STABILITY INVESTIGATION OF ETI BORAX MINE HISARCIK TAILING DAM USING FINITE ELEMENT ANALYSIS

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ABSTRACT

STABILITY INVESTIGATION OF ETI BORAX MINE HISARCIK TAILING DAM USING FINITE ELEMENT ANALYSIS

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Today, with modern technological developments, the need for raw materials has also increased, which resulted in the development of mining industry. Therefore, there should be a plan for tailing storage and constructing a reliable tailing dam to store waste materials. In this study, the current and reckoning condition of the height rising in Hisarcik tailing dam is investigated for any potential stability problems. This tailing dam is one of the main tailing impoundments in Emet Borax mine, which is located within the province of Kütahya. The required data and input parameters were obtained from the literatures and the previous site investigation report, which was prepared by one of the civil engineering research groups at JMS (Jeoloji Mühendislik Sondaj) under the supervision of ETIMINE design department. This analysis is based on the finite element method conducted via PHASE 2 v.8.01 modeling software. The results represent the dam stability estimations and recommendations.

Keywords: Tailing dam, Slope Stability, Dam Stability, Finite Element Analysis

ÖZ

SONLU ELEMANLAR ANALİZİ İLE ETİ BORAKS MADENİ HISARCIK ATIK BARAJININ STABİLİTESİNİN DEĞERLENDİRİLMESİ

Mokhtari, Pozhhan

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Günümüzde, teknolojinin gelişmesiyle beraber, hammaddelere ihtiyaç da artmaktadır ve bu da maden endüstrisinin gelişmesine neden olmuştur. Bu çalışmada Kütahya'daki Emet Boraks madeninin, Hisarcık atık barajının durağlılık problemleri ele alınarak, bu barajın yerleşlemsı ve durumunda durağlılık durumu araştırılmış ve incelenmiştir. Bu konudaki gerekli veri literatürdan incelemeleri ve daha önceki JMS Ltd. Şti. (Jeoloji Mühendislik Sondaj), inşaat mühendisi araştırma grubunda yapılan araştırmalardan temin edilmiştir. Analiz sonlu elemanlar PHASE 2 v.8.01 modelleme yazılımıyla tamamlanmıştır. Elde edilen sonuçlarda, ilgili barajın durağlılığı konusunda değerlendirmeler ve öneriler surulmuştur.

Anahtar Kelimeler: Atık Barajı, Şev duraylılığı, Baraj duraylılığı, Sonlu Elemanlar Analizi

TO DADJOU

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NOMENCLATURE

c : Cohesion (kPa)

φ: Internal Friction Angle

γ : Unit Weight

ρ : Density

SSR : Shear Strength Reduction

SRF : Strength Reduction Factor

FEM : Finite Element Method

SF : Safety Factor

E : Modulus of Elasticity

υ : Poisson's ratio

CHAPTER 1

INTRODUCTION

1.1 General

The relationship between mankind and the nature is a controversial topic in most sciences and since the man found out that he could use nature as a resource, the mining emerged in pre-historic times, the period that digging the ground to obtain the ore became widespread. The modern mining concept is tending to use facilities to explore and exploit ores to prepare them for industrial consumption and mining industry have been one of the most chief sectors for many developed countries. Besides the benefits of mining industries, the outpourings of mining process cannot be neglected because of environmental effects and several health risks. As it is defined, tailings are by-products of mining industry, in which the minerals are extracted from the ore with particle sizes, which ranging from medium sand to silt or clay size (EPA, 1994). As it is legitimated in the early 20th century, all mining industries should prepare disposal and storage facilities; that is why the new types of impoundments and tailing dams are proposed. These structures are engineered to use waste materials themselves as the containments of the dam (EPA, 1994).

UNEP (2000) represents that there are more than 20,000 mines all over the world and each of them has one or more tailing dams. For instance, there are almost 1000 active metal mines in United States (Randol, 1993and most of them have at least one or more tailing dams which is joined together as a group. EPA guesstimates that there may be

more than thousand tailings dams related to the active non-coal mining, and tens of thousands of inactive or abandoned impoundments.

In mining industry, the share of tailing is much bigger than the ore product and it should be disposed in a safe way. For example, in gold industry just a few hundreds of ounces are the beneficial layout besides tons of dry tailing which is generated due to ore processing. Similarly, in copper industry and other mines, low-grade ores and high amount of tailings cause the determination about the way of managing the residues by the mines officials (Vick, 1990). Therefore, tailing disposal is a distinct section of each mine. These include "the disposal of dry or congealed tailings in impoundments or free-standing piles, backfilling underground mine workings and open-pits, subaqueous disposal, and the most common method, the disposal of tailings slurry in impoundments" (California Mining Association, 1991).

Recently, with the progresses in mining industries, impoundments have been structured for disposing the fine-grained tailings and in some cases these impoundments reach several ten meters in height and cover areas of several hundred/thousand square meters (EM 1110, 1970).

Tailings are generally stored in surface impoundments, which commonly consist of raised embankments (Vick, 1990). The raised embankments are generally constructed using one of the following raising methods: upstream, downstream, or center line (Vick, 1990). The upstream construction is the most economical and oldest method, which begins with the starter dam that is capable of passing seepage water and constructed at the downstream toe. The downstream portion should be resistant to piping (Vick, 1990). The discharging tailings are settled cursorily from the crest of the starter dam using spigots or cyclones. The deposition develops a dike and a extensive beach with coarse material which forms the foundation of the next dike. The most important factor in the upstream construction method is that the tailing beach should form a capable foundation for supporting the next dike and for this purpose the discharge should contain 40 to 60 percent sand at the least. In case of rather low percentages of sand, the upstream method is not applicable and the downstream method is superseded due to the grain-size distribution (Vick, 1990).

The downstream method is similar to the construction method of conventional water storage dams and like the upstream method, it initiates with a starter dam constructed from compacted borrow materials. This starter dam also has a major role to decrease seepage through the dam by utilizing low permeable material such as clay and silt (Vick, 1990). Since the downstream method provides a higher degree of stability, it is much more applicable than the upstream method, specifically in a potential earthquake zone or for high dams, the heights of which exceed 15 m. Furthermore, dam raises are not basically dependent upon the tailing deposits for foundation strength (Vick, 1990). On the other hand, due to the large volume of filling material required to raise the dam, it dramatically increases the construction costs (Brawner et al., 1973).

The centerline method resembles to the upstream and downstream methods as the embankment begins with a starter dam (Vick, 1990). This method has some advantages and disadvantages compared to the upstream and downstream methods. The centerline of the embankment is maintained as filled and advanced raises are placed on both the beach and the downstream surface and the tailings on the downstream slope should be compacted to prevent shear failure (Vick, 1990). It should be noted that this embankment type is not reliable enough to be considered for permanent storage of large volumes of tailing, but because of the low amount of filling material, this method is applicable to mining sites all over the world (Brawner et al., 1973).

According to the safety consideration, concerns have been gradually raised about the stability and environmental effects of tailing dams. Tailing material commonly includes chemical and toxic ingredients and therefore it may be hazardous. In order to diminish these concerns, these embankments usually rely on a certain amount of controlled seepage to improve stability.

Figure 1. 1 The Merriespruit Tailings Dam failure in South Africa (Tailings.org, 2001)

Connivance and ignoring the stability of dam and the safety of the impoundment may cause irrecoverable events like loss of life (e.g. Figure1.1). There are tangible increases in the number of failures and accidents all over the world due to inaccurate design and bad supervision (UNEP, 1996). Hence, the stability and safety of tailing dams is essential for the sustainability of the mine environment and natural resource. For this purpose, stability analysis and regular monitoring are indispensable.

Since the numbers of tailing dams are increasing, failures in such dams and the damage caused by such accidents also increase over time (UNEP, 1996). The failure of a tailing dam ultimately causes the release of the stored tailings deposit in the surrounding locality. Ensuring the stability of tailing dams while the mine is active and also after the mine closure is vital.

Recently, there were substantial advances in understanding of the physical and chemical processes present in tailings deposits. It has been perceived that alternative placement methods and impoundment functioning procedures result in deposits with significantly different engineering properties. Advantage has been taken from these differences to develop alternative tailings dam construction methods appropriate to different tailings types, impoundment sites, climatic conditions and impoundment performance requirements (Robertson et al., 1982).

Several tailing dams contain billions of tones of mineral processing industry wastes. Most tailing dams are sequentially raised by using the "upstream" construction method following the level of the impounded tailings during the loading process. This method, while available at low costs, implies a number of specific hazards for dam stability. These hazards require a methodical assessment and unceasing monitoring and control during siting, construction, and loading of the dam. Experiences show that these conditions often are not maintained (Makadisi et al., 1978).

1.2 Problem Statement

The mining industry has experienced several substantial dam failures so far. Tailing dams are often built with steep slopes using different kinds of materials with coarse fraction. Besides, a number of external actions which rise due to the mining operations, atmosphere and other factors affect stability of dams. Other reasons may cause various characteristics that increase the vulnerability of tailing dams, such as using the local fills (soil, coarse waste, overburden from mining operation and tailings), dam rising for increasing the embankment loading capacity, poor and limited design criteria, a lack of stability requirements regarding continuous control during the emplacement operation and the exceeding cost for remediation (Rico et al., 2008).

Moreover, some mining activities and tailing dams are located in high-risk regions in terms of seismic activities. The stability investigation of Emet Borax Mine Hisarcik tailing dam is obligatory due to the same reasons. Besides, the mentioned tailing dam is scheduled to be left as an abandoned impoundment in the near future and this makes it much more crucial to evaluate the stability by considering the fact that slope geometry and changing water level may result in failures. Also, the low-scale dam height rising is considered for the dam, which is going to be carried out in the near future. Hence, the results of the stability analysis could affect the decision making.

1.3 Objective of the Thesis

As mentioned in the earlier pages, the stability analyses of tailing dams are very important in mining engineering. Therefore, the first objective of this thesis is to

investigate the stability of Emet Borax Mine Hisarcik tailing dam in its current and static loading condition by use of PHASE 2 v.8.01, which is a Finite Element Method (FEM) based software. Since Turkey is one of the seismically active regions in the world, dynamic stability analysis for the current condition and the raised dam is essential, which forms the second aim of this research. In both static and dynamic analysis, the safety factor (Strength Reduction Factor) and the displacements are discussed. Finally, the results of the current study are evaluated using similar cases.

Briefly, the objectives of this research study include the following items:

(i) To investigate the tailing dam stability and displacements under current loading condition by use of PHASE 2 v.8.01, which is a Finite Element Method (FEM) based software.

(ii) To estimate and predict the probable displacement and deformation due to the seismic activity and relevant critical SRF (Strength Reduction Factor).

(iii) To investigate the stability of tailing dam for presumptive raised dam.

1.4 Methodology of the Study

To reach the objectives, the following steps and approaches were adopted:

1. Literature survey was done to obtain general information about designing and planning the tailing impoundments and tailing dams, embankment, foundation material, the geotechnical and geo-mechanical properties of tailings, construction methods of tailing dams (Upstream, Downstream, Centerline), developing pore pressure during the construction stages, failure modes of tailing dams, methods and conditions of stability analysis for tailing dams, seismic analysis of dams, and the selection of numerical method for stability analysis.
2. The reported data which were prepared by “EMET BORAX MINE GENERAL MANAGEMENT” were surveyed. The required data were obtained from the official reports of Emet Borax Mine which were prepared by JMS and also literature reviews were performed to verify the data.

3. The current and the condition of the raised dam (future plan) were analyzed by the use of the PHASE 2 v.8.01 modeling software, which is finite element method-based software.
4. The slope stability evaluations for upstream and downstream parts of the dam were performed.
5. The obtained results were discussed and the conclusions and recommendation were represented.

1.5 Layout of Thesis

The thesis consists of five main parts:

1. Overview about the tailing dam construction and design methods, the previous studies about the slope stability analysis and failure mechanisms of the tailing dams are given in Chapter 2.
2. General information about Emet Borax Mine, which is located in Kütahya province, and the related technical and non-technical data about the study area are given in Chapter 3.
3. The determination of the analysis method (FEM) and its consequences, slope stability, static deformation and dynamic analysis are given in Chapter 4.
4. The modeling results that were obtained using PHASE2 v.8.01 for the current condition of the tailing dam and the future plans about rising the dam height are given in Chapter 5.
5. Conclusions and recommendations are presented in Chapter 6.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

When the minerals are extracted from the ore, tailing is produced as a by-product in the mining industry. Generally, the residues consist of particles with fine grained and are normally blended with water until they obtain suitable pumping viscosity. The size of the particles varies from medium sand to silt or clay. Usually, tailing slurries are sent to the disposal area by pipelines. There are some different techniques to distribute them such as subaerial discharge (with spigots), subaqueous discharge (slurry is injected below the water surface) and thickened discharge (slurry with low water content). The tailing coarse particles (i.e. sand) settle close to the point of discharge, and fine particles (i.e. silt and clay, generally termed as slimes) run down the beach into the impoundment and reside there (Davies, 2002).

Tailings are produced in huge quantities annually, and may contain some toxic chemicals which may be harmful to the environment. Therefore, it is necessary to store tailings in an environmentally safe and economically affordable way. The mixture like slurries is deposited in storages and their processes are done via dam-like structures. Tailings are generally stored in surface impoundments, which commonly consist of raised embankments. The raised embankments are generally constructed with either of the following raising methods: upstream, downstream, or center line (ICOLD, 1989).

It is expressed that storage facilities are an indispensable part of the mining industry and they are designed so as to retrain the tailings. These dams do not only keep the waste

material from the mining process but also there is a huge amount of contaminated water. It should be noted that there are tangible differences between tailing dams and water retention dams. The technology of tailings dam design and geotechnical principles of water dams are based on the same trend; however, “the presence of saturated tailings, solid as the stored medium, versus water only, presents unique challenges and design characteristics”. The gradation of mine tailings obviously varies from silty medium fine sand to clayey silt. The sections of tailings dam design vary significantly. For instance, the dam can be totally made of unprocessed tailings with upstream construction, or the dam may be made of borrowed material with little or no reliance on the tailings. The impounded tailing solids have hydraulic conductivity and shear strength properties that can be used to the advantage of the designer. On the other hand, sulphide-rich mine tailings have a potential to oxidize and leach metals through acid rock drainage. Seepage control for environment – not dam safety – has become a critical design parameter which can lead to much lower tolerances for seepage losses from the impoundment, compared to water dams (ICOLD, 2001).

In contradiction with water dams the tailing dams are usually closed at the end of the life of mine (normally 20 years), but the tailings cannot be detached for withdrawing (ICOLD, 2001). Therefore, design must satisfy the safe withdrawing and low or zero maintenance. This means that the design, construction, operation, and closure of tailing dams have certain fundamental differences when compared to the conventional water storage dams. Some of these differences grow the benefit of the dam designer, whereas some others increase the complexity and difficulty. The typical differences between tailing dams and water dams are listed in Table 2.1:

Table 2. 1 Summary of differences between water dam and tailing dam (Vick 1990)

Component	Tailing Dam	Water Dam
Stored material	Tailing solids, process water (various contaminant levels) & runoff water	Water
Regulatory regime	Ministry of Mines, Ministry of Environment	Ministry of Public Works, Regional Authorities, National Dam Associations
Operating Life	Finite operating life (5 to 40 years)	Typically designed as 100 years, but "as long as required by society".
Construction Period	Staged over mine life of 2 to 25+ years	Usually 1 to 3 years
Closure	Infinite closure period, try for "walk away" design	Often not addressed, but facility may be decommissioned
Engineering	Medium to high level	High level
Continuity of Engineering	Varies: Owner and engineer may change frequently during the construction life	Usually one engineering firm to design and construction
QA/QC	Generally good for starter dam and variable levels during operations. Can be at a low level for some mining companies	High level
Consequence of Failure	Tailing Debris flow resulting in physical damage and environmental contamination	Water inundation damages
Dam Section	Can vary during the design life e.g. transition to centerline, or downstream	Usually a consistent section

Moreover, the construction of tailing dams' progresses stage by stage and their height is raised over time.

As it can be seen, the impoundments of tailings slurries are the most usual method of disposal and these impoundments are preferred because between other methods, they are "economically affordable and relatively easy to operate" (Environment Canada 1987). The other factor is that tailings impoundments can be and are designed to perform multifunctional, including treatment functions. These include (Environment Canada 1987):

• Removal of suspended solids by sedimentation
• Precipitation of heavy metals as hydroxides
• Permanent containment of settled tailings
• Equalization of wastewater quality
• Stabilization of some oxidizable constituents (e.g., thiosalts, cyanides, flotation reagents)
• Storage and stabilization of process recycle water
• Incidental flow balancing of storm water flows.

Tailing dams are designed and constructed in three different types (Vick, 1990). The “Downstream” type is the method that raising progress is in the direction of flow and whiles in “Upstream” method the dam is raised through the other side of body which called as upstream side of the dam. Finally, if the structure raises the dam on both sides, this is the “Centerline” method (Vick, 1990).
Before going in detail for these three major types, it should be considered for retain tailings in impoundments, there are three basic types of structures, the raised embankment and the retention dam. Since the raised embankments are the most common than retention dam, the three major types of impoundment construction consider being in “Ring-Dike”, “In-Pit” and dug pit types. The factors such as economical, natural topography and site conditions are mainly considerable for optimal design (Vick, 1990).

Today, most operational tailings dams are in the form of the Valley design. Since, costs are usually related to the amount of the materials which are used for the construction of the embankment or dam (i.e. its size), main savings can be recognized by shrinking the

size of the dam and by maximizing the use of materials in vicinity, mainly the tailings themselves. On the other hand, using natural depressions to contain tailings is another advantageous reason which causes the dam sizes reduction, As far as the sides of the valley serve to contain tailings. Valley-type impoundments as it is shown in Figure 2.1 can be constructed individually, where the tailings are deposited behind a dam or embankment; or in multiple forms of the dam, in which case a series of embankments contain the tailings in connected "stair-step" impoundments (Vick, 1990).

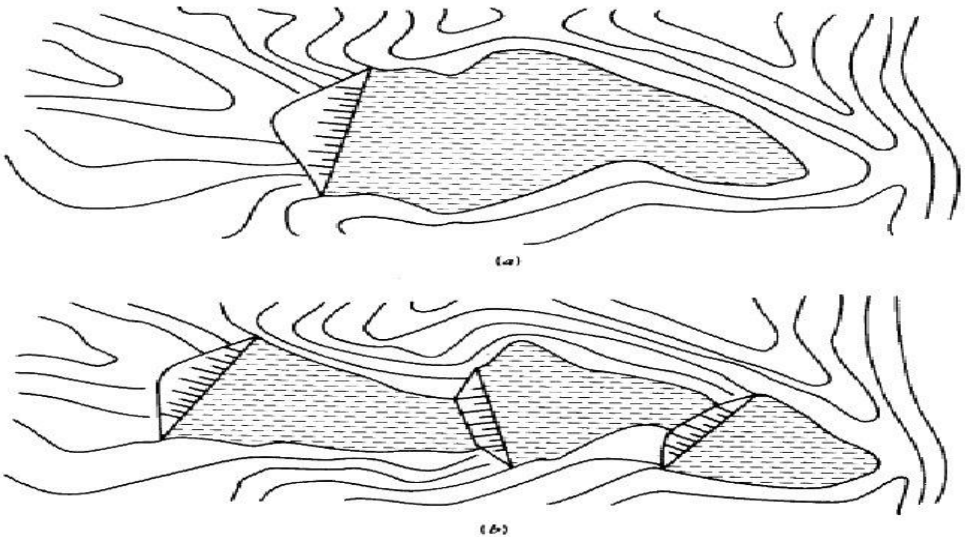

Figure 2. 1 Single (a) and Multiple (b) Cross-Valley Impoundments (Vick 1990)

The other types of the valley impoundments are applied in some other conditions like disproportionately large drainage catchment areas or the lack of proper valley topography. One of them is the side-hill impoundment, which is divided into two subsections, single and multiple types. As it is shown in Figure 2.2, the layout which consists of a three-sided dam constructed against a hillside is called side-hill. Actually, this design is appropriate for a slope of less than 10 per cent. The sharper slopes need more filling material and in the case of downstream method, it is not affordable to utilize this method (Vick, 1990).

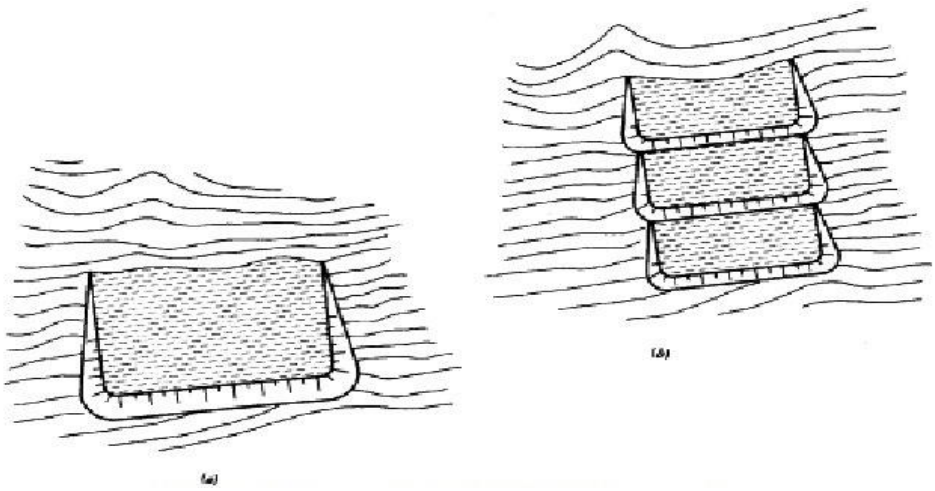

Figure 2. 2 Single (a) and Multiple (b) Side-Hill Impoundments (Vick 1990)

In the absence of natural topographic depression, the Ring-Dike impoundment formation is appropriate. As it is shown in Figure 2.3, instead of one large embankment, more embankments are needed to contain the tailings in this type of impoundment. The same method of construction is employed as in valley dams but due to the length of dike/dam, more local materials are used in the further phases of the construction (Vick, 1990).

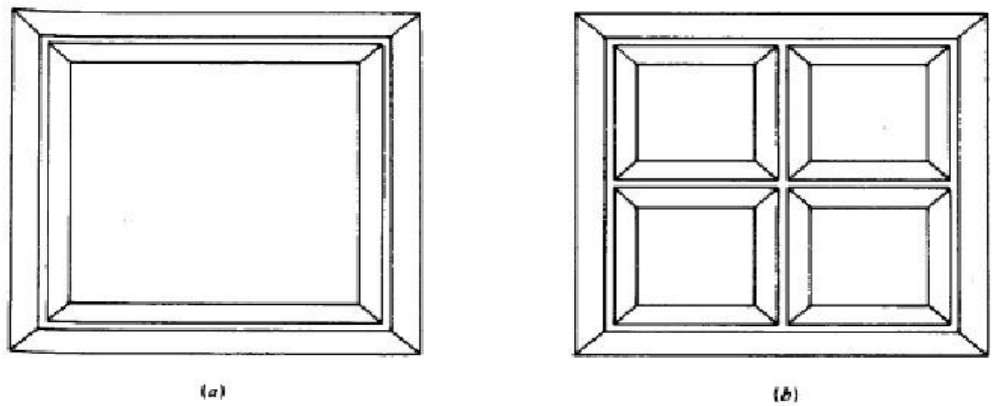

Figure 2. 3 Single (a) and Multiple (b) Ring-Dike Impoundment (Vick 1990)

As it is mentioned, unlike water-retention dams, tailing dams cannot be constructed to completion but raise consecutively as the impoundment fills. As it is mentioned before

(see Chapter 1), There are three different methods for dam construction for sequentially raising the tailing dam body. These methods include Upstream, Downstream and Centerline which are shown in Figure 2.4. As it can be seen, for each of them is constructed in four rising lifts by using the construction material and storage capacity increases gradually with succeeding lift. Since the infilling materials and the cost of placement are associated with the life of dam, the total cost is much lower than retention dams (Vick, 1990).

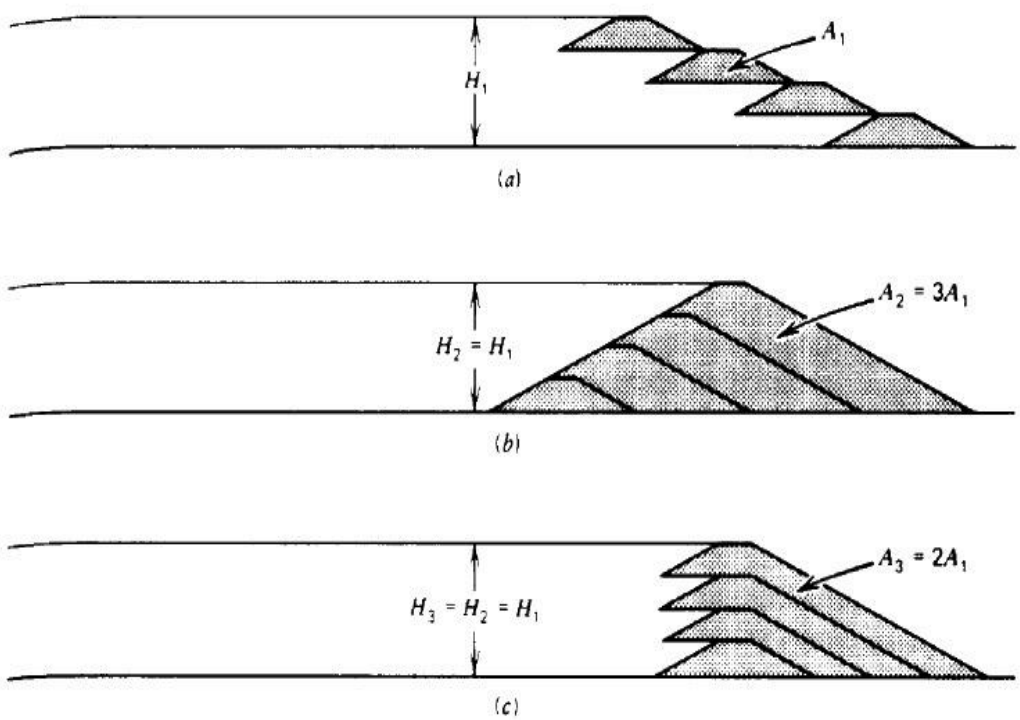

Figure 2. 4 Types of sequentially raised tailings dams (a) Upstream (b) Downstream (c) Centerline (Vick 1990)

Recently, the upstream method is mostly preferred for the construction of the tailing dam by the prominent mining companies around the world (Wei, 2008).

In the upstream embankments, which are the most popular type, new parts of the dam will be constructed on top of the older parts of slurries impoundment (see Figure 2.5) and the dam crest moving “upstream”. Therefore, the method is called upstream method (Vick, 1990).

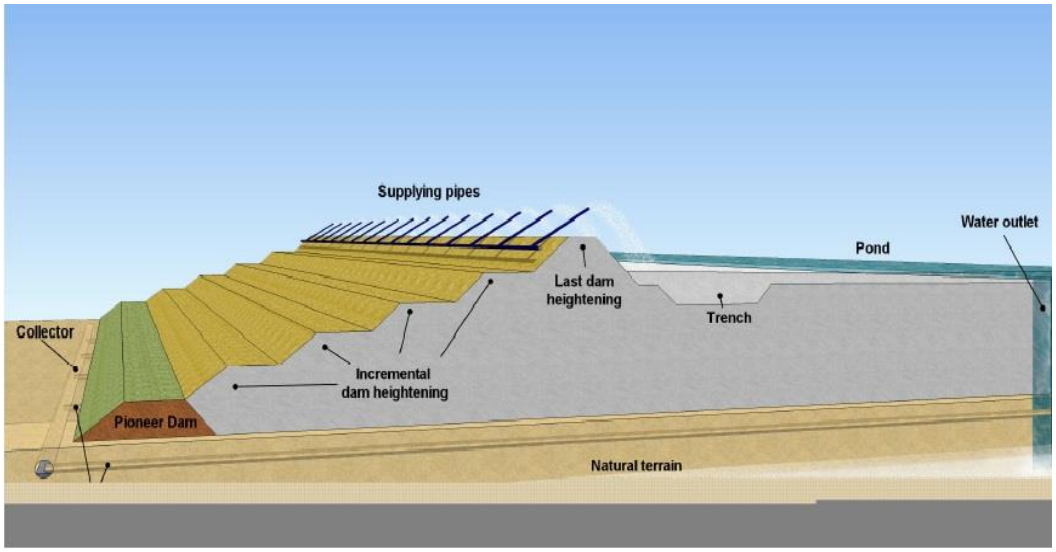

Figure 2. 5 Upstream Dam Particles (USSD, 2010)

It seems that the lower cost of the upstream method has made it more applicable, yet it should be noted that constructing this dam needs substantial care and attention as it has a higher risk of failure than the other methods. Hence, the stability of dam stands out as the main concern in the construction of tailing dams which employ the upstream method.

Another drawback of the upstream method is the susceptibility to liquefaction when severe seismic movements occur, which is caused by earthquakes, mine blasting operation or even due to the heavy mining vehicles. Besides, since tailing particles need time to consolidate, there is a limitation for increasing the height of the upstream dam and it is recommended that the raising of crest should not be more than 15 meters annually, otherwise the particles do not settle appropriately and it might cause excessive pore pressure within the deposit and accordingly a reduction in stability (ICOLD, 2001). Due to the aforementioned items, the comparison of the methods is given in Table 2.2:

Table 2. 2 Comparisons of Surface Impoundment Embankment Types (Vick 1983)

Comparison of Surface Impoundment Embankment Types				
	Water Retention	**Upstream**	**Downstream**	**Centerline**
Mill Tailings Requirements	Suitable for any type of tailings	At least 40-60% sand in whole tailings. Low pulp density desirable to promote grain-size segregation	Suitability for any type of tailings	Sand or low-plasticity slimes
Discharge Requirements	Any Discharge procedure suitable	Peripheral discharge and well-controlled beach necessary	Varies according to design details	Peripheral discharge of at least nominal beach necessary
Water Storage Suitability	Good	Not suitable for significant water storage	Good	Not recommended for permanent storage. Temporary flood storage acceptable with proper design
Seismic Resistance	Good	Poor in high seismic areas	Good	Acceptable
Raising Rate Restrictions	Entire embankment constructed initially	Less than 4.5-9 m/yr most desirable. Greater than 15 m/yr can be hazardous	None	Height restrictions for individual raises may apply
Embankment Fill Requirements	Natural Soil Borrow	Natural Soil, Sand, tailings, or mine waste	Sand tailings or mine waste if production rates are sufficient, or natural soil	Sand tailings or mine waste if production rates are sufficient, or natural soil
Relative Embankment Cost	High	Low	High	Moderate

As it can be seen in Table 2.2, the upstream method is much more affordable but less stable than the others.

Mine tailings are leftover materials of the mining operations produced during the extraction of valuable minerals from ore. It should be distinguished from waste rocks that are discarded along with soil and organics, which is called overburden. The particle size and the composition of mine tailing depend on the type of ore that is extracted and the mining method. For instance, the hard-rock metal mining ore is crushed and after the mineral processing, the particles may be found very fine and also may be used for mineral extraction because it contains chemicals (Vick, 1990).

Generally, tailing materials include ground rock and waste materials which are generated in processing part of the mine. For extracting the wanted product in mining plant some mechanical and chemical process are used. This process produce waste material which is called tailings and these tailings source is the inefficiency percentage of product extraction. Moreover, it is not possible to reuse the chemicals and applicable

materials (Ritcey, 1989). The unrecoverable and uneconomic metals, minerals, chemicals, organics and process water are discharged, normally as slurry, to a final storage area commonly known as a Tailings Management Facility (TMF) or Tailings Storage Facility (TSF). Not surprisingly, the physical and chemical characteristics of tailings and their methods of handling and storage are a greatly growing concern. Tailings are generally stored on the surface either within the retaining structures or in the form of piles (dry stacks), but they can also be stored underground in mined-out voids with a process commonly referred to as a backfill. Backfilling can provide ground and wall supports, improve ventilation, provide an alternative to surface tailings storage and prevent subsidence (Ritcey, 1989).

The disposal of mining waste material is a main problem of mining industries and also drastic worries for environmental issues. Most often, the chemical which uses for extracting the material from the ore are toxic (e.g. Cyanid in gold mines) and they remain until the end of the impoundment life. Actually, these materials could leach out into the underground water and cause contamination. Besides, rock could contain the chemicals like arsenic and mercury that are dangerous. On the other hand the acid mine drainage is the common problem in mining. Sulfide compounds are also including in hard rock mines like gold mines and in case of extraction releases and there would be concern to store it in a safe way while this compound produces sulfuric acid in contact with water and air (Davies, 2002).

In modern hard-rock metal mines, usually, tailings are stored in pits which are lined by clay or geo-synthetics. In some big mines the tailings are sending back to the original pit while in some large mines, they use all the possibilities like valleys and lakes. The pits which are using for disposal are usually covered with water, for diminishing the rate of the sulfuric acid. For some cases, huge dams keep back the tailings and water which need the long-term maintenance. The gold mine in Alaska would use such a "tailings impoundment" as it is shown in Figure 2.5. Some tailings impoundments (e.g. at Red Dog Mine) also need ongoing treatment of water that flows out of the storage. In both case of normal and well-treated storage impoundment the long period of mining process should be considered to prevent creation of acid and toxic releasing into the environment (EPA, 1994).

In disposals like “dry-stack” types, the tailings are passing some stages to dry and bury in a covered and lined pit. This method indwells less space and the safety of the impoundment is more satisfactory in term of any seismic activity. Also, this does not need any active water treatment (particularly relevant where precipitation is high) (EPA, 1994).

Figure 2. 6 Earthen tailings impoundment dam at Fort Knox gold mine in Alaska (Northern Alaska Environmental Center, 2004)

Though, the dry-stack method for disposal are enough expensive, specifically, for up-front costs. Another con if this method is about the solid tailings that should be transported by truck vehicle or conveyor instead pipelines. The covering of the pit must maintain in eternity, but its maintenance need less work than a huge tailing storage with dam (Franks, 2011). In Alaska, Pogo Mine, Greens Creek Mine, and Nixon Fork Mine use dry-stacking for tailings disposal, which is shown in Figure 2.6.

Figure 2. 7 The "dry-stack" tailings facility at Pogo Mine, Alaska (Dave Chambers, 2004)

According to the tailing impoundment facility design, there are many factors which effect of site selection in optimum manner and selecting the disposing method as well (Ritcey, 1989). The two factors of environment and ground conditions are the most important parameters that control the storing method of tailing. These items also have ultimately impacts on design of facility, built and operation of mining and closing at the end. So, some alternative method should be considered to store and discharge and these techniques require specific location. These parameters are usually considered in pre-construction stages and project developments while the trade-off studies are performing. Accordingly, the option selection which is applicable in this study can be taken through to the feasibility stage to estimate the risks and operational factors related to the environmental, social and economic aspects with the sufficient level of confidence (ASCE, 2002).

Briefly, there are two topics which are considerable for tailings and these two are production and characteristics. As same as tailing production, the tailing disposal is usually recognized as the most important resource of environment effects for most mining operations (Vick, 1990). This is not unbelievable that the storage of the impoundment is often exceeding the total in-situ volume of the ore which is processed. During the last century, the amount of tailing materials are increased drastically and also higher amounts of ore are extracted due to the advances in mining technology. In

19960's, thousands of tons of tailings were produced per day until 2000 and this amount are increasing day by day (Vick, 1990).

Nowadays, there are some mines which produce more than 200,000 tons of tailing each day. Considering the techniques in mineral processing could help to determine about the ways of production of tailings and methods of storages (Ladd, 1991).

Actually, the characteristics of tailings vary greatly and depend on the mineralogy. The process in term of the physical and chemical trends uses to extract the economic product. Ritcey (1989) reported that the same tailing materials could include different mineralogy and thus will have various chemical and physical characteristics. Consequently, the tailing material properties should be recognized to make the proper decision about the final deposition determination and potential of short and long term liabilities and environmental effects. As soon as, the characteristics of the tailing are obtained from the lab tests and plant tests, the required design requirements can be recognized to mitigate environmental effects and determination of optimal performance of operation.

The deliverance of the water in the impoundment and the volume of it which return pumping to the processing plant are important and also are the main parameters for designing. Moreover it will influence on the water balance of mining project (costs of make-up water). This freeing is dependent on the physical properties of the tailings and can be assessing via laboratory tests which reveal the tailing solid concentrations. These factors can help about determination of the storage type to prevent the discharge of water to tailing storage area (e.g. paste and dry stacking techniques). In addition, it may mitigate the seepage and evaporation (Ritcey, 1989).

Below are the factors which help the determination of the storage design and its requirements. The following characteristics of the tailings should be established (based on EC, 2004):

- Chemical composition (including changes to chemistry through mineral processing) and its ability to oxidize and mobilize metals.
- Physical composition and stability (static and seismic loading).
- Behavior under pressure and consolidation rates.

- Erosion stability (wind and water).
- Settling, drying time and densification behavior after deposition.
- Hard bind behavior (e.g. crust formation on top of the tailings)

2.2 Stability Analysis of Tailing Dams

Generally, designing of the tailing impoundment is associated with the tailing material properties and using the information related to the tailing characteristics, construction material, and specific factors about site like topography, geology, hydrology and seismicity and costs. Also the dynamic interplay between these factors are effecting on the location (or sitting) and the impoundment design. As far as, water is main aspect of tailing, the principles of hydrology (applied to the flow of water through and around the tailings embankment) dictate many of the rules of tailings impoundment design. Actually, the water level is the major parameter which influence on the dam stability, these principles are dramatic concerns in tailing impoundment design (Davies, 2002).

On the other hand, slope stability is another factor that should be considered. In order to assess the slope stability, design and storage processes, which are the main items that affect the evaluation of slope stability during its life, should be established. One of the significant factors in this evaluation is shear strength and the other aspects with the equal value of importance are sample disturbance, variability in materials, possible variation in compaction water content and the density of fill materials, anisotropy, loading rate, creep effects on possibility of partial drainage. Finally, the last one responsible for the design selection is the dam designer and the data from laboratory are the crude information for decision making (Frelund, 1997). Generally, all the factors can be gathered like what is represented in Table 2.3.

To compute the shear strength, there are some stages to be performed, which can be classified as below: (1) Drained and un-drained condition (2) Laboratory Strength test (3) Linear and non-linear strength envelopes; these envelopes correspond to Mohr – Coulomb failure criterion. For total stresses, the equation is expressed as:

$$\tau = c + \sigma \tan \varphi \qquad (2\text{-}1)$$

Where σ is the normal stress, c is the intercept of the failure envelope with the τ axis and τ is the shear strength. Also ϕ is the slope of the failure envelope and the quantity c is usually named the cohesion and angle ϕ is the internal friction angle. Compression is assumed to be positive in the following discussion.

Considering Table 2.3, it is understandable that the loading process of impoundment by taking attention dam stability into consideration could be performed due to three critical conditions.

First method which is known as "at the end of the construction" considers the stability analysis by apply the drained strength parameters in drained soils and undrained ones for low draining soils. To determine and measure the amount of drainage during the construction, the analysis of the construction is obligatory (U.S. Army Corps. Of Engineers, 2003).

Generally, the permeability value greater than 10^{-4}cm/sec for material which is considered to be fully drained through the construction, while the materials with permeability value less than 10^{-7} cm/sec are defined as undrained at the end of the construction (U.S. Army Corps. of Engineers, 2003). In some examples while the drainage is expected considerable but not complete during the construction, stability should assume the fully drained analysis and undrained condition. Also the design should consider less than the obtained amount to prevent any stability consequence (ICOLD, 1989). Some factors which are observed in case of undrained conditions pore pressure are the soil saturation degree, soil density and imposed load on it (U.S. Army Corps. of Engineers, 2003).

Basically, most critical condition should be estimated by statistics and while the drainage of soil is not complete, usually, it is assumed to be fully drained or completely undrained and while we face to the undrained condition, important factors like degree of saturation, soil density and imposed load administer the pore pressure. It should be mentioned that the undrained laboratory test results are not viable in some cases, so there should be a total stress analysis rather than effective stress one. Nonetheless, in

term of free drainage soil stability, the effective stress us much more applicable. Moreover, the staged construction may be obligatory for embankments which are built on soft clay foundation. Consolidated triaxial test in case of undrained is also applicable to determine the strength of partial consolidation during the staged construction (U.S. Army Corps. of Engineers, 2003).

Increasing the pore pressure could cause failures and this might be considered as one of the main factors of instability in both the downstream and upstream methods of construction and generally it will have an effect in both intermediate stages and much more in the full height of dam. On the other hand, some other items touch the pore pressure, like the moisture content and the construction rate and type of the soil. The soil compressibility could increase the pressure specifically in clayey soils. Hence, in low plastic silts and silty sands, according to their low compressibility, low pore pressure may develop (U.S. Army Corps. of Engineers, 2003). For low preamble soils, pore water pressure is assigned zero for the total stress analysis. In case of drained soils, determination of pore water pressure is necessary for stability analysis during and also in period of end of end of construction. Water levels can make to chance for pore water pressure determination just in vicinity of slope (U.S. Army Corps. of Engineers, 2003).

Table 2. 3 Shear Strengths and Pore Pressures for Static Design Conditions (U.S. Army Corps. Of Engineers, 2003)

Design Condition	Shear Strength	Pore Water Pressure
During Construction and at the End of the Construction	free Draining Soils-Use drained shear strengths related to the effective stresses (1)	Free draining soils-Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations if there is no flow, or using steady seepage analysis techniques (flow nets or finite element analyses)
	Low-permeability soils - use un-drained strengths related to total stresses (2)	Pore water pressures from filed measurements, hydrostatic pressure computations for no-flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses, or finite difference analyses)
Steady - State Seepage Conditions	Use drained shear strengths related to the effective stresses	
Sudden Drawdown Conditions	Free draining soils -use drained shear strengths related to effective stresses	Free draining soils- First stage computations (before drawdown) - steady seepage pore pressures as for steady seepage condition.
		Second - and third - stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water level.
	Low-permeability soils - Three-stage computations: First stage - use drained shear strength related to effective stresses; second stage - use un-drained shear strengths related to consolidation pressures from the first stage; third stage- use drained strengths related to the effective stresses, or un-drained strengths related to consolidation pressures from the first stage, depending on which strength is lower - this will vary along the assumed shear surface	Low-permeability soils - First stage computations - steady-state seepage pore pressure as described for steady seepage condition. Second - stage computations - total stresses are used; Pore water pressures are set to zero.
(1) Effective stress shear parameters can be obtained from consolidated-drained (CD-S) tests (direct shear or triaxial) or consolidated-undrained (CU,R) triaxial tests on saturated specimens with pore water pressure measurements. Repeated direct shear or Bromhead ring shear tests should be used to measure residual strengths. Un-drained strengths can be obtained from unconsolidated-undrained (UU, Q) tests. Un-drained shear strength can also be estimated using consolidated-un-drained (CU-R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the "R" or "total stress" envelope and associated c and phi , from CU,R tests should not be used.		
(2)For saturated soils phi=0, Total stress envelopes with phi > 0 are only applicable to partially saturated soils.		

Also pool level is another factor that should be considered. Most often, the crucial pool level for end of construction stability is possibly minimum through the upstream slope. In some others, it would be suitable to take the higher pool level in downstream slope for end of construction stability.

Thus, as it is resulted, pore water pressure can be predicted by steady-state seepage analysis techniques like finite element analysis or flow nets or for non-hydrostatic condition and also hydrostatic pressure computations for no flow condition (U.S. Army Corps. of Engineers, 2003).

Stage construction analysis is another loading condition which is appropriate for all embankment construction types in case of soft foundations. Nevertheless, this analysis is normally carried out for fast constructed upstream embankments. Generally, the upstream embankment rising range is changing between 4.5 and 9 m/year. It is believed that the excessive pore pressure spoils suitably and stage analysis for construction is not considered (Zardari, 2011).

The excessive pore pressure in the first raise of embankment in addition to the initial pore pressure may be generated because of the seepage. The tangible portion of excessive pore pressure in first raise spoils before the second rising. Experimentally, complete pore pressure dissipation may not happens and the residual is always exist. Therefore, after the embankment height raising there would be two kinds of pore pressures, one of which is residual and the other is excessive pore pressure which is caused by the second upraising (Zardari, 2011).

In order to provide sufficient time for the consolidation and pore water dissipation, tailing dams are constructed in stages, which intensify the stability of foundation and embankment. The construction stage of tailings dam is critical for the dam stability. Developing in slimes or in the foundation soil may be caused by positive excessive pore water pressure during the construction of embankment. By utilization of vertical drains, upstream dam stability may improve and by location the berms ate the downstream side it will increase incrementally (Ladd, 1991).

According to the hypothesis, probable dam failures may occur incrementally and pore pressure dissipation which is induced by shear stress may decrease the possibility, so the analysis of staged construction is consolidated drained case. Alternatively, if the

reason of the failure is not pore pressure dissipation which is induced by shear stress, the staged construction which is considered as consolidated undrained strength analysis (UU) is carried out. The factor of safety in case of undrained strength analysis could be acceptable, because the undrained shear strength is less than drained shear strength for typical consolidates soils (Ladd, 1991).

Another condition or the third condition is known as long-term stability analysis which is applicable when there is no quick altering in external loading and embankment reach to the maximum height.

Besides, the pore pressure and the stresses are equal at the inner part of the dam body, so the steady-state seepage condition exists on full level of the reservoir and positive pore pressure has been developed by the soil under the phreatic surface (Ladd, 1994).

In the last condition, it is better to utilize the long-term stability analysis, since there is enough time after the construction to develop the hydrostatic conditions. In term of steady-state seepage the hydrostatic condition is a specific type in which no flow exists. Internal stress and pore pressure are assigned to be in equilibrium condition form the gradually raised embankment due to the staged construction and it is predictable which the steady-state seepage condition advance when the embankment attains the maximum height. However, all this granted offers a low prediction of long-term pore pressure.

If any further description could be considered about the status in the long-term condition, some factors like shear strength properties, pool level, surcharge pool and pool water pressure should be observed.

By definition, in the long term all soils, regardless of permeability, are assumed drained. In term of effective stress parameters (c', φ'), the long-term condition is analyzed, but by considering the drained strength parameters (Cedegren, 1977).

For the level of the pool, the spillway crest elevation or maximum pool storage is the maximum level of water which is assumed to be maintained until producing the steady-state seepage condition. While the intermediate pool level is considerable in the stability analysis, it should be ranged from none to the maximum storage pool level. Also, this level is predicted to exist long enough to develop steady-state seepage (Cedegren, 1977). Often, the temporal pool which is known as surcharge pool is higher than the storage pool. Adding the load to the driving force but normally does not continue long

enough to steady seepage condition. There is an assumption that intermediate pool levels exist over a period long enough to develop steady-state seepage. Moreover, for all the analysis, the tail-water levels could be suitable for different pool levels (U.S. Army Corps. of Engineers, 2003). It is noticeable that the steady state seepage condition for the downstream slope is hazardous for downstream designed slope because of the seepage forced toward the slope. The downstream side stability condition must be analyzed while the surcharge pool level is maximum. The surcharge pool level stability analysis should be carried out by considering the drained strength properties. Also it should be assume that the possibility of steady-state seepage at the surcharge pool level (Fellenius, 1936).

By using the drained strength due to the effective shear strength, the long-term stability is analyzed and in order to do this analysis proper pore pressure according to the long term condition is employed. Also, in this condition there is no attention to soil permeability and all of them are assume drained (U.S. Army Corps. of Engineers, 1990). As it is mentioned, for all analyses, the appropriation of tail-water levels for various pool levels is important. Besides, the pore water pressure used in the analysis should signify the field condition of water pressure and the steady-state seepage in the long-term condition. The analyzed pore pressure can be estimated from (1) Filed measurement of pore pressures in existing slopes, (2) Past experience and judgment (3) Hydrostatic pressure computations for conditions of no flow, (4) Steady-state seepage analyses using such techniques as flow nets or finite element analyses (U.S. Army Corps. of Engineers, 2003).

As it is documented, in most cases, pore pressures may result in boost shear strength while omitting the shear induced by pore pressure and it should be considered in the long-term analyses. Also, some researches claimed that effective stress parameters could be used in the long-term analysis, assuming any shear-induced pore pressures at failure. Sometimes, very loose tailing deposits characteristics are similar to the naturally sensitive clay deposits. A primary slide in a sensitive clay slope may cause in a fast changing in the loading condition at the slope toe and may cause a slides resulting in slope failure (Bishop, 1955). In fine, conventional dams are viewed as an asset. As a result, their construction, loading construction, operation, and maintenance receive a

high standard of care and attention from the owners, who often retain in-house dam engineering expertise unlike tailings dams, which have been viewed by their owners as an unprofitable, money-draining part of the mining operation (Bureau, 2003).

2.3Failure Modes and Slope Analysis Criteria

As it was mentioned before, in order to determine the safety factor of tailing embankment, a sort of analysis through the failure surfaces is required. There are some different failure mechanisms for tailing dams which are vulnerable in some ways (Table, 2.4).

Table 2. 4 Failure Mechanisms of Tailing dams + causes and remedial measurement (ANCOLD, 2011)

Failure Mode	Cause	Remedy
Overtopping	> Inadequate hydrological or hydraulic design > Loss of freeboard due to crest settlement	> Gabions, mine pit waste or surrounding borrow material may be quickly imported to aid the strength of embankment > Opening of emergency pumps and spillways
Slope Instability	> Overstressing of foundation soil and dam fill > Inadequate control of pore pressure	> Soil reinforcement and strengthening measures > Installing a drainage trench at the toe of downstream face and/or horizontal bore drains. Filters can prevent the entry of fill material into the drain
Internal erosion	> Inadequate control of seepage > Bad filter and drain design > Poor design or construction control resulting in cracks or leakage through conduits	> Raising downstream embankments with drainage blanket > Installation of horizontal bores to relieve pressure > Installation of deep trenches towards downstream face
External erosion	> Inadequate slope and toe protection	> Vegetation of the downstream face > Placing crushed mine waste on the downstream embankment face > Construction of berms on downstream face > Placing rock fill such as mine pit waste adjacent to the toe > Filling of crack with suitable material
Earthquake action	> Steep Slopes > Liquefaction of embankment and foundation soil	> Filling of cracks with a suitable material
Damage to decant system	> Excessive settlement > Chemical attack on concrete/steel	> Opening of emergency pumps or spillways

The most probable failure mechanism for the tailing embankment is circular or rotational failure. It is called rotational failure because of the failure shape, which is like a horizontal cylinder or the segment of a circle, and occurs along the face of an embankment. In fact, for a stable dam slope, the item that resists in front of movement is shear strength. Instability appears while the shear stress and shear strength are equal (Vick, 1990). Rotational failure as shown in Figure 2.8 is because of the inappropriate control of pore pressure, soil overstressing, water table change, permeability of foundation soil altering, disturbance of embankment by vibration and foundation material settlement (CANMET, 1977).

Figure 2. 8 Circular or Rotational Failure Mechanism (Calgary Tailing Dam, 1998)

Thus, the transitional or rotational movement is considered to take place on the assumed or known potential slip surface below the soil mass and this failure mechanism is one of the most popular failure modes in tailing dams.

The second failure mechanism called as the foundation failure is a well-known failure for the earth fill structures. As it is shown in Figure 2.9, when a weak soil or rock layer is found in the sophomoric dam foundation depth below the structure, Movement play an important role through the failure plane if the structure produce bigger stress than strength of the soil in weak layer (CANMET, 1977).

Figure 2. 9 Foundation Failure (Hungary tailing dam, 2010)

This suggests that the key factor of the foundation instability is the location of phreatic line where the fully saturated zone within the embankment exists. In an unsafe dam this level is upper than estimated and will cause the sliding of the zone beneath the phreatic line. To avoid this failure mechanism, the most important factor is to establish an appropriate drainage system (CANMET, 1977). Almost 20% of dam failures have been caused by piping (internal erosion caused by seepage). Seepage often occurs around hydraulic structures, such as pipes and spillways, through animal burrows, around roots of woody vegetation, and through cracks in dams, dam appurtenances, and dam foundations (CANMET, 1977). Piping presents the subsurface erosion through the seepage lane within or beneath the dam, which ends up with the generation of channel permitting concentrated flow. Sometimes the cause of piping is the embankment face seepage with proper and enough velocity which could erode the dam face. Toe is the first place of the downstream dams in which erosion is initiated and progresses through the reservoir by forming channels or pipes under the dam. The progressive erosion is promoted due to the void space in the direct channel creation from the tailing impoundment to the dam face (CANMET, 1977). Thus, excessive piping may culminate in a local or general failure of the foundation or embankment like what is shown in Figure 2.10.

Figure 2. 10 Failure caused by piping (Ajkai Timfoldgyar / Kolontar tailings dam, 2010)

For high precipitation, protection measures against erosion are needed (Figure 2.11). The two other main places which are vulnerable to erosion are embankment faces and abutments. The erosion may affect from the storm-water along the contact line among the abutment and the embankment (CANMET, 1977). Normally, erosion could be avoided by use of storm-water diversion methods, so the faulty design or maintenance might be the reasons. Embankment faces erosion may be caused from the tailings lines ruptures installed on the embankment crest. Along the contact among the abutment and embankment may suffer from the erosion because of the storm water flow concentration (USBR, 1987).

Figure 2. 11 Gümüş Tailing Dam failure cause by the Face erosion (Gümüş, 2011)

For cross-valley dams, the common modes of failure are liquefaction, which takes place when the toughness and the soil strength are reduced by an earthquake or other types of quick loading. Tailing deposits generally include unconsolidated soil, with grain sizes similar to those in saturated deposits and they are vulnerable to temporary suspension in water (Vick, 1990). When the soil saturated, liquefaction may occur. Briefly, the spaces among soil particles are totally filled with water. The soil particles are suffered from the pressure which is applied by water and effects on the particle packing (Davies, 2002).

The behavior of liquefied tailings seems as a viscous fluid which may pass through the thin openings and flow to dramatic distances (CANMET, 1977). Even in small dam failures, the considerable releasing of suspended impounded materials is probable. On the other hand, the mining operations like blasting may trigger and increase the water pressure, besides earthquake phenomenon (Davies, 2002).

Vick (1977) represented the factors which affect the liquefaction potential as follows:

• *Soil type* - Uniform grain size materials, mostly in fine sand sizes (the typical gradation of a tailings material), are the most susceptible to liquefaction.

• *Relative density or compactness* - The more compact or dense a given material is, the bigger resistance it will cause. It will result in liquefaction.

• *Initial confining pressure at the time subjected to dynamic stress* - This offers an opportunity in certain areas to prevent liquefaction by applying overloads to loose deposits.

• *Intensity and duration of the ground shaking* - Liquefaction may occur due to an intensive earthquake, or due to prolonged earth movement.

• *Location of the water table* - A high water table is damaged. Consequently, a tailings deposit constructed on a pervious foundation or a dam with a phreatic line kept low by providing adequate internal drainage features may have a greatly reduced potential for liquefaction (Vick, 1977).

Drainage facilities and maintaining a low pond surface compact the fill materials at the construction stage and confining pressure can control the probable liquefaction. If the tailing embankment is constructed by compacted fine sand, it will increase the density and also reduce the susceptibility of liquefaction as in Figure 2.12. In more than 60% of

the instances of this compaction of dam material to achieve the range of densities, satisfactory protection is provided (CANMET, 1977).

Figure 2. 12 Disaster caused by liquefaction (Aberfan Disaster, 1966)

The embankment could be stable against liquefaction failure but this is only possible by enabling that the phreatic surface is maintained below the embankment surface or the embankment materials possess a relative density of 60% or greater (U.S. Environmental Protection Agency, 1994).

Overtopping, which commonly occurs due to flooding, is the pervasive failure cause. The reason that stimulates the overtopping failure is the volume of run-on entering the impoundment. It can enter through inappropriate surface water flows or excessive storm-water flows which surpass the impoundment capacity. Since the tailing consists of destructive materials, the friction caused by rapid flow over the instable dam crest may rapidly erode a ditch in the fill material. Moreover, the increasing of pore pressure created by large storm-water inflow may cause the liquefaction of unconsolidated impounded sands and slimes. So, a major failure can occur like the one illustrated in Figure 2.13, as a result of the sustained high flow over the crest of an embankment within minutes (CANMET, 1977).

Figure 2. 13 Overtopping failure in tailing dam in Virginia, South Africa (tailing.info. 2012)

Analyzing the tailing dam incidents may provide us with useful information about the failure mechanisms and the most probable mode of failures in tailing dams. Recently, some researches have been conducted by environmental associations to represent the reasons of tailing dam failures all over the world. ICOLD (International Commission on Large Dams) reported more than 220 dam failures according to the USCOLD (U.S Commission of Large Dams), which documented all the observations and researches about the causes of dam failures and instabilities. In most cases, a dam failure results from a combination of different reasons such as seepage, overtopping and foundation failure (ICOLD, 2004).

On the other hand, the effects of tailing dam failures include certain aspects that grab the attention to a bigger consideration about the dam stability.

Table 2. 5 Tailing Dam Failure Severity Impact (Robertson GeoConsultants Inc, 2003)

Consequences Severity (Directed Costs)	**Biological Impacts and Land Use**	**Regulatory Impacts and Concerns**	**Public Concern and Image**	**Health and Safety**
Extreme (>$ 10M)	Catastrophic Impact on habit (irreversible and large)	Unable to meet regulatory obligations or expectations; shut down or severe restriction of operations	Local, international and NGO outcry and demonstrations, results in large stock devaluation: severe restriction of license to practice	Fatality or multiple fatalities expected
High ($1-$10M)	Significant , irreversible impact on habit or large, reversible	Regulatory (more than once per year) or severely fail regulatory obligations or expectations- large increasing fines and loss of regulatory trust	Local, international or NGO activism resulting in political and financial impacts on company license to do business and in major procedure or practice changes	Severe injury or disability likely: or some potential for fatality
Moderate ($0.1-$1M)	Significant irreversible impact on habit	Occasionally (less than one per year) or moderately fail regulatory obligations or expectations- fined or censured	Occasional local, international and NGO attention requiring minor procedure changes and additional public relations and communications	Lost time or injury likely: or some potential for serious injuries; or small risk of fatality
Low ($0.01-0.1M)	Minor impact on habit	Seldom or marginally exceed regulatory obligations or expectations. Some loss of regulatory tolerance, increasing reporting.	Infrequent local, international and NGO attention addressed by normal public relations and communications	First aid required; or small risk of serious injury.
Negligible (< $ 0.01 M)	No measurable impact	Do not exceed regulatory obligations or expectations	No international/ NGO attention	No concern

By considering the aforementioned information in Table 2.5, which indicates the direct cost of dam failures, the necessity of tailing dam stability analyses is much more tangible. There are several methods to analyze the tailing dam safety, yet the conventional methods of slope stability analysis are based on the concept of limit equilibrium and they enable the calculation of the minimum factor of safety, which is a unit-less indicator of the stability. The aim of the method is to analyze the stability of any mass of soil or rock, assuming incipient failure along a potential sliding surface. As it can be seen in Figure 2.14, the failure surface of a simple shape is assumed and the surface is considered to be a free body.

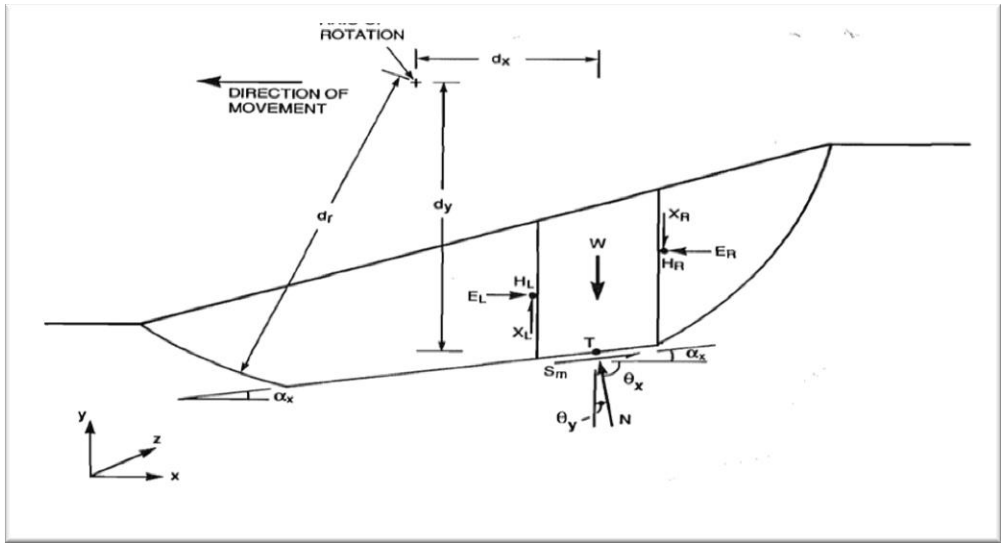

Figure 2. 14 Schematic of Limit Equilibrium Method as an analytical view (Bishop, 1955)

The cross-section through the failed mass in the x direction shows the common axis of rotation and the forces on the column. d_x is the moment arm for the weight of a column; d_y is the vertical distance from the axis of rotation to the center of the column base, d_r is the moment arm for shear resistance on the circular portion of the slip surface; E_L is the inter-column normal force on the left front plane of a column; H_L is the horizontal inter-column shear force on the left front plane of a column; X_L is the inter-column shear force on the left, the side plane of a column; W is the weight of a column; X_R is the inter-column shear force on the right side plane of a column; H_R is the horizontal inter-column shear force on the right front plane of a column; E_R is the intercolumn normal force on the right front plane of a column; T is the horizontal shear force at the base of a column in a plane perpendicular to movement; α_X is the angle between the horizontal and the shear force at the base of a slice in the direction of movement; θ_X is the angle between the horizontal and the normal force at the base of a column in the plane of movement; S_m is the shear force mobilized at the base of the column in the plane of movement; and θ_y is the angle between the vertical and the normal force at the base of a column in the plane of movement.

The sliding mass is divided into a number of slices. The disturbing and resisting forces above the assumed failure surface are estimated to enable the formulation of equations concerning the force equilibrium or moment equilibrium (or both) of the potential sliding surface (Tavenas. F and Leroueil, S. 1980).

Obviously, the limit equilibrium methods based on analytical solutions consider: the weight of the sliding block (W), c, ϕ, u, the geometry of the slope, seismic acceleration, the position of tension crack and external loads. They are used for the design and selection of the appropriate remedial measure.

Due to the malfunction in limit equilibrium method, in some cases such as the lack of concentration in (i) the stress distribution within a slope either above or below the assumed failure surface, (ii) the progressive failure actually happening due to break-up of the mass and the associated stress redistribution and deformations, the deformation analysis would be applicable.

The overall stress distribution controls the deformations, movements and development of failure zones within a slope depending on its material properties, and stress concentrations may have a significant influence on the initiation and growth of a sliding surface. Therefore, the knowledge of stress and displacement distribution in the slopes is a requirement and this can be analyzed by using numerical techniques, such as (i) the Finite Element Method (FEM) and (ii) the Distinct Element Method (DEM) (Tavenas, 1979).

In the following chapters, the failure mechanisms and causes will be analyzed using the Finite Element Method (FEM).

CHAPTER 3

GENERAL INFORMATION ABOUT STUDY REGION

3.1 Description

Emet is located 100 km southwest of Kütahya, which was one of the important ceramic centers in Anatolia in the past. There are two main borax mines in the Emet region. One of them is the Espey Colemanite mine, 5 km to the northeast of Emet, and the other one is the Hisarcik Colemanite mine, 11 km to the south of Emet (Figure 3.1). The Emet Borax mines were first discovered by Gawlik (unpubl.) during the lignite exploration work for the Mineral Research and Exploration Institute of Turkey (MTA) in 1956 (Tosun, 2011).

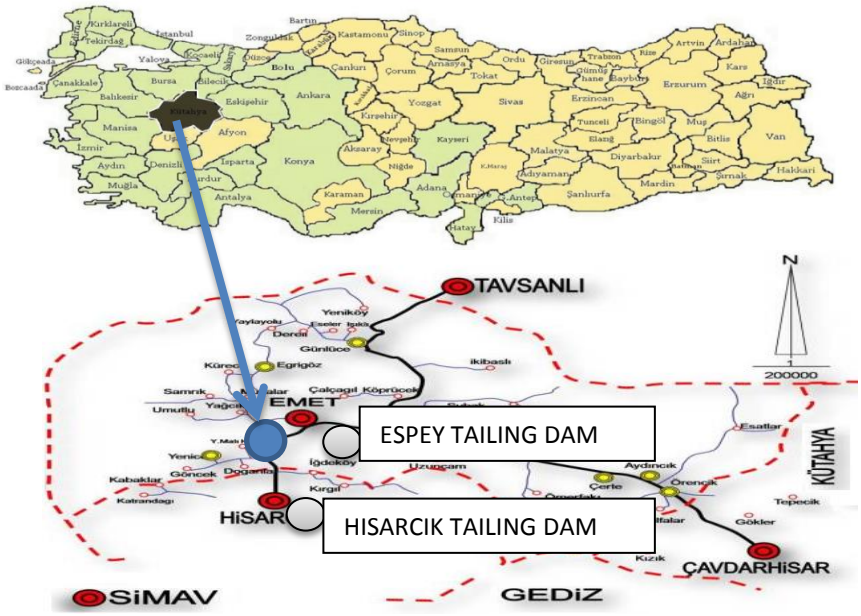

Figure 3. 1 Emet Borax Mine Tailing Dam (JMS, 2012)

In 1958, the Colemanite Mine of Emet has been established by the Etibank-Emet, but an operation site was put into operation in 1962 (Tosun, 2011). Again in 1998, it was degraded to the construction site and the determined area for this site is located 4 km to the south of Hisarcik, which is in “HAMAM KÖY” region, and 3.5 km to the north, which is in the Emet city in Espey region (Tosun, 2011).

The products of the mine are processed and concentrated in Hisarcik and Espey and prepared for sale. Besides, in 2004 an important factory was established, which produces more than 100,000 ton/year of “Sodium Borate” and because of an increasing demand for tailing impoundment, Hisarcik tailing dam was constructed (DSI, 2002).

3.2. Geological Information of the Dam Area

The geological information of the vicinity of Emet mine has been prepared by the JMS Company according to the geotechnical investigation of tailing dams (Baget, 2013). This report contains detailed data about the formations and the age of the ground layers. According to the information, Emet area and its vicinity are surrounded by the Cenozoic and Paleozoic units. The rock formations within the region are classified into several metamorphic groups (JMS, 2013). One of the main formations is Saricasu formation, which includes the Paleozoic period, which is the oldest unit of the study area. This unit contains mica-quartz schist, shale, meta conglomerate, phyletic and quartzite layers. All these units show the lateral-vertical transitions since the argillaceous schist and phyllite layers are weak but mica-quartz schist is in medium hardness, and quartzite layers are significantly resistant. The visible thickness of the Saricasu formation is around 1000-1200 m (JMS, 2013). Moreover, another major formation near the Hisarcik area is Kirkbudak, which dates to the Mezozoic (Jurassic) period. This unit is located outside the dam region and sandstone, siltstone and sandy limestone are the most explored layers (JMS, 2013). The sandstones are dark yellow, brown colored and cemented calcite in the neighboring areas and the sandy limestone is a mixture of white, gray, and green (sandstone and limestone are thick-bedded and sparsely fractured). The formation is mid-competent and the apparent thickness of the formation in Kirkbudak is about 200

m (JMS, 2013). Between the aforementioned formations, the Kizikbuk formation belongs to the Cenozoic and consists of dark yellow colored convex coal and interbedded tuff, sandstone, marl, clay and argillaceous limestone. These layers are the main properties of the Espey tailing dam foundation. Sandstone and argillaceous limestone have medium strength, while marl and clay have low-medium strength. The apparent thickness of Kizilbuk formation varies between 300 and 600 m (JMS, 2013). The Hisarcik area is surrounded by the Emet formation, which includes white, cream-colored, thick-bedded, lacustrine clay and also limestone. In fact, the main layer in this region is made up of limestone but on the upper level, it is much fractured and the decomposition reduces the strength, whereas the lower levels are pretty tough and rugged (JMS, 2013 and Tosun, 2011). Since the study is going to analyze the stability of Hisarcik tailing dam, which is considered in this location, the detailed site investigation could represent more accurate information about the conditions.

3.3 General Characteristics of "Hisarcik" Tailing Dam

The Emet Borax mine is located to the southwest of Kütahya province, which is one of the most important Borax mines in Turkey. The site is located 4 km to the south of Hisarcik, between "Hamamkoy", and 3.5 km to the north of Emet at the region, which is called Espey. In fact, the extraction and concentration process is performed in Hisarcik and Espey, which is the mineral processing center to prepare products for sale. In 2004, the factory was established with a capacity of 100,000 tons and it is obvious that the mentioned factory needs a proper tailing dam (Tosun, 2011).

The construction project of Hisarcik tailing dam has been launched around the concentrator, which is one of the main three facilities of Emet mine and determined as a suitable field for tailing impoundment (Figure 3.2). The geotechnical analysis has been launched to investigate the stability of the dam in front of natural phenomena and hazards (Tosun. 2002).

Figure 3. 2 Hisarcik Tailing Dam - Vision from North (JMS, 2013)

The Hisarcik tailing dam project was launched for Emet mineral processing plant, which is one of the three main facilities of the Emet Borax mine complex, and therefore the definitive decision was made to evalute its stability for the possibility of any failure occurrences. One of the analyses was about the horizontal and vertical displacement and the safety factor of dam which was considered to estimate the stability of the tailing dam. Consequently, the study was launched to predict the construction design method and the disaster potential of the tailing dam and any remedial measurements that are required (Tosun, 2011).

The capacity of Emet Borax mine is estimated at about 100,000 tons/year of Borax (sodium borate). The capacity of the estimated waste impoundment was about 9,079,456 m^3 and the decision to increase the capacity was taken and this amount would be about 4,314,351 m^3 (JMS, 2013). The height of the dam is about 35 m and the dam elevation is about 791 (JMS, 2013). The total area for the Hisarcik tailing dam, which is shown in Figure 3.3, is about 3809 m^2, which is projected in 1:5,000 scale maps (Tosun, 2011). The planning of the tailing dam was performed by the geological and geotechnical surveys that were adjusted to the Solid Waste Control Regulation (SWCR). The total area of the impoundment with the body of the dam project and the

additional waste is approximately 19 hectares in size. To select the appropriate location of a tailing dam, the proximity of other dams, residential units, construction and operation expenses are considered (Tosun, 2011).

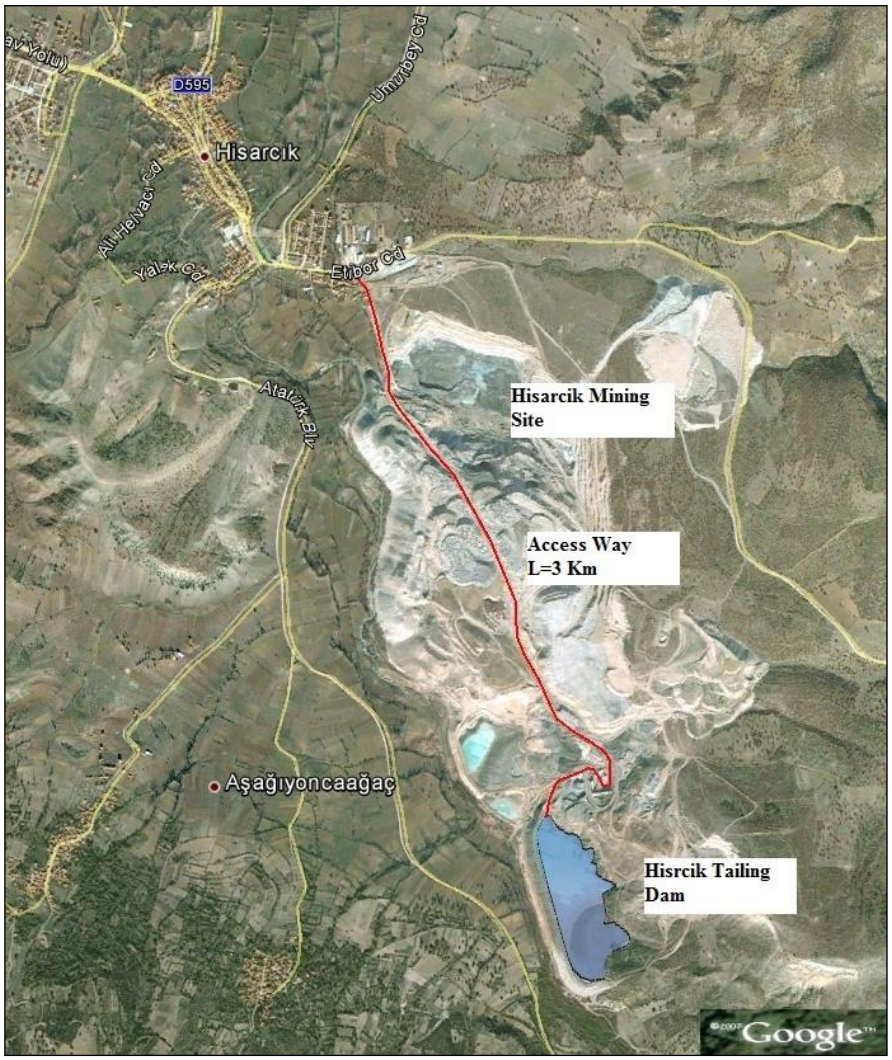

Figure 3. 3 Hisarcik Mining Facilities (Google Earth, 2013)

In order to assess the current status of the foundation, surveys were initiated while the tailing dam begun to develop. Eleven geological engineering reports were prepared and basic drilling was carried out to obtain data about the water pressure and the ground

water level in the impoundment. The impermeability of the membrane was achieved by using the leakage analysis (Tosun, 2011).

According to the geological information related to the dam location, the material types of the dam foundation consist of clay, gravel and sand at the thin upper level and there is a limestone layer which is the main formation embedded in deep level (Tosun, 2002). The impoundment foundation is illustrated in Figure 3.4.

Figure 3. 4 Hisarcik Impoundment (DSI, 2002)

In Hisarcik tailing dam, a geo-synthetic clay mask was applied as shown in Figure 3.5 (DSI, 2002).

Figure 3. 5 Hisarcik Tailing dam body and foundation

Pins and needles are used to link each other and geo-membrane with composite material laminated as it can be seen in Figure 3.6 (Tosun, 2011).

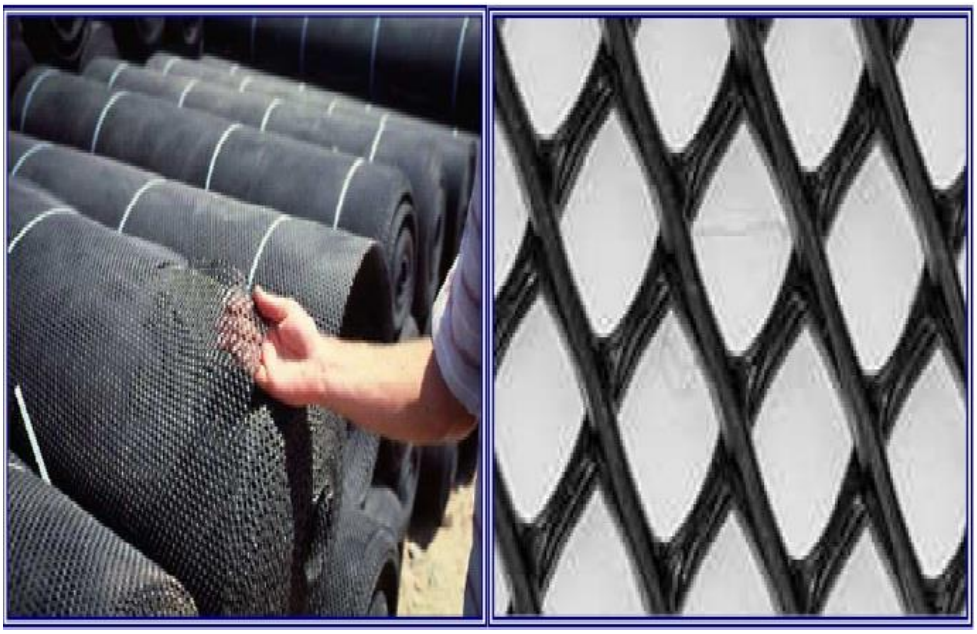

Figure 3. 6 Geotextile & application of dam (DSI, 2002)

The dam was constructed in three stages; the height of dam was about 17.5m from the ground. At the second stage, the height was raised for 11.5m and the crest elevation was selected as 791.000m and at this stage, the clay core was designed in a near-vertical position. In the third stage (filling stage), the elevation changed from 796.00 to 797.00 and there was no transition zone between the clay core and the rock fill. The dam impoundment was coated with 50 cm of impermeable clay blanket and connected with the clay core (Baget, 2013).

The characteristics of the dam, which was built in the type of rock fill with clay core, are given in Table 3.1.

Table 3. 1 General Characteristics of Hisarcik Tailing Dam (JMS, 2013)

Dam Characteristics	Properties
Dam type	Upstream inclined clay core rock fill
Dam purpose	Waste storage
Height (from ground)	17,5 m (I. step)
	29,0 m (II. step)
	34,0 m (III. step)
First stage elevation	779.50 m
Second stage elevation	791.00 m
Current condition crest elevation	796.00 m
Solid tailing volume	2.130.000 m^3
Upstream slope	45°
Downstream slope	$\mathbf{22}^\circ$

In order to increase safety, a 30-m-wide and 15-m-high rock fill that is 950m from the ground was constructed and for this aim 750,000 tons of rock filling material was transported to the toe from the Hisarcik concentration section (Baget, 2013). Figure 3.7 illustrates the upstream slope of the tailing dam.

Figure 3. 7 Hisarcik Tailing Dam rock fill slope (JMS,2013)

3.4 Hisarcik Tailing Dam Rehabilitation

The latest stability analysis of the Hisarcik tailing dam, which was performed by Emet Borax mine engineering management, showed significant problems in the stability of the dam body and possible failures were predicted. Also, some engineering designs were launched to estimate the possibility of dam height rising. Therefore, a sort of rehabilitation work should be organized in order to enhance the slope stability.

After some geological and geotechnical investigations, it was decided to design the rehabilitation project due to the material injection. This injection is designed to boost the impermeability of the dam layers. As it is noted, the Hisarcik tailing dam consists of two major structures, which are known as clay fill and rock fill. Based on the geological surveys in the longitudinal section of the dam, it is determined that 15m to 20m injection in the inner depth of clay fill and the transition section among the clay fill and the rock fill is applicable (Baget, 2013). The assumed coordinates and the direction of the injection boreholes are illustrated in Figure 3.8.

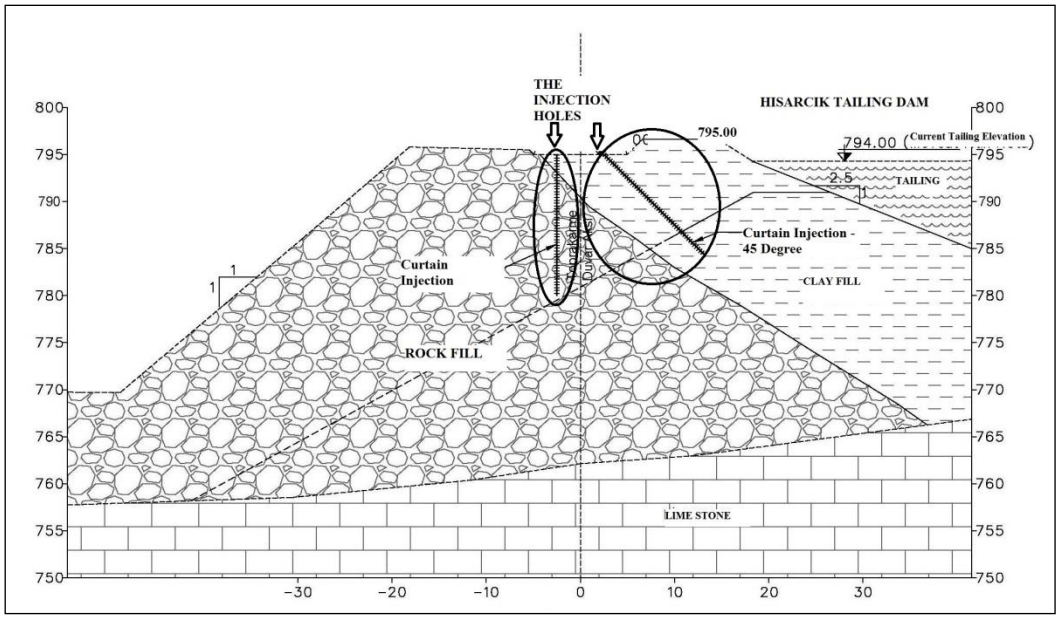

Figure 3. 8 Injection boreholes in dam body (Baget, 2013)

As it is seen in Figure 3.8, two rows of the injection are employed to mitigate the leakage of dam upper levels. The distance between these two rows is considered to be about 5m and these processes have been scheduled in line with the DSI (State Hydraulic Works) specifications and instruction. For this purpose, a group of geological engineers supervised the operation (JMS, 2013). The injection excavation process was initiated by launching six schemes of stripping and shaping the injection, while the first row of it was applied to the clay fill layer by an inclination of 45° from the ground and the second row of injection curtain was applied perpendicularly to the transition zone among the clay fill and the rock fill while this trend starts from the clay fill layer surface and passes the border zone to enter the rock fill layer. The total number of injections is assigned to be about 390 and the length of it is estimated at about 6380m (Baget, 2013). Table 3.2 summarizes the characteristics of the injection and application type.

Table 3. 2 Characteristics of injection and application type (JMS, 2013)

Characteristics	1st and 2nd Curtains Injection
Bore Spacing	5m
Phase Depth	5m
Phase Number	4
Angle	45
Injection Method	Up to bottom
Bore Numbers	195
Bore Depth	Between 15 - 20m
Total Bore Depth	3195 m

The following step right after the injection process should be determining the success of the percentage of injection application. To do this, continuous pressurized water is applied to the zone in several stages to evaluate the leakage amount and if there is any unexpected passing through the area, it will prove an unsuccessful and inaccurate design for injection. The observations have shown that the injection has intensified the impermeability and the results are satisfactory (Baget, 2013).

The material properties for injection mixture are as below:

1. **Cement**: The cement mixture is prepared based on the Turkish standards (TS 10157). Two types of the cements were chosen to be included in the mixture; (i) Sulphate resistant cement and (ii) Normal Portland cement (Baget, 2013). The sulphate resistant cement is in compliance with TS 10157 standard and in such cement the amount of tricalcium aluminate does not exceed 3.5%. Also, it should meet the TS-19 and OP 325 standards (JMS, 2013).
2. **Water**: Water must be clean, fresh and free from organic materials, salts, alkaline, oils, and any kind of waste materials, hazardous or undesirable

materials. When it is needed, water samples must be provided systematically to the laboratory. Sodium sulfate and other ions must be within the limits so that they do not damage the concrete. The strength of the mixture which is made using waste water must not be less than 10% of the mixture, which is made in the same manner with drinking water (JMS, 2013).

3. **Sand**: According to the injection type, and by considering the administration approval and instruction, in the first step, sand grout up to 200% of the weight of cement should be added to the mortar. It consists of stable, durable and tough grains the harmful material of which is less than 5%. Grains are mostly round or cubic shaped and they should be thin or medium-size (JMS, 2013).
4. **Bentonite:** As it is requested by the project supervisor, the bentonite is used in the injection mixture. The bentonite specification are in accordance with the TS 977 standard. The specifications are as below:

- In hydrometric analysis, the remaining quantity passed from 200 sieves must be up to 2.5%.

- In dry sieve analysis, the quantity, which passed from 149 micron space sieve, must be at least 98%.

- The moisture content must be 10% by weight.

- In device for measuring direct viscosity, the viscosity must be 600 rev / min and at least 30 s.

To verify and confirm the bentonite quality, it is assigned to evaluate the experimental results taken from the injected zone right after each bentonite delivery.

CHAPTER 4

HISARCIK TAILING DAM STABILITY ANALYSIS

4.1 Study Description

In order to analyze the stability of the dam, the geotechnical parameters must provide that this information can be gathered from different sampling and evaluation such as geological, seismological, and field investigations and laboratory testing. In this study, the input data were taken from the previous surveys and site investigations and mainly the stability investigation that was done by the research group of JMS and Eti Mine engineering section. The resulting technical report contained information about the material properties, the geometrical data, and dynamic analysis due to the simple modeling method. All the input data are utilized to run the finite element method so as to analyze the dam condition and safety. The present study is an analytical work to promote another analysis in order to revise the inputs and outputs achieved previously. The second step was surveying and revising the reported data and the analysis that were reported by the "ETI MINE GENERAL MANAGEMENT". The required data were obtained from the official reports of ETI Mine. The dam condition and stability under the static and dynamic loading were analyzed by applying the plastic parameters to consider the yielding condition of material and feasible seismic loading. For this purpose, the PHASE 2 v.8.01 modeling software was employed, which is applicable for analysis based on the finite element method. The slope stability evaluation for the upstream and downstream parts of the dam as considered via the same software. Also, the Strength Reduction Factor (SRF) was computed for possible field loading

conditions. Consequently, the obtained results have been discussed and compared to the results of similar studies and the conclusions and recommendations were represented.

4.2 Analysis Method – Finite Element Method

The numerical technique, generally known as the Finite Element Method (FEM), is used to solve the partial differential equations (PDE). Also, it is less often applicable for integral equations. It can be said that it refers to dividing the complicated problems to small elements that can be solved in relation to each other.

Generally, limit equilibrium analysis is the most popular method to represent the current condition of the study case. However, due to the priorities of the FEM like expressing the non-linear behavior of the rock and soil masses, the influence of discontinuities, the progressive failure and the associated stress redistribution also influence the initial tectonic stress system; nowadays, it has become much more applicable. On the other hand, the main assumption of limit equilibrium method is the occurrence of sliding failure through the rock or soil masses along the slip surface (Rocscience Inc., 2004). The privilege of this method is the simplicity of understanding and the ability which can perform the sensitivity analysis for input parameters. Regardless of the utilization of minimum input data, the output safety factors of slope usually satisfy the expected range of deformations. The technique does not consider the stress-strain behavior of rock and soils. It makes arbitrary assumptions to determine the condition (Azadegan, 2012).

In the assessment of slope safety, engineers primarily use the factor of safety values to determine how close or far slopes are from failure (Hemmah et al., 2006). Recently, by progress in technology and the availability of greater memory resources and computation, adding the low cost of method, the finite element method has been recognized as a powerful and reliable alternative. The safety factor of slopes is expressed by the shear strength reduction factor and the method has certain advantages such as predicting stresses and the deformation of support elements like piles, anchors and geotextiles (Dawson et al., 1999). Safety factor indicates the current condition of the structure (in this case, the upstream and downstream slopes) and the accuracy of the

output is related to the accuracy of geotechnical input data. The inflection of the finite element method is the main reason of its being commonly applicable in slope stability analysis due to the elastic-perfectly plastic material strength formulation format as non-linear. Griffiths (1999) offers other privileges such as (1) Elimination of a priori assumptions on the shape and location of failure surfaces, (2) Eliminated assumption regarding the inclinations and locations of inter-slice forces, (3) Ability of the model for investigation of progressive failure, (4) Calculation of deformations at slope stress levels and (5) Capability of performance in a wide range of conditions (Griffiths et al., 1999). In Figure 4.1, the stress analysis structure of a typical earth dam which is analyzed by the finite element method is shown.

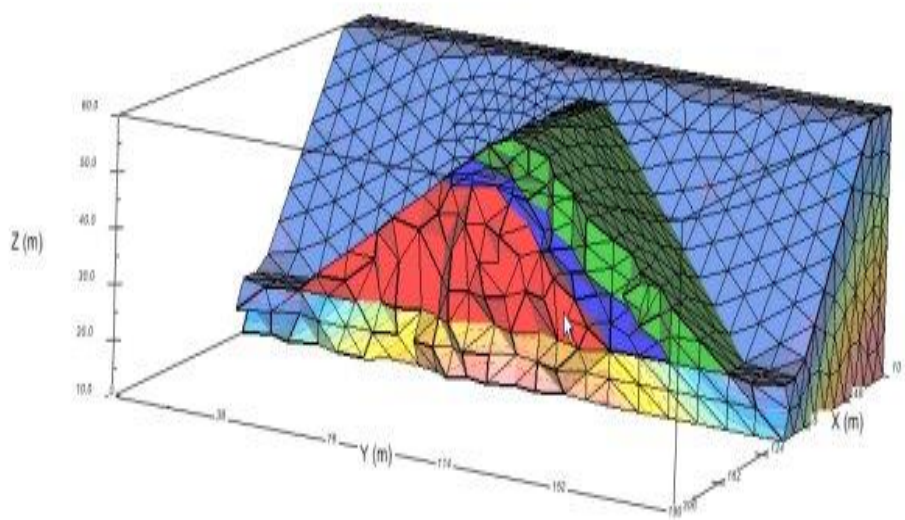

Figure 4. 1 Stress Analysis of Earth Dam Site using Finite Element Method (soilvision.com)

The safety factor in slope stability analysis is defined as: (Duncan, 1996)

$$F = \frac{Shear\ strenght\ of\ material\ (rock\ or\ soil)}{Shear\ strength\ required\ for\ equilibrium} = \frac{t}{t^*} \tag{4.1}$$

The same trend is applicable for the limit equilibrium method.

In general, two sub-methods are applicable to analyze the slope stability: (1) Gravity Increase Method and (2) Strength Method. Below is the representation of differences among these two trends:

(1) Gravity Increase Method: Increase "g" until the slope becomes unstable and equilibrium solutions no longer exist (Hammah, 2004).

- ✓ $g(t) = g_{base} * f(t)$ Where g_{true} is the actual gravitational acceleration. (4.2)
- ✓ $(F.S)_{gi} = \frac{g_{limit}}{g_{true}}$ (4.3)

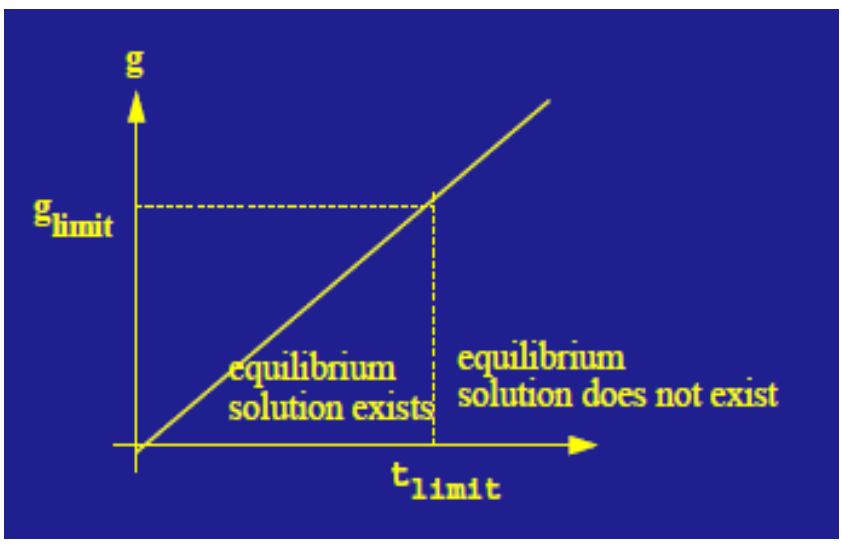

Figure 4. 2 Gravity Increased Method plot (Iowa ASCE Geotech, 1999)

(2) Strength Reduction Method: Decrease the strength parameters of the slope until slope becomes unstable and equilibrium solutions no longer exist (Zeinkiewicz, 1991).

- ✓ Y (t) = $Y_{base} * f(t)$ Where Y_{base} are the actual strength parameters. (4.4)

- ✓ $(F.S)_{sr} = \frac{Y_{base}}{Y_{(t_{limit})}} = \frac{1}{f(t_{limit})}$ (4.5)

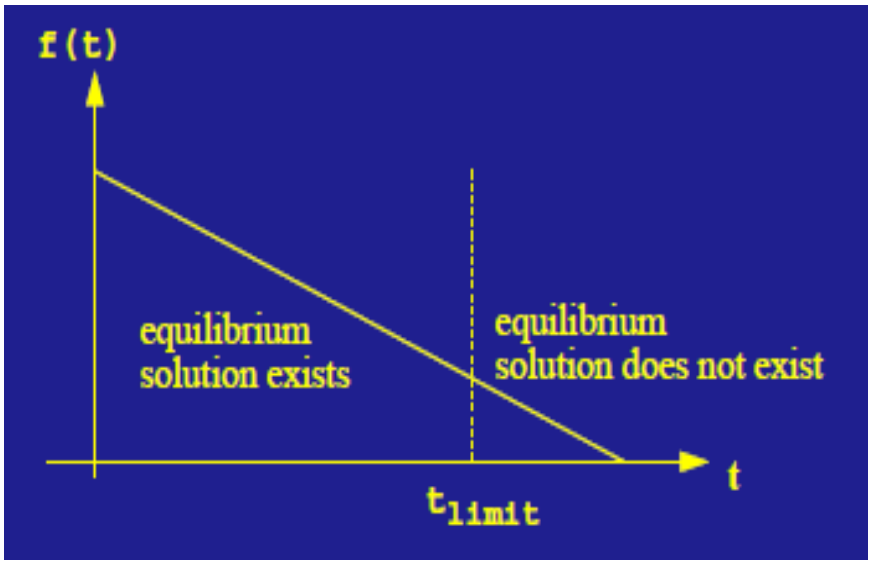

Figure 4. 3 Strength Reduction Method Plot (Iowa ASCE Geotech., 1999)

Nowadays, both methods are applicable for the slope safety factor. The result of the strength reduction method is typically lower than what is obtained in the gravity increase method. For instance, in soil slope, the results could be different and lower for strength reduction while slopes are lower or the easy grade of side slope. Through the analysis of the results, it can be obtained that if the grade of side slope is below 1 or 1.5, the gravity increase method is not recommended for severe increases, and when the slope side is more than 1.5, this method can be calculated. In rock slope, the numerical examples show that the method of finite element calculation results and the gravity increasing method are basically the same (Zeinkiewicz, 1991).

The Shear Strength Reduction (SSR) method provides advantages such as the prediction of stresses and the deformation of support elements at failure, and it generally visualizes the development of failure mechanisms (Hammah et al. 2004). Specifically, in case of several probable failure modes, the SSR method is much more applicable since this method automatically finds the critical failure mechanisms (Hammah et al., 2004). In contradiction with the SSR method, the limit equilibrium method is not affected by certain properties like Young's modulus (E), Possion ratio (ν) and angle of dilation (ψ). Also the post-peak material behavior is one of the outlines which is absent in the limit equilibrium method (Hammah et al., 2004). Based on these differences, it can be perceived that the deformation of the structure is not considered in the limit equilibrium method and this could make the FEM more applicable and precise for cases related to the slope stability (Griffiths, 1999).

In the SSR analysis, instability and non-convergence in the model results indicate the failure condition and this convergence is due to three main factors including (a) stopping criterion, (b) stopping criterion tolerance value and (c) the number of iteration before solution convergence. It should be noted that in this analysis the material type is assumed as an elasto-plastic one and the trend of the method is going to reduce the shear strength parameters with the constant factor until the failure occurs (Rocscience Inc., 2004).

Since the case in this study is related to slope stability of tailing dam, the advantages of the SSR method are taken into consideration. One of the main advantages of this method over the limit equilibrium is the elimination of assumption about failure mechanism due to the type, shape and probable surface locations while it evaluates the critical mechanism automatically. Besides, the assumption about inter-slice forces is omitted by the SSR (Abolfazl .S et al 2007). Moreover, this method can correct the deformation properties of materials at different stress levels through the slope. Another point that could be noticed about the FEM is the modeling of the construction sequences like embankments which consists of several stages and also this method is applicable for complex slope configurations and soil deposits (Rocscience Inc., 2004).

Consequently, the SSR is the best fit for 2D and 3D modeling of slopes and seepage induced instabilities as it is shown in Figure 4.4 as a sample model.

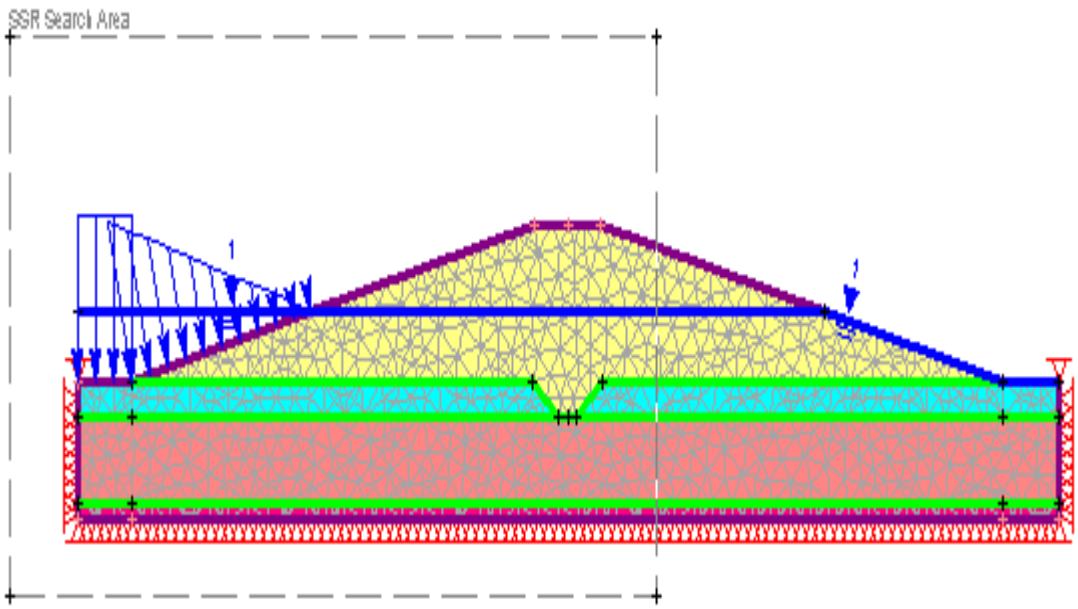

Figure 4. 4 Example of SSR method analyses in case of dam stability (Rocscience Inc., 2004)

4.3 Hisarcik Tailing Dam Embankment Characteristics

The stability of dam, foundation and fill materials in term of engineering properties are evaluated by taking into account the prevailing geological conditions and also the determination of engineering properties of the fill material. There would be some important criteria for this determination of the proposed backfill material, the selection of adequate materials and supplies and changing cases (USBR, 1987c). The climate conditions of the region and inevitable changes in the condition of water content and also compaction density are effective factors which are included in the stability analysis. Besides, the fine-grained filler material under high lateral pressure and the reduction of internal friction angle of the soil mass are especially important for high embankment dams which should be taken in account (USBR, 1991).

For the stability analysis of the Hisarcik tailing dam, there are three different materials that form the core zone and the foundation of tailing dam, which involves foundation,

rock fill and clay fill (Baget, 2013). The foundation is defined as limestone. During the planning and design of the dam, the material properties have been allocated according to the geotechnical studies based on the laboratory (Tosun, 2011).

JMS (2013) has performed a sort of borehole drilling to define the detailed characteristics of dam for the current condition. According to a total of 13 disturbed and 9 undisturbed samples of the tests, clay is used as a feature to the dam material. The ground is covered by thin parts including a mixture of low plasticity clay (CL) and high plasticity clay (CH). The mixture of low cohesion grain size material and silt has made the foundation upper side and limestone is the main structure for the foundation. For the raised dam the same qualification of clay material is used (JMS, 2013).

During the operational phase, usually the clay core effective stress average envelope is obtained by applying loading with the corresponding value. In the sudden inset, the lowest envelope parameters which are transformed by effective base values are taken into consideration (EM 1110, 1970). The analysis in both cases considers the pore water pressure during the installation which exists in the clay fill, which is taken into account by a coefficient of ($r_u = 0.30$) (EM1110, 1952).

As mentioned before, the achievements of the drilling in Hisarcik tailing dam body, which were performed for the crest, upstream and downstream slopes, represent the geological cross-section that illustrates the upstream impervious clay core and the downstream rock fill as the main materials for construction properties (JMS, 2013). According to these studies related to the dam body, which can be seen in Figure 4.5, the characteristics of the site are:

Impervious clay core thickness: 15.0 m

Upstream slope steep of the dam body: 1 / 2.5 = almost 22°

Downstream slope steep of the dam body: 1/1 = 45°

Dam body height: 34.0 m

Topographic slope of the dam lake area: 4-5 degrees

Figure 4. 5 Hisarcik Tailing Dam crest and upstream (JMS, 2013)

The strength properties of upstream layer are based on the undisturbed samples analysis, which is taken from the drilling strength test. The ranges of the engineering parameters of the clay-fill are given below (JMS, 2013):

Unit weight: 18.04 to 18.32 kN/m^3

Triaxial compression test (UI): 58.84 to 71.60 kPa

Internal friction angle (Ø): 6°-10°

As it can be seen, the strength values are obtained from undrained and undisturbed samples. Also, another set of data related to the undisturbed consolidation test samples is obtained in which the ranges of parameters are given as below:

Swelling percentage (%): 1.02 to 1.51

Inflation pressure from 10.78 to 16.18 kPa

By considering these drilling samples and results, the geotechnical properties of the body materials of the Hisarcik tailing dam are reported as in Table 4.1 (JMS, 2013).

Table 4. 1 The material properties of Hisarcik tailing dam body (JMS, 2013)

Unit	Unit Weight (kN/m^3)	Poisson Ratio	Young's (Elastic) Modulus (MPa)	Internal Friction Angle (^{0})	Cohesion (kPa)
Clay Fill (Undrained)	18.28	0.10	30	6	63.74
Rock Fill	21	0.25	120	45	0
LimeStone	25	0.30	500	Uniaxial Compressive Strength: 29420kPa m_b: 1.6 s: 0.003	

As it is illustrated in Table 4.1, the information about the geo-mechanical properties of the layers of the dam body can be realistic but for a greater precision, similar literature references should be checked.

Since in this study the long-term behavior of dam is taken into consideration, there should be some information about the drained tests results for the upstream clay fill material, while no drained test is performed by the geological lab section of the JMS Company. Hence, the literature data related to similar cases are studied to evaluate the material property as an input data for the modeling.

The drained analysis is applicable while the rate of loading is low and permeability is high and also it should be mentioned that the drained analysis may be carried out by utilizing a constitutive model based on effective stresses in which the material model is specified in terms of drained parameters (Potts, 2001). The shear strength test includes two major types as (i) the direct shear strength test and (ii) the triaxial shear strength test. The vane shear test and the unconfined compression test are other types of shear

tests that are applicable to specific cases. The two parameters that can be obtained from these tests are internal friction angle (ϕ) and cohesion (c). The tests results are plotted on a graph with the peak (or residual) stress on the x-axis and the confining stress on the y-axis. The y-intercept of the curve which fits in the test results is the cohesion, and the slope of the line or curve is the friction angle. As a general statement, the internal friction angle is the maximum angle to which the specimen could resist the force, which stimulates it to slide. Cohesion can be explained as an attraction force which exists between the minerals. These two parameters are the most effective items on the factor of safety evaluation in investigations about slope stability analysis.

According to the definitions about shear strength properties such as internal friction and cohesion, the aforementioned literature survey is carried out to find the proper values of strength parameters and also to compare the undrained and drained tests results. These layouts are gathered in Tables 4.2 and 4.3 about the drained and undrained material properties, respectively.

Table 4. 2 Shear Strength parameter for drained samples

No	REFERENCE	MATERIAL	E (Mpa)	ϕ (°)	C (kPa)	DESCRIPTION
D1	Sonmez et al., (1998)	CLAY	_	21	9	comapct Marl+soft clay
		ROCK	_	_	_	_
D2	Oral (2010)	CLAY	30	20	5.9	_
		ROCK	_	_	_	_
D3	Billiton (2009)	CLAY	35	20	5	Clayey liner
		ROCK	120	40	0	Crushed Rock (also 37°)
D4	Agency (August 1994)	CLAY	25	20	5	Silty Clay
		ROCK	80	35	0	_
D5	Stanciucu, M., (2005)	CLAY	27	13	8	Pliocene clays
		ROCK	40	18	5	Alluvial Rock
D6	Ormann (2011)	CLAY	_	_	_	_
		ROCK	40	42	1	Rockfill
D7	Zardari (2011)	CLAY	20	22	1	_
		ROCK	40	42	1	_
D8	Paul (2000)	CLAY	54	26	5	sandy clay
		ROCK	158	42	3	Shell

Table 4. 3 Shear Strength parameter for undrained samples

No	REFERENCE	MATERIAL	E (Mp)	ϕ (°)	C (kPa)	DESCRIPTION
UD1	Fell (2005)	CLAY	_	7	85	Reinforced clay
		ROCK	_	55	0	igneous rock
UD2	Liang (1998)	CLAY	_	0	49.69	Medium gritty blue clay
		ROCK	_	45	0	Free drainage rock fill
UD3	Liang (1998)	CLAY	_	0	50.77	Stiff gritty blue clay
		ROCK	_	42.3	0	Rockfill
UD4	Dalvand (2010)	CLAY	46	12	50	silicate clay
		ROCK	250	35	2.6	Rock Mass
UD5	Ghafoori (2011)	CLAY	50	12	56	Undrained Clay
		ROCK	250	54.7	7.3	_
UD6	Breitenbach (2009)	CLAY	25	10	62	mixed clay (+loam)
		ROCK	100	25	0	_
UD7	SPENCER (1967)	CLAY	22	16	61.66	_
		ROCK	_	_	_	_
UD8	ORAL (2010).	CLAY	30	0	75	Sandy clay
		ROCK	65	42	0.1	_
UD9	Abolhassani (1988)	CLAY	_	2	70	silty clay
		ROCK	_	42	0	travertine
UD10	Mihai (2008)	CLAY	40	17	65	_
		ROCK	100	45	0	_
UD11	Kutzner (1997)	CLAY	25	1	58	cohesive plastic soil
		ROCK	100	40	3	Crushed Rock(igneous and sedimentary)

As it can be seen in Tables 4.2 to 4.3, according to the similar analysis types, there are some differences among the drained and undrained parameters and the average of the given data for each parameter could represent the reliable input for analysis. By using the data given in 4.2 and 4.3, summary statistics for shear strength parameters are obtained (Table 4.4 - 4.8). The first factor which is considerable about the rock fill is internal friction angle of it and again the average of the given parameters could be acceptable for it. In Table 4.4, the plot represents the tolerance of rock fill friction angle in samples and the average amount is 45°. Hence, the reported value for the rock fill friction angle could be employed in the analysis.

Table 4. 4 Summary statistics of internal friction angle for rock fill samples obtained from the literature

Material	Parameter	Mean	Standard Deviation	Range	Min	Max	Coefficient of Variation
Rock Fill	Internal Friction Angle (°)	45.4	6.35	18.8	40	58.8	0.13

It should be noted that “D” is a symbol for drained samples and “UD” is for undrained ones. Regarding the clay fill material properties, both undrained and drained parameters are considered. Since the input parameters are assumed as drained ones in this study, the undrained outline of the literature survey is just for the evaluation of the reported values. The first item that seems to be not realistic in the values reported by the JMS is the Poisson ratio of undrained clay, which in most cases is assumed to be about 0.45. On the other hand, the validity of the shear strength parameters can be represented as averages of the similar samples. The obtained amounts for undrained cohesion of clay are shown in plot that is illustrated in Table 4.5, which takes the amount of 62 kPa as an average. Based on this average, the 63.74 kPa is evaluated as a feasible value for the undrained cohesion for clay fill.

Table 4. 5 Summary statistics of clay cohesion for undrained samples, obtained from the literatures

Material	Parameter	Mean	Standard Deviation	Range	Min	Max	Coefficient of Variation
Undrained Clay	Cohesion (kPa)	61.85	11.48	35	50	85	0.18

The internal friction angles of the undrained clay are generally expected to be between 0° and 10° and of course, these kinds of values are obtained from the common data released by the empirical and theoretical experiences. Furthermore, in the combination of clay material with some others, this measured amount could be different as it is indicated in Figure 4.8, which is the plot for various ranges of undrained friction angle for clay.

Table 4. 6 Summary statistics of clay internal friction angle for undrained samples, obtained from the literatures

Material	Parameter	Mean	Standard Deviation	Range	Min	Max	Coefficient of Variation
Undrained Clay	Internal Friction Angle (°)	7	6.69	17	0	17	0.19

The amount of 7° for the internal friction angle in case of clay material due to the assumed range, which is 0°<$\varphi_u < 10°$, could be acceptable. Therefore, compared to the range of JMS reported values which are shown in Table 4.1 for this parameter, it can be perceived that the obtained range is plausible to be employed as an undrained friction angle.

Besides the proposed undrained parameters for shear strength properties, in this study, it is assigned to get the drained values as input data. Thus, as it is mentioned above, there are no drained test results in the reports. That's why, the current investigation input parameters are based on the revision of the similar projects related to the clay core rock fill dams. The predictions about shear strength parameters in case of drained condition are gained from empirical and theoretical sources as well.

The assumption about the drained cohesion of the clay represents the low range while there is no mixing material. It is believed that the drained cohesion of clay could be counted as a value close to zero but since there is always a tangible impurity percentage

in clay fill material, this value could be increased. Table 4.7 represents different values of drained clay cohesion.

Table 4. 7 Summary statistics of clay cohesion for drained samples, obtained from the literatures

Material	Parameter	Mean	Standard Deviation	Range	Min	Max	Coefficient of Variation
Drained Clay	Cohesion (kPa)	5.15	2.55	8	1	9	0.19

As it is shown in Table 4.7, the average amount for drained cohesion in case of clay material is considered to be about 5 kPa. It is assigned to take the drained cohesion range between 0 kPa and 5 kPa for this study to find the optimum parameter which causes the convergence in results. According to the other parameters and the plastic behavior of clay that is based on the JMS outlines which represent low plasticity in many samples (PI<5), it is assumed to take the 5 kPa as the input parameter for the dynamic analysis of the dam.

Similarly, for the drained internal friction angle of clay, the range between 15° and 20° is defined and the samples from the literature are summerized in Table 4.8 to make a decision about the proper values of drained friction angle that can be used as an input data in dam modeling.

Table 4. 8 Summary statistics of clay internal friction angle for drained samples, obtained from literatures

Material	Parameter	Mean	Standard Deviation	Range	Min	Max	Coefficient of Variation
Drained Clay	Internal Friction Angle (°)	19.37	3.15	10	13	23	0.16

The result of the different sources about the same practice indicates the amount of 19° for the drained friction angle in case of clay type material. Since the assumed range about this parameter satisfies the average, 19° could be employed for the analysis.

Up to here, the material properties of the dam body are defined and can be applied to the analysis, yet there is also another point to be in investigated. While the analysis is going to evaluate the tailing dam characteristics, it should include the tailing specifications for a more realistic interpretation. Since there is no detailed information about the tailing properties and the type related to the Hisarcik impoundment, this research relies on the reported parameters due to the sample tests which is carried out by Tosun's team in 2011. On this basis, the geo-mechanical properties of tailing material are as in Table 4.9.

Table 4. 9 Hisarcik dam tailing properties (Tosun, 2013)

Material	Unit Weight (kN/m3)	Poisson Ratio	Young's Modulus (MPa)	Internal Friction Angle (°)	Cohesion (kPa)
TAILING	14	0.3	20	20	0

As it can be perceived from the tailing properties, it contains of fine, homogeneous and plastic soil like material. The evaluation of the tailings material properties is done according to the literature survey given in Table B1 and B2.

By considering Table 4.1, which contains the JMS reported data about the geotechnical properties of Hisarcik tailing dam body and also the obtained parameters from the literature which are summerized in Tables 4.4 to 4.8 and added the information in Table 4.4, the applicable parameters for analysis can be verified. Accordingly, the material properties which are applicable in the stability analysis of the Hisarcik tailing dam are represented in Table 4.10.

Table 4. 10 Hisarcik dam and impoundment material properties

Material	Unit Weight (kN/m3)	Poisson Ratio	Young's Modulus (MPa)	Internal Friction Angle (°)	Cohesion (kPa)
LIMESTONE	25	0.3	500	uniaxial compressive strength: 29420 kPa m_b =1.6, s=0.003	
ROCK FILL	21	0.25	120	45	0
TAILING	14	0.3	20	20	0
Drained CLAY FILL	18.28	0.1	30	19	5

As it can be seen in Table 4.10, most of the input data in terms of strength parameters are assumed to satisfy the Mohr-Coloumb failure criterion except limestone, which were prepared to be applied for the Hoek-Brown failure criterion. Hoek-Brown failure criterion has been found to work well for most rocks of decent to logical quality, in which the rock mass strength is controlled by tightly interlocking angular rock pieces, and in case of limestone, this criterion could produce more accurate results.

4.4 Seismic Analysis in Study Area

The fundamental aspects of the dynamic analysis of tailing dams depend on the geological and tectonic investigation in the dam site and its vicinity. These affect the primary factors of the seismic design of the dam. These factors include safe basement, local conditions, the size of the dam and the function of structure which is important in dam damages and any probable failures (Vick, 1983).

The selection of parameters for the evaluation of the dam safety due to the seismic activities are referred to as the specific requirements and provides a minimum level of tasks which are carried out in a series of stages. The geology and seismicity analysis that are typically performed for the embankments should be considered on a regional scale and the seismic history of the formation in the certain area should be studied to recognize the geological characteristics. In some sections of dam sites, the analysis significantly explains all the geological formations by considering the specific condition which may require the evaluation of large regional studies (Ulusay et al., 2004).

According to the experts' vision, the regional geological studies should cover an area of at least 100 km in diameter (ICOLD, 1989). In this case, there is a major fault in distance of 300 km, which is not considered in the study area, and it could have effects on stability (Tosun, 2011). Since the site area includes a region with a high degree of seismic activity, it seems that the fault should be considered in the research (Tosun, 2011).

ICOLD (1989) highlighted that for these kinds of studies, the dam site identification of physico-geographic and tectonic environment, identifying the geological history of dam area (e.g. geological formations, rock types and definition of the deposits floor, folds, active cracks or joints structural formation, etc.), regional tectonic mechanism, the interpretation of faults and fault type or types (the location and description of faults and shear zones, faults, faults capacity in case of earthquake generation or the fault displacement generated by earthquake) and a scope about the related values of fault activity (average shear rate, the time interval between large earthquakes, etc.) are required.

In high seismicity regions, the effects of earthquake motion should be estimated and it is an important engineering problem. The strong ground motion may cause large inertia forces (ICOLD, 1989). Analyzing the stresses generated by the dam deformation and the determination of the deformation results is quiet difficult and complex. For this purpose, a large number of analyses that are assumed to be more easily accessible have done, most importantly, the plain strain simulation (Tosun, 2011).

In concrete dams, additional lateral forces and moments under the influence of excessive cracking are expected to occur in the body and in this situation the overall stability is evaluated. In order to obtain the initial information about dam material parameter in case of taking the seismic effects into account, in preliminary design, it must be considered that the selected material must supply the optimum strength and avoid the risks associated with a hazard ratio to be low enough (USCOLD, 1996).

Earthquake effects are added to the analysis method in order to determine the behavior of fill dam in term of seismic activity. These pseudo-static analyses include the shear beam method to form the experimental model by the finite element method. The pseudo-static analysis method, which is proposed for the dynamic analysis of embankments, has been used for more than 50 years (Tosun, 2011). It is considered as a detailed method, a precise solution and also an effective method to learn about the mechanism of the effects of earthquakes on the center of dam (Makdisi, 1987).

The dam area directions are surrounded by an active fault which has enlonged from south to south-west. This fault has caused some earthquakes. According to the data, there are two recorded instances with magnitudes of 6.5 in 110 years (1880-1990).

Observing the existing seismological maps of the dam location has represented that the dam is located at the first degree region and the set of seismic analysis has been performed (Tosun, 2011). This map was prepared by the Ministry of Public Works and Settlement by considering the latest knowledge and information. The prepared map was approved by the Government of Turkey in 1996 (Tosun, 2011). The endangered zones are determined by using the peak ground acceleration contour map, which has been calculated with the probabilistic method. It is assumed that a normal construction, which has 50 years of economic life, may not be damaged by an earthquake larger than the expected maximum acceleration values with a 90% of probability. For important constructions or buildings that have longer economical lives, the maximum acceleration values should be calculated (Tavenas, 1979). As it is noted, according to the information which is released by the Turkish Ministry of public works and Settlements, the tailing dam region is ranked as a 1st degree in terms of seismic activity zone in Figure 4. 11.

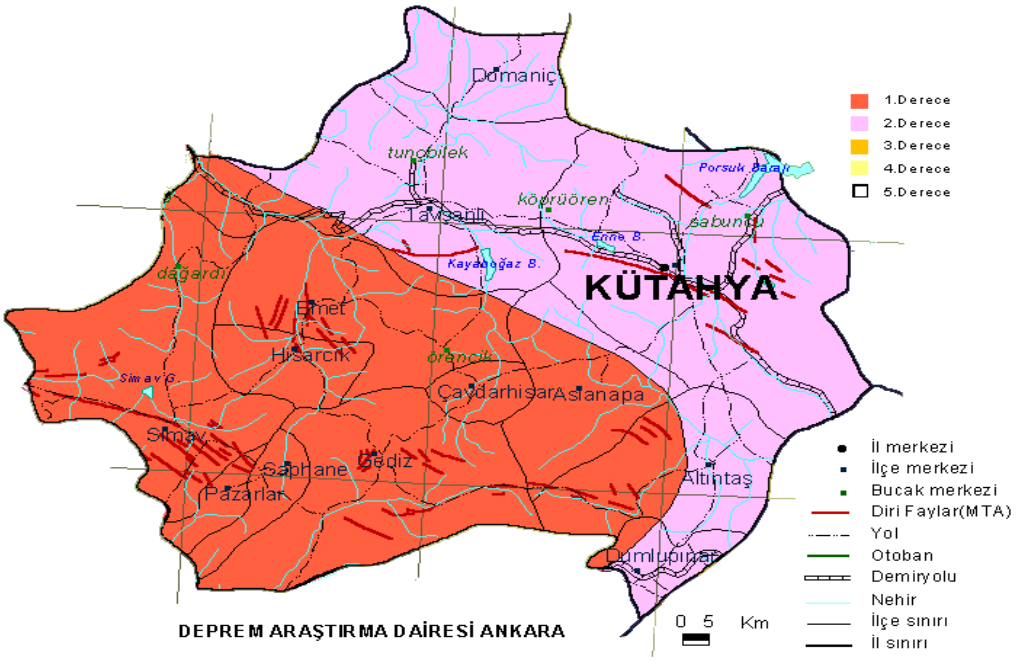

Figure 4. 6 Turkey earthquake map around the project location (www.deprem.gov.tr)

The earthquake zones of Turkey are classified as follows according to the expected acceleration values:

1st degree earthquake zone	:	more than 0.4g
2nd degree earthquake zone	:	between 0.3g - 0.4g
3rd degree earthquake zone	:	between 0.2g - 0.3g
4th degree earthquake zone	:	between 0.2g - 0.1g
5th degree earthquake zone	:	less than 0.1g

g: gravity (981 cm/s*s)

This map was prepared by using the report named "A Seismic Zones Map of Turkey Derived from Recent Data", which was prepared by Polat Gülkan, Ali Koçyiğit, M.Semih Yücemen, Vedat Doyuran and Nesrin Başöz (METU Civil Engineering Dept. Earthquake Engineering Research Center) and presented to the Ministry of Public Worka and Settlement, General Directorate of Disaster Affairs as a final report of the project 92-03-03-18 in January 1993 (Tosun, 2011).

By concentration about the seismic activities in the western Anatolia region, Kütahya province is one of the most active locations and according to the “annotated catalogue of earthquakes” which was published in 1952, Emet is located among three major earthquake zones in western Anatolia: (1) Sındırgı-Simav-Gediz depression zone, (2) Emet depression zone and (3) Kütahya. To discuss about seismic hazard analysis in term of filled dam stability analysis, the seepage and foundation analysis and the dynamic analysis of the current situation are needed. The earthquakes which occurred in in the history entail the energy accumulation in the crust of the earth produced by sudden movements (Tosun, 2011). Besides, the most important natural disasters in the region are caused by earthquakes and the vibrations from these movements give birth to the displacements in structure (USCOLD, 1996). For this reason, the probability for earthquakes and the behavior of structure under the seismic load takes into account the optimization of the dam foundation design, which is an important part to know about the treating in case of any seismic activity. For this purpose the seismic analysis for the dam site is the first stage of the stability analysis.

For the dam safety in case of earthquakes, the realization of the significant seismic parameters related to the structure design is an obligation. Dam failures, either directly

to the body or foundation, occur generally because of the fault movements. It will give the choices for selecting the parameters in terms of structured safety (USBR, 1999).

The location of the dam is defined to assess seismic hazard analysis and all potential seismic resources and also to estimate the potential impact of each resource, all of which are considered. Then seismic studies with different principles of deterministic seismic hazard analysis and statistical studies are implemented. There is another map which is illustrated in Figure 4.12, which shows the quantity and quality of earthquakes with magnitudes that are higher than 4 (M>=4) which occurred after 1900 and by focusing on Kütahya provience and its vicinity in western Anatolia. It is illustrated that the distributions of earthquakes with magnitude M=> 4 and M=<5 related to the project area are bigger than the others. Moreover, four seismic activities for magnitude of more than 5 and less than 6 (5<=M =< 6) are registered.

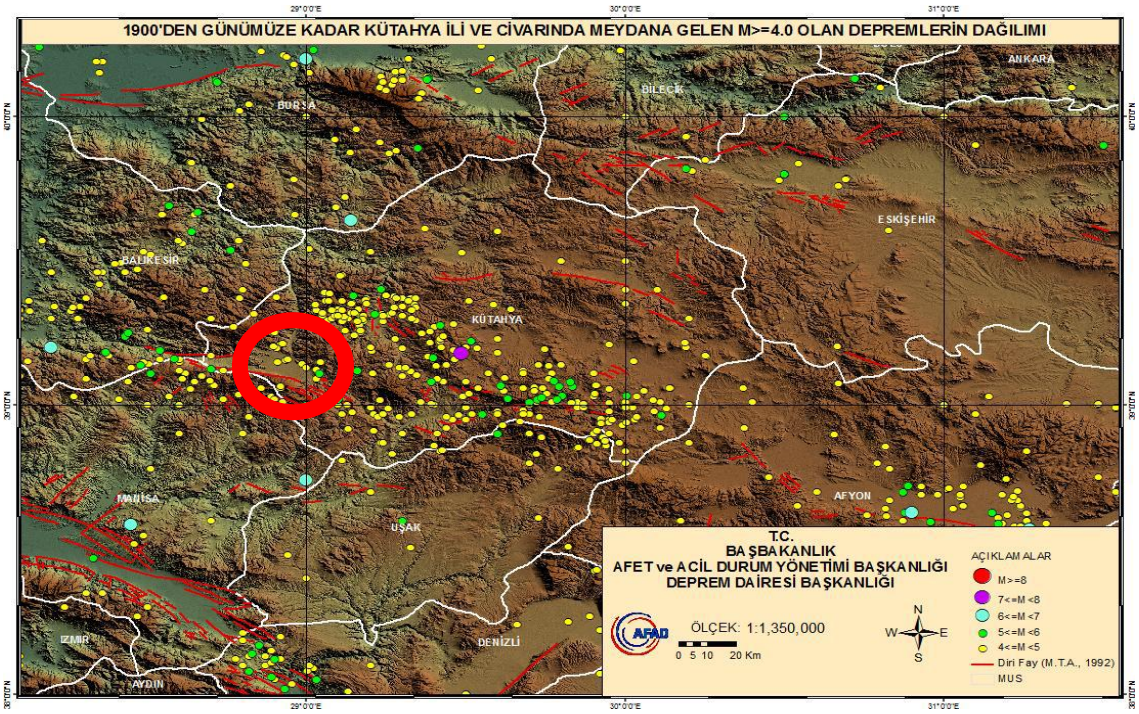

Figure 4. 7 Map of Turkey earthquake with M>=4 from 1900 until now (JMS, 2013)

The seismic activities map of the dam region shows the importance of seismic analysis of any structure inside the zone; accordingly, the Hisarcik tailing dam stability analysis has become more considerable as well. The ground categories are the main factor that

should be defined to figure out the effects of any activity on the structure and for this purpose, Table 4.11 is prepared.

Table 4. 11 Ground Categories (based on the regulations defined by the Turkish Ministry of Social Security and Settlement for structures in disasters area, 1999) (Baget, 2013)

Ground Categories	Ground Categories Description	Standard Penetration (N/30)	Relative Stiffness (%)	Free Pressure Resistance (kPa)	Shear Wave Velocity (m/s)
(A)	Massive volcanic rocks and undifferentiated metamorphic rocks, hard cemented sedimentary rocks Very dense sand and gravel..... Hard clay and silty clay.....	- >50 >32	- 85-100 -	>1000 - >400	>1000 >700 >700
(B)	Loose volcanic rocks such as tuff and agglomerate, discontinuity planes weathered cemented Sedimentary rocks..... Tight sand, gravel..... Very stiff clay and silty clay...	- 30-50 16-32	- 65-85 -	500-1000 - 200-400	700-1000 400-700 300-700
(C)	Soft discontinuities planes decomposed metamorphic and cemented sedimentary rocks Rocks..... Medium dense sand and gravel..... Solid clay and silty clay.....	- 10-30 8-16	- 35-65 -	<500 - 100-200	400-700 200-400 200-300
(D)	The groundwater level is high, soft, thick alluvial layers ... Loose sand ..... Soft clay, silty clay	- <10 <8	- <35 -	- - <100	<200 <200 <200

By considering the ground formation categories and the characteristics of the material which form the ground, the classification of local ground formations should be carried out as Table 4.7.

Table 4. 12 Local ground classes (based on the regulations defined by the Turkish Ministry of Social Security and Settlement for structures in disasters area, 1999) (JMS, 2013)

Local Ground Classes	Ground Categories according to Table 4-6 and Top Floor Layer Thickness (h_1)
Z1	(A) Ground Category $h_1 \leq 15$ m. of the (B) ground category
Z2	$h_1 > 15$ m. of the (B) ground category $h_1 \leq 15$ m. of the (C) ground category
Z3	15 m. $< h_1 \leq 50$ m. of the (C) ground category $h_1 \leq 10$ m. of the (D) ground category
Z4	$h_1 > 50$ m. of the (C) ground category $h_1 > 10$ m. of the (D) ground category

The classification of the ground helps defining the corner periods characteristics ($T_A and\ T_B$). There is also a reference source for adjusting the ground classes to proper values for the spectrum characteristics of a period, which is shown in Table 4.8.

Table 4. 13 Corner periods characteristics (based on the regulations defined by the Turkish Ministry of Social Security and Settlement for structures in disasters area, 1999) (JMS, 2013)

Local Ground Classification according to the Table 4.7	T_A (S)	T_B (S)
Z_1	0.1	0.3
Z_2	0.15	0.4
Z_3	0.15	0.6

The last step for denoting the acceptable ground acceleration to be employed for tailing dam analysis is to fit the obtained data and interpret them. For this purpose, Table 4.14 is prepared to gather the information until now and propose the suitable effective ground acceleration in the Hisarcik zone.

Table 4. 14 Hisarcik area ground categories and seismic characteristics (Baget, 2013)

Project Site Materials	**Ground Category**	**Local Ground Classes**	**Corner periods characteristics (s)**		**Effective Ground Acceleration Coefficient**
			T_A	**T_B**	
Impermeable clay Filling Material	B_3	Z_2	0.15	0.40	0.40
Limestone Filler	B_1	Z_2	0.15	0.40	0.40
Kocaçay Formation	A_1	Z_1	0.10	0.30	0.40

As it is mentioned in Table 4.14, the assumed value for effective ground acceleration coefficient is 0.4, which should be applied to the all layers.

However, the studies about the Hisarcik tailing dam consist of ground motion observations based on the characteristics of tailing dams. The results contain appropriated seismic parameters and data about impoundment location, the size of the earthquakes, the largest earthquake ground acceleration and estimated values. Hence, the study could go forward to the analysis and results.

CHAPTER 5

MODELING AND RESULTS

5.1 General Description

This study involves the stability analysis of the Hisarcik tailing dam. The impacts of seismic activities on the dam stability are investigated and the results show the stability situation of the dam in the current condition and the raised height of the dam. The middle section of the dam has been analyzed by use of the finite element method using PHASE 2 v.8.01 software and the dam model has been divided into different layers according to the material properties and defined geometry. As a result, the stability analysis of the dam is carried out.

There are several notations about the minimum expected safety factors for tailing dams. For instance, the Swedish safety guidelines document GruvRIDAS (2007) suggests a minimum safety of 1.5 for the stability of tailing dam at the end of the construction and during normal operation condition. Besides, U.S Army Corps. of Engineers (2003) recommends the same SF value for the downstream slope while the long term conditions is desired.

As it is shown in Table 5.1, four different analysis conditions such as the end of the construction (including staged construction), long-term (steady seepage, maximum storage pool, spillway crest or top of gates), maximum surcharge pool and rapid

drawdown has been assumed. For each case, a different required minimum safety factor is proposed.

Table 5. 1 Minimum SF, new earth and rock-fill dams (U.S Army Corps. Of Engineers, 2003)

Analysis Condition	Required Min SF	Slope
End of Construction (Including staged construction)	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool	1.4	Downstream
Rapid drawdown	1.1-1.3	Upstream and Downstream

Another reference for the minimum safety factor is ANCOLD (1997), which is based on the loading conditions. For all the steady state seepage, earthquake and construction, there are minimum required factors of safety, which are represented in Table 5.2.

Table 5. 2 Recommended Factors of Safety (ANCOLD, 1999)

Factor Of Safety	Significance
Less than 1.0	Unsafe
1.0 - 1.2	Questionable
1.3 - 1.4	Satisfactory for cuts, fills, questionable for dams
1.5 - 1.75	Safe for dams

5.2 Dam Model Characteristics

There are some examples of 2D and 3D modeling software for the simulation of the environment and they are applicable to different types of structures. Also, different methods are employed for this analysis. Phase2 v.8.01 is well-known and powerful software in term of elasto-plastic finite element stress analysis. It can be applied to any kind of underground and surface excavations or embankments. Also, it can be used for a different range of engineering problems including slope stability analysis. In this study, one tailing dam is assigned to be modelled for the stability analysis via the finite element method and the corresponding software (Phase2 v8.01). The main factor which affects the accuracy of modeling is the precision of the geometry while the main purpose of the analysis is the stability of upstream and downstream slopes. Another point which is equally important about the model is the mesh density, which plays the main role in the feasibility of layouts in case of multi-stages dams.

Mesh density in finite element modeling is an important aspect because it has major effects on the accuracy. Usually the numbers of elements can be determined according to the topological considerations (Rocscience Inc., 2004). In the past, it was not common to analyze each particle of the models to investigate the behavior of different parts separately. Nowadays, however, it is possible to employ high-speed computers and represent all the main components of the finite element model. The best fit for the mesh density is the optimum condition to satisfy the topological characteristics. The desired mesh density depends on several factors such as stress gradients, loading type, boundary conditions, the element shape and type and the required degree of accuracy (Rocscience Inc., 2004). In most models, the results do not considerably change after the constant amount of mesh density. In these cases, the errors of the layout play the main role in selecting the appropriate mesh density. Based on several analysis and experiences which are carried out via PHASE2 v.8.01, an error of 4% to 10% is acceptable for the accuracy of analysis. In addition, the result of the modeling is going to be interpreted in real project estimations (Rocscience Inc,, 2004).

In this study, the first and the most important aspect is the determination of the suitable geometry for the tailing dam, which could adjust the realistic dimensions. To complete this task, the design is assumed to be carried out based on the reported geometry of the Hisarcik tailing dam and the prepared maps. One of the maps that can be referred to is illustrated in Figure 5.1:

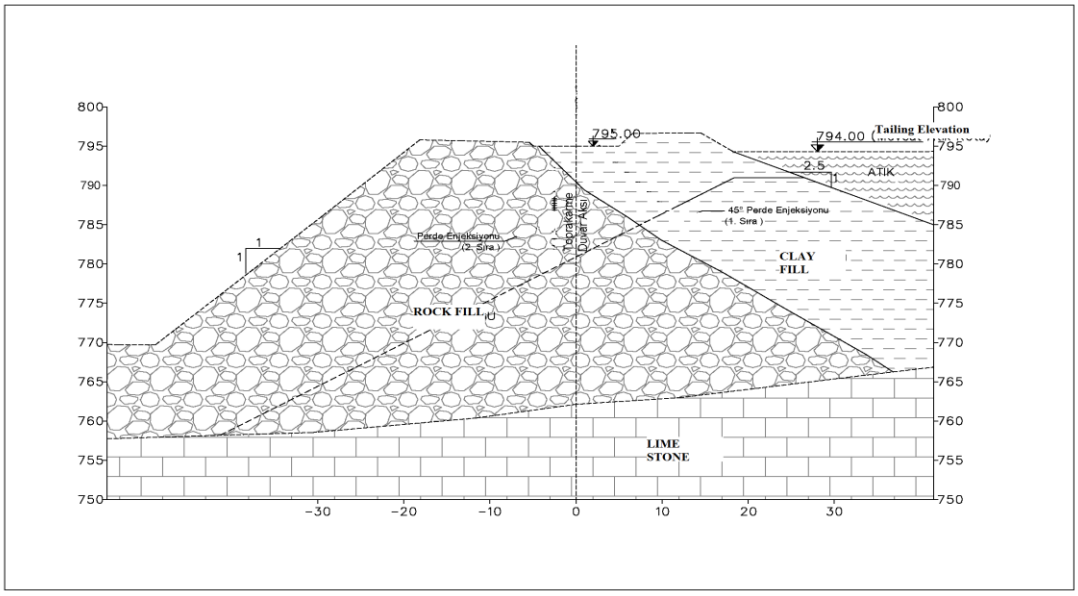

Figure 5. 1 Hisarcik tailing dam map (JMS, 2013)

In this map, the dam upstream slope has an inclination of about 45° and the downstream slope is approximately 22°. Also, the elevation of each coordinate in the dam body is indicated. According to these data, the dam height is assumed to be about 35m. Hence, the representation of the dam model could be assigned as proposed in Figure 5.2.

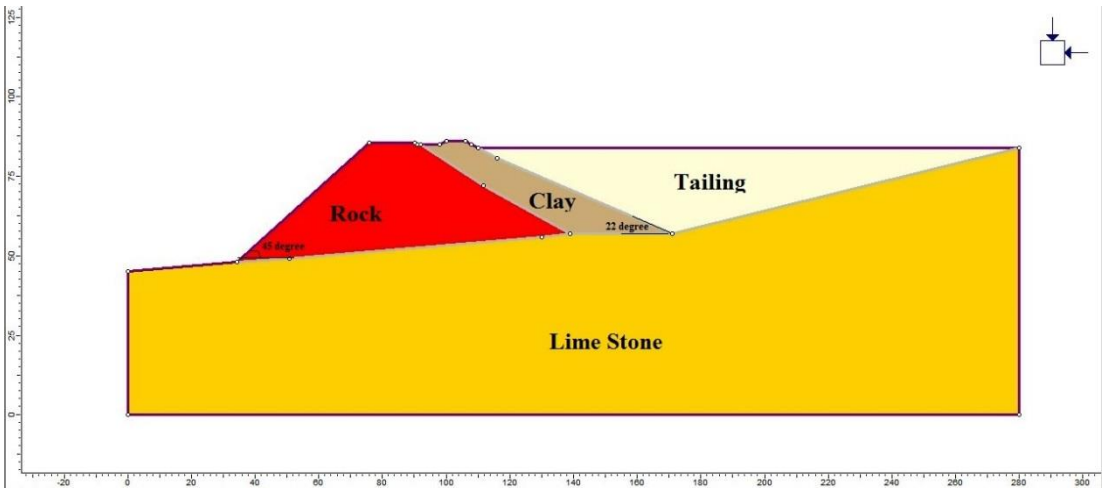

Figure 5. 2 Dam model characteristics

The other specifications that should be considered in the model are the boundary condition and the mesh discretization. Based on the research and literature review in similar cases (Rocscience Inc., 2004), the boundary condition of the dam should be determined such that it restrains in both vertical and horizontal (X and Y) directions. Besides, several mesh discretization samples are applied to the model in order to obtain the optimal mesh density by considering the model characterization and topological aspects. All the experienced mesh densities are determined according to the number of the nodes on external boundaries. Table 5.3 contains all the changes respecting to the change in displacement amount.

Table 5. 3 The different mesh density for Hisarcik dam model

	CLAY	ROCK	TAILING	LIME STONE	DESCRIPTION	CLAY DISPLACEMENTS	
MODEL Number	NUMBER OF NODES ON EXTERNALS				The amount of displacements	MAX (cm)	MIN (cm)
1	2	5	10	20	The max total displacement in right slope of clay layer 4 cm decrease to 2 cm through the layer.	4	2
2	5	10	15	30	The max total displacement in right slope of clay layer 4 cm decrease to 2 cm through the layer.	4	2
3	13	17	25	50	The max total displacement in right slope of clay layer 5 cm decrease to 2 cm through the layer.	5	2
4	15	25	35	70	The max total displacement in right slope of clay layer 5 cm decrease to 3 cm through the layer.	5	3
5	15	25	50	75	The max total displacement in right slope of clay layer 6 cm decrease to 3 cm through the layer.	6	3
6	30	45	65	100	The max total displacement in right slope of clay layer 6 cm decrease to 3 cm through the layer.	6	3
7	60	80	100	140	The max total displacement in right slope of clay layer 6 cm decrease to 3 cm through the layer.	6	3
8	100	145	160	200	The max total displacement in right slope of clay layer 6 cm decrease to 3 cm through the layer.	6	3
9	120	160	180	200	The max total displacement in right slope of clay layer 6 cm decrease to 3 cm through the layer.	6	3
10	200	250	300	350	The max total displacement in right slope of clay layer 6 cm decrease to 3 cm through the layer.	6	3

For more elaboration, Figure 5.3 and Figure 5.4 are plotted to show the tolerance of the maximum and minimum displacement due to increases in mesh density.

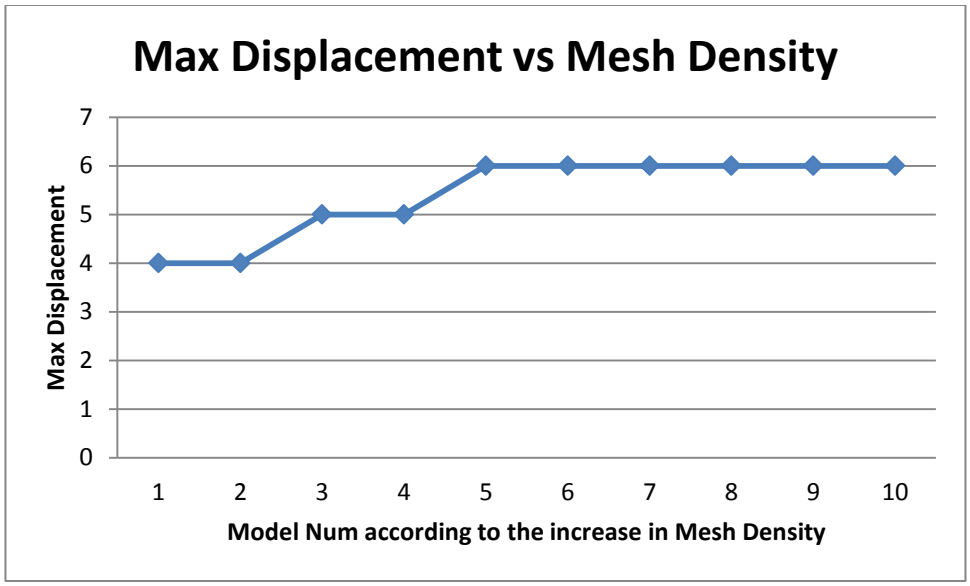

Figure 5. 3 Maximum Displacement changes due to increase in mesh density (in Clay fill)

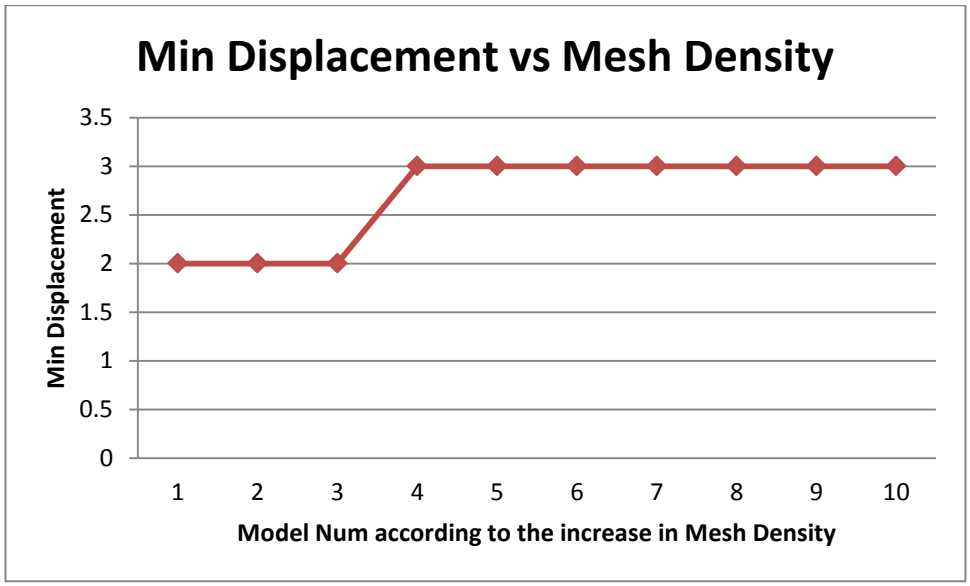

Figure 5. 4 Minimum Displacement changes due to increase in mesh density (in Clayfill)

Therefore, it is clear that after model number 5, there is no variation in the magnitude of displacements in clay fill material. Actually, the clay fill layer is selected to be checked due to the tangible changes that are expected to be observed in this layer. As it is noted about the importance of mesh density in the finite element analysis, it could be perceived that the optimum mesh density should include factors like accuracy of stresses and displacements. It also represents the deformations and altering inside the elements. Hence, in this case, it is preferred to take model number 5, which is defined as 70 nodes on limestone externals, 50 nodes on tailings externals, 25 nodes on rock fill externals and 15 nodes on clay fill externals as well. The result of the aforementioned interpretations is illustrated in Figure 5.5 as Hisarcik tailing dam body model.

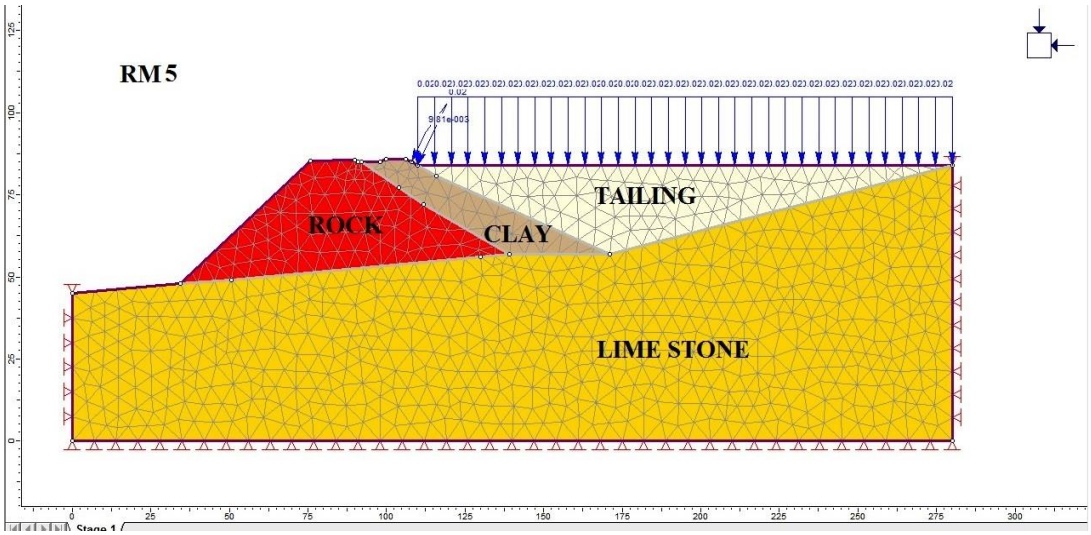

Figure 5. 5 Hisarcik tailing dam in Phase2 v 8.01

Up to now, the proper mesh density and geometry has been defined. By running the model in the software, the current static loading and dynamic layouts can be obtained and discussed.

At the second stage, the dam model is prepared to represent the assumption about the dam condition while its height is raised. The reference about the dam geometry is the map which is presented by the dam engineering design of the EtiMine Company. This map for the future plan is shown in Figure 5.6.

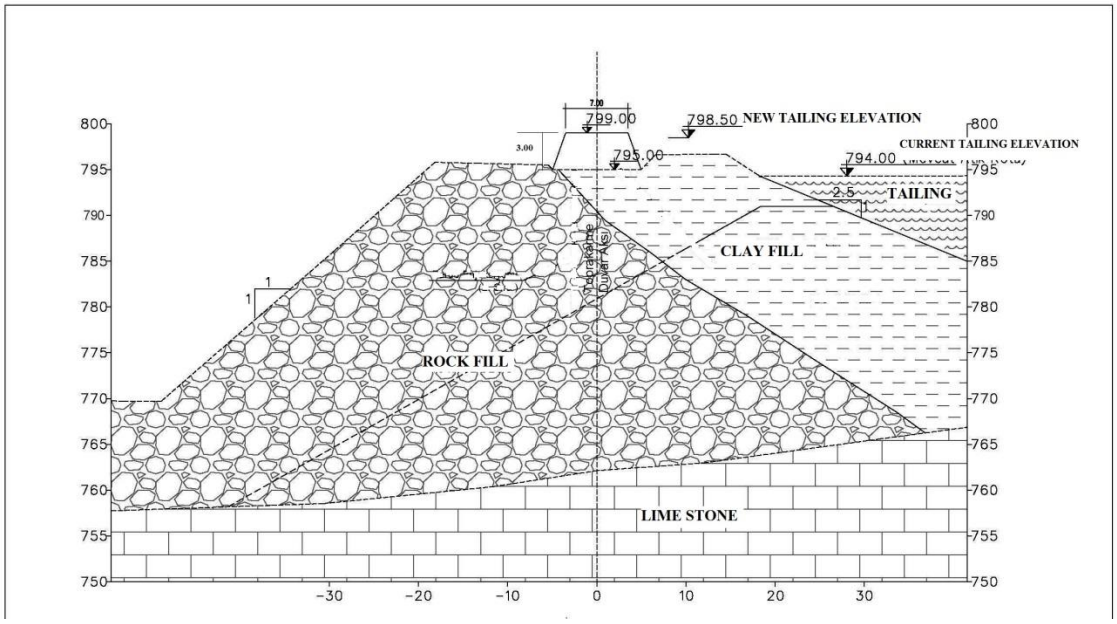

Figure 5. 6 Map of the Hisarcik tailing dam in case of height rising (JMS, 2013)

As it is shown in Figure 5.6, a low amount of height rise is designed for the dam. This elevation change is considered to be located at the dam crest, within inequality of that part. The width of the new surcharge of the dam is considered to be about 7m while the height is 3m. The height determination is based on the decision made by Emet Borax mine engineering management on abandoning the tailing dam in the near future.

Based on the presented map and other factors such as proper mesh density or size, the dam model is prepared for the analysis. In this step, it is assumed to investigate the dam stability by applying the dynamic loading. The predicted dam is represented in Figure 5.7 below:

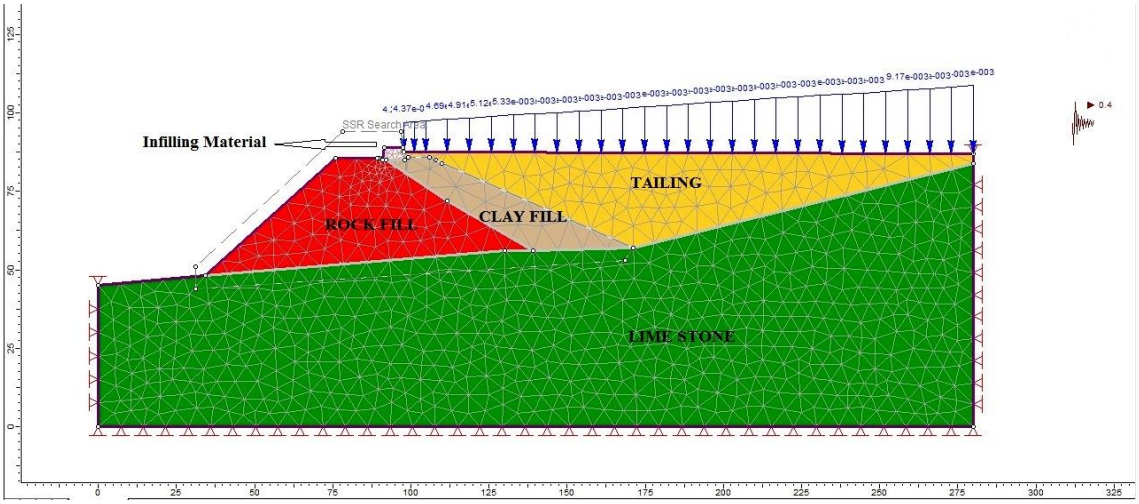

Figure 5. 7 Tailing dam model with height rising

Figure 5.7 represents the dam model while the defined rise of height is applied and since this amount is quiet low, Figure 5.8 illustrates the exaggerated section of the dam crest to show the changes in geometry and elevations.

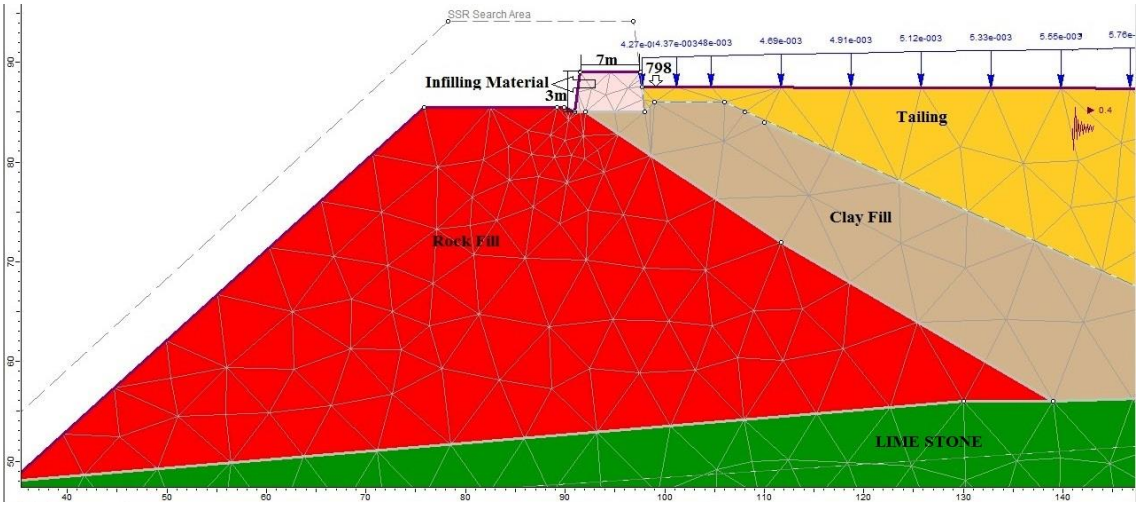

Figure 5. 8 Tailing dam model with a focus on dam crest

The reported information about the geotechnical property of the infilling material, which is used for dam height rise, is not accurate enough to be used as an input parameter in the analysis. The only reference about the characteristics of infilling material is shown in Table 5.4.

Table 5. 4 Infilling material gradation (JMS, 2013)

Sieve Screen Size D (mm)	Percent Passing P %
125 (5”)	100
75 (3”)	85-100
12.5 (1/2”)	25-100
2 (No. 10)	15-100
0.59 (No. 30)	10-65
0.075 (No.200)	<15
Uniformity Coefficient	Cu>5
Plastic Index	PI<6

As it can be seen in Table 5.4, the sieve analysis represents the coarse size material that is mixed by the clay type materials. For considering all the aspects of the infilling material and the effects of it on the dam stability, the detailed information should be available about the reinforced property of material and the amount of compaction. Hence, the infilling material is considered to be as a surcharge and the related material property is assumed to have average parameters between Rock fill and Clay fill. The only point that should be mentioned is about the cohesion amount of the infilling material. To improve the accuracy of the results, two different cohesions are assumed for this mixture. It is considered to have “Infilling 1” and “Infilling 2” as material properties, which are used for the dam height rise. However, the only difference is the cohesion value. Table 5.5 shows the geotechnical properties of the filling material. The parameters of the other layers are the same as in Table 4.5.

Table 5. 5 Properties of different infilling material

Material	Unit Weight (kN/m3)	Poisson Ratio	Young's Modulus (MPa)	Friction Angle (°)	Cohesion (kPa)
Infilling Material (1)	19	0.2	70	30	2
Infilling Material (2)	19	0.2	70	30	0

By applying two different values to the model, the various amounts of critical SRF and maximum shear strength are obtained. This variety also affects the movements but the elastic modulus has more significant impacts on displacements.

5.3 Results of the Analysis

As it is noted, the analysis of the Hisarcik tailing dam was performed by utilizing the Phase2 v8.01 software. Total stresses, displacements and also maximum shear strain are calculated by two dimensional plain strain finite element analysis. Also, elastic and elastic-perfectly plastic material parameters for Mohr-Coulomb and Hoek-Brown, which are the presumptive failure criteria in PHASE2 v.8.01, are applied to the model.

The first step of the dam modeling is to interpret the results of the dam condition in terms of static loading. The best indicator for this interpretation is the total displacement outline as it is shown in Figure 5.9.

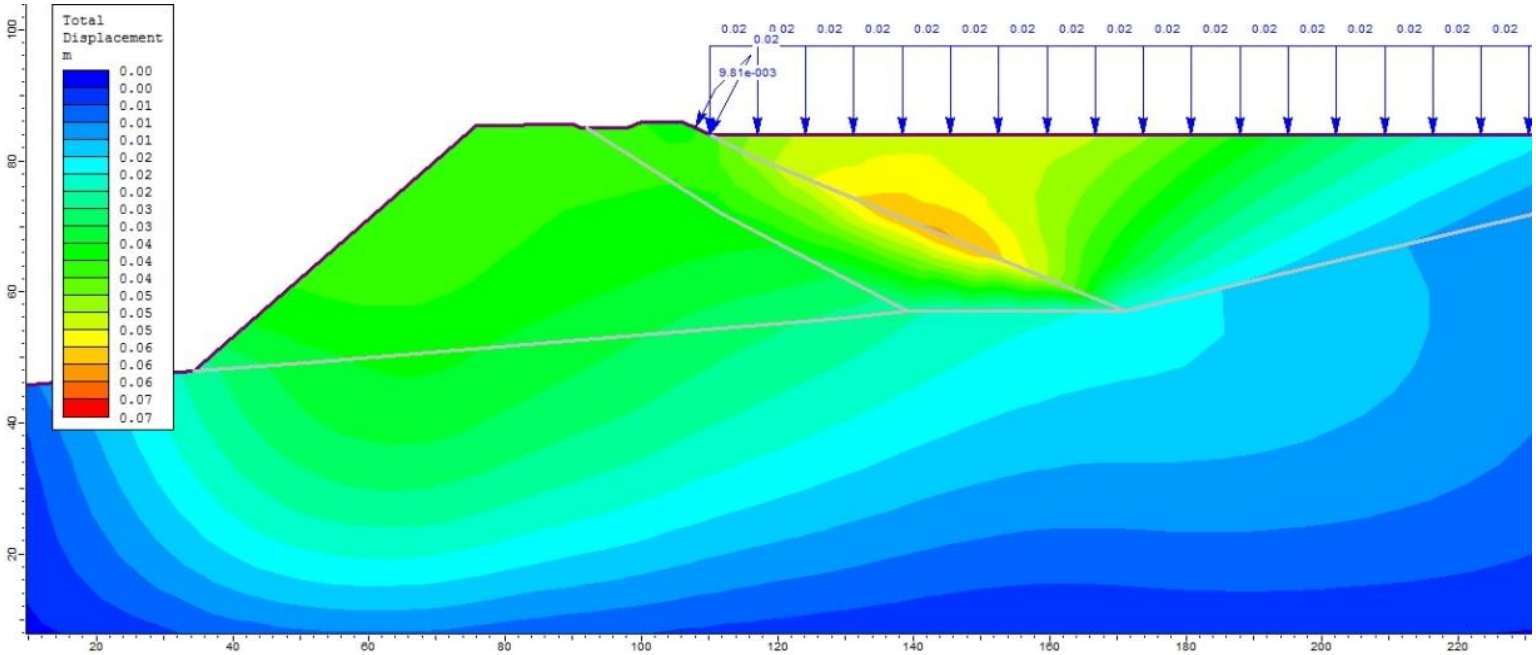

Figure 5. 9 Total displacements due to the static loading

The total displacements of the dam in case of static loading are illustrated in Figure 5.6. According to Table 5.3, the maximum displacement magnitude, which is about 6 cm, corresponds to the clay fill layer. This amount of displacement for the dam body is the first step to predict some deformation in dynamic loading analysis when static loading presents. Since the results represent the analysis in one stage of the ponded water loading, the interpretation seems feasible, also the values are evaluated by comparing them to the literature (e.g Table A.1) about similar cases. So, as a second step, the dynamic load should be applied to the dam by adding the seismic coefficient in the analysis process and also enabling the strength reduction factor option.

The shear strength reduction option in Phase2 is applicable to automatically carry out the finite element slope stability analysis and to estimate the critical strength reduction factor for the model. In most cases, this value resembles the factor of safety in order to investigate the slope stability. Actually, the basic concept of the SSR method includes some steps like reduction in the strength parameter by a certain factor. This process will be continued until the model determines the critical strength reduction factor or safety factor of the slope. Since the study observes the displacements and deformation related to the dam body and to obtain the reliable results in the stability analysis of the Hisarcik tailing dam, there should be a limitation for the SSR analysis to avoid the existence of tailing material in the analysis. Phase2 v.8.01 has a solution for this problem, which is called SSR search area. This option is defined to enclose the section of the model and it gives the opportunity to concentrate on the analysis inside the desired area of the model. By this option, it could be possible to avoid the existence of the tailing section in the SSR process. The representation of the model by applying the seismic coefficient and defining the SSR area is shown in Figure 5.10.

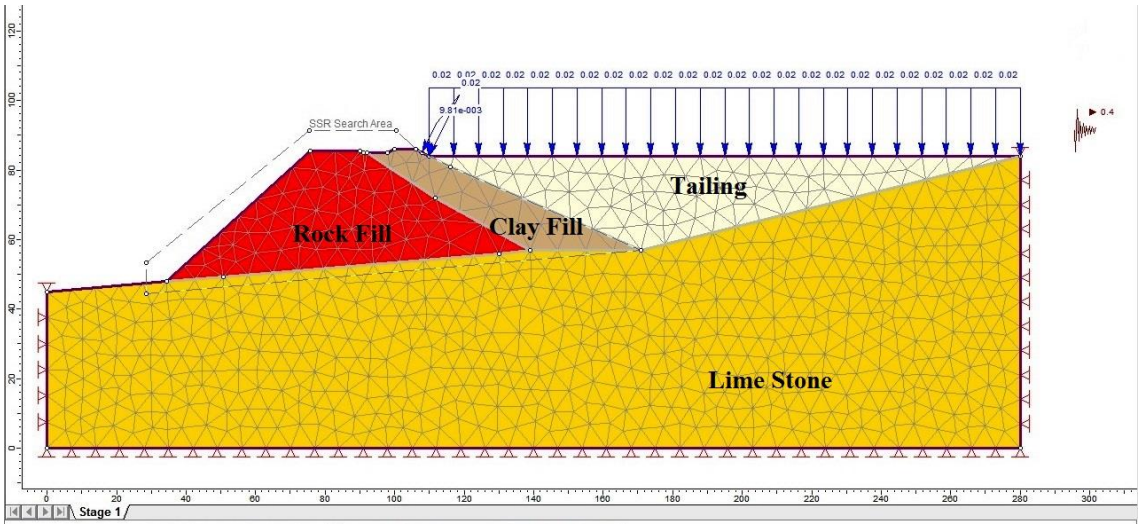

Figure 5. 10 Hisarcik tailing dam in Phase2 v 8.01 in term of dynamic loading

It should be mentioned that according to the same mesh density investigation like what has been done for the static loading analysis, a similar number of nodes in layer externals are appropriate for the dynamic loading condition analysis. The layout of the stability analysis of the dam by considering the dynamic loading includes several factors like shear strain and displacements. The total displacement result of the analysis is illustrated in Figure 5.11. Figure 5.11 indicates that the resultant vertical and horizontal displacements have been concentrated in clay layer as it is expected in the previous analysis.

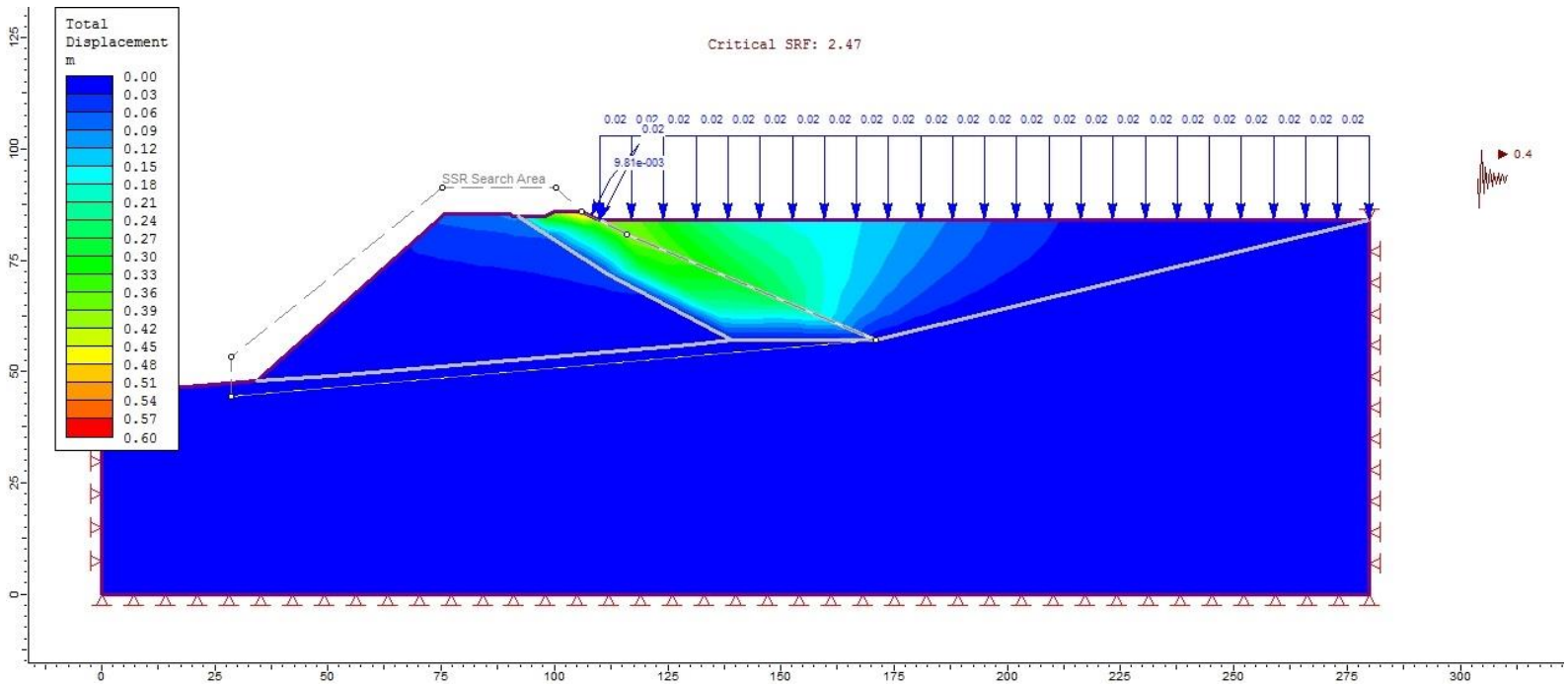

Figure 5. 11 Total displacement due to the dynamic loading

According to Figure 5.11, in clay fill crest section, the high amount of displacement is observable and this amount could be estimated about 45 cm. Also, as it is mentioned before, the displacement distribution in the tailing layer is omitted by applying the SSR area option and by enclosing the desired section of the dam for the analysis. Besides, there is a lower limit for displacement in inner slope, which is estimated about 5 cm. Moreover, for the rock layer, the maximum and minimum values are estimated to be about 10 cm and 1 cm, respectively. Again the most considerable displacement is related to the dam crest and inequality or roughness in the head of the dam. The ups and downs through the crest have caused the drastic movement in that section. Another factor that can help interpret the result is the maximum shear strength. In the strength reduction factor method, the maximum shear strain counters are assumed to define the critical failure zones. The outline counters for this analysis, in term of the maximum shear strain, show the concentration in the clay layer the same as that of displacement. Similarly, the dam crest shows a higher amount of shear strain when compared to the other parts. Therefore, the most crucial part of the dam could be the dam crest. Figure 5.12 shows maximum shear strain result for the dynamic loading analysis.

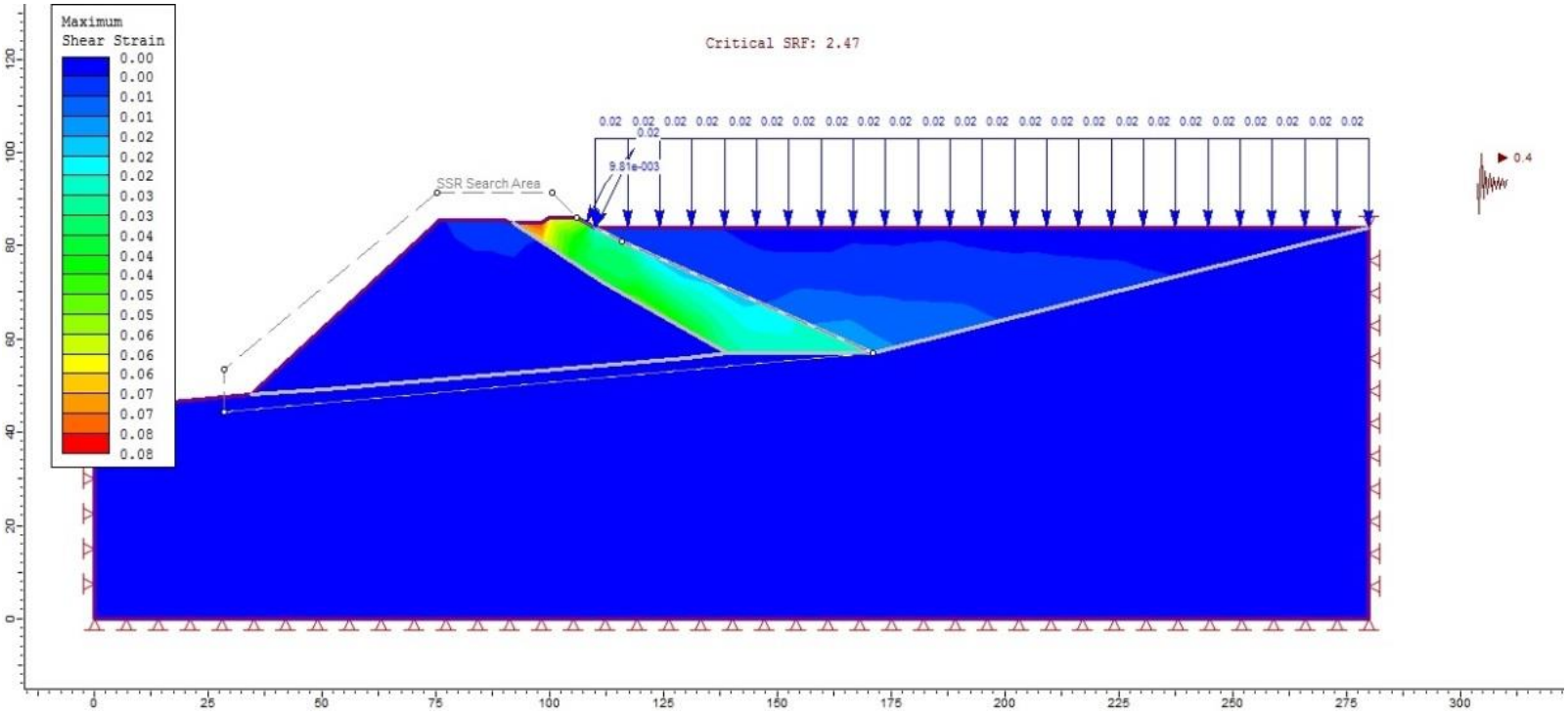

Figure 5. 12 Maximum shear strain due to the dynamic loading

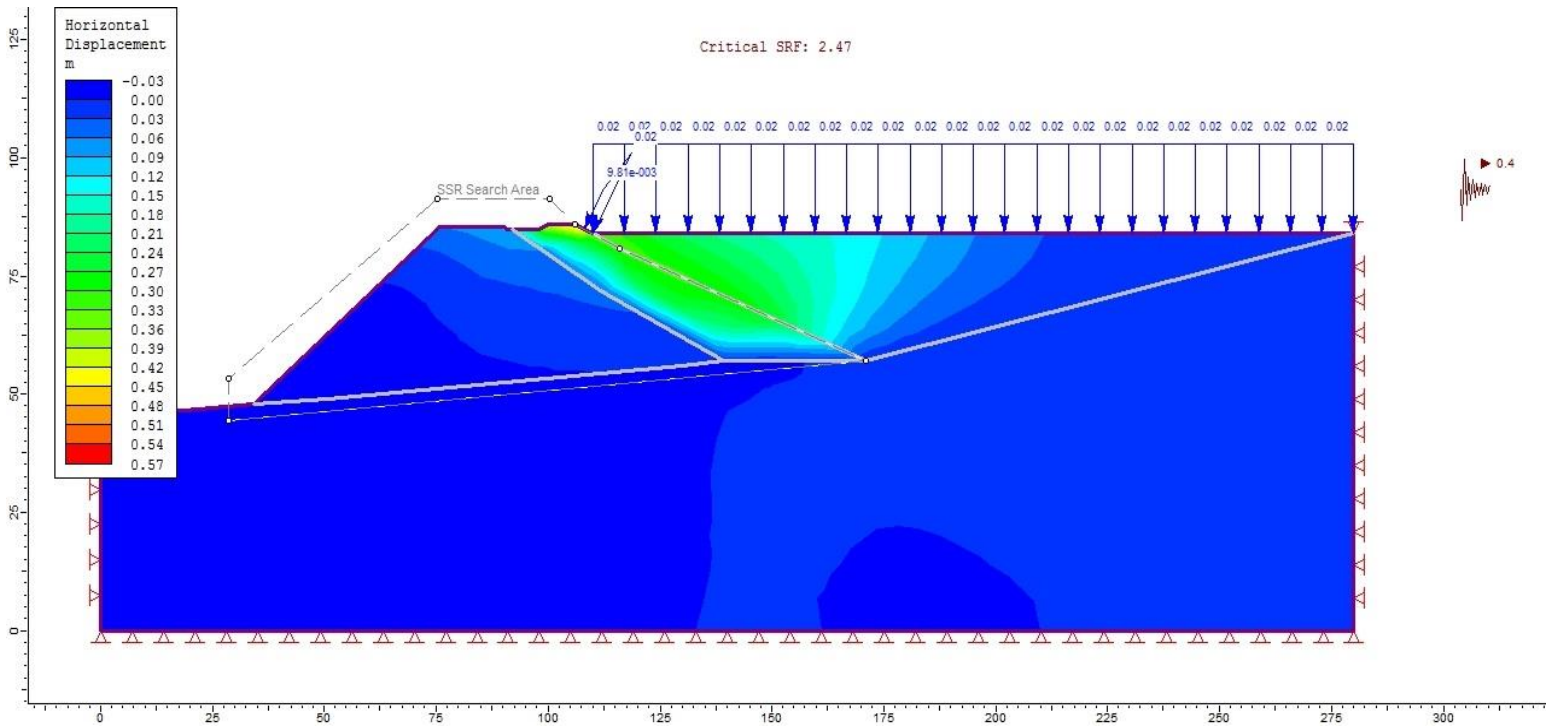

Figure 5. 13 Horizontal displacement due to the dynamic loading

The other options of displacement could represent detailed information about the movements through the dam body. In Figure 5.13, the horizontal displacement is illustrated. In other words, the magnitude of the movements in the X direction is important to show the effect of seismic activities.

The result indicates that the horizontal displacement has a more significant role in the total displacement compared to the vertical one. The maximum amount of horizontal displacement is about 38 cm in the dam crest and the minimum amount is estimated to take 4 cm inside the clay fill layer. This outline for the rock layer is at a maximum amount, around 9 cm and the minimum amount is 1 cm.

In addition, the critical strength reduction factor is estimated to be around 2.47. It means that all the results are feasible by considering this magnitude. This amount could be considered as an acceptable for the safety of the dam. However, it should be noticed that the dam is going to be full and a new project for height rise should be determined. Therefore, in the following step, the model should be involved so that the height rise and the same analysis can be carried out.

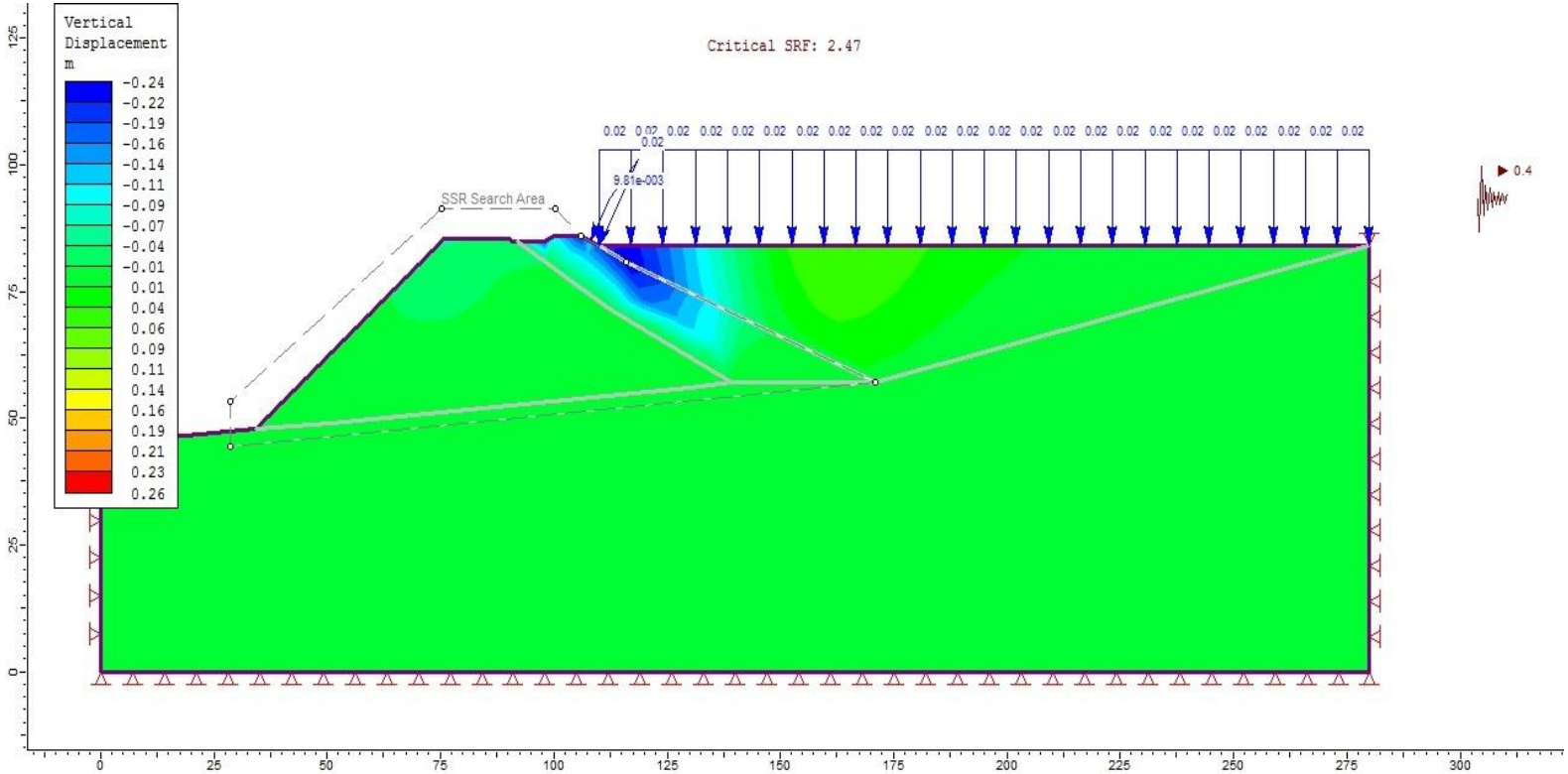

Figure 5. 14 Vertical displacement due to the dynamic loading

The vertical movement or the movements through the Y direction is shown in Figure 5.14 as well.

The maximum magnitude of the vertical displacement is estimated to be about 28 cm while the minimum amount is 2 cm. For rock, the maximum movement amount is 3 cm and the minimum one is 1 cm. The result of the vertical displacement reveals the movement concentration through the dam body.

The detailed description about the dam height rising was mentioned in the previous part. Since there are two different infilling materials, the results are illustrated for each of them separately.

Firstly, the results are the output of the dam model by applying the “infilling 1” material properties, which means that the cohesion is assumed to be 2 kPa. To interpret the changes related to the dam height rising, the first layout can be considered as the total displacement, which is shown in Figure 5.15.

As it is shown in Figure 5.15, the total displacement in the new filling layer is more than the other parts. Also, the amount of movement in the clay fill layer is increased. The maximum value of movement in the new layer is about 45 cm and in the clay fill layer it is about 39 cm. Moreover, the minimum displacement for the filling layer is around 7 cm and it is estimated to be about 5 cm for the clay fill layer.

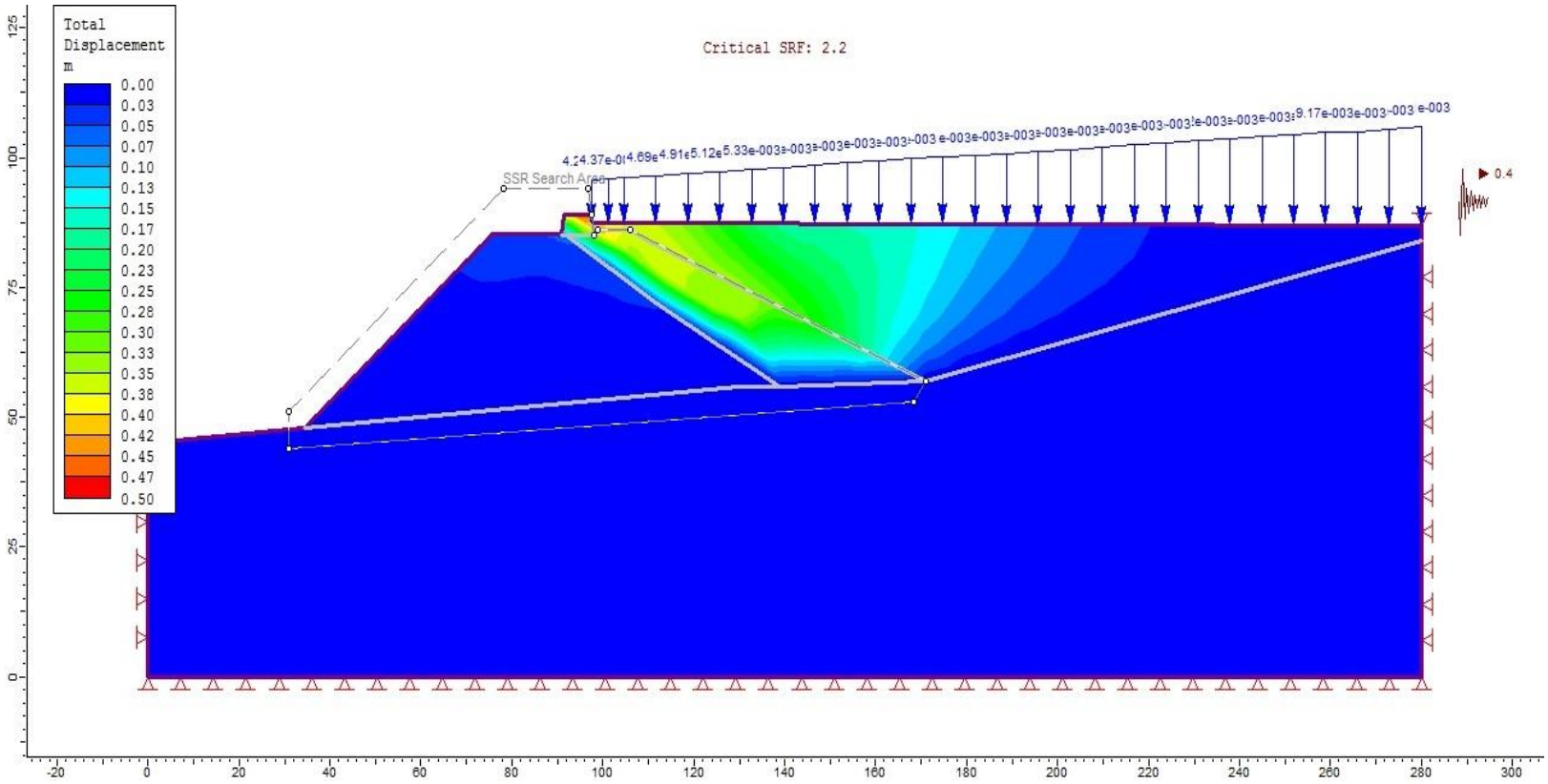

Figure 5. 15 Total displacement in case of dam height rising (infilling 1)

The maximum shear strength is illustrated in Figure 5.16 with the critical failure zone.

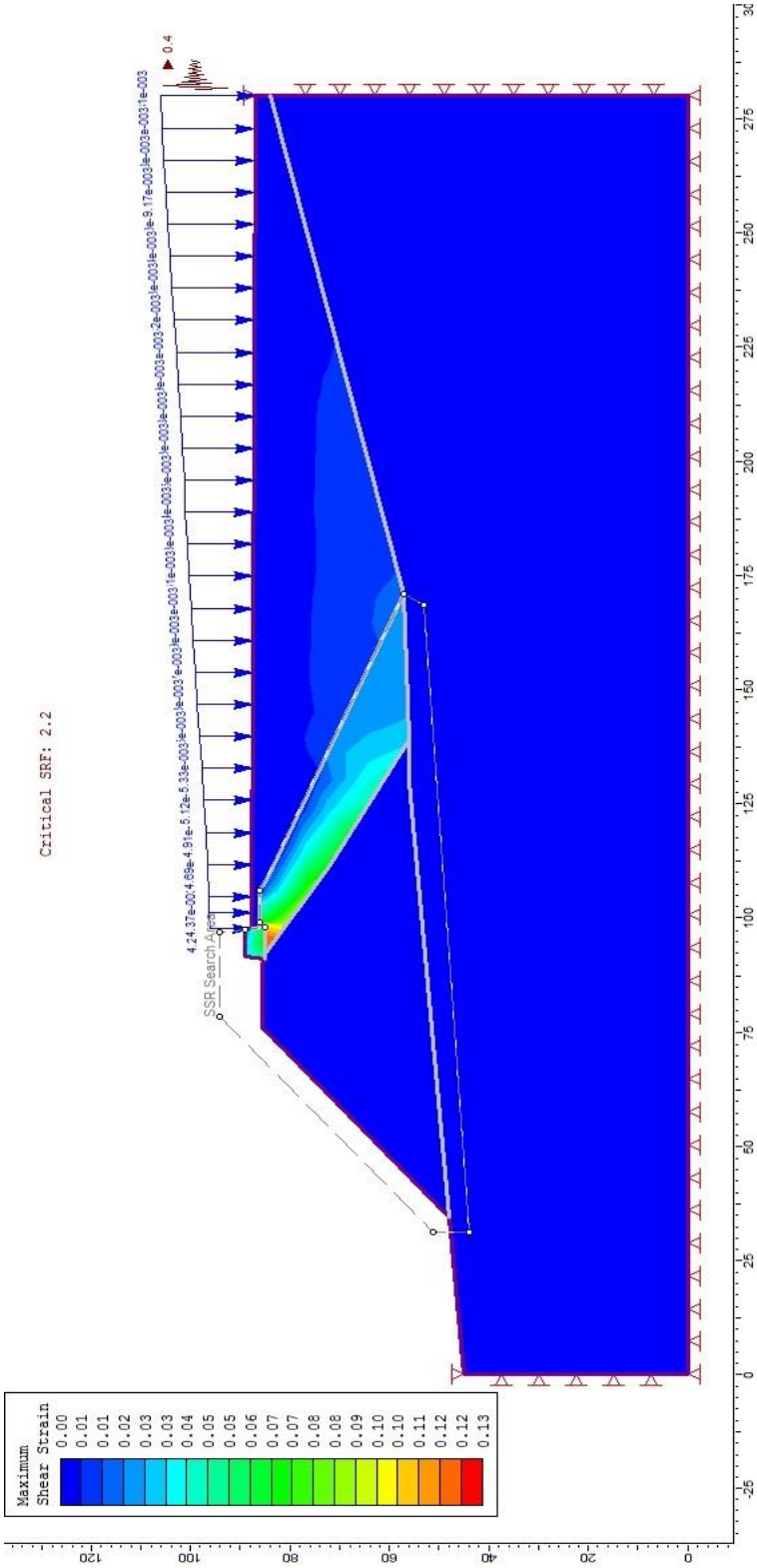

Figure 5. 16 Maximum shear strain in case of dam height rising (infilling 1)

As it can be seen from Figure 5.16, the loading of the new stage has caused a significant shear strain in the transition zone among the clay fill and the infilling area. It means the most critical area is exactly located beneath the basement of the height rising plan.

The horizontal displacement of the dam body could represent the behavior of the new geometry. In Figure 5.17, the horizontal movement is shown while the tailing and dam body elevation is increased.

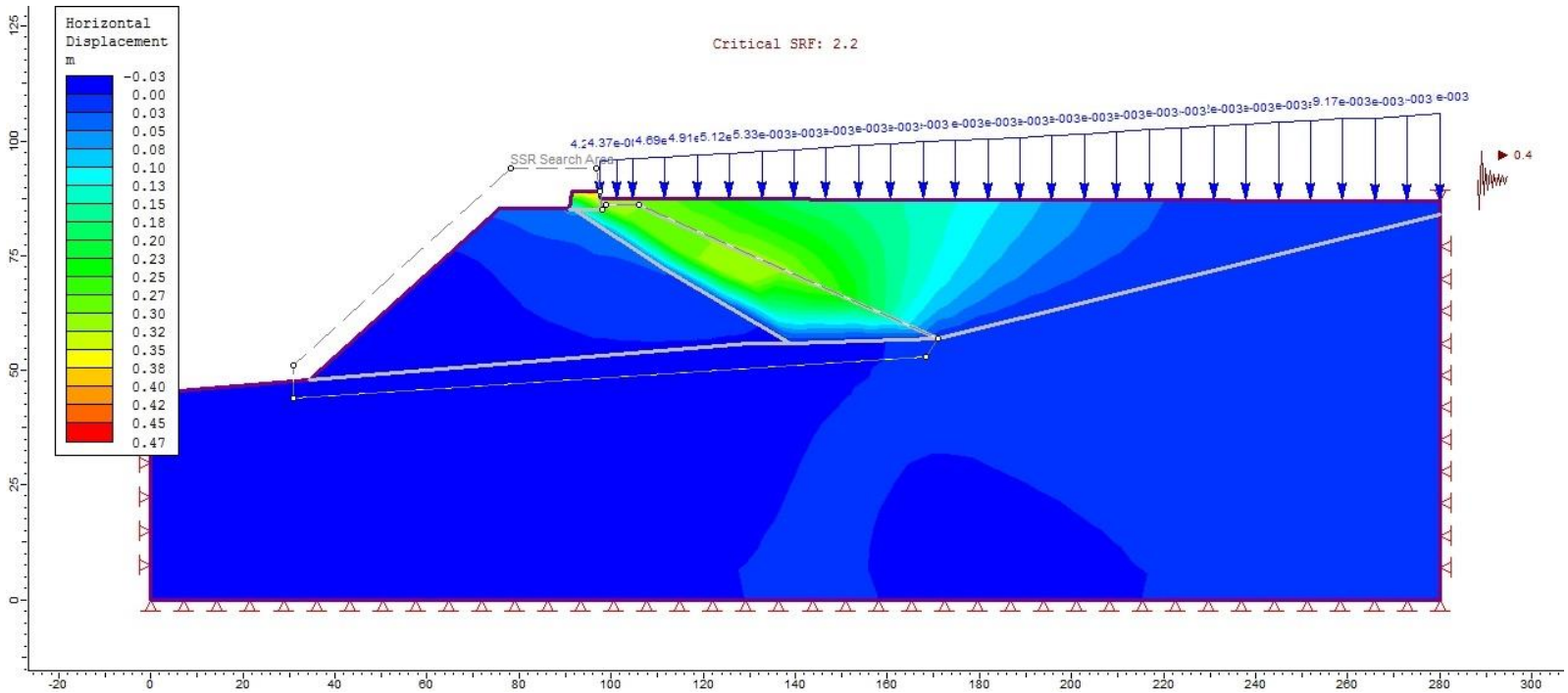

Figure 5. 17 Horizontal displacement in case of dam height rising (infilling 1)

The results have shown a high amount of displacement in the new stage. The maximum amount is about 39 cm and for the clay fill layer it is 31 cm and the minimum values are 7 cm and 5 cm, respectively. These amounts show the movements in the X direction.

The vertical displacement is also important to illustrate the impact of the new loading stage. Figure 5.18 shows the vertical displacement for the raised dam height.

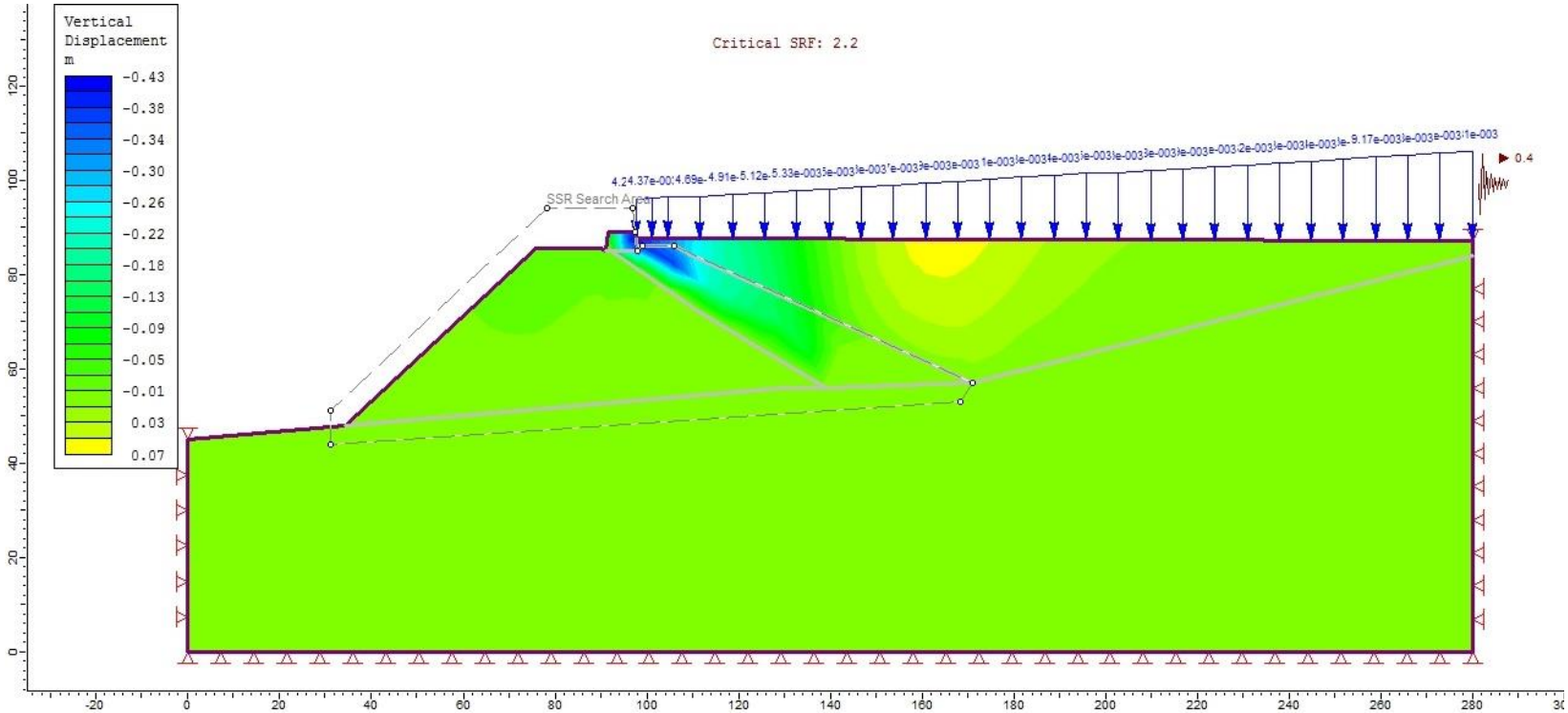

Figure 5. 18 Vertical displacement in case of dam height rising (infilling 1)

As it is indicated in Figure 5.18, the maximum amount of the vertical movement in the new stage and the clay fill are 26 cm and 23 cm, respectively. Also, the minimum displacements are 1 cm for the new stage and 1 cm for the clay fill.

It is obvious that by applying the extra loading to the dam body, specifically in the dam crest section, the magnitude of displacements have increased. Now, the upcoming results have shown the stability investigation while the cohesion of the infilling material is decreased to zero. As it is mentioned above, for distinguishing among the two different cohesions, they have been called with two different names; the stability analysis results by applying “infilling 2” material properties for the height rise are shown in Figure 5.19.

To compare the results of the two different cohesions on the analysis, the same outputs could make the analogy. The first one is the total displacement which is shown in Figure 5.19.

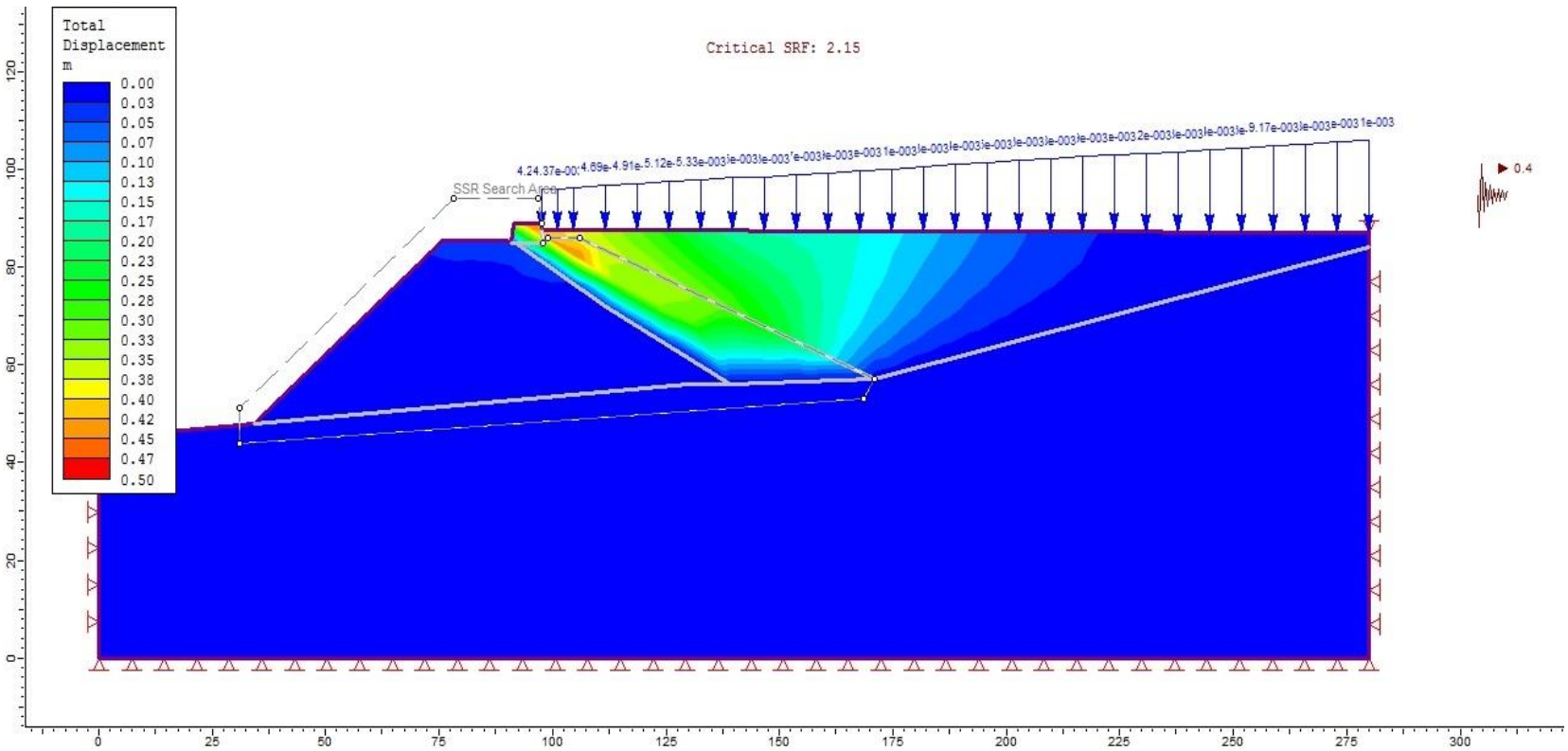

Figure 5.19 Total displacement in case of dam height rising (infilling 2)

The maximum value of movement in the new layer is about 49 cm and about 42 cm in the clay fill layer. Moreover, the minimum displacement for the infilling layer is around 7 cm and for the clay fill layer estimated about 5 cm.

The maximum shear strain of the cohesion-less filling material is shown in Figure 5.18 to bold the critical failure zones.

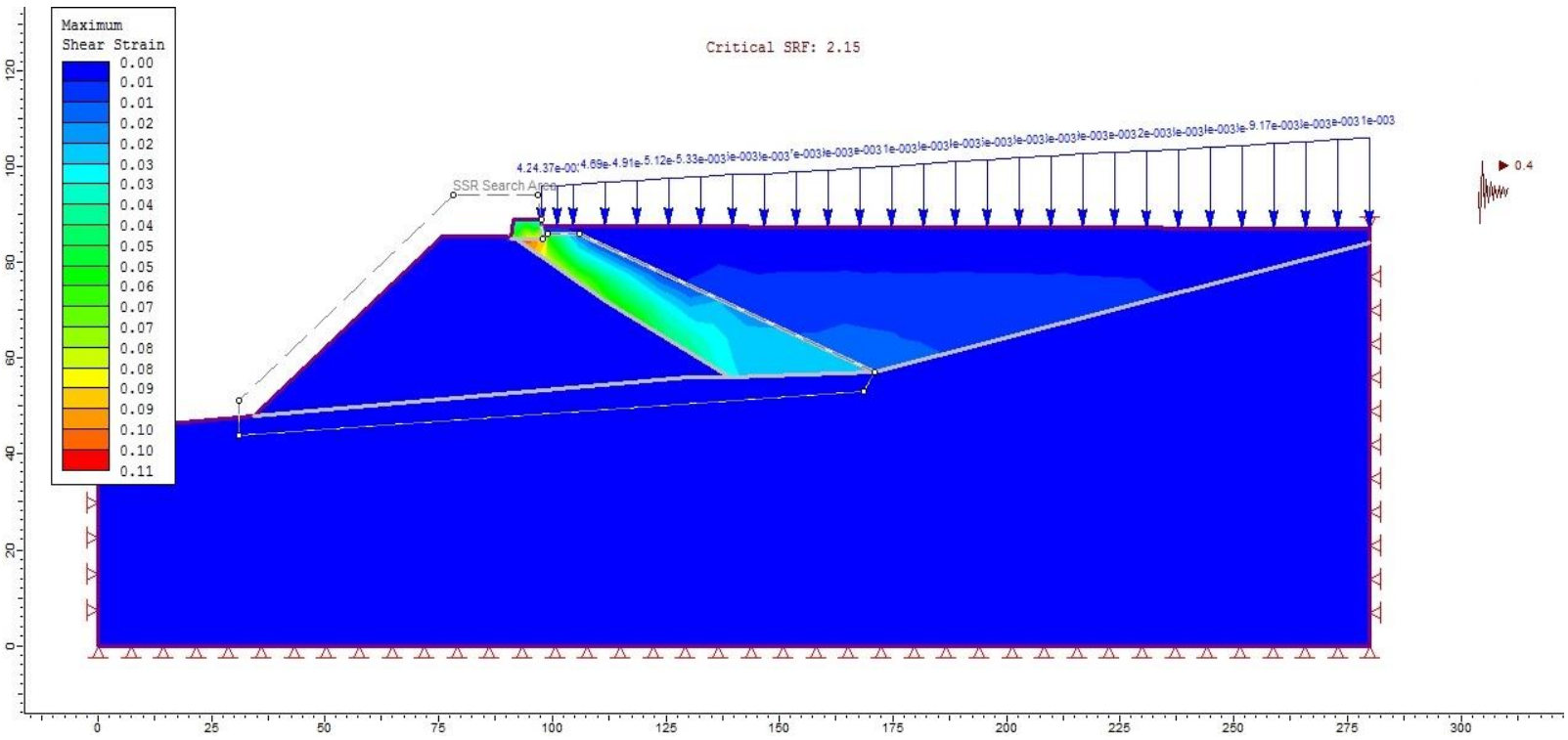

Figure 5. 20 Maximum shear strain in case of dam height rising (infilling 2)

As it can be seen, still, the critical zone is considered to be between the basement of the new stage and the clay fill crest. The applied surcharge effects have impressed the zone, which includes the crest and borders between the rock fill and the clay fill.

Again, to understand the movements of the dam body in detail, the horizontal and vertical displacements outputs could help. The horizontal displacement is indicated in Figure 5.21.

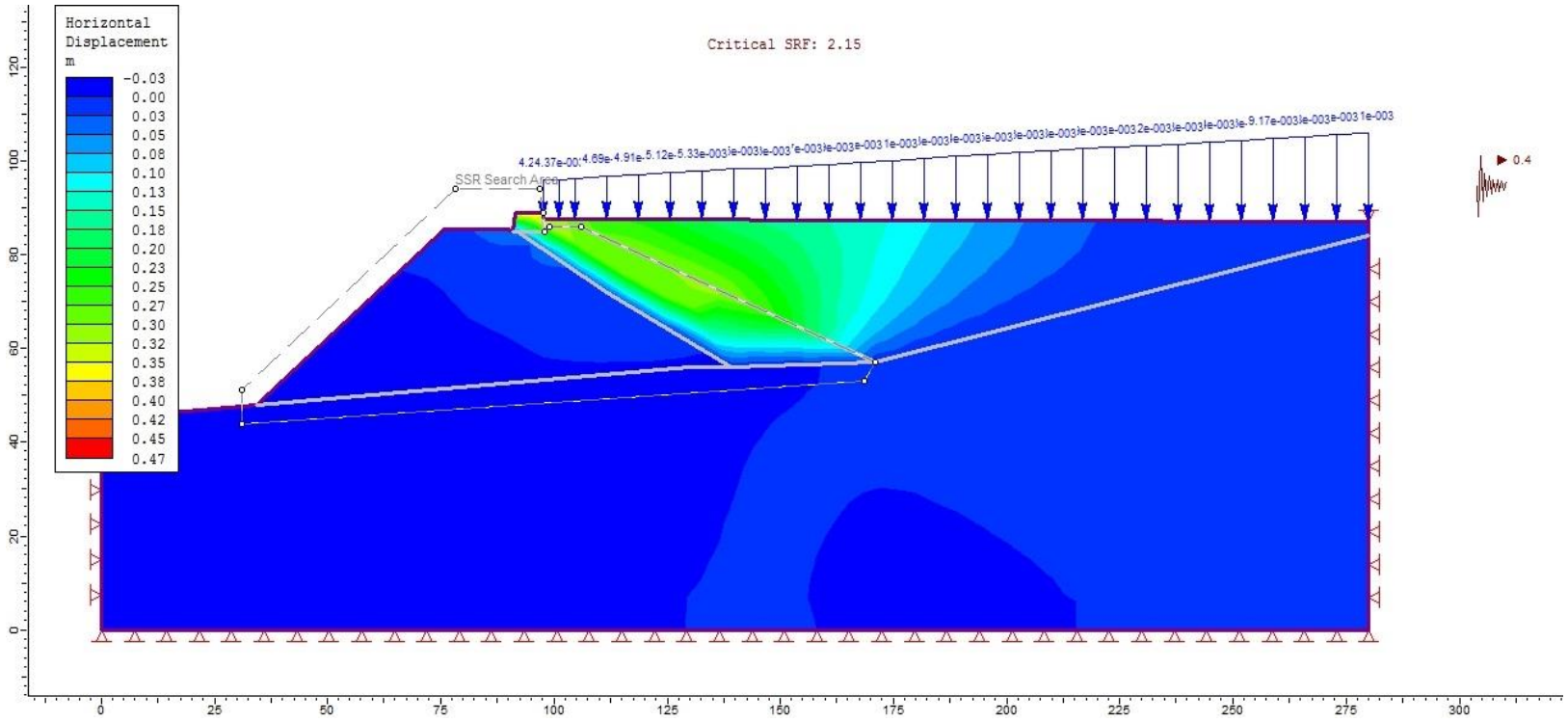

Figure 5. 21 Horizontal displacement in case of dam height rising (infilling 2)

The results have shown the maximum amount of displacement 42 cm for the new stage and 34 cm for the clay fill material. Also the minimum displacements are 7 cm for the new stage and 5 cm for the clay fill. The vertical displacement is shown in Figure 5.22.

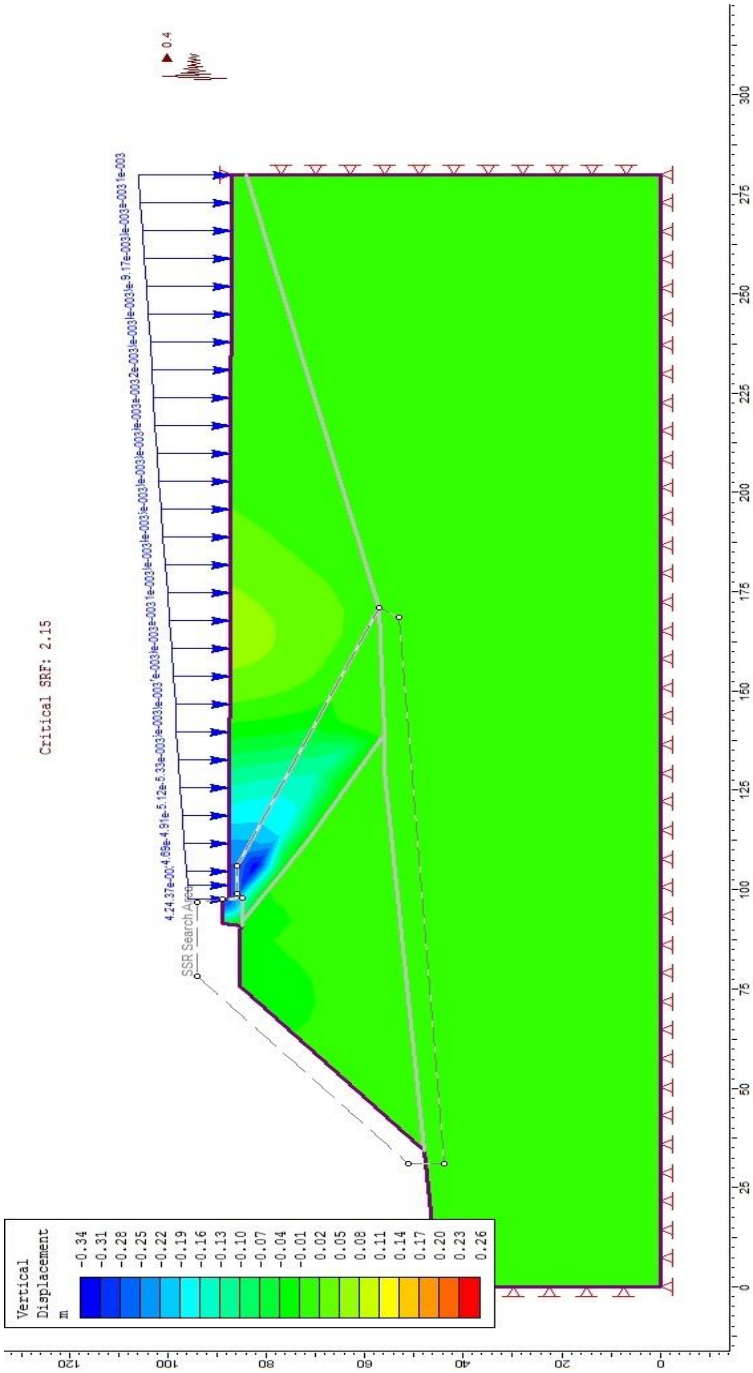

Figure 5. 22 Vertical displacement in case of dam height rising (infilling 2)

The displacements, which were illustrated in Figure 5.22, are the vertical movements that occur through the dam body. The maximum amounts of the displacements are 26 cm for the new stage and 23 cm for the clay fill layer. Also the minimum values are estimated 1 cm for the new infilling layer and 1 cm for the clay fill.

By comparing the results for both cohesion values, it is perceived that the dam height rising will apply more loading to the dam body. Since the new stage has been considered as a surcharge, more movements are observed and also the strength factor has decreased. For more elaboration, applying the bigger load to the system will change the results and in this case, two different cohesions are checked for this loading. In "infilling 1" case, which material properties are applied (c=2 kPa), the strength is predicted to be relatively more than the "infilling 2" condition, where (c=0 kPa) is employed. The results verify this prediction and all the displacement options have endorsed it. The magnitude of the total displacement in the first condition (applying infilling 1 property as cohesion) is 4 cm less than the second condition (applying infilling2 property as cohesion). On the other hand, this extra loading is affected on the clay fill layer and also the rock-fill. Also, Table A.1 and Figures A.1 and A.2 represent the stability analysis of rock-fill slope using SLIDE v.5.0. Both of them have increased relatively and the significant compression can be observed on clay layer.

To confirm the obtained displacements amounts from the analysis, it is required to find the viable references. By surveying the literature about the similar works which are summarized in Table 4.15, results have been checked.

Table 4. 15 The maximum distribution of total displacement based on the literature review

Ref.No	REFERENCE	SF	MAX DISPLACEMENT (m)
1	Debarghya Chakrabort, 2009	2.27	0.55
2	Mihai, 2008	2.84	0.43
3	Choudhury, 2011	1.97	0.63
4	Liang, 2010	1.83	0.35
5	Song, 2012	2.5	0.65
6	Zlagnean, 2009	2.4	0.57
7	Smith, 2010	1.9	0.32
8	Bingbing, 2008	1.75 (SRF)	0.48
9	YaoYao, 2008	2.51	0.55
10	Mosquera, 2000	1.78	0.46

According to the represented amounts of total displacement, Table 4.16summerzied the statistics of data collected from the literatures.

Table 4. 16 Summary statistics of the max total displacement, obtained from the literature

Parameter	Mean	Standard Deviation	Range	Min	Max	Coefficient of Variation
Total Displacement (m)	0.5	0.11	0.33	0.32	0.65	0.2

By considering the Tables 4.15 and 4.16 and the obtained amount for total displacement, the result of the analysis could be acceptable. The range of the displacements in term of dynamic analysis of current condition and raised height dam shows the range of 40 to 57 cm. According to the gained amount in Table 4.15, thesis numbers are included in the range.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

In this study, the stability of Emet Borax mine tailing dam (Hisarcik) located in Emet, Kütahya is investigated for both current static condition and dynamic loading in the current and future plans. The input parameters for analysis were obtained from the reported data for Emet Borax mine document, which was prepared by the JMS Company. Besides, the other required parameters are obtained from the literature reviews. The finite element method (FEM) is employed to evaluate the dam stability. For this purpose, Phase2 v8.01, which is FEM-based numerical modeling software, is used.

According to the literature surveys, the upstream method for constructing tailings dams is the most vulnerable method, which has caused several incidents. This is the reason that justifies the stability analysis of the Hisarcik tailing dam. As it is noted, the geotechnical properties of the tailing dam were determined with the help of the previous site investigations and also controlled with the values of similar properties in the literature. The first stage of the stability analysis is conducted for the static loading condition that includes the current condition of dam. Secondly, the stability is investigated by applying the dynamic loading due to seismic activities. By considering the stability investigation due to the shear strength reduction method, the current elevation of the dam is inquired. Also there is a plan for the future height rise which is added to the dam geometry and the stability is predicted as well. According to the results of the analysis, the following conclusions have been drawn.

I. The dam's current condition and presumptive height raised dam are modeled via PHASE2 v.8.01, which is a finite element method-based software and the safety results analysis were compared with the proposed values in literature. The results have shown acceptable amounts of stability for the dam body.

II. In order to analyze present condition of the dam, the body of dam is assumed to be affected by static loading considering the ponded water pressure. Also dynamic loading is applied by adding the proper coefficient of seismic activity. Based on similar works, the feasibility of displacements in term of static loading condition is investigated. The shear strength reduction (SSR) method is employed to estimate the safety value of the dam in terms of the dynamic loading condition for both the current and after height rising. The SRF or safety value seems to be satisfactory and the dam stability in the current condition could be reliable. However, since the total head of ponded water level is going to reach the dam crest, special attention must be given to the ponded water level.

III. The lateral displacement of clay-fill layer, which is illustrated in the dynamic analysis, could be constant while the loading of ponded water level is going to increase by the time. This, in turn, will stimulate more ponded water loading.

IV. For the dam stability in terms of raised height, the material properties are assumed with two different cohesions. For each of them the displacements and shear strain outputs are checked. Based on the basic information about the infilling material, the dam is analyzed and the stability was found satisfactory. But there should be more data about the material properties and the construction type to obtain much more accurate results.

V. For the entire analysis, the safety factor (SRF value), by considering the reported geotechnical properties of dam body and tailings, according to Sowers (1979), is satisfactory for stability. Also, based on the "Hisarcik Atik

Barajı Teknik Raporu" which was prepared by Etimine engineering and the design section and the JMS Company, the current condition of the the tailing dam was found safe.

Based on the inferred conclusions, some recommendations are given below.

1. According to the geological reports, at the down side of the Hisarcik Tailing dam, there is an old valley. No hydrological studies have been carried out by the mining company in the area. Since this rock valley has been aged between 500 to 1000 years, some destruction possibility is susceptible, so, there should be remedial measurements to intensify the stability of tailing dam down-side.

2. Due to the last injection through the dam body layers and increasing the resistance of impermeable zone, the stability of the dam has been increased. However, there should be a plan for dam height rising even if the dam is going to be abandoned in the near future. Moreover, the retention wall must be constructed at the down side of the dam.

3. Seepage may cause erosion and accordingly an unexpected failure. In this study, no seepage analysis has been done but according to the previous studies, to avoid this hazard, drainage liners should be devised through the upper side of the dam by clay and rock materials.

4. Researches on the rates of mining operations also show that the facilities need additional tailing impoundment. For new projects, more static and dynamic reliability experiments should be carried out in order to satisfy almost 20 to 25 years of reservoir demand.

REFERENCES

Abolfazl, S., Ali. P., Mohyeddin, S. B., Amir Hossein, A. S. (2007). “Geotechnical characteristics of copper mine tailings: A case study”. Geotechnical Geology. Engineer. Vol.25. pp: 591-602.

Abolhassani, F (1988). « Unusual Behaviour of an Earth-Rockfill Dam”. Second International Conference on Case Histories in Geotechnical Engineering, 347-354.

Agency, U. E. (1994). “DESIGN AND EVALUATION OF TAILINGS DAMS”, United States: USE Office of Solid Waste.

Ambraseys, N.N., Douglas, J., Karma, S.K. and Smit, P.M., (2005), “Equations for the estimation of strong ground motions from shallow crustal earthquakes using data from Europe and Middle East: horizontal peak ground acceleration and spectral acceleration”, Bulletin of Earthquake Engineering, 3, 1-53.

ASCE. (2002). “American Society of Civil Engineers Los Angeles section geotechnical group”. Recommended procedures for implementation of DMG special publication 117: Guidelines for analyzing and mitigating landslide hazards in California. Southern California Earthquake Center.

ANCOLD. (2011). “Australian National committee on Large Dams”. Guidelines on tailing dams: Planning. Design. Construction. Operation and closure.

ATAMAN G. and BAYSAL 0. (1978), “Clay mineralogy of Turkish borate deposits”. Chem. Geol., 22, p.233-247

Azadegan, O. (2012). “Finite Element Modeling of Stability of A New Device of PLT set Bracing”. EJGE, 17, 511.

Baba, K., (2002), “On a Consideration for a Non-linear Analysis of Rockfill” 9th Regional Seminar on Earthquake, September 7-17, Istanbul.

Babu, G.S., Srivastava, A., & Sahana, V. (2007). “Analysis of stability of earthen dams in kachch resgion”, Gujarat, India, Engineering Geology, 94(3-4), 123-136.

Baget, (2013), -Baraj,Geoteknik,Temel-“Emet Mine Tailing Dams Rehabilitation Report”, EtiMine Engineering Section general management.

Baoshan Zhang, C. L. (1983). “Shear wave velocity and geotechnical properties of tailings deposits”. Bulletin of the International Association of Engineering Geology - Bulletin de l'Association Internationale de Géologie de l'Ingénieur, pp 347-353.

Barton, N.R. and Bandis, S.C. (1990). “Review of predictive capabilites of JRC-JCS model in engineering practice”. In *Rock joints, proc. int. symp. on rock joints,* Loen, Norway, (eds N. Barton and O. Stephansson), 603-610. Rotterdam: Balkema.

Billiton, B. (2009). Tailing Storage Facility Design Report. Olympic Dam Expansion Draft.

Bingbing, Z. B. (2008). Study on Slope Stability Analysis of Tailing Dam Based on FEM Strength Reduction Method. JOURNAL OF YANGTZE RIVER SCIENTIFIC RESEARCH INSTITUTE, 53-57.

Boore, D.M, Joyner, W. B. and fumal, T.E., (1993), “Estimation of response spectra and peak acceleration from Western North American Earthquakes: An interim report”, U.S. Geol. Surv. Open-File Rept. 93-509, 72 p.

Boore, D.M., Joyner, W. B. and Fumal, T.E., (1997), “Equations for estimating horizontal response spectra and peak acceleration from Western North American earthquakes: A summary of recent work”, Seis. Res. Let., 68, 128-153.

Bishop, A.W., (1955), “The Use of the Slip Circle in the Stability Analysis of Slopes”, Geotechnique, V.5, No.1, 7-17

Brawner, C.O. and D.B. Campbell. (1973). "The Tailings Structure and its Characteristics - A Soil's Engineer's Viewpoint." In: *Tailings Disposal Today*, Proceedings of the First International Tailings Symposium, Tucson, Arizona, October 31, November 1, 2 and 3, 1977. C.L. Aplin and G.O. Arsall Jr. (Editors).

Britenbach, A. J. (2009). “Improvement in the stability of upstream method phosphate tailings dams with rock fill shells”. Phosphate Conference in Marrakech, Morocco, COVPHOS III , 11.

Budhu, M., (2000), “Soil Mechanics and Foundations”, John Wiley and Sons, New York, 585 p.

Bureau, G.J., (2003), “Dams and appurtenant facilities in Earthquake Engineering”, Handbook (edited by Chenh, WF and Scawthom, C.) CRS press, Bora Raton, Chapter 26, 47 p.

Campbell, K.W., (1981), “Near-source attenuation of peak horizontal acceleration”. Bulletin Seism. Soc. Am., vol. 71, N.6, 2039-2070.

Campbell, K.W. and Bozorgnia, Y., (1994), “Near-source attenuation of peak horizontal acceleration from worldwide acceleograms recorded from 1957 to 1993”, Proc. Fifth U.S.National Conference on Earthquake Engineering 3, Chicago, Illinois, 283-292.

CANMET. (1977), “Pit Slope Manual . In Canada Center For Mineral and Energy”, Technology Report (p.136). Canada: CANMET.

California Mining Association, (1991), Mine Waste Management. Edited and Authored by Ian Hutchison and Richard D. Ellison. Sponsored by the California Mining Association, Sacramento, CA.

Choudhury, D. (2011). “Seismic Behavior of Tailings Dam Using FLAC3D”. American Society of Civil Engineers, 34.

Cedegren, H.R., (1977), “Seepage, Drainage and Flow Nets” John Wiley and Sons, Second Edition, New York.

Cutting, J. (2007). “Geotechnical characteristics of copper mine tailing”. Geotechnical and Geological Engineering, 591-602.

Dai, F.C., Lee, C.F and Zhang, X.H., (2001), “GIS-based geo-environmental evaluation for urban land-use planning: A case study”, Engineering Geology, vol. 61, pp.257-271.

Dalvand, A. (2010). “Study of base geomechanical properties of Eyvashand dam, khorramabad north Iran,” Indian Journal of Fundamental and Applied Life Sciences ISSN: 2231-6345, 257-267.

Davies, M. P., McRoberts, E.C., Martin, T. E. (2002). “Static liquefaction of taliling dams- A review of the basics”. In proceeding of tailings and mine waste. Fort Collins Balkema Publishers. January. Pp:3-15.

Debarghya Chakraborty, D. C. (2009). Investigation of the Behavior of Tailings Earthen Dam Under Seismic Conditions. American J. of Engineering and Applied Sciences, 559-564.

DSI, (2002), “Emet Yeni Borik Asit Tesisi Atik Baraji-Kesin Proje Mühendislik Jeolojisi Revize Raporu” (yayınlanmamış)

EM 1110., (1952), “Soil Mechanics Design-Seepage Control” Civil Works Construction, Part CXIX, Chapter 1 (1110-2-1901).

EM 1110., (1970), “Engineering and Design- Stability of Earth and Rockfill Dams” Engineering and Design Manual, US Army Corps of Engineers.

EPA. (1994). “Technical report design and evaluation of tailing dams”. U.S environmental protection agency office of solid waste special waste branch. 401M Street. SW Washington. DC20460.

Erdik,M.,DoyruranV.,Gülkan, P. ve Akkaş, N., (1985), “Türkiye'de deprem tehlikesinin istatisiki açıdan değerlendirilmesi”, Orta Doğu Teknik Ünıversitesi Deprem Mühendisli ği Araştırma Merkeziö 116 s.

FEMA, 2005. Federal Guideline for Dam Safety: Earthquake Analyses and Design of Dams. Fell, R., McGregor, P. and Stapledon, D. (1992), “Geotechnical Engineering of Embankment Dams”, Balkema, 675 p.

Fell, Robin P. M. (2005). “Geotechnical Engineering of Dams”. LONDON: A.A. BALKEMA PUBLISHERS.

Franks, DM, Boger, DV, Côte, CM, Mulligan, DR. (2011). “Sustainable Development Principles for the Disposal of Mining and Mineral Processing Wastes”. Resources Policy. Vol. 36. No. 2. pp 114-122.

Fraser, W.A. and Howard, J.K., (2002), “Guidelines for use of the consequence-hazard matrix and selection of ground motion parameters”, technical publication, Department of Water Resource, Division of Safety of Dam, 7 p.

Frelund, D. G., Krahn. J. (1997). “Comparison of slope stability methods of analysis”. Canadian Geotechnical Journal. 14(3). pp:429-439.

Germanov, T. (2007). Geotechnical properties of industrial waste deposits in Bulgaria. Man Made deposits , 95-100.

Ghafoori, G. R. (2011). Investigation of the Geological and Geotechnical Characteristics of Daroongar Dam, Northeast Iran. Geotechnical and Geological International Journal.

Google Earth v.2011.

Griffith, D., & Lane P. (1999). “Slope Stability by Finite Elements”. Geotechnique, 49(3), 387-403

Gutenberg, B., Richter, C.F., (1942). “Earthquake magnitude intensity energy and acceleration”. Bulletin of the Seismological Society of America 32 (3) (July).

Gülkan, P. and Kalkan, E., (2002), “Attenuation modeling of recent earthquake in Turkey”, Journal of Seismology, 6, No. 3, 397-409.

Helvaci, C., (2002), “the boron isotope geochemistry of the Kirka borate deposits”, western Turkey: Geochimica et Cosmochimica Acta, v. 59, p. 3599–3605.

Helvaci, C., (1986), “Geology, mineralogy, and geochemistry of the borate deposits and associated rocks at the Emet valley”, Turkey: Unpubl. Ph.D. thesis, University of Nottingham, UK, 338 p.

Hammah, R., Curran, J., Yacoub, T., & Corkum, B. (2004). “Stability Analysis of Rock Slopes using the Finite Element Method”. Proceesings of the ISRM Regional Symposium EUROCK 2004 and the 53rd Geomechanics Colloquy. Slazburg.

Hammah, R., Yacoub, T., Corkum, B., & Curran , J. (n.d). “A Comparison of Finite Element Slope Stability Analysis with Conventional Limit-Equilibrium Investigation”. Toronto, Canada: Rocscience Inc.

ICOLD, (1989), “Selecting seismic parameters for large dams-guidelines and recommendations: ICOLD committee on seismic Aspects of Dams Design”, Bulletin 72.

ICOLD, (1996). “A guide to tailing dams and impoundments-design, construction”, use and rehabilitation. Bulletin no(106).

ICOLD, (2001). “Tailing dams – risk of dangerous occurrences”. Lessons learnt from practical experiences. Bulltin 121. ISSN 0534-8293.

ICOLD, (2004), “ Working group on guidelines for the seismic assessment of dams-Final report (edited by N.Reilly)”, ICOLD Europen Club.

Janbu, N., (1973), “ Slope Stability Computation”. Embankment Dam Engineering, Casagrande Memorial Volume, Wiley, New York, 47-86.

Jantzer, A. B. (2005). Material properties of Tailings from Swedish mines . 8.

Jiminez, M.J, Giadini, D. And Grunthal, G., (2001), “Unified Seismic Hazard Modeling Throughout the Mediterranean Region” Bolettino di Geofisica Teorica ed Applicata, Vol.42, N.1-2, Mar-Jun., 3-18.

JMS, (2013).-Jeoloji Mühendislik Sondaj- “Rehabilation of Hisarcik and Espey Tailing Dams ”. Edited by EtiMine general Management, May.2013.

Khaled, S. M. (2012). Geotechnical Investigation of Uranium Tailings. Saskatchewan : University of Regina .

Kramer, S.L., (1996), “Geotechnical earthquake engineering”, Prentice Hall, New Jersey, 653 p.

Kutzner, C. (1997). “Earth and Rockfill Dams:Principles for Design and construction”. Hofheim, Germany: CRC Press.

Ladd, C.C. (1991). “Stability evaluation during staged construction”. J. Geotech. Engrg. Div., ASCE, 117(4): 540-615.

Ladd, C.C. and Foott, R. (1994). “A new design procedure for stability of soft clays”. J. Geotech. Engrg. Div., ASCE, 100(7): 763-786.

Lade, P. V., Liggio, C. D., Yamamuro, J. A. (1998). “Effects of non-plastic fines on minimum and maximum void rations of sand”. Geotechnical Testing Journal. Vol.21 (4). December.pp:336-347.

Law, K. T., and Refahi, K., Lam, T., Ko, P., and Hassan, P., (2005), “Seismic deformation of Waba Dam” CDA Annual Conference, Calgary.

Liang a, O. N. (1998). A reliability based approach for evaluating the slope stability of embankment dams. Engineering Geology 54, 271–285.

Liang, J. Z. (2010). Seismic Evaluation of Tailing Storage Facility. Australian Earthquake Engineering Society, 7.

Makadisi, F.I., and Seed, H.B. (1978), “Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformation” Journal of Geotechnical Engineering, ASCE, Vol.104, N.GT7, 849-867.

MIAC. (2013). “Tailings storage facilities in Western Australia”. Resources safety and environmental affairs, 24.

Mihai, S. D. (2008). Tailings Dams and Waste-rock Dumps Safety Assessment using 3D . International Association for Computer Methods and Advances in Geomechanics (IACMAG), India.

Mosquera, T. H. (2000). “COMPARATIVE STABILITY ANALYSIS OF TAILINGS STORAGE FACILITIES”. McGill University , 11.

Morgenstren, N.R., and Price, V.E., (1965), “The Analysis of Stability of general Slip Surface”. Geotechnique, Vol.15, No.1.

ORAL, Y. Z. (2010). DEFORMATION BEHAVIOUR OF A CLAY CORED ROCKFILL DAM IN TURKEY. MSc Thesis in METU, 116.

Ormann, Linda M. A. (2011). Numerical Analysis of Curved Embankment of an Upstream Tailings Dam. EJGE, 14.

Paul, D. K. (2000). “SEISMIC SAFETY ANALYSIS OF A HIGH ROCK-FILL DAM SUBJECTED TO SEVERE EARTHQUAKE MOTION”. 12WCEE, 8.

Quille, M. E., & O'Kelly, a. B. (2010). Geotechnical Properties of Zinc/Lead Mine Tailings from Tara Mines, Ireland. Geoenvironmental Engineering and Geotechnics, pp. 111-117.

Rico M, Benito G, Salgueiro AR, Diez-Herrero A, Pereira HG (2008b) “Reported tailings dam failures- A review of the European incidents in the worldwide context”. Journal of Hazardous Materials 154: 846–852.

Ritcey, G. M, (1989), “Tailings management: Problems and solutions in the mining industry”, Elsevier, ISBN 0444873740, 970 p.

Robertson, A. M. (November 2001). Geotechnical Stability Considerations in the designand reclamation of Tailing Impoundments. Geotechnical stability in surface mining symposium , 12.

Robertson, A. MacG., Fisher, J.W. and Van Zyl, 0. (1982), "The Production and Handling of Dry Uranium and Other Tailings". Procedure on Uranium Mill Tailings Management, Fort Collins, Colorado, USA.

Rockwitz, R., (2000), “Reviewing probabilistic Soils modeling”, Computers and Geotechnics, Vol.26, pp. 199-223.

Rocscience Inc. (2001-2004). “Application of the Finite Element Method to Slope Stability”. Toronto, Ontario, Canada.

Rocscience Inc. (2005). “Phase 2 Version 6.0 Finite Element analysis for Excaviations and Slopes”. Toronto, Ontario, Canada.

SAYIT, E. (2012). “AN INVESTIGATION OF GEOTECHNICAL CHARACTERISTICS AND STABILITY OF A TAILINGS DAM”. Ankara: METU.

Sharma, S., (1996), “Slope Stability and Stability Methods” England, 620 p.

Singh, B., & Varshney, R. (1995). Engineering for Embankment Dams, Rotterdam: A.A Balkema.

Slide, (2004), “2D limit-equilibrium Analysis”, Rockscience, Toronto, Canada.

Smith, P. (2010). “The Research on the Stability of the Fine Grained Soil Tailing Dam” . OISUD, 90-97.

Song. A (2012). Seismic Induced Slope Displacement of a Tailings Embankment – Comparison of Numerical and Simplified Analysis Methods . Knight Piesold, 10.

Sonmez, H., Ulusay, R., (1998). A Practical Procedure for the Back Analysis of Slope Failures in Closely Jointed Rock Masses. Int. J. Rock Mech. Min. Sci. Vol. 35, No. 2, pp. 219±233.

Sowers, G. F. (1979). “Introductory soil mechanics and foundations”. Macmillan.

Spencer, E., (1967), “A method of analysis assuming parallel interslices technique”, Denver, 860 p.

Stanciucu, M. (2005). Evaluation of waste embankment slope stability: Valea Manastirii, Gorj, Romania. Bull Eng Geol, 22.

Tavenas, F. and Leroueil, S. (1980). “The behaviour of embankments on clay foundations”. Canadian Geotech. J., 17(2): 236-260.

Tavenas, F., Mieussens, C. and Bourges, F. (1979). “Lateral displacements in clay foundations under embankments”. Canadian Geotechnical J., 16(3): 532-550.

Tosun, H., (2013), Hisarcik Tailing Dam General Report, Eti Mine General Management, Ankara, 285 p.

Tosun, H., Batmaz, S., (2007), “Describe the evaluation of design principles adopted for CFRDs in Turkey and discuss problems encountered during construction”, International Water Power & Dam Construction, May 2007

Tosun, H., N.A. Abu-Zaid, (2002) “Pade Approximation for Extended Boundary Condition Method in Electromagnetic Scattering Analysis”,, Microwave and Optical Technology Letters, Vol. 32, NO. 2, , pp:100-101

U.S. Army Corps. Of Engineers. (2003). “Engineering and Design, Slope Stability (Vol. Engineer Manual)”. Washington, DC: Department of Army Corps. Of Engineers.

USBR, (1987a), “Design of Small Dams”, United States Bureau of Reclamation, Third Edition, Denver, 860 p.

USBR 1987b, “Seepage Analysis and Control”, Embankment Dams-Design Standards: 13 Bureau of Reclamation, Denver.

USBR, (1992), “Static Deformation Analysis”, United States Bureau of Reclamation, Third Edition, Denver, 329 p.

United States committee on large dams (USCOLD). (1996).”Tailing Dam Incidents”. Denver.Co.

U.S. Environmental Protection Agency. (1994). “Design and evaluation of Tailings Dams”. Special Waste Branch. Washington, DC: U.S. Environmental Protection Agency Office of Solid Waste.

United Nations environmental program (UNEP). (1996). “Tailing Dams Incidents 1980-1996”. Mining Journal Ltd. London.

Ulusay, R., Tuncay, E., Sönmez, H., and Gökçeoğlu, C., (2004). “An attenuation relationship based on Turkish strong motion data and iso-acceleration map of Turkey”. Engineering Geology, 74, 265-291.

Vick, S. G. (1990). “Planning design and analysis of tailing dams”. BiTech Publisher Ltd. Vancouver. Canada. 1990. ISBN:0-921095-12-0.

Vick, S. G. (1983). “Planning design and analysis of tailing dams”. Wiley & Sons. New York. Pp:369. ISBN: 0-471-89829-5.

Villavicencio A, P. B. (nd). Estimation of the variability of tailings dams properties in order to perform probabilistic assessment. Revue, 12-20.

Wei Z, Yin G, Li G, Wang JG, Wan L, Shen L (2008) “Reinforced terraced fields method for fine tailings disposal”. Mineral Engineering 22: 1053–1059.

Wieland, M. & Malla, S., (2005).” Seismic evaluation of a 117 m high embankment dam resting on a thick soil layer”. Proc. 12th European Conf. on Earthquake Engineering, London, UK, Paper 128.

Wyllie, D.C. & Mah, C. (2004). “*Rock slope engineering: civil and mining”*. 4th edition, New York: Spon Press.

Zardari, M. A. (2011). “Stability of Tailing Dams, Focus on Numerical Modeling”. Luleå, Sweden: Luleå University of Technology, Division of Mining and Geotechnical Engineering.

Zeinkiewicz, O.C., Taylor, R.L., (1991), “Finte Element Method”, v.2, Mc.Graw Hill.

APPENDIX A

ROCK SLOPE STABILITY ANLYSIS USING SLIDE v.5.0

According to the reported map (JMS, 2014) the rock fill slope has a 45° steepness and it could be considered as a possible instable part of the dam body. The rock fill slope stability is analyzed by utilizing the SLIDE v.5.0 by applying two different failure criteria; (i) Mohr-Coulomb and (ii) Hoek-Brown.

It should be mentioned that the proper parameters for Hoek-Brown criterion is obtained from the literature review. Table A.1 contains the results of the surveys.

Table A. 1 Rock fill material parameters by considering the Hoek-Brown failure criterion (Sonmez. H , Ulusay.R (1998))

PROPERTY	MEAN	Std.dev.	Min	Max
UCS (kPa)	10000	2500	1000	20000
m_b	0.68	0.18	0.0086	1.44
s	0.0002	0.00007	0.00008	0.0007

The required input data and all the aspects of the dam model are the same as what is employed for the PHASE 2 v.8.0.1 software and actually are exported from it. So, the applicable data for the SLIDE v.5.0 are obtained from Figure 5.1 for geometry and Table 4.5. The slope stability of rock fill by considering the two different failure criteria are obtained due to the failure surfaces and the relative factor of safety.

(i) Mohr-Coulomb

By applying the material properties which are required in Mohr-Coulomb method the slope stability analysis are as in Figure A.1.

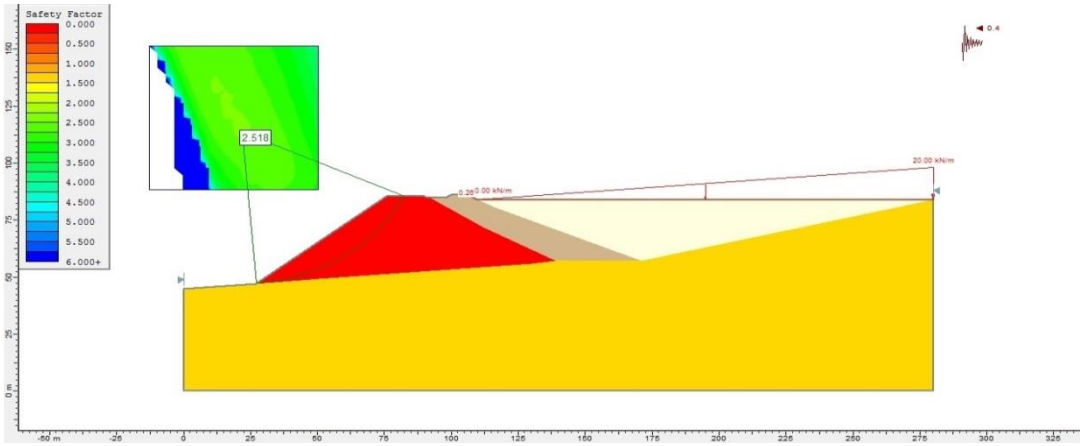

Figure A. 1 Rock fill slope probable failure surface and safety factor (Considering the Mohr-Coulomb failure criterion)

(ii) Hoek-Brown

By applying the material properties which are required in Hoek-Brown method the slope stability analysis are as in Figure A.2.

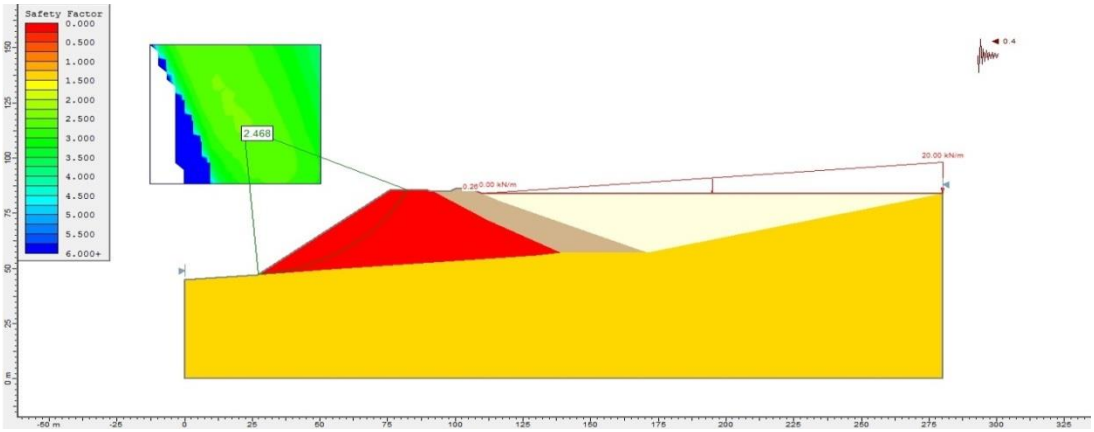

Figure A. 2 Rock fill slope probable failure surface and safety factor factor (Considering the Hoek-Brown failure criterion)

As it can be seen, the difference between the applied failure criteria and their material properties made the difference between two amounts of safety factors. The amount of the factor of safety in terms of Mohr-Coulomb failure criterion is about 2.51 and 2.46

for Hoek-Brown failure criterion. However, the obtained amount represents that the PHASE 2 results were satisfactory.

APPENDIX B

TAILINGS MATERIAL PROPERTIES EVALUATION

The material properties related to the tailings are also obtained from the reported data by Tosun (2013). For evaluation of the accuracy of the material properties, literature surveys are performed. The obtained data is mentioned in Table B.1.

Table B. 1 Tailing material properties according to the literatures

No	REFERENCE	Unit Weight (kN/m3)	E (Mpa)	ϕ (°)	C (kPa)	DESCRIPTION
No.1	Jantzer, 2005	14	20	19.4	2.9	Fine - Homogeneous
No.2	Robertson, 2001	12	25	18.2	1.1	_
No.3	Zhang, 1983	17	18	23.7	0	soild like Material
No.4	Jantzer, 2005	13	26	23.9	0	Sample from Beach of embankment
No.5	Germanov, 2007	15	20	21	0.65	Fine sandy soil
No.6	SAYIT, 2012	14	17	20	0	Fine
No.7	Quille and O'Kelly, 2010	16	20	21.4	0.1	Zinc/Lead Mine Tailings
No.8	Villavicencio, nd	12	19	17.2	0.344	_
No.9	Khaled, 2012	11	14	15	0.7	Uramium Tailing
No.10	Mendes 2013	15	20	20	0	Fine and plastic
No.11	Cutting, 2007	14	21	23	1	_
No.12	MIAC, 2013	10	22	20	0	

Based on the literature surveys in Table B.1, the statistical layout data about tailing material properties are as presented in Table B.2.

Table B. 2 Summary statistics for tailing material properties.

Material	Parameter	Mean	Standard Deviation	Range	Min	Max	Coefficient of Variation
Tailing	Unit Weight (kN/m3)	13.5	2.06	6	10	16	0.15
	Elastic Modulus (Mpa)	20.1	3.24	12	14	26	0.16
	Internal Friction Angle (°)	20.4	3.1	11.7	15	26.7	0.15
	Cohesion (kPa)	0.5	0.1	2.9	0	2.9	0.2

By considering Table 4.4, which represents the reported tailing material and the obtained results from literature reviews, the reported data are reliable and they can be used as an input parameter for tailing material in the analysis as it is done.