. However, a different effect of earthquakes liquefaction (Casagrande, 1976). Loosely deposited, fully saturated sand (void ratio larger than critical void ratio) may be compacted by seismic tremors (contractant deformation) so that the increased pore-water pressure practically balances the effective stress and the soil liquefies (Seed and Peacock, 1971). Grave damages may be the consequence; constructions, like buildings and embankments, literally vanish into the ground, pipelines break, etc. A way to prevent this liquefaction is the treatment of the soil by vibro flotation, i.e., the shocks of the vibrator cause the soil to collapse so that the void ratio decreases below the critical limit and no dangerous pore-water pressures can build up through eventual earth tremors. Loess soils, in which the voids are filled by air instead of water, are also able to liquefy under the impact of earth tremors as air pressure instead of water pressure builds up. Casagrande and Shannon (1947/48) report on a large-scale loess slide in the Chinese province Kansu, triggered by an earthquake on December 16, 1920, that took 100,000 lives. The reports by Close and McCormick ought to convince any expert that such sudden movements of huge masses of loess can only be explained by a temporary, very drastic reduction of shear strength of the material, as compared to its original strength before the earthquake. This raises the question of whether the pores in the lower strata of the loess were saturated to a degree that made liquefaction possible or whether, during the collapse of the loess structure, the stress was temporarily taken up by the large volume of the air enclosed in the pores. Casagrande

believes that the liquefaction was caused by the air pressure building up in the voids. That the air enclosed in the pores of powdery material may cause a flow can be observed on a smaller scale when, for example, flour or cement is quickly poured from a bag. However, if huge masses of loess, several hundred meters high, are shaken so strongly that their structures collapse, then it is understandable that it requires a longer period of time until the excess pressure of the air in the pores decreases again. During this period the shear strength of the material is at a very low value. If we can assume the credibility of the report, this may explain the great velocity by which a whole mountainside of loess can cover many square kilometers of a valley "within seconds," burying villages and damming up rivers. Regarding the predictability of such loess slides, cracks are bound to show at the head of a potential slide or signs of failure should be observable at the foot, but a sudden strong vibration may cause the collapse of the relatively weak loess structure and thereby its liquefaction. As a stabilization measure the soaking of the endangered area with water was suggested, so that the formerly loose soil structure settles under the effect of the water and becomes stable and firm. When carrying out this treatment, care must be taken not to create an excess pore air pressure; therefore, soaking by sections is recommended. Deformations caused by earthquakes in natural and man-made slopes may develop in three consequent stages. In the first stage, which is co-seismic, gravity as well as seismic forces, which during the relatively short duration of an earthquake may bring about instability, can generate a failure surface or active a pre-existing slip plane causing permanent displacements of the slope. Co-seismic displacements, unless the residual strength of the material in which they occur drops to very low values, are usually small and are controlled by the magnitude and duration of the application of the earthquake inertia forces, the geometry of the slope, and the undrained strength of the material mobilized during the earthquake. The second stage, which is post-seismic, follows immediately after the earthquake if the fast residual undrained shear strength on the slip surface generated by the shock is less than that required to maintain static equilibrium; i.e. if the initial factor of safety of the slope at the end of the earthquake is less than one. A drop in resisting forces will thus lead to an acceleration of the mass downslope. In this case, down-slope displacements, initiated by the earthquake will continue with an outward movement of the toe, the velocity of motion providing the kinetic energy necessary for further transport of slide material. As a result of this, when the mass comes to rest in a new

position of equilibrium its factor of safety will be greater than one. During this second stage driving forces will be due purely to gravity, while resisting forces will continue to depend on the undrained residual strength available on the pre-existing slip surface generated during the first stage and on the resistance due to the growth of the toe. In this stage, displacements will be large if the strength drop is large or if toe support is drastically reduced or lost due to unfavorable topography or extreme loss of strength. A third stage of displacements, which is not examined here, may follow. In this stage further movements may develop as a result of creep and consolidation processes, as well as from destabilizing hydrostatic forces if deep open cracks produced by the shock are filled with surface or ground water. Additional movements may occur, which will be slow and associated with progressive failure and drained strength of the soil. This third stage may follow immediately the first stage if, at the end of the earthquake, the post-seismic static factor of safety FoS is close to one, or if the residual factor is less than one (Veder, 1981).

2.11 Methods of Stability Analysis

There are different ways and methods for analyzing the stability of slopes, proposed by numerous researchers. Hopkins, Allen and Deen (1975) presented a review on several methods. Generally the methods are divided into three categories, based on the number of equilibrium equations to be satisfied.

2.11.1 Methods which satisfy overall moment equilibrium

The most well-known methods in this category are the Fellenius method (1936), the simplified Bishop method (1955), the $\varphi = 0$ method (Taylor, 1937; Huang, 1975), and the logarithmic spiral method (Taylor, 1937; Huang and Avery, 1976). The Fellenius method has been used extensively for many years because it is applicable to nonhomogeneous slopes and is very amenable to hand calculation. This method is applicable only to circular failure surfaces and considers only the overall moment equilibrium; the forces on each side of a slice are ignored. Where high pore pressures are present, a modified version of the Fellenius method, based on the concept of submerged weight and hereafter called the normal method (Bailey and Christian, 1969), is available. The factor of safety obtained by the normal method is usually slightly smaller than that given by the simplified Bishop method. In the simplified

Bishop method, overall moment equilibrium and vertical force equilibrium are satisfied. However, for individual slices, neither moment nor horizontal force equilibrium is satisfied. Although equilibrium conditions are not completely satisfied, the method is, nevertheless, a satisfactory procedure and is recommended for most routine work where the failure surface can be approximated by a circle. Bishop (1955) compared the factor of safety obtained from the simplified method with that from a more rigorous method in which all equilibrium conditions are satisfied. He found that the vertical interslice force, S , could be assumed zero without introducing significant error, typically less than one percent. Hence, the simplified procedure, which set the vertical interslice force to zero, gives approximately the same result as the rigorous procedure, which satisfies all the equilibrium conditions. The $\phi = 0$ method involving circular failure surfaces was used to develop the stability charts for the total stress analysis (Veder, 1981).

2.11.2 Force equilibrium methods

Several methods have been proposed which satisfy only the overall vertical and horizontal force equilibrium as well as the force equilibrium in individual slices or blocks. Although moment equilibrium is not explicitly considered, these methods may yield accurate results if the inclination of the side forces is assumed in such a manner that the moment equilibrium is implicitly satisfied. Arbitrary assumptions on the inclination of side forces have a large influence on the factor of safety. Depending on the inclination of side forces, a range of safety factors may be obtained in many problems. Force equilibrium methods should be used cautiously, and the user should be well aware of the particular side force assumptions employed. The procedure most amenable to hand calculation is the sliding wedge method in which the active or passive earth pressure theory is employed to determine the side forces. This procedure has been used by Mendez (1971) and the Department of Navy (1971). Another procedure, as used by Seed and Sultan (1967), is to assume the shear stress along the failure surface as a quotient of shear strength and factor of safety, and determine the factor of safety from force equilibrium (Huang, 1983).

2.11.3 Moment and force equilibrium methods

Included in this category are the methods by Janbu (1954, 1973), Morgenstern and Price (1965), and Spencer (1967). The basic concept in these methods is the same;

the difference lies in the assumption of the interslice forces. If both moment and force equilibrium are satisfied, the assumption on interslice forces should have only small effect on the factor of safety obtained. All these methods can be applied to both circular and noncircular failure surfaces. In Janbu's method, the location of the interslice normal force, or the line of thrust, must be arbitrarily assumed. For cohesion less soils, the line of thrust should be selected at or very near the lower third point. For cohesive soils, the line of thrust should be located above the lower third point in a compressive zone (passive condition) and somewhat below it in an expansive zone (active condition). This method is very easy to use and does not require as much individual judgment as Morgenstern and Price's method.

In Morgenstern and Price's method, a simplifying assumption is made regarding the relationship between the interslice shear force, S , and the normal force, E . Having made the assumption and obtained the output from the computer, all the computed quantities must be examined to determine whether they seem reasonable. If not, a new assumption must be made. Bishop (1955) indicated that the range of equally correct values of safety factor might be quite narrow and that any assumption leading to reasonable stress distributions and magnitudes would give practically the same factor of safety. Whitman and Bailey (1967) solved several problems using Morgenstern and Price's procedure and the simplified Bishop method and found the resulting difference was seven percent or less, usually less than two percent. In Spencer's method, it is assumed that the interslice forces be parallel; i.e., the angle of inclination, δ , or $\tan^{-1}(S/E)$, is the same at every vertical side. By considering the force and moment equilibrium for each slice, two recursive formulas can be derived for determining the two unknowns F and δ .

In contrast to all the above methods, the finite element method determines the normal and shear stresses along a failure surface by considering the elastic properties of soils in terms of Young's modulus and Poisson's ratio. All the above methods are deterministic in that the shear strength of soils, the loadings applied to the slope, and the required factor of safety are assumed to be known. In reality, a large variation in shear strength, and possibly also in loading, exists (Huang, 1983).

2.12 Types of Failure Surfaces

The purpose of stability analysis is to determine the factor of safety of a potential failure surface. The factor of safety is defined as a ratio between the resisting force and the driving force, both applied along the failure surface (Fig 2.9). When the driving force due to weight is equal to the resisting force due to shear strength, the factor of safety is equal to 1 and failure is imminent. The shape of the failure surface may be quite irregular, depending on the homogeneity of the material in the slope. This is particularly true in natural slopes where the relic joints and fractures dictate the locus of failure surfaces. If the material is homogeneous and a large circle can be formed, the most critical failure surface will be cylindrical, because a circle has the least surface area per unit mass, the former being related to the resisting force and the latter to the driving force. If a large circle cannot be developed, such as in the case of an infinite slope with depth much smaller than length, the most critical failure surface will be a plane parallel to the slope. If some planes of weakness exist, the most critical failure surface will be a series of planes passing through the weak strata. In some cases, a combination of plane, cylindrical, and other irregular failure surfaces may also exist. If the actual failure surface is quite irregular, it must be approximated by either a series of planes or a cylinder before a stability analysis can be made (Huang, 1983).

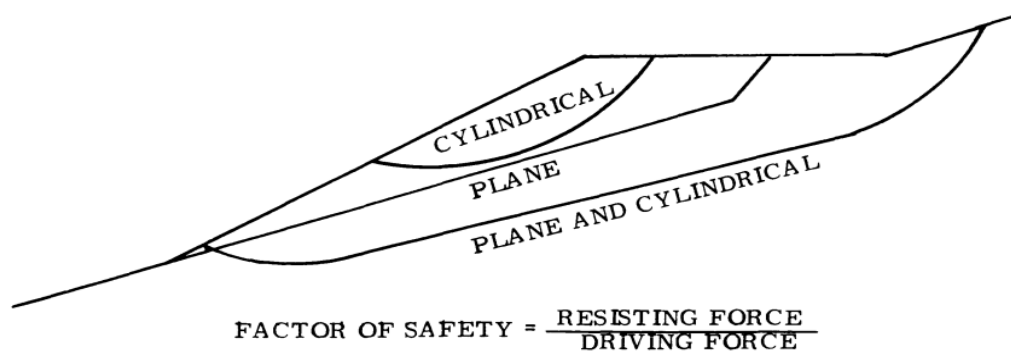


Figure 2.9 : Types of failure surfaces (Huang, 1983).

2.12.1 Plane failure surfaces

The reason that a plane failure exists is because the original hillside is not properly scalped and a layer of weak material remains at the bottom. The resisting force along the failure surface can be determined from the Mohr-Coulomb theory as indicated by Eq:

$$s = c + \sigma_n \tan \phi \quad (2.1)$$

In which s is the shear strength or resisting stress, c is the cohesion, (σ_n is the stress normal to failure surface, and ϕ is the angle of internal friction. Note that c and ϕ are the strength parameters of the soil along the failure surface. For a failure plane of length L and unit width, the resisting force is equal to $cL + W \cos \alpha \tan \phi$, where W is the weight of soil above failure plane and α is the degree of natural slope. Note that $W \cos \alpha$ is the component of weight normal to the failure plane. The driving force is the component of weight along the failure plane and is equal to $W \sin \alpha$. The factor of safety is a ratio between the resisting force and the driving force, or:

$$F = \frac{c + N \tan \phi}{T} = \frac{cL + W \cos \alpha \tan \phi}{W \sin \alpha} \quad (2.2)$$

The case becomes more complicated if the failure surface is comprised of two or more planes. Fig. 2.10 shows a failure surface along two different planes. The sliding mass is divided into two blocks. Both the normal and the shear forces on each plane depend on the interacting force between the two blocks and can only be determined by considering the two blocks jointly. The lower block has a weight, W_1 , and a length of failure plane, L_1 ; the upper block has a weight, W_2 , and a length of failure plane, L_2 . Figure below shows the free-body diagram for each block. The shear force along the failure plane is equal to the shear resistance divided by the factor of safety. For the two blocks shown, there are four unknowns: N_1 , N_2 , P , and F , where N_1 and N_2 are the forces normal to the failure plane at the lower and upper blocks, respectively; P is the force between the two blocks; and F is the factor of safety. Based on statics, there are also four equations; i.e., in each block the summation of forces in the horizontal and the vertical directions is equal to 0. Since there are four equations and four unknowns, one can solve for the factor of safety. If the weaker planes are not continuous or the location of the critical planes is not known appropriately, it is necessary to try different failure planes until a minimum factor of safety is obtained (Huang, 1983).

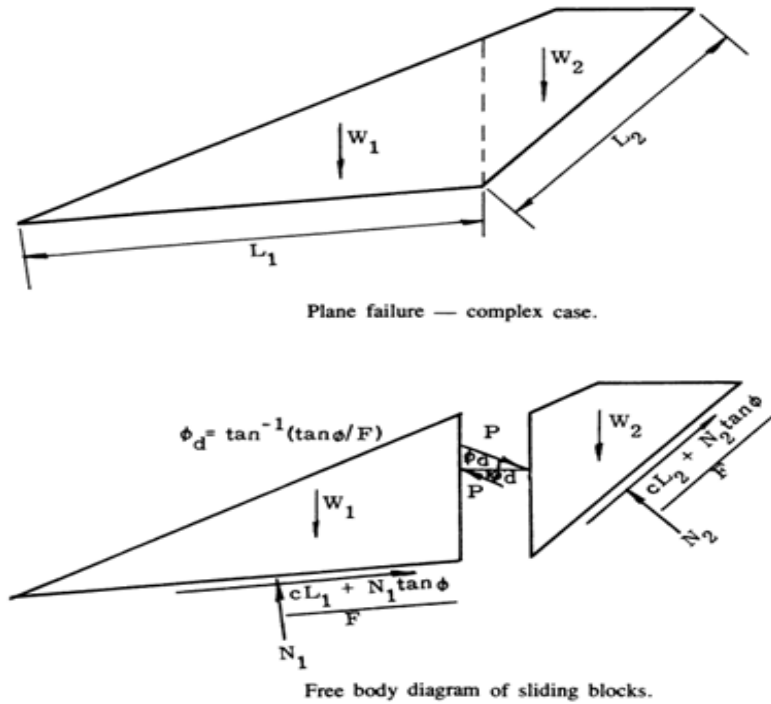


Figure 2.10 : Failure surface along two different planes and free-body diagram for each block (Huang, 1983).

2.12.2 Cylindrical failure surfaces

To find the minimum factor of safety for a cylindrical failure surface, a large number of circles must be tried to determine which is most critical. Figure 2.11 shows one of the many circles for which the factor of safety is to be determined. The sliding mass is divided into n slices. The i th slice has a weight, W_i , a length of failure surface, L_i , an angle of inclination, θ_i , and a normal force, N_i . The factor of safety is a ratio between the resisting force and the driving force. According to the Mohr-Coulomb theory, the resisting force in slice i is $cL_i + N_i \tan \phi$. Note that N_i depends on the forces on the two sides of the slice and is statically indeterminate unless some simplifying assumptions are made. The driving force is equal to $W_i \sin \theta_i$ which is the component of weight along the failure surface. The driving force is independent of the forces on the two sides of the slice because whenever there is a force on one side of the slice, there is a corresponding force, the same in magnitude but opposite in direction on the adjacent side, thus neutralizing their effect. The factor of safety can be determined by:

$$F = \frac{\sum_{i=1}^n (cL_i + N_i \tan \phi)}{\sum_{i=1}^n (W_i \sin \theta_i)} \quad (2.3)$$

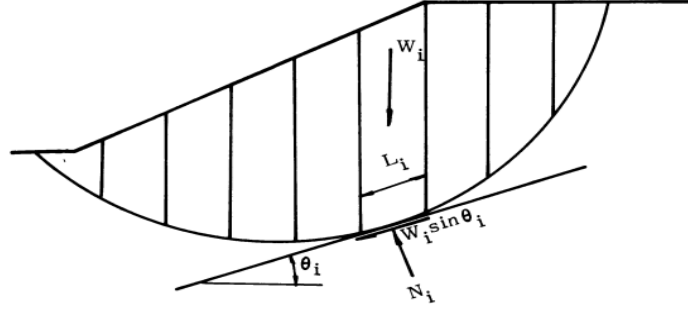


Figure 2.11 : One of the many circles to determine FoS (Huang, 1983).

When the failure surface is circular, the factor of safety can be defined as the ratio between two moments. Because both the numerator and the denominator in Eq. 2.3 can be multiplied by the same moment arm, which is the radius of the circle, it makes no difference whether the force or the moment is used. In the Fellenius method, it is assumed that the forces on the two sides of a slice are parallel to the failure surface at the bottom of the slice, so they have no effect on the force normal to the failure surface, or (Huang, 1983):

$$N_i = W_i \cos \theta_i \quad (2.4)$$

Therefore previous equation becomes:

$$F = \frac{\sum_{i=1}^n (cL_i + W_i \cos \theta_i \tan \phi)}{\sum_{i=1}^n (W_i \sin \theta_i)} \quad (2.5)$$

2.13 Action of Water in Soil

Existence of water in the soil layers has a great importance, especially for landslide cases. The momentary absence of water does not preclude that it may appear in a most damaging manner, for example, after heavy precipitation. It often takes quite a number of years until water becomes active, for example, as pore-water pressure builds up behind the slopes of cuts. A lot of experience and the study of relevant literature is needed to predict such occurrences, and to take timely preventive measures (Veder, 1981).

2.13.1 Action of pore water

Pore water may have a damaging effect in the soil by different processes which has been studied by different researchers (Bauer et al., 1980; Bishop, 1957; Skempton, 1977b).

2.13.1.1 Concentrated pore water action at potential or actual slip surfaces or at walls of cracks in clayey-silty homogeneous soils

This case is typical for conditions in brown London clay, as described in detail by Skempton (1977a,b). His observations regarding the generation of pore water pressure at the slip surface in seemingly homogeneous brown clay are very interesting. Pore-water pressure continuously increases over many years until it finally leads to retarded landslides (Skempton, 1977a,b). It has not been established where this water comes from, perhaps by hydrostatic pressure from an uphill water source, and it is questionable whether in such cases the installation of slightly rising drainage pipes would lead to a stabilization (Veder, 1981).

2.13.1.2 Concentrated pore water action at the zone of division between Clayey-Silty soils of different nature

Pore-water action at the zone of division is quite frequent. Pore-water pressure building up between soil layers with different chemical and physical properties often causes spontaneous landslides. Hydrostatic pressure may build up on account of a connection with an uphill collecting area supplying the water but may also be explained by electro osmotic processes (Veder, 1981).

2.13.1.3 Pore water acting in relatively permeable Silty Sand layers of several decimeters thickness interposed between two relatively impermeable Silty-Clayey layers

In many cases the sand layer wedges out at the surface and shows distinct wetness. During periods of heavy rainfall, water flow in the sand layer may become so strong that overpressure builds up, causing the sliding of the upper impermeable layer owing to a decrease of effective stress. Spontaneous landslides of several meters may take place within a few days (Veder, 1981).

2.13.2 Action of water that is streaming in different types of soils

Sometimes we consider a case in which there is a constant strong source of water. This issue affects soil layers in different ways.

2.13.2.1 Sandy soils

A retrogressive erosion may be happen due to water streaming into and out of uniform silty sand soils. In numerous landslides it has been observed how strong water flow can set sand in motion so that it was flowing out like a thin liquid. So this is a very important issue. This soil material, also called quicksand, sometimes only has an inclination of 1 to 2 degrees, and since it develops quite spontaneously, it causes great damage. Quite justly, quicksand is a menace for the designing engineer (Veder, 1981). Stabilization alternatives include:

- Extensive soil exchange, i.e., removal of quicksand.
- Solidification of the quicksand horizon by grouting with chemicals.
- Drainage by slightly rising horizontal drainage lines (Kenney et al.. 1977).
Usually it is also necessary to install a Terzaghi filter where the sand layer wedges out at the slope toe.

2.13.2.2 Clayey-Sandy silt soils

Water flowing in and out of a slope of clayey-sandy silt will only cause movements if a constant strong water source is present. Under such conditions, the fine particles are not washed out nor does quicksand form, but the whole viscous mass creeps downhill. This mass moves continuously (e.g. several millimeters per day), passing without stopping over any constructions, roads, channels, etc. This type of movement of an embankment toe will cause the embankment to cave in. In a case history of a stabilization where water flowing from permeable limestone mountains dislocated the embankment toe until the roadway failed. The pulpy soil was replaced by a stone wedge which provided the necessary drainage and also increased friction strength in this critical zone.

In another case history for the stabilization of constantly and heavily watered clayey-sandy silt layers, it was decided to excavate up to 5 meters deep drainage trenches, at the bottom of which plastic tubes were placed in gravel and wrapped in filter fabric.

The installation of slightly rising, bored drainage pipes was not considered because the affected area was too large for this method (Veder, 1981).

2.13.3 Action of flowing surface waters

2.13.3.1 Water in vertical or steep, fissured soil layers

In soil layers with that kind of properties, surface water enters the fissures, distends them, softens the marl, and reduces its shear strength. In a case history of this kind, owing to the slope-parallel stratification, whole blocks slid down and landed on the railway below. Here construction of a retaining wall would be a solution but it could be avoided by a number of measures, especially the collection of surface waters in open drainage channels (Veder, 1981).

2.13.3.2 Water mixed with superficial soil

Water flowing on the surface, together with the action of weathering, loosens the structure of the topsoil. When the flow of water increases, for example during heavy rainfalls, a mixture of water, humus, and fractured and weathered rock and soil rushes downhill as a pulpy mass, destroying and carrying along trees, roads, and buildings. These are the much feared debris avalanches, which can only be prevented by extensive constructions (Veder, 1981).

2.13.3.3 Water undercutting slope toes

Flowing water, for example, a river, may cause the toe of a slope to slide. To achieve a permanent stabilization, additional measures are of course required, such as the construction of a wall following the course of the riverbank, to protect the slope toe against further erosion and to prevent undercutting (Veder, 1981).

2.13.3.4 Solifluction

Solifluction is a special type of sliding and flowing strata close to the surface. There the soil thaws only to shallow depths during the short summer seasons. Water from melting and precipitation saturates the topsoil and it flows like heavy snow slush, even on rather level slopes (less than 4 degrees); the subsoil is impermeable since it is firmly frozen. During the Pleistocene, solifluction was quite common and an important factor in the formation of the terrain morphology. This type of sliding is very dangerous because it happens in slopes even with an inclination less than 10

degrees which most of the times nothing happens in slopes with that amount of inclination (Veder, 1981).

3. THE ELAZIĞ EARTHQUAKE

In this part we will have a review of Elazığ earthquake to find out about the region and also the ground motion of earthquake. The slope failures in the region are important for us in this thesis. The general information about the slopes like inclination and also the geology of affected area are presented in this part.

On March 8, 2010, an earthquake shook Kovancılar district and the surrounding villages of Elazığ province at 04:32 (02:32 GMT) local time. Elazığ locates on the East Anatolian Fault (EAF), almost at the north-eastern end, where EAF meets the well-known North Anatolian Fault (NAF) at the Karlıova triple junction. Magnitude and source characteristics of the earthquake are defined by various institutions as given in Table 3.1. In the table, h hypo is the depth of hypocenter of the earthquake and M is the local magnitude (ML) for the top two rows and the moment magnitude (M_w) for the other rows. The earthquake hit a populated region with about 10000 residents and caused 42 human casualties and injuries of 137 individuals (Celep et. al, 2010). According to the Governorship of Elazığ approximately 3000 residential and 181 office buildings were heavily damaged. Stockbreeding sector is seriously affected with a loss of more than 3000 farm animals due to the collapse of barns. Most of the damages were observed in Okçular, Yukarı Demirci, Kayalık and Göçmezler villages within the political boundaries of Kovancılar district. Although Kovancılar and Palu city centers were the two closest districts to the epicenter, most buildings withstood against the shock without experiencing damages. Followed by the main event, however, an aftershock of $ML = 5.6$ at 07:47GMT caused damages in Palu district. The characteristics of this aftershock is shown in table 3.2. Figure 3.1 shows the location of the region which is affected by earthquake (Celep et. al, 2010).

Table 3.1 : 08.03.2010 Elazığ–Kovancılar Earthquake characteristics (Celep et. al, 2010).

Source	Time (GMT)	Latitude (N)	Longitude (E)	h_{hypo} (km)	M
DEMA ^a	02:32:30	38.7752	40.0295	5.01	5.8
KOERI ^b	02:32:31	38.807	40.100	5.00	6.0
USGS ^c	02:32:34	38.873	39.981	12.00	6.1
EMSC ^d	02:32:35	38.84	40.00	10.00	6.0

^a Turkish Prime Ministry-Disaster and Emergency Management Agency, DEMA.

^b Kandilli Observatory and Earthquake Research Institute.

^c United States Geological Survey.

^d European-Mediterranean Seismological Centre.

Table 3.2 : 08.03.2010 Elazığ-Palu Earthquake characteristics (Celep et. al, 2010).

Source	Time (GMT)	Latitude (N)	Longitude (E)	h_{hypo} (km)	M
DEMA	07:47:37	38.7355	40.0090	5.00	5.6
KOERI	07:47:38	38.781	40.066	5.00	5.5
USGS	07:47:40	38.767	40.239	10.00	5.5
EMSC	07:47:40	38.73	40.20	2.00	5.5

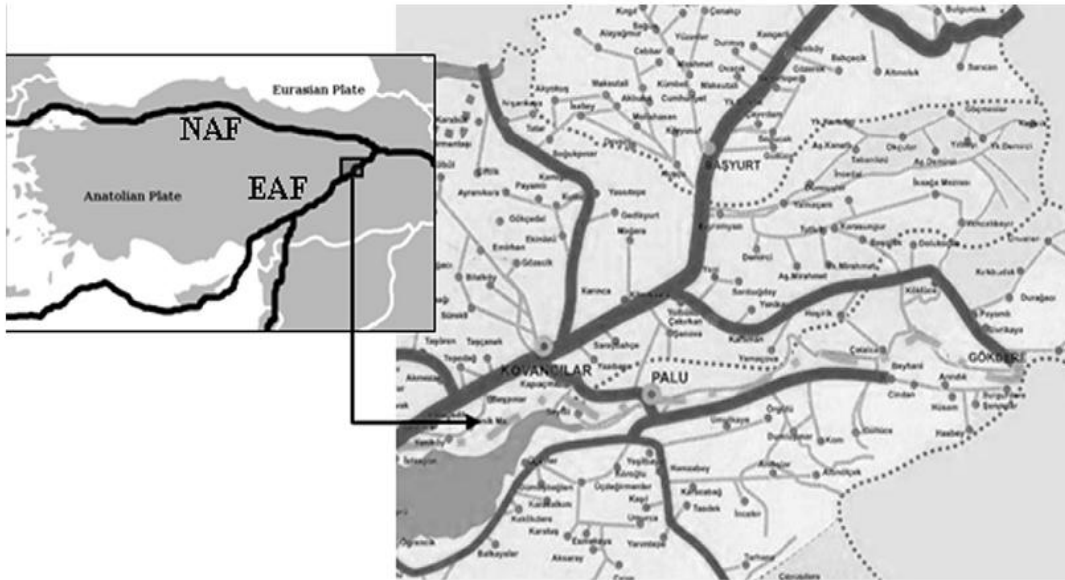


Figure 3.1 : Region affected by the earthquake (Celep et. al, 2010).

3.1 Seismo-tectonic Characteristics of the Region

3.1.1 Seismic sources and historic earthquakes in the region

Each of the strong motions of March 8, 2010; namely the main Kovancılar earthquake and the major Palu aftershock, occurred nearly at the east end of the EAF zone. Starting from Bingöl-Karlıova at the north-east and towards Antakya at the south-west, EAF extends approximately 550 km long. With left-lateral strike slip moving segments, EAF constitutes the tectonic boundary between the Anatolian Plate and the northward-moving Arabian Plate defined the active tectonic zones on

EAF from east to west as Karlıova-Bingöl segment; Palu-Hazar Lake segment; Hazar Lake-Sincik segment; Çelikhan-Erkenek segment; Gölbaşı-Türkoğlu segment and Türkoğlu-Antakya segment. Historical studies indicate that Elazığ and its surroundings have experienced destructive earthquakes in the past (Celep et. al, 2010).

3.1.2 Seismic activity from March 8 to 18, 2010

Until March 18, a number of 275 aftershock motions are recorded pursuing the main event on March 8, 2010, including four of them with magnitudes $M_L \geq 5.0$. As emphasized previously, the major $M_L = 5.6$ aftershock hit Palu district causing partial or total collapse of some adobe and stone buildings. These aftershocks are shown in Figure 3.2 (Celep et. al, 2010).

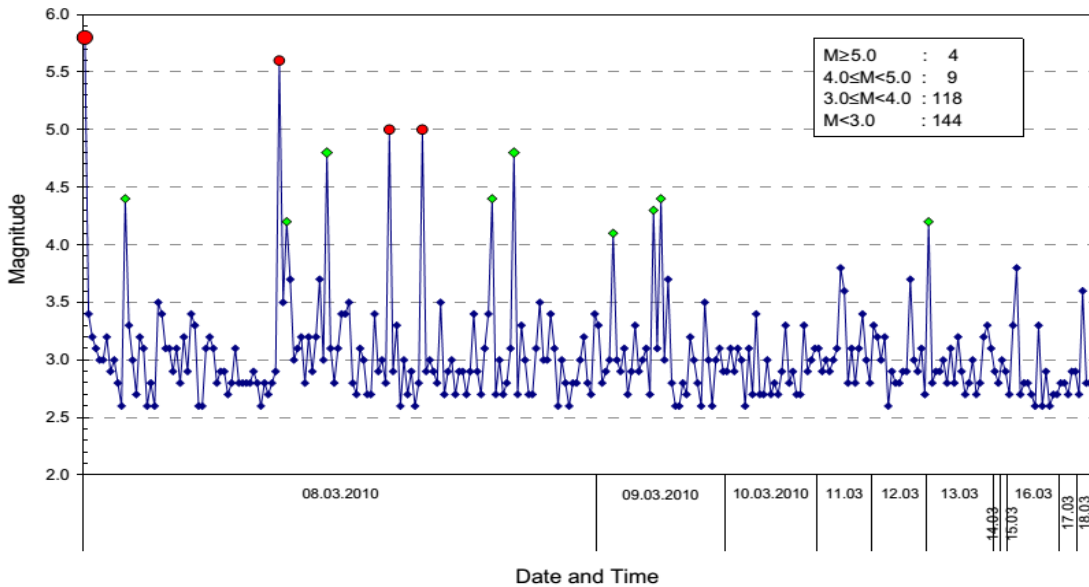


Figure 3.2 : Distribution of aftershock magnitudes along time scale (Celep et. al, 2010).

3.2 Evaluation of Strong Ground Motion

3.2.1 Characteristics of strong motion records

Characteristics of the strong motion accelerogram provided by the National Strong Motion Recording Stations of DEMA are given in Table 3.3 for Kovancılar–Elazığ Earthquake (02:32GMT $M_L = 5.8$) at the epicentral coordinates of 38.7752N–40.0295E, where R_{epi} is the epicentral distance from the station and PGA is the peak ground acceleration. The strong motion accelerogram having $R_{epi} = 12.2$ km has the

smallest epicentral distance. It was recorded at the Palu station located in the Directorate of Meteorology Building. However, the maximum PGA values of 0.063g, 0.068g and, 0.031 g for NS, EW and UD directions, respectively, are found to be surprisingly low compared to the damage observed within the region.

Detailed information for the major aftershock of $ML = 5.6$ occurred at 07:47GMT on the epicentral coordinates 38.7355N– 40.0090E is given in Table 3.4. PGA for NS, EW and UD directions are measured as 0.078g, 0.049g and 0.056g, respectively, at the closest recording station with $R_{epi} = 8.0$ km.

The strong motion acceleration-time histories are processed by taking into account the characteristics of the previous earthquake motions recorded in the region and the engineering intensities are determined. Figure 3.3 shows the time histories for acceleration, velocity and displacement as well as for Fourier Amplitude Spectra (FAS). In the process, initially, baseline correction is performed for the three components of the records of the main event for Palu station. Later, for removing the undesirable frequency (noise) contents in the records, a causal fourth order Butterworth band filter having corner frequencies found from FAS, is applied to the acceleration records. The processed time histories yielded the peak ground velocities of 6.0 cm/s, 5.9 cm/s and 3.1 cm/s, for the NS, EW and UD components, respectively.

The maximum displacement is calculated as 2.3 cm for the NS component. The effective durations t_{eff} , of the horizontal components are also computed assuming that t_{eff} , is the time interval between the 95% and 5% values on the Arias Intensity (AI) curves. From the plots of AI (%) given in Figure 3.4, the effective durations are calculated as 15.5 s and 13.8 s for the NS and EW components, respectively (Celep et. al, 2010).

Table 3.3 : Characteristics of the strong motion records of $ML = 5.8$ Kovancılar Earthquake (Celep et. al, 2010).

Station information			R_{epi} (km)	PGA (cm/s ²)		
Name	Latitude (N)	Longitude (E)		NS	EW	UD
Palu Dir. Meteo. Bldg.	38.69577	39.9319	12.2	62.00	66.50	30.00
Bingöl Dir. Pub. Works Bldg.	38.89708	40.5032	43.3	55.31	34.27	25.50
Elazığ Dir. Pub. Works Bldg.	38.67043	39.19267	73.6	5.56	4.77	3.85
Diyarbakır Dir. Pub. Works Bldg.	37.93088	40.20278	95.2	3.44	5.10	2.59
Karlıova Municip. Garage	39.29345	41.00883	102.5	11.59	17.84	8.95
Batman Dir. Pub. Works Bldg.	37.873	41.15112	140.3	7.62	5.44	2.52
Mardin Dir. Pub. Works Bldg.	37.32632	40.72374	172.4	2.54	2.46	1.68
Adıyaman Dir. Pub. Works Bldg.	37.762	38.267	190.9	2.50	2.24	1.64

Table 3.4 : Strong motion records of ML = 5.6 Palu Earthquake (aftershock)
(Celep et. al, 2010).

Station information			R_{epi} (km)	PGA (cm/s ²)		
Name	Latitude (N)	Longitude (E)		NS	EW	UD
Palu Dir. Meteo. Bldg.	38.69577	39.9319	8.0	76.5	48.00	55.00
Bingöl Dir. Pub. Works Bldg.	38.89708	40.5032	46.5	14.49	10.20	7.07
Elazığ Dir. Pub. Works Bldg.	38.67043	39.19267	71.3	3.54	4.21	3.33
Karlıova Municip. Garage	39.29345	41.00883	106.5	2.86	3.86	1.55

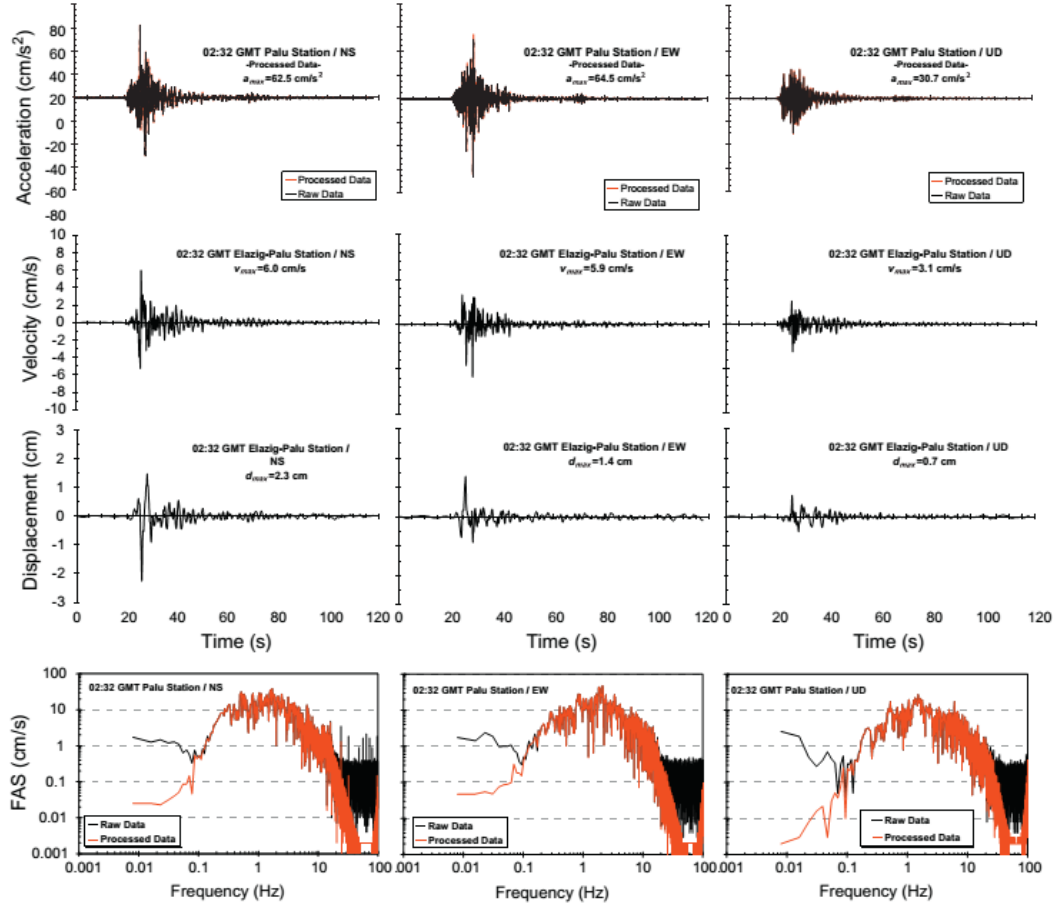


Figure 3.3 : Processed records: top three rows for acceleration, velocity and displacement histories and the bottom row for the comparison of FAS for raw and processed acceleration-time histories (Celep et. al, 2010).

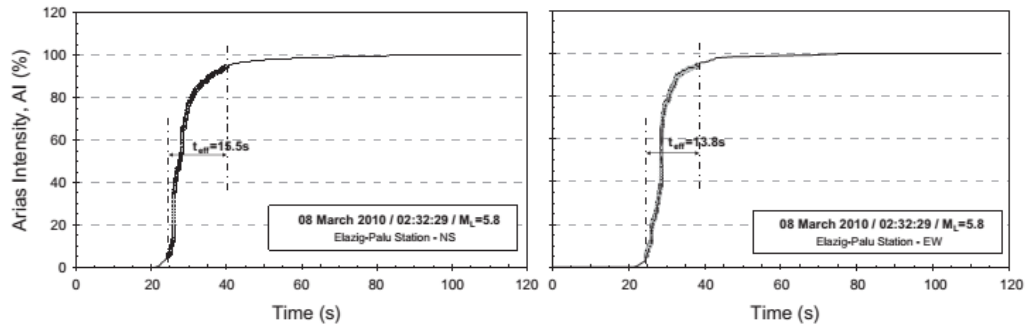


Figure 3.4 : AI curves for the horizontal components and the effective duration teff. (Celep et. al, 2010).

3.3 Geological and Geotechnical Characteristics of the Site

In the affected area, mainly metamorphic, magmatic and sedimentary rocks are dominant. The reason of the sedimentary layers in the area is Murat river, which runs southwards in a valley . Limestone, Pliocene deposits, sandstone, conglomerate and volcanic rock units outcrop at the mountain of Murat and the Valley of Murat River. Pliocene deposits, which consist of clay, sand and gravel, appear at the other parts of the site. Limestone is observed at the north-east part of Kovancılar and its villages. Failures related to geological and geotechnical characteristics of the site and the corresponding parameters of the motion are summarized below. (Celep et. al, 2010).

3.3.1 Slope failures in the area

A lot of slope failures are observed in the earthquake region during the Kovancılar (ML = 5.8) and the Palu Earthquakes (ML = 5.6). These failures were observed particularly on the embankments along Palu-Köklüce road within the valley of the Murat River and on the natural slopes along the southern parts of the Murat Mountain, where Okçular, Yukarı Demirci and Kayalık villages are located. Figures 3.5, 3.6, 3.7 and 3.8 show the view of damaged villages. The villages, where heavy damages occurred, are located on soil layers consisting of clay, sand and gravel mixture. Rubbles also cover the slope of the hills. Okçular, Yukarı Demirci and Kayalık Villages are settled on such slopes. Figure 3.8 shows the Yukarı Demirci Village located on the slopes of the neighboring mountain. (Celep et. al, 2010).



Figure 3.5 : Aerial photo of damaged Okçular village (Yilmaz & Uran, 2010).



Figure 3.6 : A general view of damaged Okçular village (Yilmaz & Uran, 2010).



Figure 3.7 : Location of Yukarı Demirci Village (Celep et. al, 2010).



Figure 3.8 : Tension cracks on the side of the road in the region (Celep et. al, 2010).

Inclinations of the natural slopes vary from 10 up to 35 degrees. Clays with silt, sand and gravel are dominated in the site. The thickness of the clayey layer decreases up to the hill. Weathered limestone, sandstone and volcanic units are outcrop. The groundwater table lies just below the interface between soil layer and rock units. Rubbles consisting of silty, clayey sandy stones have fallen down from the steep slopes around the road (Fig. 3.9). Fig. 3.10 shows the slope failures along a stream in the region. Similar slope failures are observed on the slopes of the Murat Mountain along the Murat River Valley, some of which caused damages especially in poor quality structures with inadequate foundations. Fig. 3.11 exhibits a building influenced by the sliding of the foundation soil with the inclination angle of approximately 35 degrees. The building has only a ground story and a partial basement. During the earthquake, settlement induced shear cracks were developed on the walls of the structure due to slope failure. Tension cracks also occurred on the soil around the structure, which clearly indicate the motion under the foundation of the building (Fig. 3.12). From the failure and the observations on the visible part of the sliding surface, the entire shape of the failure surface seems to be planar or curved having a very large curvature radius. The regions, where slope failures are observed, generally lay on the slopes of hills consisting of clay, sand and gravel mixture. Inclinations of the slopes of hills generally vary between 10 and 35 in the areas of settlement. Therefore, it is assumed that the internal friction angles of the layers are between 20 and 35, which correspond to loose to the medium dense soils. Slopes of streams next to Yukarı Demirci Village as shown in Fig. 3.10. The maximum displacements on the ground surface at foundation levels in the villages of Yukarı Demirci, Okçular and Kayalık were observed to be up to 20 cm, which is comparable to the calculated displacement values within a few centimeters (Celep et. al, 2010).



Figure 3.9 : Failure surface (Celep et. al, 2010).



Figure 3.10 : Slope failures around the stream in the region (Celep et. al, 2010).



Figure 3.11 : A rural building on slope at Yukarı Demirci Village (Celep et. al, 2010).



Figure 3.12 : A view of the building from backyard (Celep et. al, 2010).

4. FINITE ELEMENT METHOD

The finite element method (FEM), sometimes referred to as finite element analysis (FEA), is a computational technique used to obtain approximate solutions of boundary value problems in engineering. Simply stated, a boundary value problem is a mathematical problem in which one or more dependent variables must satisfy a differential equation everywhere within a known domain of independent variables and satisfy specific conditions on the boundary of the domain (Hutton, 2004). Boundary value problems are also sometimes called field problems. The field is the domain of interest and most often represents a physical structure. The field variables are the dependent variables of interest governed by the differential equation. The boundary conditions are the specified values of the field variables (or related variables such as derivatives) on the boundaries of the field. Depending on the type of physical problem being analyzed, the field variables may include physical displacement, temperature, heat flux, and fluid velocity to name only a few (Hutton, 2004).

The Finite Element Method has become so popular in recent years. Recently different programs and codes have been available for FEM analysis. In this study the Finite Element program, PLAXIS 2D version 2012, has been used to conduct analysis on slopes. This program is one the most prominent tools for FEM analysis, which has lots of useful features. The advantages of a FE approach to slope stability analysis over traditional limit equilibrium methods can be summarized as follows:

- No assumption needs to be made in advance about the shape or location of the failure surface. Failure occurs 'naturally' through the zones within the soil mass in which the soil shear strength is unable to sustain the applied shear stresses.
- Since there is no concept of slices in the FE approach, there is no need for assumptions about slice side forces. The FE method preserves global equilibrium until failure is reached.

- If realistic soil compressibility data are available, the FE solutions will give information about deformations at working stress levels.
- The FE method is able to monitor progressive failure up to and including overall shear failure.

4.1 Constitutive Soil Models

Probably the most important part of the numerical modelling process is the selection of the constitutive model and determination of the corresponding model parameters. Real soil behavior may involve several features that can be observed and measured in lab tests and in situ, but which are still difficult to capture in a homogenized continuum formulation.

Currently, various constitutive models for soils have been developed. These models cover a wide range of soil features such as anisotropy, cyclic loading, creep etc. The selection of a constitutive model for a geotechnical application depends on the mechanical properties of the soil (i.e. permeability, stiffness and strength), previous history of the soil and the stress changes that will occur in future (Wood and Muir, 1990). In the finite element analysis, reliable predictions can be achieved by using an appropriate constitutive model for a particular geotechnical problem. Soil behaviour may be modelled at different degree of accuracy. Many constitutive models are developed to characterize and interpret soil behaviour accurately. The simpler soil models need less data but the more complex models need more accurate data for modelling (Brinkgreve, 2005). Every constitutive model requires different parameters and every model has limitations. Some constitutive models are available in the Plaxis program. A brief description of these models is given below. (Plaxis Material Models Manual, 2012).

4.1.1 Linear elastic model (LE)

The Linear Elastic (LE) model is based on the Hooke's law of isotropic elasticity. It is the simplest model available which only involves two basic parameters, Young's modulus (E) and Poisson ratio (ν). The Linear Elastic model is not appropriate for modeling soil behavior because soil behavior is highly nonlinear and irreversible, but

it is appropriate to model stiff volumes in soil like concrete walls. (Plaxis Material Models Manual, 2012).

4.1.2 Mohr coulomb model (MC)

The Mohr Coulomb (MC) model is a linear elastic and perfectly plastic model. It involves five parameters, both elastic and plastic behavior parameters, i.e. Young's modulus (E) and Poisson's ratio (ν) for soil elasticity; friction angle (ϕ) and cohesion (c), for soil plasticity and the dilatancy angle (ψ). Although the increase of stiffness with depth can be taken into account, this model does not include stress-dependency, stress-path dependency or anisotropic stiffness. The stiffness behavior below the failure line is assumed to be linear elastic according to Hook's law. Hence, the model does not accurately predict deformation behavior before failure, especially in situations where the stress level is changing or multiple different stress paths are followed. For tunnel construction, the MC model predicts too wide settlement trough. As far as its strength behavior is concerned, this model is suitable to analyze the stability of dams, slopes and embankments. However, the model does not show softening behavior after peak strength. The soft soils, like normally consolidated clays, generally show a decreasing mean effective stress during undrained shearing, whereas the Mohr-Coulomb model would predict a constant mean effective stress in this case, which results in an over-prediction of the shear strength. The simplicity of the MC-model allows a fast and simple application, but does not account for typical characteristic of soils, such as the stress-dependent stiffness behavior and irreversible strains due to primary isotropic compression. These essential aspects are covered by the more advanced HS-model, an elastoplastic model with isotropic hardening. (Plaxis Material Models Manual, 2012).

4.1.3 Soft soil model (SS)

The Soft Soil (SS) model is a Cam-Clay type of model for predicting the behavior of normally consolidated soils (clays, clayey silts and peat). The Hardening Soil model supersedes the SS model. The SS model does not over predict the shear strength for over consolidated states of stress. The SS model works well in primary loading conditions, such as embankment or foundation construction. The model has no advantages over the Mohr-Coulomb model in unloading situations, such as excavations or tunnel construction (Plaxis Material Models Manual, 2012).

4.1.4 Soft soil creep model (SSC)

The Soft Soil Creep (SSC) model is suitable for prediction of creep related settlement of embankments and foundations in soft soils (normally consolidated clays, silts and peat). The SSC model over predicts the range of elastic behavior in unloading situations such as excavations and tunnel construction. In these situations, the SSC model hardly supersedes the Mohr-Coulomb model (Plaxis Material Models Manual, 2012).

4.1.5 Modified cam clay model (MCC)

The Modified Cam Clay Model (MCC) uses a logarithmic relationship between the mean effective stress and the void ratio. Hence, the model involves a linear stress-dependency of the stiffness, which is more realistic for normally consolidated clays. For over-consolidated states of stress, the MCC model uses Hook's law of isotropic elasticity. For highly over-consolidated soils, the MCC model predicts an unrealistic elastic range which leads to an unrealistic high peak strength, followed by softening behavior until the critical state is reached. Hence, the MCC model is not suitable for highly over-consolidated soils. The model is suitable for soft soils such as normally consolidated clays (Plaxis Material Models Manual, 2012).

4.1.6 Hardening soil model (HS)

The Hardening Soil (HS) model (Brinkgreve & Vermeer, 1997; Schanz & Vermeer, 1998) is a true second order model for soils in general (soft as well as hard soils), for most types of applications. The model, in an undrained loading shows a reduction in mean effective stress for soft soils and an increase in mean effective stress for hard soils, respectively. The hardening Soil model is an advanced elasto-plastic model soil model and from the aspect of stiffness, it is more accurate than Mohr-Coulomb model because it considers three more stiffness parameters in modeling. This model can accurately predict displacements and failure for general types of soils in various geotechnical applications. The model does not include anisotropy and time dependent behavior. The results of this model is a better approximation of real soil behavior which is shown in the Figure. 4.1 (Plaxis Material Models Manual, 2012).

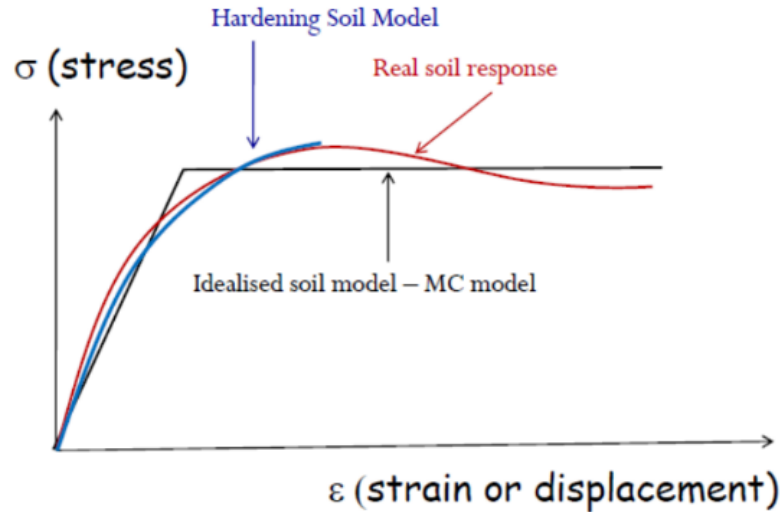


Figure 4.1 : Comparison of real soil behavior with HS and MC models (Plaxis 2012 manual).

This model is a double hardening model based on the Mohr-Coulomb failure criterion. It consists of two main types of hardening which are shear hardening and volumetric hardening. Table 4.1 shows the input parameters for this soil model.

Table 4.1 : Input parameters of HS model (Plaxis Material Models Manual, 2012)

Symbol	Parameter
ϕ'	Internal friction angle
c'	Cohesion
ψ	Dilatancy angle
E_{50}^{ref}	Reference secant stiffness from drained triaxial test
E_{oed}^{ref}	Reference tangent stiffness for oedometer primary loading
E_{ur}^{ref}	Reference unloading/reloading stiffness
m	Power for stress-level dependency of stiffness
ν_{ur}	Unloading/reloading Poisson's ratio
K_0	Coefficient of earth pressure at rest = $1 - \sin \phi$
R_f	Failure ratio (0.9~1)

4.1.7 Hardening soil model with small strain (HSsmall)

The Hardening Soil model with small strain stiffness (HSsmall) is a modified version of the original Hardening Soil model. At low strain levels, most soils show a higher stiffness than at engineering strain levels, and the stiffness varies non-linearly with

strain. This behavior is captured by HSsmall model. (Plaxis Material Models Manual, 2012). At low strain levels, soil behaviour is different and most soils behave stiffer. This behaviour of soils is defined in HSsmall model with two new parameters: small strain shear modulus (G_0^{ref}) and strain level at which the shear modulus reduces to 70% of the small strain shear modulus ($\gamma_{0.7}$). The HSsmall model is able to calculate the displacements more precisely and also in dynamic applications, HSsmall model accounts for hysteretic material damping (Plaxis Material Models Manual, 2012).

4.2 Selection of Appropriate Soil Model

The selection of a constitutive model should be based on an evaluation of the capabilities (and limitations) that the model has to describe the essential features of soil behavior for the application at hand. In that respect, the constitutive model provides the qualitative description of soil behavior, whereas the parameters in the model are used to quantify the behavior. The composition of the model plus parameters can be regarded as the ‘artificial soil’ that is used in the finite element model, which should be representative for the real soil behavior in the application. Before considering the numerical model in full detail, it makes sense to evaluate the behavior of the ‘artificial soil’ (= model + parameters) separately in particular stress paths. Therefore it is useful to check the behavior of the soil in simplified soil lab tests simulations in element tests or using a single stress-point algorithm (Brinkgreve R.B.J. 2013). The Hardening Soil with Small Strain Stiffness model has been chosen for modeling the slope failure under earthquake loading. This model is the best available model in the Plaxis program which is better for conducting dynamic analysis.

4.2.1 Hardening soil with small strain model

Recently, the HS-model was extended with small strain stiffness (HSsmall). The HSsmall model is a modification of the HS-model that accounts for the increased stiffness of soils at small strains. At low strain levels most soils exhibit a higher stiffness than at engineering strain levels, this stiffness varies non-linearly with strain. HS small model is suitable for the analysis of both static and dynamic tasks. The behaviour of soil is assumed elastic in the hardening soil model during

unloading and reloading. Nonetheless, elastic strain range in soils is very small. Most soils display higher stiffness at low strain levels. As strain amplitude increases, soil stiffness decreases nonlinearly. In Figure. 4.2, reduction of stiffness is shown.

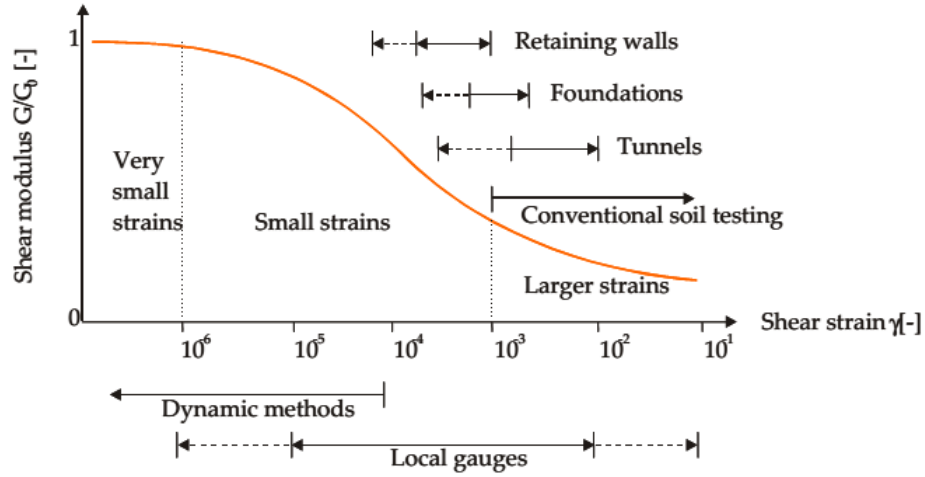


Figure 4.2 : Reduction of shear modulus with typical strain ranges (Atkinson & Sallfors,1991).

The original Hardening Soil (HS) model assumes elastic material behavior during unloading and reloading. However, the strain range in which soils can be considered truly elastic, i.e. where they recover from applied straining almost completely, is very small. With increasing strain amplitude, soil stiffness decays nonlinearly. Figure gives an example of such a stiffness reduction curve. It shows the characteristic stiffness-strain behavior of soil with typical strain ranges for laboratory tests and structures. One feature of soil behavior that was still missing in the HS-model is the high stiffness at small strain levels ($<10^{-5}$). Even in applications that are dominated by ‘engineering strain levels ($>10^{-3}$)’, small-strain stiffness can play an important role. It is generally known that conventional models over-predict heave in excavation problems. These models also over-predict the width and under-predict the gradient of the settlement trough behind excavations and above tunnels. Small-strain stiffness can improve this. Moreover small-strain stiffness can be used to model the effect of hysteresis and hysteretic damping in applications involving cyclic loading and dynamic behavior. (Plaxis Material Models Manual, 2012).

The HSsmall model is based on the Hardening Soil (HS) model and uses almost entirely the same parameters. In fact, only two additional parameters are needed to describe the stiffness behavior at small strains. These are the initial or very small-

strain shear modulus G_0^{ref} , and the shear strain level $\gamma_{0.7}$ at which the secant shear modulus G is reduced to 70% of G_0^{ref} . The following equation shows the corresponding relationship between G_{0ref} and $\gamma_{0.7}$.

$$\frac{G_s}{G_0} = \frac{1}{1 + \frac{3}{7} \times \frac{\gamma}{\gamma_{0.7}}} \quad (4.1)$$

The advanced features of the HSsmall model are most apparent in working load conditions. Here, the model gives more reliable displacements than the HS-model. When used in dynamic applications, the HSsmall model also introduces hysteric material damping. Thus, this advanced constitutive model is especially suited for analysis of ductile structures, both for static and dynamic applications. The material parameters of the HSsmall model can be obtained by conducting classical laboratory tests, i.e. triaxial tests and resonant-column tests without special instrumentation. In soil dynamics, small-strain stiffness has been a well known phenomenon for a long time. In static analysis, the findings from soil dynamics have long been considered not to be applicable. Seeming differences between static and dynamic soil stiffness have been attributed to the nature of loading (e.g. inertia forces and strain rate effects) rather than to the magnitude of applied strain which is generally small in dynamic conditions (earthquakes excluded). As inertia forces and strain rate have only little influence on the initial soil stiffness, dynamic soil stiffness and small-strain stiffness can in fact be considered as synonyms. The advantage of using the HSsmall in combination with dynamic calculations becomes obvious (Plaxis Material Models Manual, 2012).

4.3 Model Parameters

The material parameters which are used in HSsmall model are given in Table 4.2. The two differences between this model and original HS model are two new parameters.

Table 4.2 : Input parameters of HSsmall model (Plaxis Models Manual, 2012).

Symbol	Parameter
ϕ'	Internal friction angle
c'	Cohesion
ψ	Dilatancy angle
E_{50}^{ref}	Reference secant stiffness from drained triaxial test
E_{oed}^{ref}	Reference tangent stiffness for oedometer primary loading
E_{ur}^{ref}	Reference unloading/reloading stiffness
m	Power for stress-level dependency of stiffness
ν_{ur}	Unloading/reloading Poisson's ratio
G_0^{ref}	Reference small strain shear modulus
$\gamma_{0.7}$	Shear strain magnitude at $0.722G_0$
K_0	Coefficient of earth pressure at rest
R_f	Failure ratio

4.4 Determination of Model Parameters

Determination of model parameters is one of the most important parts of modelling. The more accurate and reliable parameters we have means more precise and close to reality results. To determine the model parameters, different results of laboratory tests and also correlations based on the results are used. For example laboratory tests like Standard Penetration Test (SPT), Cone Penetration Test (CPT), Sieve Analysis, Atterberg limit tests and lots of other tests are used. Using each of the tests and correlations will be discussed in this chapter.

4.4.1 Determination of internal friction angle

Internal friction angle of soils is determined by performing laboratory tests or in-situ tests. In laboratory, Angle of Internal Friction, can be determined in the laboratory by the Direct Shear Test or the Triaxial Stress Test. In field, penetration tests could be used to determine internal friction angle of cohesionless soils. Hatanaka and Uchida (1996) developed an empirical correlation formula between $N_{1,60}$ values and internal friction angle. This formula is represented in formula below.

$$\phi' = (20N_{1,60})^{0.5} + 20 \quad (4.2)$$

Researchers also have prepared different tables based on soils SPT value or other test results. Some of them are summarised in Tables 4.3, 4.4 and 4.5.

Table 4.3 : Empirical values for friction angle of granular soils based on the standard penetration number (Bowels, Foundation Analysis).

SPT Penetration, N- Value (blows)	ϕ (degrees)
0	25 - 30
4	27 - 32
10	30 - 35
30	35 - 40
50	38 - 43

Table 4.4 : Relationship between soil friction angle and standard penetration number for sands, (Peck 1974, Foundation Engineering Handbook).

SPT Penetration, N-Value(blow)	Density of Sand	ϕ (degrees)
<4	Very loose	<29
4 - 10	Loose	29 - 30
10 - 30	Medium	30 - 36
30 - 50	Dense	36 - 41
>50	Very dense	>41

Table 4.5 : Relationship between , friction angle and standard penetration number for sands, (Meyerhof 1956,Foundation Engineering Handbook).

SPT Penetration, N-Value(blow)	Density of Sand	ϕ (degrees)
<4	Very loose	<30
4 - 10	Loose	30 - 35
10 - 30	Medium	35 - 40
30 - 50	Dense	40 - 45
>50	Very dense	>45

Bowles (1988), based on his works, summarizes the values of ϕ , D_r , corrected SPT values (Tables 4.6 and 4.7).

Table 4.6 : Values of ϕ and D_r based on SPT N'_{70} (Bowles, 1988).

Description		Very Loose	Loose	Medium	Dense	Very Dense
Relative Density (D_r) (%)		0-15	15-35	35-65	65-85	85-100
SPT N'_{70}	fine	1-2	3-6	7-15	16-30	>30
	medium	2-3	4-7	8-20	21-40	>40
	coarse	3-6	5-9	10-25	26-45	>45
Φ	fine	26-28	28-30	30-34	33-38	<50
	medium	27-28	30-32	32-36	36-42	
	coarse	28-30	30-34	30-34	40-50	

Table 4.7 : Relation between SPT N-Value and consistency of fine grained soil(Bowles, 1988).

SPT N-Value (No Correction)	Consistency
< 2	Very Soft
3-4	Soft
5-8	Medium Stiff
9-15	Stiff
16-30	Very Stiff
> 30	Hard

4.4.2 Determination of cohesion

Cohesive soils are clay type soils. Cohesion is the force that holds together molecules or like particles within a soil. Cohesion, C , is usually determined in the laboratory from the Direct Shear Test. Undrained shear Strength, S_{uc} , can be determined in the laboratory using the Triaxial Test or the Unconfined Compressive Strength Test. There are also correlations for S_{uc} with shear strength as estimated from the field using Vane Shear Tests. The cohesion C can be derived using the formula below.

$$C = \frac{q_u}{2} \quad (4.3)$$

Where:

C = cohesion, kN/m^2

q_u = Unconfined compression strength, kN/m^2

In literature, there many correlation formulas between standard penetration value and undrained shear strength of clay. Undrained triaxial tests and standard penetration tests on insensitive clays are performed to derive correlations. The general form of correlations follows:

$$q_u = kN_{1,60} \quad (4.4)$$

Where, q_u is unconfined compression strength of clay, k is site-dependent constant and $N_{1,60}$ is corrected SPT-N value. Terzaghi and Peck (1967) proposed a table (Table 4.8), about consistency of cohesive soils.

Table 4.8 : Consistency of cohesive soils (Terzaghi and Peck, 1967).

Consistency	SPT-N	q_u (kPa)
Very Soft	<2	<25
Soft	2-4	25-50
Medium	4-8	50-100
Stiff	8-15	100-200
Very Stiff	15-30	200-400
Hard	>30	>400

Cohesion value of soils is determined by using unconfined compression strength.

$$c = q_u/2 \quad (4.5)$$

Where, c is cohesion and q_u is unconfined compression strength.

4.4.3 Determination of dilatancy angle

The dilatancy angle of sands is related to the density and internal friction angle of soil. Generally, dilatancy angle of soil is a smaller value than internal friction angle. For quartz sands dilatancy angle is calculated as:

$$\psi = \phi - 30 \quad (4.6)$$

Where, ψ is dilatancy angle and ϕ is internal friction angle of soil. For the soils whose internal friction angle is smaller than 30° , the dilatancy angle is zero (Plaxis Material Models Manual, 2012).

4.4.4 Determination of stiffness parameters

There are three different stiffness parameters and five other parameters required to model hyperbolic stress-strain curve. These parameters are:

- Secant stiffness in standard drained triaxial test, E_{50}^{ref}
- Tangent stiffness for primary oedometer loading, E_{oed}^{ref}
- Unloading/reloading stiffness, E_{ur}^{ref}
- Power for stress-level dependency of stiffness, m
- Poisson's ratio for unloading/reloading, ν_{ur}
- Failure ratio, R_f
- Reference stress for stiffness, p^{ref}
- K_0^{NC}, K_0 value for normal consolidation (Schanz & Vermeer, 1998).

Hyperbolic stress-strain relation between in primary loading for a standard drained triaxial test is shown in Figure 4.3.

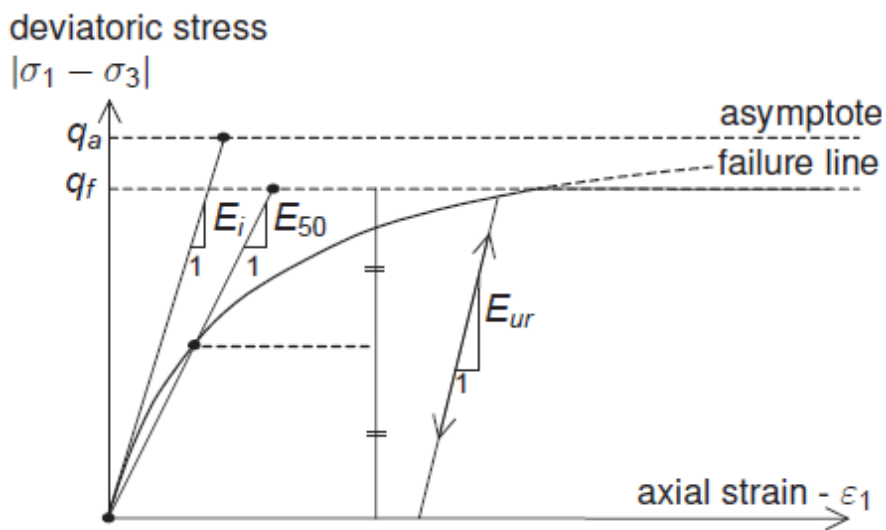


Figure 4.3 : Hyperbolic stress-strain relation between in primary loading for a standard drained triaxial test (Plaxis Material Models Manual, 2012).

The secant modulus parameter, E_{50} , is determined by using strain curve at 50% of the ultimate shear strength in triaxial test. Also, E_{50} can be determined by using initial modulus, E_0 . Based on the works of Mayne (2007), E_{50} can be obtained by using modulus reduction curves. These curves are shown in Figure 4.4.

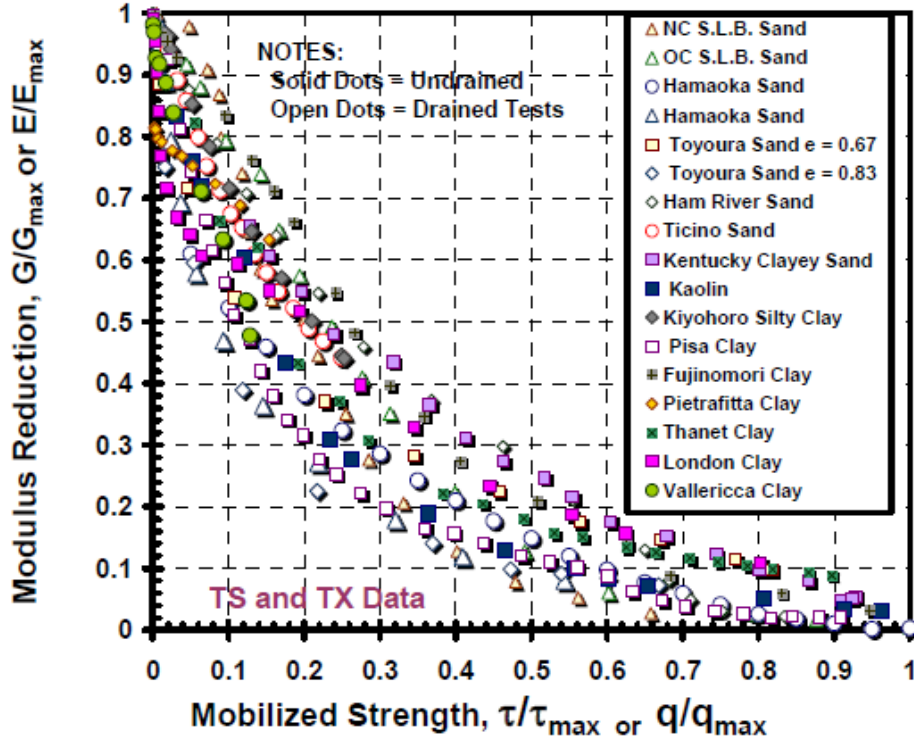


Figure 4.4 : Monotonic modulus reduction curves static torsional and triaxial shear data on clays and sands (Mayne, 2007).

The confining stress dependent stiffness modulus for primary loading is calculated by using following equation:

$$E_{50} = E_{50}^{ref} \left(\frac{c \cos \phi - \sigma'_3 \sin \phi}{c \cos \phi - p^{ref} \sin \phi} \right)^m \quad (4.7)$$

Where, E_{50}^{ref} is reference stiffness modulus corresponding to the reference pressure, p^{ref} and m is power for stress-level dependency of stiffness.

Tangent stiffness for primary oedometer loading, E_{oed}^{ref} , is used to model stiffness for compression behaviour. Determination of E_{oed}^{ref} is shown in Figure 4.5. E_{oed}^{ref} is calculated by following equation:

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cos \phi - \sigma'_3 \sin \phi}{c \cos \phi - p^{ref} \sin \phi} \right)^m \quad (4.8)$$

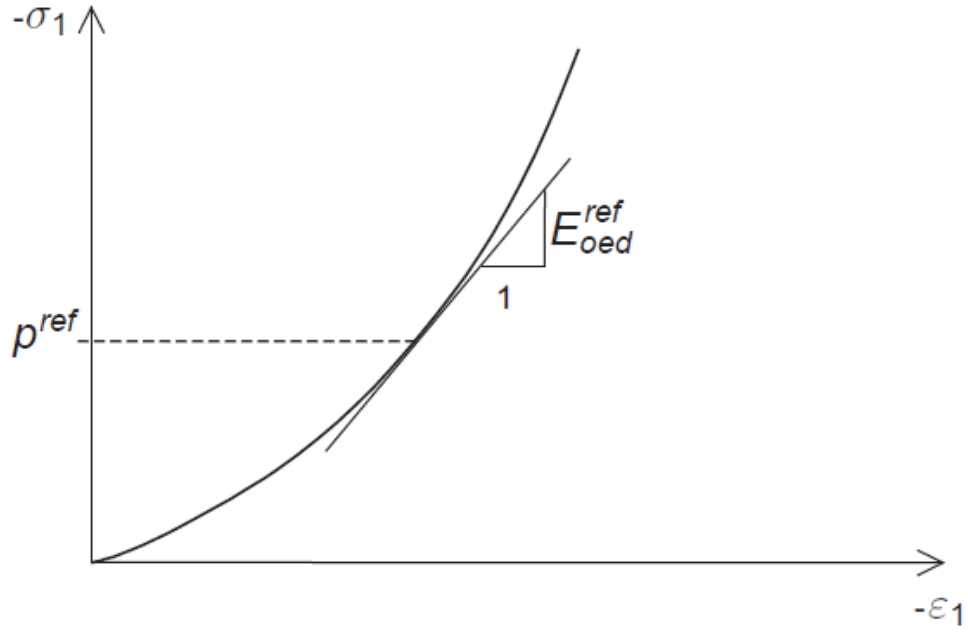


Figure 4.5 : Definition of Tangent stiffness for primary oedometer loading, E_{oed}^{ref} (Plaxis Material Models Manual, 2012).

E_{oed}^{ref} Can be derived from eodometer test results where E_{oed}^{ref} is the tangent stiffness modulus as indicated in Fig 4.5, at reference pressure $p^{ref} = 100kN/m^2$. Unloading/reloading stiffness, E_{ur}^{ref} is used for unloading and reloading stress paths and calculated for a reference stress as:

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cos\phi - \sigma'_3 \sin\phi}{c \cos\phi - p^{ref} \sin\phi} \right)^m \quad (4.9)$$

It is suggested in Plaxis Material Models Manual (2012) that E_{ur}^{ref} can be estimated as:

$$E_{ur}^{ref} = 3E_{50}^{ref} \quad (4.10)$$

Both E_{ur} and E_{50} can be extracted from standard drained triaxial test by plotting the deviator stress q versus the axial strain ϵ_1 .

4.4.4.1 Correlation of stiffness modulus

Vermeer P.A. et al (1985), proposed a correlation between oedometer stiffness E_{oed} , Plasticity index I_p and effective vertical stress σ'_1 , in case if deviator stress q versus the axial strain ϵ_1 , is not available.

$$E_{oed} \approx \frac{500}{I_p} * \sigma'_1 \quad (4.11)$$

As it was mentioned before, PLAXIS hardening soil model uses a reference stress of $p^{ref} = 100kN/m^2$. It can be substituted in previous correlation for vertical effective stress. Then the correlation above will be changed into:

$$E_{oed}^{ref} \approx \frac{50000}{I_p} (kPa) \quad (4.12)$$

The primary loading stiffness of clay can be measured precisely by oedometer test. For normally consolidated clays an approximation relation can be obtained when the reference oedometer stiffness is about half the reference triaxial stiffness at 50% of strength.

$$E_{oed}^{ref} = 0.5 * E_{50}^{ref} \quad (4.13)$$

There is also another correlation proposed by Vermeer, which is shown below.

$$E_{oed}^{ref} = \frac{500kPa}{w_L - 0.1} \quad (4.14)$$

4.4.5 Power law m

The power m is used to set the amount of stress dependency. This parameter can be obtained by plotting the exponent m against plasticity index at very small strains which is shown in Figure 4.6. This method was proposed by Viggiani et al (1997), and Hicher (1996). They also proposed another correlation by using liquid limit instead of plasticity index. Generally for sandy soil this value is between 0.5 and 1. For clayey soil this value varies between 0.9 and 1.

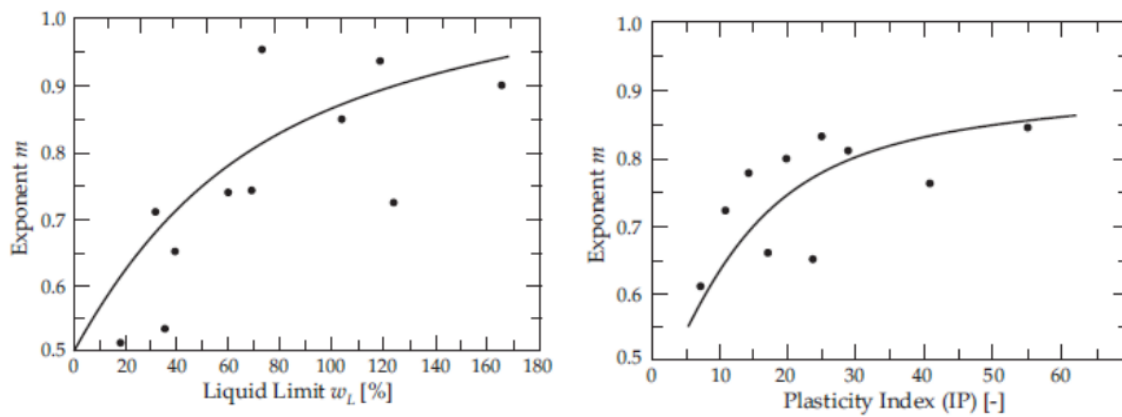


Figure 4.6 : Power law exponent m as function of plasticity index and liquid limit (Benz T.2007a).

4.4.6 Poisson's ratio for unloading/reloading, ν_{ur}

ν_{ur} is defined as Poisson's ratio for unloading or reloading. The value of Poisson's ratio in this thesis is 0.2.

4.4.7 Small strain stiffness parameters G_0^{ref} and $\gamma_{0.7}$

The formulation of small strain stiffness is derived from the S shape curves obtained from plotting the shear modulus G being as a logarithmic function of shear strain γ . These curves are shown in Figure 4.7. The main characteristics of this S shaped plot are first the small strain shear modulus G_0 and then the shear strain at which the shear modulus is 70% of G_0 which is shown by $\gamma_{0.7}$.

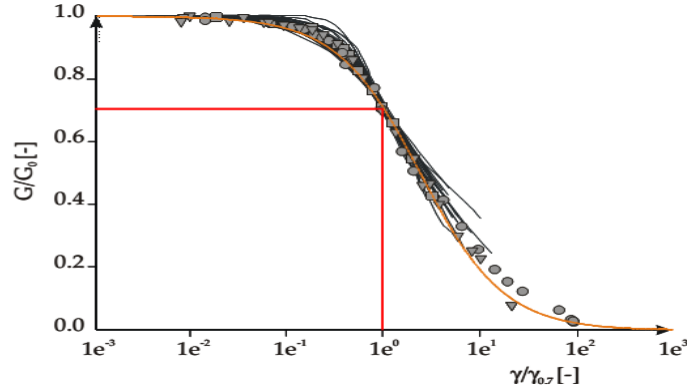


Figure 4.7 : S shaped curves for reduction of shear modulus against shear strain (Plaxis 2012 manual).

To define these new parameters properly we can say G_0^{ref} is the reference shear modulus at very small strain ($\varepsilon < 10^{-6}$). $\gamma_{0.7}$ is the shear strain at which the secant shear modulus (G_s) is reduced to 70% of G_0^{ref} . The parameters of small strain stiffness can be derived from the classic triaxial laboratory tests which is shown in Figure 4.8.

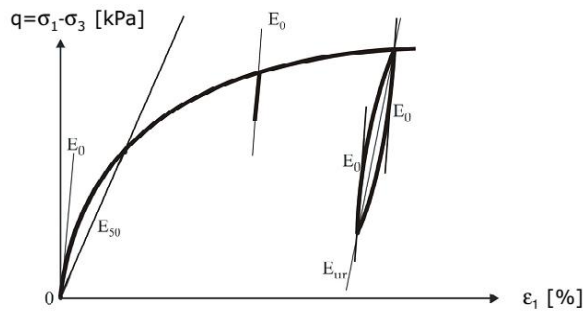


Figure 4.8 : Stiffness parameters from triaxial test (brinkgreve R.B.J 2004).

Figure 4.8 shows the derivation of stiffness parameters from the well-known triaxial test. There are different relations to calculate the stiffness parameters such as:

$$E_0 = 2(1 + \nu_{ur})G_0 \quad (4.15)$$

And then by substituting the reference values, the formula changes to:

$$G_0^{ref} = \frac{E_0^{ref}}{2(1+\nu_{ur})} \quad (4.16)$$

The relationship between E_0 and E_{ur} can be estimated from the chart by Alpan (1970), assuming:

$$\frac{E_{dynamic}}{E_{static}} \cong \frac{E_0}{E_{ur}} \quad (4.17)$$

Alpan proposes a chart for this issue which is shown in Figure 4.9:

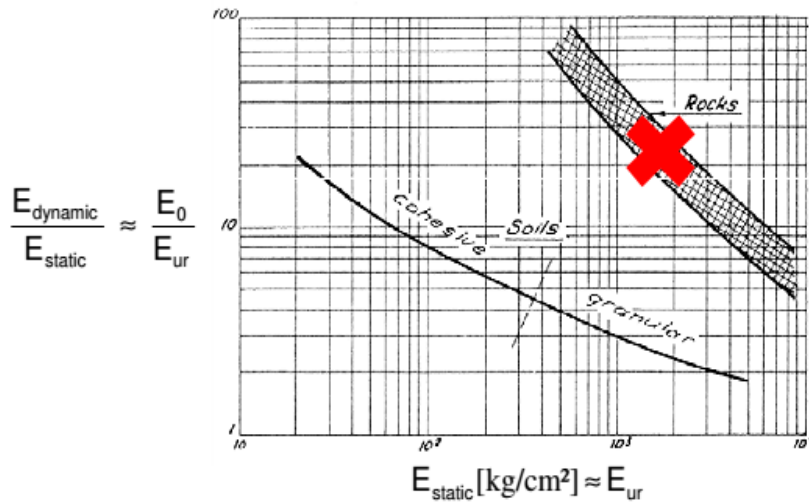


Figure 4.9 : Chart proposed by I. Alpan (1970).

For estimating the values of $\gamma_{0.7}$ some charts and correlations have been proposed by different researchers. For example the correlation between threshold shear strain and plasticity index may be used to determine the $\gamma_{0.7}$. Vucetic and Dobry (1991), propose a chart which is shown in Figure 4.10. Depending the value of plasticity index, the value of shear strain can be read at $\frac{G}{G_0} = 0.7$. This curves are mainly used for clay soils. Another chart proposed by Seed and Idriss (1970), for sandy soils is available also and is shown in Figure 4.11.

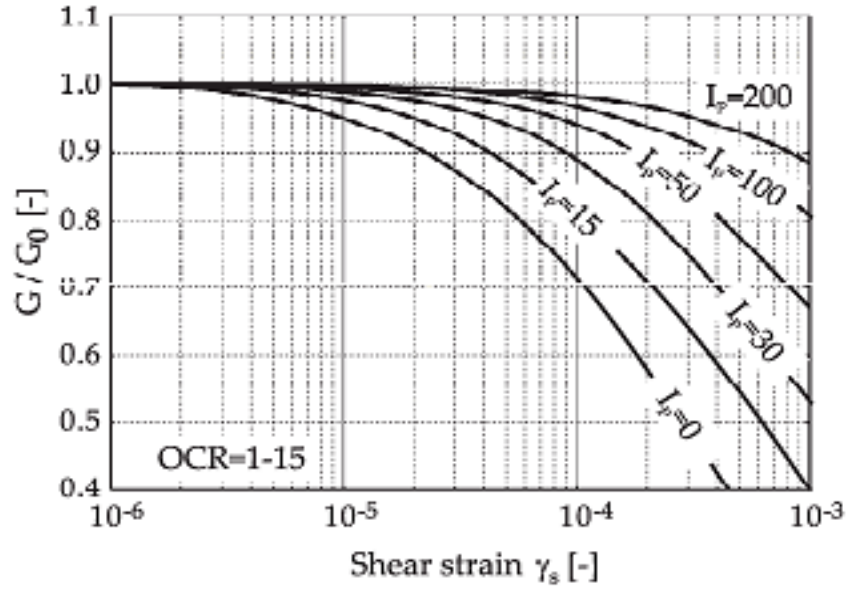


Figure 4.10 : Influence of plasticity index on stiffness reduction (Vucetic and Dobry, 1991).

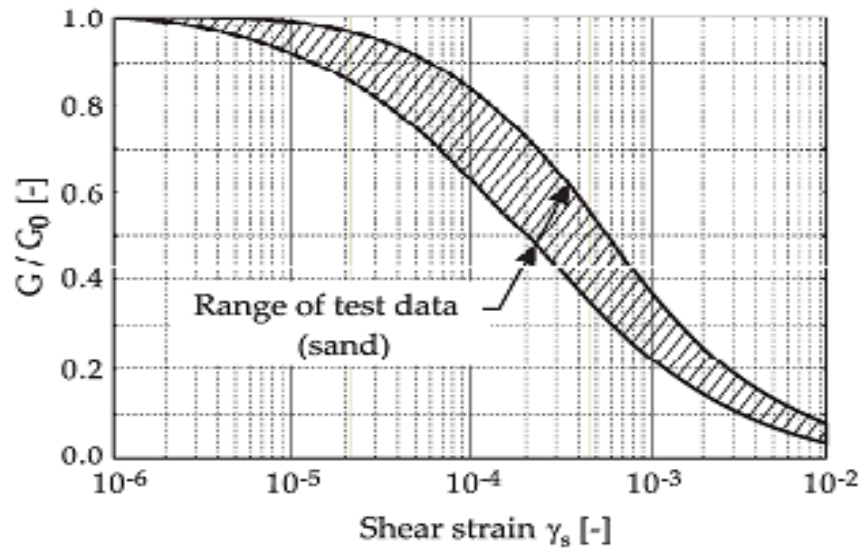


Figure 4.11 : Stiffness reduction curve (Seed and Idriss, 1970).

There are another ways for calculating G_0 . Atkinson J. (2007), suggests that the simplest way to calculate the shear modulus at small strain is to use the shear wave velocity and the correlation below:

$$G_0 = \frac{\gamma}{g} * V_s^2 \quad (4.18)$$

γ is the unit weight of soil g indicates gravity which is equal to 9.81 m/s^2 . Then using the value obtained from correlation above and correlation (4.15), E_0 is obtained. Figure 4.12 shows another way of estimating the value of shear modulus at

small strains is using the correlation of cone resistance value which has been derived from the CPT test and use the chart proposed by Robertson and Campanella (1983).

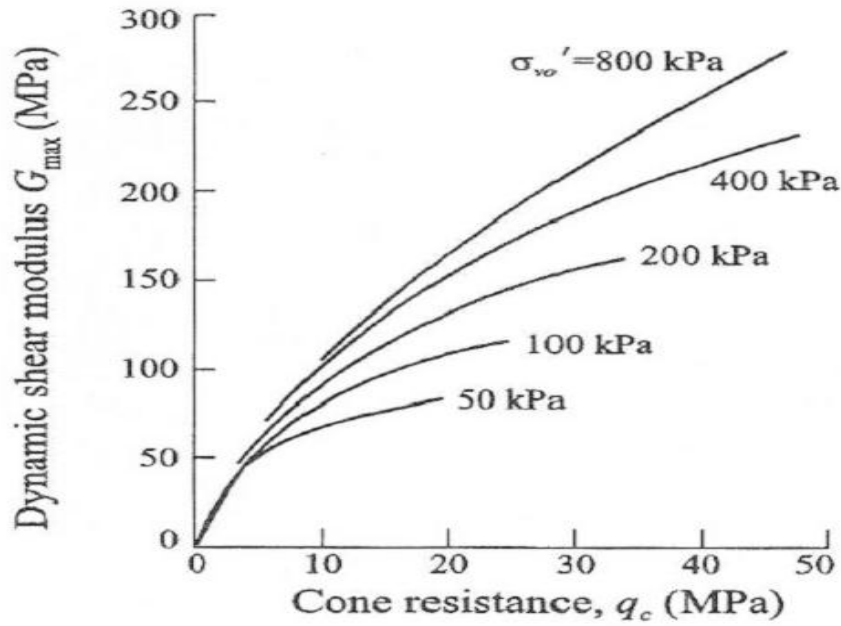


Figure 4.12 : Correlation between cone resistance and dynamic shear modulus(Plaxis manual, 2012).

Based on the works of Mayne (2007), E_{50} can be obtained by using modulus reduction curves. These curves are shown in Figure 4.13.

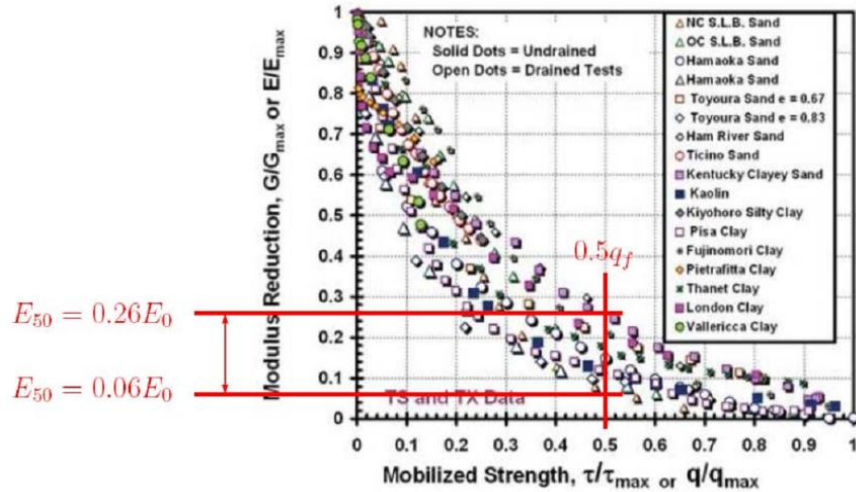


Figure 4.13 : Monotonic modulus reduction curves static torsional and triaxial shear data on clays and sands (Mayne, 2007).

Based on Mayne's works, E_{50} can be obtained by using modulus reduction curves which are indicated above. The values are between 0.06 to 0.26 of E_0 value. Then we can calculate other parameters assuming $E_{oed}^{ref} \cong E_{50}^{ref}$ and $E_{ur}^{ref} \cong 3 * E_{50}^{ref}$ (Plaxis

materials model manual 2012). Figure 4.14 shows the relationship between E_{oed} and E_{50} .

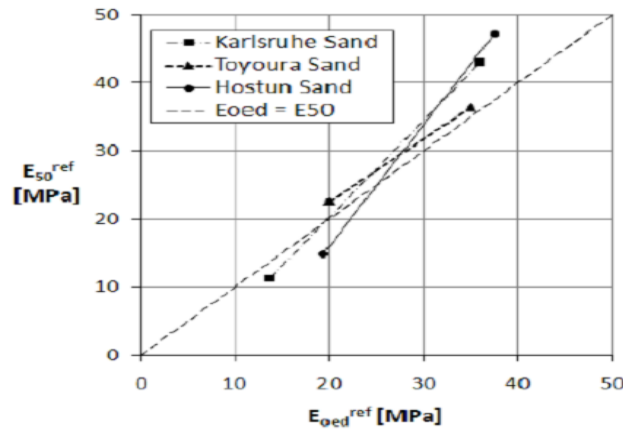


Figure 4.14 : Relationship between E_{oed} and E_{50} (Kempfert, 2006).

4.4.8 Correlation between SPT N value and Shear wave velocity

As it was mentioned above, to calculate $G_{0 \text{ or } max}$, correlation 4.18 can be used. But first the value of shear wave velocity have to be obtained. Shear wave velocity (V_s) is a valuable indicator of the dynamic properties of soil. G_{max} And V_s are primarily functions of soil density, void ratio, and effective stress, with secondary influences including soil type, age, depositional environment, cementation, and stress history (Hardin and Drnevich 1972a, b). The best way to obtain this value is conducting field tests and plot shear wave logs. There are also some correlations for estimating shear wave velocity. These correlations are based on SPT value which have been derived by different researchers around the world. Table 4.9 represents the Effect of increase of various factors on G_{max} And V_s [(Dobry and Vucetic (1987) as reported by EPRI (1991))].

Table 4.9 : Effect of increase of various factors on G_{max} And V_s [Dobry and Vucetic [1987] as reported by EPRI [1991].

Increasing Factor or Parameter	Influence on G_{max} And V_s
Confining pressure $\uparrow$	Increase with σ'_{v0} $\uparrow$
Void Ratio $\uparrow$	Decreases with increased void ratio $\downarrow$
Geologic Age $\uparrow$	Increase with Geologic Age $\uparrow$
Cementation $\uparrow$	Increase with Cementation $\uparrow$
OCR $\uparrow$	Increase with OCR $\uparrow$
Frequency of Cyclic Loading $\uparrow$	Increase with strain rate $\uparrow$

The Caltrans Seismic Design Criteria classifies sites based on V_s of the top 30 m of the soil profile (V_{s30}). Sites are divided into the six categories (Soil Profile Types A through F) presented in Table 4.10. For site classification, V_{s30} is calculated as the time for a shear wave to travel from a depth of 30 m to the ground surface, not the arithmetic average of V_s to a depth of 30 m. As shown in Equation (4.19), the time-averaged V_{s30} is calculated as 30 m divided by the sum of the travel times for shear waves to travel through each layer. (Dobry et al. 2000). The travel time for each layer is calculated as the layer thickness (d) divided by V_s . Table 4.10 represents the shear wave velocity values for different types of soils.

$$V_{s30} = \frac{30}{\sum \frac{d}{V_s}} \quad (4.19)$$

Table 4.10 : Soil Profile Types A through F(Caltrans/NEHRP, 2006).

Site Class	Soil Profile Name	V_{s30}
A	Hard Rock	$> 1500 \text{ m/s}$
B	Rock	$760 \sim 1500 \text{ m/s}$
C	Very dense soil and soft rock	$360 \sim 760 \text{ m/s}$
D	Stiff Soil	$180 \sim 360 \text{ m/s}$
E	Soft Soil	$< 180 \text{ m/s}$
F	Soils requiring site-Specific evaluation	...

Higher cost and lacked in respective expertise made shear wave velocity unable to be measured routinely in industry and engineering practice. By making use of the

abundant standard penetration tests (SPT) data in site investigation program, statistical means to obtain the relationship between shear wave velocity (V_s) and standard penetration resistance (SPT-N) is greatly supported by researchers and industries. Most of the researchers including Anbazhagan et al. (2012) utilized uncorrected SPT-N in establishing relationship while Sitharam and Anbazhagan (2008) developed using corrected values and obtained great correlation pattern. Nevertheless, Uma Maheswari et al. (2010) reported that the corrected and uncorrected N values predicted V_s with equal accuracy. Numerous relations between SPT blow count, N , and shear wave velocity, V_s , exist in the literature. The most common functional form for the relations proposed in the literature is $V_s = A \cdot N^B$, where the constants A and B are determined by statistical regression of a data set. The N -values are typically not corrected for overburden, but sometimes are corrected for hammer energy, rod length, and sampler inside diameter, in which case N would be replaced by N_{60} . V_s Values are typically not corrected for overburden (Wair et al, 2012). The relationships used in this thesis, are listed in Table 4.11. It has to be mentioned that these selected relationships were the most accurate ones based on the type of soils and the region of study.

Table 4.11 : Correlations for estimating V_s

Researcher's Name	Correlations based on SPTN
Fujiwara (1972)	$92.1 \times N^{0.337}$
Imai et al. (1976)	$89.8 \times N^{0.341}$
Ohta and Goto (1978)	$85.35 \times N^{0.348}$
Imai and Tonouchi (1982)	$97 \times N^{0.314}$
Iyisan (1996)	$51.5 \times N^{0.516}$
Kiku et al (2001)	$68.3 \times N^{0.292}$

4.4.9 Brinkgreve and Engin formulas for estimation of soil parameters

There are a lot of useful correlations based on soil characteristics such as relative density for sandy soils and plasticity index for clayey soils. Brinkgreve and Engin (2007), proposed formulas for estimation of sandy soils properties based on their relative density value (D_r). The main goal of the empirical formulas is to give a reasonable first order approximation of soil behavior in FEM calculations, covering a wide range of sands. Even if the relative density is not precisely known, it could be

estimated on the basis of very preliminary soil data. The formulas have been derived by regression analysis on a collection of soil data (general soil data, triaxial test data, oedometer test data, etc), which was a study done by Jeffries and Been (2006) and others. The relative density value is used as a percentage value in the formulas. The proposed formulas are as follows:

$$\gamma_{unsat} = 15 + 0.4RD/100 \text{ (kN}_m^3\text{)} \quad (4.20)$$

$$\gamma_{sat} = 19 + 1.6RD/100 \text{ (kN}_m^3\text{)} \quad (4.21)$$

$$E_{50}^{ref} = 60000RD/100 \text{ (kN}_m^2\text{)} \quad (4.22)$$

$$E_{oed}^{ref} = 60000RD/100 \text{ (kN}_m^2\text{)} \quad (4.23)$$

$$E_{ur}^{ref} = 180000RD/100 \text{ (kN}_m^2\text{)} \quad (4.24)$$

$$G_0^{ref} = 60000 + 68000RD/100 \text{ (kN}_m^2\text{)} \quad (4.25)$$

$$m = 0.7 - RD/320 \quad (4.26)$$

$$\gamma_{0.7} = (2 - \frac{RD}{100})10^{-4} \quad (4.27)$$

$$\varphi' = 28 + 12.5RD/100 \text{ (}^\circ\text{)} \quad (4.28)$$

$$\psi = -2 + 12.5RD/100 \text{ (}^\circ\text{)} \quad (4.27)$$

$$R_f = 1 - RD/800 \quad (4.29)$$

4.4.10 Calculation of safety factor in plaxis

Safety analysis in Plaxis uses a so-called phi/c reduction method in which the strength of the soil materials will be reduced step by step with a factor ΣM_{sf} until either failure is reached for a stable value of ΣM_{sf} , or the maximum number of calculation steps is reached. The theory of this type of FOS calculation is that when failure occurs by reducing the strength of soil gradually, the correspondent strength reduction factor can be considered as a factor of safety on soil strength. In this approach the cohesion and the tangent of friction angle are reduced in the same proportion.

$$\Sigma M_{sf} = \frac{\tan \varphi'_{input}}{\tan \varphi'_{reduced}} = \frac{c'_{input}}{c'_{reduced}} \quad (4.30)$$

4.4.11 Fem model

A HS-small under drained condition has been chosen for analyses on different types of soil profiles, because the material in most parts of affected zone are determined to be granular with a low degree of saturation and excess pore pressures will not be generated considerably. The basic mesh elements used in these analysis are 15 noded triangular elements. This type of element provides a fourth order interpolation for displacement and the numerical integration involves 12 Gauss points (stress points). It is a very accurate element that produces high quality stress results. A medium mesh density has been chosen for both Kocaeli and Elazığ cases based on the average element size calculations. The average element size (AES) represents the average side length of the element used and is an average dimension, which is representative of the mesh thickness. Kuhlmeier and Lysmer (1973), suggested that the element size should be no larger than $\lambda/8$, where λ is the wave length with the maximum frequency of interest. The table below shows the maximum element size of each layer. The AES in this study is around 4m which is quite enough. It has to be mentioned that the maximum element size of Elazığ case is bigger than Kocaeli case, so the medium mesh density and also coarse mesh density will be enough for that case. Maximum element sizes for Kocaeli case are given in Table 4.12.

Table 4.12 : Maximum element size of each layer of Kocaeli case.

Layer	Thickness (m)	Vs (m/s)	Natural Vibration Period	Frequency	Wave length	Maximum element size
Silty Sand 1	1.4	150	0.037	26.79	5.6	0.7
Silty Sand 2	4	220	0.073	13.75	16	2
Silty Sand 3	9	250	0.144	6.94	36	4.5
Average Sand	4	260	0.062	16.25	16	2
Rock	47	1000	0.235	4.02	195	25

4.4.12 Model boundaries

Model boundaries are introduced to limit the extent of the finite element model and calculation time. It has to be validated whether the outcome of the finite element model is not influenced by the particular choice of the model boundaries. In this thesis, validation has been done by redoing the analysis with model boundaries taken further away from the main modelling object and comparing the results. Changes in stress and strain near the model boundaries, have to be checked if they are relatively small, otherwise the tests must be done again. In the case of a dynamic analysis, users should check that there is no spurious reflection at the model boundaries. In dynamic analysis the geometry of model should be modified especially when the behavior of buildings are being studied. (Brinkgreve et al., 2013).

4.4.13 Absorbent boundaries

If the goal of analysis is to conduct a dynamic type, it is necessary to take additional measures to perform a better analyses. In order to a dynamic analysis, special boundary conditions have to be defined in order to justify the fact that soil is a semi-infinite medium in reality. These boundaries are utilized reduce the infinite domain (which is the aim for analysis), to a finite domain. These boundaries are used to prevent waves from reflecting back from the model boundaries. Otherwise reflected waves will cause disturbance which leads to wrong results. To avoid these reflections, absorbent boundaries were applied (Brinkgreve et al., 2011).

4.4.14 Initial condition

To have an accurate prediction, it is necessary to initialize the stress in the model as much as possible in correspondence with the situation in reality. The initial situation in the model may involve total or effective stress components, pore water pressures, pre-consolidation stress, void ratio and other state parameters, depending on the constitutive model(s) being used. In Plaxis there are two options for determination of initial condition. The K_0 procedure and gravity loading procedure. In cases like slope analysis the gravity loading procedure is used (Plaxis manual 2012). Most soil constitutive models involve at least some sort of stress-dependency. Moreover, the initial stress state directly influences the forces in soil retaining structures. In the case of time-dependent behavior, the initial state may have influence on the settlement rate. In an effective stress analysis, it is essential to create a realistic distribution of

initial pore water pressures. Generated pore pressures should be validated against measured pore pressure distributions in the field.

4.4.15 Dynamic input

4.4.15.1 Newmark alpha and beta

To determine the numerical time integration according to the implicit Newmark scheme, the Newmark alpha and beta parameters are used in the manual settings of the iterative procedure group. These parameters must satisfy the following condition:

$$\text{Newmark } \alpha \geq 0.25(0.5 + \beta)^2 \quad (4.31)$$

$$\text{Newmark } \beta \geq 0.5 \quad (4.32)$$

In this thesis the default values of $\beta = 0.5$ and $\alpha = 0.25$ are used.

4.4.15.2 Boundary C1 and boundary C2

These boundry multipliers can simulate effects of a damper which helps to absorb the stress on the boundries. A damper is used instead of applying fixities in a certain direction. The damper ensures that an increase in stress on the boundary is absorbed without rebounding. The use of absorbent boundaries is based on the method described by Lysmer and Kuhlmeyer (1969). The normal and shear stress components absorbed by a damper in x-direction are as follows:

$$\sigma_n = -C_1 \rho V_p U_x \quad (4.33)$$

$$\tau = -C_2 \rho V_s U_y \quad (4.34)$$

Boundary C1 and C2 are relaxing coefficients used to improve the wave absorption on the absorbent boundaries. C1 corrects the dissipation in the direction normal to the boundary and C2 does it in the tangential direction. If waves are coming in the direction perpendicular to the boundary, there is no need of relaxation ($C1 = C2 = 1$). Since we are dealing within earthquakes in which waves are coming in arbitrary directions, C2 requires adjustment. The standard values suggested by Plaxis are $C1 = 1$ and $C2 = 0.25$. Another values should also be checked to insure better absobtion in the model boundries. In this thesis, C2 parameter have been checked with the values of 0.1, 0.25, 0.3, 0.4, 0.5 and 0.75. The best and more realistic results were obtained using the values between 0.25 and 0.4.

4.4.15.3 Dynamic multipliers

Multipliers are used to activate the dynamic loads while doing dynamic analysis. There are different options for dynamic multipliers, such as defining a harmonic load or to read a dynamic load multiplier from a data file. The file may be in plain ASCII or in SMC (Strong motion CD-ROM) format. Prescribed displacements are used for entering the ground motion data or harmonic load in the analysis. In case of prescribed displacement a consideration has to be made whether the input has to be considered as displacement, velocity or acceleration. An ASCII file can be created with any text editor. There are different sources which provide the SMC files or the text files for numerous ground motion data around the world such as USGS, PEER, European Strong Ground Motion Center data bases or in Turkey the website of Strong Ground Motion of Turkey (kyhdata.deprem.gov.tr), can be used. In this thesis, strong ground motion records of Kocaeli and Elazığ earthquakes are used, which are shown below. Seismo Signal commercial computer software has been used to modify the ground motion records. For Kocaeli case, acceleration, velocity, displacement and spectrum graphs of Yarımca record are used which are shown respectively in Figure 4.15. Acceleration of Kocaeli record is more than 300 gal which is obviously more than acceleration of Elazığ record which is about 65 gal. It has to be mentioned that figures with better quality are presented in appendices chapter (Figs. A.1, A.2, A.3, A.4 and A.5). For Elazığ earthquake acceleration, velocity, displacement and spectrum graphs are shown respectively in Figure 4.16. Properties of record used for Elazığ ground motion is shown in table 4.13.

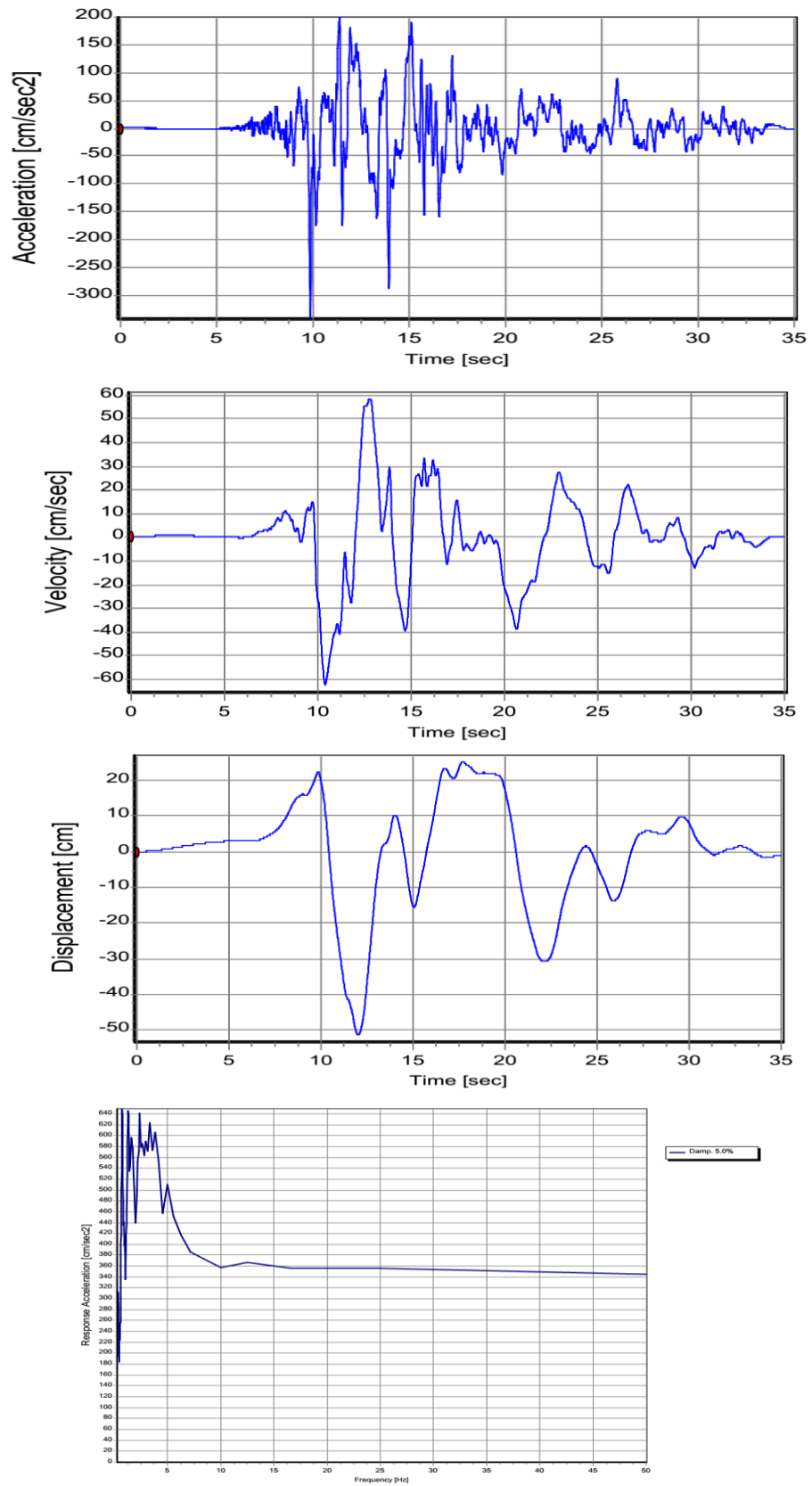


Figure 4.15 : Acceleration, velocity, displacement and spectrum graphs of Kocaeli earthquake..

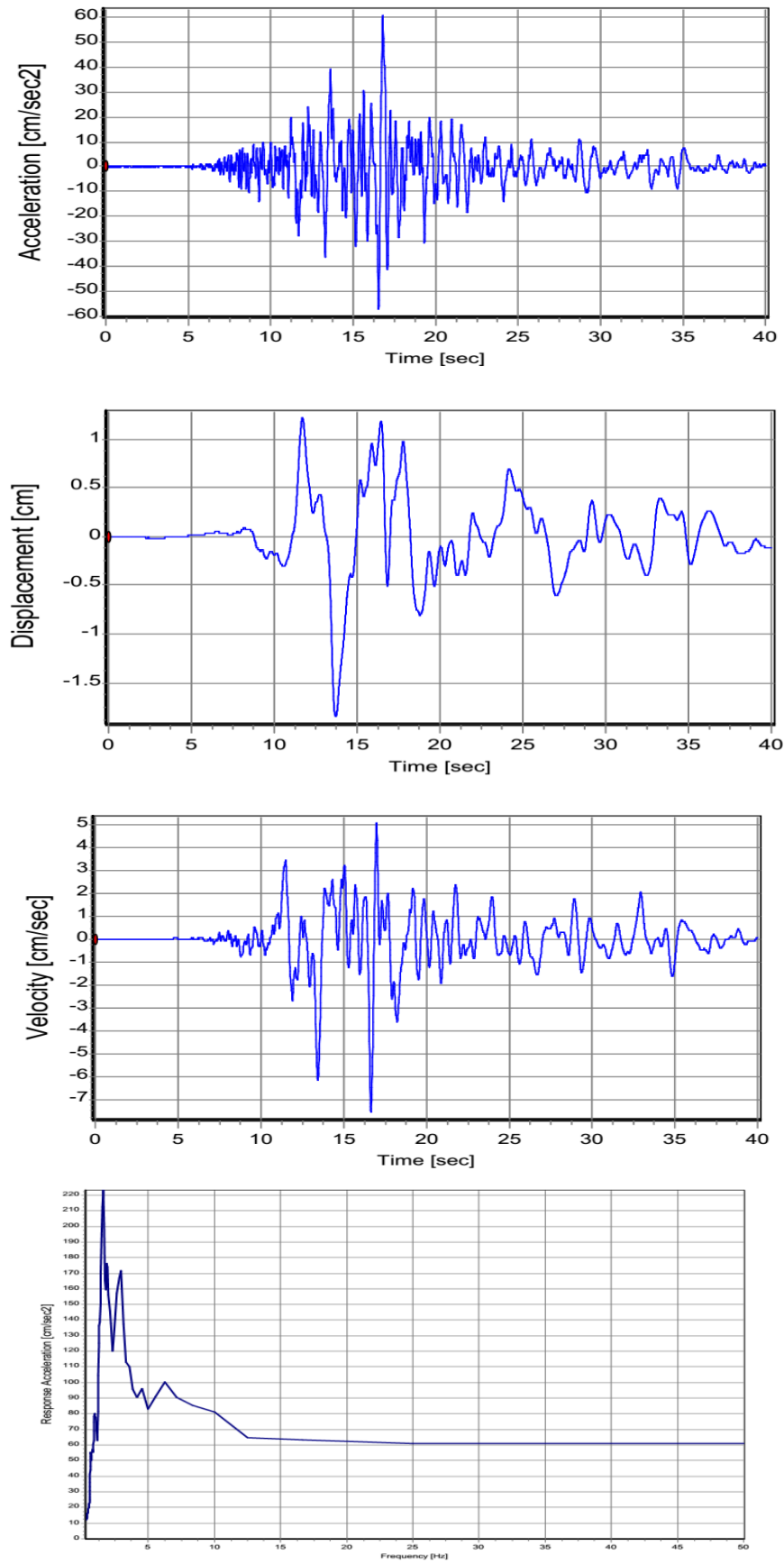


Figure 4.16 : Acceleration, velocity, displacement and spectrum graphs of Elazığ earthquake.

Table 4.13 : Properties of record used for Elazığ ground motion.

Place	ELAZIĞ PALU
Epicenter Coordinates	38.77520N-40.02950E
Earthquake Depth (km)	5
Earthquake Magnitude	5.8 ML
Station Altitude (m)	1000
Raw PGA Values (gal)	(N-S) 62 (E-W) 66.5 (U-D) 30
Station ID	2303
Station Coordinates	38.69577N-39.93190E

4.4.15.4 Material damping

We first have to define material damping. This is described as the decrease of the amplitude of vibration in soil with respect to time. (Das and Ramana 2011). In the dynamic analysis conducted by Plaxis, viscous properties of soil, friction and development of plastic strains, causes material damping to a certain extent (Brinkgreve et al., 2011). In real soil behavior the observed damping appears to be more. Therefore an extra damping is required to simulate the realistic damping effects in soils (Brinkgreve et al., 2011). In time domain analysis material damping includes viscous and hysteretic soil damping and is frequency dependent. The material damping is simulated by means of the well-known Rayleigh formulation. The damping matrix, C , is assumed to be proportional to the mass matrix, M , and to the stiffness matrix, K , through the coefficients α and β according to the following relation:

Rayleigh α parameter is used for determination of the influence of mass in the damping of the system and Rayleigh β parameter is used for determination of the influence of stiffness in the damping of the system.

$$C = \alpha M + \beta K \quad (4.35)$$

The parameters of Rayleigh formulation are being derived as follows (Plaxis 2012 Manual):

$$\alpha = 2\omega_1\omega_2 \frac{\omega_1\xi_2 - \omega_2\xi_1}{\omega_1^2 - \omega_2^2} \quad (4.36)$$

$$\beta = 2 \frac{\omega_1\xi_2 - \omega_2\xi_1}{\omega_1^2 - \omega_2^2} \quad (4.37)$$

If we accept that $\xi_1 = \xi_2$, then the formulas above changes to:

$$\alpha = 2\xi \frac{\omega_1\omega_2}{\omega_1 + \omega_2} \quad (4.38)$$

$$\beta = 2\xi \frac{1}{\omega_1 + \omega_2} \quad (4.39)$$

Where ξ represents damping, ω_1 is the first angular frequency and, ω_2 is the second angular frequency. ω_1 And ω_2 can be calculated as follows:

$$\omega_1 = 2\pi f_1 \text{ and } \omega_2 = 2\pi f_2 \quad (4.40)$$

Where f_1 and f_2 are first and second natural vibration frequency respectively. In order to calculate f_1 and f_2 the natural period of ground should be calculated as follows:

$$T = \frac{4H}{V_s} \quad (4.41)$$

$$f = \frac{1}{T} \quad (4.42)$$

Where H is the thickness of soil layers and V_s is the shear wave velocity of soil layers. A 5% value for damping has been chosen for the analyses in this study.

4.4.15.5 Dynamic sub steps

For each additional time step, Plaxis computes the number of substeps necessary to reach the estimated end time with the sufficient accuracy on the basis of the generated mesh and computed δt critical. These numbers of additional steps and dynamic sub steps are indeed directly related to the accuracy of the computations. A time step taken by Plaxis in each computation step can be represented by the following equation

$$\delta_t = \frac{\Delta_t}{n.m} \quad (4.43)$$

Where Δt is the duration of dynamic loading (time interval of the earthquake), n is the number of dynamic sub steps and m is the number of additional steps. In general, the time step used in the program should be equal to the time step used in the input signal. In our cases which are Kocaeli and Elazığ earthquakes, the time step of input signal is 0.005 for both of accelerograms so for Elazığ case the values has been chosen as follows:

$$\Delta_t = 40 \text{ s}$$

$$m = 100$$

$$n = 80$$

For Kocaeli case the values has been chosen as follows:

$$\Delta_t = 35 \text{ s}$$

$$m = 100$$

$$n = 70$$

4.5 Review of Literature

4.5.1 Introduction

In this part of the thesis, the importance of studying the behavior of foundations and buildings near or on slopes and also the benefits of using the Finite Element Method in analysis of slopes will be discussed briefly. Furthermore, previous studies about different landslides and also slope stability analysis are presented.

4.5.2 Finite element analysis and its application in slope analysis

The majority of slope stability analyses performed in practice still use traditional limit equilibrium approaches involving methods of slices that have remained essentially unchanged for a long time. The finite element method represents a powerful alternative approach for slope stability analysis which is accurate, versatile and requires fewer a priori assumptions, especially, regarding the failure mechanism. Slope failure in the finite element model occurs naturally through the zones in which the shear strength of the soil is insufficient to resist the shear stresses.

Griffiths, D. V. & Lane, P. A. (1999) described several examples of finite element slope stability analysis with comparison against other solution methods, including the influence of a free surface on slope and dam stability. It was argued that the finite element method of slope stability analysis is a more powerful alternative to traditional limit equilibrium methods and its widespread use should now be standard in geotechnical practice.

They developed a finite element code and used six different parameters to model the slope in program. Used parameters were: friction angle, cohesion, dilation angle, Young's modulus, Poisson's ratio and unit weight. Programs used in this paper are based closely on Program 6.2 by Smith & Griffiths (1998), the main difference being the ability to model more general geometries and soil property variations, including variable water levels and pore pressures. Further graphical output capabilities have been added. The programs are for two-dimensional plane strain analysis of elastic-perfectly plastic soils with a Mohr-Coulomb failure criterion utilizing eight-node quadrilateral elements with reduced integration (four Gauss points per element) in the gravity loads generation, the stiffness matrix generation and the stress redistribution phases of the algorithm. The soil is initially assumed to be elastic and the model generates normal and shear stresses at all Gauss points within the mesh. These stresses are then compared with the Mohr-Coulomb failure criterion. If the stresses at a particular Gauss point lie within the Mohr-Coulomb failure envelope, then that location is assumed to remain elastic. If the stresses lie on or outside the failure envelope, then that location is assumed to be yielding. Yielding stresses are redistributed throughout the mesh utilizing the visco-plastic algorithm (Perzyna, 1966; Zienkiewicz & Corneau, 1974). Overall shear failure occurs when a sufficient number of Gauss points have yielded to allow a mechanism to develop. Slope stability represents an area of geotechnical analysis in which a nonlinear FE approach offers real benefits over existing methods. Griffiths, D. V. & Lane, P. A. (1999) showed, slope stability analysis by elastoplastic finite elements is accurate, robust and simple enough for routine use by practising engineers. Perception of the FE method as complex and potentially misleading is unwarranted and ignores the real possibility that misleading results can be obtained with conventional 'slip circle' approaches. The graphical capabilities of FE programs also allow better understanding of the mechanisms of failure, simplifying the output from reams of paper to manageable graphs and plots of displacements.

Duncan did a review of FE analysis of slopes concentrated mainly on deformation rather than stability analysis of slopes; however, attention was drawn to some important early papers in which elastoplastic soil models were used to assess stability. Smith & Hobbs (1974) reported results of $\phi_u = 0$ slopes and obtained reasonable agreement with Taylor's (1937) charts. Zienkiewicz et al. (1975) considered a c', ϕ' slope and obtained good agreement with slip circle solutions. Griffiths (1980) extended this work to show reliable slope stability results over a wide range of soil properties and geometries as compared with charts of Bishop & Morgenstern (1960). Subsequent use of the FE method in slope stability analysis has added further confidence in the method (e.g. Griffiths, 1989; Potts et al., 1990; Matsui & San, 1992). Duncan mentioned the potential for improved graphical results and reporting utilizing FE, but cautions against artificial accuracy being assumed when the input parameters themselves were so variable. Wong (1984) gave a useful summary of potential sources of error in the FE modelling of slope stability, although recent results, including those presented in this paper, indicate that better accuracy is now possible.

As it was mentioned before the Finite Element method is a very useful tool to analyse slopes and landslides. A landslide occurs in a slope near Dahr Elbaidar in Lebanon during roads excavation works. Ali Fawaz, Elias Farah and Fadi Hagechade (2014), analyzed the stability of slope based on the numerical simulation using Plaxis software. The mechanical parameters of soil layers constituting the slope were evaluated from laboratory results and numerical simulations of in-situ pressuremeter tests. The study of the slope consists to determine the failure surface and the corresponding safety factor. This coefficient is calculated taking into consideration the influence of factors that contribute slope instability and following the use of several methods of reinforcement to strengthen the slope.

In order to conduct the analysis and get the soil data, drilling boreholes in different types of soil and executing in-situ pressuremeter tests at different depths were done and soil samples were also taken from these boreholes to be tested in the laboratory. The experimental pressuremeter tests were numerically simulated using Plaxis software. Once the soil parameters were determined, the slope was modeled with Plaxis. They started the modeling from the initial state of the slope before any excavation and analyzed the safety factor and the surface of failure corresponding to

this state. An excavation performed during the construction of a highway was also modeled and generally they focused on the parameters which have probably caused the landslide. They also tried some remedies for preventing landslid, like piles and nails. After performing analyses on slope the results showed that the decrease of the mechanical parameters of the soil due to the increase in water content was the main cause of the slide. The homogeneity within the layers leads to a circular failure mode which turns to plane in case of the presence of a thin layer of low mechanical characteristics. The drainage was an effective way to stabilize the slope. When the slope is fully drained, the stability is ensured even without any strengthening methods. The partial drain needs to be accompanied by another means of reinforcement such as piles or nails. The distribution of the piles or nails along the slope was more efficient to ensure stability than the use of a high inertia member disposed in one place of this slope.

In another case S.Y. Liu , L.T. Shao and H.J. Li (2015) studied the factors of safety and critical slip surfaces obtained by the limit equilibrium method (LEM) and two finite element methods (the enhanced limit strength method (ELSM) and strength reduction method (SRM)) and compared them. They did several representative two-dimensional slope examples and analysed them. Using the associated flow rule, the results showed that the two finite element methods were generally in good agreement and that the LEM yielded a slightly lower factor of safety than the two finite element methods did. Moreover, a key condition regarding the stress field is shown to be necessary for ELSM analysis. The SRM and ELSM are the main finite element slope stability methods currently employed. Comparisons between the LEM and finite element analyses of slope stability illustrate the advantages and limitations of these methods for practical engineering problems. The SRM was used for slope stability analysis as early as 1975 by Zienkiewicz et al. This method was later termed the “shear strength reduction technique” by Matusi and San (1992), but the name is now typically shortened to the “strength reduction method”. During the last ten years, many researchers have applied the SRM to analyse slope stability problems and have compared the SRM and LEM. The primary advantage of the SRM is that the critical slip surface is found automatically from the shear strain, which increases as the shear strength decreases. However, the SRM suffers from the important limitation of being unable to locate other slip surfaces (i.e., local minima).

They compared the factors of safety resulted from these analysis. In slope stability analysis, the FOS describes the structural capacity of an embankment or slope, either natural or excavated, beyond the expected or actual loads. In this work, comparisons are performed to determine the correlation among the factors of safety of these three methods.

The FOS and the critical slip surfaces obtained by the LEM and two finite element methods (ELSM and SRM) were compared. Several representative two-dimensional slope examples are analyzed, and the results obtained using the three methods are compared. Under the assumption of the associated flow rule, the critical slip surfaces from the two finite element methods and the limit equilibrium method are generally in good agreement; the FOS obtained from the limit equilibrium method is slightly less than those from the two finite element methods. Moreover, in the SRM, a number of separate finite element analyses must be performed to obtain the FOS and critical slip surface; thus, this is the most time-consuming solution. When a slope model is in an elastic state, it was proven that the directions of the shear stresses at any point along the slip surface must follow the same trend towards the slide for the application of the ELSM. Increasing the Poisson's ratio was shown to be an effective way to satisfy this condition. In the application of the SRM, once a slope reaches the limit equilibrium state by the strength reduction analysis, in which the associated flow rule is applied, the ELSM is shown to be useful in determining the critical slip surface as a technical measure. The critical slip surface determined by the ELSM is located in the centre of the band of equivalent plastic strain. The search technique combining PSO with the pattern search technique to locate the slip surface of the slope was shown to be feasible and reliable in the stability analysis of these examples. This technique could also be applied in the LEM.

The presence of water table is one of the most important issues which affects the stability of slopes. Especially in cases which earthquake plays the role as a trigger. Numerous researchers have studied this issue, for example L. Lu, Z.J. Wang, M.L. Song, K. Arai (2015) have evaluate slope stability during seismic loading. They have defined Seismic failure by means of deformations. They assumed if cumulative plastic deformation exceeds a critical value of deformation determined by static slope stability analysis, then the seismic failure is happened. The numerical procedure proposed by them relies on finite-element analysis of dynamic stress and deformation in slopes to assess their stability during earthquakes considering the effect of ground

water level. They applied this procedure to a hypothetical slope and then to an actual slope with high ground water levels. Figures 4.17, 4.18, 4.19, 4.20, 4.21, 4.22, 4.23, 4.25 and 4.26 show some part of their study and results. Figure 4.24 shows the profile of collapsed soil.

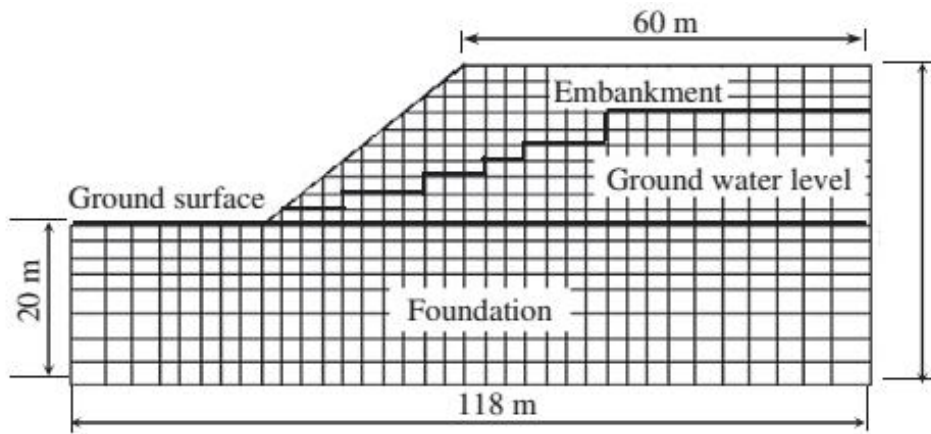


Figure 4.17 : FE meshing for the hypothetical case study (Lu et al, 2015).

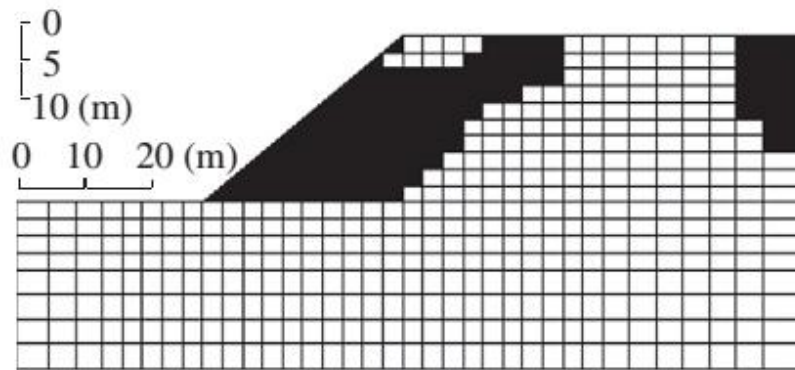


Figure 4.18 : Yield region without ground water table (Lu et al, 2015).

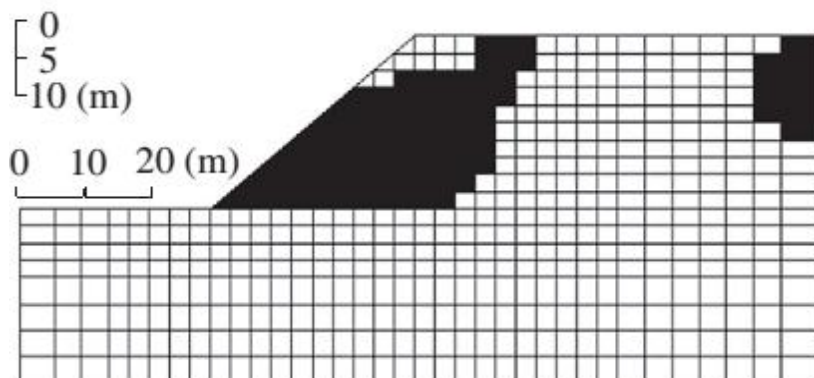


Figure 4.19 : Yield region with ground water table effect (Lu et al, 2015).

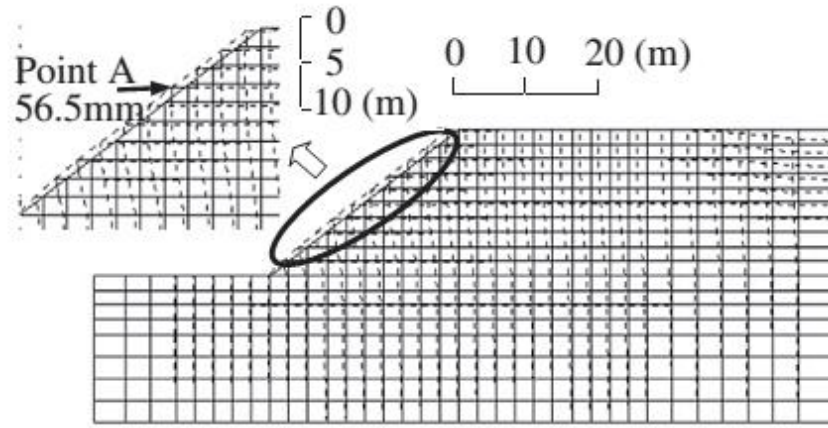


Figure 4.20 : Plastic deformation field at collapse without ground water (Lu et al, 2015).

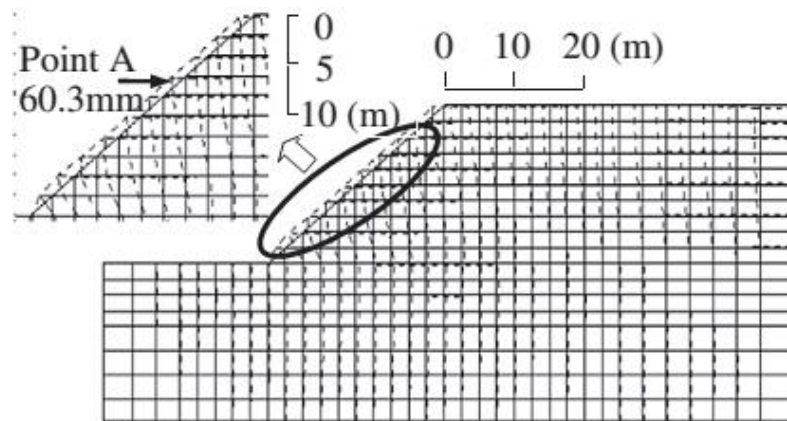


Figure 4.21 : Plastic deformation field at collapse with ground water (Lu et al, 2015).

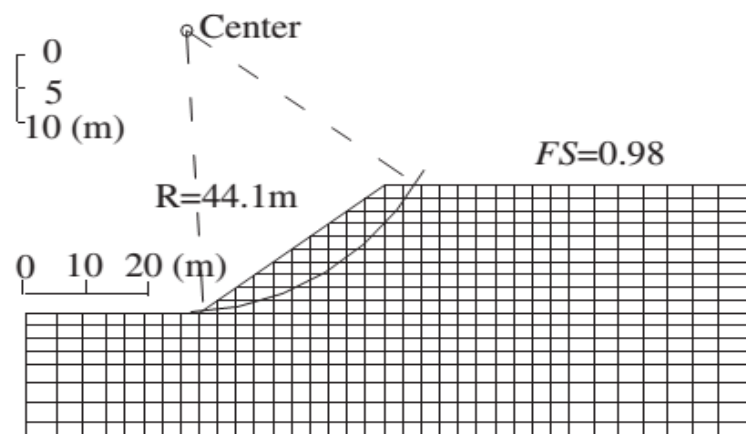


Figure 4.22 : Results from the slip circle method by Bishop (hypothetical case) Case without ground water (Lu et al, 2015).

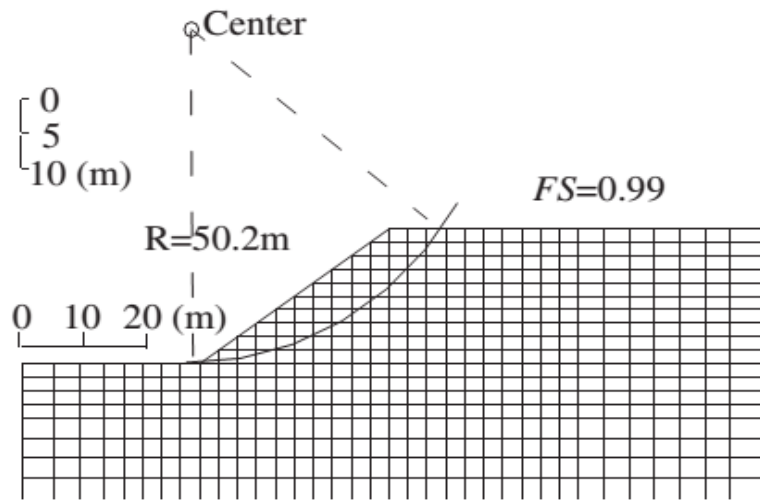


Figure 4.23 : Results from the slip circle method by Bishop (hypothetical case) Case with ground water (Lu et al, 2015).

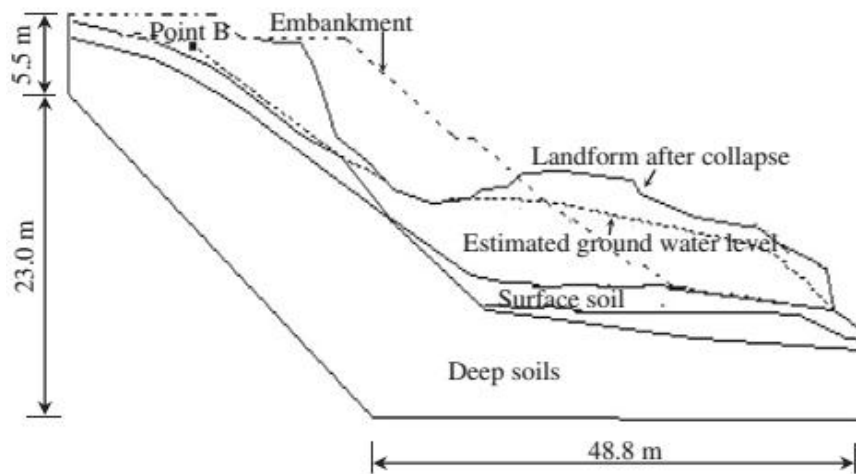


Figure 4.24 : Profile of the slope collapse during the Noto Hanto Earthquake (Actual case) (Lu et al, 2015).

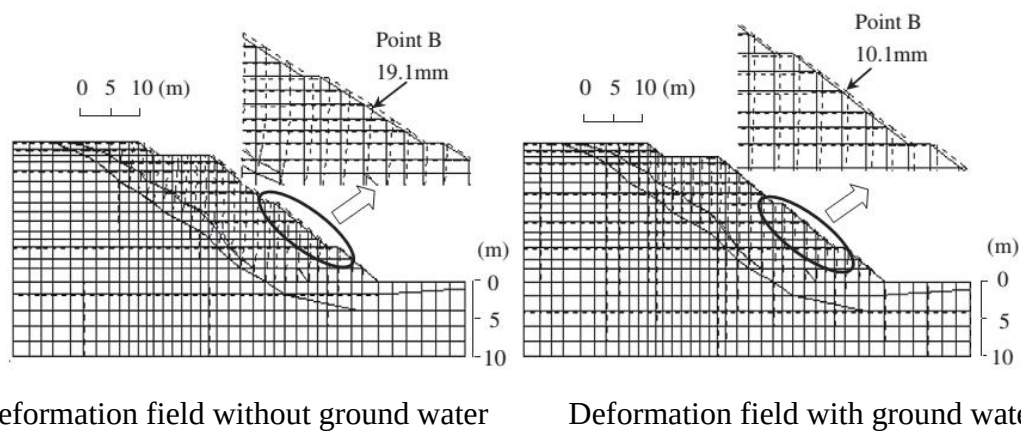


Figure 4.25 : Plastic deformation fields at collapse (actual case) (Lu et al, 2015).

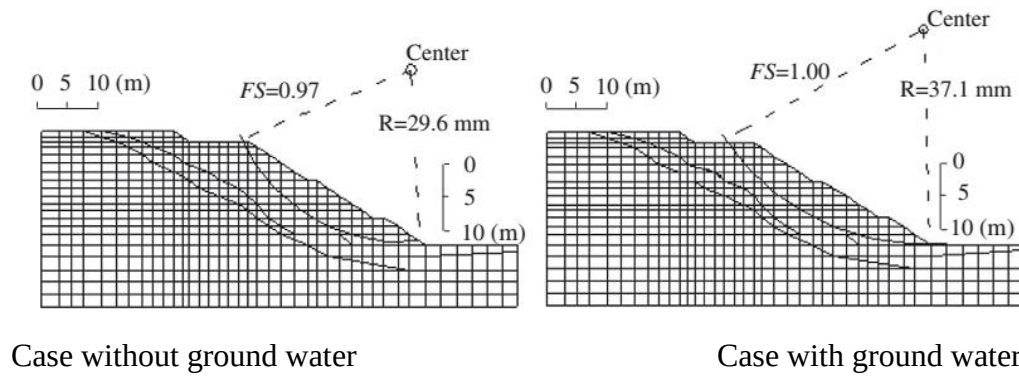


Figure 4.26 : Results from the slip circle method by Bishop (actual case) (Lu et al, 2015).

After all analysis they concluded that it takes less seismic energy to fail the slope with ground water than did the slope without ground water because ground water induces the slope failure at a lower horizontal seismic intensity. The reliability of the proposed procedure was verified based on the fact that the corresponding value of seismic intensity at the collapse of the slope nearly coincided with the result from the conventional slip circle method.

The horizontal displacement toward the slope increased rapidly due to the effect of the ground water. Additionally, a greater plastic deformation at collapse was obtained from the static stability analysis for the slope with ground water in any case.

The cumulative plastic displacement with ground water was larger than that in the case without ground water. This tendency reduced the elapsed time until collapse. Thus, it was observed that the slope with ground water collapsed within a shorter elapsed time than did the slope without ground water. The case studies showed that the proposed method can produce realistic predictions of the effect of ground water. If the duration of vibration is extended until collapse, the slope may not actually collapse. Therefore, it is important to accurately estimate the damping constants to improve the reliability of the proposed procedure. This subject must be investigated further using a large number of applications of the proposed procedure to actual slope failures.

One of the aspects of studying the earthquake induced landslides is the behavior of foundations on or near the slope, which is a major concern in this project too. The behavior of foundation near slope under seismic loading is another important issue which has to be considered. Numerous researchers are interested in this case as Gazetas R. Kourkoulis, I. Anastasopoulos, F. Gelagoti (2010). They studies the

combined effects of earthquake-triggered landslides and ground shaking on foundation-structure systems founded near slope crests. They used plane strain nonlinear finite element dynamic analyses to do evaluate the behavior of foundation near the slope. The soil constitutive model was calibrated against published data to simulate the (post -peak) softening behavior of soil during a seismic event and under the action of gravitational forces. Flexural beam elements were used to model the foundation and frame columns and beams, while the possibility of sliding and separation between the foundation and the underlying soil is considered through the use of special frictional gap elements. They have studied the effects of foundation type on the position of the sliding surface, on the foundation total and differential displacements. They did four parametric study which are sketched in Figure 4.27(a–d). They all refer to a 23 degree slope, 30 m high, consisting of post-peak softening sandy clay with the properties summarized directly in Figure 4.27a. The total upslope thickness of this soil layer is 40 m and is underlain by a 20 m thick stiff sandy soil layer down to bedrock. While Figure 4.27a refers to the free-field problem, i.e. of the slope without any structure affecting it, Figures 4.27(b–d), examine the interaction of the precarious slope with a number of idealized structures not far from its crest: a rigid uniformly loaded raft foundation (Fig. 4.27b), a three-bay single-story frame on isolated footings (Fig. 4.27c), a similar frame founded on a rigid raft (Fig 4.27d). The horizontal base excitation is uniformly applied along the x-axis, utilizing a set of both real accelerograms and idealized mathematical pulses (Ricker wavelets). The selected time histories cover a wide range of seismic motions, ranging from medium intensity records of relatively short duration (e.g. Kalamata, 1986; Pyrgos, 1993; Aegion, 1996; Monastiraki, 1999; Ricker 3, Ricker 1) to very strong accelerograms (e.g. Kobe, 1995 JMA and Takatori, Imperial Valley 1994) featuring a large number of significant cycles (Fig. 4.28). Finite element mesh is shown in Figure 4.29. Some of their results are shown in Figures. 4.30, 4.31 and 4.32.

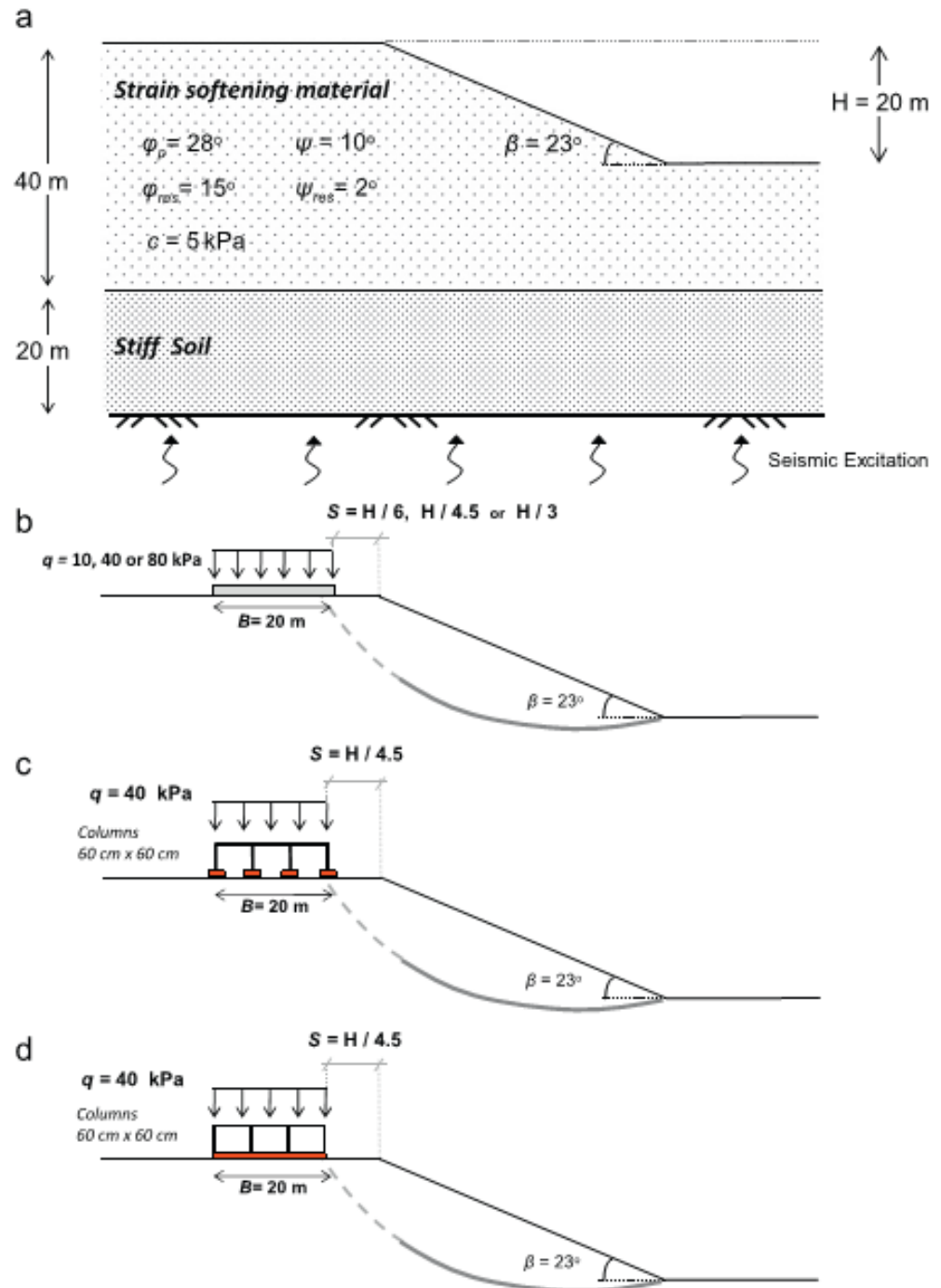


Figure 4.27 : Geometry and soil properties of the analyzed slope: (a) at the free-field, (b) with a rigid uniformly loaded slab foundation, (c) with a 3-bay single storey frame on isolated footings, and (d) with a 3-bay single storey frame on rigid raft. (Kourkoulis et al, 2010).

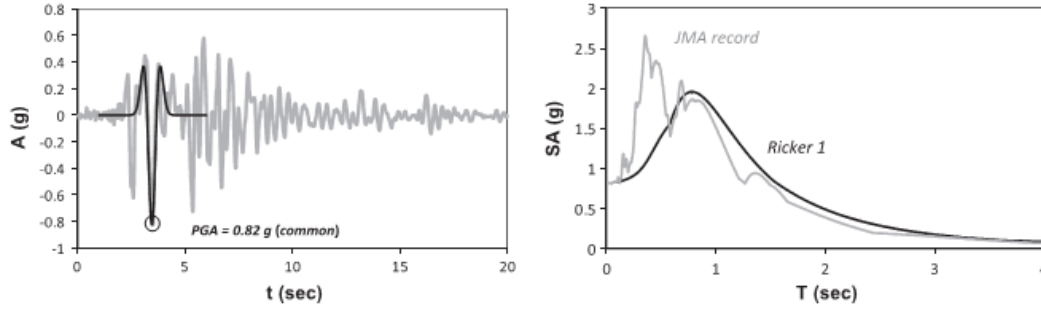


Figure 4.28 : The two seismic excitations discussed in the paper (Kobe JMA and Ricker 1), along with their 5% damped response acceleration spectra. (Kourkoulis et al, 2010).

The numerical analysis is performed in three consecutive steps:

- (1) Gravity loading (static analysis) establishment of the initial state of stress in the slope.
- (2) Seismic excitation (dynamic analysis) triggering of shear band (depending on the intensity of seismic shaking, a shear band extending from toe to crest may develop, with plastic strains reaching or exceeding their peak-strength values, at least in a few locations).
- (3) Post-earthquake response (analysis in the time domain) evolution of the seismically triggered landslide after the end of shaking due to the action of gravity (if a continuous shear-band has developed during seismic shaking and soil elements have reached their residual strength, deformations will keep increasing becoming, in principle, infinitely large).

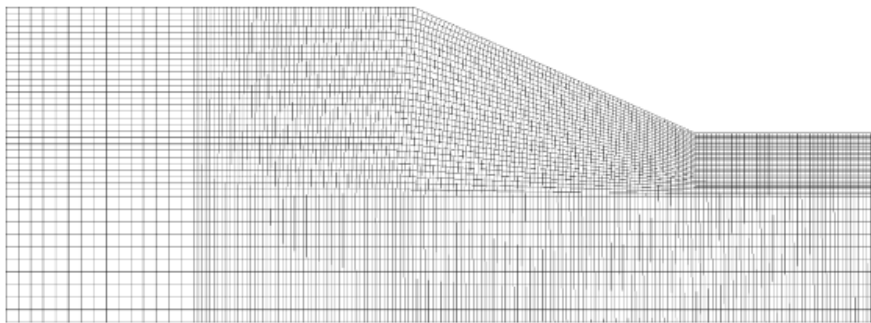


Figure 4.29 : Finite element mesh. In the area of the slope the discretization is denser (0.5 m 0.5 m quadrilateral elements); towards the lateral boundaries the mesh is coarser. Elementary transmitting boundaries ensure insignificant contamination of the response with spurious wave reflections. (Kourkoulis et al, 2010).

The finite element model used in analysis has been shown above. They tried different foundation type and also various load on foundations. Then they studied the effect of distance of foundation from the crest of slope.

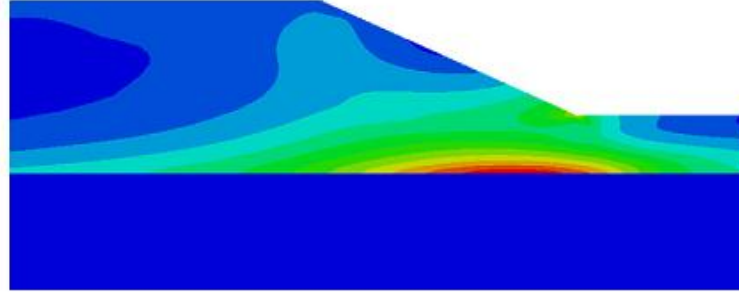


Figure 4.30 : Free-field dynamic analysis with the Ricker 1 wavelet at 0.82 g; plastic strain contours upon completion of seismic shaking. Despite the exceedance of the pseudo-static yield acceleration (0.2 g), no shear band (failure) mechanism develops. (Kourkoulis et al, 2010).

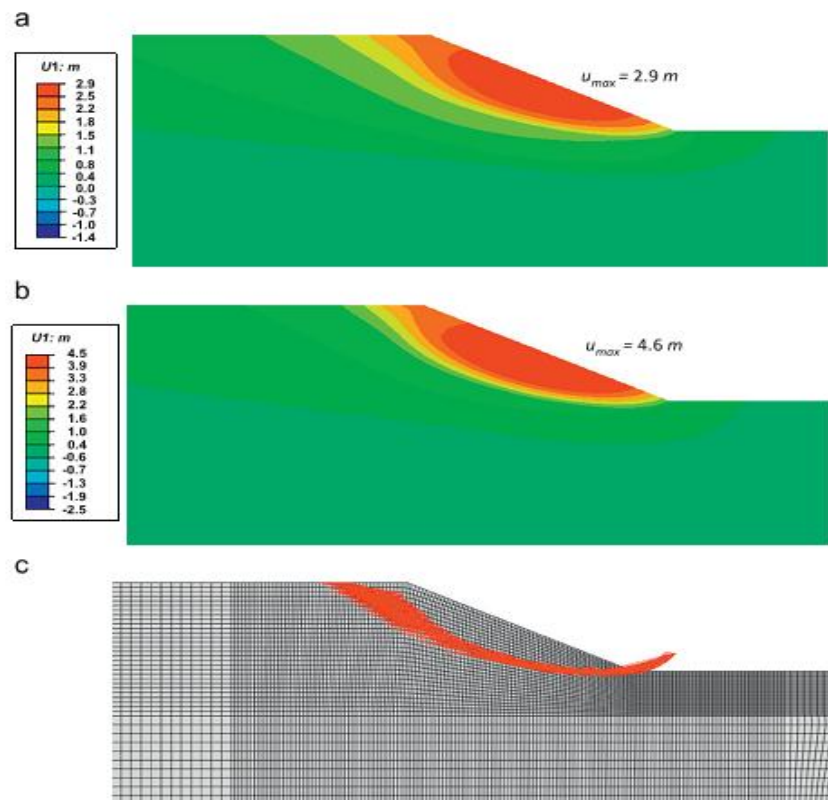


Figure 4.31 : Free-field dynamic analysis with the JMA record (0.82 g): Contours of horizontal displacement (a) upon completion of seismic shaking, and (b) post seismically (i.e. after completion of Step 3); (c) deformation vectors along the generated sliding surface at the end of seismic shaking. (Kourkoulis et al, 2010).

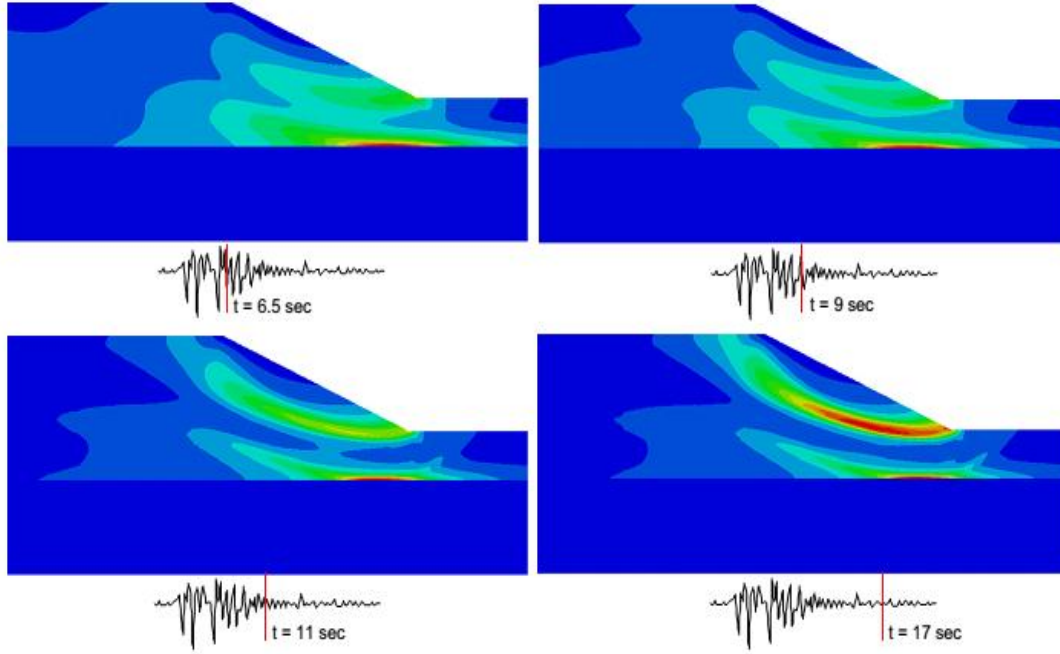


Figure 4.32 : Free-field dynamic analysis with the JMA record (0.82 g): evolution of plastic shear strains during seismic shaking. (Kourkoulis et al, 2010).

Fig. 4.33 shows the distribution of the seismically induced plastic shear strains for the foundation loaded with $q = 40$ kPa and placed at different locations, relative to the crest of the slope.

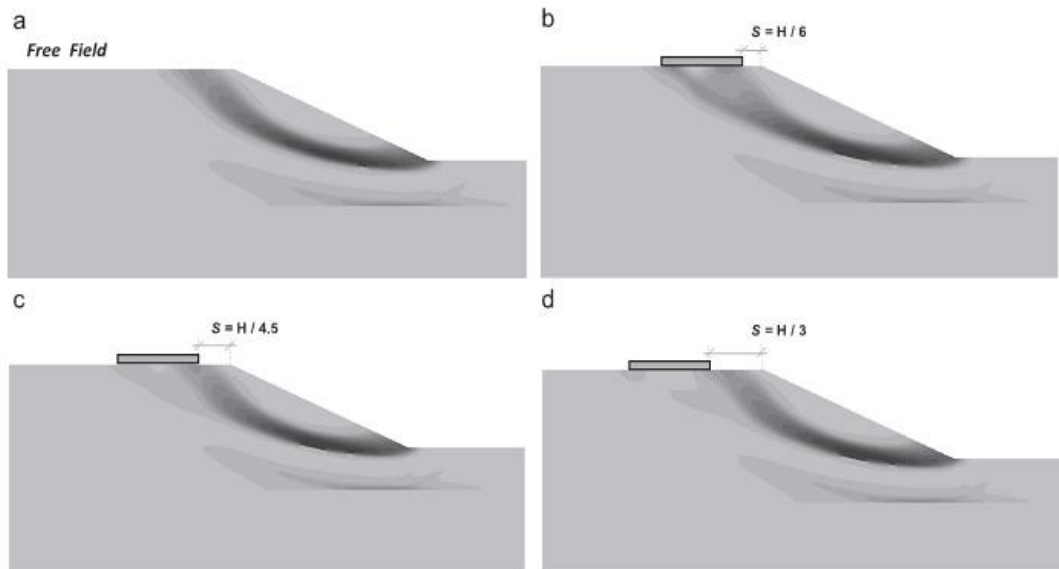


Figure 4.33 : Rigid $B = 20$ m raft foundation with surcharge load $q = 40$ kPa placed parametrically at $S = H/6$, $H/4.5$ and $H/3$ (Kobe JMA seismic excitation): distribution of seismically induced plastic shear strains at the end of seismic shaking (Step 3). (Kourkoulis et al, 2010).

The following observed during analysis:

(a) When the footing is located relatively close to the crest ($S = H/6$), the failure surface shifts its path from its original free-field position towards the inner part of the slope, outcropping on the left edge of the footing; a secondary plastic zone emerges at the right edge of the foundation. Apparently, in this case a bearing capacity type failure mechanism is partially activated due to limited support of the foundation from the downhill sliding wedge. As a result, the foundation rotates extensively. To have a better understanding, in Figure 4.34 shows the distribution of contact pressures underneath the foundation. The top figure (Fig. 4.34a) plots the distribution of static contact pressures, followed below (Fig. 4.34b) by the contact pressures after completion of seismic shaking (end of Step 3). The change is quite substantial, accompanied by a reduction in the effective contact area of the raft: detachment of foundation from the bearing soil at the two edges. The time history of horizontal displacement u_h of the foundation (Fig. 4.35a) reveals nonzero horizontal velocity at the end of seismic shaking, implying that the downhill sliding continues after the end of the earthquake. As it would be expected, the rotation γ of the foundation is increased with the decrease of its distance from the crest of the slope (Fig. 4.35 b), (Kourkoulis et al, 2010).

(b) Moving the footing farther from the crest ($S = H/4.5$), the generated shear zone becomes diffused and deviates slightly towards the foundation slab, which again loses contact from the underlying soil at both edges. The separation is largest under the crest side of the foundation, which now (perhaps as a beneficial result of this detachment) survives the landslide (i.e., no further displacement takes place (Fig. 4.35 a)), (Kourkoulis et al, 2010).

(c) When the distance of the foundation from crest increases even more ($S = H/3$ for $q = 40$ kPa) the footing is located on a safe region. In such a case, the existence of the structure hardly modifies the plastic shear strain field (Fig. 4.33d), and failure occurs just beyond the right (closer to the slope) edge of the foundation, which never loses contact with the underlying soil (Kourkoulis et al, 2010).

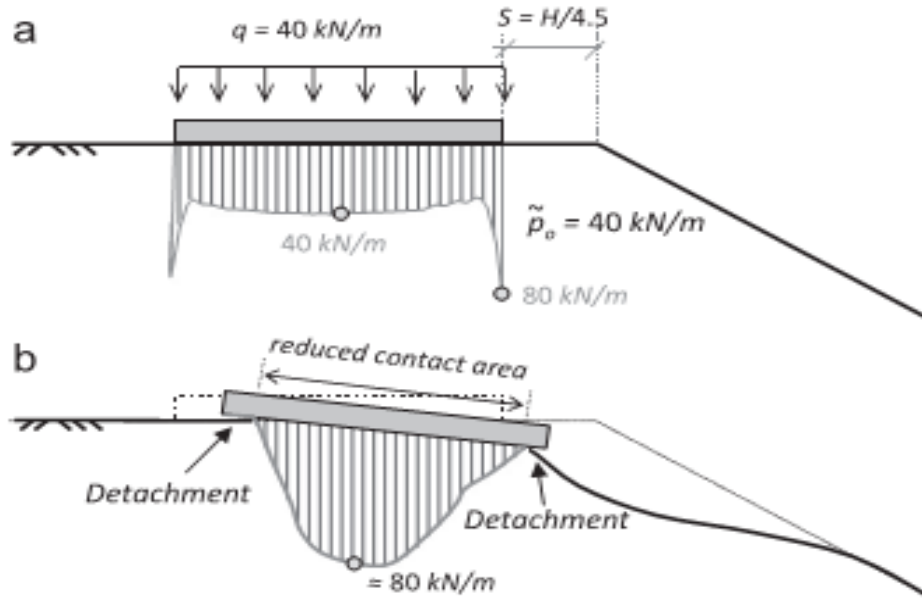


Figure 4.34 : Rigid $B = 20$ m raft foundation with surcharge load $q = 40$ kPa at distance $S = H/4.5$ from the slope crest (Kobe JMA seismic excitation): (a) static contact pressures and (b) contact pressures after the end of seismic shaking (Kourkoulis et al, 2010).

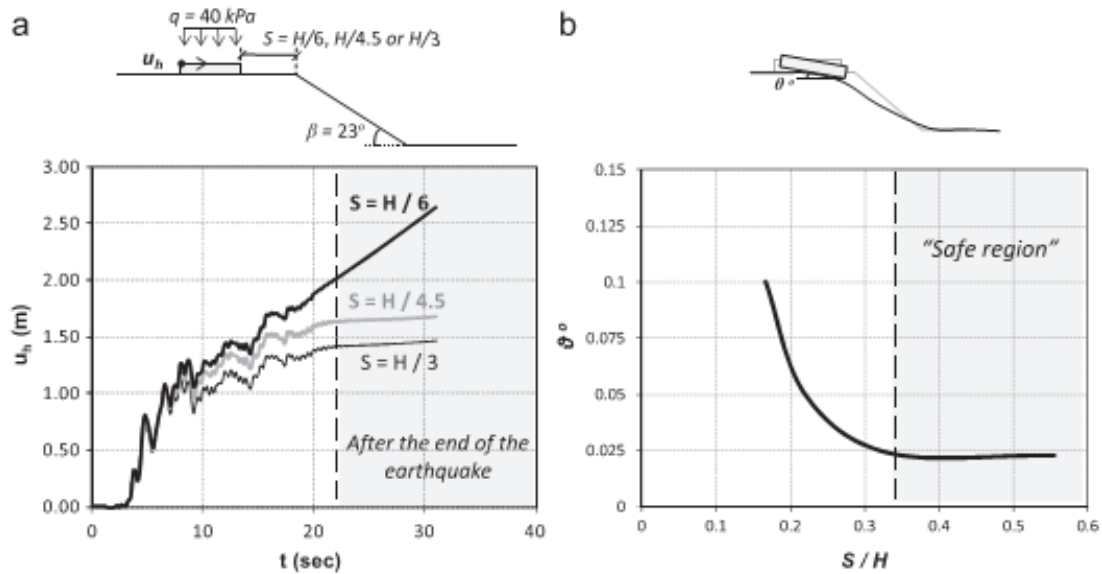


Figure 4.35 : Rigid $B = 20$ m raft foundation with surcharge load $q = 40$ kPa (Kobe JMA seismic excitation) placed parametrically at different locations relative to the crest of the slope: (a) time-history of horizontal displacement u_h of the foundation; and (b) foundation rotation y as a function of its normalized distance S/H from the slope crest (Kourkoulis et al, 2010).

Studying the effect of different surcharge loads also yields to interesting results. Some of the most important results are listed below, which are also one of the goals

of the thesis. The effect of different types of loading will be evaluated for Elazig and Kocaeli cases too.

- a) Increasing q from 20 to 80 kPa gives rise to a diversion of the zone of excessive shearing deformation (Fig. 4.36). This deviation of the path of the shear band is particularly conspicuous for $q = 80$ kPa (Fig. 4.36d), in which case the shearing due to shaking of the slope interacts with the rudimentary bearing capacity mechanism under the footing (local shearing), (Kourkoulis et al, 2010).
- b) The foundation detachment from the supporting soil due to seismically induced slope instability that was observed with the light surcharge load ($q = 20$ kPa) is diminished with the heavier load ($q = 40$ kPa), practically disappearing for $q = 80$ kPa. Note that the remarkably large detachment of the lightly loaded foundation (only its middle 1/4 remains in contact) will generate huge bending moments on the raft. (Kourkoulis et al, 2010).
- c) Figure 4.37a reveals that the heavily loaded foundation is not safe: it has positive velocity at the end of shaking, and its displacement (outward and downward) continues to grow under the sole action of gravity (Step 4). Foundation rotation γ increases markedly with q (Fig. 4.37b), (Kourkoulis et al, 2010).

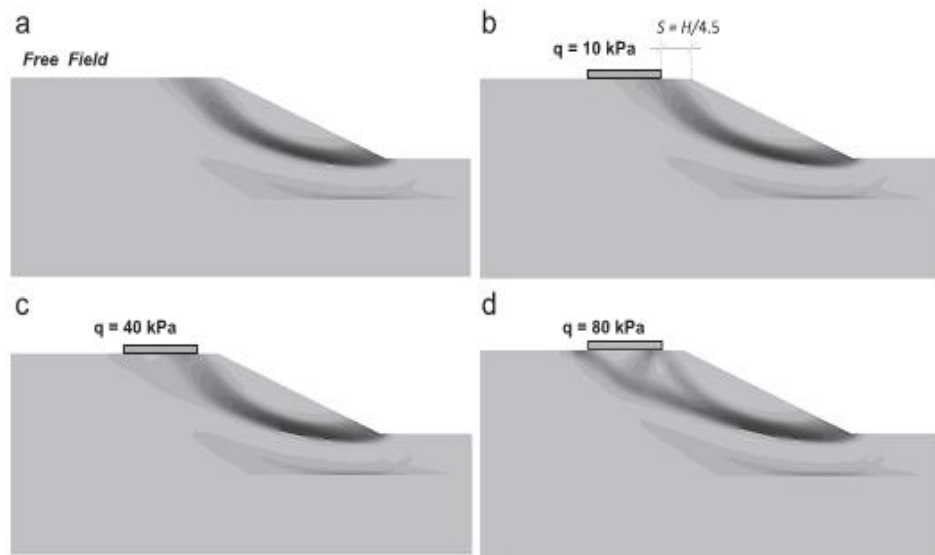


Figure 4.36 : Rigid $B = 20$ m raft foundation placed at $S = H/4.5$ with parametrically varied surcharge load q (Kobe JMA seismic excitation): distribution of seismically induced plastic shear strains at the end of seismic shaking (Step 3) (Kourkoulis et al, 2010).

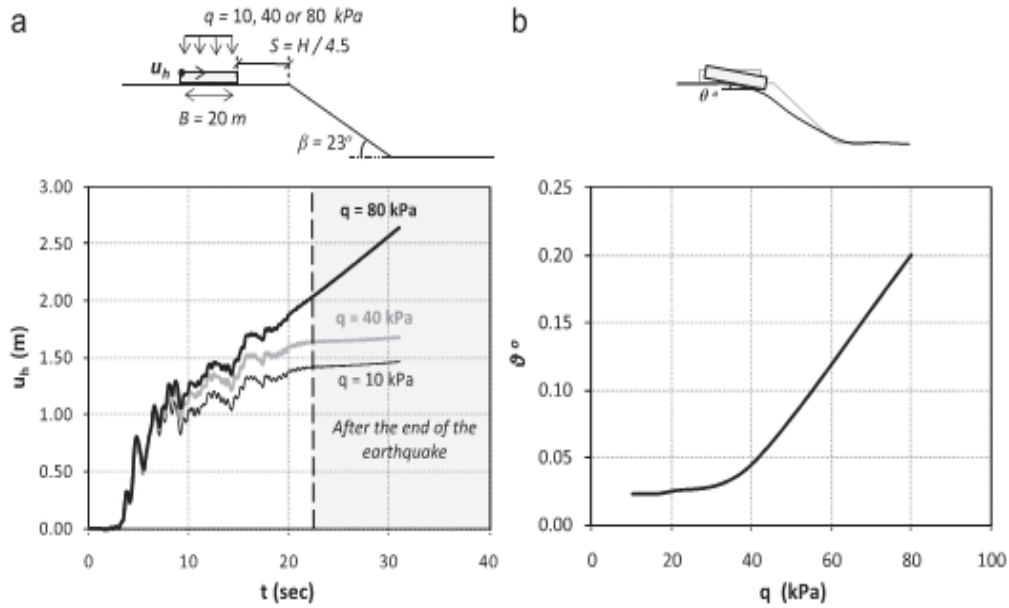


Figure 4.37 : Rigid $B = 20\text{ m}$ raft foundation placed at $S = H/4.5$ with parametrically varied surcharge load q (Kobe JMA seismic excitation): (a) time-history of horizontal displacement u_h of the foundation; and (b) foundation rotation y as a function of the surcharge load q (Kourkoulis et al, 2010).

Figures. 4.38, 4.39, 4.40 and 4.41 show the results for different analysis done by authors of the study. Figures 4.38 and 4.39 show the interstorey drift in four columns of the resisting frame first on the isolated footings and then on raft foundation.

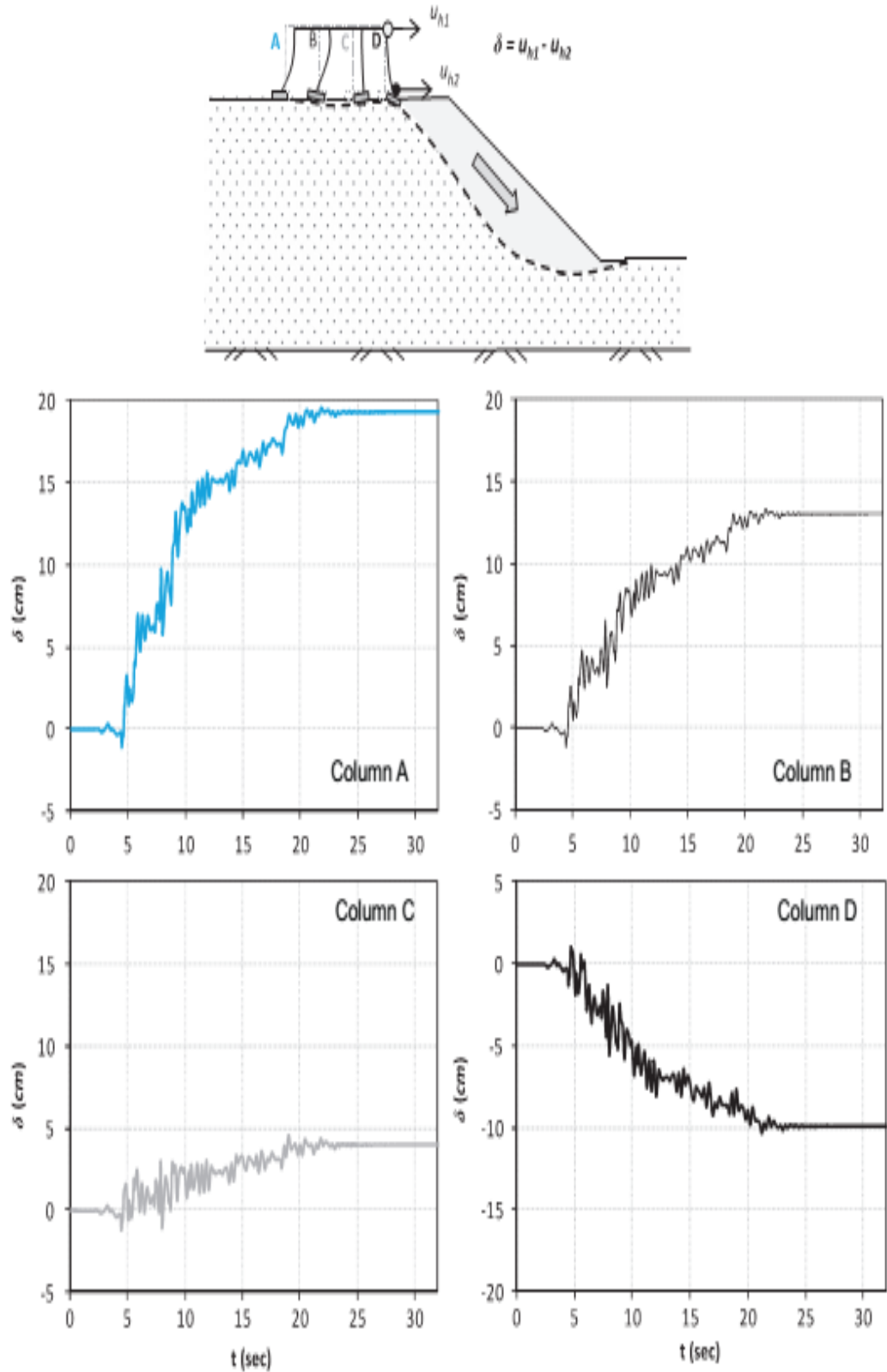


Figure 4.38 : Typical B = 20 m moment resisting frame founded on isolated footings, at distance $S = H/4.5$ (Kobe JMA seismic excitation): time histories of interstorey drift d developing in the four columns of the frame (Kourkoulis et al, 2010).

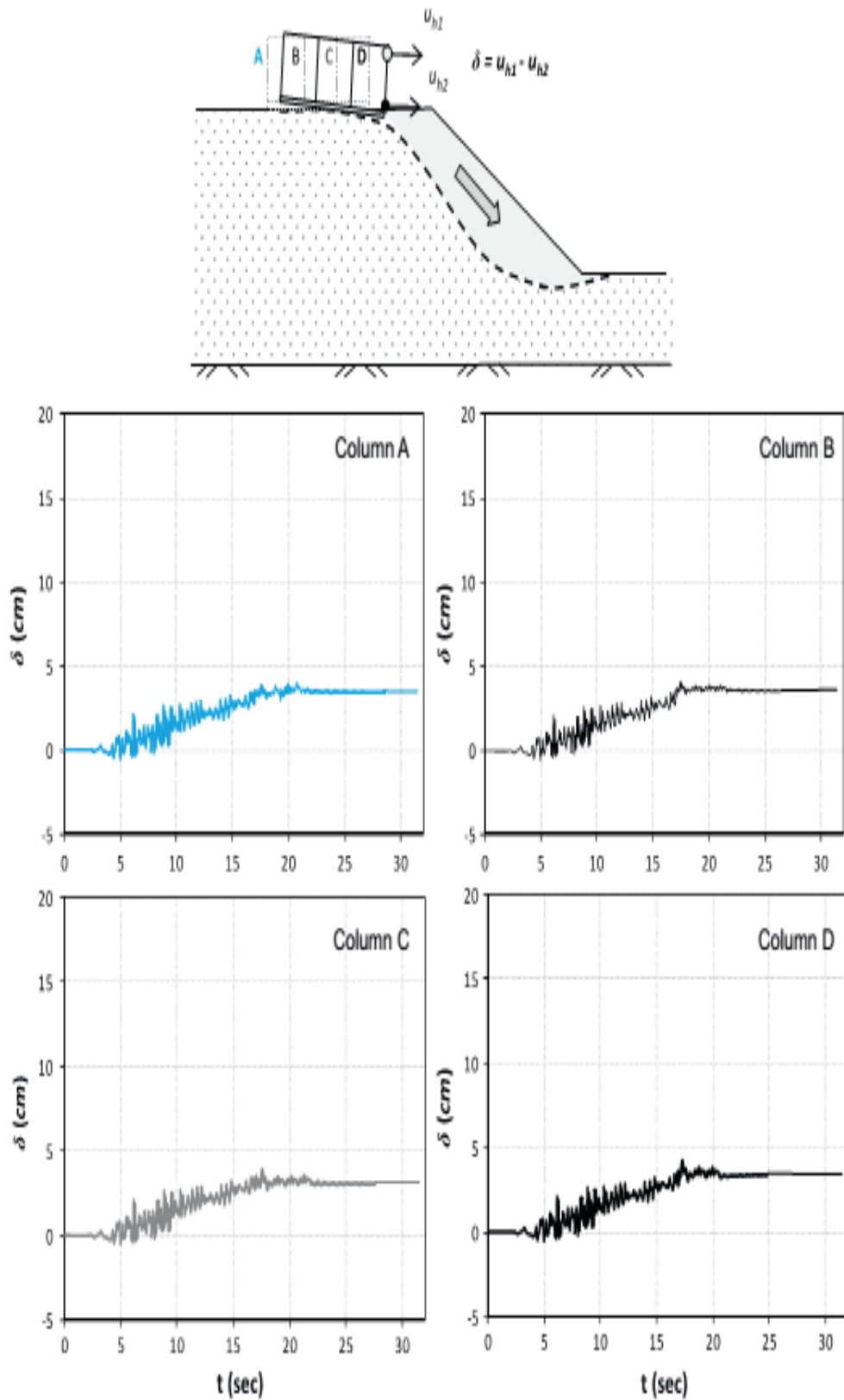


Figure 4.39 : Typical B=20 m moment resisting frame founded on a rigid raft foundation, at distance $S=H/4.5$ (Kobe JMA seismic excitation): time histories of interstorey drift d developing in the four columns of the frame (Kourkoulis et al, 2010).

Figure 4.40 shows the sketch elucidating the mechanism of gap formation underneath the rigid foundation slab, normalized contact pressures p/q after the end of seismic shaking; and foundation bending moments before (grey line) and after the end of the earthquake

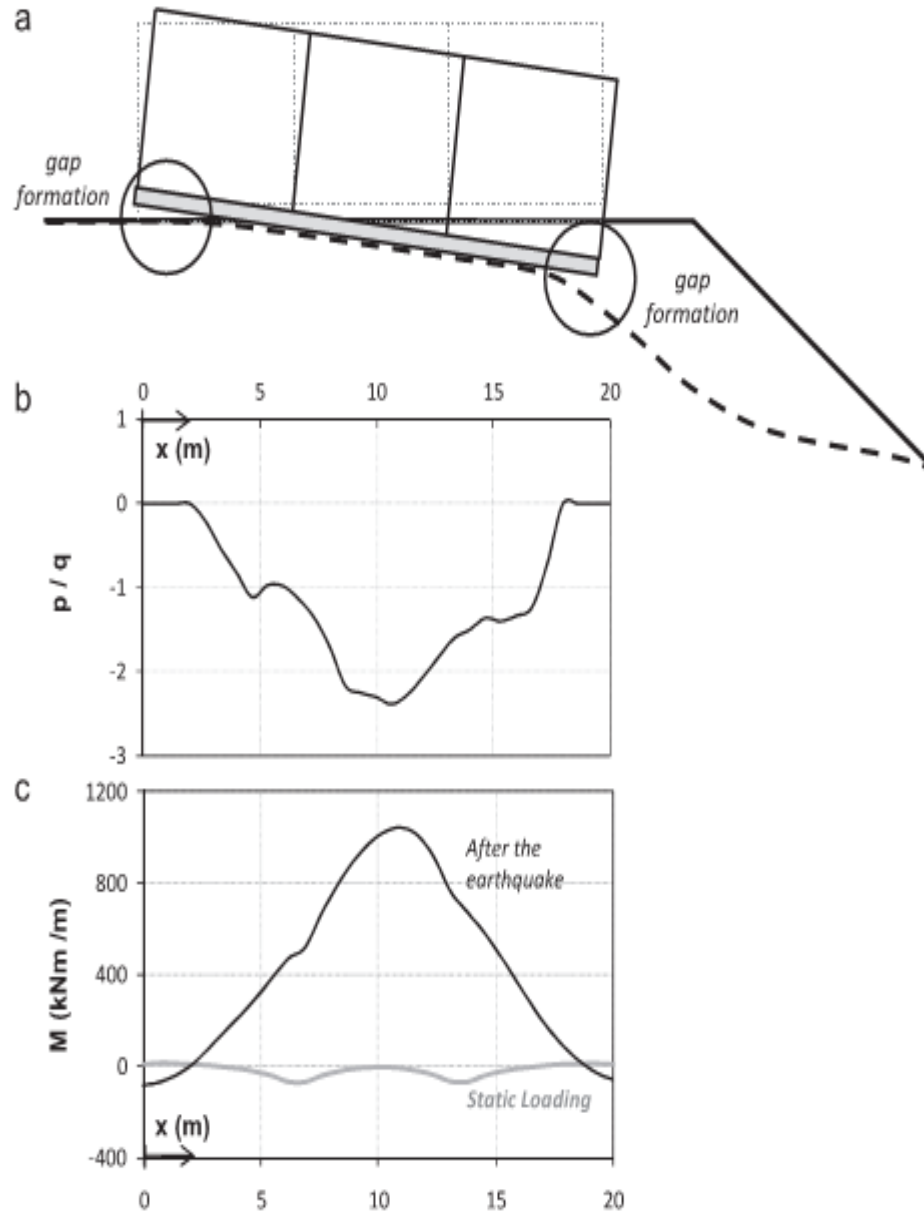


Figure 4.40 : Typical $B = 20$ m moment resisting frame founded on a rigid raft foundation, at distance $S = H/4.5$ (Kobe JMA seismic excitation): (a) sketch elucidating the mechanism of gap formation underneath the rigid foundation slab; (b) normalized contact pressures p/q after the end of seismic shaking; and (c) foundation bending moments before (grey line) and after the end of the earthquake (solid black line) (Kourkoulis et al, 2010).

Figure 4.41 shows the evolution of normalized foundation contact pressures p/q (where q is the static contact pressure) at three characteristic locations.

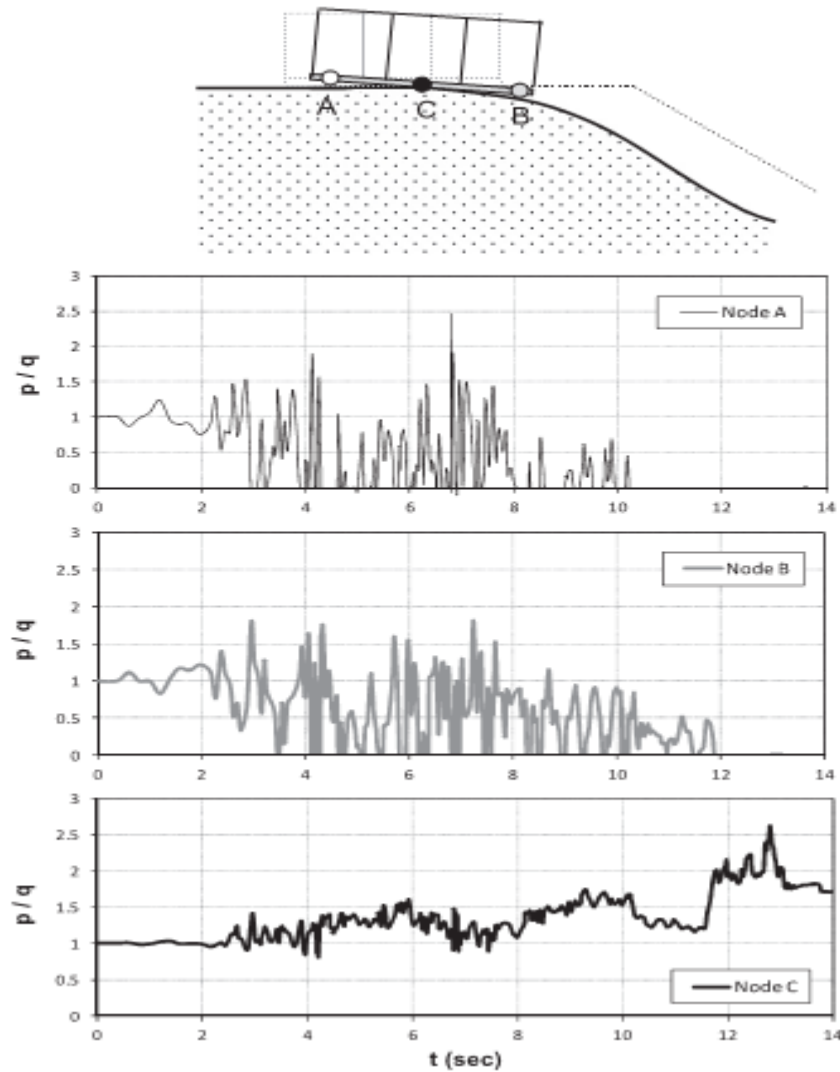


Figure 4.41 : Evolution of normalized foundation contact pressures p/q (where q is the static contact pressure) at three characteristic locations (Kourkoulis et al, 2010).

The most important conclusions of the study are listed below:

A key conclusion of the study, which is also a good help for this thesis too, is that finite-element (FE) analyses can be successfully utilized to model the generation of sliding failure surfaces in the slope, reproducing similar failure surfaces with those derived from venerable limit-equilibrium and limit-analysis methods, and leading to similar yield accelerations. But the capability to treat realistically the dynamic response to ground shaking is an exclusive attribute of the numerical (FE) methodology, not of the pseudo static limit equilibrium analysis methods.

An additional capability of the numerical (FE) methodology is that any structure-foundation system can be placed on top of the slope. As a result, we studied the interplay between dynamic effects arising from the ground vibration and the quasi-static (kinematic) effects arising from the downward movement of a shallow sliding soil wedge. It was thus determined that a rigid raft foundation, even if located at a small distance (of the order of $H/4$) from the crest of a slope which is in a precarious equilibrium and fails during very strong shaking, can protect the superstructure from both falling (being dragged) with the sliding soil mass and from suffering large damaging internal forces. This is in qualitative agreement with several case histories of structures that have survived the combined effects of strong seismic shaking and of soil downward sliding. The penalty to pay, however, is: (a) appreciable rotation and lateral displacement of the whole system which may impair the serviceability of the structure, and (b) generation of large bending moments and shear forces in the foundation slab. By contrast, as in fact most engineers would have intuitively predicted, placing a structure with isolated footings close to a seismically unstable slope would not be a prudent decision (Kourkoulis et al, 2010).

As it was mentioned before, evaluating the behavior of buildings on slopes or near them is another part of the study and for Elazig earthquake which caused lots of landslides and damaged buildings on slopes, has to be studied carefully. A small case of Kocaeli earthquake will be presented also and comparisons between two cases will be explained.

Seismic behavior of buildings on slopes have been studied by different researchers around the world. Ahmed Abdelraheem Farghaly (2014), has worked on a case study on hillside Slope of Doronka village in Egypt (Fig 4.42). This study is a good case because of similarities between the affected region and also the existing buildings in the area, and our case which is Elazığ region.



Figure 4.42 : The techniques of constructing building on hillside slope in Doronka village (Farghaly, 2014).

Two recorded accelerographs of Northridge (1989) and El Centro (1940) earthquakes (Fig. 4.43), were chosen as seismic sources for analysis.

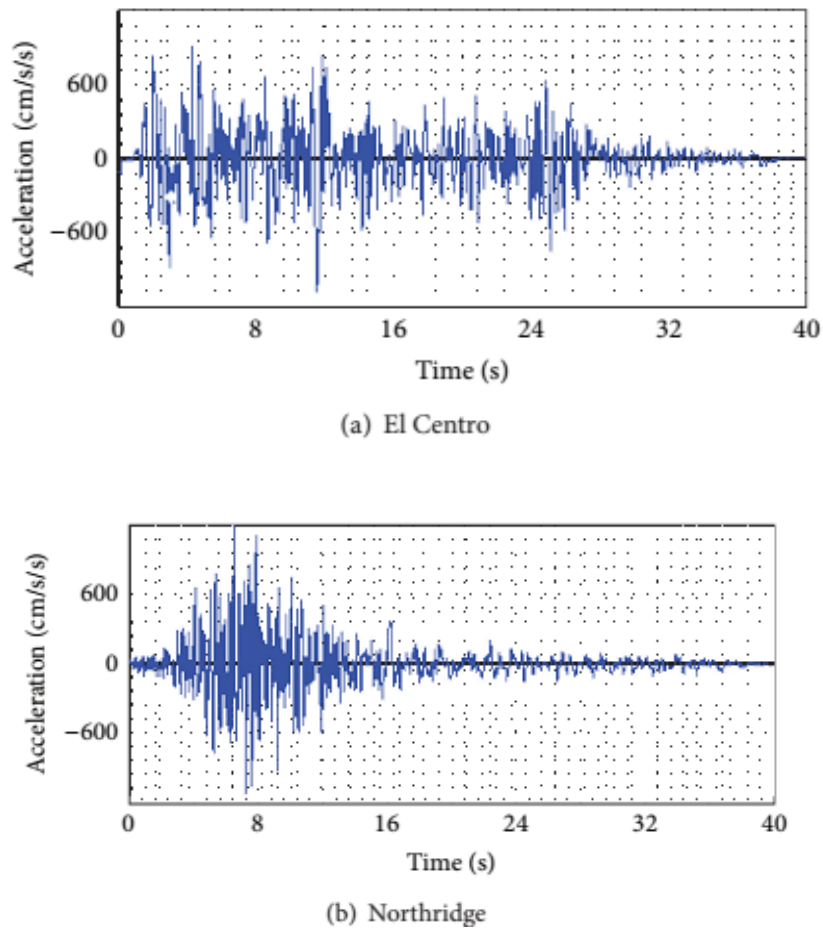


Figure 4.43 : Accelerograms used for analysis (Farghaly, 2014).

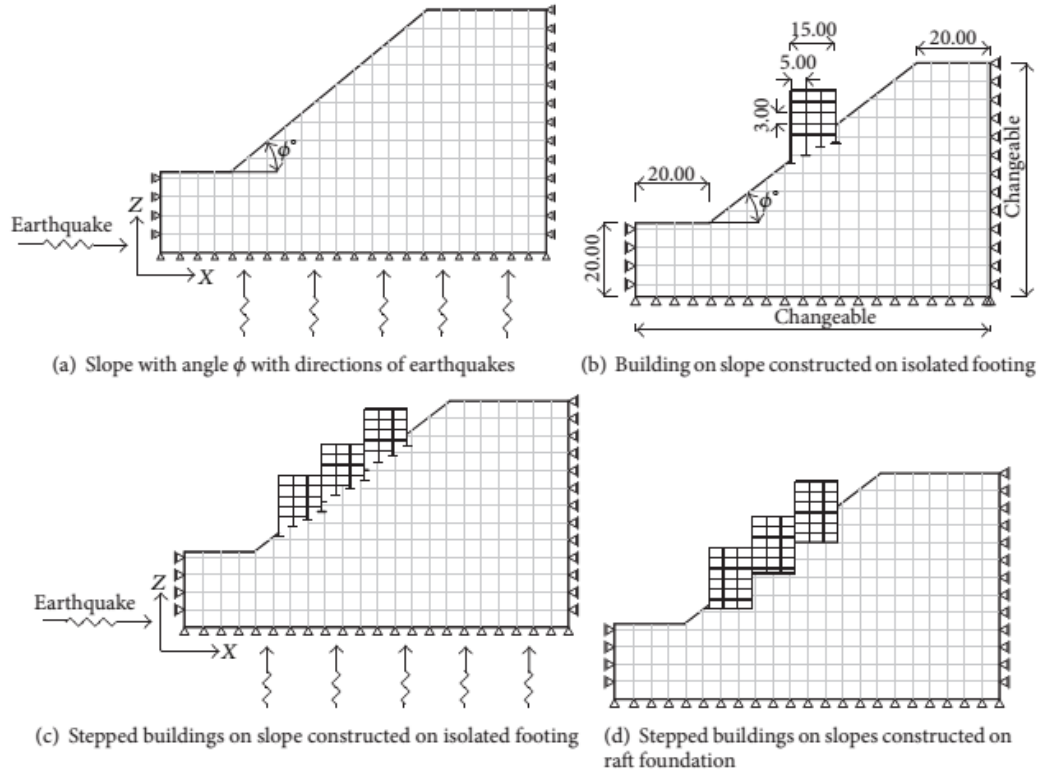


Figure 4.44 : Finite element mesh: in the area of the slope the discretization is denser ($0.5 \text{ m} \times 0.5 \text{ m}$ quadrilateral elements) (Farghaly, 2014).

The beams and columns were modeled as frame elements in the building model showed above. The column dimension was $50 \times 50 \text{ cm}$ and the beam section was $25 \times 50 \text{ cm}$ all over the height of the building. The foundation is chosen to be either raft or isolated footing in the analyses. A 4 story building was modeled with a 3 m height for each storey and a 15 m length, as shown in Figure 4.44. The inelastic analysis is performed, and the failure criterion used for both the soil and the structure members has to be stated as well. A Beam element was loaded with constant distributed loads equal to 2.50 ton/m at all floor levels, in addition to own weight of elements.

The author has studied the behavior of hillside slopes with two criteria. The first one is the performance of the building with a different foundation system to seismic loads on the hillside slope and presenting the straining actions of them resulting from two earthquakes (El Centro and Northridge earthquakes) analyses in steps, that is, first 0.25 g, 0.5 g, and 1 g, in increasing order of the PGA and the second part is to evaluate the performance of hillside slope (slope angles 60° , 45° , and 25°) under static and seismic loads with and without constructed building to evaluate the static and dynamic stability of the hillside slope.

In order to compare the results of analysis, some figures have been prepared. The description of each symbol is shown below:

- I 60°: Building on 60° slope angle founded on isolated footing
- R 60°: Building on 60° slope angle founded on raft footing
- I 45°: Building on 45° slope angle founded on isolated footing
- R 45°: Building on 45° slope angle founded on raft footing
- I 25°: Building on a 25° slope angle founded on isolated footing
- R 25°: Building on a 25° slope angle founded on isolated footing
- Fixed ISO: Building with fixed, stepped isolated footing
- Fixed raft: Building with fixed raft footing

Figures 4.45, 4.46, 4.47 and 4.48 show some of analysis and obtained results.

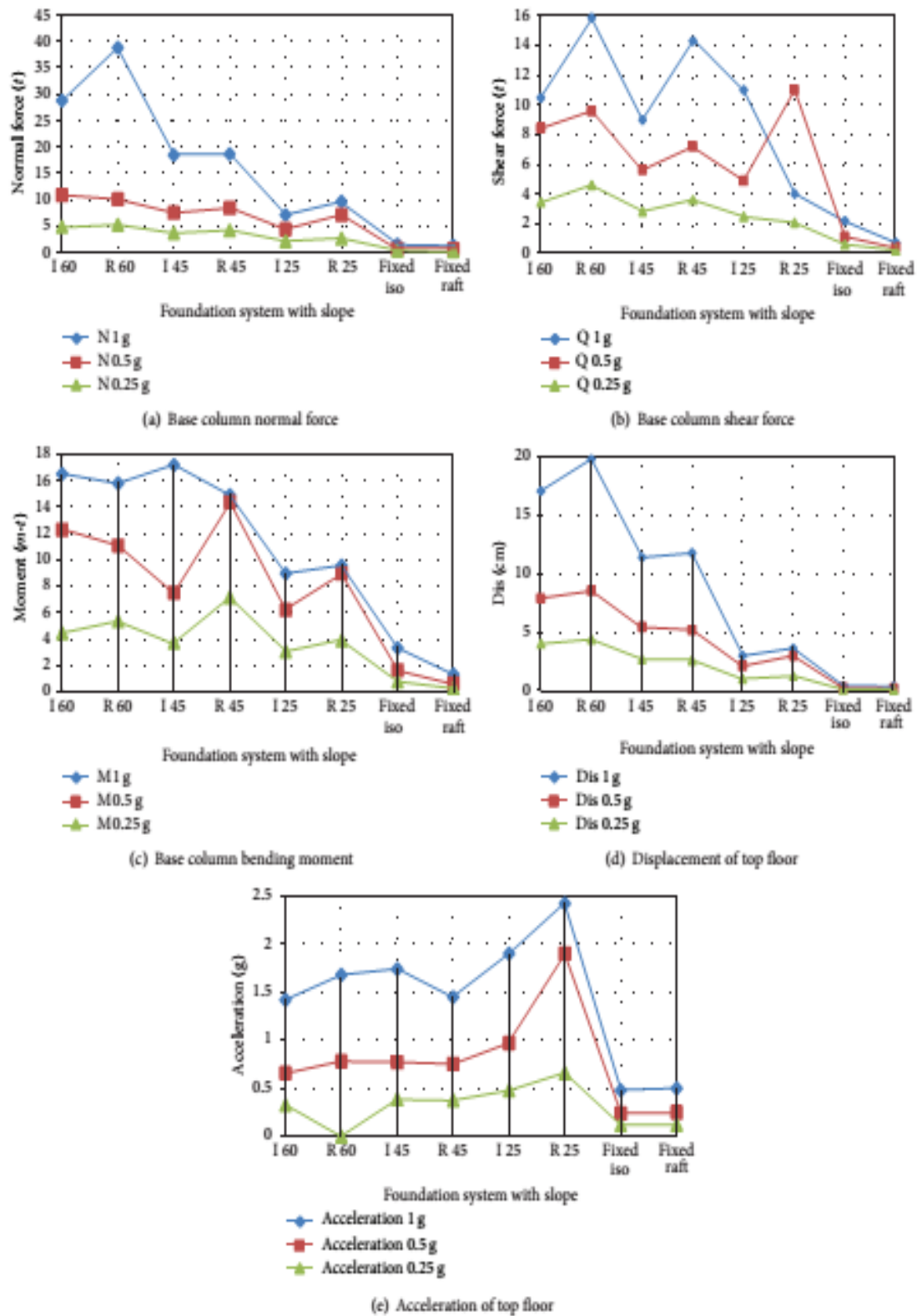


Figure 4.45 : Results for different types of foundation systems and slope angles with El Centro earthquake excitation (Farghaly, 2014).

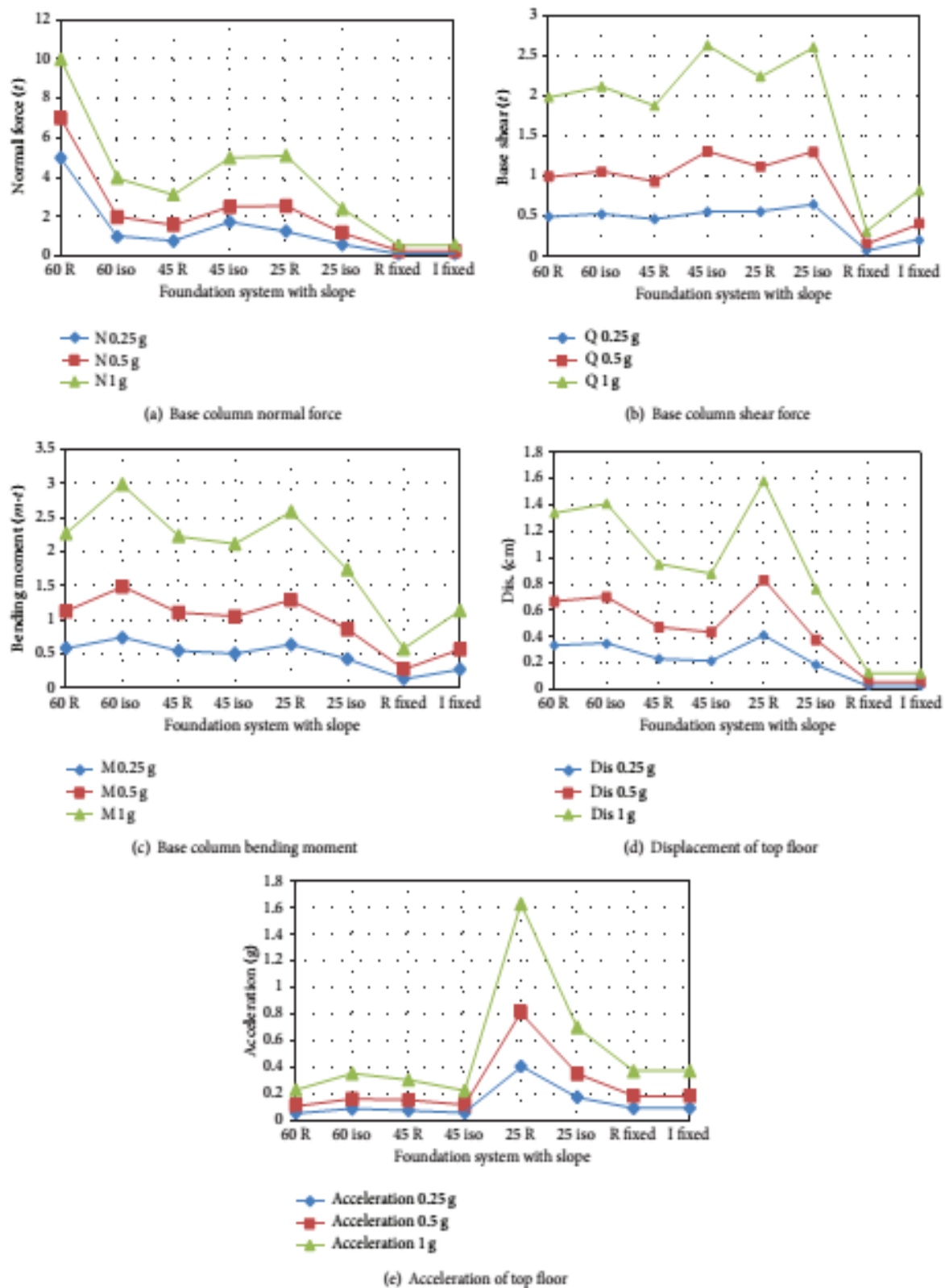


Figure 4.46 : Results for different types of foundations and slope angles with Northridge earthquake (Farghaly, 2014).

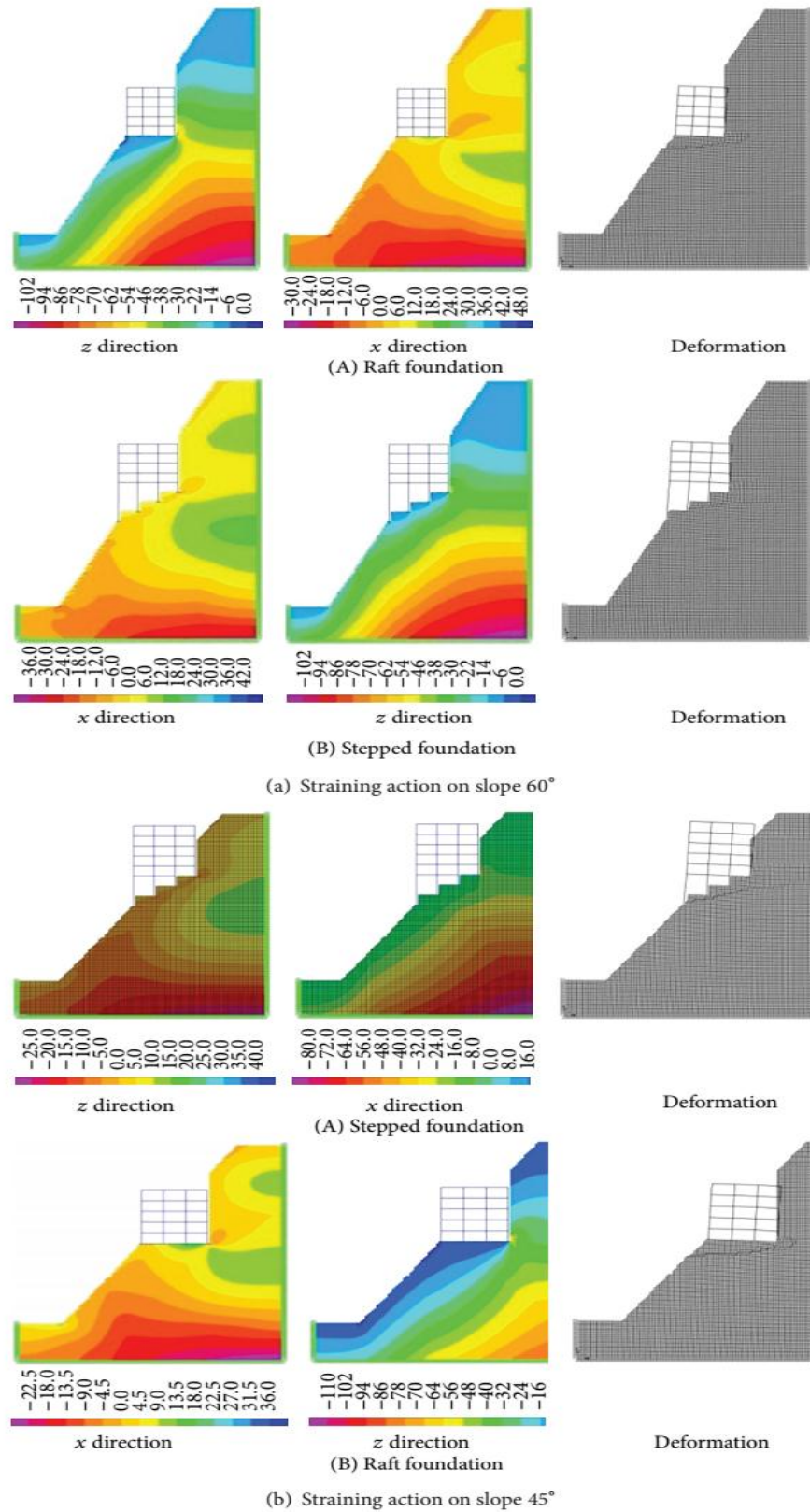


Figure 4.47 : Results for different types of foundation systems and slope angles with El Centro earthquake excitation with PGA equal to 1 g (Farghaly, 2014).

Same analysis were done using Northridge earthquake excitation with PGA equals 1g.

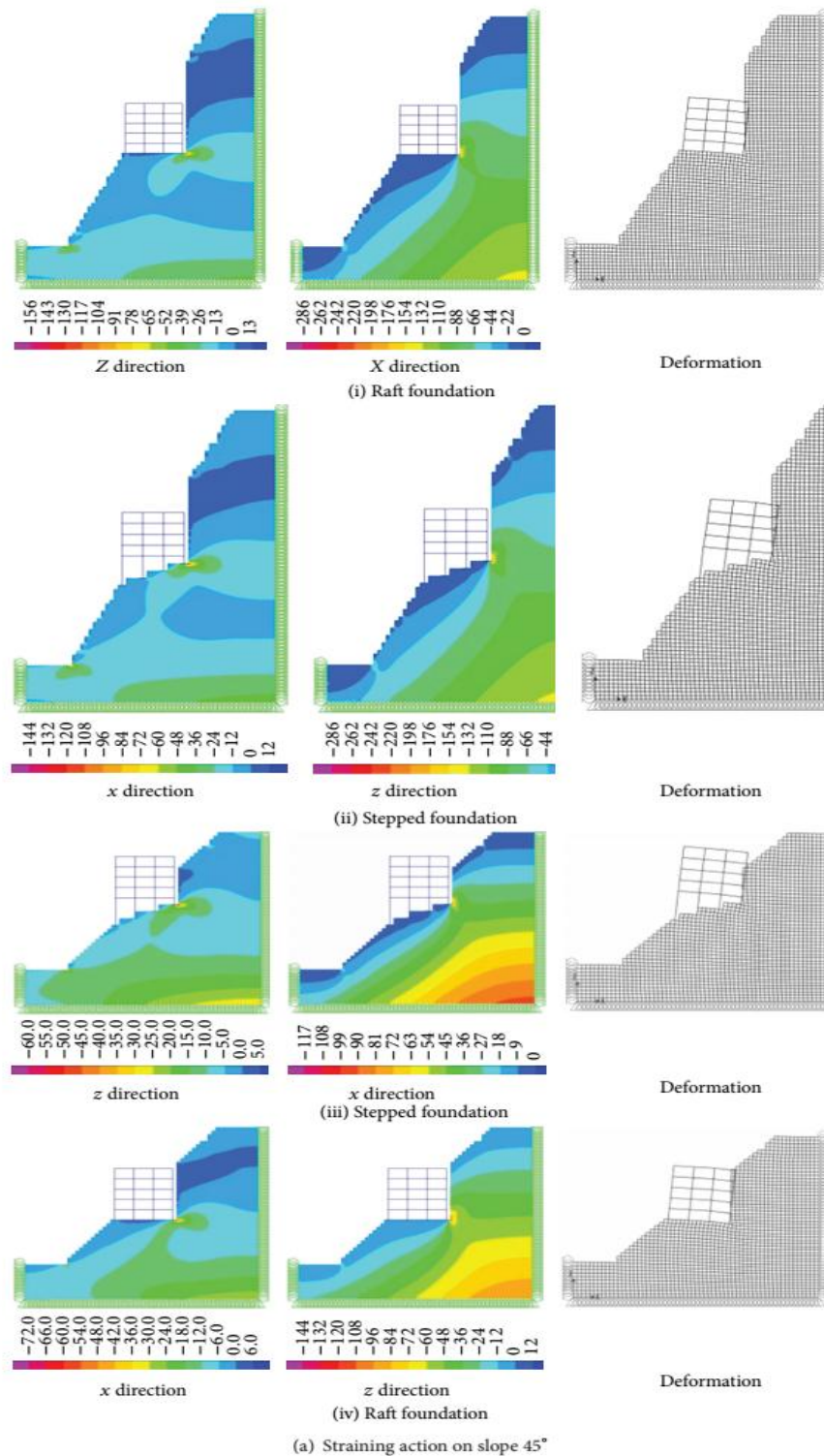


Figure 4.48 : Results for different types of foundation system and slope angle with Northridge earthquake excitation with PGA equals 1 g (Farghaly, 2014).

Some of the interesting results which were derived during analysis are explained below:

The stability of slopes (without constructing buildings) (with slope angles range from 60° to 25°) was acceptable in the high PGA for the study of rocky soil. No tensile stress was found in the hillside slope. Compression stress in a hillside slope angle 60° decreased by 1.5 times than hillside slope angle 45° and decreased by 1.4 times than hillside slope 25° under seismic excitations. The interplay between dynamic inertial effects arising from the ground vibration and the quasistatic “kinematic” effects arising from the downward movement of a shallow sliding soil wedge are studied. It was also concluded that a rigid raft foundation, located on a slope with a big angle which is in a precarious equilibrium and fails during very strong shaking, cannot protect the superstructure from both falling (being dragged) with the sliding soil mass and from suffering large damaging internal forces. The author recommends to flatten the slope from 60° to 45° to avoid wedge failures at all isolated stepped footing locations (normal force decreased by 50% for a slope angle 60° than slope angle 45° and base shear decreased by 40% in a slope angle 60° than slope angle 45°). Stepped isolated footing represents the best solution for both dynamic performance of superstructure constructed on hillside slope and the stability of the slope (columns normal force for isolated footing decreased by 1.33 times than raft foundation and decreased by 1.45 times in base shear). The effect of the tie beams to connect stepped footing has been ignored (Farghaly, 2014).

5. RESULTS AND DISCUSSIONS

5.1 Analysis of Elazığ Case

The location of Yukari Demirci village has been shown in figures 5.1 and 5.2.



Figure 5.1 : Areal view of Yukari Demirci village.

Figure 5.2 depicts a general view of slopes which have an inclination range from 10 to 35 degrees. The pictures has been produced using Google Earth computer program. This program uses photos taken by satellites to simulate the real world.

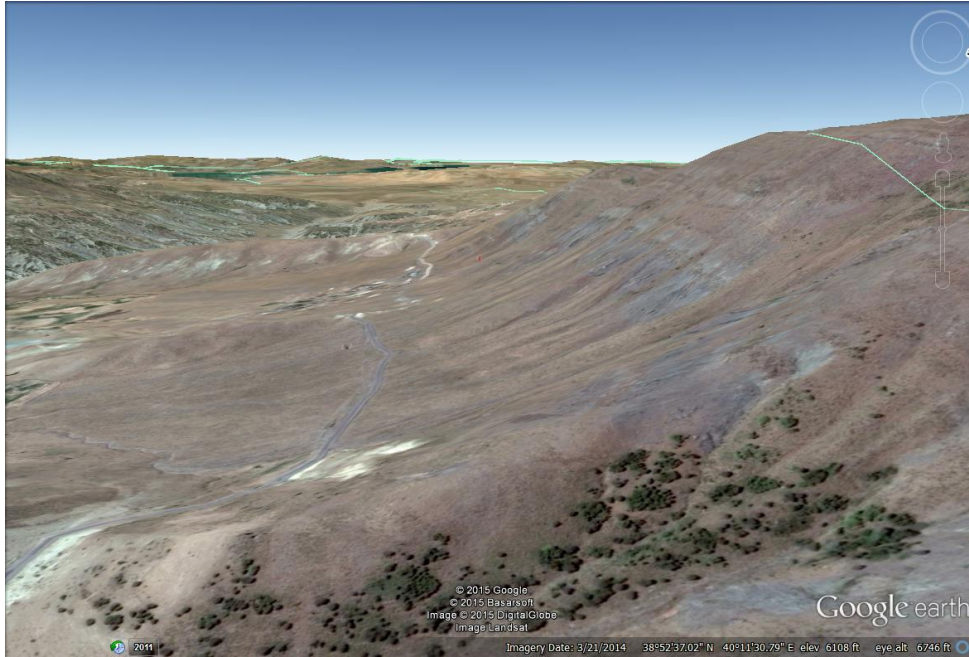


Figure 5.2 : General view of slopes in studied area.

5.1.1 The analyses procedure

First of all, a model with 510 m length and 185 m height has been chosen. The inclination of slope is 25 degrees, which can be considered as a general slope inclination of the area. The inclination of slopes at the earthquake affected area, varies between 10 to 35 degrees. So to evaluate the behavior of soil in affected area a typical slope with mentioned geometry has been chosen. There are three types of soil materials in the affected area, clayey sand (SC), silty sand (SM) and high plasticity clay (CH). All three types of main soil materials have been analyzed under Elazığ earthquake excitation and the displacements and also safety factors of mentioned slopes have been calculated and compared to each other. In another series of analyses, the behavior of slopes under a distributed static load, which can be a representative of a building load, have been studied. Finally the behavior of two types of buildings have been studied under Elazığ and Kocaeli earthquakes excitations. The Kocaeli earthquake has been one of the strongest ground motions in Turkey and here, has been chosen to compare the effect of a strong earthquake and a weaker one like Elazığ on structures in the affected area. For this series of analysis,

two types of sandy soil and clayey soil have been chosen to represent the soil types. The soil material used have the properties of loose to medium dense soils. The buildings have been modeled as a two and a five storey building with typical material properties, which are more strong than the rural typical homes. Two earthquake excitations of Kocaeli and Elazığ have been applied to models and their results have been obtained and compared to each other. The results of the analyses have been shown and discussed in this chapter.

5.1.2 Studying the effect of the yield acceleration on slope displacements

The yield acceleration for seismic stability of slopes is defined as the threshold average acceleration acting upon a sliding mass above which permanent deformations occur. It corresponds to a factor of safety with respect to sliding of unity (Prater, 1979). One of the best methods for analyzing the effect of yield acceleration is the Newmark (1965) method. The purpose of the Newmark (1965) method is to estimate the slope deformation for those cases where the pseudostatic factor of safety is less than 1.0. The Newmark (1965) method works assuming that the slope deformation will happen only during those moments of the ground motion when the out-of-slope earthquake forces cause the pseudostatic factor of safety to become less than 1.0. When this happens, the slope will no longer be stable. The longer that the slope is subjected to a pseudostatic factor of safety below 1.0, the greater the slope deformation. For Elazığ case, the yield acceleration a_y and the upper bound of the permanent displacement d_{max} occurred in a slope failure are calculated utilizing the approach proposed by Newmark. Making use of the yield acceleration a yield coefficient of acceleration $k_y = \frac{a_y}{g}$ can be defined as well (Celep et al, 2010). When the maximum acceleration of an earthquake does not exceed the yield acceleration, it is expected that slope failure does not occur. Many different equations have been developed utilizing the basic Newmark (1965) method to evaluate the permanent displacement caused by an earthquake motion. Two of them which are defined by Newmark are:

$$d_{max} = 0.5 \frac{a_{max} \times v_{max}^2}{a_y^2} \quad (5.1)$$

$$\log d = 0.9 + \log \left[\left(1 - \frac{a_y}{a_{max}} \right)^{2.53} \left(\frac{a_y}{a_{max}} \right)^{-1.09} \right] \quad (5.2)$$

The magnitude of the slope deformation is related to the following factors:

- Horizontal yield acceleration a_y : The higher the horizontal yield acceleration, the more stable the slope is for any given earthquake.
- Peak ground acceleration a_{max} : The peak ground acceleration represents the highest value of the horizontal ground acceleration. The greater the difference between the peak ground acceleration a_{max} and the horizontal yield acceleration a_y , the larger the downslope movement will be.
- Length of time: The longer the earthquake acceleration exceeds the horizontal yield acceleration a_y , the larger the downslope deformation.
- Number of acceleration pulses: The larger the number of acceleration pulses that exceed the horizontal yield acceleration a_y , the greater the cumulative downslope movement during the earthquake.

To use the equations (5.1) and (5.2), the first step is to determine the pseudostatic factor of safety, using the methods proposed by Terzaghi and other researchers. Anytime the pseudostatic factor of safety is less than 1.0, the next step is to reduce the value of the seismic coefficient K_h until a factor of safety exactly equal to 1.0 is obtained. The value of K_h that corresponds to a pseudostatic factor of safety equal to 1.0, can be converted to the yield acceleration. Substituting the values of the peak ground acceleration and the yield acceleration into Eq. (5.1) or (5.2), the slope deformation can be determined in centimetres.

In equation (5.1), v_{max} and a_{max} correspond to the maximum velocity and acceleration of the earthquake motion. For the Kovancilar Earthquake, the numerical evaluation is carried out for the two records, namely, for the Palu-EW component of the Kovancilar Earthquake (ML = 5.8), and the Palu-NS component of the Palu Earthquake (ML = 5.6), (Table 5.1). For the evaluation of the yield coefficient of the acceleration, only the internal friction of soil is required and cohesion is neglected for the simplicity of the calculations (Celep et al, 2010).

Table 5.1 : Characteristic parameters of Kovancılar and Palu Earthquakes(Celep et al, 2010).

Place	Magnitude	Max Acc(cm/s^2)	Max Velocity (cm/s)	Max Disp(cm)
Kovancılar	5.8	64.5	5.9	1.17
Palu	5.6	74.9	10.8	1.49

Figure 5.3 indicates the relationship between the maximum displacement and coefficient of the yield acceleration. Figure 5.4 shows the variation of d_{max} with different values of a_{max}/a_y ratio. As it is seen, the maximum displacement decreases as the yield acceleration increases. The maximum displacements on the ground surface at foundation levels in the villages of Yukarı Demirci, Okçular and Kayalık were observed to be up to 20 cm, which is comparable to the calculated displacement values within a few centimeters. (Celep et al, 2010).

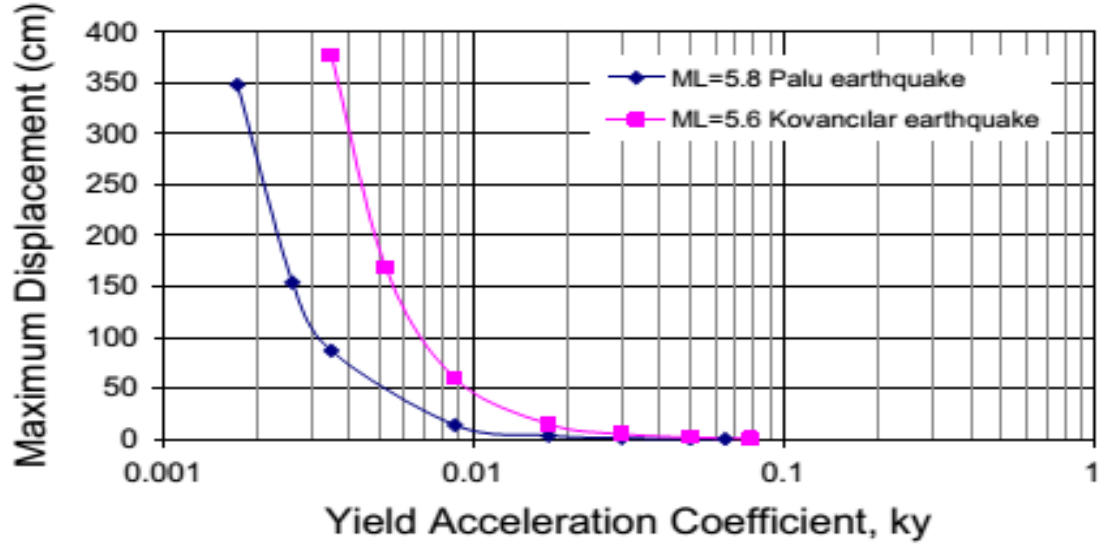


Figure 5.3 : The relationship between the yield acceleration coefficient and the expected maximum displacement (Celep et al, 2010).

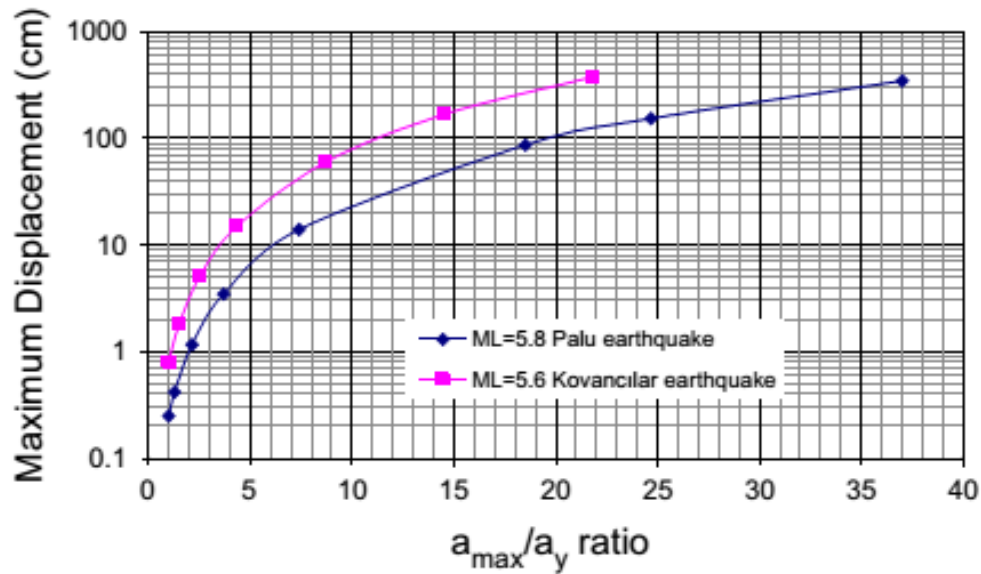


Figure 5.4 : The relationship between the maximum displacement and the a_{max}/a_y ratio (Celep et al, 2010).

5.1.3 Soil properties at studied area

Sieve analysis data and Atterberg limits test results were available from a fieldwork performed in the area. Sieve analysis curves have been shown in Figure 5.5. There are eight different soil samples from different locations in the affected area. All of grading curves have been shown in figure below. Table 5.2 shows the soil classification based on the available data.

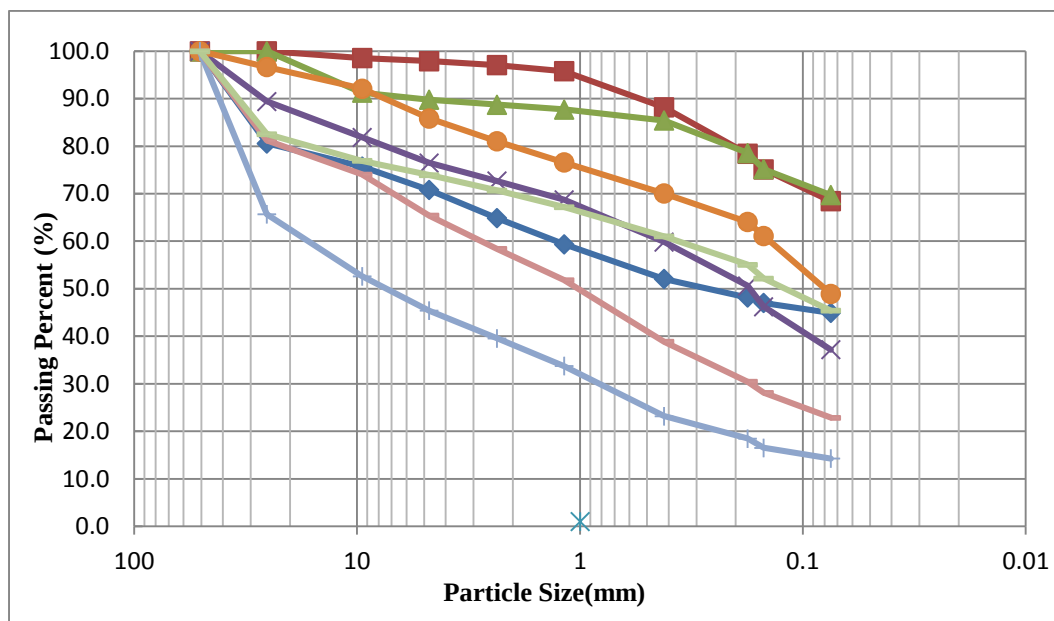


Figure 5.5 : Sieve analysis curves in the affected area.

Table 5.2 : Soil classification.

Sample	G(%)	S(%)	FC(%)	LL	PL	PI	Soil Type
Göçmezler	29	26	45	39	20	19	GC
Kayalık1	2	30	68	60	22	38	CH
Kayalık2	10	20	70	67	24	43	CH
Okçular1	24	39	37	30	19	11	SC
Okçular2	14	37	49	37	21	16	SC
Y.Demirci1	55	31	14	22	np	np	GM
Y.Demirci2	35	43	22	32	27	5	SM
Y.Kanatlı	26	29	45	40	20	20	SC

5.1.4 Selected soil properties

Soil parameters have been derived by estimations based on the properties of soil. Generally, there are 4 different soil types in the studied area. All of four soil types have been used in the modeling of the affected area. The gravely sand layer has been found in the same area which silty sand is dominant so it is not modeled separately in the analyses but its effect on parameters has been studied in the literature and the possible influence on material properties has been considered in analysis of silty sand layer. The same goes for clayey gravel layer which is found in the area that clayey sand type is dominant. Studied slopes generally consist of three main layers. Most of the slopes consist of a deep bedrock and weathered rock layer (soft rock), and the rest of slope is located on soft rock which are four different soil types listed above. The layers generally consist of a mixture of sand, clay and gravel. The thickness of layers decreases up to the hill and rubbles and sandstone and limestone are outcrop up to the hills.

5.1.4.1 Properties of bedrock

According to the Plaxis manual (2012), the best soil model to represent bedrocks is the linear elastic model so bedrock has been modeled using linear elastic soil model as a nonporous material. A typical value of shear wave velocity around 1000 m/s has been selected for rock layer. It also has to be mentioned that the rock layers have been modeled with HSsmall soil model too, but there was not any considerable difference in results. Table 5.3 represents the parameters of rock layer.

Table 5.3 : Properties of bedrock

Layer Type	γ	$E(kN/m^2)$	ν	$V_s (m/s)$	$G(kN/m^2)$
Bedrock	25	6.2E+06	0.2	1000	2.3E+06

The weathered rock (soft rock), layer has a decreased amount of shear wave velocity which is about 750 m/s and other stiffness parameters are decreased in accordance to this shear wave velocity value. In order to calculate the shear wave velocity using SPT N values, different formulations by numerous scientists have been proposed, which were discussed in previous chapters. In order to have an estimation of SPT N value for each of soil types, the related values based on the material and also the fact that studied soil types are categorized as soft to medium dense soils, have been derived from different sources.

5.1.4.2 Parameters of clayey sand (SC)

Parameters which have been used for modeling the soil layers in finite element program have been shown in Table 5.4. These are the average of all values based on the SPT N values for loose to medium dense soils, which could be used in the analyses based on the fact that the soil layers specifications vary from loose to medium dense soils, so there are other parameters that can be used in the analyses but all of them correspond to the characteristics of loose or medium dense soils.

Table 5.4 : Selected soil parameters for soil layers.

Material	Clayey Sand (SC)	Silty Sand (SM)	High Plasticity clay (CH)
N average	7	8	5
Vs (m/s)	175	180	150
Dr (%)	30	30	-
Material Model	HS Small	HS Small	HS Small
$\gamma_{\text{unsat}}(\text{kN}/\text{m}^3)$	17	18	20
$\gamma_{\text{sat}}(\text{kN}/\text{m}^3)$	19	20	20
ϕ	32	33	18
ψ	2	3	0
c (Kpa)	1	1	25
E (Kpa)	-	-	-
G (Kpa)	-	-	-
E_{50}^{ref} (Kpa)	22000	27000	20000
$E_{\text{oed}}^{\text{ref}}$ (Kpa)	22000	27000	23000
$E_{\text{ur}}^{\text{ref}}$ (Kpa)	66000	81000	69000
m	0.6	0.6	0.9
ν_{ur}	0.2	0.2	0.2
G_0^{ref} (Kpa)	65000	70000	50000
$\gamma_{0.7}$	0.00017	0.00017	0.0005

The layers have been modeled using HSsmall soil model as drained layers. Thickness of layer varies from 0 on the top of the slope where sandstone and limestone are outcrop, to several meters down to the hillside. Layers have been assumed to be loose so the related SPT value would be between 4 and 10 for sandy layers (Bowles 1988). An average SPT N value of 7 has been selected for the sandy layers and an average SPT N value of 5 for clayey layers have been chosen and calculations have been done based on selected values.

5.1.4.3 Parameters of silty sand (SM)

The silty sand layer is categorized as a sandy soil like clayey sand, but the influence of silt makes a little different, such as unit weight of soil or stiffness parameters. The parameters for slity sand layer is shown in Table 5.5.

Table 5.5 : Selected soil parameters for silty sand layer.

Material	Silty Sand (SM)
N average	8
Vs (m/s)	180
Dr (%)	30
Material Model	HS Small
$\gamma_{\text{unsat}}(\text{kN}/\text{m}^2)$	18
$\gamma_{\text{sat}}(\text{kN}/\text{m}^2)$	20
ϕ	33
ψ	3
c (Kpa)	1
E (Kpa)	-
G (Kpa)	-
$E_{50}^{\text{ref}}(\text{Kpa})$	27000
$E_{\text{oed}}^{\text{ref}}(\text{Kpa})$	27000
$E_{\text{ur}}^{\text{ref}}(\text{Kpa})$	81000
m	0.55
ν_{ur}	0.2
$G_0^{\text{ref}}(\text{Kpa})$	70000
$\gamma_{0.7}$	0.00017

5.1.4.4 Parameters of high plasticity clay (CH)

The high plasticity clay layer has some differences compared to previous layers. The parameters for this layer is shown in Table 5.6.

Table 5.6 : Parameters for clayey layer

Material	High Plasticity Clay(CH)
N average	5
Vs (m/s)	150
Dr (%)	-
Material Model	HS Small
$\gamma_{\text{unsat}}(\text{kN}/\text{m}^2)$	20
$\gamma_{\text{sat}}(\text{kN}/\text{m}^2)$	21
ϕ	18
ψ	0
c (Kpa)	25
E (Kpa)	-
G (Kpa)	-
$E_{50}^{\text{ref}}(\text{Kpa})$	20000
$E_{\text{oed}}^{\text{ref}}(\text{Kpa})$	23000
$E_{\text{ur}}^{\text{ref}}(\text{Kpa})$	69000
m	0.6
ν_{ur}	0.2
$G_0^{\text{ref}}(\text{Kpa})$	50000
$\gamma_{0.7}$	0.0005

5.1.5 Analysis on slopes with different materials

As it was mentioned before the analyses have been done in three series. The first series of analysis have been done on a typical 25 degrees slope. The second series have been done to evaluate the effect of external loading on slope under earthquake excitation. And last series have been done to evaluate the behavior of building on slopes. The results of all analyses will be presented in this chapter. The lists of performed analyses in this thesis are given in Tables 5.7 and 5.8. First table shows the list of analyses conducted using Plaxis version 2012 program. In second table, list of analyses done by GeoStudio program is given.

Table 5.7 : List of analyses performed by Plaxis.

Analysis Type	Structure/Load	Upper Soil	Slope Angle (°)
1	No	Sand	25
2	No	Clay	25
3	Yes / Upper Part	Sand	25
4	Yes / Middle Part	Sand	25
5	Yes / Upper Part	Clay	25
6	Yes / Middle Part	Clay	25
7	Yes / Two Storey	Clay	15
8	Yes / Five Storey	Clay	15
9	Yes / Two Storey	Clay	30
10	Yes / Five Storey	Clay	30
11	Yes / Two Storey	Sand	15
12	Yes / Five Storey	Sand	15
13	Yes / Two Storey	Sand	30
14	Yes / Five Storey	Sand	30

Table 5.8 : List of analyses performed by Geostudio.

Analysis Type	Structure / Load	Upper Soil	Slope Angle (°)
1	No	Sand	25
2	No	Clay	25

Figures 5.6 and 5.7, show the model and mesh used for first and second series of analyses. All three types of soil material have been applied in the model and analyzed under Elazığ earthquake excitation. The results were similar to the field work results. All of the slopes had a total displacement about 15 to 35 cm after termination of ground motion, which are in the range of the values obtained in the field work (around 20 cm). The analyses showed that behavior of slopes is related to the consistency of soil material. The results indicated that medium dense soils had less deformations unlike looser soils. All slopes had a safety factor around 1 to 1.3 after the end of earthquake excitation. These values explain the vulnerability of

slopes in the area during earthquake. The presence of ground water table has been considered in the models but as it was told in the previous chapters, it locates between the soil layer and the rock layer and analysis showed a few centimeter difference between these two conditions of considering or neglecting the ground water table in the model.

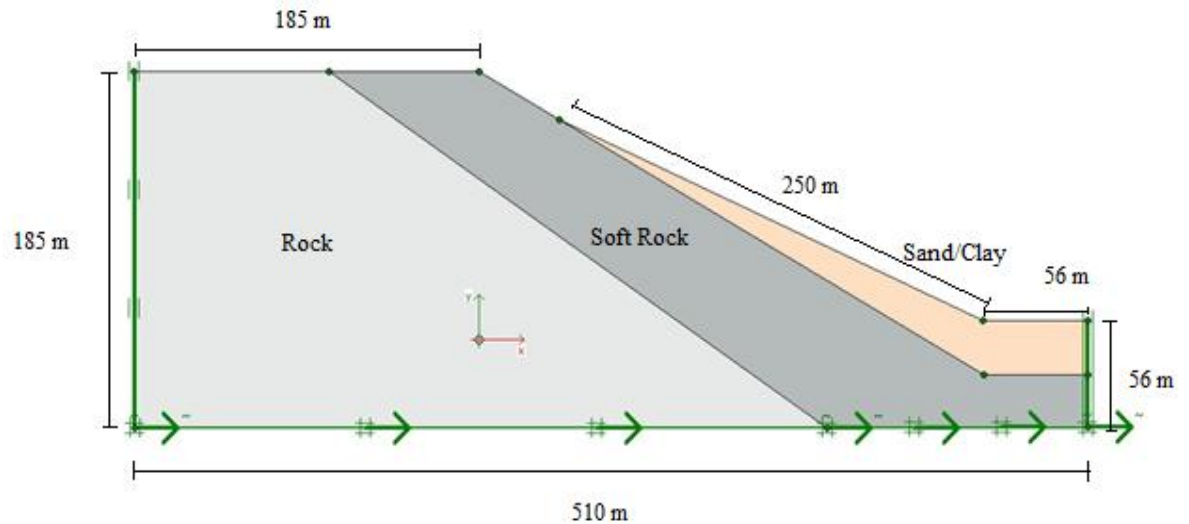


Figure 5.6 : Typical model of 25 ° slope.

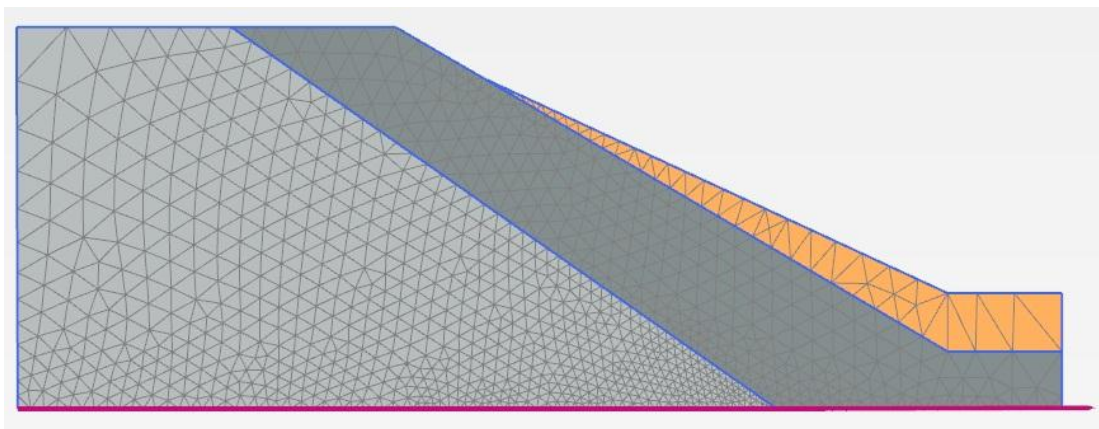


Figure 5.7 : Finite element model mesh.

Figure 5.8 shows the total displacement of a slope with clay material. Total displacements have been calculated on bottom, middle and upper parts of slope surface. The middle part of slope has the most total displacement with 35cm of deformation. Upper and bottom parts of slope have 11 cm of total displacements.

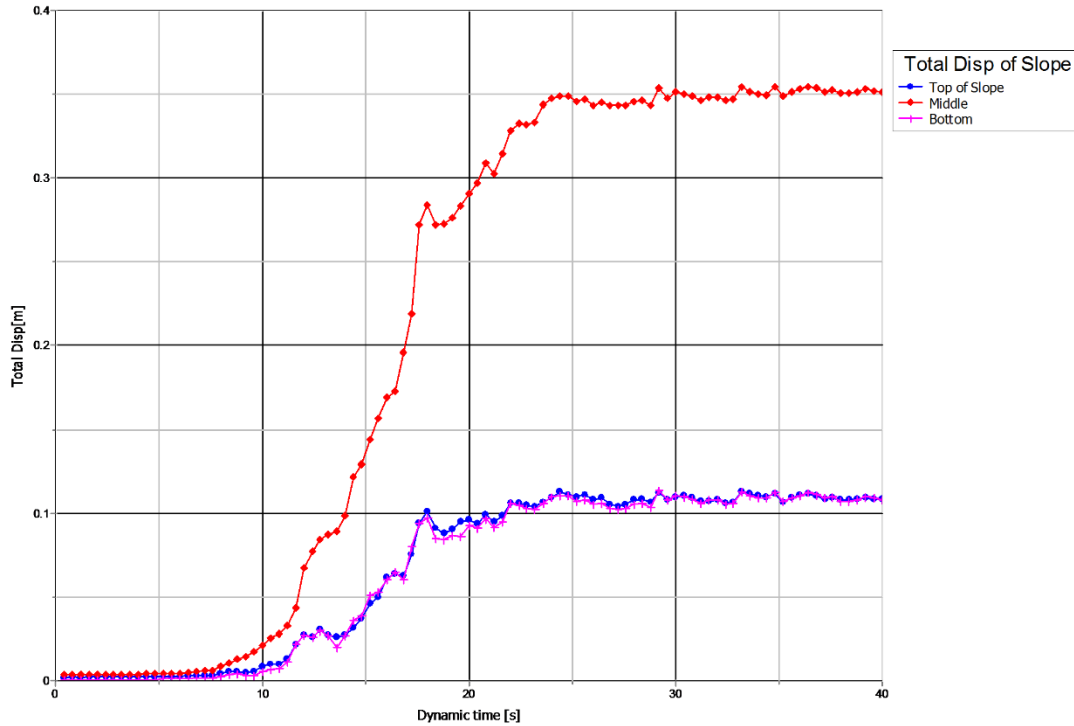


Figure 5.8 : Total displacement of clayey layers.

Results for sandy layer is shown in Figure A.6 in appendices. Maximum displacement in sandy layer is obtained to be 25 cm in the middle of slope, upper and bottom of slope are 16 cm and 7 cm respectively. Cross section of displacements for sandy and clayey layers are shown in figures A.7 and A.9 in appendices. In both of cross sections it is obvious that displacements in surface of slopes are more compared to the depth in which soil layer is close to the rock layer. Cross section of shear stresses in the model are shown in Figures A.8 and A.10 in appendices.

Figure 5.9 shows the safety factor of clayey layer, which is about 1.09 after earthquake. Deformations in sandy layers were less compared to clayey layers and obtained safety factors were around 1.4 on the end of earthquake.

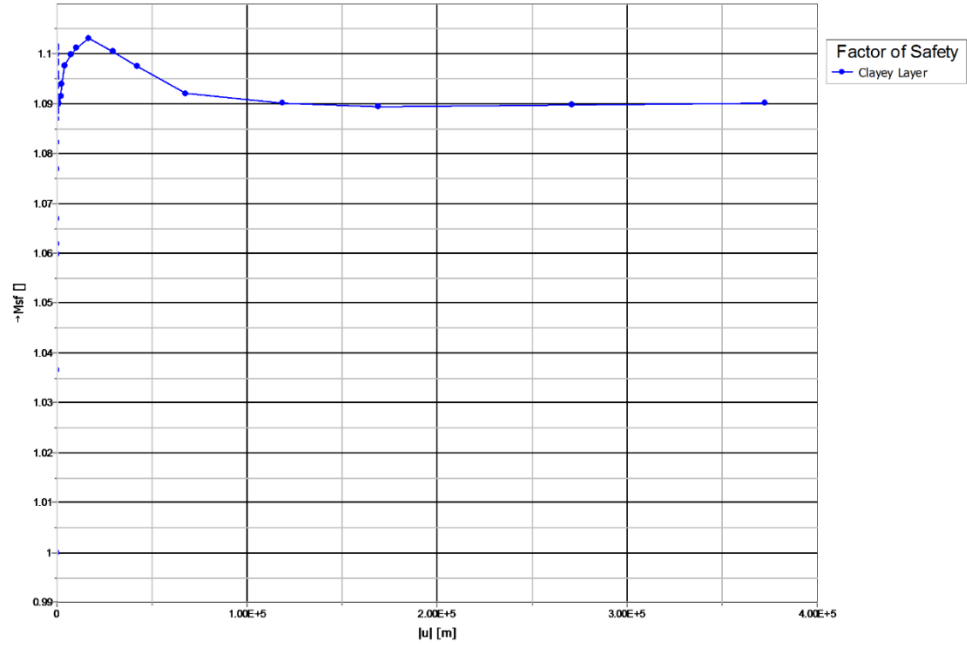


Figure 5.9 : Safety factor of clayey layer.

In order to validate the model and obtained results and also having a better understanding of behavior of slopes during earthquake, the slope have been modeled in GeoStudio 2012 computer program and Newmark analysis have been carried out. The model has the same geometry used in Plaxis analysis (Fig. 5.10). The results of analysis for sandy layer have been shown below.

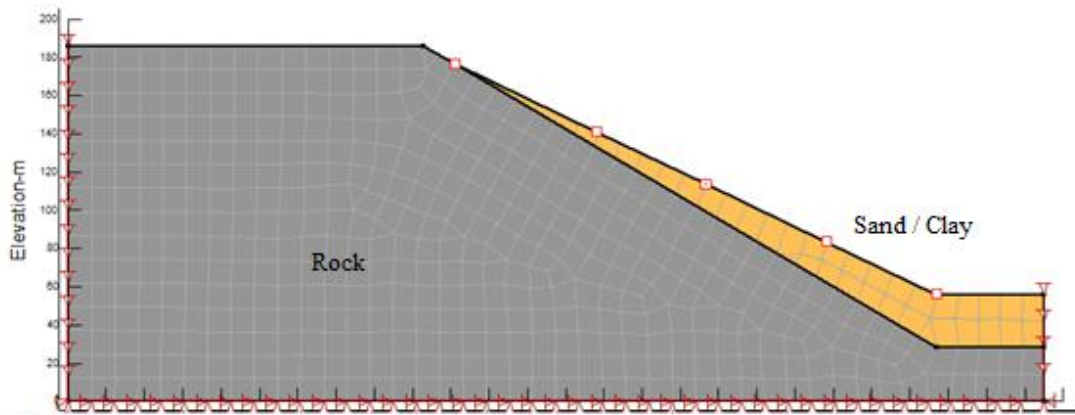


Figure 5.10 : Finite element model and mesh in Geostudio.

Figure 5.11 shows the deformation of sandy slope during earthquake excitation. The result indicates a 22 cm of displacement at the end of earthquake which is in the range of results obtained from Plaxis.

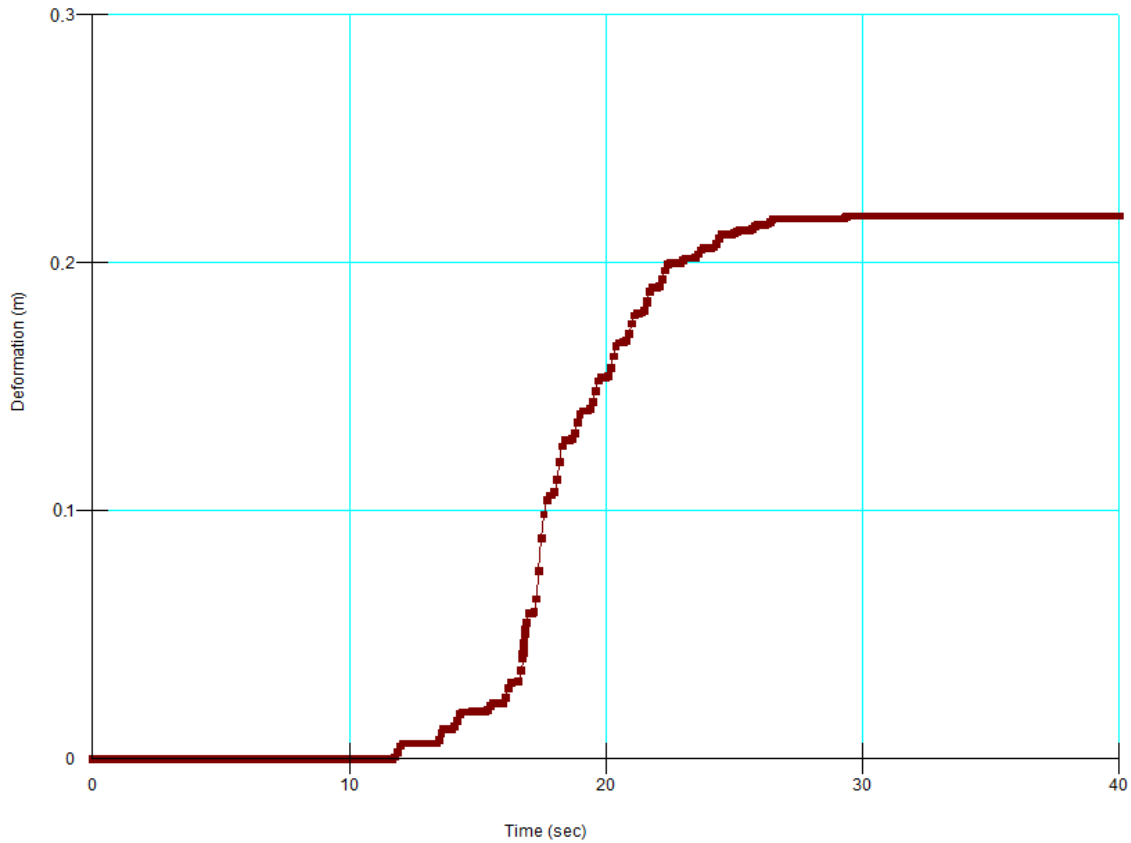


Figure 5.11 : Deformation of slope under Elazığ ground motion record.

The results for clayey layer is shown in figure A.11 in appendices. Analysis in Geostudio shows about 45 cm of displacement which is a bit more than results from Plaxis. Figure 5.12 shows the alteration of safety factor during time in slope. Safety factor falls below 1 frequently during earthquake and proves the instability of slope during ground motion. The least value is 0.77 which happens after 17 seconds from initiation of ground motion. Geostudio analysis result for clayey layer is shown in figure A.12 in appendices. The least value is 0.7 which happens after 16 seconds from initiation of ground motion. Overall safety factor is less compared to the sandy layer. Analysis for clayey layer showed that the safety factor values are below 1 during most of the earthquake time interval even in the last ten seconds of time interval.

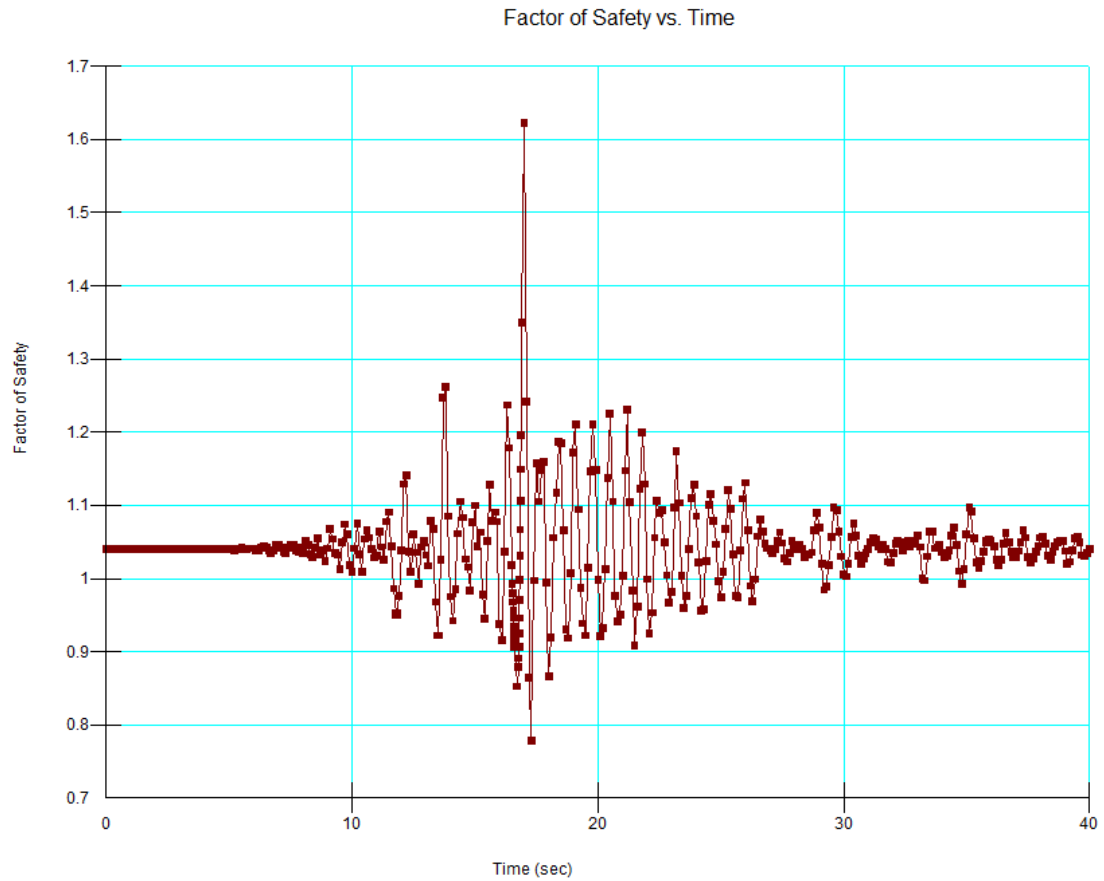


Figure 5.12 : Variation of safety factor of slope during earthquake.

Figure 5.13 shows the relation between average horizontal acceleration and factor of safety during earthquake. From the figure we can determine the yield acceleration of slope which is about 0.13 m/s^2 . Result for clayey layer is shown in Figure A.13 in appendices. The value for yield acceleration can be determined using the relation between average horizontal acceleration and factor of safety during earthquake. For clayey layer this value is about 0.05 m/s^2 . Compared to the value for sandy layer, it is understood that clayey layer could become instable with a less horizontal acceleration which is in accordance with the results for displacements.

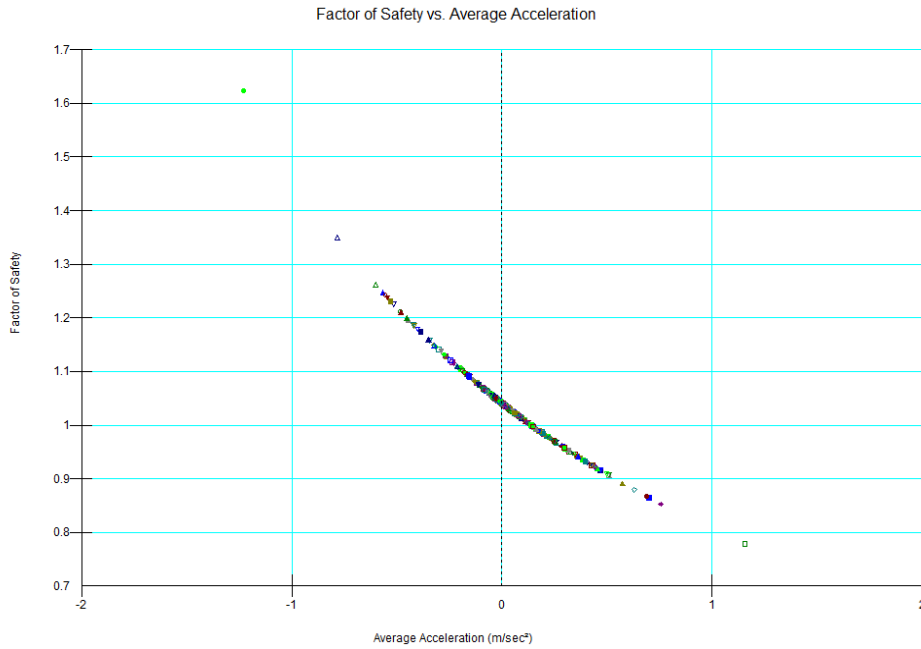


Figure 5.13 : Relationship between factor of safety and average acceleration.

5.1.6 Analysis of applied load on slope

The second series of analysis have been performed considering a distributed applied load of 15 kPa on a foundation having 10 meters of width and a thickness of 40 cm, which has been introduced in the model as a plate element. The analysis include two different conditions which account for the distance of foundation from rock layer and the elevation of located foundation. In another words, the importance of the location of foundations on slopes are studied. First the results of analysis on clayey soil type are shown. The analysis have been repeated for sandy layer. Figure 5.14 shows the applied load on the slope. In the first analysis, applied load is located in a higher elevation, where it is closer to the rock layer.

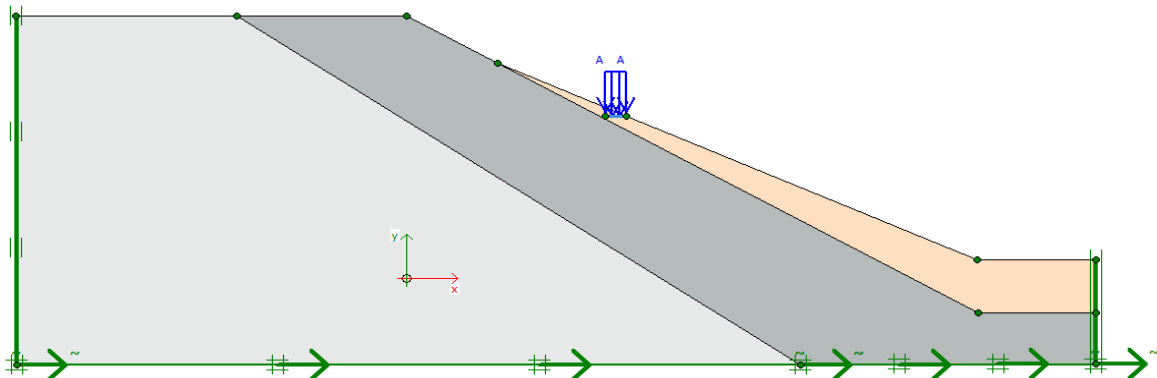


Figure 5.14 : Applying load on top of slope with clayey material.

Figures 5.15, 5.16 and 5.17 show the deformed mesh, displacement shadings and shear failure bands of slope. Figure 5.16 indicates that the maximum displacement of clayey layer is about 35 cm and it happens in the middle of slope affecting a range of 100 m on the surface of slope. Obviously the shear failure bands which used to be at middle part of slope, have been expanded toward the applied load and have caused the tilting of foundation.

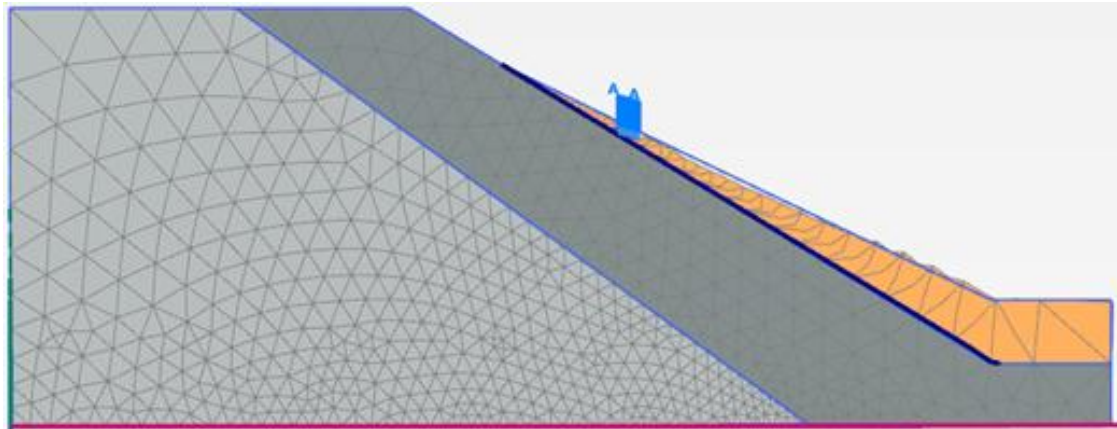


Figure 5.15 : Deformed mesh of layers.

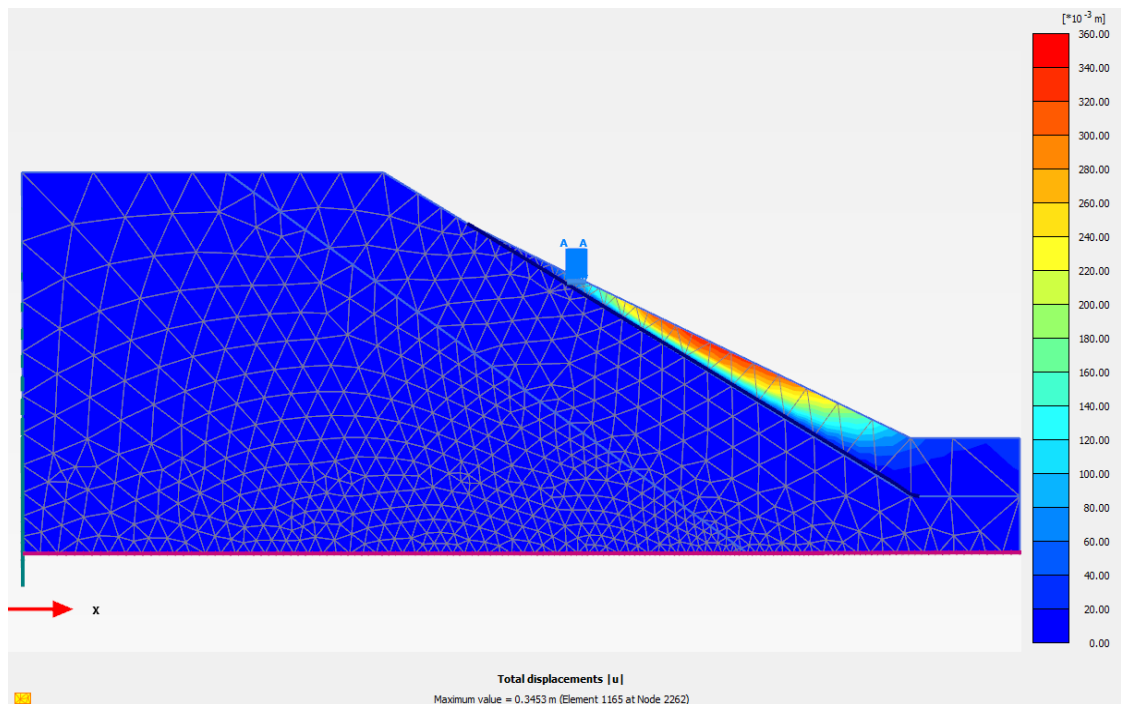


Figure 5.16 : Displacement shadings of layers .

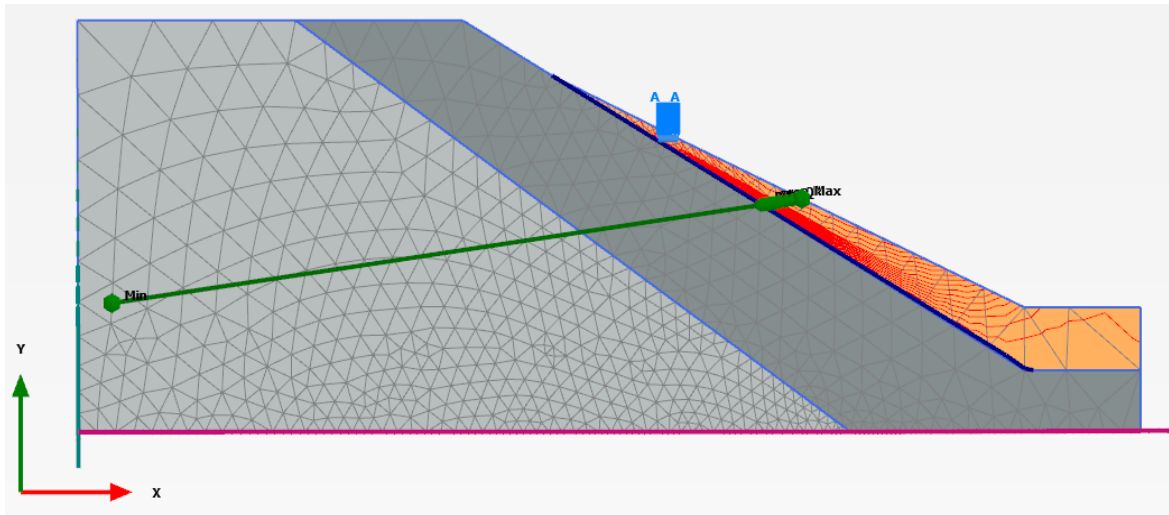


Figure 5.17 : Maximum displacement and shear failure bands of slope.

Figures 5.18, 5.19, 5.20 and 5.21 show the results for the condition that the applied load is located on the middle of slope, far from the rock layer. Displacements are about 50 cm which are more compared to the previous condition.

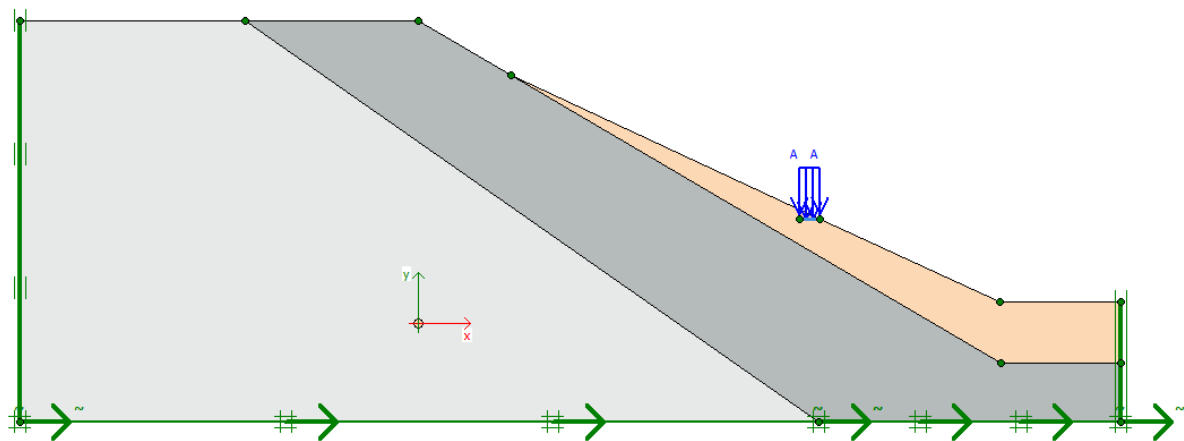


Figure 5.18 : Location of applied load in second analysis on clayey layer.

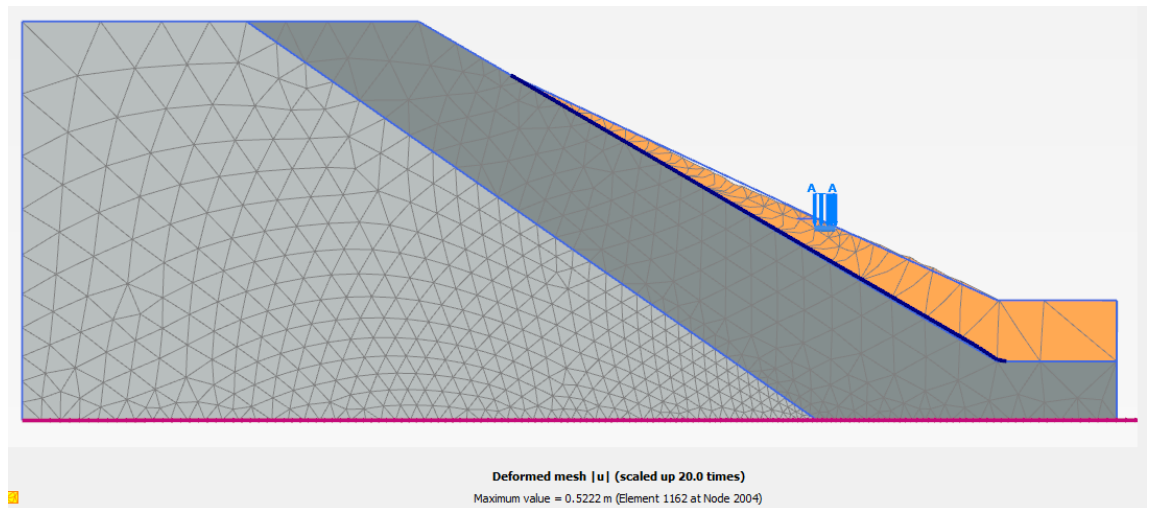


Figure 5.19 : Deformed mesh of layers.

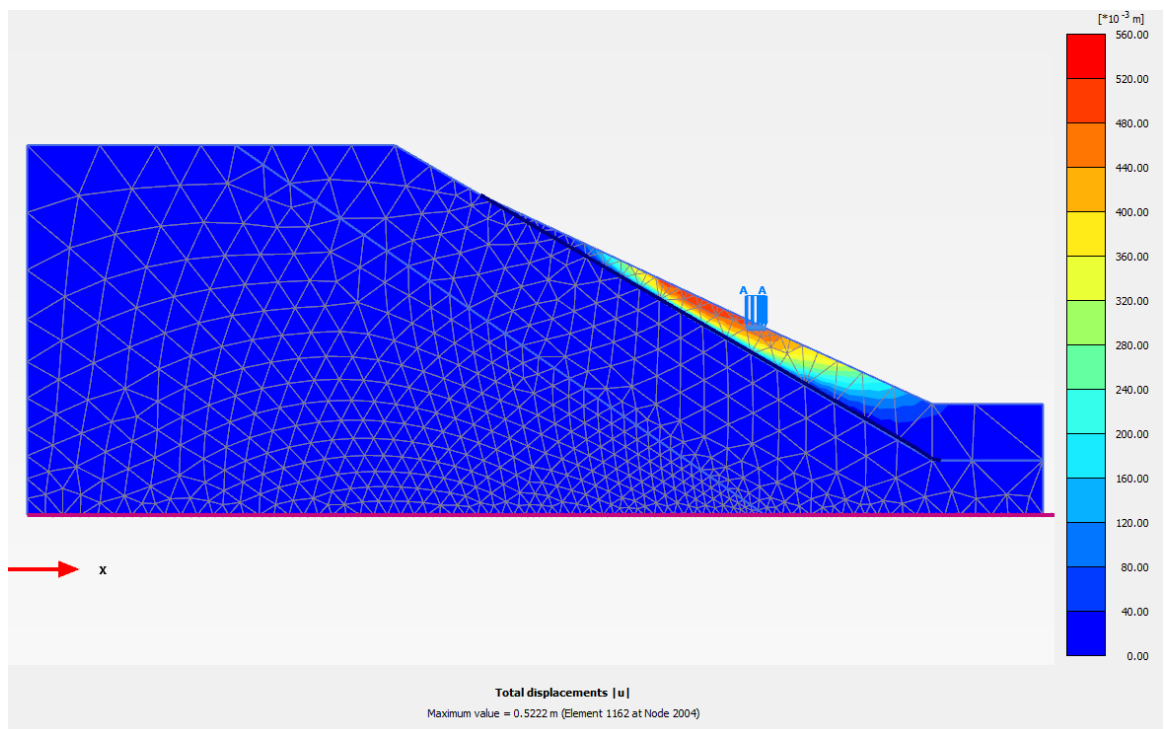


Figure 5.20 : Displacement shadings of layers .

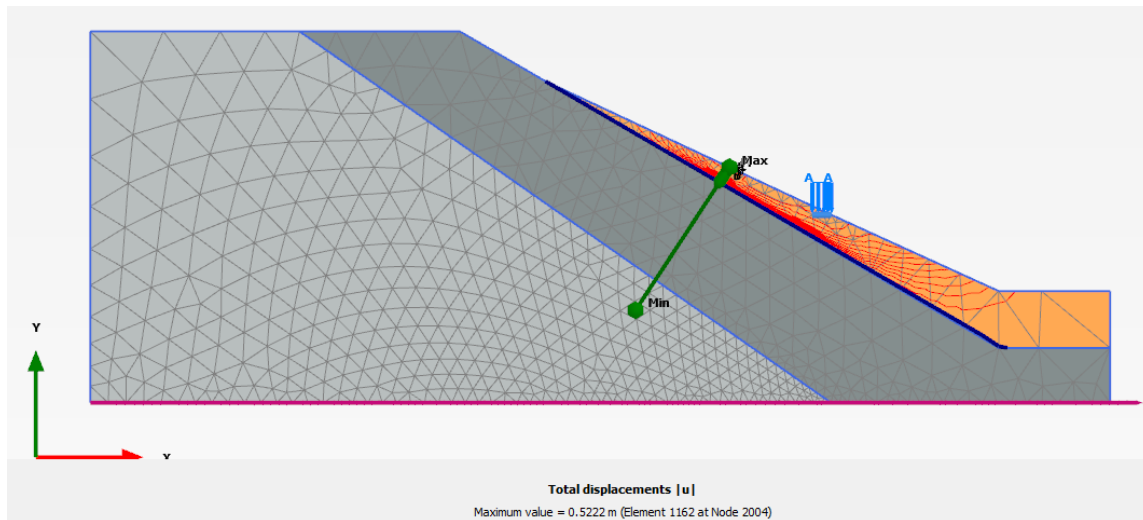


Figure 5.21 : Maximum displacement and shear failure bands of slope.

Figure 5.22 shows the safety factors for both conditions. For the condition that foundation is located on the middle of slope, factor of safety is about 1.14 and in other case factor of safety is about 1.18.

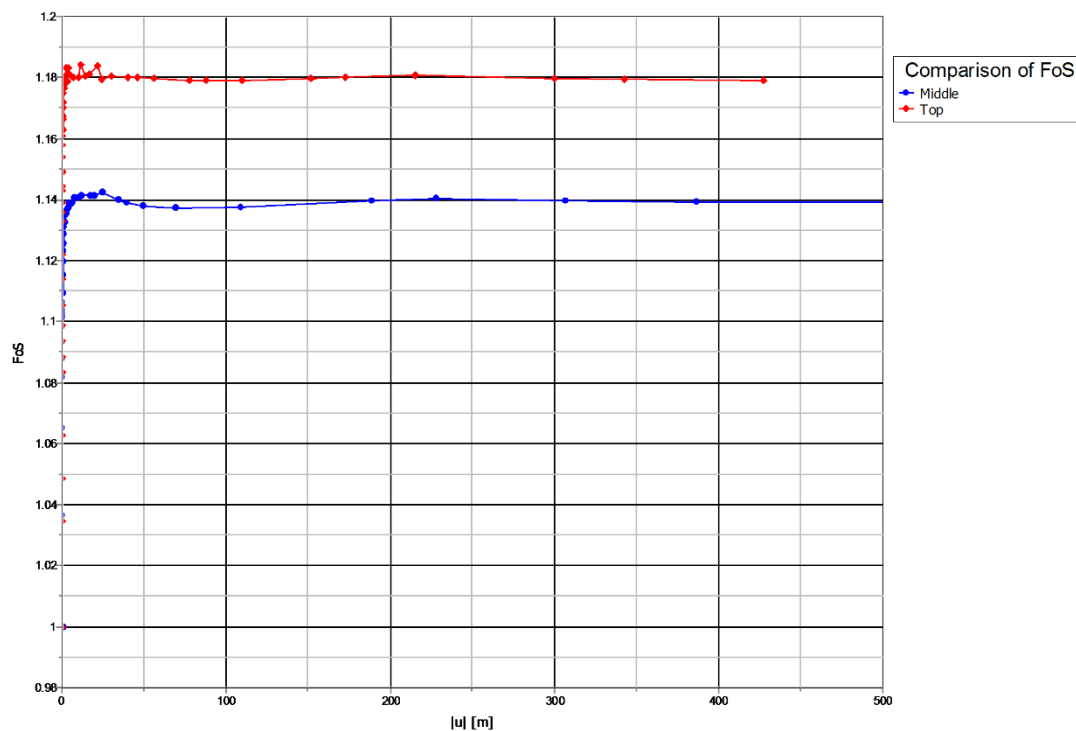


Figure 5.22 : Comparison of safety factors for clayey layers.

Figures 5.23 and 5.24 show the rotation of foundations in both conditions. Obviously when the foundation is located at more elevated part of slope it has more rotation.

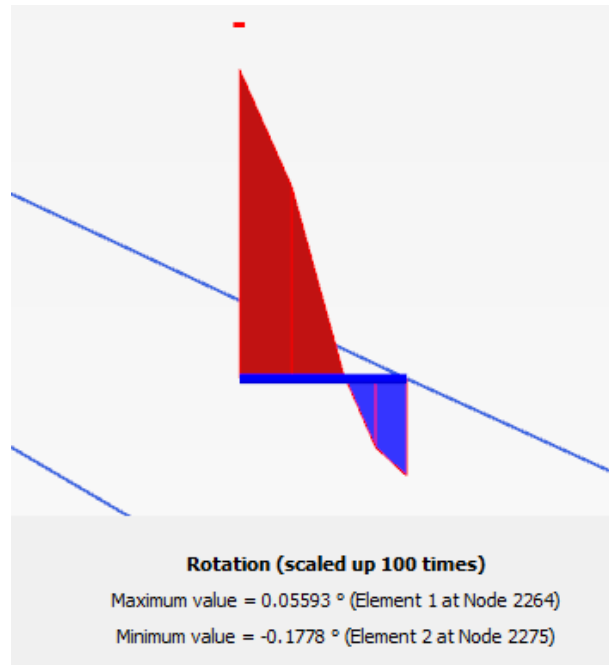


Figure 5.23 : Rotation of foundation in case it locates on the middle of slope.

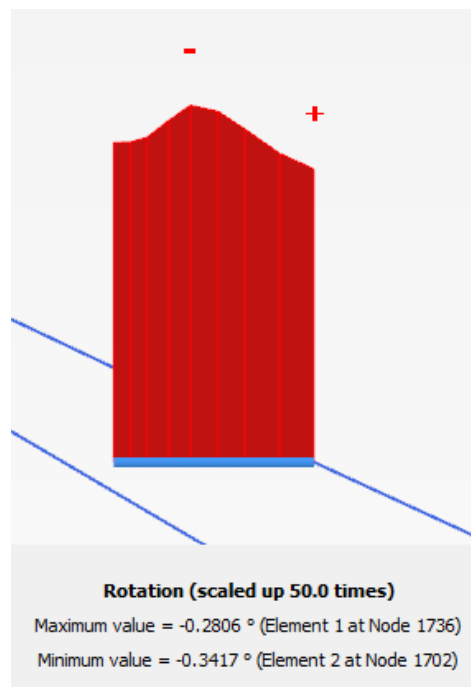


Figure 5.24 : Rotation of foundation in case it locates on top of slope.

The analysis have been repeated for sandy layer. The same procedure has been followed and results has been shown below. Fig. 5.25 shows the finite element model with the sandy layer.

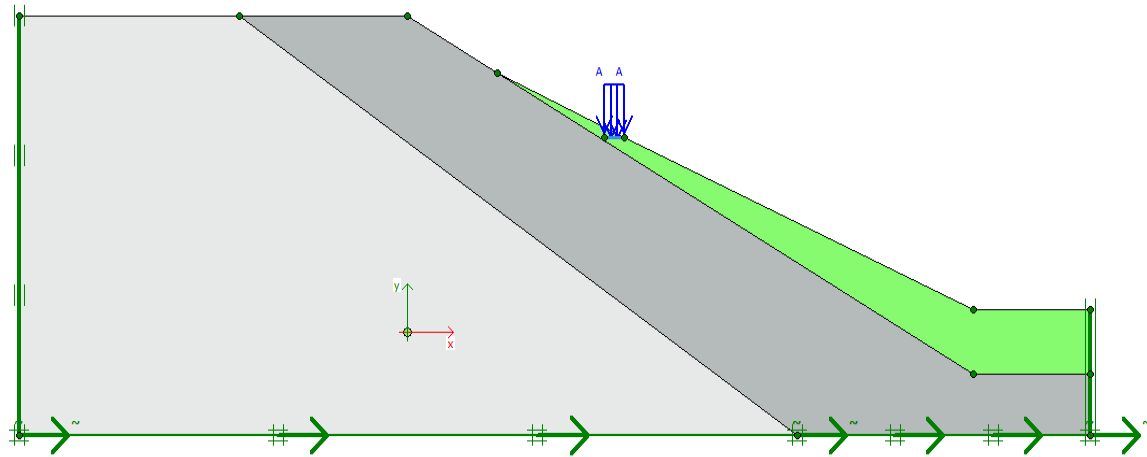


Figure 5.25 : Applying load on top of slope with sandy material.

Figures 5.26, 5.27 and 5.28 show the deformed mesh, displacement shadings and maximum displacement of slope.

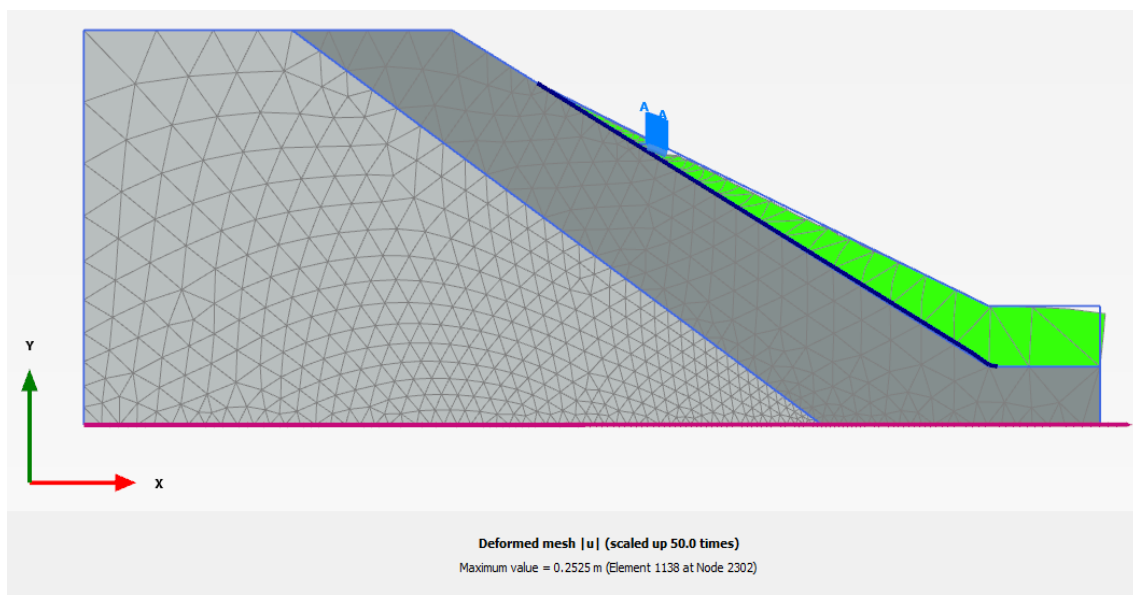


Figure 5.26 : Deformed mesh of layers.

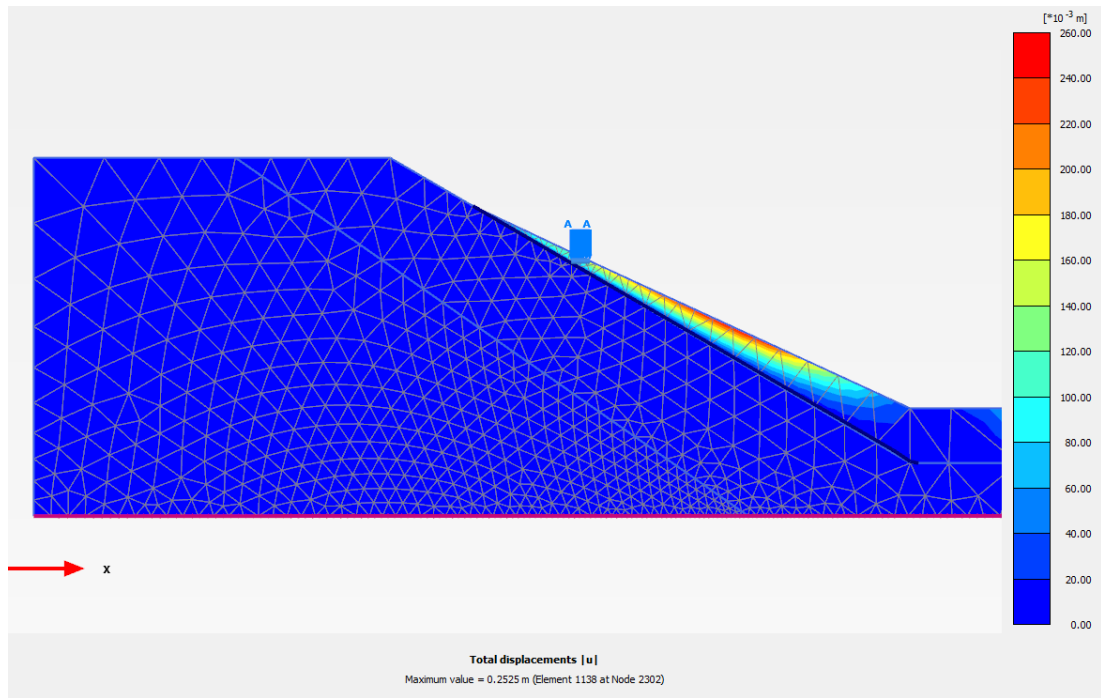


Figure 5.27 : Displacement shadings of layers .

Figure 5.28 indicates that the maximum displacement of sandy layer which is around 25 cm, occurs on the middle of slope and again on the surface of slope.

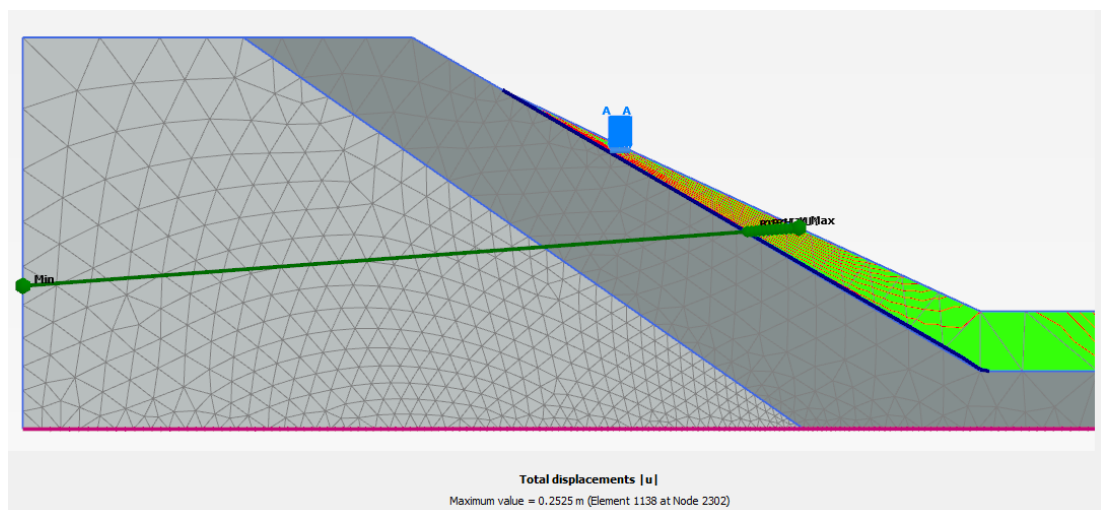


Figure 5.28 : Maximum displacement and shear failure bands of slope.

. Fig 5.29 shows the effect of the applied load on soil underneath the foundation.

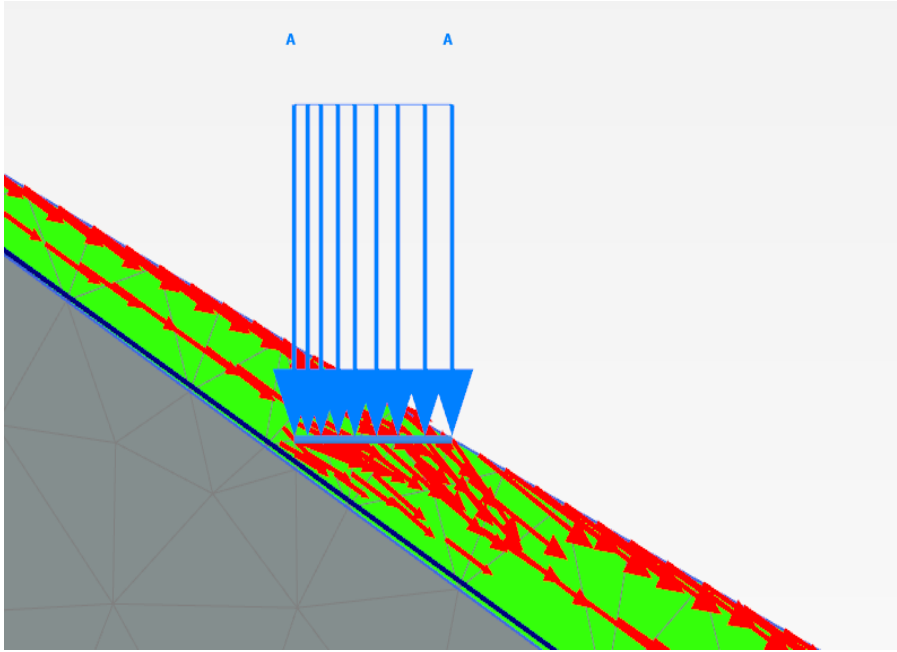


Figure 5.29 : The effect of applied load on soil underneath the foundation.

Figures 5.30, 5.31 and 5.32 show the results for the condition that the applied load is located on the middle of slope and far from the rock layer. Displacements are less (20cm), compared to the clayey layer. Another difference is how the sandy and clayey layers act under earthquake excitation. In clayey layers it seems that most particles move connected to each other and have a very similar displacement but for sandy case it is unlike the clayey case and particles move on top of each other.

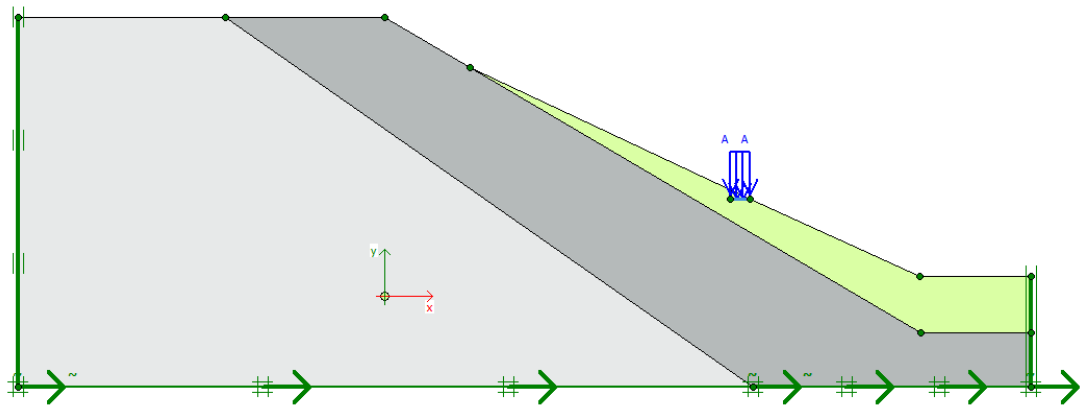


Figure 5.30 : Location of applied load in second analysis on sandy layer.

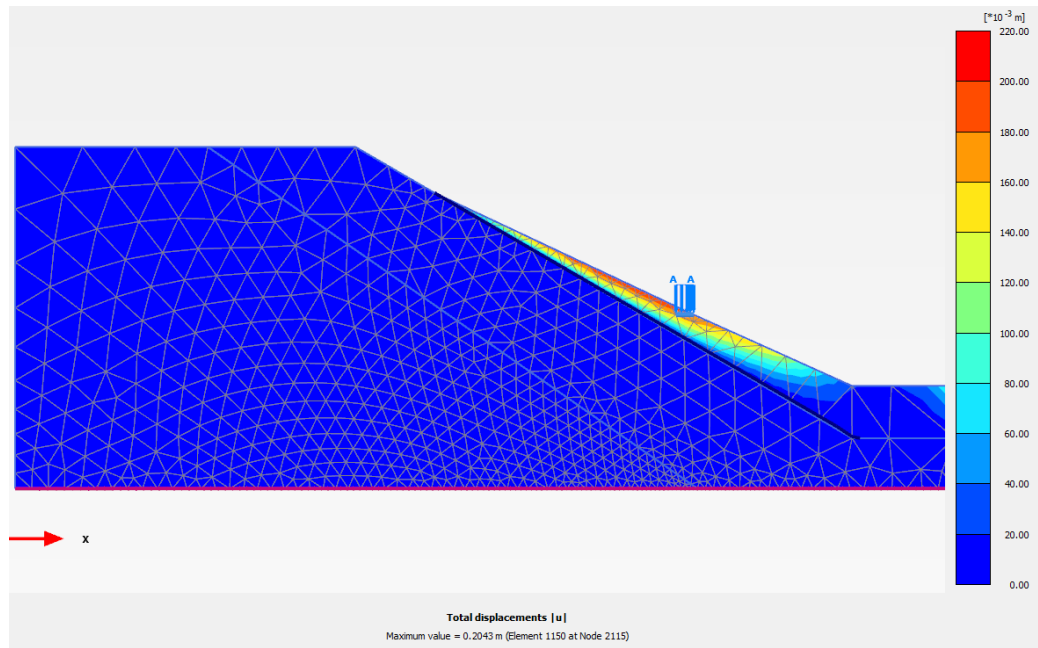


Figure 5.31 : Displacement shadings of layers.

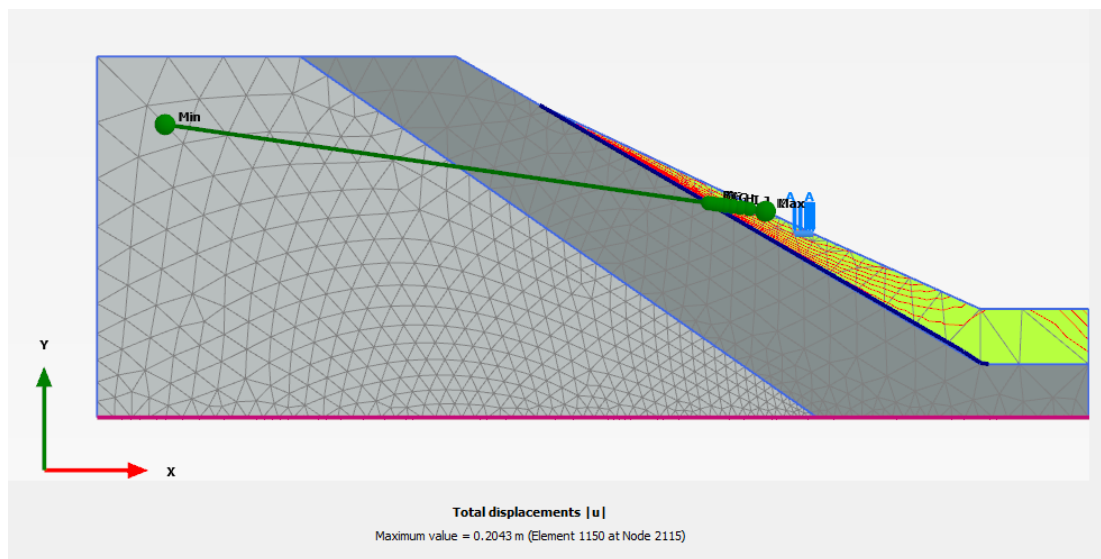


Figure 5.32 : Maximum displacement and shear failure bands of slope.

Fig. 5.33 shows the safety factors for both conditions. Both of them are around 1.35.

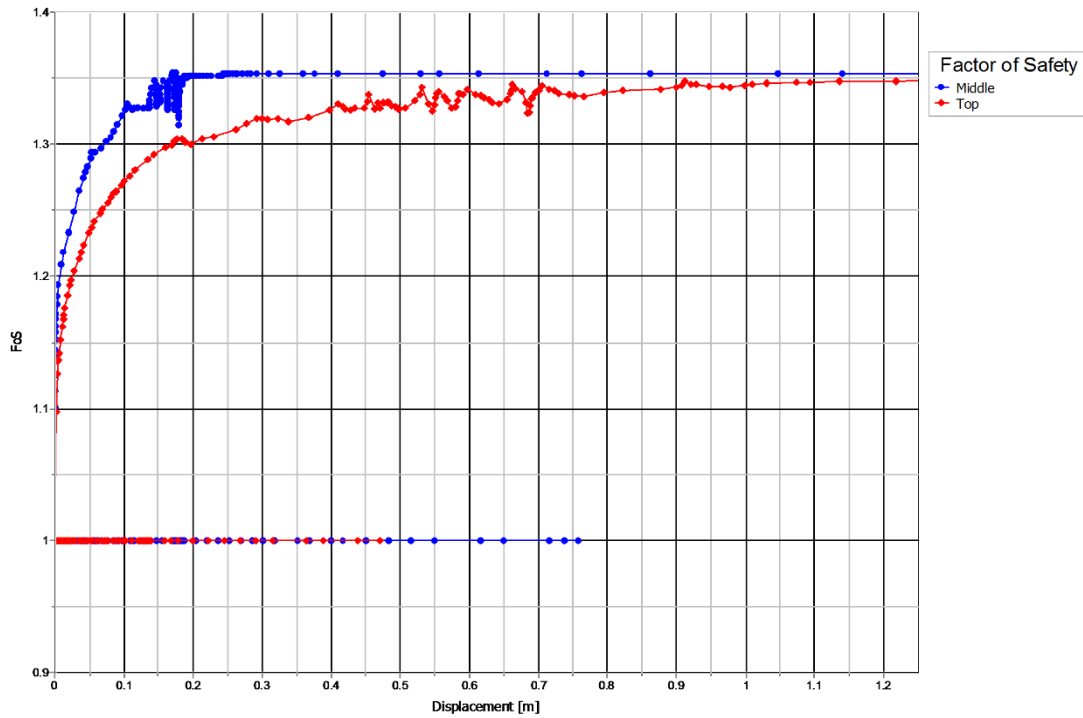


Figure 5.33 : Comparison of safety factors for sandy layer.

Figures 5.34 and 5.35 show the rotation of foundations in both conditions. Obviously when the foundation is located at the upper part of slope it has more rotation. The direction of rotation is different in this case compared to clayey case. In sandy case, right side of foundation rotates downwards but in clayey case it rotates upwards

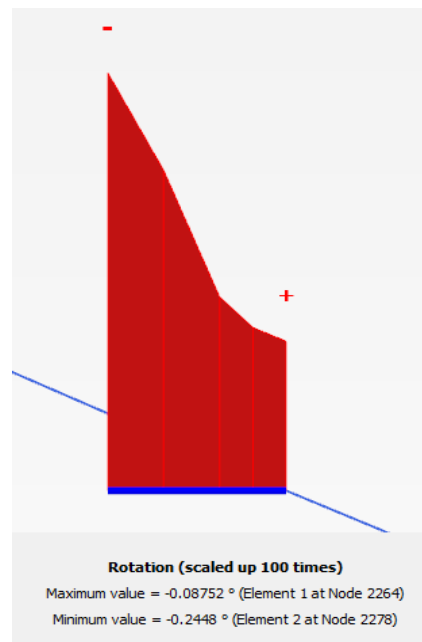


Figure 5.34 : Rotation of foundation when it locates on the middle part of slope.

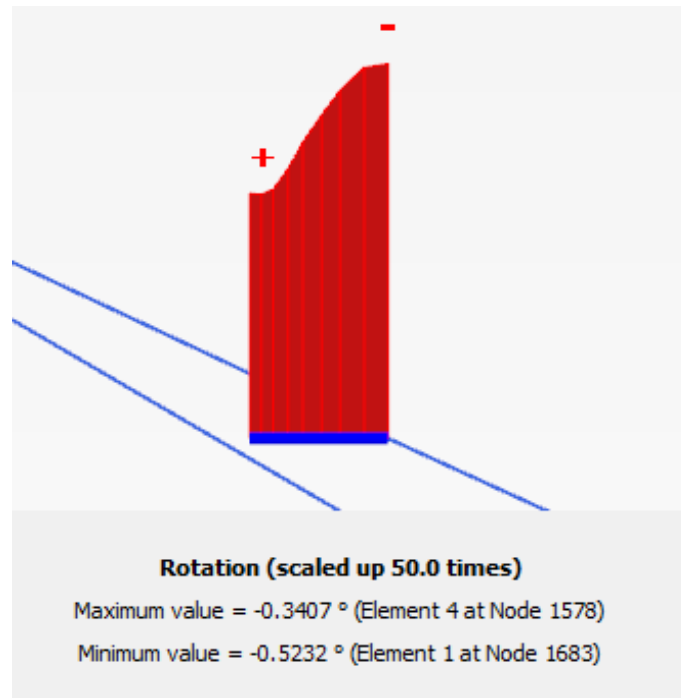


Figure 5.35 : Rotation of foundation when it locates on elevated part of slope.

A comparison of displacements of foundations in four different mentioned cases is shown in figure A.14 in appendices. The figure indicates that the condition in which the foundation is located on the middle part of clayey layer has the most displacement which is around 45 cm. In situations that the foundation are located on middle part of silty layer, upper part of silty layer and upper part of clayey, displacements are 17, 13 and 10 cm respectively. The thickness of both sand and clay layers are more in the middle of slopes compared to the upper part of slope so this is the reason that much more displacements are observed in the middle of slopes.

5.1.7 Dynamic behavior of buildings on slopes

The last series of analyses have been done to study the behavior of buildings on slopes. Two different slopes inclinations have been chosen as 15 and 30 degrees. The soil materials also have been selected as a sandy layer and a clayey layer. Soft to medium stiff parameters have been used to obtain the best results.

First the analyses have been conducted on a slope with 15 degrees inclination. Two and five storey buildings have been modeled on sandy and clayey layers and also two earthquake excitation of Elazığ and Kocaeli have been applied to model. The Kocaeli excitation has been chosen as a stronger ground motion case and the results of applying this accelerogram have been compared to main Elazığ case. The second

analysis have conducted on a slope with 30 degree inclination. The same procedure have been done with this case. Displacements at the end of ground motion are compared to each other.

The modeled buildings consist of 2 or 5 floors with a basement. The width of buildings is considered to be 10 meters. Each storey has a 3 meters of height. The building is modeled with plate elements. The material parameters for modeling the buildings are given in Table 5.9. The weight of floors and walls are considered to be $5kN/m^2$. A model with a length of 300 m has been chosen for modeling. The height of 15 degrees slope is 74m and height of 30 degrees slope is 140m. The building locates in the middle of models. The boundaries are placed in a distance of ten times the buildings width to have the appropriate distance for dynamic analysis.

Interface elements are also used in basement floor to model the interaction between soil and building.

Table 5.9 : Properties of building material.

Parameter	Name	Plates of building	Plates of basement	Unit
Material type	Type	Elastic,Isotropic	Elastic	-
Normal Stiffness	EA	$8.0 \cdot 10^6$	$1.3 \cdot 10^7$	kN/m
Flexural rigidity	EI	$6.0 \cdot 10^4$	$1.7 \cdot 10^5$	kNm ² /m
weight	W	10	20	kN/m/m
Poisson's ratio	ν	0	0	-
Rayleigh damping	α	0.23	0.23	-
	β	$8.0 \cdot 10^{-3}$	$8.0 \cdot 10^{-3}$	

The models used in analysis have been shown in Figures 5.36, 5.37, 5.38 and 5.39. All of them have been analyzed with both Elazığ and Kocaeli earthquake excitations and also with two different sandy and clayey soil materials. The results are shown in the next pages.

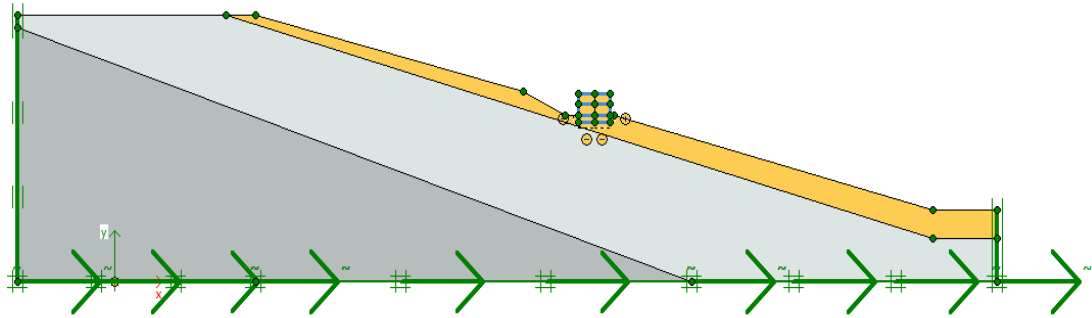


Figure 5.36 : Finite element model of 2 storey building on a slope with 15 degrees of inclination.

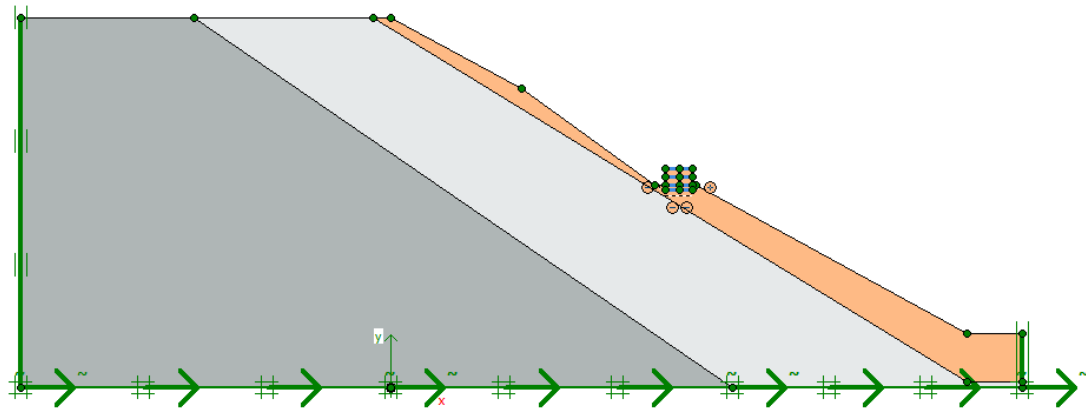


Figure 5.37 : Finite element model of 2 storey building on a slope with 30 degrees of inclination.

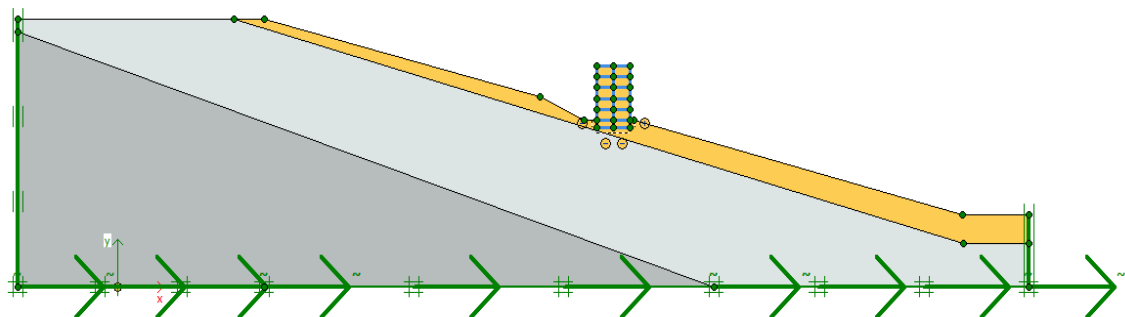


Figure 5.38 : Finite element model of 5 storey building on a slope with 15 degrees of inclination.

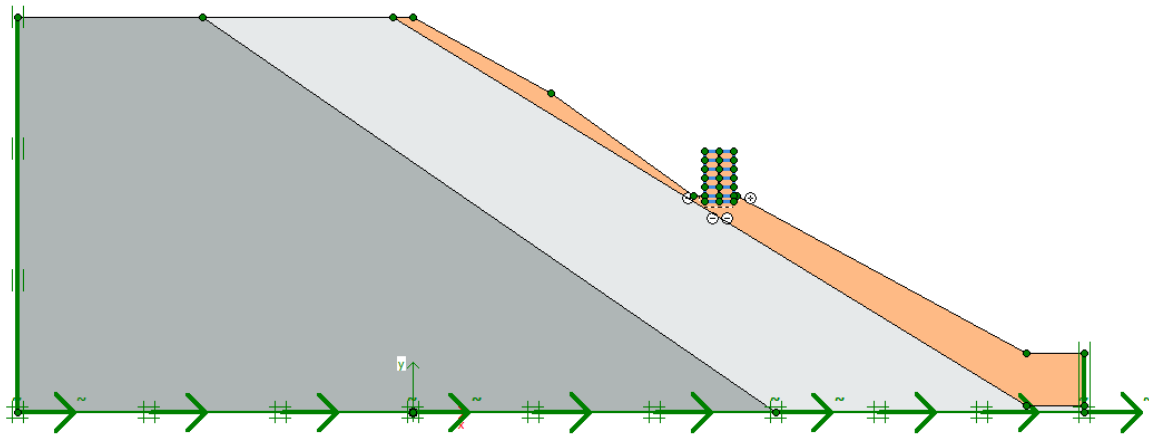


Figure 5.39 : Finite element model of 5 storey building on a slope with 30 degrees of inclination.

Figures 5.40, 5.41, 5.42, 5.43, 5.44, 5.45, 5.46 and 5.47 show the vertical and horizontal displacements of top floor resulted in all of the analyses. Table 5.10 summarizes the displacements resulted from both Elazığ and Kocaeli ground motions. Obviously there is a considerable difference in results if they are compared to each other from four different aspects of inclination of slopes, the motion records, type of soil materials and height of buildings. Figure 5.40 shows the comparison between vertical displacements of top of buildings on a slope with 15 degrees of inclination under Elazığ ground motion record. In this case the 5 storey building on sandy soil has the most displacement which is less than 1 cm and not so harmful.

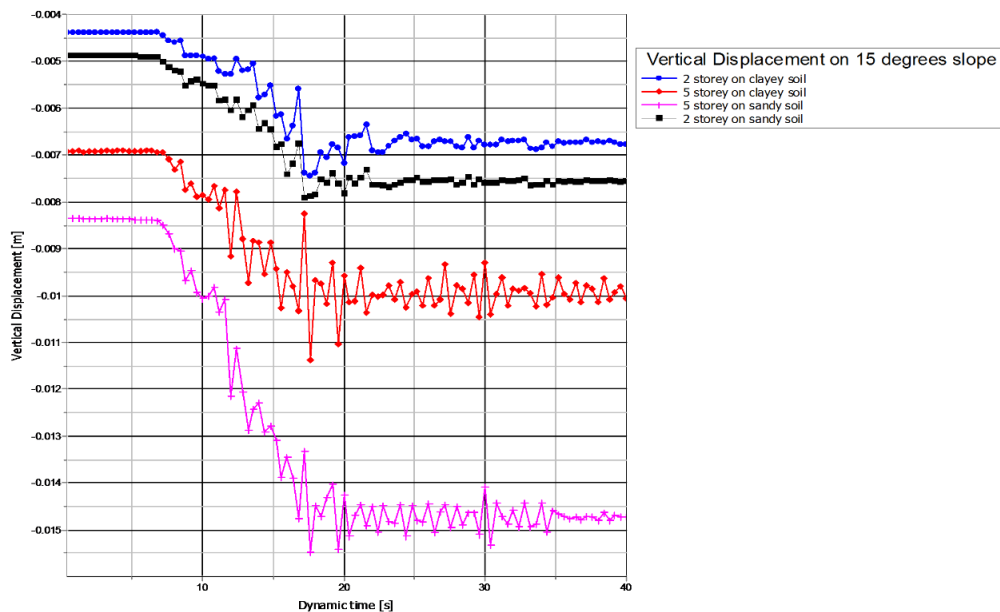


Figure 5.40 : Vertical displacement of top of buildings under Elazığ earthquake.

Figure 5.41 shows the comparison between vertical displacements of top of buildings on a slope with 30 degrees of inclination under Elazığ ground motion record. In this case the 5 storey building on clayey soil has the most displacement which is about 10 cm. So importance of slope inclination is considerable. Figure 5.42 shows the comparison of displacements on 15 degrees inclined slope but this time under Kocaeli ground motion record. In this case the 5 storey building on clayey soil has the most displacement which is about 8 cm. So the effect of a stronger ground motion is understood.

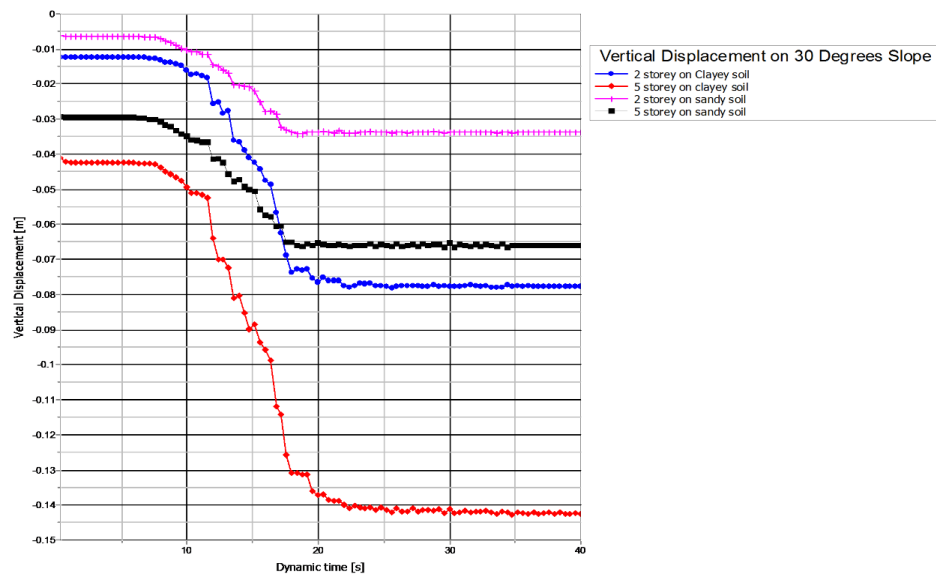


Figure 5.41 : Vertical displacement of top of building under Elazığ earthquake.

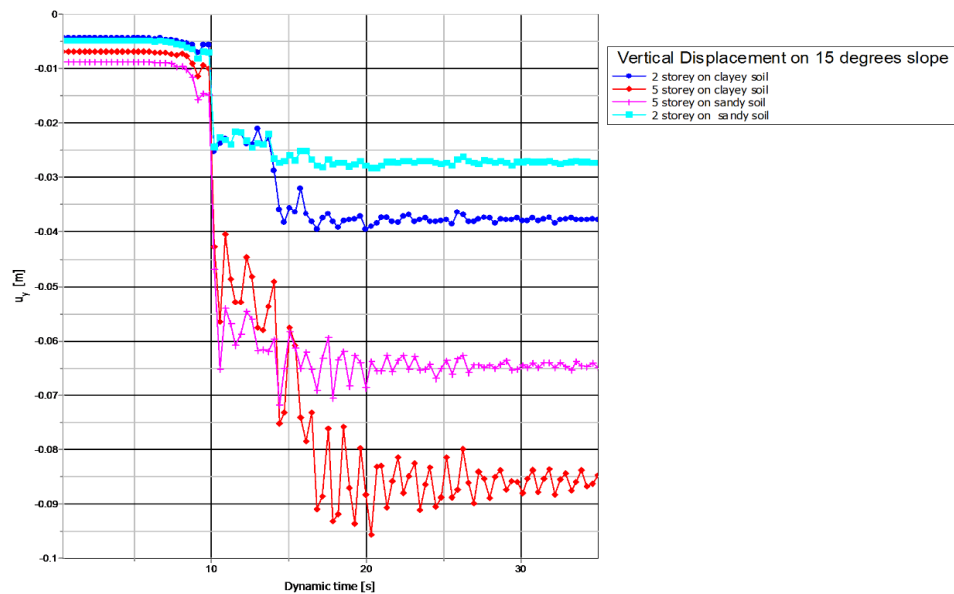


Figure 5.42 : Vertical displacement of top of building under Kocaeli earthquake

Figure 5.43 shows the comparison between vertical displacements of top of buildings on a slope with 30 degrees of inclination under Kocaeli ground motion record. In this case the 5 storey building on clayey soil has the most displacement which is about 2m. Comparison between horizontal displacements are shown in Figures 5.44-5.47.

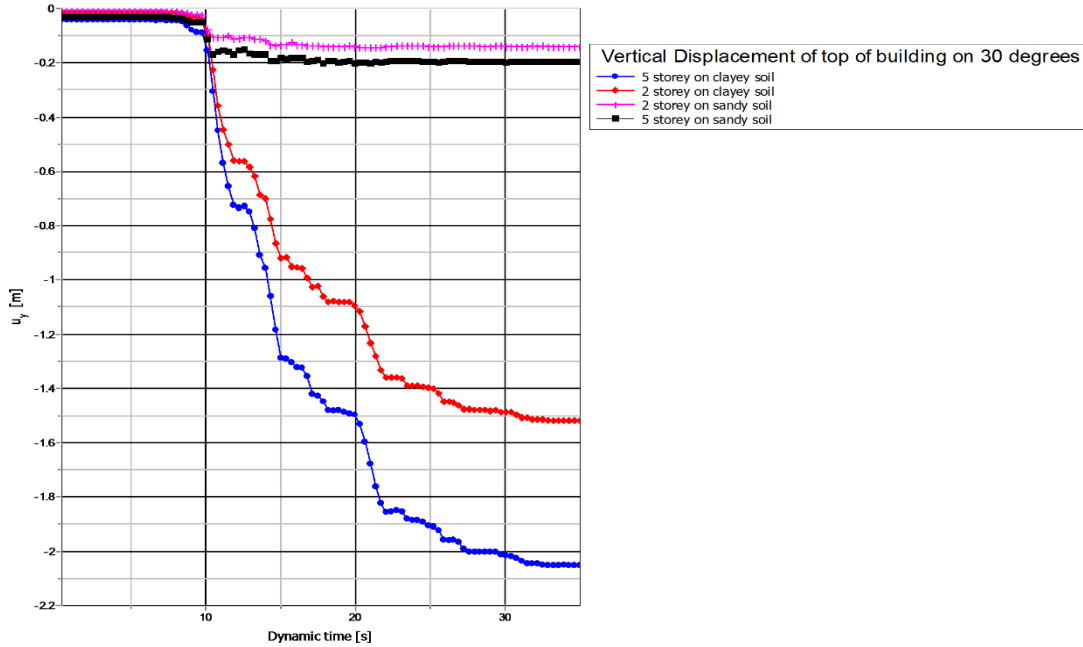


Figure 5.43 : Vertical displacement of top of building under Kocaeli earthquake

In case the building is located on a 15 degrees inclined slope under Elazığ ground motion (Fig. 5.44), maximum horizontal displacement happens when the 5 storey building is located on sandy soil and the displacement is about 2 cm. Analysis for the same case with Kocaeli ground motion record, resulted in 20 cm of horizontal displacement for the 5 storey building on clayey soil (Fig. 5.46). In 30 degrees inclined slopes, 5 storey building on clayey soil has the maximum horizontal displacement with the value of 15 cm for Elazığ ground motion record (Fig. 5.45), and 3m for Kocaeli ground motion record (Fig. 5.47).

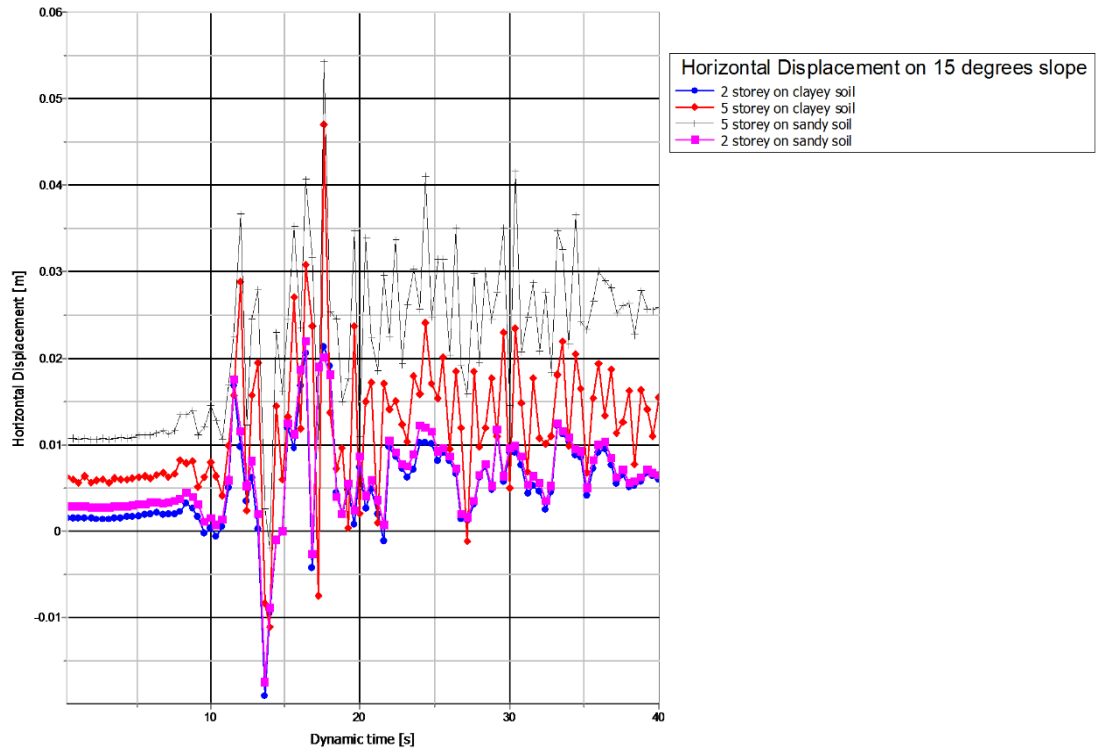


Figure 5.44 : Horizontal displacement of top of building under Elazığ earthquake

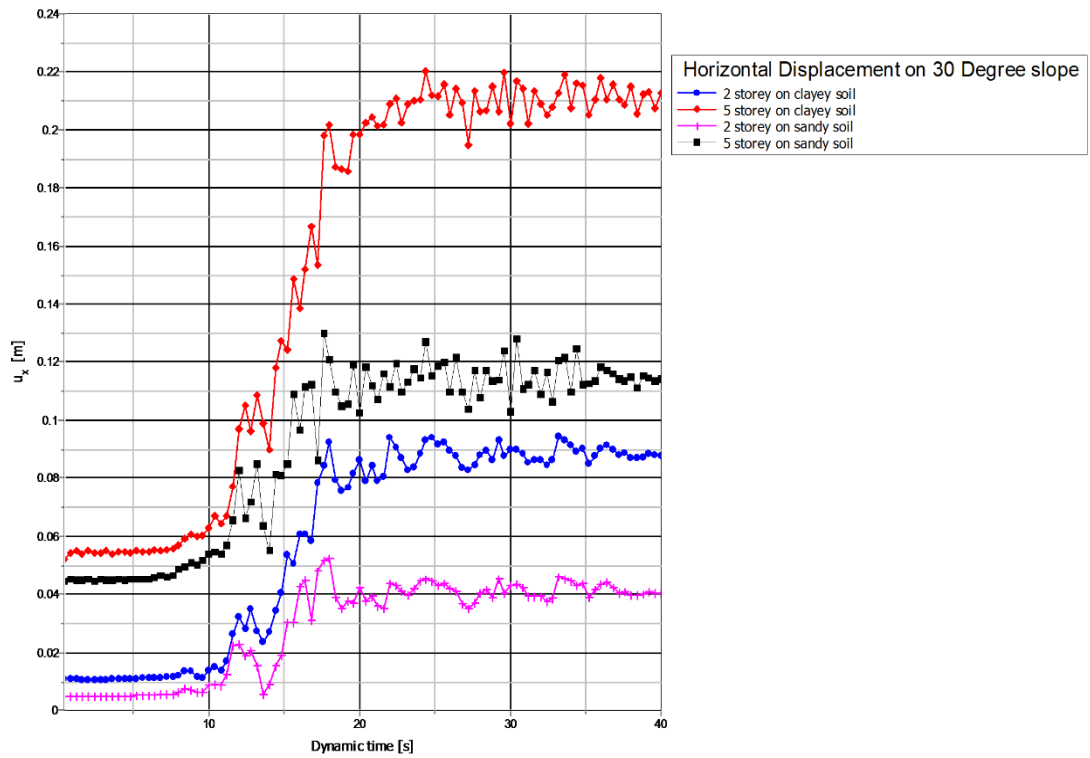


Figure 5.45 : Horizontal displacement of top of building under Elazığ earthquake.

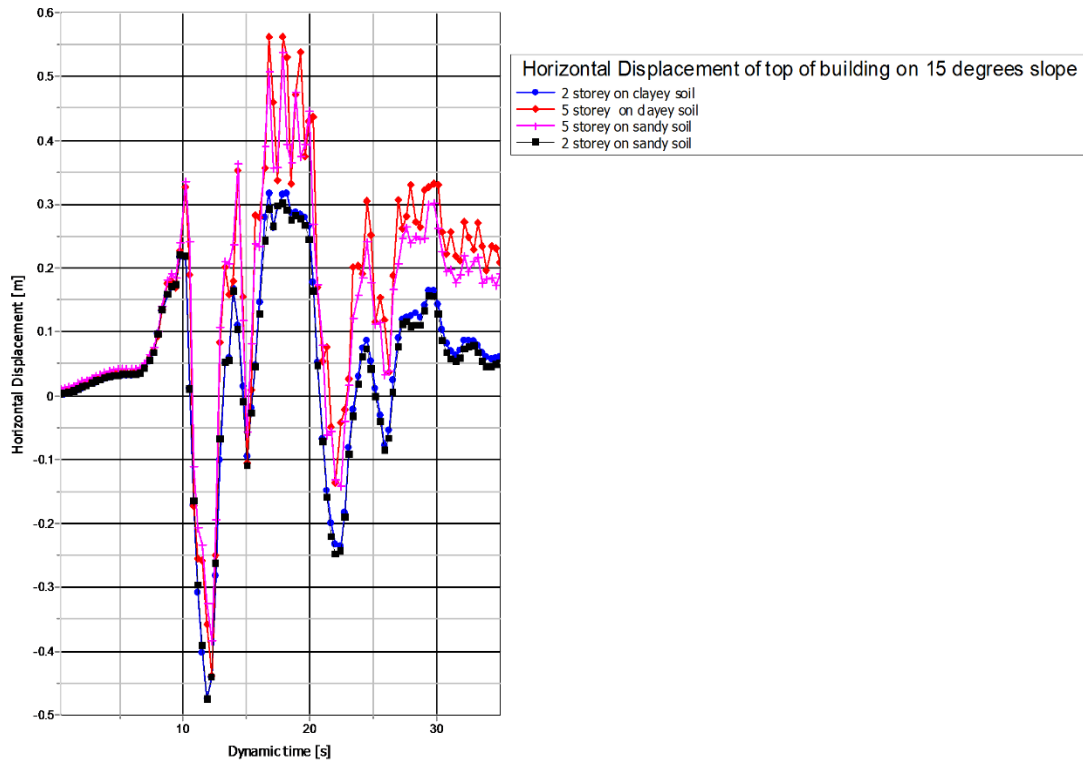


Figure 5.46 : Horizontal displacement of top of building under Kocaeli earthquake.

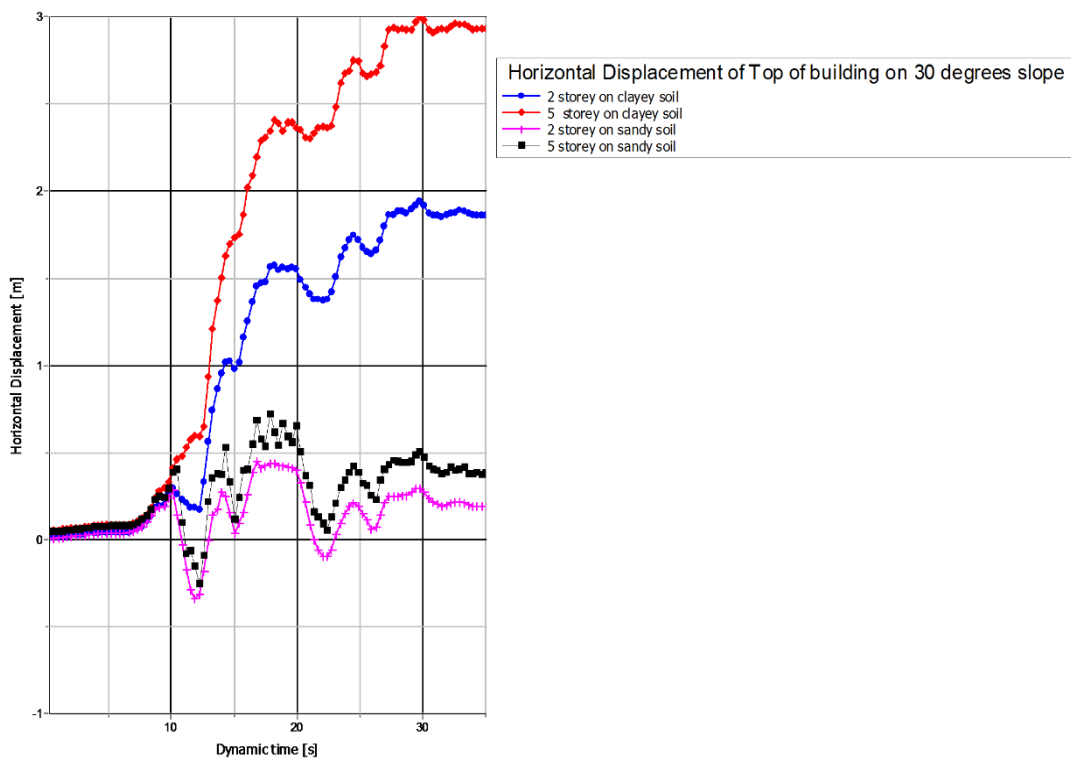


Figure 5.47 : Horizontal displacement of top of building under Kocaeli earthquake.

Table 5.10 : Results of analysis for 15 and 30 degrees inclined slopes respectively.

Soil Type	Storey	Eq Record Motion	Displacement(m)	
			Ver	Hor
Sandy	2 Storey	Elazığ	0.003	0.008
		Kocaeli	0.027	0.05
	5 Storey	Elazığ	0.006	0.011
		Kocaeli	0.065	0.19
Clayey	2 Storey	Elazığ	0.0025	0.007
		Kocaeli	0.03	0.06
	5 Storey	Elazığ	0.003	0.01
		Kocaeli	0.085	0.2
Soil Type	Storey	Eq Record Motion	Displacement(m)	
			Ver	Hor
Sandy	2 Storey	Elazığ	0.031	0.034
		Kocaeli	0.15	0.25
	5 Storey	Elazığ	0.031	0.06
		Kocaeli	0.2	0.4
Clayey	2 Storey	Elazığ	0.06	0.08
		Kocaeli	1.5	1.8
	5 Storey	Elazığ	0.1	0.14
		Kocaeli	2.00	2.7

Figures 5.48, 5.49, 5.50 and 5.51 show the tilting of buildings (visually), on slopes with different types of soil materials. The figures have the same scale factor.

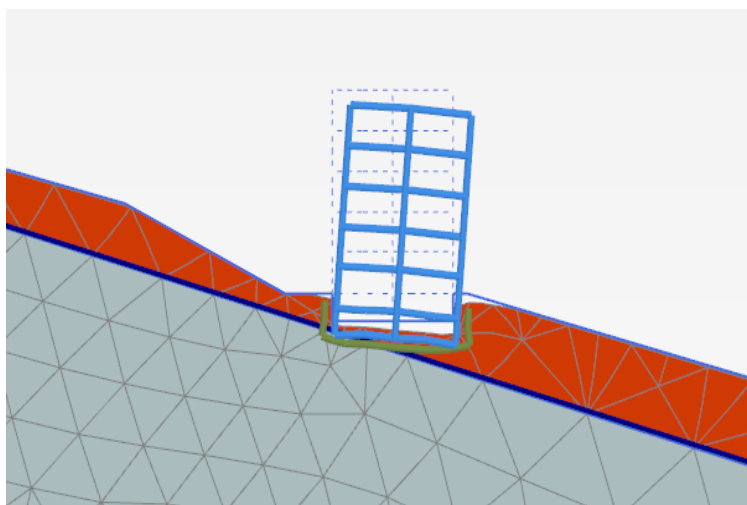


Figure 5.48 : Tilting of a five storey building on a 15 degrees inclined clayey slope

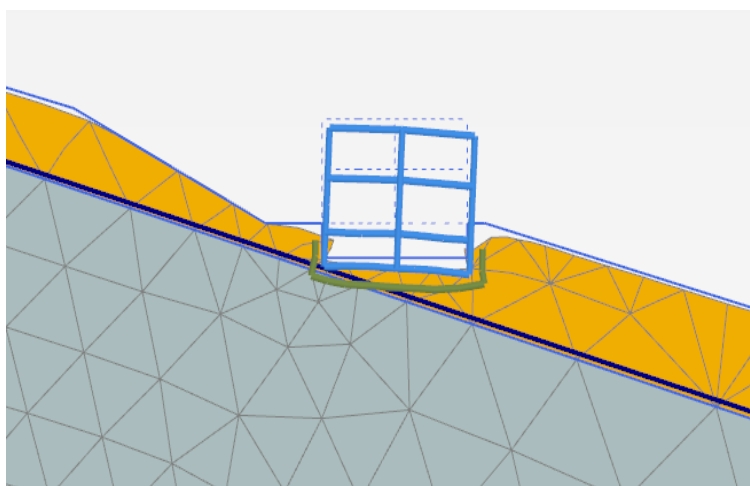


Figure 5.49 : Tilting of a two storey building on a 15 degrees inclined sandy slope.

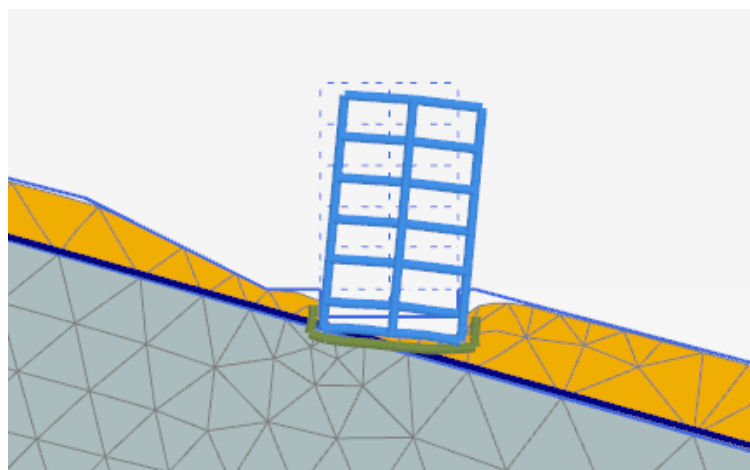


Figure 5.50 : Tilting of a five storey building on a 15 degrees inclined sandy slope.

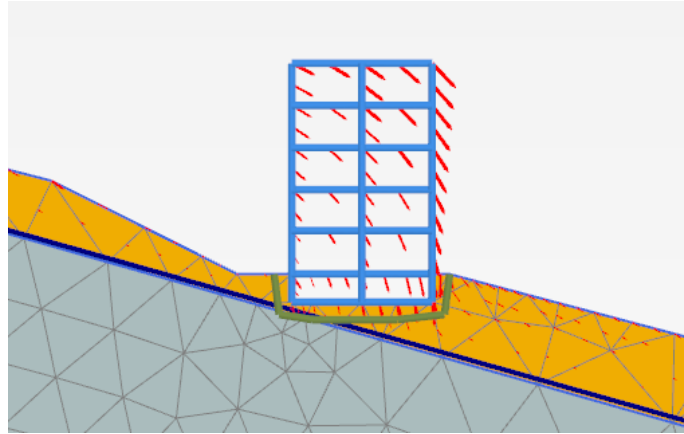


Figure 5.51 : Displacement vectors of a five storey building on a 15 degrees inclined sandy slope.

Analysis indicate that in clayey slopes rotation is more compared to sandy soils. The inclination of slopes has a great influence on displacement and rotation of buildings. Displacement of 5 storey buildings are more compared to 2 storey buildings. Figures 5.52 and 5.53 show the calculated values for horizontal acceleration of top, middle and basement floors of 5 a story building on a 30 degrees inclined slope with clayey and sandy soil materials respectively.

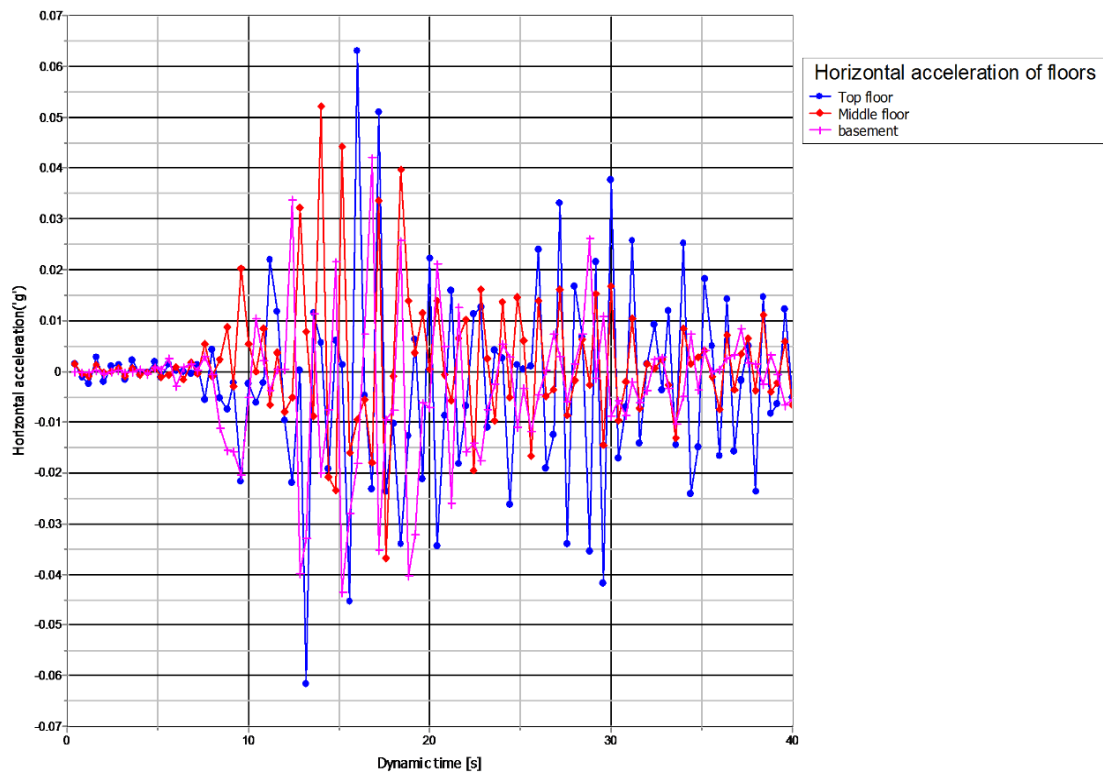


Figure 5.52 : Horizontal acceleratoinis of floors on 30 degrees inclined clayey soil under Elazığ ground motion.

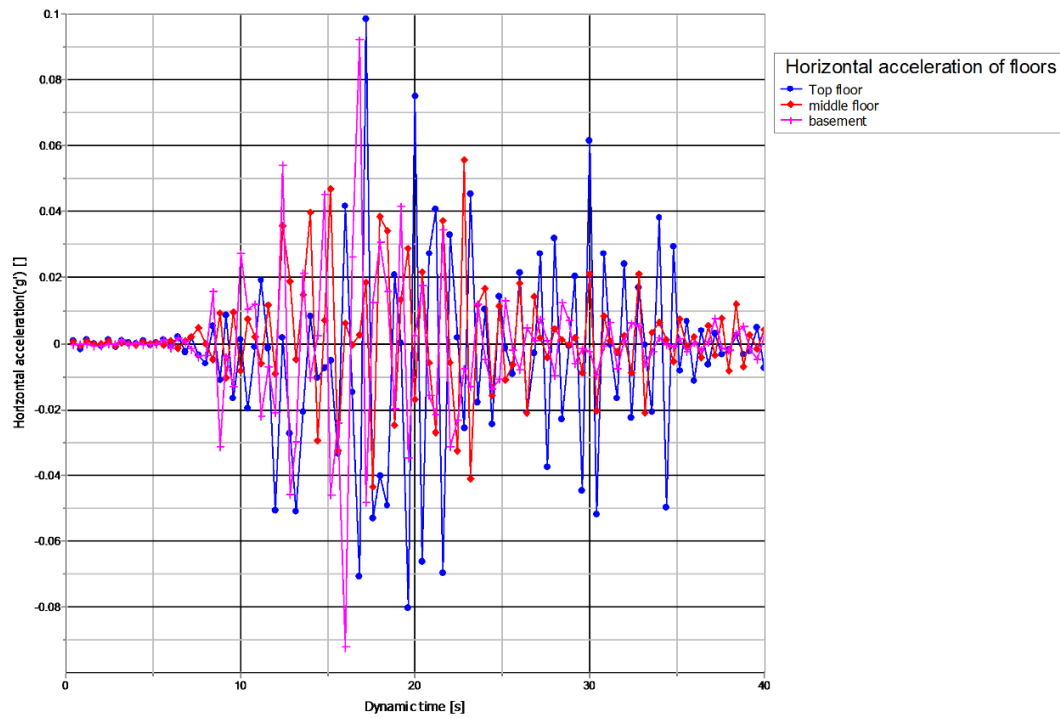


Figure 5.53 : Horizontal acceleratoin of floors on 30 degrees inclined sandy soil under Elazığ ground motion.

Figures 5.54 and 5.55 show the horizontal accelerations on the floors of a 5 storey building located on sandy and clayey slopes respectively. The buldings loacted on a 15 degrees clayey slope hava more acceleration compared to other conditions.

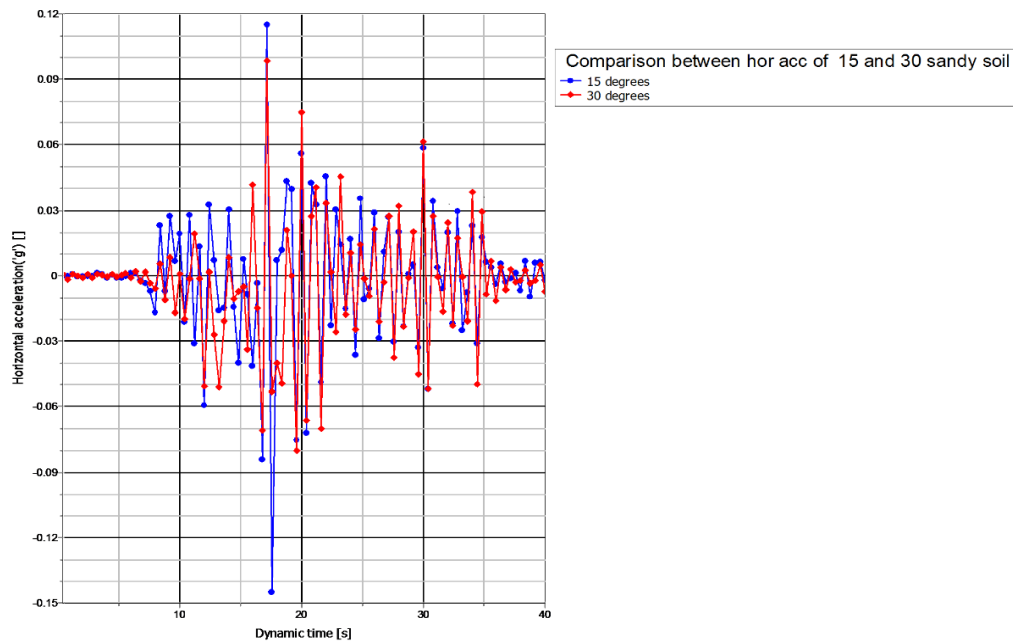


Figure 5.54 : Comparison of horizontal accelerations on 15 and 30 degrees sandy slopes.

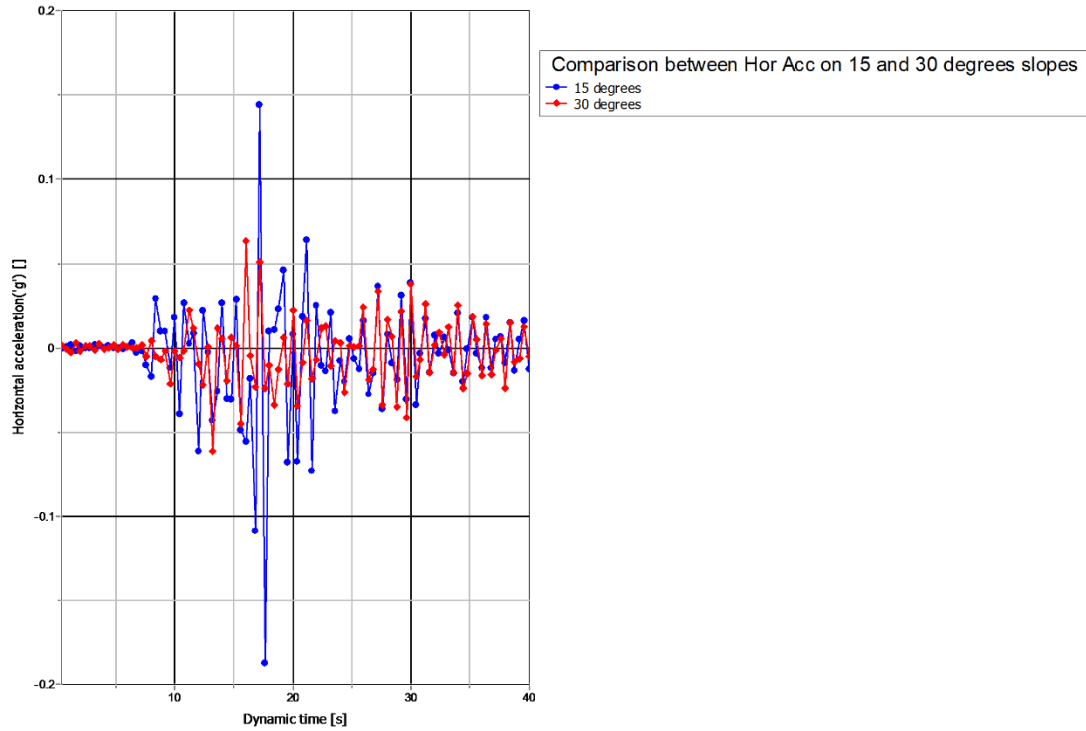


Figure 5.55 : Comparison of horizontal accelerations on 15 and 30 degrees clayey slopes.

5.2 Analysis of Kocaeli Case

In order to consider this important earthquake in the thesis, this case has been analyzed by finite element program, Plaxis too. The results are quite similar and also excess pore pressure generation which has been calculated during the earthquake, proves the great possibility of liquefaction phenomenon in this case, which is also the same conclusion of the researchers of that incident. Another reason of analyzing this case by Finite Element method is to validate the model used by PLAXIS program and finally the behavior of foundations located in Kocaeli case is studied which was missing from the original study. The results of analysis will be discussed in next parts. A brief review of Kocaeli earthquake and slope failure in Değirmendere and Halidere is presented below:

The 17 August 1999 Kocaeli-Turkey earthquake was an event of magnitude M_w 7.4 and caused extensive landslides, subsidences and liquefaction-induced ground deformations along the coast of Izmit Bay. K. Onder Cetin, Nihat Isik, Berna Unutmaz (2003), studied this earthquake and also the liquefaction induced slope failure caused by the Kocaeli 1999 earthquake. In addition to a brief discussion of

geologic setting and ground motions during Kocaeli earthquake, the observed ground deformations were discussed, along with the results of site investigations and in situ index tests including standard penetration tests (SPT), static cone penetration tests (CPT) and piezocone (CPTU) tests. The results of these site investigation studies were then used as a basis for calculation by finite difference analysis method. Figures 5.56 and 5.57 show the view of affected area before and after earthquake.



Figure 5.56 : View of Degirmendere nose before the earthquake (Cetin et al,2003).

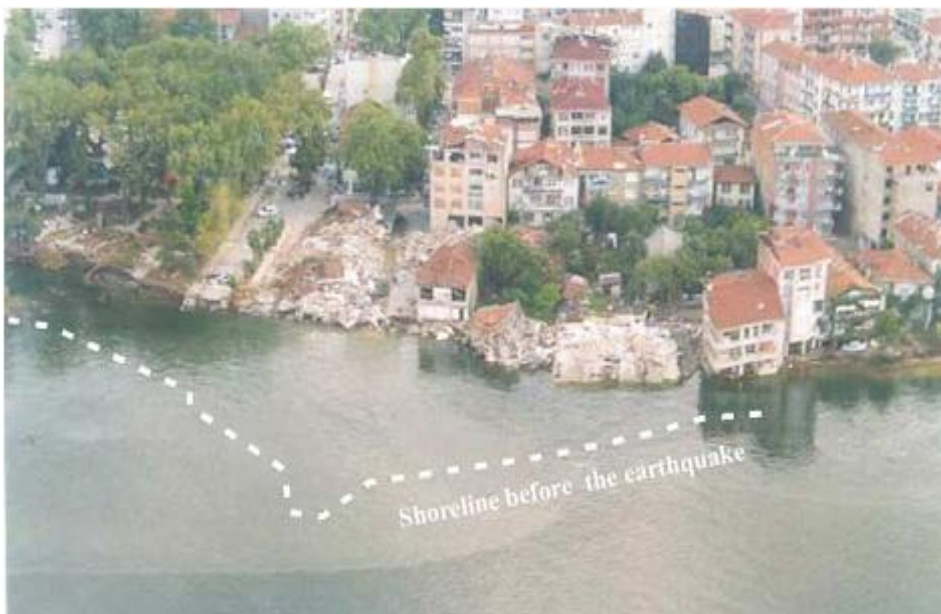


Figure 5.57 : View of Degirmendere nose after the earthquake (Cetin et al,2003).

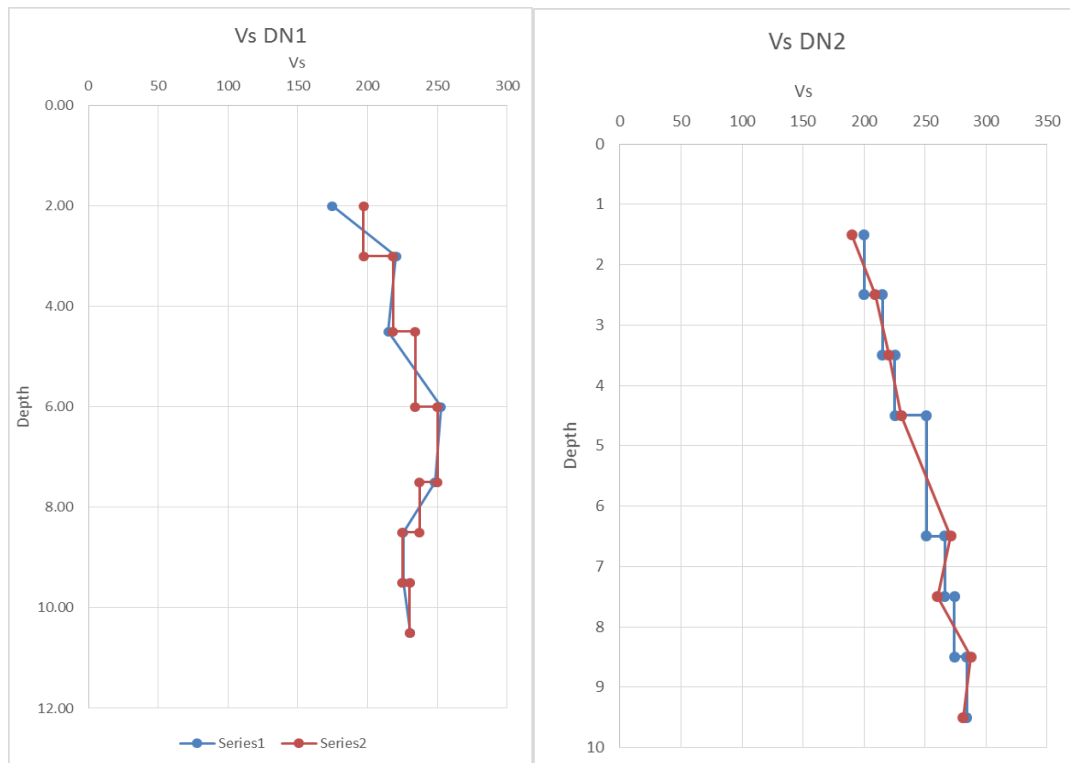
The parameters used in Plaxis have been derived by correlations based on SPT N values from two borehole logs. The model has been modified to be used in HS small model. The unit weight of soil values, were available from the original study. The parameters of stiffness and strength have been derived using the correlations and formulas which have been explained in previous chapter. The derived parameters and also related charts have been shown in the next page. The analysis in this part have been done considering four different conditions. First the slope has been analyzed without any extra load on it. The aim of the analysis is to obtain the displacements and shear failure bands due to the ground motion. Then the analysis have been carried out considering a 50 kPa load on the slide prone area and also the effect of distance of the load from the slope crest has been studied with another analysis. Finally the effects of a heavier load, which is selected as 100 kPa, have been investigated by another analysis and the results of them have been compared to each other. Evaluating the behavior of slope under each condition, was the aim of these analyses.

5.2.1 Parameters used in finite element analysis:

Table 5.11 summarizes the parameters of soil layers which have been used in the finite element analysis. Figure 5.58 shows the alteration of shear wave velocity with increasing depth.

Table 5.11 : Parameters used for soil layers in Finite Element analysis.

Material	Silty Sand1	Silty Sand 2	Silty Sand3	Average Sandy Soil	bedrock
Vs (m/s)					1000
Material Model	Hssmall	Hssmall	Hssmall	Hssmall	Linear Elastic
γ_{unsat}	18	18	20	20	20
γ_{sat}	19.2	19.2	20	20	20
ϕ	31	32	33	34	-
ψ	1	2	3	4	-
c (Kpa)	1	1	1	1	-
E (Kpa)	-	-	-	-	5.00E+06
G (Kpa)	-	-	-	-	2.00E+06
E ref 50	11000	20000	25000	30000	-
E ref Oed	11000	20000	25000	30000	-
E ref ur	33000	60000	75000	90000	-
m	0.5	0.5	0.5	0.5	-
v ur	0.2	0.2	0.2	0.2	0.2
G ref 0	20000	40000	45000	60000	-
γ 0.7	0.00017	0.00017	0.00017	0.00017	-

**Figure 5.58** : Shear wave velocity based on SPT N values from two bore holes.

The parameters for Rayleigh damping values are obtained and shown in Table 5.12.

Table 5.12 : Rayleigh damping values.

Layer	Thickness (m)	V_s (m/s)	Natural Vibration Period	f_1 Hz	f_2 Hz	w_1	w_2	α	β
Silty Sand 1	1.4	150	0.037	26.79	10	168.30	62.83	4.58	0.0004
Silty Sand 2	4	220	0.073	13.75	10	86.39	62.83	3.64	0.0007
Silty Sand 3	9	250	0.144	6.94	10	43.63	62.83	2.58	0.0009
Average Sand	4	260	0.062	16.25	10	102.10	62.83	3.89	0.0006
Rock	30	1000	0.12	8.33	10	52.36	62.83	2.85	0.0009

The cross section of affected area is shown in Fig 5.59. The locations of boreholes and also the ground water table are shown in figure below.

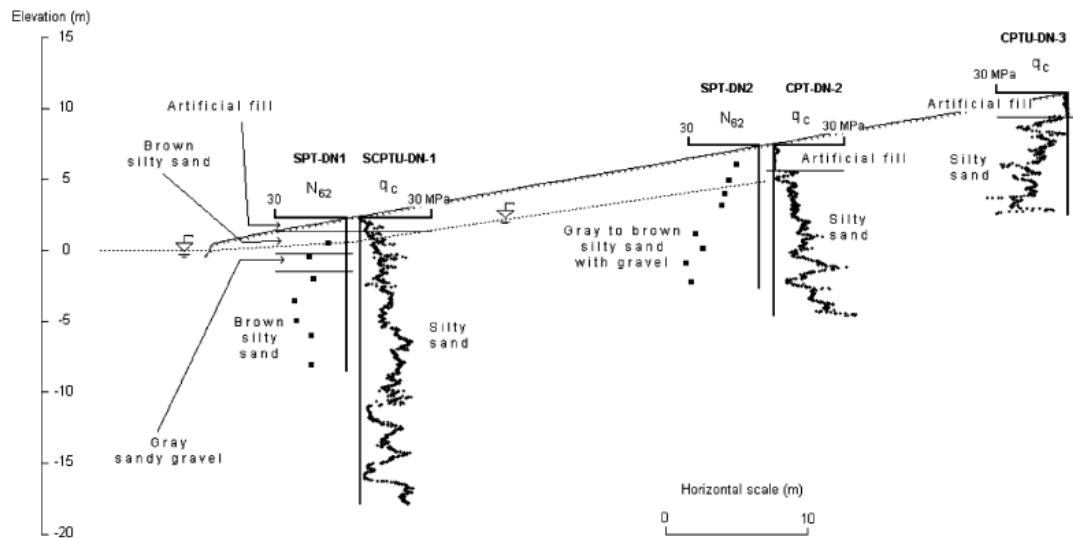


Figure 5.59 : Cross-section of Degirmendere nose site (Cetin et al,2003).

5.2.2 Ground motion data

Yarımca ground motion record was used in order to conduct the analysis. So the same accelerogram has been downloaded from strong ground motion database of Turkey, and modified to produce a maximum acceleration of 0.35 g. The accelerogram has been shown in Figure 5.60.

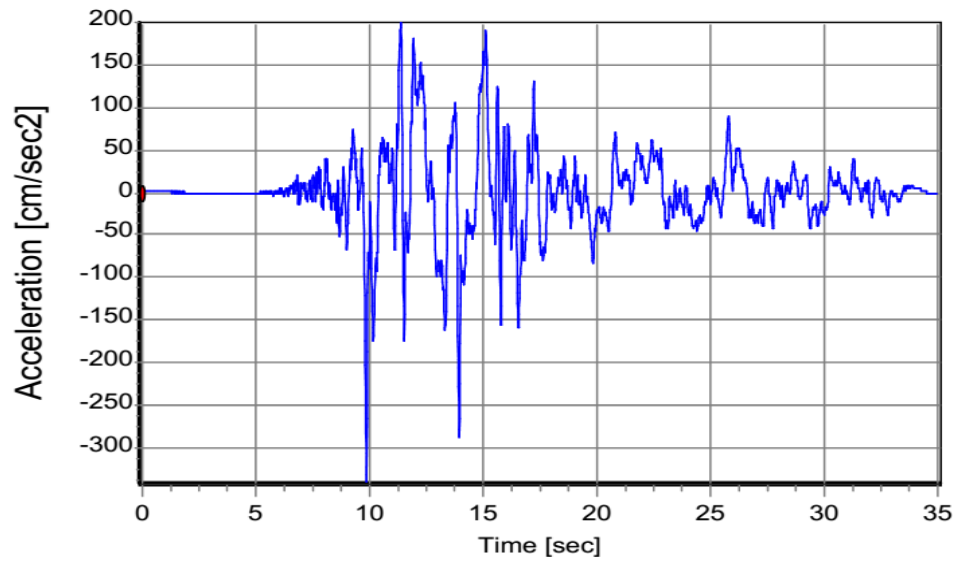


Figure 5.60 : Yarımcı record which is used for the analysis.

5.2.3 Finite element model

Figure 5.61 shows the soil profile analyzed by finite difference method in original study. The soil profile used in finite element method is shown in Figure 5.62.

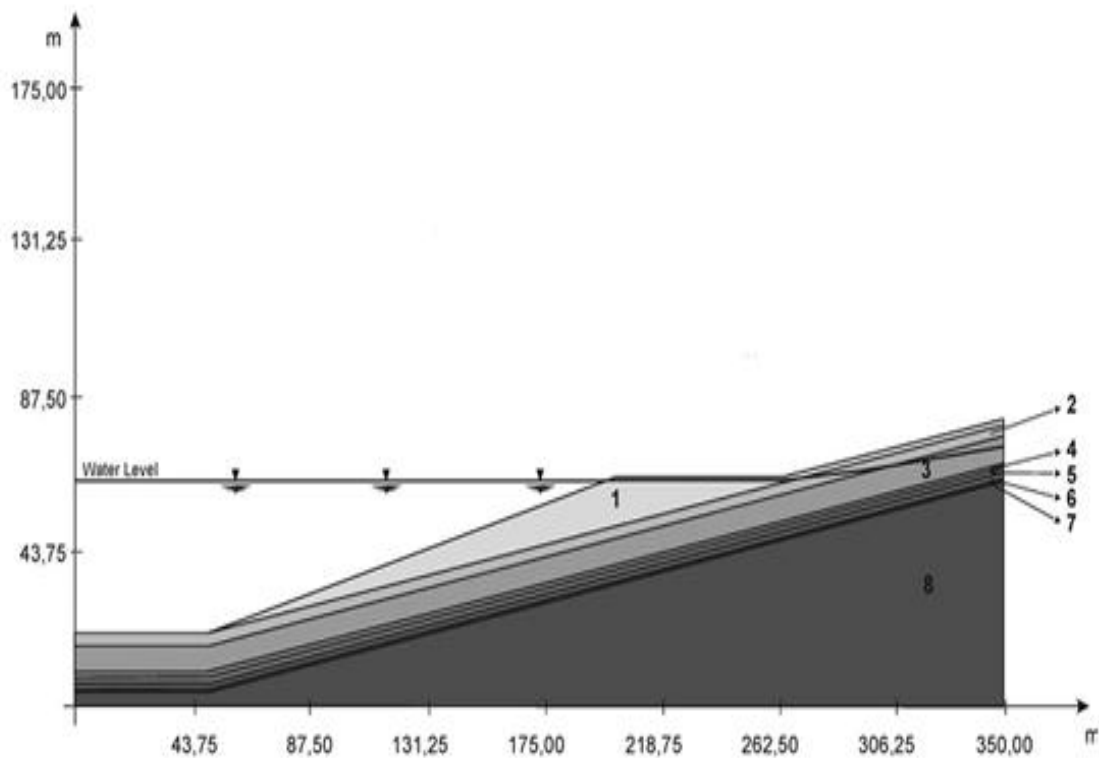


Figure 5.61 : Soil profile (Cetina et al, 2003).

In order to model the soil profile in Plaxis, original soil profile has been modified based on the fact that it is not appropriate to model very thin layers in Plaxis. Layers 4,5,6 and 7 are modeled as one contiguous layer with average soil parameters. The soil profile used in soil profile is shown in Figure 5.62.

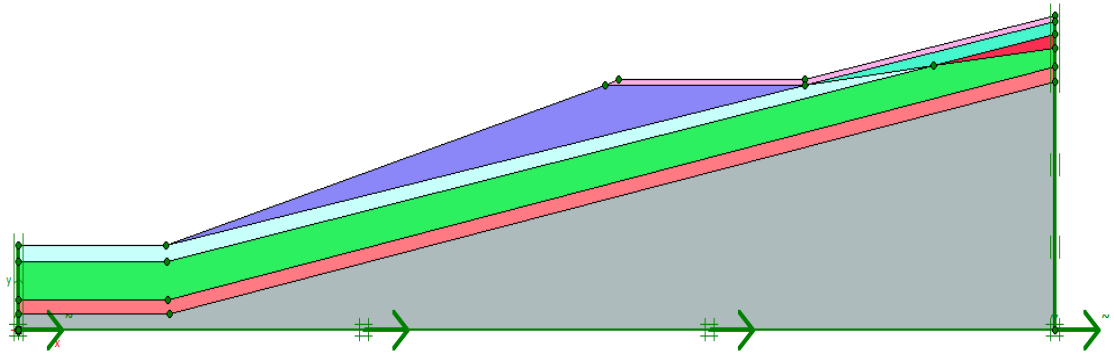
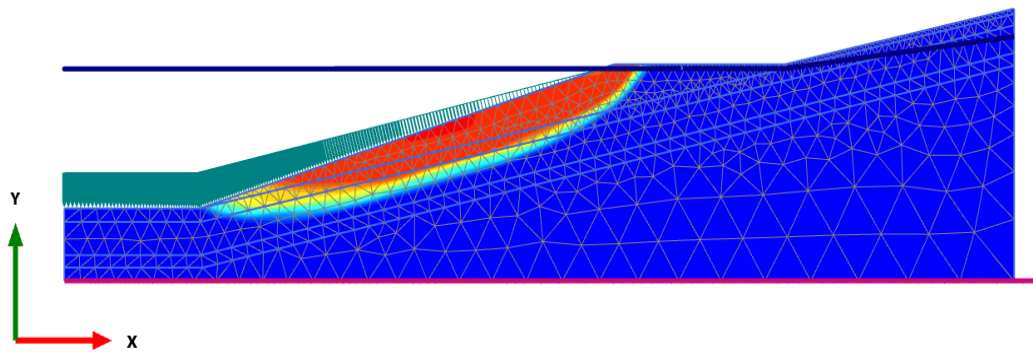


Figure 5.62 : Soil profile used in finite element analysis program.

5.2.4 Results and discussions

5.2.4.1 Analysis without considering a distributed load

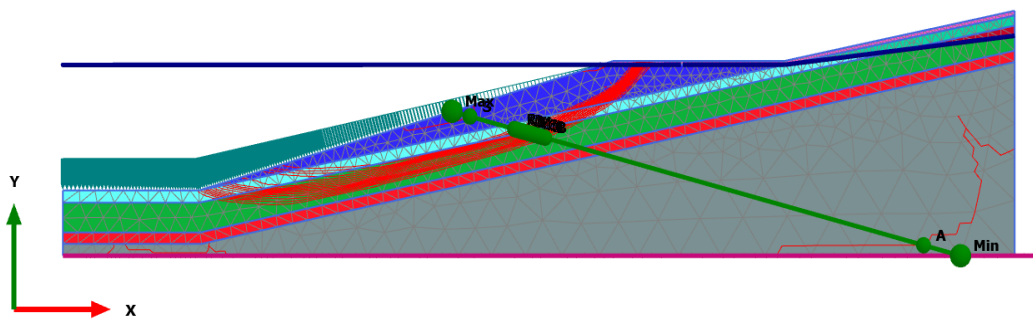
First analysis were done without considering the existence of extra load on slope. The results were different from the results of finite difference method. The reason could be the fact that Plaxis is unable to model liquids as well as other programs like Abaqus or Diana. Large displacements about 2 meters, happened at the end of earthquake duration. Displacement caused by safety analysis were obtained to be around 50 meters but it cannot be accepted as the displacement caused by earthquake. All of the displacements shown in the figures are results from safety analysis. The values are not valid but shape of shear failure bands and the generation of excess pore pressure is the same as results from finite difference analysis. The safety factors were very small which means the slope has suffered from lack of safety when Kocaeli earthquake happened ($FoS \approx 0.8$). Figures 5.63 and 5.64 show the shear failure bands and displacements of slope under Kocaeli earthquake.



Total displacements |u|

Maximum value = 45.10 m (Element 848 at Node 2375)

Figure 5.63 : Total displacement of slope after earthquake excitation.



Total displacements |u|

Maximum value = 45.10 m (Element 848 at Node 2375)

Figure 5.64 : Shear failure bands of slope.

Figure 5.65 shows the generation of pore water pressure during earthquake time interval. Excess pore pressure values have been calculated in four different depths as shown below. Obviously the point in the lowest depth (about 10m), has the most excess pore water pressure generation. As it can be seen in Figure 5.64, the shear failure bands pass through sandy layers with the same depth.

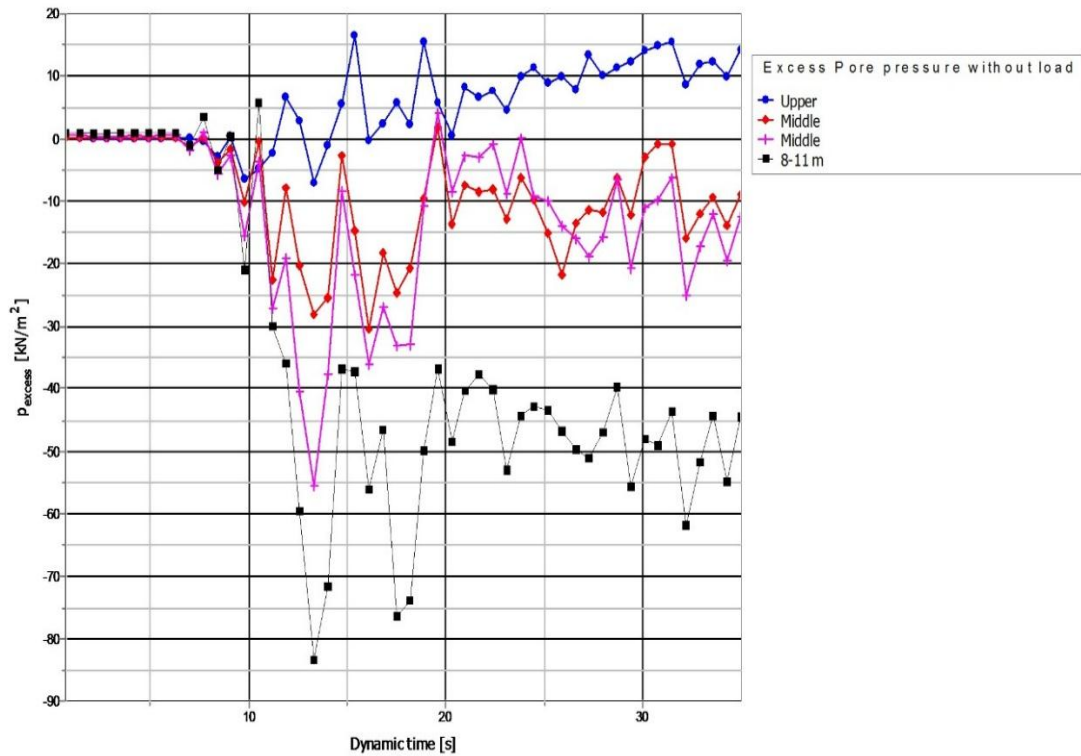


Figure 5.65 : Excess pore pressure generation in soil layers through depth.

5.2.4.2 Analysis considering a 50 kPa distributed load

Second analysis were done considering a 50 Kpa static permanent distributed load over a 20 meters width foundation in a distance of 20m from the crest. The results were interesting when the shear failure mechanism changes by applying the load.

Figure 5.66 shows the soil profile with applied load and Figure 5.67 show the initial condition of studied area before earthquake. The results of static analysis obviously shows no failure mechanism because of applied load. So without the effect of ground motion, the slope is stable but some displacements under the static load, which could be assumed as the load of a building, may be resulted.

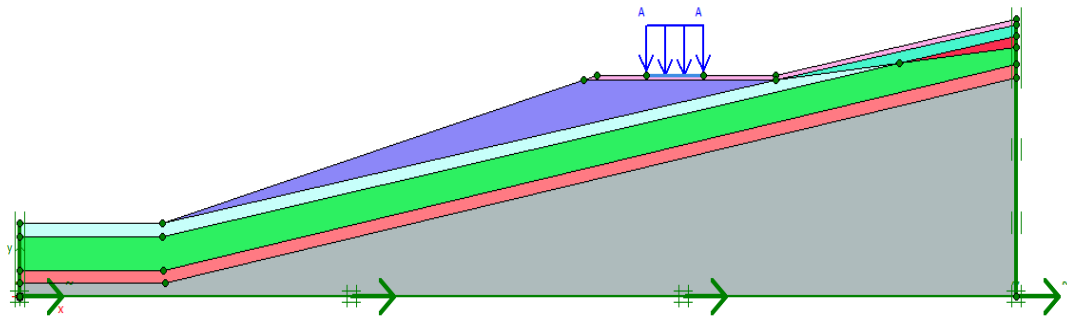


Figure 5.66 : Soil profile with applied load used in finite element analysis program.

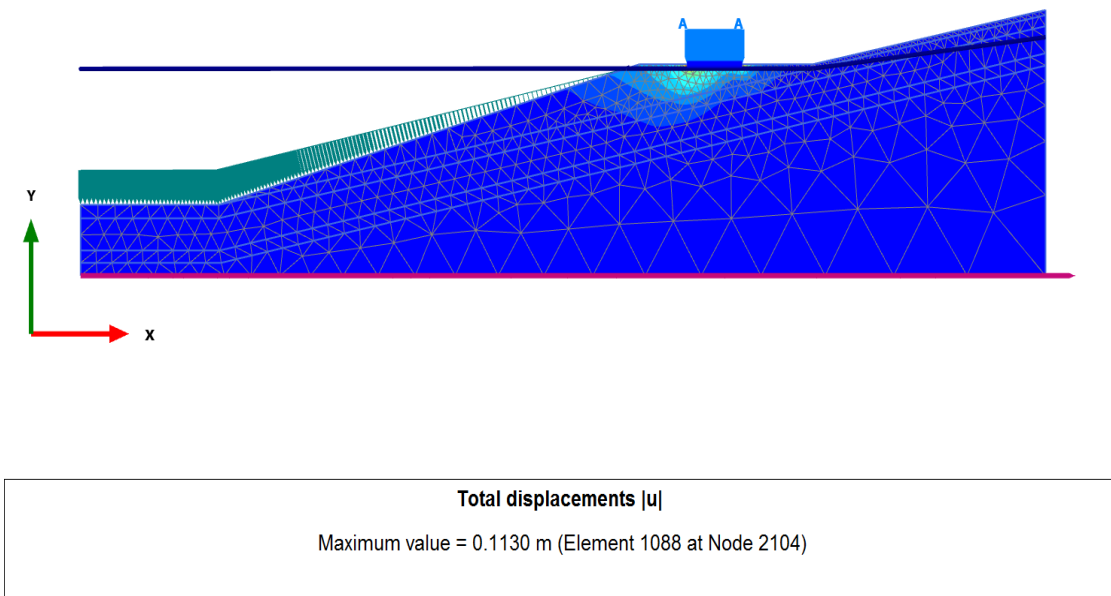


Figure 5.67 : Total displacement of foundation, resulted of applying load, without existing of earthquake effect.

Figures 5.68 and 5.69 show the results of applying Kocaeli earthquake ground motion record to the soil profile. The displacements increase to several meters (in safety analysis) and around 2m under the foundation, while it was only about 10 cm before earthquake excitation. Figure 5.70 shows the failure mechanism in case of presence of the loads of buildings in the affected area. The failure mechanism spreads toward to the right where the foundation is. So the instability of buildings will be a minimum consequence of Kocaeli earthquake.

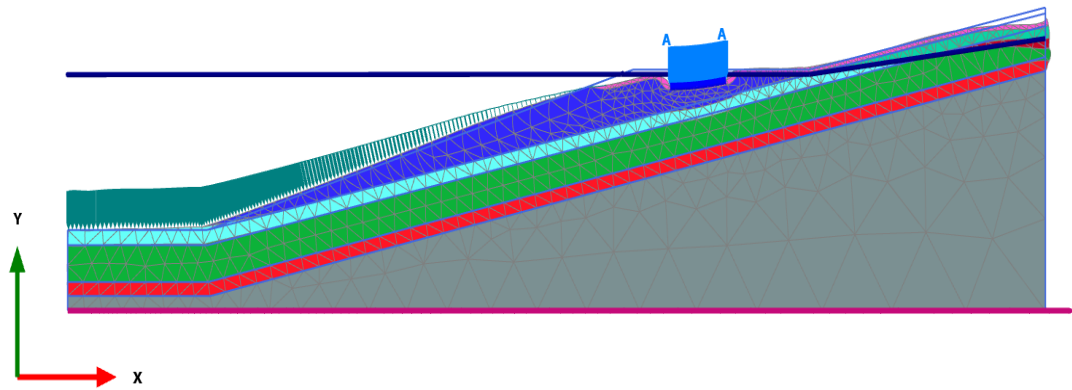
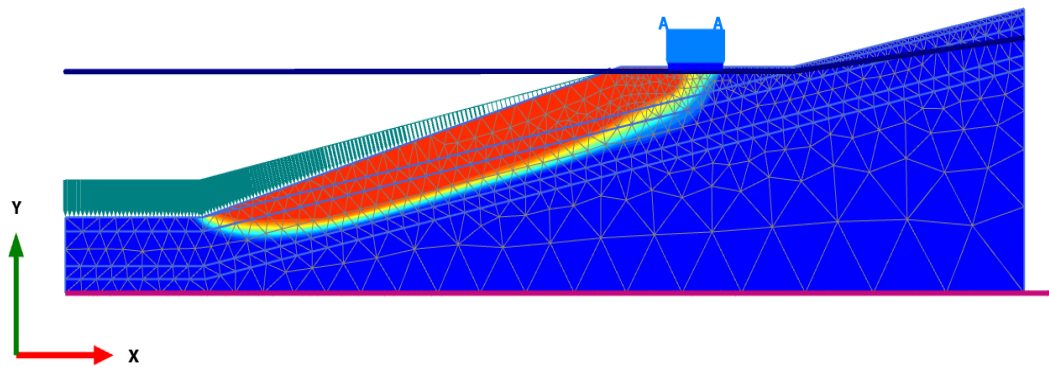


Figure 5.68 : Deformed mesh after earthquake.



Total displacements $|u|$

Maximum value = 45.96 m (Element 943 at Node 2234)

Figure 5.69 : Total displacement of foundation after earthquake effect.

Figure 5.70 shows the shear failure bands caused by earthquake. Obviously the most critical shear failure bands are located in the main sandy layer which is similar to the results from finite difference analysis.

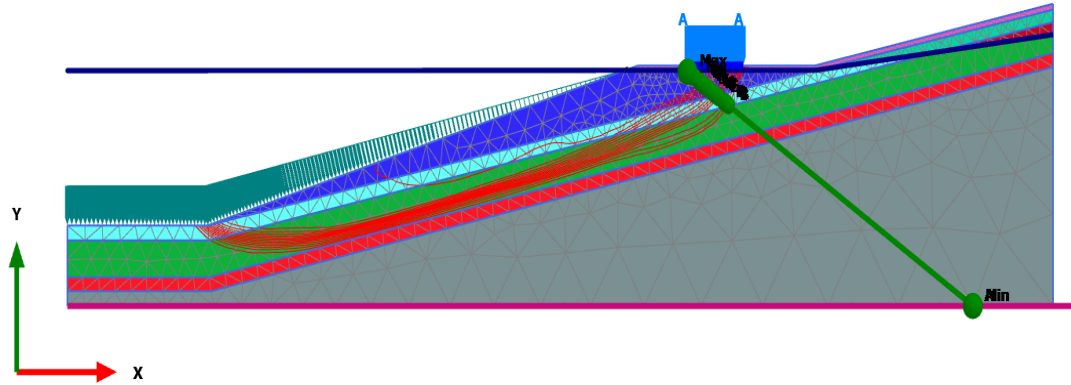


Figure 5.70 : Shear failure bands of slope resulted of applying load, considering the earthquake effect.

Figure 5.71 shows the generation of excess pore pressure during dynamic time. Generation of excess pore pressures have been calculated in three different depths using three points in the model which have been selected under the applied load. One in the upper soil layer, second one in the middle of profile and the third one in the main sandy layers which is located in 8-11 meters deep in the ground. The more excess pore pressure generation during earthquake in main sandy layers is obviously seen in the figure and is agreed with the idea of liquefaction in that layers, which was proposed by the authors of original study. Figure 5.72 shows the total displacement of foundation in this condition.

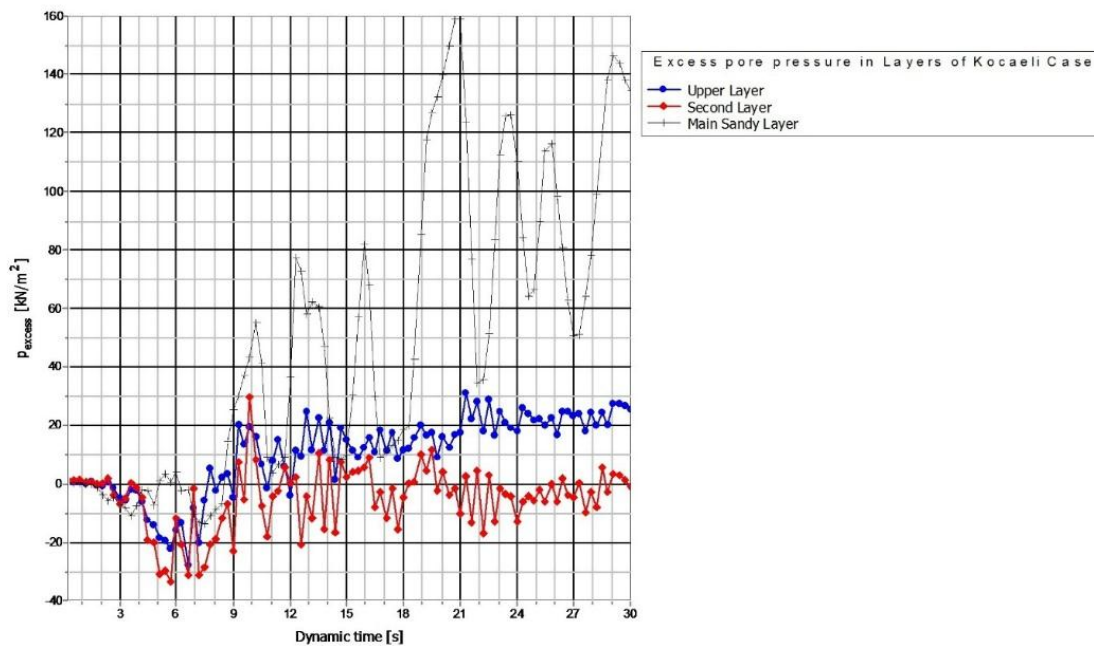


Figure 5.71 : Generation of excess pore water pressure in layers during earthquake.

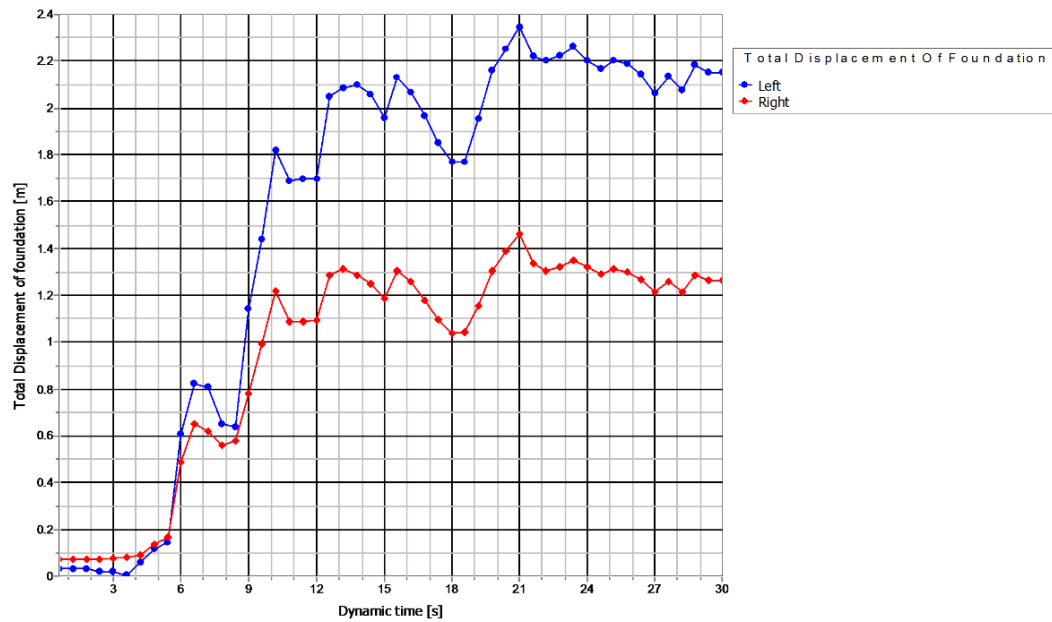
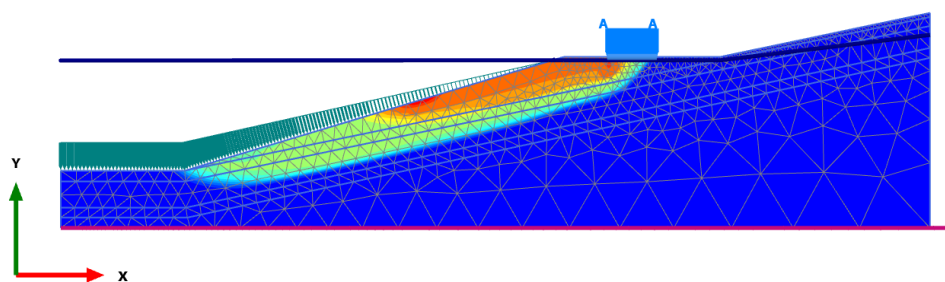


Figure 5.72 : Total displacement of left and right sides of foundation.

5.2.4.3 Analysis considering a 100 kPa distributed load

The third analysis was performed using a 100kPa distributed load, applied in a distance of 20 meters from slope crest. The aim of the analysis was to determine the behavior of the previous condition with a heavier load.

Figure 5.73 indicates that, due to heavier loading the slope movement is more and also the shear failure mechanism tends to move toward the right side of the foundation which is unlike the case that the load has the half amount of current load.

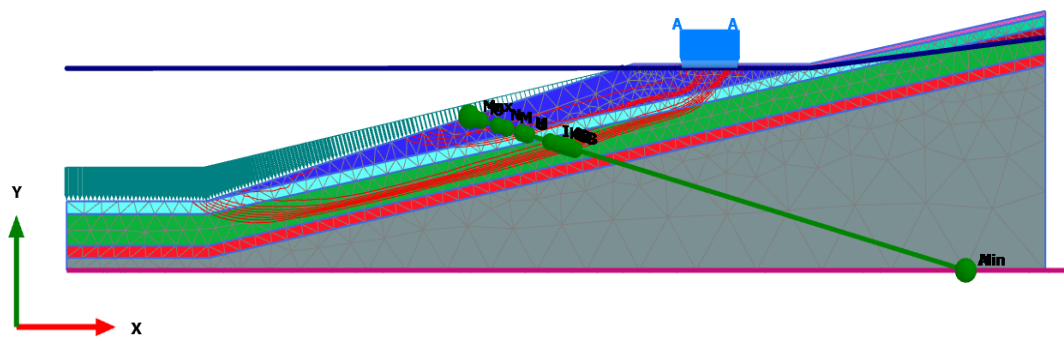


Total displacements |u|

Maximum value = 62.34 m (Element 735 at Node 6567)

Figure 5.73 : Total displacement of the heavier loading case.

Fig. 5.74 shows the shear failure bands of slope when a heavier load is applied in the model. This figure obviously indicates the effect of heavier loading on the slope which could be caused by a heavier building. Unlike the case with lighter load, most parts of foundation is under influence of failure and deformations are severe. Figures 5.75 and 5.76 show the comparison between horizontal and vertical displacements of left side of the foundations under three different conditions. Tilting of foundation is considerable in 100 kPa case according to values of vertical and horizontal displacement.



Total displacements |u|

Maximum value = 62.34 m (Element 735 at Node 6567)

Figure 5.74 : Shear failure bands of slope resulted of applying heavier load, considering the earthquake effect.

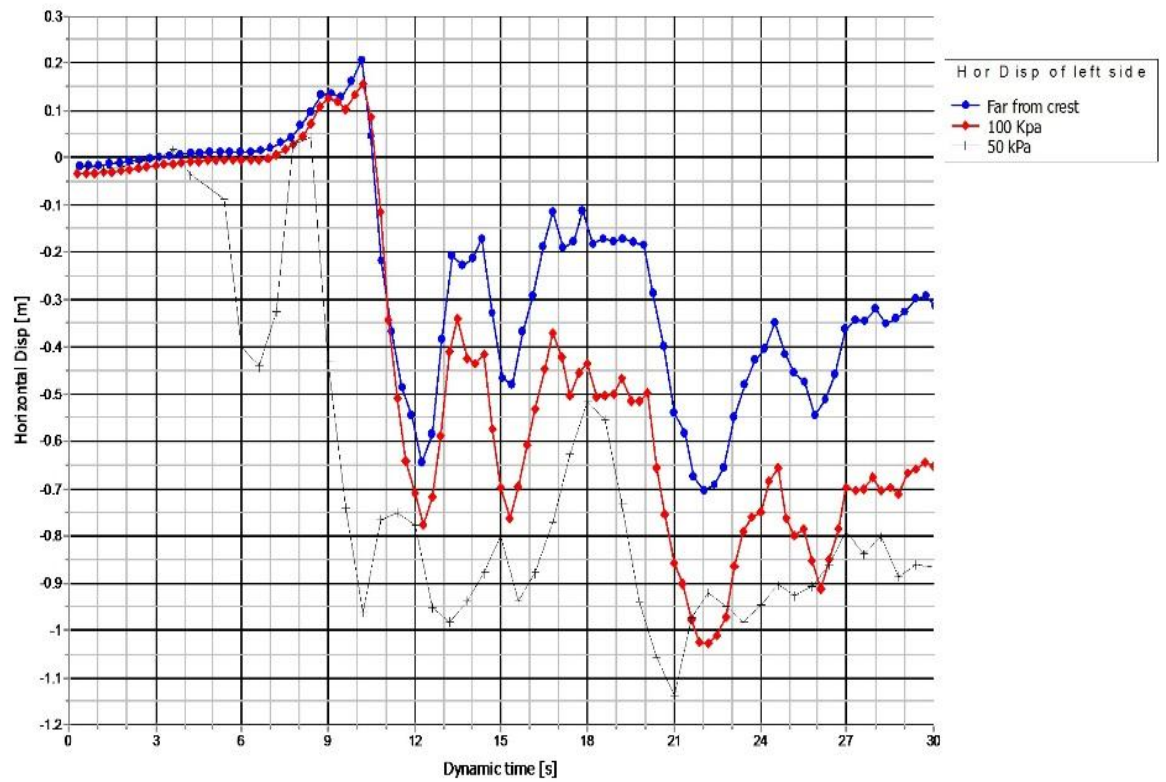


Figure 5.75 : Horizontal Displacement of left side of foundation.

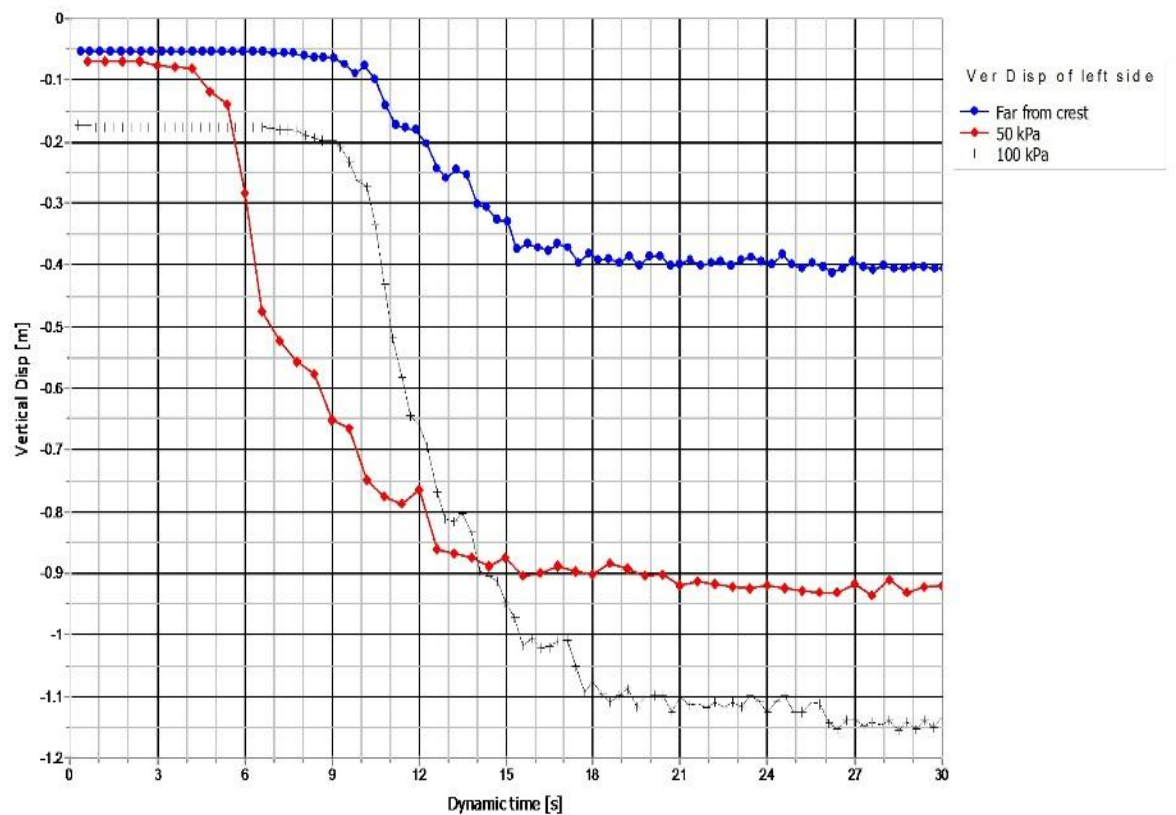


Figure 5.76 : Vertical Displacement of left side of foundation.

5.2.4.4 Analysis considering a longer distance of applied load from slope crest.

The last analysis are done considering the increased distance of applied load from the slope crest. The 50 Kpa applied load has been put far from the slope crest which is shown in Figure 5.77 and Kocaeli earthquake excitation has been applied again. The results of analysis are shown below.

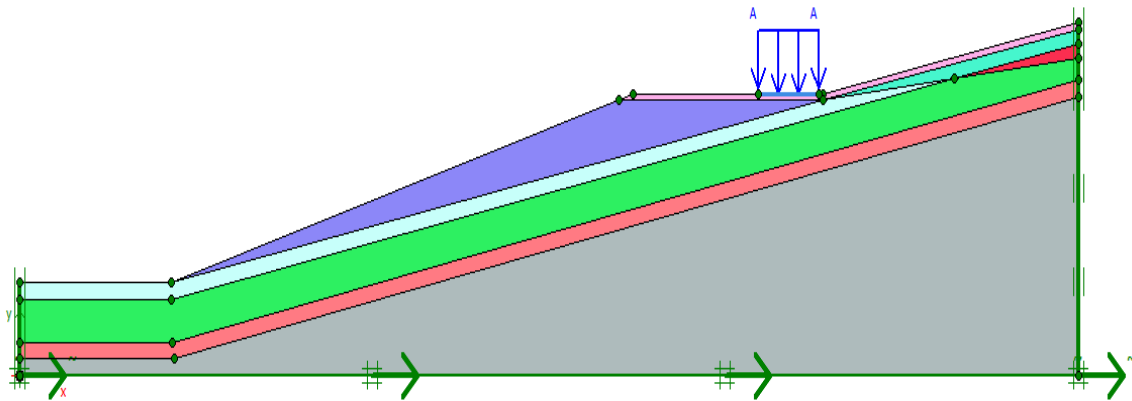
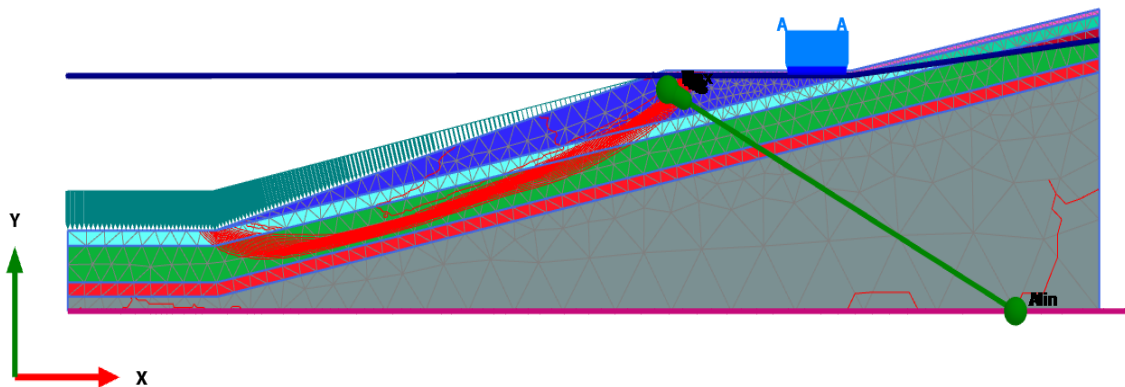


Figure 5.77 : Soil profile with applied load located far from slope crest.

Figure 5.78 shows the shear failure bands after termination of earthquake motion. Unlike the previous case which shear bands tend to spread toward the foundation, the applied load does not have any considerable effect on failure of slope.



Total displacements $|u|$

Maximum value = 48.15 m (Element 823 at Node 3922)

Figure 5.78 : Failure shear bands of slope resulted of applying load, considering the earthquake effect.

Figure 5.79 shows the total displacement of two sides of foundation. Obviously the deformations are less compared to cases studied before.

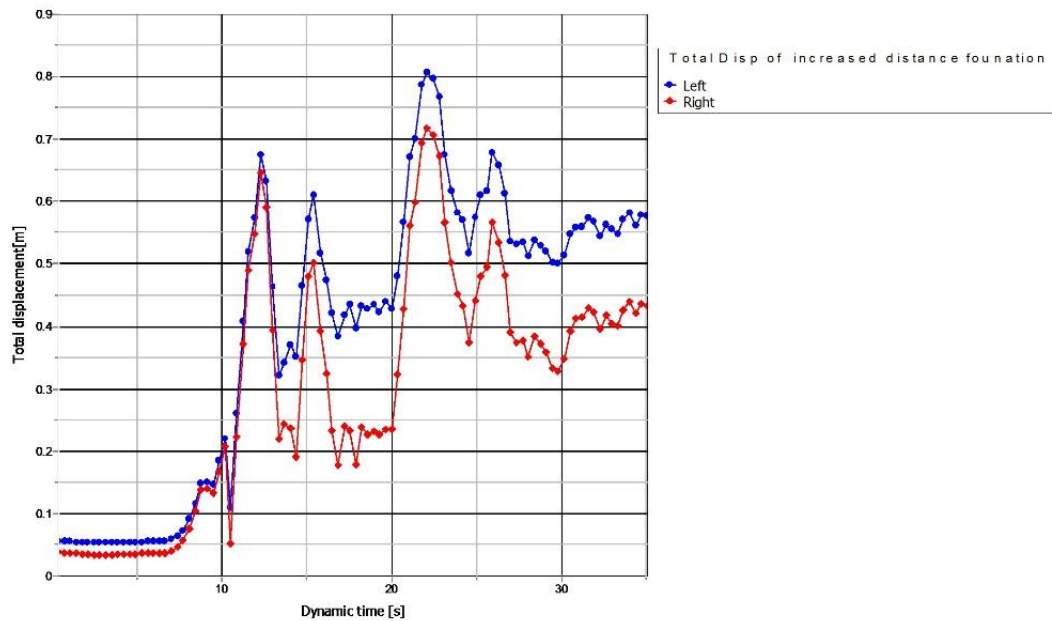


Figure 5.79 : Total displacement of left and right side of foundation, located far from the crest.

Figure 5.80 shows the difference between total displacements of a foundation closer to slope crest and another one which is located far from the crest. So if the structures were have been built far from the crest, the number of casualties and failure of buldings could be less.

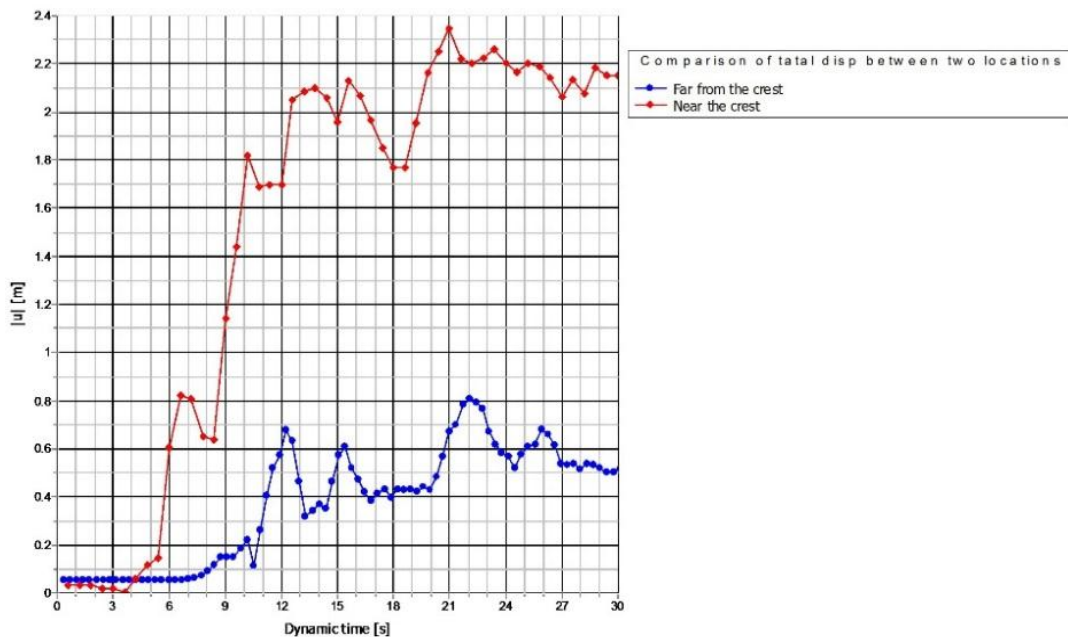


Figure 5.80 : Total displacement comparison between two conditions.

6. CONCLUSIONS AND RECOMMENDATIONS

Studying the behavior of slopes and buildings on or near them has always been a field of interest for geotechnical engineers. Recently a lot of structures such as buildings are being constructed on slopes of hilly areas. So they must be completely studied and evaluations must be done about how they behave under earthquake excitation.

In this thesis the Elazığ earthquake was the subject of study. The huge number of landslides and failure of buildings on hillside areas make this incident an important case. To investigate the effect of earthquake excitation on behavior of slopes in the affected zone, the finite element program, Plaxis 2D Version 2012, has been used. The finite element method has been one of the most efficient and reliable tools for studying slopes in recent years.

In order to perform the analyses, the models available in Plaxis have been studied and the best model for conducting dynamic analysis has been chosen which is the Hardening Soil with Small Strain stiffness model.

In order to determine appropriate soil model parameters, literature have been reviewed and the model parameters and the methods for derivation of parameters have been studied.

The soil material parameters have been obtained by estimation of parameters based on the results of sieve analysis which were done in the affected area.

First of all, a model with a length of 510 m and a height of 185 m has been chosen. The inclination of slope was chosen to be 25 degrees, which can be considered as a general slope inclination of the area. The inclination of slopes at the earthquake affected area, varies between 10 and 35 degrees. So to evaluate the behavior of soil at affected area a typical slope with mentioned geometry has been chosen. There are three types of soil material in the affected area, clayey sand (SC), silty sand (SM) and high plasticity clay (CH). All three types of main soil materials have been analyzed

under Elazığ earthquake excitation and the displacements and also safety factors of mentioned slopes have been calculated and compared to each other. In another series of analyses, the behavior of slopes under a distributed static load, which can be a representative of a building load, have been studied. Finally the behavior of two types of buildings have been studied under Elazığ and Kocaeli earthquakes excitations. The Kocaeli earthquake was one of the strongest ground motions in Turkey and in this thesis, ground motion record of this earthquake has been chosen to compare the effects of a strong earthquake and a weaker one like Elazığ earthquake on structures in the affected area. For this series of analysis, two types of sandy and clayey soils have been chosen to represent the most critical soil types. The soil materials used in analysis, have the properties of loose to medium dense soils. The buildings have been modeled as a two and a five storey buildings with the material properties of an ordinary frame building, which are stronger than the rural typical buildings. Two ground motion records of Kocaeli (1999) and Elazığ (2010) earthquakes have been applied to models and results of analysis have been obtained and compared to each other.

The analysis on the general slope showed a displacement about 15 to 35 cm for different materials, which can be accepted as a correct answer compared to real displacement which was around 20 cm. The nature of landslide and the displacement of soil particles are also similar to the deformation results of analysis. Safety factor for slopes have been calculated (around 1.1), and the fact that slopes were vulnerable against failure during earthquake have been clearly understood.

The analysis of applied load on different locations of slope showed that the slope deformations make the foundations so vulnerable against failure. For clayey layer, deformations and rotations were more severe and this issue should be taken into account if there is a need to build structures in such area.

The analysis on behavior of buildings showed the importance of location of building and the material on which the structure is being built. The inclination of slopes has a direct relationship with the displacements of buildings. Vertical displacements on a 30 degrees inclined slope for Elazığ earthquake ground motion, are generally in the range of 1 to 10 cm and they decrease to 1 cm and even less on 15 degrees inclined slope. The buildings in affected area are weaker compared to modeled building so it has to be mentioned that the displacement will be more in those types of buildings.

Same analysis with Kocaeli ground motion record indicates that how severe the displacements will be if stronger earthquakes happen in the region. Displacements increase even up to 2 meters in clayey soils. It is well understood that soft to medium dense soil types in the region are vulnerable against failure and must be treated well before building structures on them. For example different kinds of soil improvements could be done in order to have a better soil condition. So it is better to build structures with not much of height, for example 2 or 3 storey buildings will be safe depending on the used building materials, building properly engineered structures and also the condition of ground soil. It is better to avoid building structures on or near very inclined slopes to reduce the risk of failure of buildings.

The analysis for Kocaeli case showed the effect of excess pore pressure generation in soil layers which was not a considerable issue in Elazığ case. It also indicates that the distance from slope crest plays an important role in behavior of structures near slopes. And also the effect of building heavier structures on slope prone areas have to be precisely evaluated before any action.

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APPENDICES

APPENDIX A: Extra Figures

APPENDIX A

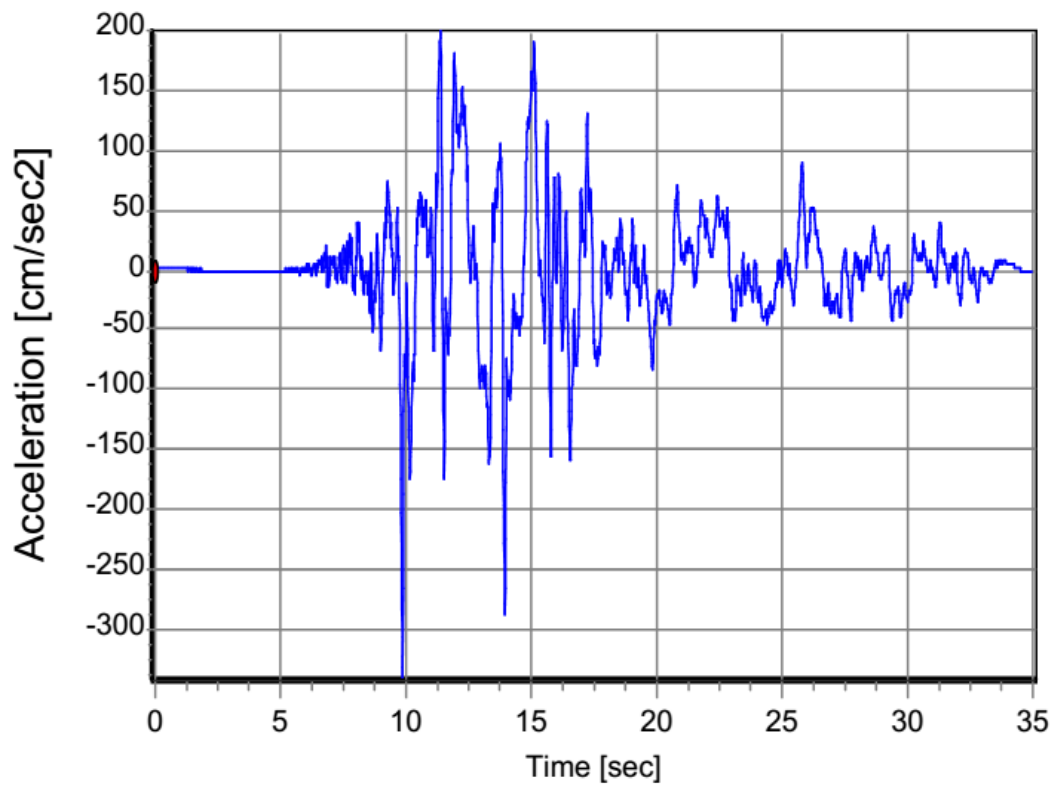


Figure A.1 : Acceleration record of Kocaeli earthquake.

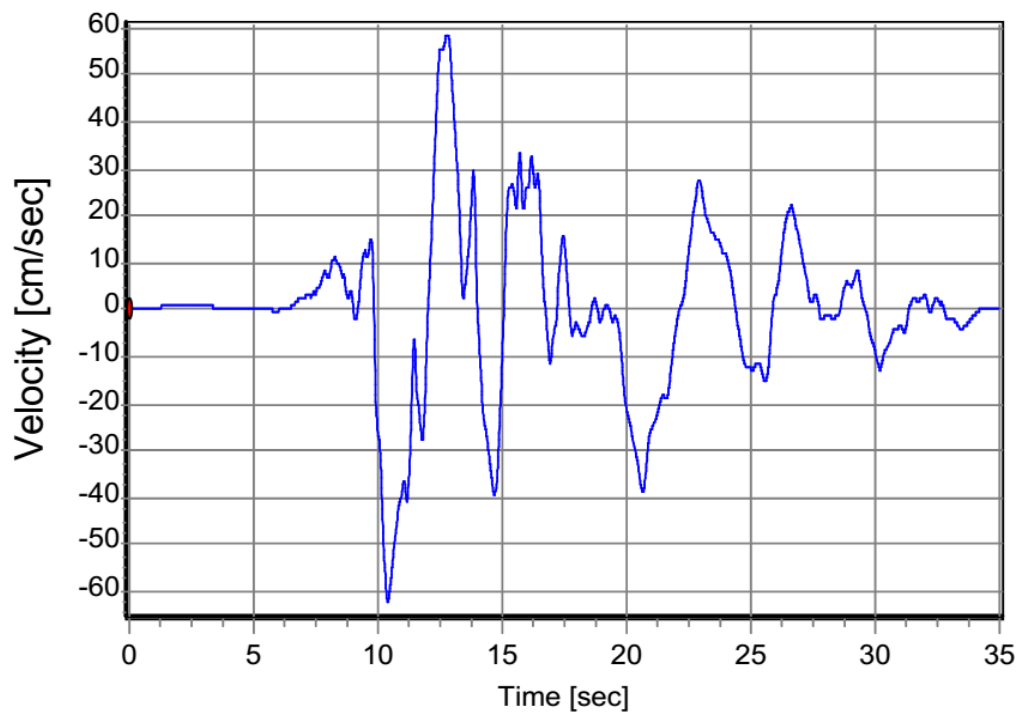


Figure A.2 : Velocity record of Kocaeli earthquake

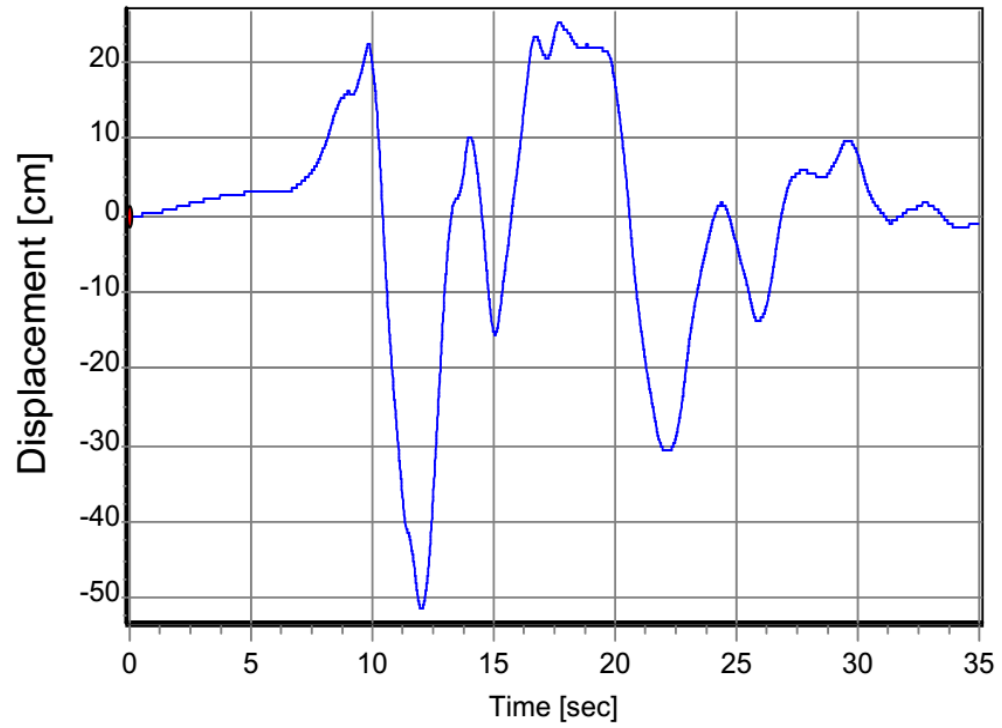


Figure A.3 : Displacement record of Kocaeli earthquake.

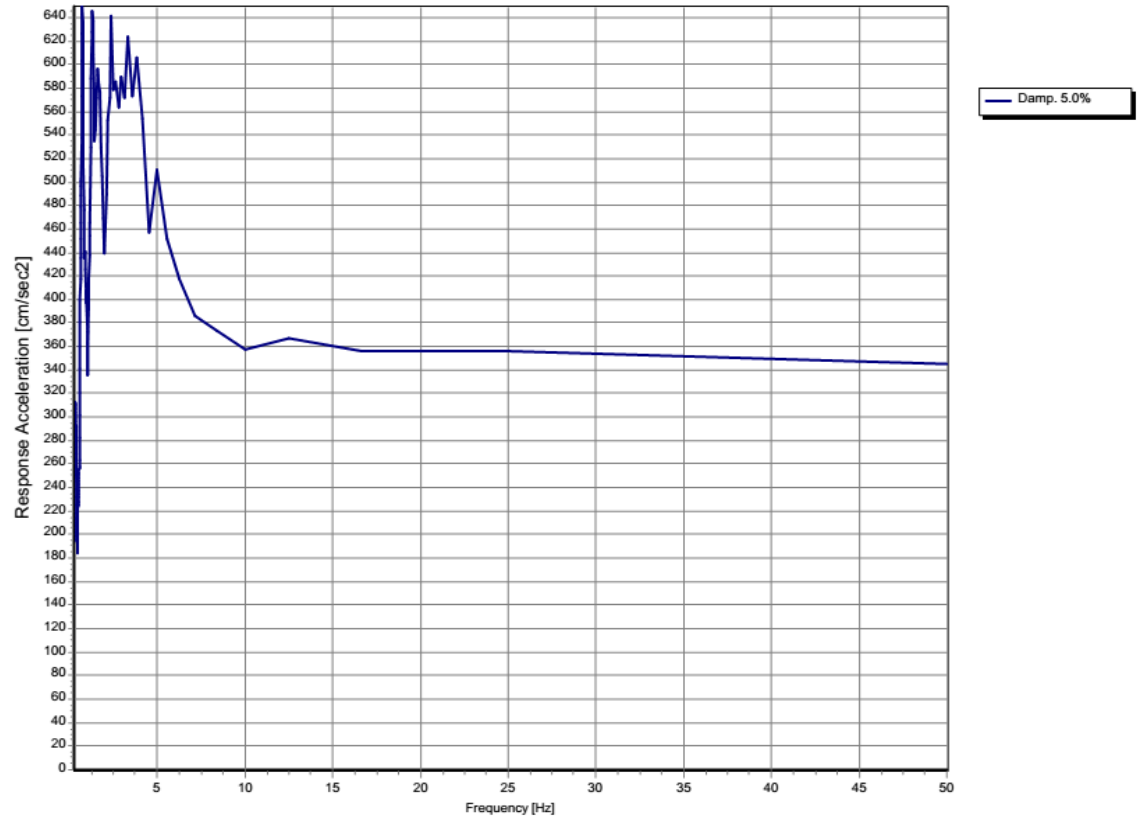


Figure A.4 : Spectrum graph of Kocaeli earthquake.

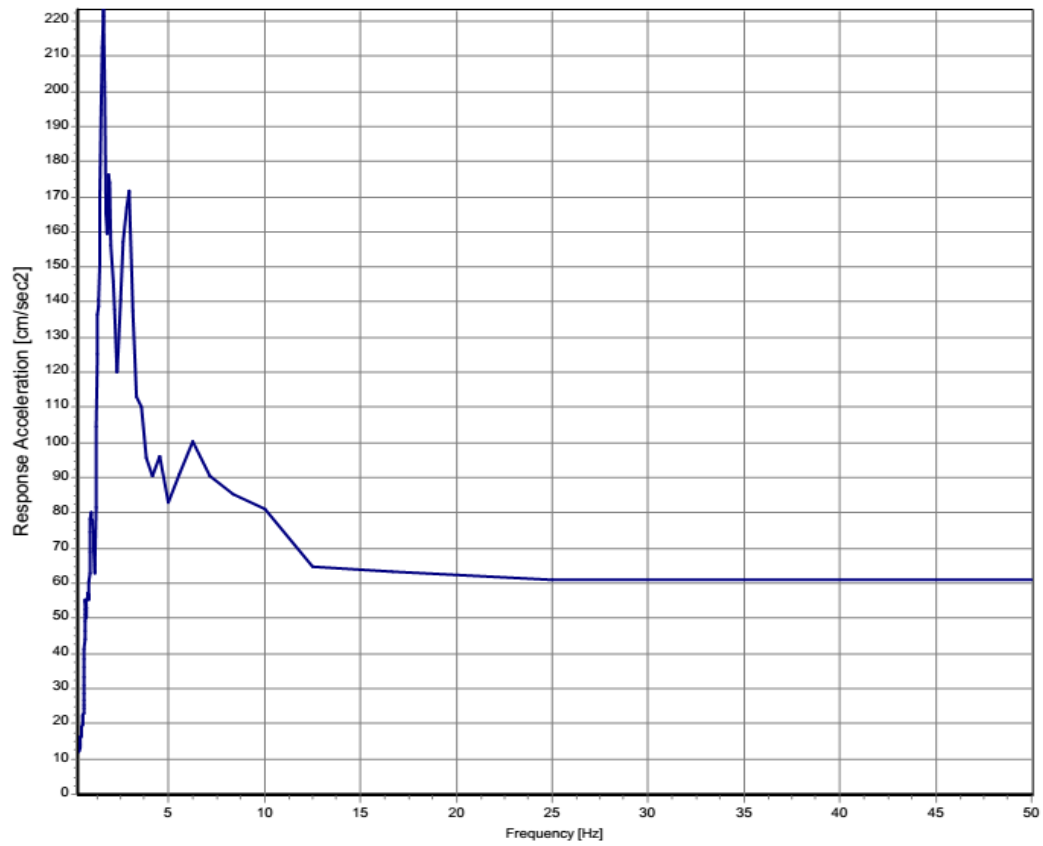


Figure A.5 : Spectrum graph of Elaziğ earthquake.

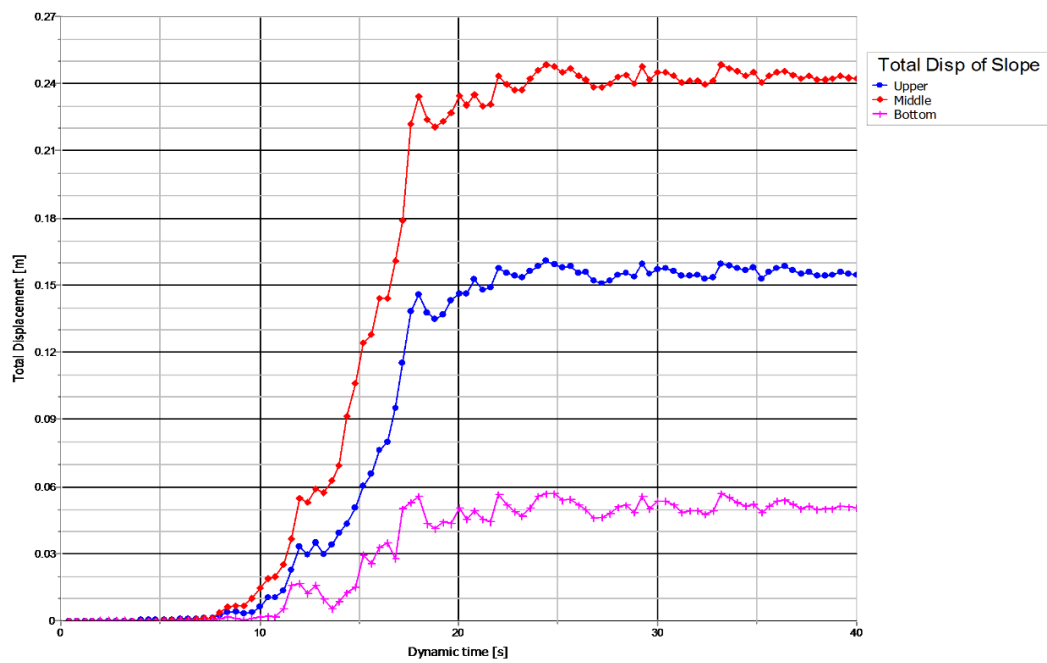


Figure A.6 : Total displacement of sandy layer in first series of analysis.

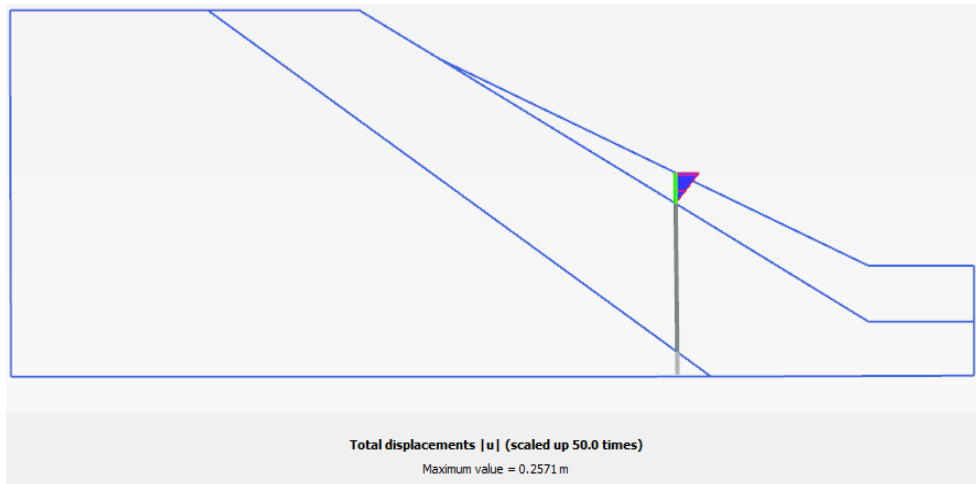


Figure A.7 : Cross section of displacements of sandy layer in first series of analysis.

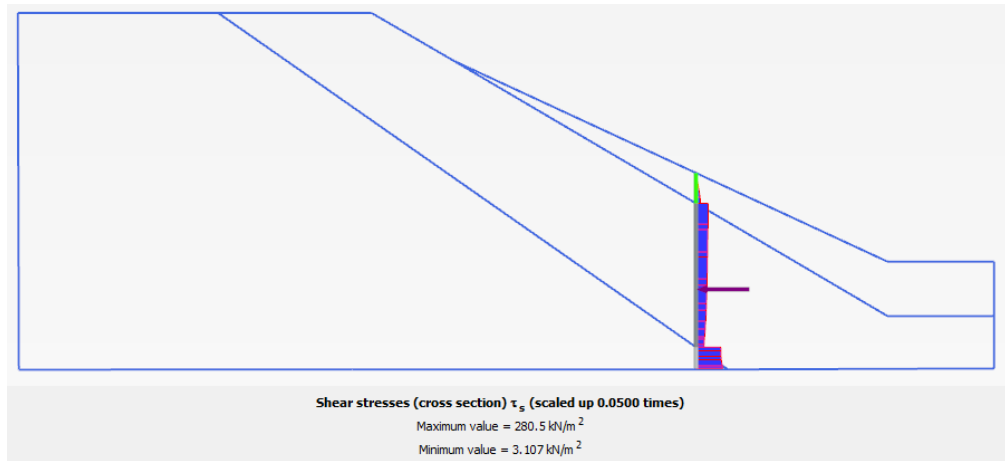


Figure A.8 : Cross section of shear stresses of sandy layer in first series of analysis.

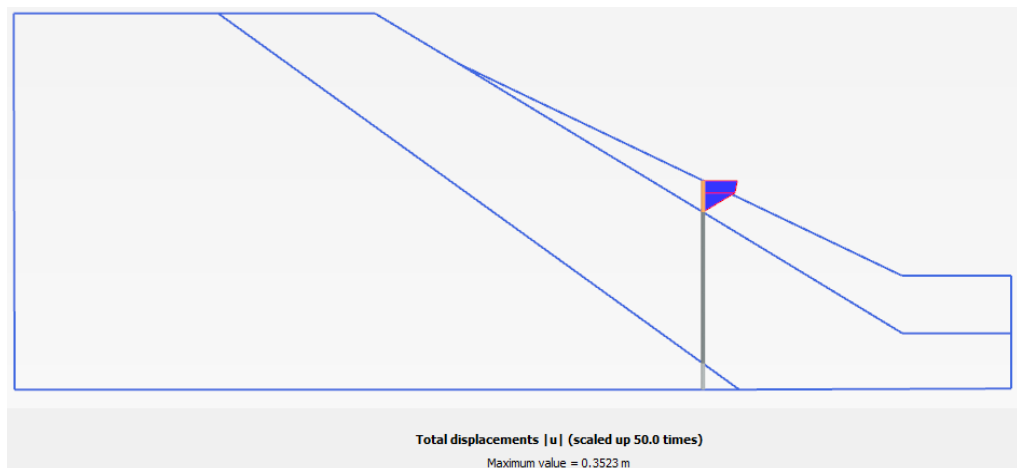


Figure A.9 : Cross section of displacements of clayey layer in first series of analysis.

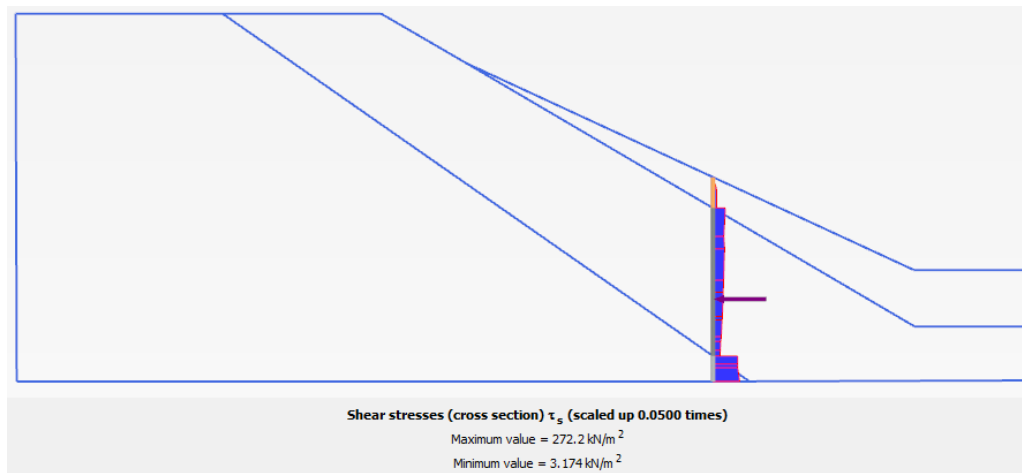


Figure A.10 : Cross section of shear stresses of clayey layer in first series of analysis.

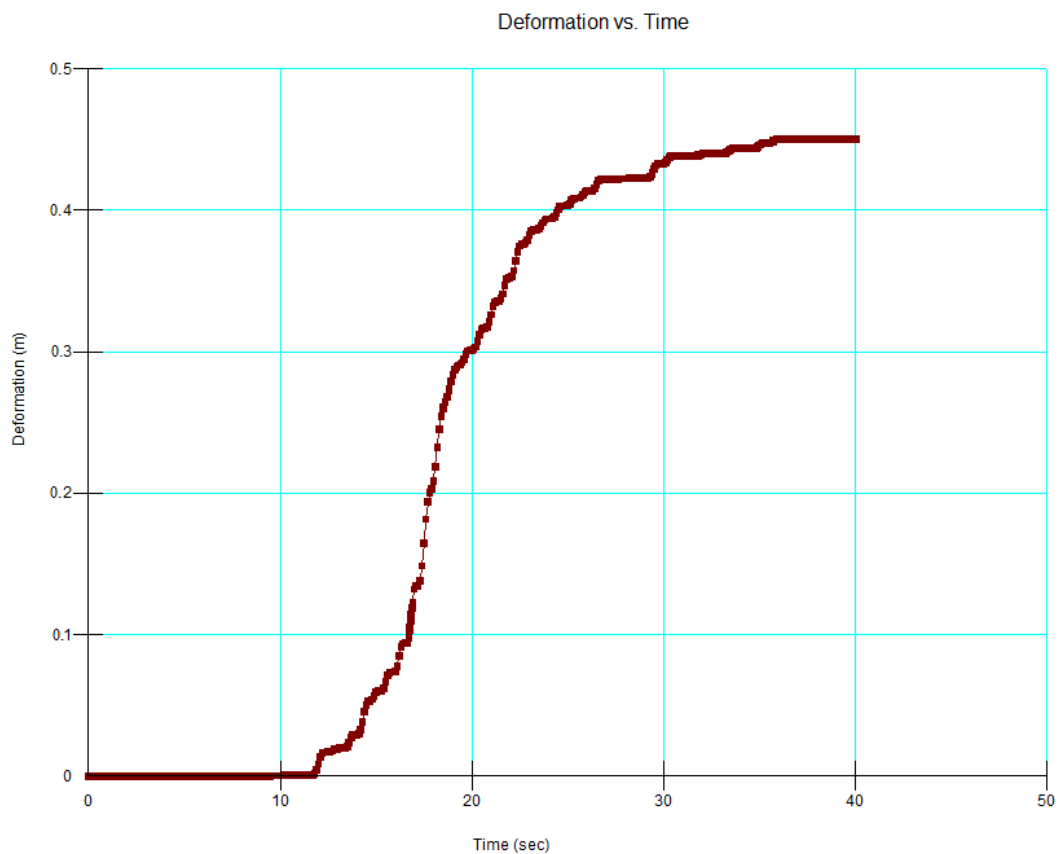


Figure A.11 : Displacements during time interval for clayey layer (Geostudio).

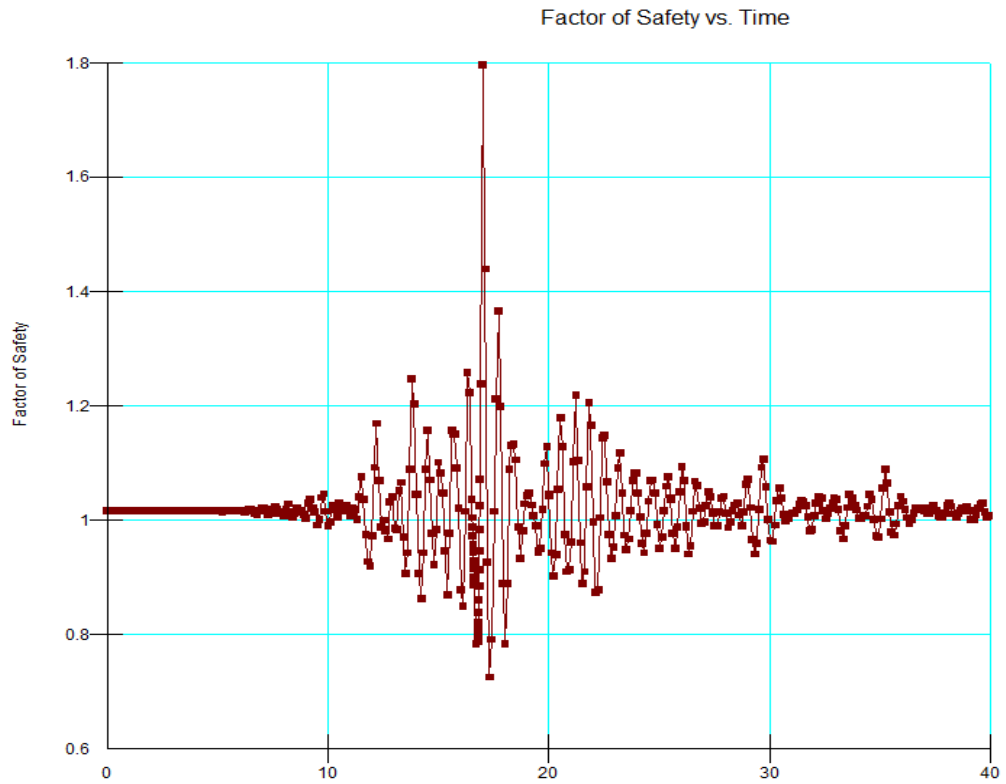


Figure A.12 : Variation of safety factor of clayey layer during earthquake (Geostudio).

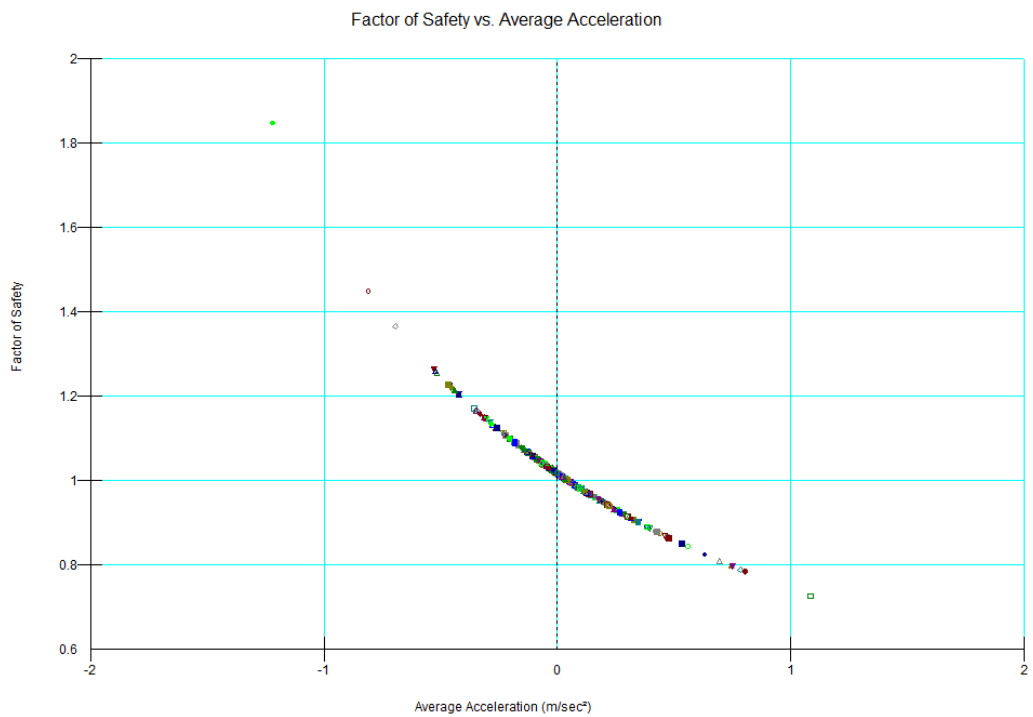


Figure A.13 : Relationship between average horizontal acceleration and safety factor of clayey layer (Geostudio).

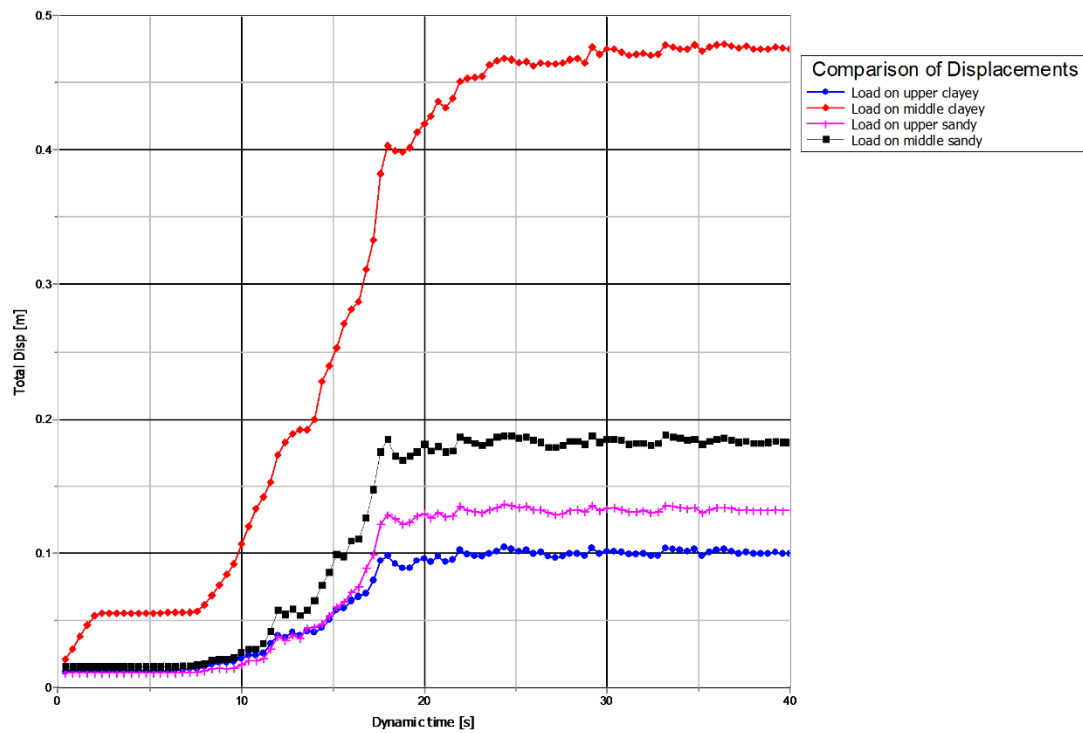


Figure A.14 : Comparison between the displacements of foundations located on four different condition in second series of analysis.

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