DEFORMATION BEHAVIOUR OF A CLAY CORED ROCKFILL DAM IN TURKEY

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

DECEMBER 2010

Approval of the thesis:

DEFORMATION BEHAVIOUR OF A CLAY CORED ROCKFILL DAM IN TURKEY

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ABSTRACT

DEFORMATION BEHAVIOR OF A CLAY CORED ROCKFILL DAM IN TURKEY

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December 2010,97pages

In this study, Bahçelik Dam, which is located in Kayseri Province, is investigated by means of horizontal movement due to reservoir loading and seepage inside the core and body. Two dimensional plain strain finite element analyses are carried out in order to find total stresses, displacements and pore water pressures. Mohr-coulomb soil model is used to represent elastic behavior of rock-fill material. Since there is no information about material used in dam body, material parameters are determined by sensitivity analyses being in the range of data acquired from literature survey. Calculated displacement and pore water pressures are compared to the data taken from field survey on actual dam body. As a conclusion remark, it is beleived that the horizontal displacement behaviour of two systems, such as real dam and computer modelling, would not match excatly since the materials used in real dam body would behave as plastic whereas that used in computer modelling assumed to be elastic.

Keywords: Rockfill dam, Bahçelik dam, clay cored dam, finite element, deformation, seismic performance

ÖZ

TÜRKİYEDE BULUNAN KİL MERKEZLİ KAYA DOLGU BİR BARAJIN DEFORMASYON DAVRANIŞI

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Aralık 2010,97 sayfa

Bu çalışmada, Kayseri ili içerisinde bulunan Bahçelik Barajı, rezervuar etkisinde oluşan yatay deplasman ve baraj gövdesinde ve çekirdeğinde oluşan sızıntılar yönünden incelenmiştir. Toplam gerilme, deplasman ve boşluk suyu basıncını temin etmek için iki boyutlu düzlem şekil değiştirme metodu kullanılarak sonlu elemanlar metodu analizi yapılmıştır. Kaya dolgunun elastik yapısını temsil etmesi için Mohr-Coulomb zemin modeli kullanılmıştır. Baraj gövdesinde kullanılan malzemelerle ilgili bir bilgiye sahip olunmadığından, literatür araştırmasında elde edilen sınırlar içinde hassaslık analizleri yapılarak malzeme parametreleri belirlenmiştir. Saha incelemesinden elde edilmiş olan deplasman ve boşluk suyu basıncı değerleri sayısal modelden elde edilen değerler ile karşılaştırılmıştır. Sonuç olarak, gerçek baraj gövdesinde kullanılan malzemeler ile sayısal modellemede kullanılan malzemeler plastik ve elastik olarak iki farklı davranış sergileyeceğinden, analiz sonucunda çıkacak olan yatay deplasman eğrileri tam olarak çakışmayacağı düşünülmektedir.

Anahtar Kelimeler: Kaya dolgu baraj, Bahçelik barajı,Kil çekirdekli baraj, sonlu elemanlar, deformasyon, sismik performans

To my wife and my family

ACKNOWLEDGEMENTS

I would like to express my sincere thanks to my supervisor Asst. Prof. Dr. Nejan Huvaj Sarıhan for her constant encouragement and support throughout my study.

I wish to thank Prof. Dr. Kemal Önder Çetin for his invaluable suggestions.

I wish to give my immeasurable thanks to my partner in life, Sevinç Ünsal Oral for her continuous support during my studies. Her infinite love and infinite insistence was certainly a key to my success.

I would like to thank to my parents Nesrin and Lütfi Özden Oral, and my brother Murat Anıl Oral. Their unconditional love, continued encouragement and belief in my abilities nurtured me throughout this process. Essentially words are not enough to thank my family for their endless support.

Finally, I would like to thank all my friends, for their friendship and fellowship in nearby or kilometers away. I feel lucky to have their friendship.

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LIST OF ABBREVIATIONS

$\mathbf{a_{max}}$	Peak Horizontal Ground Surface Acceleration
g	Acceleration of Gravity
σ_v	Total Vertical Stress
σ'_v	Effective Vertical Stress
FS	Factor of Safety
E	Elastic Modulus
ν	Poisson's Ratio
γ	Shear Strain
σ	Normal Stress
k	Seismic Coefficient

CHAPTER 1

INTRODUCTION

1.1 Background and Motivation

Dams are gaining more attention in recent years due to the rise of the environmental awareness and “renewable energy” and “sustainability” concepts. Earth embankment dams are preferred for their ease of construction and relative economical advantage over concrete gravity dams.

Rockfill dam is a type of earth dam where a compacted central clay core is supported on the upstream and downstream sides by compacted rockfill material. Rockfill dams are preferred in areas where abundant quarried or processed rockfill material is available for construction. In recent years rockfill dams, especially the impervious-faced rockfill dams (IFRD), are being built all around the world using asphalt or concrete as the impervious material in the upstream face of the dam. It has been frequently reported in the literature that cracks develop in the impervious face of these dams, causing seepage and instability problems. Such observed deformations led researchers to further study the deformation behavior of rockfill material.

Laboratory testing of rockfill material is very difficult because of the large size of particles. Instead, the stress-strain behavior can be studied through observed deformations in rockfill dams. Also, deformation of, for example, clay-cored rockfill dams can provide an approximate upper limit to the expected deformation in impervious-faced rockfill dams.

Some empirical guidelines have been proposed in the literature to estimate the crest settlement of a rockfill dam. However these empirical relations can lead to very large errors since they are not considering the construction stages, or rockfill material type etc. (Clements, 1984). It is useful to broadly define the "normal/expected" deformation behavior of rockfill dams in terms of magnitude, rate or trend. This would provide some guidance to identify potentially "abnormal" deformation behavior, indicating marginal stability, or instability. Observing real deformations in the dam body or calculating deformations for different possible conditions can provide an early warning, so that remedial actions can be taken in time, for example by lowering the reservoir level.

In this thesis, firstly, types of dams and instrumentation equipment will be discussed. Then a 65-m-high rockfill dam, Bahçelik Dam, which is constructed in Kayseri province, will be analyzed by using 2D plane strain finite element method. The measured pore pressures, horizontal and vertical displacements will be compared with pore pressures and displacements obtained from numerical analyses. Lastly, the behavior of the dam under seismic forces will be evaluated.

1.2 Objective of The Study

The main objective of this study is to compare the deformations obtained from finite element modelling of a rockfill dam with real measured values. Within the confines of this thesis, a simple material model will be preferred, since it requires less number of input parameters to be determined and inputted, as compared to more sophisticated material models. By this, the validity, accuracy and adequeacy of the simple material model can be checked. The results of this study could be useful in: (1) simpler prediction of future deformations of rockfill dams and, consequently, of their safety; and (2) improved design criteria for future rockfill dams (freeboard evaluation, limiting permissable deformations etc.), (3) determination of the most efficient location of instrumentation.

1.3 Thesis Organization and Scope

This thesis is composed of six main sections. Contents of each chapter are summarized as follows:

In the first chapter, the research statement and introductory comments are presented.

Chapter 2 gives a general literature review for the embankment dams, their history, typical deformation behavior, instrumentation on dams including types of instrumentation and some background on the seismic design of embankment dams.

Chapter 3 provides information on the location and properties of the selected rockfill dam, namely Bahçelik Dam in Kayseri. The surrounding topography, available information about the instrumentation on Bahçelik Dam, its relation to seismicity of Kayseri province and steps followed in the seismic analysis of the dam are described.

Chapter 4 explains the analyses procedure of Bahçelik Dam including finite element modeling and seismic design.

Chapter 5 describes the results obtained from the analyses. The horizontal deformations as well as pore water pressures calculated by finite element modeling are compared with the measured data from field instrumentation. The results of the seismic stability analyses of the dam together with expected deformations are presented.

Chapter 6 summarizes the research findings and presents concluding remarks.

Finally in the appendix all necessary information and detailed results of the analyses are given.

CHAPTER 2

LITERATURE SURVEY

The first earth fill dam known to have been built was called Nimrod's Dam and it was built in Mesopotamia, north of Baghdad across the Tigris around 2000 BC. The construction purpose of the dam was to divert the flow in the river to reduce the threat of flooding and to help irrigate the crops. The dam was built of earth and wood.

The first steps were taken for modern day rock fill dam construction in California during the mining gold rush era about 150 years ago. Drill and blast mining techniques by miners provided an abundant supply of rock materials for use in dam construction. Gold mining in the 1850's also required a large and steady supply of water for sluicing and extracting the heavier gold nuggets from alluvial placer deposits. The miners used the rock quarry materials to construct water storage dams in remote areas by hand or with available mine haul and dump equipment.

A historical summary of the use of rockfill in embankment design and construction was presented by Hunter and Fell (2003) in Table 1.

Table 1: Historical summary of rockfill usage in embankment design (Hunter and Fell 2003)

Approximate Time Period	Method of Placement and Characteristics of Rockfill	Comments
Concrete Face Rockfill Dams		
Mid to late 1800's to early 1900's	Dumped rockfill with timber facing	Early embankments constructed with timber facing. Typically of very steep slopes (up to 0.5 to 0.75H to 1V). First usage of concrete facing in the 1890's. Height limited to about 25 m.
1920's to 1930's	Main rockfill zone dumped in high lifts (up to 20 to 50 m) and sluiced, although the sluicing was relatively ineffective. A hand or derrick placed rockfill zone was used upstream.	Rockfill typically sound and not subject to disintegration. Dam heights reaching 80 to 100 m. For high dams, cracking of the facing slab and joint openings resulted in high leakage rates (2700 l/sec Dix River, 3600 l/sec Cogswell, 570 l/sec Salt Springs).
Late 1930's to 1960's	High pressure sluicing used for the main rockfill zone. Rockfill still very coarse.	Cracking of face slab, particularly at the perimeter joint, and high leakage rates a significant issue with higher dams (3100 l/sec at Wishon, 1300 l/sec at Courtright).
From late 1960's	Rockfill placed in 1 to 2 m lifts, watered and compacted. Reduction in particle size. Usage of gravels and lower strength rock.	Significant reduction in post-construction deformations due to low compressibility of compacted rockfill. Significant reduction in leakage rates; maximum rates typically less than 50 to 100 l/sec. Continued improvement in plinth design and facing details to reduce cracking and leakage.
Earth and Rockfill Dams		
1900 to 1930	Dumped rockfill	Use of concrete cores with dumped rockfill shoulders at angle of repose. Limited use of earth cores. Dam heights up to 50 to 70m.
1930's to 1960's	Earth core (sloping and central) with dumped rockfill shoulders.	Use of earth cores significant from the 1940's due to the difficulties with leakage of CFRD. Increasing dam heights up to 150 m.
From 1960's	Use of compacted rockfill. Typically placed in 1 to 2 m lifts, watered and compacted with rollers.	Improvements in compaction techniques. Early dams compacted in relatively thick layers with small rollers. Gradual increase in roller size and reduction in layer thickness reduced the compressibility of the rockfill. Significant increase in dam heights in the mid to late 1970's, up to 250 to 300 m.

Nowadays, still the most preferable dam type is clay core earth fill dam because of the easiness of construction and the easiness of obtaining construction material. In earth fill dams either the material of the excavated area may be used or the required amount of soil may be transported from the closest deposit area.

Earth dams are massive dams similar to gravity dams except that they are made of soil. The dam is made watertight, with a core wall and filled with an impervious center usually made of clays.

According to International Commission of Large Dams (ICOLD), a rock fill dam is an embankment type of dam, which depends primarily on rock material for its stability. As rock fill dams must contain an impervious zone comprising a substantial volume of the dam - the term Rock fill dam usually represents a dam that contains more than 50% of compacted or dumped pervious fill. The dam is dependent for water tightness on an impervious upstream blanket or an impervious core.

2.1 Definition of Embankment Dams

According to the predominant fill material used, the embankment dams are divided into two groups as earth and rock-fill dams. If the local borrow materials are not so adequate, earth dams with impervious cores are constructed. Instead of using inclined upstream vertical core, using impervious core at the center of the dam is much more desirable since the contact pressure between the core and foundation is higher than the previous. So, it will help to prevent leakage and provide greater stability to earthquake loading.

An earth dam is constructed using suitable soils such as sand, gravel, clay etc., obtained from mining areas by transporting them to site area or using the material after excavation of dam area. The materials are compacted in layers by mechanical machines such as tamping rollers, sheepfoot rollers, heavy pneumatic tired rollers, vibratory rollers, etc.

If a dam is composed of mainly fragmented rock with an impervious core, it is called a rock-fill dam. Mostly, impervious core is separated from the main rock shells by several transition zones built of properly graded material. Similar to earth dams, rock-fill zones are compacted in layer thicknesses of about 30 to 60 cm by mechanical compactor machines.

In the construction of a rock-fill dam, a wide range of materials can be used from sound, free draining rock to the more friable materials such as sandstone and silt shale materials. The friable materials are better for filling the gaps to provide better compaction but since the shear strength of these materials are not as high as sound rock fill; the stability design of the slope should be studied carefully.

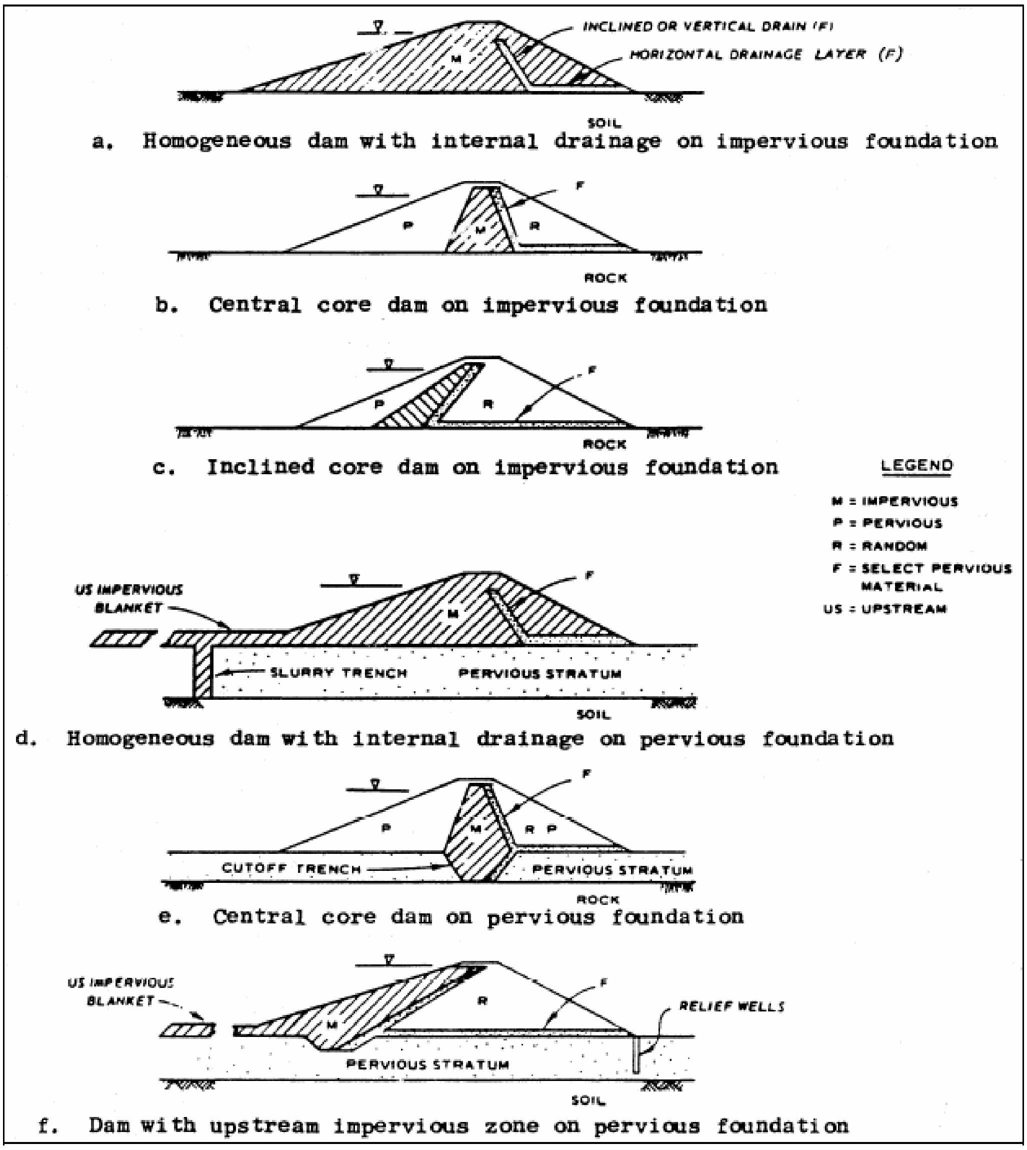

Figure 1: Typical cross sections of earth-fill dams (www.theconstructor.org)

2.2 Selection of Embankment Type

2.2.1 General

An earth dam or a rock-fill dam, rather than a concrete dam, may be preferred for the following conditions; a wide stream valley, lack of strong abutments, considerable depths of soil overlying bedrock, poor quality bedrock from a structural point of view and existence of a sufficient capacity for a spillway.

2.2.2 Topography

Topography is the main element which effects the selection of type of dam. If the site is in a V-shaped valley with strong sound rock abutments, an arch dam would be a perfect choice. If there is a relatively narrow valley with high, rocky walls, the dam type would be a rock-fill or concrete dam. However, a wide valley with a deep overburden would suggest definitely an earth dam. Also, composite structures with partly concrete and partly earth may be used for irregular valleys (Golze 1977, Singh and Sharma 1976, Goldin and Rasskazov 1992).

2.2.3 Geology and Foundation Conditions

The geology and foundation condition is one of the main elements effecting the selection of suitable dam type for that site. The geology and foundation conditions at the dam site may dictate the type of dam suitable for that site. Because of its high shear strength and resistance to erosion and seepage, competent rock foundations develop some restrictions for the selection of dam type which would be built in that site. If it is well compacted, gravel foundations are good for earth or rock-fill dams. In order to provide seepage control and/or effective water cutoffs, special site improvements shall be performed. Also, the liquefaction potential of gravel foundations should be investigated (Sykora et al. 1992).

Silt or fine sand foundations are good for low concrete and earth dams but not for rock-fill dams. Settlement, prevention of piping, excessive percolation losses, and protection of the foundation at the downstream embankment toe from erosion are the main problems. Since it has low foundation shear strength, non-dispersive clay foundations may be used for earth dams with flat embankment slopes. Concrete and rock-fill dams are not suitable for silt or fine sand foundations because of the requirement of flat embankment slopes and tendency for large settlements (Golze 1977, Bureau of Reclamation 1984).

2.2.4 Materials Available

The availability of materials in a distance of hauling nearby the site of dam will affect the type and also cost of the dam. The material from excavating dam foundation, spillway, outlet works, powerhouses and other appurtenant structures would be used as soils for embankments, rocks for embankments and riprap and concrete aggregate (sand, gravel and crushed stone). Although, using the materials directly from excavation would be the most economic and cost-saving way, they can be stockpiled for later use. If suitable soils for an earth-fill dam can be found in nearby borrow pits, an earth dam may prove to be more economical. The availability of suitable rock may favor a rock-fill dam. The availability of suitable sand and gravel for concrete at a reasonable cost locally or onsite is favorable to use for a concrete dam (Golze 1977, Bureau of Reclamation 1984).

2.3 Instrumentation

2.3.1 Types of Instrumentation

Depending on the layout, the type of the project and the construction techniques that are used in the site, the type, quantity and location of the instrumentation tools may vary. Available instruments that may be used during or after construction can be listed as following: piezometers located in the foundation abutment and/or embankment, surface monuments, inclinometers, pressure cells, accelerographs (in areas of seismic activity), settlement plates within the embankment, movement indicators, strain indicators.

2.3.2 Discussion of Devices

2.3.2.1 Piezometers

Pore water pressure in the embankment, foundation and abutments is an item that affects the safety of a dam. Piezometer observations should be made in periodic times in order to get an idea about seepage conditions, effectiveness of seepage cutoff and the performance of drainage system.

In order to evaluate pore water pressures accurately in several cross sections, piezometers should be placed in several groups in vertical planes perpendicular to the axis of the dam. If the piezometers, which are placed at each cross section, should extend into the foundation and abutments, the measurements would be more realistic and useful. There are various types of piezometers that could be installed in a dam. A very simple Casagrande type piezometer is shown in Figure 2.

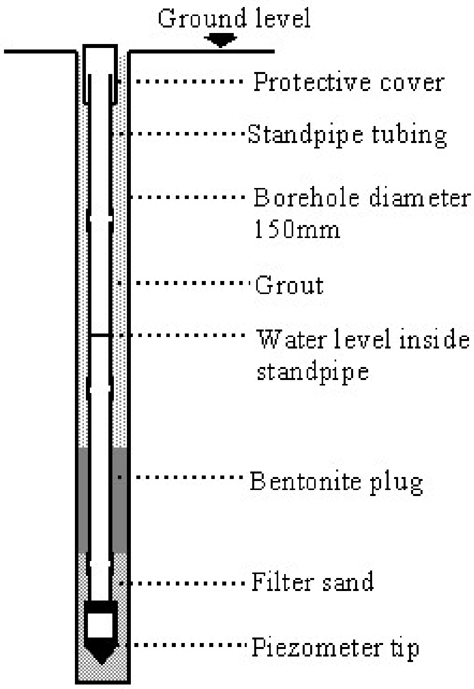

Figure 2: Diagram of borehole with a Casagrande piezometer www.canterbury.gov.uk)

2.3.2.2 Surface Monuments

The items which are located in the crest and at upstream and downstream slopes and used for measuring both vertical and horizontal movement are called surface monuments. By taking reference to a fixed offsite point that is stable/non-moving, the movements of the surface monuments should be measured in certain time periods. The surface monuments, which are composed of steel or brass rod, should be embedded in the crest or embankment so that it would not be affected by external weather conditions. All surface monuments should be protected from construction equipment.

The nominal horizontal spacing between surface monuments should be as follows: 15 m intervals for crest lengths up to 150 m, 30 m intervals for crest lengths up to 300 m and 60 m to 120 m intervals for longer embankments. In order to have data for a longer period of time and to be aware of any possible danger due to movements, surface monuments should be placed as early as possible after the completion of dam construction.

2.3.2.3 Inclinometers

Inclinometers are devices that are used for measuring the horizontal deformation at certain depth. These devices are frequently used in landslide studies to detect the depth of the slide plane. It is simply composed of a tube inserted in the ground and an inclinometer probe attached with a cable which is sent down the tube to measure any tilt in the tube with depth. These measurements are taken at certain time intervals to observe the deformations at different times. Inclinometers are used usually at high dams, dams on weak deformable foundations and dams composed at least in part of relatively wet, fine-grained soils. The embankment movements would be either parallel or perpendicular to the dam axis while the dams are constructed in deep and narrow valleys, thus, the inclinometers should be installed properly. Inclinometers should span the suspected zone of concern for movements. It is essential that these devices be installed and observed during construction as well as during the operational life of the project.

Figure 3: Inclinometer probe (www.gage-technique.com)

2.3.2.4 Pressure cells

Pressure cells (or earthcells) which are used to measure the total earth pressure inside the dam are the least common equipments. These devices are composed of two thin plates that are welded together, inside which is full of oil. Any change in the oil pressure is measured by a transducer attached to this system via a steel tubing. Although this equipment has been installed in many dams, much research has to be done to improve the success of measurement. Some pressure cell devices installed at the interface of concrete structures and earth-fill have performed very well.

Figure 4: Earth pressure cells (www.wetec.com.sg)

2.3.2.5 Accelerographs

In areas of seismic activity, in order to design important structures stronger for big earthquakes, accelerographs are used to record the data of strong ground motion. Dams are the most commonly used structures for recording data from earth movements. Although analog film-type accelerographs still exist they are being replaced more and more with digital accelerographs.

Measuring strong ground shaking, resulting from big earthquakes, is an essential tool for finding out the parameters of strong ground motion. This data is vital in understanding the high frequencies of seismogenic layers. Moreover, these measurements are primary tools used in developing experimental relationships of strong seismic properties. (http://www.bhrc.ac.ir)

Figure 5: Anaccelerograph (www.geonet.org.nz)

2.4 NEHRP (National Earthquake Hazards Reduction Program)

The National Earthquake Hazards Reduction Program (NEHRP), the purpose of which is, shortly, to reduce the risk from earthquakes on the buildings, has been founded in 1978 in the U.S.A, and is being managed by several governmental institutes such as FEMA, NIST, NSF and USGS.In this methodology, the earthquake motion at a given point on the ground surface can be represented by an elastic ground acceleration response spectrum. In the evaluation of seismic stability of earth and rockfill dams, the methodology suggested by NEHRP can be used to determine the elastic design spectrum parameters.

2.4.1 General Procedure

2.4.1.1 Site coefficients and adjusted acceleration parameters

S_{MS} and S_{M1} parameters, of which the maximum credible earthquake (MCE) spectral response acceleration, shall be determined as follows:

$$S_{MS} = F_a S_s$$

$$S_{M1} = F_v S_1$$

where F_a and F_v are defined from Table 2 and Table 3 respectively.

Table 2: Values of Site Coefficient F_a

Site Class	Mapped MCE Spectral Response Acceleration Parameter at 0.2 Second Period[a]				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	—[b]	—[b]	—[b]	—[b]	—[b]

[a] Use straight line interpolation for intermediate values of S_S.

[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 3: Values of Site Coefficient F_v

Site Class	Mapped MCE Spectral Response Acceleration Parameter at 1 Second Period[a]				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	—[b]	—[b]	—[b]	—[b]	—[b]

[a] Use straight line interpolation for intermediate values of S_1.

[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

2.4.1.2 Design Acceleration Parameters

Design acceleration parameters S_{DS} and S_{D1} shall be determined as follows:

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

2.4.1.3 Design Response Spectrum

The design response spectrum shall be developed as follows:

1. For periods less than or equal to T_0, S_a shall be taken as below:

$$S_a = 0.6 \frac{S_{DS}}{T_0} T + 0.4 S_{DS}$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, S_a shall be taken as equal to S_{DS},
3. For periods greater than T_S and less than or equal to T_L, S_a shall be takes as follows:

$$S_a = \frac{S_{D1}}{T}$$

4. For periods greater than T_L, S_a shall be taken as follows:

$$S_a = \frac{S_{D1} T_L}{T^2}$$

where:

S_{DS} = the design spectral response acceleration parameter at short periods

S_{D1} = the design spectral response acceleration parameter at 1 second period

T = the fundamental period of the structure (sec)

$T_0 = 0.2S_{D1}/S_{DS}$

$T_S = S_{D1}/S_{DS}$

T_L = Long-period transition period

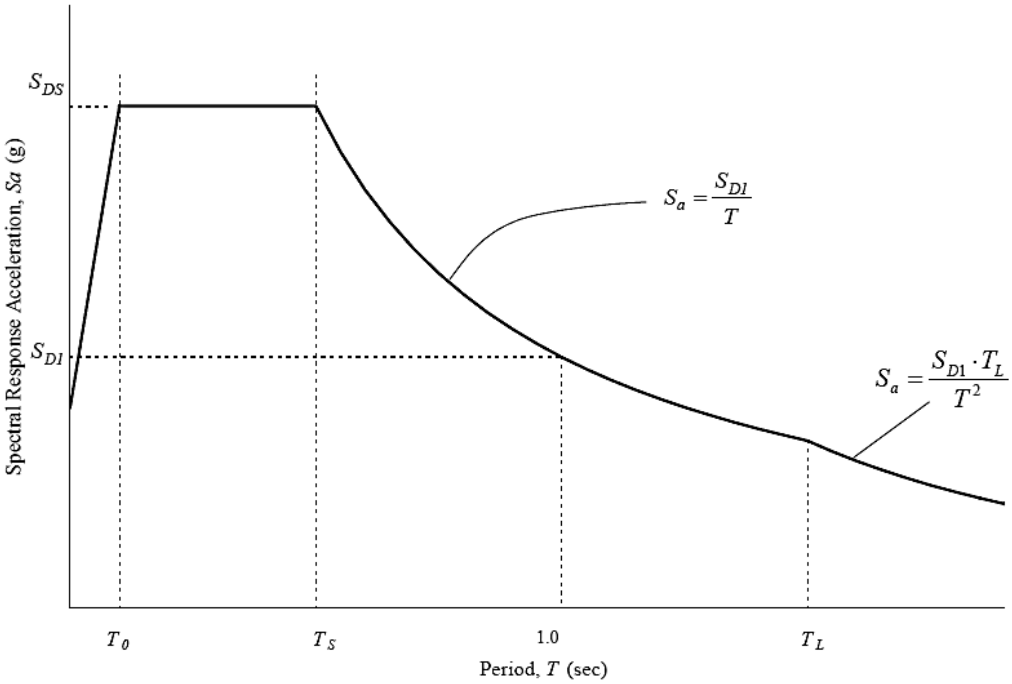

Figure 6: Design Response Spectrum

2.5 Pseudo-Static Analysis

Analyses of seismic slope stability problems using limit equilibrium methods in which the inertia forces due to earthquake shaking are represented by a constant horizontal force (equal to the weight of the potential sliding mass multiplied by a coefficient) are commonly referred to as pseudo-static analyses.

In recent years, U.S. Army Corps of Engineers have been pioneer for seismic design of new dams (which are generally considered to be among the more critical civil engineering facilities). The research includes using of a seismic coefficient of 0.1 in Seismic Zone 3 and 0.15 in Seismic Zone 4 by means of a minimum factor of safety of 1.0. But some, accepting the factor of safety 1.1 which is slightly more conservative requirement, the seismic coefficient is taken as 0.15. However, there should be an engineering judgment while using pseudo-static analyses cause of uncertainties involved in a particular analysis.

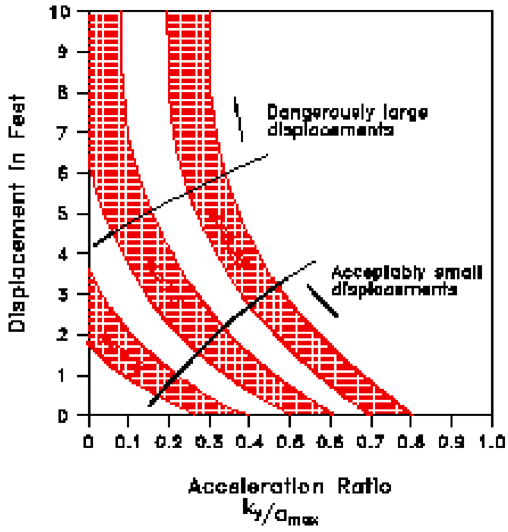

Figure 7: Typical Displacements Computed by Newmark Method (Seed, 1979)

The figure shows displacements computed by the Newmark method as a function of the acceleration ratio, k_y/a_{max}, where k_y is the critical seismic coefficient and $a_{max}$is the expected peak acceleration.

If a pseudo-static analysis using a seismic coefficient equal to one-half the peak acceleration yields a factor of safety greater than 1.0, the displacements are likely to be acceptably small. Similarly, for magnitude 7.5, 7.0, and 6.5, if the seismic coefficient is taken as one-third, one-forth and one-fifth of the expected peak acceleration, and the computed factor of safety is greater than 1.0, the displacements are likely to be acceptably small. The seismic coefficients obtained this way are shown as a function of peak acceleration and magnitude in Figure 8. (Robert Pyke, Consulting Engineer, Lafayette CA)

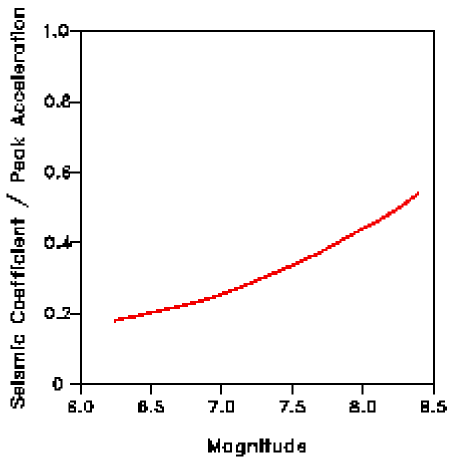

Figure 8: Curve for Obtaining Seismic Coefficient (Seed, 1979)

2.6 Deformation Behavior of Rockfill Dams

Rockfill dams are composed of material having particle sizes up to 1 m in diameter. Therefore it is very difficult to carry out laboratory shear strength tests on rockfill materials. Based on very limited laboratory triaxial test data available in the literature, it is concluded that rockfill material exhibit nonlinear, inelastic stress-strain behavior (Marsal, 1967; Marachi et al., 1972; Duncan et al., 1980, Saboya and Byrne, 1993) as can be seen in Figure 9. To represent this behavior Duncan and Chang's (1970) hyperbolic model is frequently used in the literature (Ozkuzukiran et al. 2006, Unsever 2007).

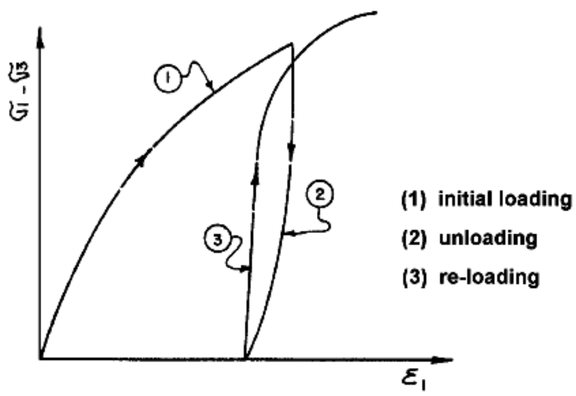

Figure 9: Typical stress-strain behavior of rockfill from a triaxial compression test (Mori and Pinto 1988).

Instead of laboratory tests, it is often more practical to look at the data collected from deformations observed in constructed rockfill dams. In this section, collected data in the literature on the vertical and lateral deformation of rockfill dams, and modulus of rockfill material will be reviewed.

Rockfill dams continue to deform long after their construction is completed, although at a decreasing rate. According to Hunter and Fell (2003) compressibility characteristics of rockfill are influenced by: degree of compaction of the rockfill, applied stress conditions and stress path, particle shape and particle size distribution, intact strength of the rock and the susceptibility of the rockfill to collapse upon wetting.

Clements (1984) studied the post-construction deformation behavior of rockfill dams by observing deformation data of 68 rock-fill dams. Settlement behavior of central core rockfill dam is shown below:

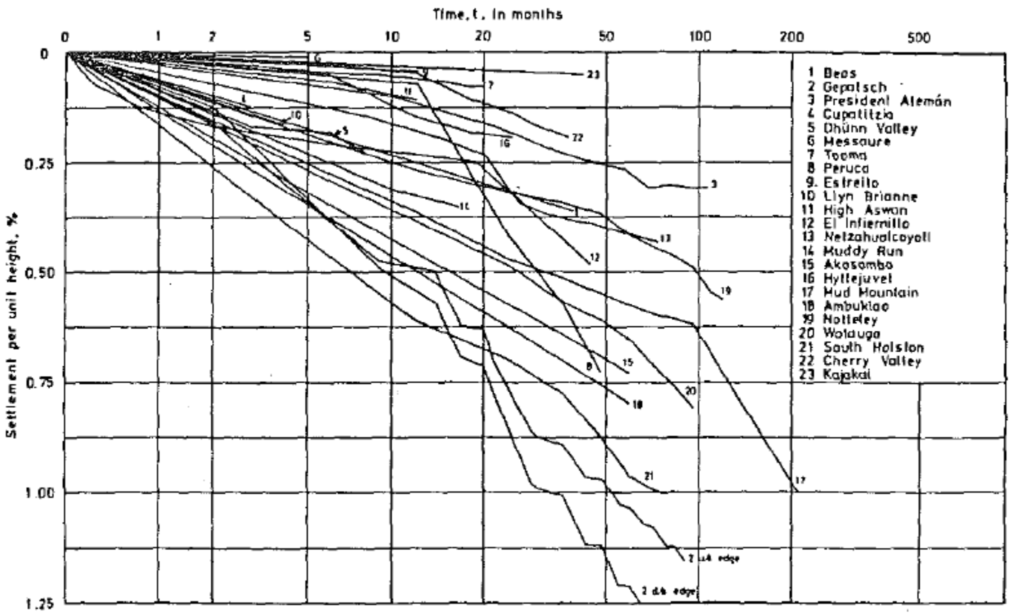

Figure 10: Crest Settlement of Central Core Dams (Clements 1984)

U.S. Bureau of Reclamation recommended, for the design of rockfill dams, a maximum crest settlement that is equal to 1%H (plus any deformation due to the settlement of the foundation), for rockfill dams with heights less than 15 m. It is noted in the literature that better compaction and sluicing decreases the crest settlements.

Lawton and Lester, by studying 11 dams which are built between 1925 and 1964, found that settlement can be expressed by an equation which is $S = 0.001H^{3/2}$. According to study, the horizontal deflection of the crest is about 50% of its settlement (Lawton 1964).

Sowers et al. (1965) found out that crest settlement of a rockfill dam equals to 0.25-1% of height by analyzing the behavior of 14 rockfill dams. Independent of dam height, cross section or the fill material type, their proposed correlation is $\Delta H = \alpha \cdot (\log t_2 - \log t_1)$ which gives the result as percentage of the height, between times t_1 and t_2 , where α is the rate of settlement changing from 0.2 to 1.05.

Soydemir and Kjaernsli (1979), for crest settlement of impervious-faced compacted rockfill dams, suggested $s = 10^{-4}.H^{3/2}$ in initial impounding, and three times this value after 10 years in service. Calculation of crest settlements using this simplified equation is found to overestimate the observed settlements on average by a factor of 3.2.

Hunter and Fell (2003) summarized the available empirical relations in the literature on the crest settlements due to first reservoir filling, as can be seen in Table 4

Table 4: Post-construction deformations reported in the literature (Hunter and Fell 2003)

Reference	Dam Type/s	Deformation Parameter	Range of Deformation (% of dam height) *1	Comments
ICOLD (1993)	"rockfill" *2	crest settlement crest displacement shoulder settlement	0.2 to 1.0% 0.1 to 0.5% 0.1 to 0.2%	Crest displacement up to 50% of the crest settlement.
Sowers et al (1965)	"rockfill" *2	crest settlement	0.25 to 1.0% (upper range for dumped RF)	14 dams, settlements up to 10 years after construction.
Clements (1984)	CFRD	crest settlement crest displacement	Up to 2.5% (dumped RF) 0 to 0.25% (compacted RF) 0 to 2.5%	Database of 68 dams.
	CCER	crest settlement crest displacement	0.05 to 1.25% -0.75% to 0.5%	
	Sloping core	crest settlement crest displacement	0.06 to 1.1% 0 to 0.6%	
Bernell (1958)	CCER with moraine core	crest settlement during first filling	0 to 0.2% (silty and sandy moraines) 0.05 to 0.3% (clayey moraines)	6 dams with moraine cores placed using the wet compaction method. Fine fractions less than 20%.
Dascal (1987)	"rockfill" dams (*2) with moraine cores	crest settlement downstream shoulder settlement crest displacement	< 0.35% (compacted RF) 0.3 to 0.55% (dumped RF) up to 0.7 to 0.8% ≥ crest settlement for compacted RF < crest settlement for dumped RF	15 Hydro Quebec dams and dikes on rock foundations
Sherard et al (1963)	"rockfill" *2	crest settlement	0.1 to 0.4% (for well constructed wetted RF)	Greater settlement for dumped RF.
	well constructed dams	crest displacement on first filling	< 25 to 50 mm	Greater displacements for dams with dumped RF.
Gould (1954)	rolled earthfill dams	crest settlement	< 0.2% in first 3 years <0.4% up to 14 years	Typical range of settlement for USBR dams.

*1 displacement is horizontal deformation, downstream displacement is positive and upstream is negative.
*2 "rockfill" dams including membrane face rockfill dams, and central and sloping earth core rockfill dams
CFRD = concrete face rockfill dam, CCER = central core earth and rockfill dam, RF = rockfill

Hunter and Fell (2003) reported that long-term rate of crest settlement in rockfill dams is mainly influenced by dam height (level of applied stress within the embankment), and intact strength of rockfill material as can be seen in Figure 11. They also noted that dams constructed in areas getting high rainfall, and dams constructed with weathered rockfill, or rockfill subject to weakening on wetting, can be expected to give greater rates.

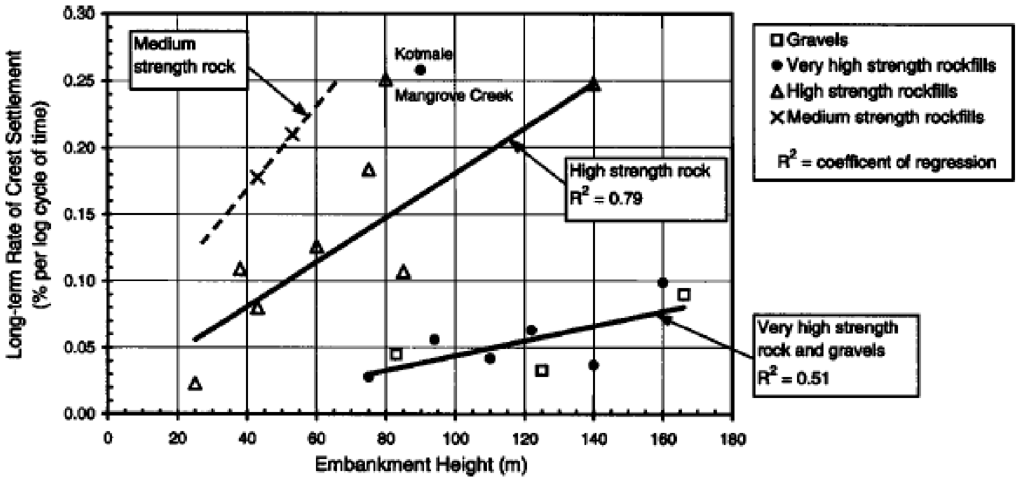

Figure 11: Long-term crest settlement rates (Hunter and Fell 2003)

For estimating rockfill modulus Fitzpatrick et al. (1985) identified rockfill modulus during construction E_{rc}, and the rockfill modulus on first filling E_{rf}, calculated from:

$$E_{rc} = \gamma H \frac{d_1}{\delta_s}$$

$$E_{rf} = \gamma_w h \frac{d_2}{\delta_n}$$

where E_{rc} and E_{rf} are in MPa; γ unit weight of the rockfill inkN/m^3; γ_w is unit weight of water in kN/m^3; δ_s settlement of layer of thickness d_1 due to the construction of the dam to a thickness H above that layer; δ_n face slab deflection at depth h from the reservoir surface; and d_2 is measured normal to the face slab as shown. H, h, d_1 , and d_2 are all measured in meters, and δ_s and δ_n are measured in millimeters. Fitzpatrick et al (1985) noted that E_{rf} is not a true modulus of the rockfill but it is an artifact ofthe method of calculation.

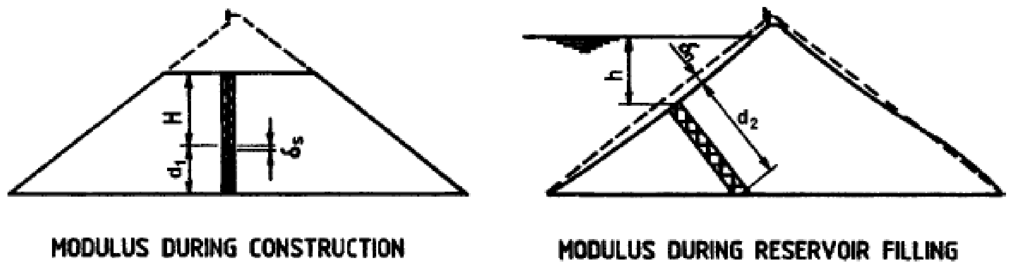

Figure 12: Rockfill modulus defined by Fitzpatrick et al. (1985)

Based on collected data from rockfill dams Hunter and Fell (2003) presented the following graph for the estimation of the secant moduli of the rockfill during construction for the typical well compacted rockfill (i.e. rockfill placed in layers 0.9 to 1.2 m thickness, water added and compacted with four to six passes ofa 10 t smooth drum vibratory roller). For reasonably compacted rockfill the values from the graph can be reduced by half:

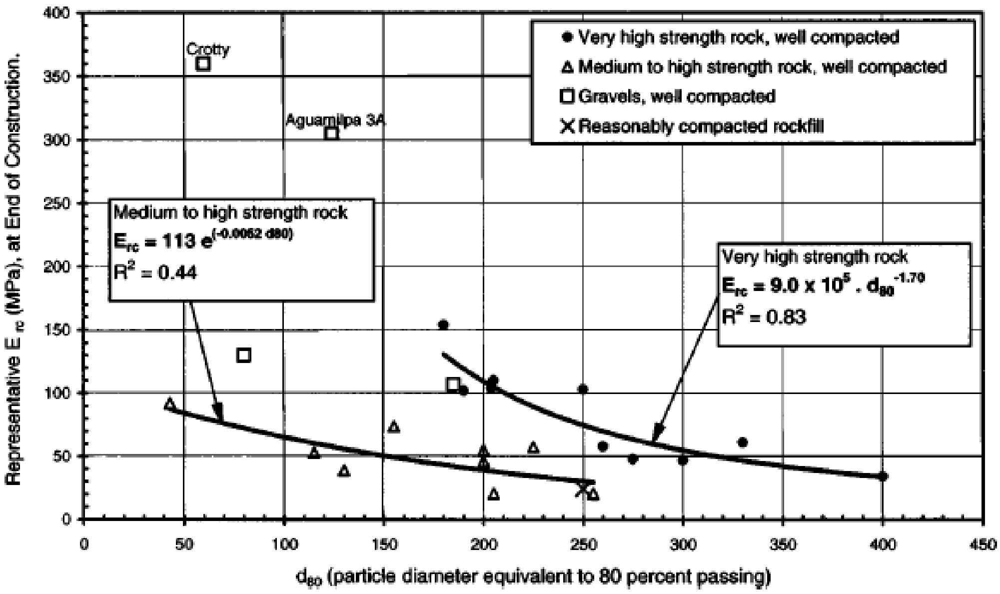

Figure 13: End-of-construction secant modulus of compacted rockfill based on particle size and unconfined compressive strength (from Hunter and Fell 2003)

Hunter and Fell (2003) presented data on suggested empirical relations between the lateral displacements in rockfill dams and their ratio to dam height (Table 5). Lateral displacement of rockfill dams on first reservoir filling: for the crest displacements typically range from 50 mm upstream (-50 mm) to 200 mm downstream, or from -0.02% to 0.20% of the embankment height. For the downstream slope (mid to upper region), displacements are typically downstream in the range from 0 up to 200 to 250 mm (or less than 0.2% of the embankment height).

Table 5: Lateral deformations of the crest of rockfill dams due to first filling of the reservoir (Hunter and Fell 2003)

Core Width	Downstream Shoulder		Core *2 Classification	No. Cases	Displacement Range		Comments
	Material	Rating *1			(mm)	% of dam height *3	
Thin to medium	Rockfill	Well-comp	CL/CH/GC/SC, wet	13	29 to 180	0.0 to 0.12	
			CL/SC/GC, dry	5	5 to 80	0.0 to 0.12	
			SM/GM – dry and wet	3	177 to 394	0.10 to 0.30	Naramata (177 mm), Cougar (375 mm), Round Butte (394 mm)
		Reas to well	CL/CH/SC/GC	5	6 to 78	0.0 to 0.11	
			SM/GM	5	6 to 535	0.03 to 0.36	LG2 (535 mm), Frauenau (250)
		Reas	All types	4	-6 to 1120 (most < 200)	-0.01 to 0.17	Svartevann = 1120 mm (SM core)
		Poor	All types	7	-39 to 480	0.09 to 0.63	No correlation to core soil type, Beliche = 347 mm (0.63%), Djatiluhur = 480 mm (0.46%)
	Gravels	-	All types	3	18 to 107	0.04 to 0.07	
Thick	All cases with thick cores			19	-64 to 285 (most –19 to 222)	-0.02 to 0.20	
	Rockfill	Reas to well	All types	1	0	-	Maroon dam
		Poor	All types	9	-64 to 285 (most –15 to 165)	-0.02 to 0.19	Sth Holston = -64 mm (-0.07%), Glenbawn = 285 mm (0.37%)
	Gravels	-	All types	4	-13 to 44	-0.01 to 0.04	
	Earthfill	-	All types	5	-19 to 222 (most –19 to 45)	-0.02 to 0.19 (-0.02 to 0.09)	Navajo = 222 mm, rest < 45 mm
Very Broad			All types	15	-236 to 229 (most –58 to 94)	-0.02 to 0.14	Rector Creek = -236 mm Mita Hills = -146 mm San Luis (slide area) = -79 mm Horsetooth = 229 mm

Notes: *1 compaction rating of rockfill; well-comp = "well-compacted", reas to well = "reasonably to well compacted", reas = "reasonable compaction", poor = "poorly compacted".
*2 symbols represent soil classification to Australian Standard AS 1726-1993, "wet" = placed at or on the wet side of Standard optimum moisture content, "dry" = placed on the dry side of Standard optimum.
*3 range of displacement as a percentage of dam height excludes possible outliers (Svartevann, South Holston, Glenbawn, Rector Creek, Mita Hills, San Luis (slide area) and Horsetooth dams).

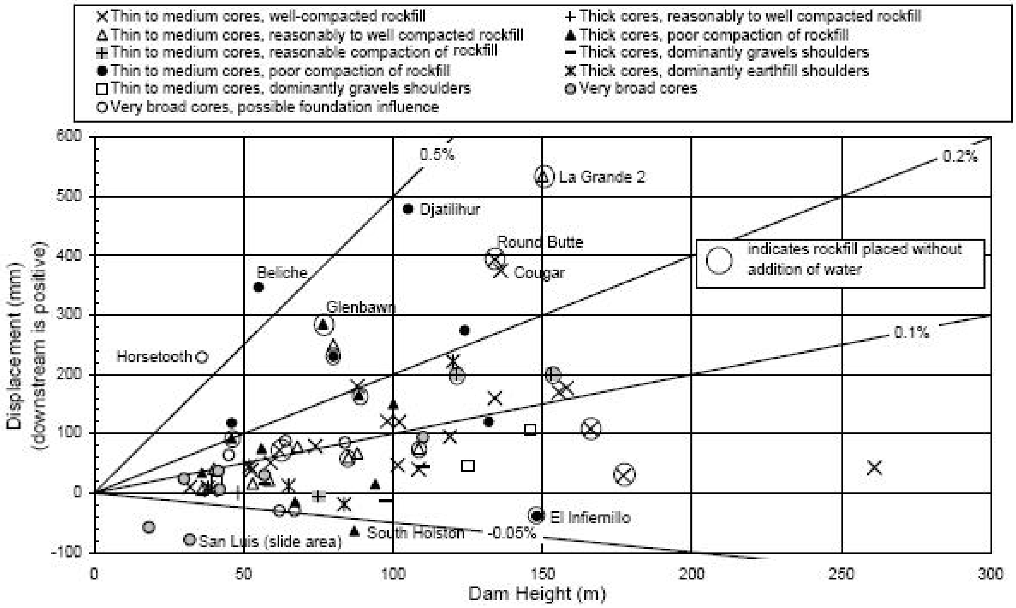

Figure 14: Lateral displacement of the crest on first filling versus embankment height (displacement is after the end of embankment construction) (Hunter and Fell 2003)

CHAPTER 3

BAHÇELİK DAM

3.1 General Information on Bahçelik Dam

Bahçelik Dam is constructed on Zamantı River which is located at Pınarbaşı town in Kayseri Province. The aim of the Bahçelik Dam is to provide irrigation for the neighborhood area and to produce power. The reservoir volume of Bahçelik Dam is 216 hm3 which is distributed in an area of 12 km2. The dam annually produces 35 GWh of energy. Construction of the dam has been completed from 1996 and 2005 (www.dsi.gov.tr). A satellite view of the dam is shown at Figure 15.

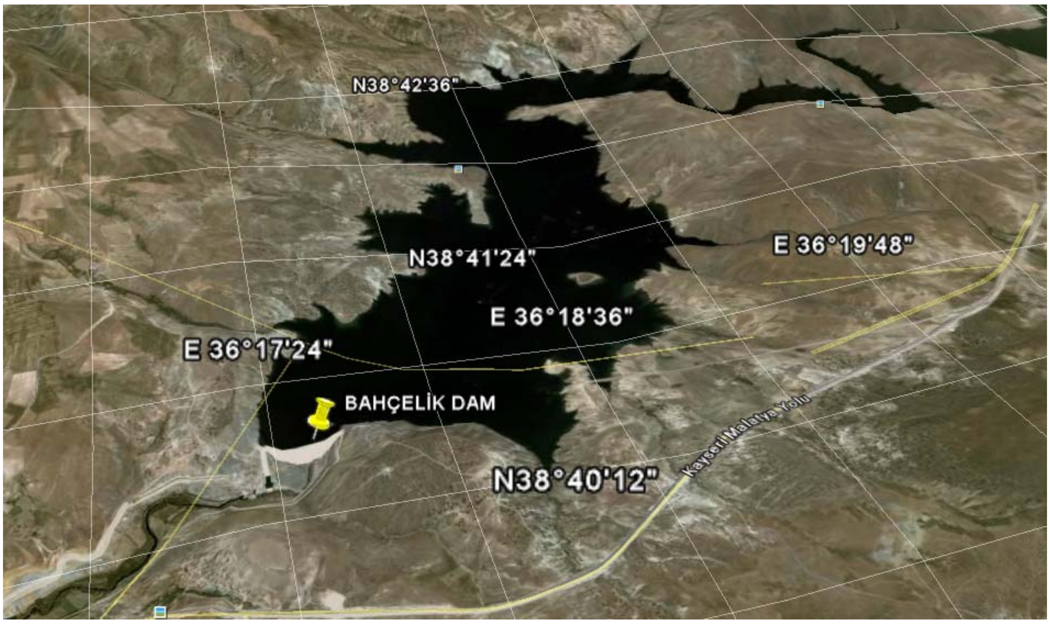

Figure 15: Satellite view of Bahcelik Dam (Google Earth)

The embankment type of dam is rock-fill with a clay core at the center. There are several layers for filtering between clay core material and rock-fill material. There are sand layer, gravel layer and crumbled rock pieces material respectively from clay material to rock-fill material.

Bahçelik Dam has a crest height of 65 m from the bottom of clay core. The crest length of the dam is about 350 m. Both, upstream and downstream faces are inclined with 2H:1V. The bottom of the dam is curved according to the topographic geometry of the valley. The geometry of the dam at the highest cross section is shown in Figure 16.

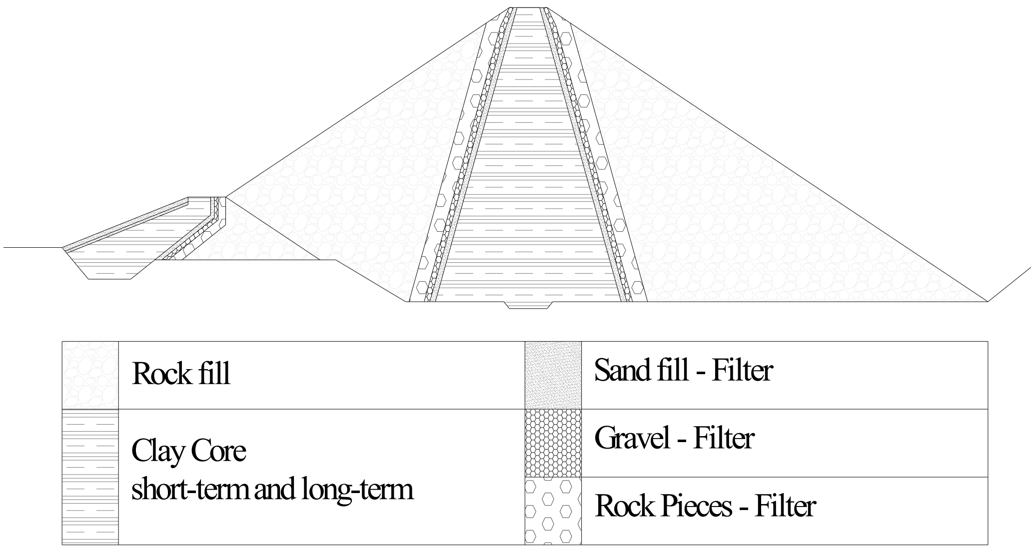

Figure 16: The geometry of Bahçelik Dam

3.2 Topography of Dam Area

Kayseri Province is located at the middle of Turkey with a mean elevation of 1330 m from sea level. It is located in a mountainous area where Mount Erciyes which is the highest and the volcanic mountain in central Anatolia exists. Nevertheless, volcanic history of the area made the geologic structure very stiff.

The geologic layers have some materials such as Eocene rock, Neocene rock, serpentine, Paleozoic rock, etc. However, the geologic layers beneath the Bahçelik Dam are Pliocene rock and greenstone. The geologic map of Kayseri Province and the Bahçelik Dam can be seen in Figure 17.

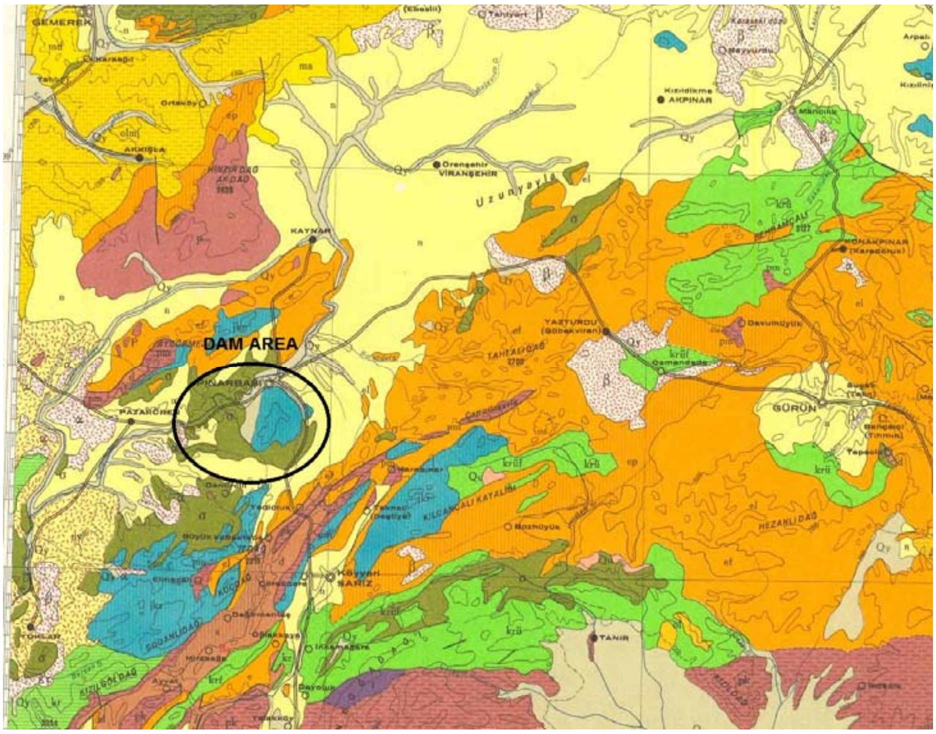

Figure 17: Geologic map of Kayseri Province
(http://www.mta.gov.tr, 14/09/2010)

Kayseri Province is between 3rd and 4th earthquake regions. One of the main faults of Turkey which is Central Anatolian Fault goes through the middle of Kayseri. Since the basin of Kayseri is composed of several types of rocks such as Eocene rock, Neocene rock, serpentine, Paleozoic rock, etc. the earthquake region comes out as given above. Figure 18 shows the Neotectonic map of Central Anatolian Fault Zone.

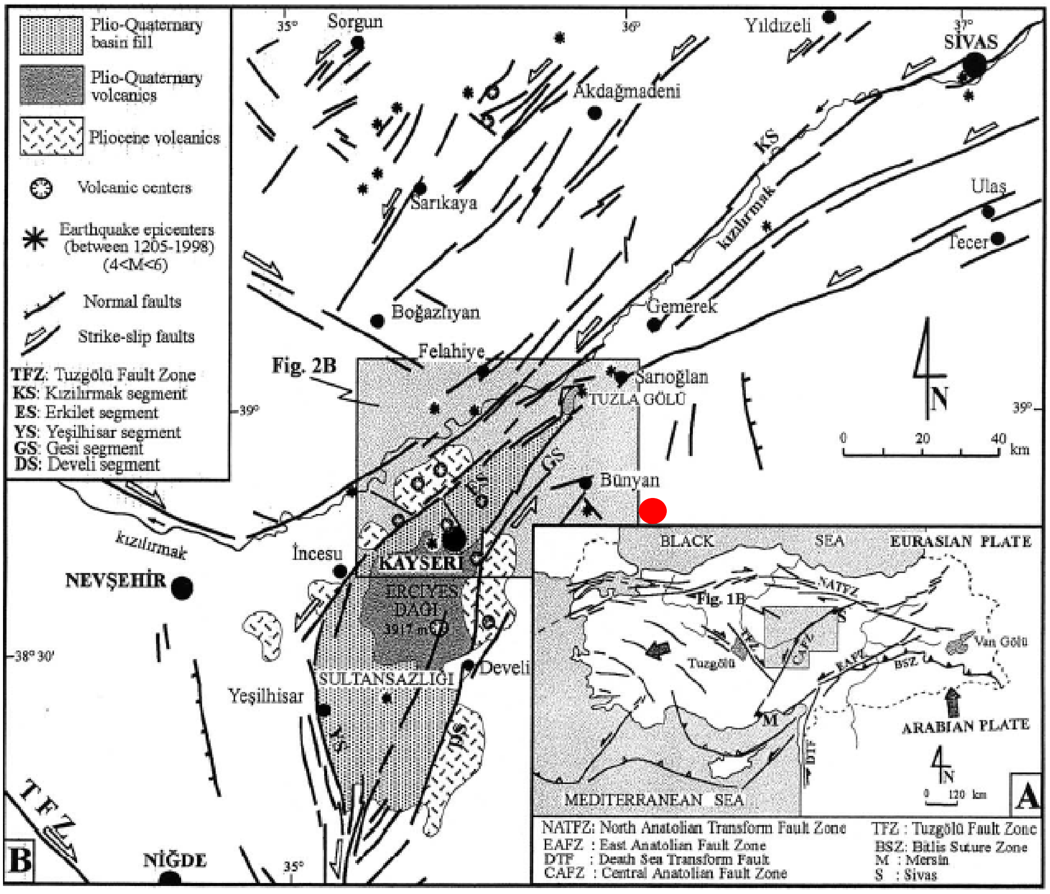

Figure 18: Neotectonic map showing the nortwestward arched segmend of Central Anatolian Fault Zone (Dirik, 2000). Dot marked in the zoomed-in view indicates the location of Bahcelik dam

3.3 Instrumentation in Bahçelik Dam

As it is explained in chapter 2, there are plenty of instrumentation techniques in a dam body. In Bahçelik Dam there are two types of instrumentation techniques. These are piezometers and surface monuments.

In this thesis, the available data spreads over two years. For piezometer data; two readings in 2008 and four readings in 2009 are available. On the contrary, for surface monuments, there are totally three readings in 2008 and five readings in 2009.

3.3.1 Piezometers

There are 39 piezometer wells located on the dam body. The piezometers are placed in three different distances from the beginning of the crest; these are 110m, 150m and 200m which are the highest cross sections. Moreover, at each cross section, each piezometer group is divided into three levels. The cross sections are shown in Figure 19.

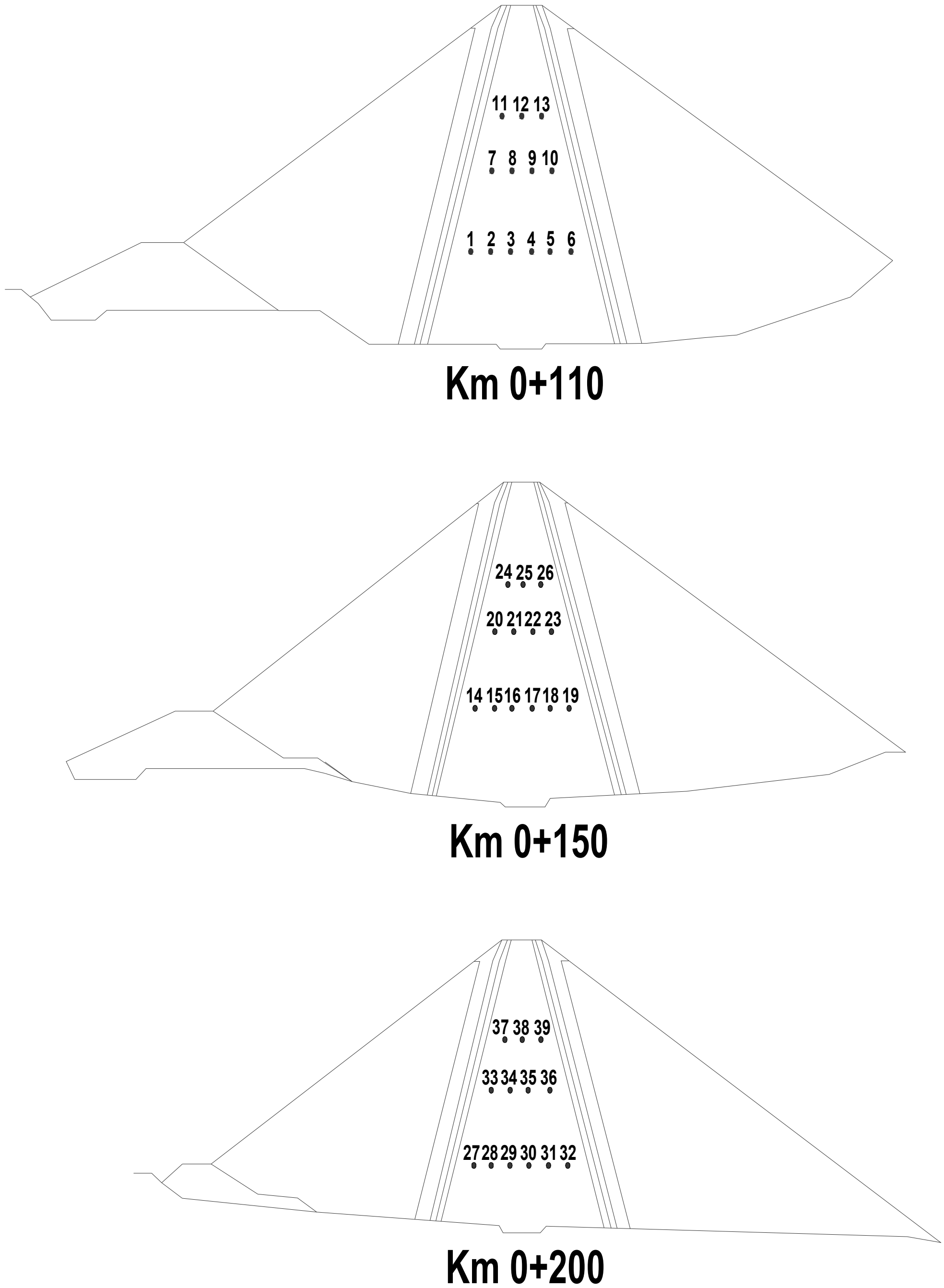

Figure 19: Piezometer locations shown on cross-sections of Bahçelik Dam

Table 6: Piezometer locations according to centerline of Bahçelik Dam

SECTION NO	NO	ELEVATION (m)	DISTANCE TO CENTER LINE (m)	
			UPSTREAM	DOWNSTREAM
Km 0+110	1	1453	12.5	
	2	1453	7.5	
	3	1453	2.5	
	4	1453		2.5
	5	1453		7.5
	6	1453		12.5
	7	1473	7.5	
	8	1473	2.5	
	9	1473		2.5
	10	1473		7.5
	11	1483	10	
	12	1483	ON THE CENTER LINE	
	13	1483		10
Km 0+150	14	1453	12.5	
	15	1453	7.5	
	16	1453	2.5	
	17	1453		2.5
	18	1453		7.5
	19	1453		12.5
	20	1473	7.5	
	21	1473	2.5	
	22	1473		2.5
	23	1473		7.5
	24	1483	10	
	25	1483	ON THE CENTER LINE	
	26	1483		10
Km 0+200	27	1453	12.5	
	28	1453	7.5	
	29	1453	2.5	
	30	1453		2.5
	31	1453		7.5
	32	1453		12.5
	33	1473	7.5	
	34	1473	2.5	
	35	1473		2.5
	36	1473		7.5
	37	1483	10	
	38	1483	ON THE CENTER LINE	
	39	1483		10

3.3.2 Surface Monuments

As it is explained in chapter 2 surface monuments are fixed measuring points on the dam surfaces. There are total 22 surface monuments on the Bahçelik Dam surface. Six of these monuments, from 1 to 6, are located at upstream face of the dam. Rest of 22 monuments is located at downstream face of the dam. Exact locations can be seen at table below. Also the locations of the monuments are illustrated on a sketch of dam top view.

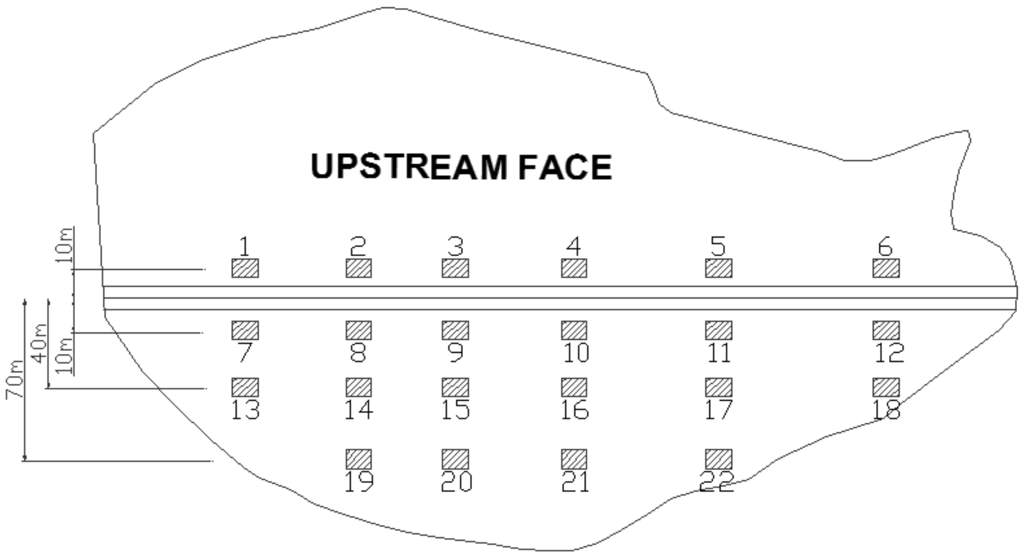

Figure 20: Surface monuments on Bahçelik Dam

Table 7: Surface monument locations on Bahçelik Dam according to centerline of dam

MONUMENT NO	SECTION NO	DISTANCE TO CENTER LINE (m)	
		UPSTREAM	DOWNSTREAM
1	0+060	10.00	-
2	0+110	10.00	-
3	0+150	10.00	-
4	0+200	10.00	-
5	0+260	10.00	-
6	0+330	10.00	-
7	0+060	-	10.00
8	0+110	-	10.00
9	0+150	-	10.00
10	0+200	-	10.00
11	0+260	-	10.00
12	0+330	-	10.00
13	0+060	-	40.00
14	0+110	-	40.00
15	0+150	-	40.00
16	0+200	-	40.00
17	0+260	-	40.00
18	0+330	-	40.00
19	0+110	-	70.00
20	0+150	-	70.00
21	0+200	-	70.00
22	0+260	-	70.00

The data acquired from DSİ (General Directorate of State Hydraulic Works) is combined together and the following graphs (Figure 21 and Figure 22) are prepared in order to get summary information. Tables of the reading for all dates are listed at Appendix D.

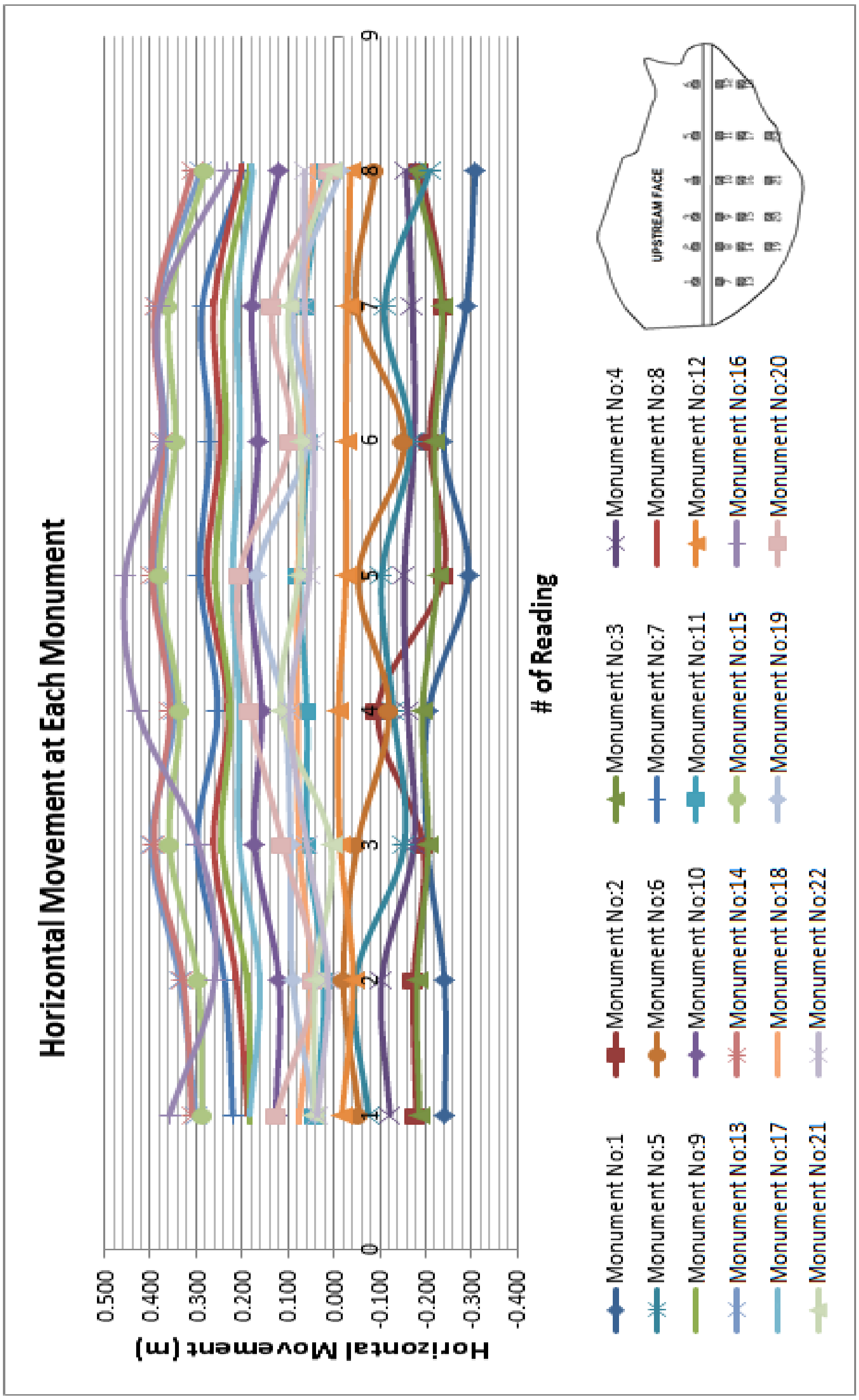

Figure 21: Bahçelik Dam surface monuments readings for horizontal deflection 1)14.08.2008, 2)14.10.2008, 3)14.12.2008, 4)22.04.2009, 5)17.06.2009, 6)06.08.2009, 7)28.09.2009, 8)18.11.2009, (Note: Time is not at equal interval scale)

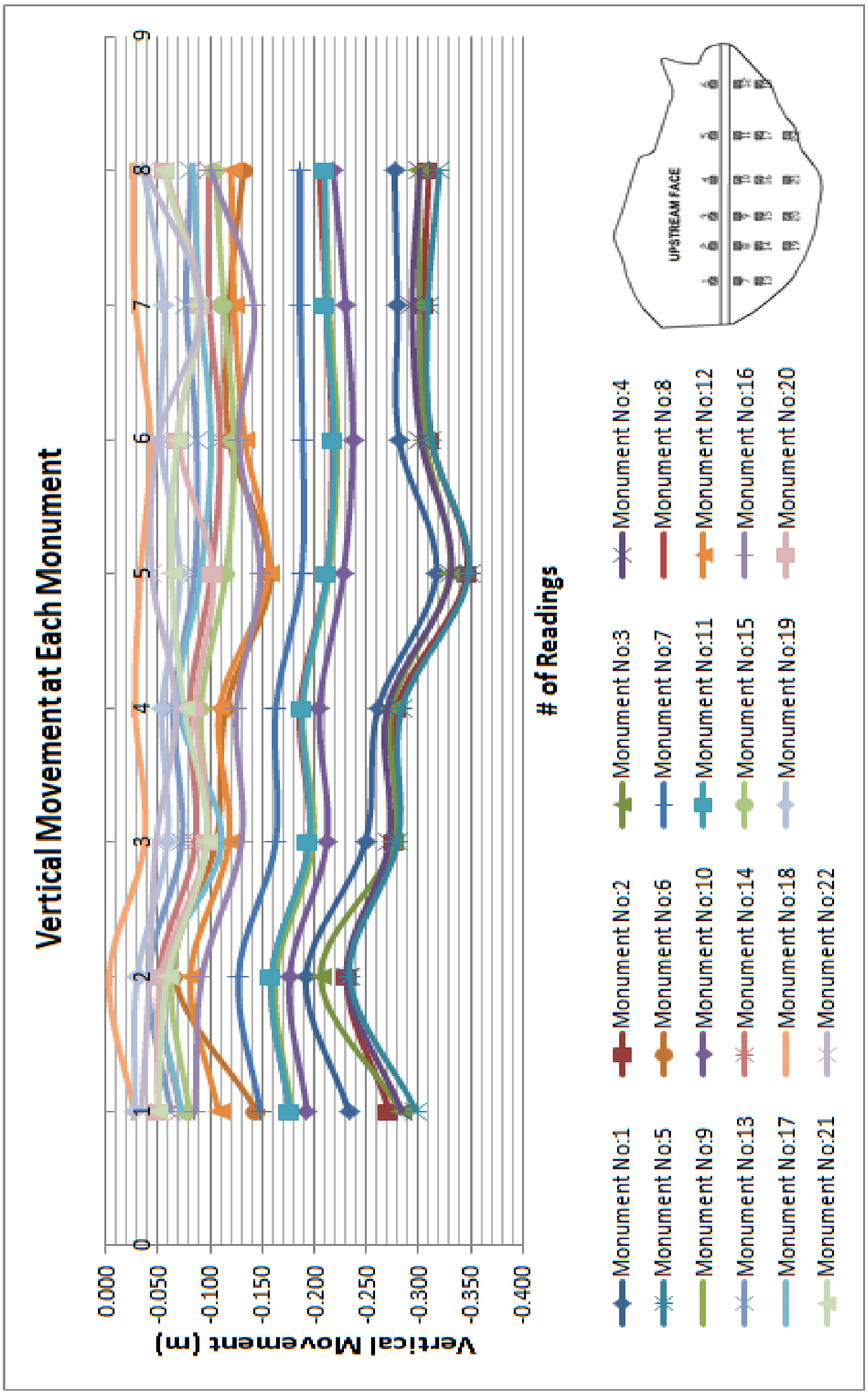

Figure 22: Bahçelik Dam surface monuments readings for vertical deflection

1)14.08.2008, 2)14.10.2008, 3)14.12.2008, 4)22.04.2009, 5)17.06.2009

6)06.08.2009, 7)28.09.2009, 8)18.11.2009

(Note: Time is not at equal interval scale)

As it can be seen easily, for horizontal movement the surface monument no 16 has the largest value. Readings at monument no 16 are fluctuating distinctively when compared to the other monuments nearby (numbers 13, 14 and 15). Since the readings are taken with a manually operated surveying device instead of a digital one, there could be operator errors. Eventually, it is ignored and the mean value of the closest three surface monuments which are no 13, no 14 and no 15 are taken into consideration. The mean envelope of the three readings is as following. From trend line of the three readings, the mean horizontal movement value comes out as 0.35 m at 40 m away from center line of the dam at downstream face.

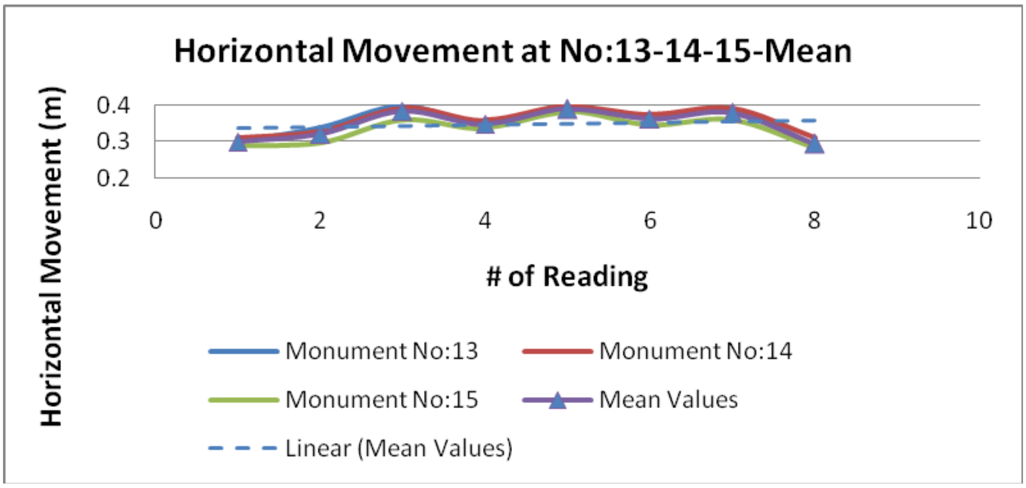

Figure 23: Mean value of no 13, no 14 and no 15 monuments

If the vertical movement on the surface monuments is evaluated, it can be observed that monument numbers 2, 3, 4 and 5 show similar values of vertical deformations with time. Monument no 5, located at the top of the dam in the upstream face, has the largest value of vertical deformation (0.35 m) among all other monuments. The fluctuation of the settlement graph could be because of the rainy season and the quantity of rain dropped to the area, in addition to possible nonuniform compaction and material densities at different locations in the dam body.

CHAPTER 4

ANALYSES OF BAHÇELİK DAM

4.1 Finite Element Modeling

R. Courant has first developed finite element analysis in 1943 by utilizing the Ritz method of numerical analysis and minimization of variational calculus to obtain approximate solutions to vibration systems. After a decade, M. J. Turner, R. W. Clough, H. C. Martin, and L. J. Topp team have published a paper which establishing a broader definition of numerical analysis concentrated on the "stiffness and deflection of complex structures".

After 1970s, generally the aeronautics, automotive, defense, and nuclear industries was using the finite element analysis but it was limited to expensive mainframe computers. Resulting in rapid decrease in the cost of computers and phenomenal increase in computing power, finite element analysis has been developed to incredible precision. Nowadays, a standard computer sold in a computer market has the ability to produce accurate results for all kind of parameters.

Generally there are two types of analysis that are used in industry; 2D modeling and 3D modeling. Since 2D modeling is simpler than 3D modeling and so on runs on a relatively normal computer, it gives less accurate results compared to 3D modeling. Within each of these modeling schemes, the programmer has to insert numerous algorithms which make the system behave linearly or nonlinearly. Since the linear modeling is less complex, it does not take into account plastic deformation. However, non-linear modeling does solve for plastic deformation and may also capable of testing a material all the way to fracture.

Bahçelik Dam has been modeled by a 2-D plane-strain finite element methodology using Plaxis software. The geometry of the problem is defined (construction in stages will be explained below) and the boundary conditions are determined. In order for the calculated deformations not to be affected by boundary conditions; suggested rules of thumb in the literature about the limits of the geometry have been used. In the finite element mesh 15-node isoparametric triangular elements are used. The refinement of mesh size can increase the accuracy of the finite element calculations. In this study meshing property is defined as fine. The number of elements are used in the mesh is 705. The finite element mesh of Bahcelik Dam can be seen in Figure 24.

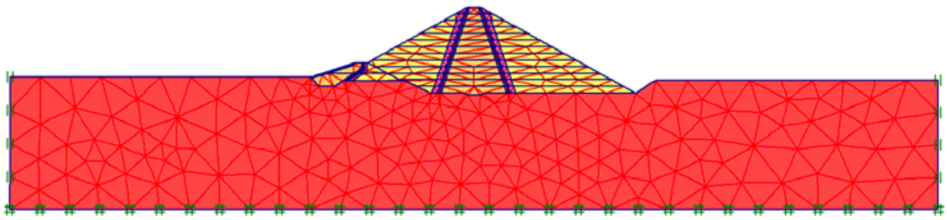

Figure 24: Model mesh

Analyses are performed in numerous stages so that the construction of the dam is realistically captured. In the beginning of construction of the dam, only coffer dam is constructed. After that, each 5 m height of dam is considered as one stage. The dam construction is finished in 13 stages (Figure 25). Then, reservoir of the dam is started to being filled. This process is divided into 3 stages from the ground level up to full reservoir level.

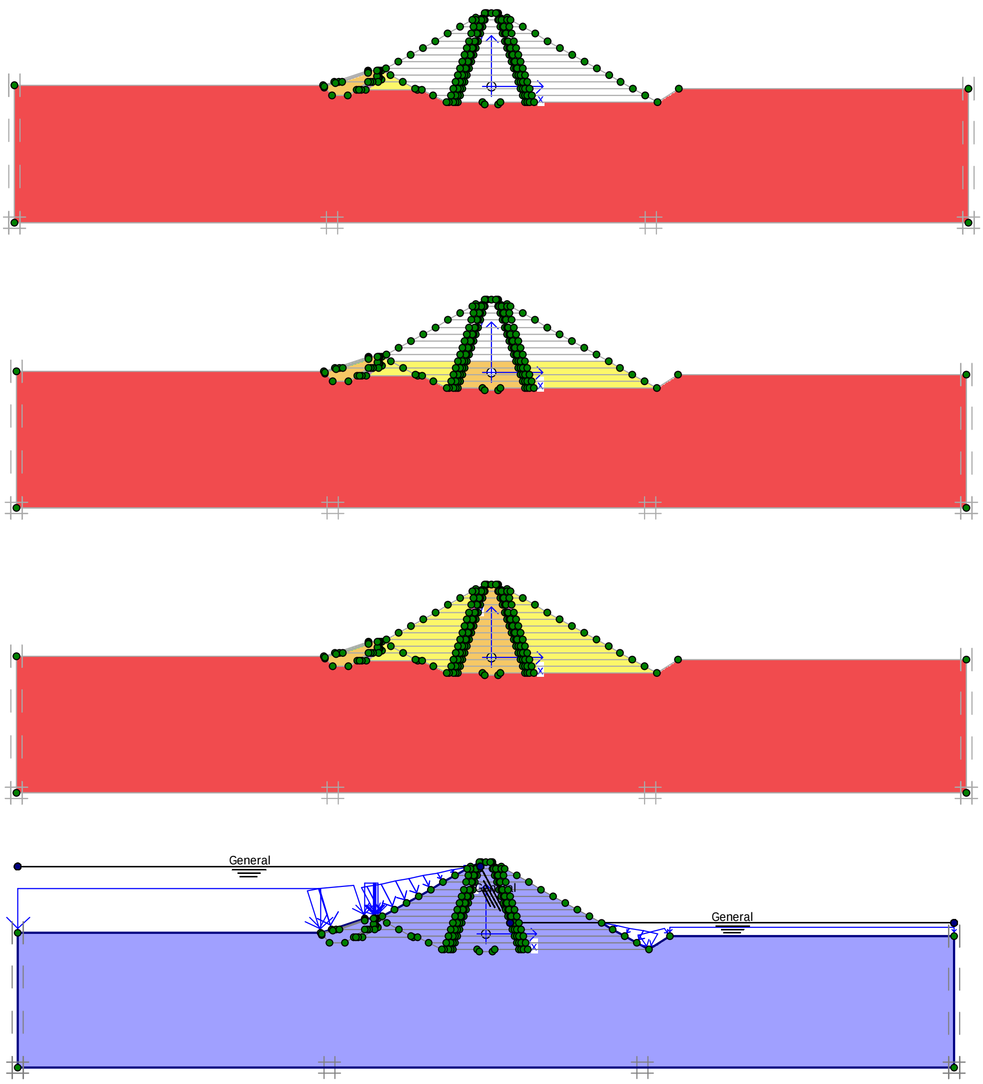

Figure 25: Stages from analyses of Bahçelik Dam

Materials are defined after the geometry of the dam has been inputted totally. The next chapter describes the selection of material model and related parameters.

4.1.1 Selection of Material Model and Parameters

The materials used in the dam body can be seen in the Figure 26. In addition to Figure 26, bedrock is also added to the model. Since there is no information about real material properties, simple linear elastic material model was selected to be used in the analyses, for all materials. For granular materials (sand, gravel and rockfill), linear elastic material model may be sufficiently accurate to represent the real behavior. However, recent studies on finite element modeling of rockfill dams suggested use of Plaxis hardening soil model for nonlinear, inelastic, stress-dependent behavior of rockfill materials (Ozkuzukiran et al. 2006, Unsever 2007). In this study, accuracy and adequacy of a simple material model (such as elastic plastic Mohr Coulomb soil model, which, as compared to more advanced material models does not require many material parameters) in the calculation of rockfill dam deformation behavior is investigated. As for the clayey material in the dam core, using a linear elastic material model will not accurately capture the true behavior of this material in the field. However, since the material properties in this study were going to be back-calculated through a sensitivity analyses and since there is no laboratory or otherwise any data on the stress-strain behavior of the clay used in the dam core, it was decided to simplify the material properties by using linear elastic model for the clayey materials as well. The error related to these assumptions can be evaluated in future studies.

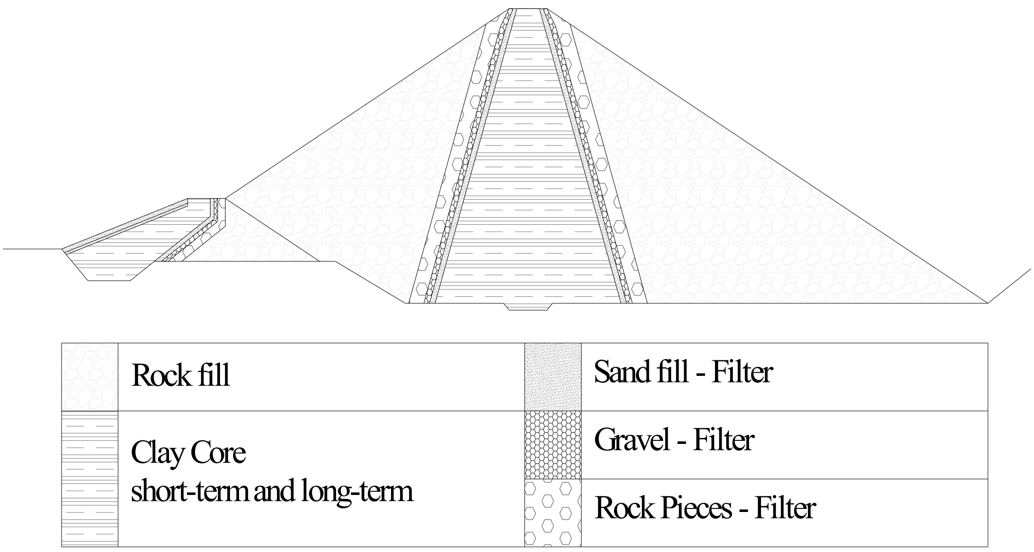

Figure 26: Materials in Bahçelik Dam

As mentioned above, since the material properties of the dam are not defined in the report supplied from DSİ, they are defined from sensitivity analysis. Records of measured movement of surface monuments are used to back-calculate material properties. In this process, material properties are varied within a probable range until calculated and measured deformations have been matched with a reasonable accuracy

The view of a typical cross section of dam and the surface monuments on this cross section are shown in Figure 27. In Figure 27 monument numbers 14, 15 and 16 have the maximum horizontal movements. In Figure 20, the monument numbers 2, 3 and 4 have the maximum vertical movements.

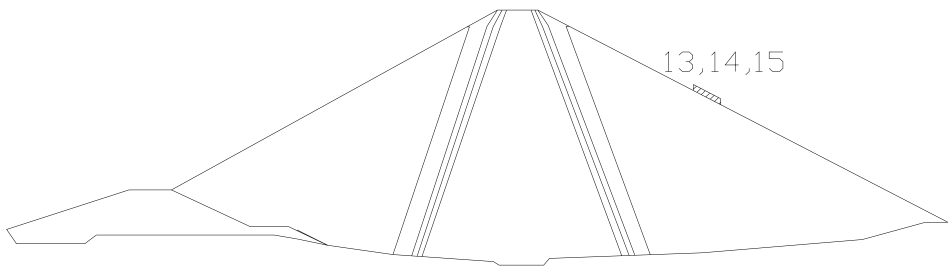

Figure 27: The monuments which has maximum horizontal movement readings

In order to match behaviors of the real dam and the Plaxis model, sensitivity analysis is performed in order to define soil parameters as accurate as possible in the range of recommended literature values. The analysis results are compared to the real case movement values in order to achieve closest parameters values. Sensitivity analysis is a technique used to determine how different values of an independent variable will impact a particular dependent variable under a given set of assumptions. This technique is used within specific boundaries that will depend on one or more input variables.

The boundaries of the material properties are taken from Table 8.

Table 8: Material property range table (Bowles, 1996)

Poisson's ratio:		Young's modulus (Values given in MPa):	
Clay, saturated	0.4-0.5	Clay	
Clay, unsaturated	0.1-0.3	. Very Soft	2-15
Sandy clay	0.2-0.3	. Soft	5-25
Silt - 0.3-0.35	0.3-0.35	. Medium	15-50
Sand, gravelly sand (not elastic but 0.3-0.4 commonly used)	0.1-1.0	. Hard	50-100
Rock	0.1-0.3	. Sandy	25-250
Loess	0.1-0.3	Glacial till	
Commonly used values (Poisson's ratio):		. Loose	10-150
		. Dense	150-720
		. Very dense	500-1440
Most clay soils	0.4-0.5	Loess	15-60
Saturated clay soils	0.45-0.5	Sand	
Cohesionless(medium & dense)	0.3-0.4	. Silty	5-20
Cohesionless(loose to medium)	0.2-0.35	. Loose	10-25
		. Dense	50-81
		Sand and gravel	
		. Loose	50-150
		. Dense	100-200
		Shale	150-5000
		Silt	2-20

Sensitivity analysis is performed with several types of properties such as elastic modulus, Poisson’s ratio etc.

Elastic modulus is a property that dramatically influences the horizontal deformations if it changes. While determining the material properties, elastic modulus was paid a special attention as it is the most significant property. So, relatively more iterations have been made for elastic modulus in sensitivity analysis. After several iterations, the materials and the properties listed below are determined.

Table 9: Material properties used in the model

	Rock fill	Clay Core short-term	Clay Core long-term	Sand Fill Filter	Gravel Filter	Rock Pieces Filter	Bedrock
γ_{dry} (kN/m^3)	23	19	19	20	20	20	24
γ_{sat} (kN/m^3)	24	19.5	19.5	20.5	20.5	20.5	25
ν (nu)	0.3	0.49	0.35	0.3	0.3	0.3	0.1
c (kN/m^2)	0.1	75	25	0.1	0.1	0.1	100
ϕ (phi) (degrees)	42 °	0 °	20 °	35 °	38 °	40 °	45 °
ψ (psi)	10 °	0 °	0 °	0 °	0 °	0 °	10 °
E (MPa)	65	30	30	25	32	35	200
Permeability (m/s)	1.0E-4	1.0E-9	1.0E-9	1.0E-4	1.0E-4	1.0E-4	1.0E-8

Strength parameters of clay are defined in two stages such as long-term and short-term parameters. Short-term clay parameters are used from the construction start time up to date of full reservoir level. At the beginning of construction phases, clay material behaves as undrained very fine material. But after some time, it starts to behave like drained material. Thus, it is important to use clay material with its two different behavior in the analyses.

4.2 Seismic Analyses (Pseudo Static Analysis)

In this section seismic hazard assessment of Bahçelik Dam is investigated. As it is explained in the first chapter, investigation area stands inside the Kayseri Province border, in the direction of west of city center, on Zamantı River.

Kayseri Province is in the third and fourth earthquake region. However, Bahçelik Dam is in the fourth region area of Kayseri Province which is not a dangerous case. (Turkey Earthquake Regions Map, Ministry of Public Works and Settlement, 1996)

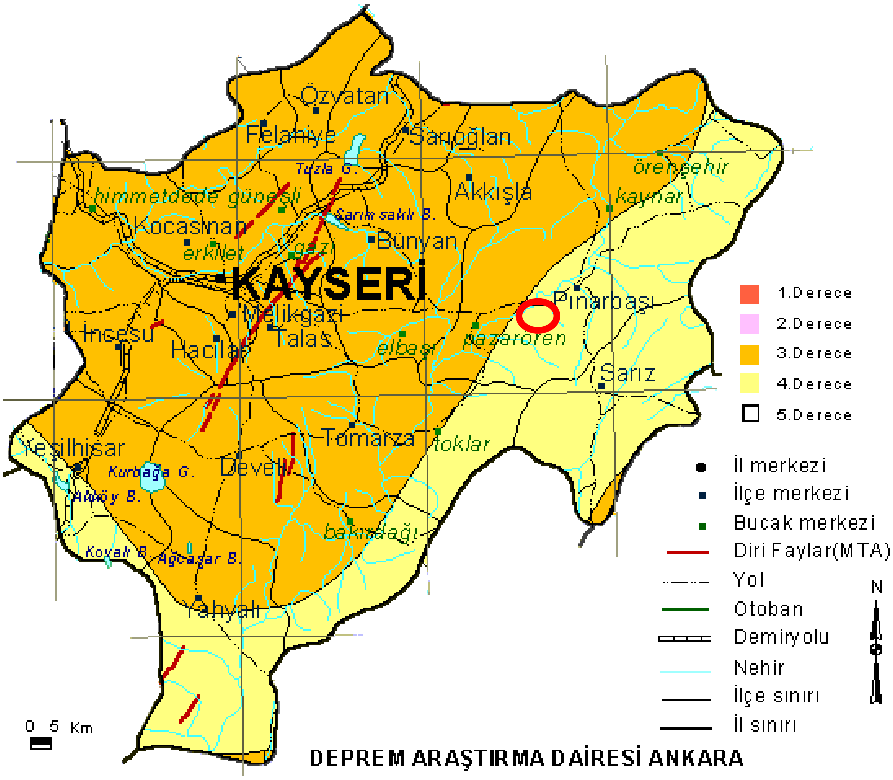

Figure 28: Kayseri Province Earthquake Regions Map; red circle shows the dam location

The properties of expected seismic shock in the area defined by design procedure prescribed in NEHRP (2003) and by using design spectrum parameters prescribed in DLH Geotechnical Design Manual (2007). Chosen design procedure and parameter definitions are shown in Figure 29.

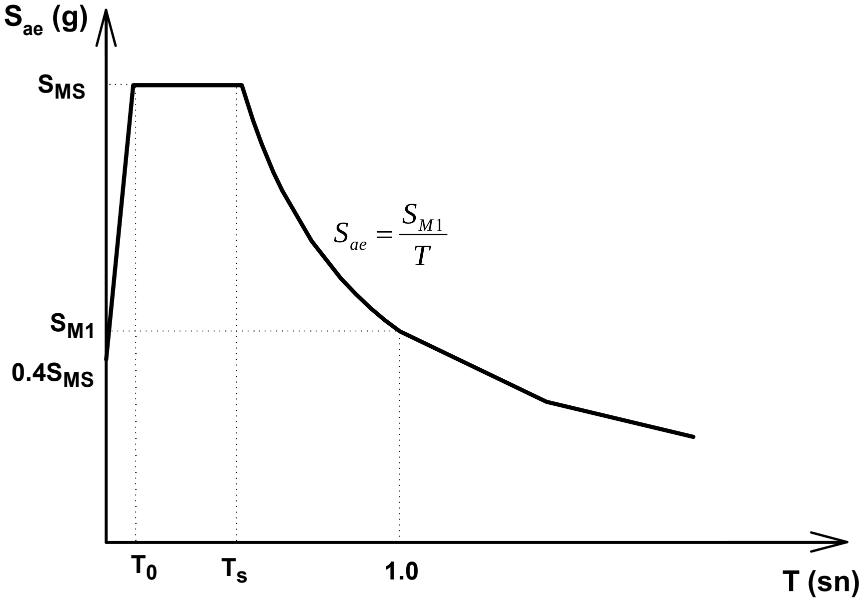

Figure 29: NEHRP Design Spectrum Parameters (m)

Elastic design spectrums are prepared with a 5% damping ratio according to recurrence time of 72 years, 475 years and 2475 years, respectively, while the economic life of the dam is thought to be 50 years. The values which are chosen above equal to probability of exceedence of 50% in 50 years, 10% in 50 years and 2% in 50 years, respectively. The NEHRP elastic design spectrum parameters are given in the Table 10.

Table 10:NEHRP Elastic Design Spectrum Parameters

Recurrence Time	Probability of Exceedence		NEHRPdesign spectrum parameters			
			S_{MS}	S_{M1}	T_0	T_S
2475years	50 years	2%	0.53	0.15	0.06	0.28
475 years	50 years	10%	0.29	0.08	0.06	0.28
72 years	50 years	50%	0.12	0.04	0.07	0.33

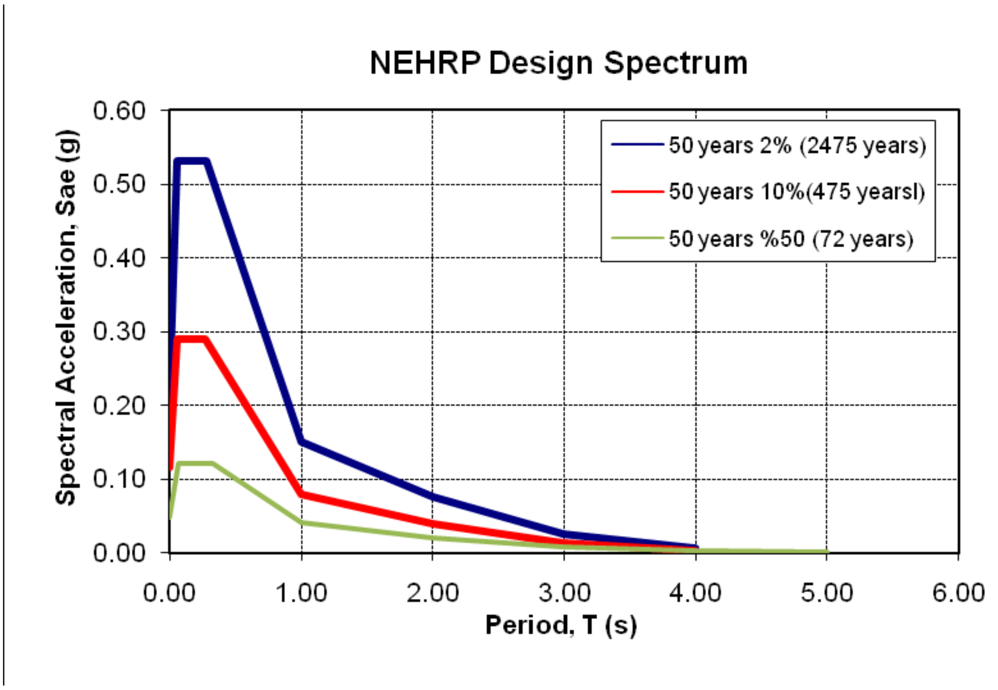

Figure 30: NEHRP Elastic Design Spectrum for Bahçelik Dam

CHAPTER 5

ANALYSES RESULTS

5.1 Finite Element Modeling Results

Analyses of the Bahçelik Dam are performed by using 2D Plaxis software. Total stresses, displacements and pore water pressures are calculated by two dimensional plain strain finite element analyses. Elastic plastic Mohr Coulomb soil model is used in the analyses in this study.

Finite element analyses are performed using below finite element mesh which is developed by 2D Plaxis software automatically. The mesh coarseness is chosen as fine since the levels of construction has been chosen as fine.

Analysis of the Bahçelik Dam is performed in 18 phases. 13 phases are used in order to represent construction procedure which is assumed to be construction progress updates at every 5 m. Four phases are used in order to represent reservoir filling. The last phase is for representing longterm behavior of the dam body.

5.1.1 Horizontal Movement

It is assumed that, most probably the fixing time of the surface monuments is just after the construction. According to this, the results will show only the deflection occurred from the time of end of construction up to now. Since there are no monument readings in our hand, the deformations during construction are not known. The results of horizontal deflection behavior of the dam body after the full reservoir condition is shown below.

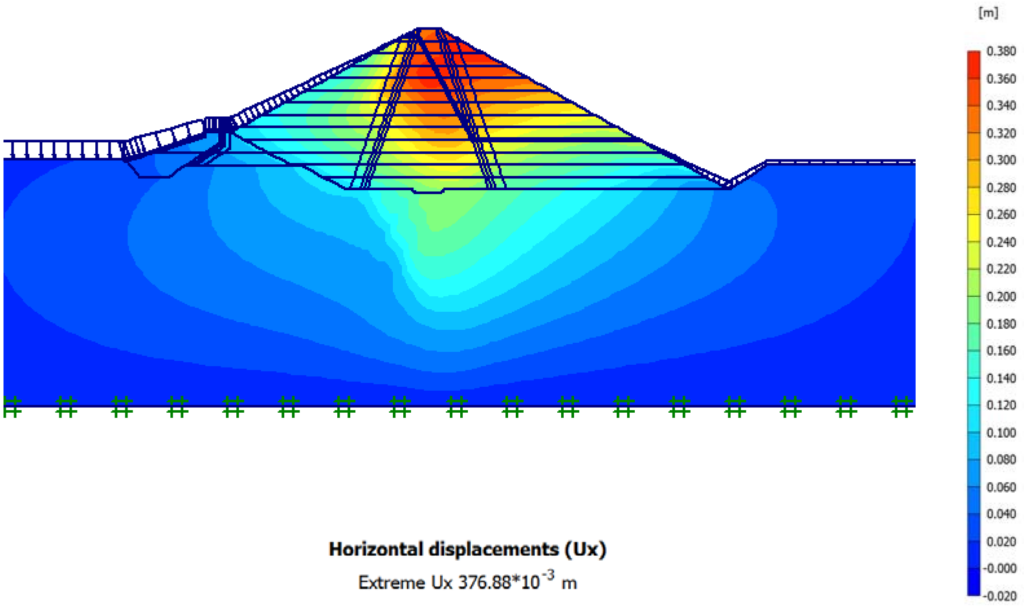

Figure 31: Horizontal displacement of the Bahçelik Dam, full reservoir condition

The comparison for the readings of surface monuments and the analysis results is shown in the below table.

Table 11: The comparison for the horizontal displacement readings of surface monuments and the analysis results

Surface Monuments		Analysis Results	
Distance from Centerline	Movement (m)	Distance from Centerline	Movement (m)
10 m	0.185	10 m	0.363
40 m	0.381	40 m	0.331
70 m	0.096	70 m	0.241

In the analysis, the maximum deflection value which is 0.381 m measured in the real dam body is considered as a target value while estimating dam body material properties. As it can be seen in the above table, the value at 40 m far from centerline is as much as the same with the desired target value. The horizontal deformation behaviors of the model and the real case are as below:

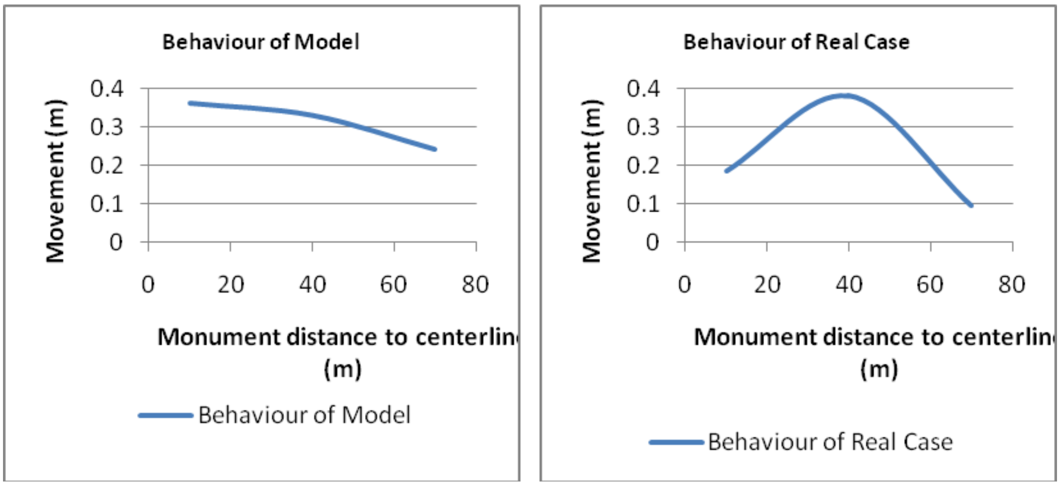

Figure 32: Horizontal displacement behaviors for computer model and real case

It may be noted that a recent study by Unsever (2007), using hardening soil model for the rockfill material, concluded that the calculated and measured deformations in rockfill dams could be within 0.5 to 2 times each other, and this order of magnitude estimation is still considered to be successful. In the current study, a simpler material model (elastic plastic Mohr Coulomb model) is used for the rockfill instead of hardening soil model, because of the minimum number of parameters required in this material model as compared to more sophisticated material models. The values obtained in this study by using such a simplified material model in the analysis is still able to calculate the horizontal deformations that are twice the measured deformations. Therefore it can be considered successful. The reasons for the discrepancy in the measured and calculated values in this study could be due to (1) the set goal of only capturing the maximum deformation value measured at a point rather than capturing the deformation behavior throughout the dam, (2) using simple material model for all soils, (3) nonuniform compacting and different material properties in real dam, (4) the possible 3D arching effect in reality due to valley shape which cannot be captured in 2D plane strain analysis in this study (5) inaccuracy in measured deformations and/or inaccuracy in our estimate of the start time of zero deformation reading etc.

5.1.2 Vertical Movement

The settlement of the dam has been also checked by PLAXIS software. Among the many construction stages only the end of construction and long-term stages are presented here.

The settlement at the end of construction has been calculated as 1.21 m maximum at top of the dam. Figure 33 shows the settlement behavior of the dam. The time versus vertical deformations plot given in Figure 22 shows that the maximum settlement during the measurement period of Bahcelik Dam was about 0.30 m. However, the zero time of installation of instruments at Bahcelik dam is not known. Therefore it is not possible to confirm the validity of the end of construction vertical movements computed by PLAXIS. However, as can be seen in Figure 10 and Table 4, the end of construction vertical deformation values of 1.25%H (H=dam height) have been reported in the literature for clay cored rockfill dams. Therefore the calculated end of construction settlement values could be reasonable, keeping in mind that in the current analysis simple material model and back-calculated material properties are used.

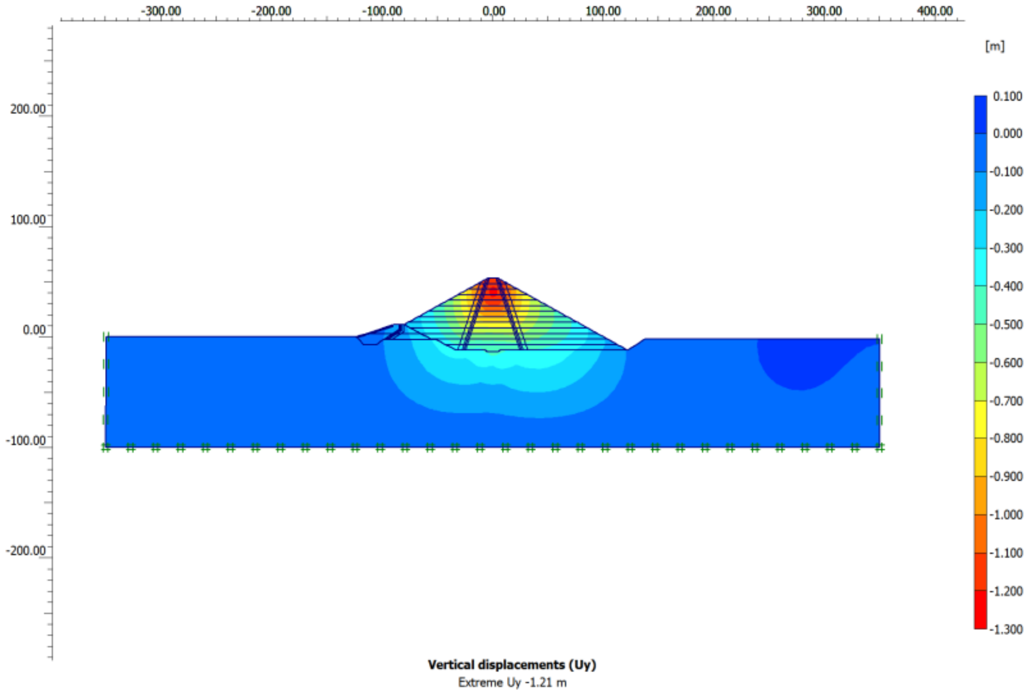

Figure 33: Vertical deflection behavior of Bahçelik Dam in finite element modeling at the end of construction

If there is reservoir water in the system, and if there is no impervious material at the upstream face of the dam, the vertical movement behavior of the dam cannot be precisely calculated by a simple material model in PLAXIS. Figure 34 shows the behavior of the model for vertical deflection while there is reservoir water in the system. Maximum upward movements of about 40 cm have been calculated by the simple elastic plastic Mohr Coulomb material model. It should be noted that in Figure 22 and in the tables given in Appendix D there have been some reported upward movements (up to values of 0.25 m) in Bahcelik dam, especially in the monuments with numbers 1-6 located in the upstream face of the dam. This is because reservoir water acts as an uplifting force causing unloading behavior in the rockfill material, and some vertical movements could be observed in upward direction. Within the confines of this thesis, a simple material model is used, which cannot take into account the increased stiffness of the rockfill material in the unloading stress path condition.

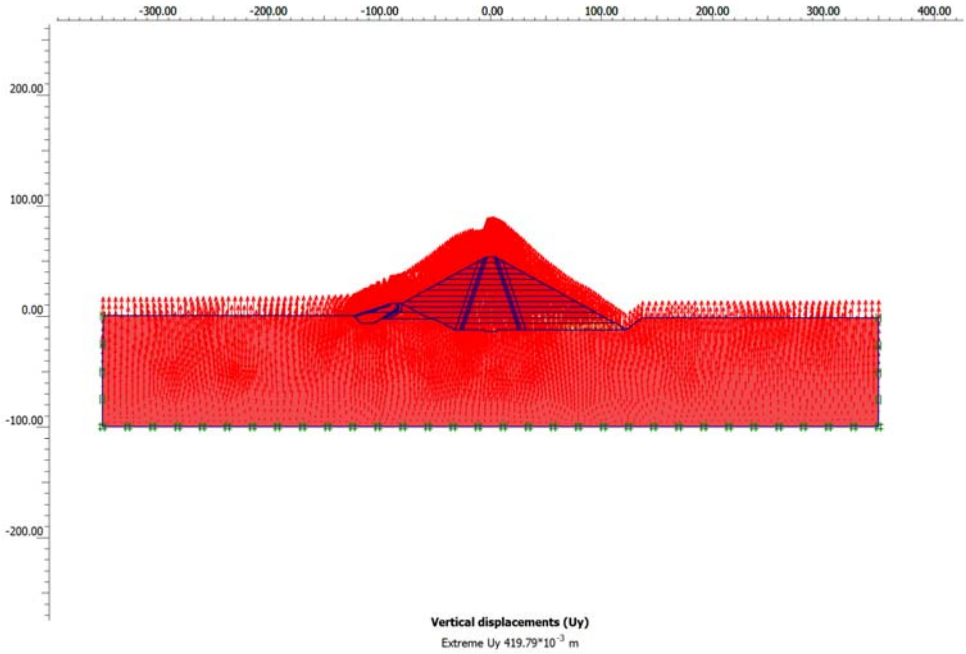

Figure 34: Vertical deflection behavior of Bahçelik Dam in finite element modeling at full reservoir

5.2 Seismic Analyses Results

In Chapter 4, seismic analyses of Bahçelik Dam procedure was explained. If it is summarized shortly; elastic design spectrums were performed with a 5% damping ratio in accordance with recurrence time of 72 years, 475 years and 2475 years, respectively, while the economic life of the dam is thought to be 50 years. The values which are chosen above equal to probability of exceedence of 50% in 50 years, 10% in 50 years and 2% in 50 years, respectively.

Seismic analysis are performed for three stages; i) just after construction finishes, ii) just after reservoir is full, iii) in longterm period.

It is thought that the most critical stage would be the second one that is just after full reservoir. Since the water in the upstream face would behave like a thrust and it would enforce the dam body during earthquake. However, without water mass in the upstream face, there would be no extra mass to produce extra deformation. Used seismic coefficients (k) during analysis are shown in Table 12.

Table 12: Seismic coefficients which are used in seismic analysis

Probability of Exceedence in 50 years	Seismic Coefficient k
2%	0.10
10%	0.06
50%	0.02

Since the value for 50% probability of exceedence in 50 years is very small, the analyses are performed only for 2% and 10% probabilities.

5.2.1 Just After Construction

The Bahçelik Dam is checked for peak ground accelerations (PGA) which have probabilities of exceedence of 2% in 50 years and 10% of 50 years, since the economic life Bahçelik Dam is assumed to be 50 years.

Figure 35 shows general view of Bahçelik Dam for horizontal displacement in an earthquake with a 2% probability of exceedence in 50 years. The later figures will show closer view of the same deformation behavior in order to express better approach for evaluating figures.

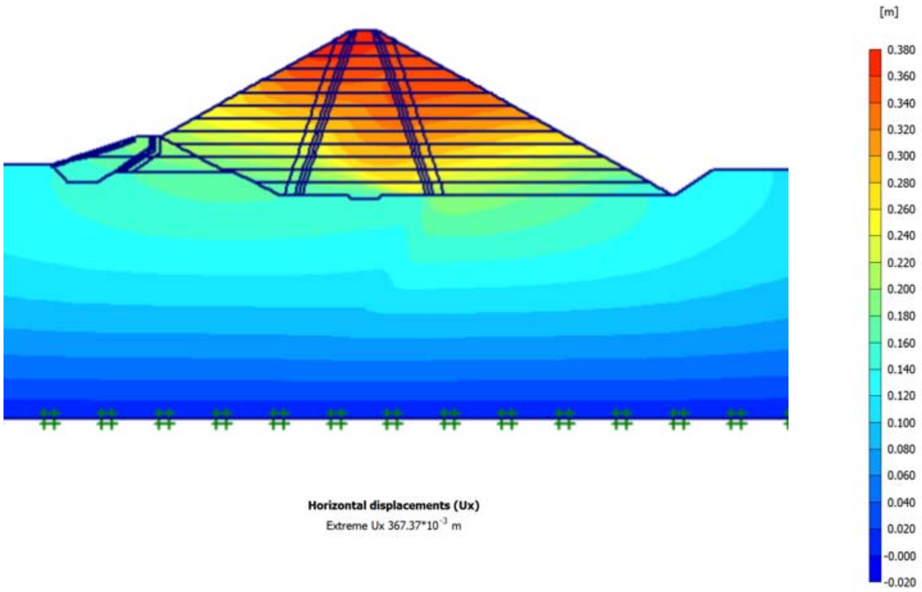

Figure 35: Horizontal displacement occurred at seismic analysis just after construction phase; k=0.1

Below figures are representing deformations at the end of seismic analyses for 2% and 10% probability of exceedence.

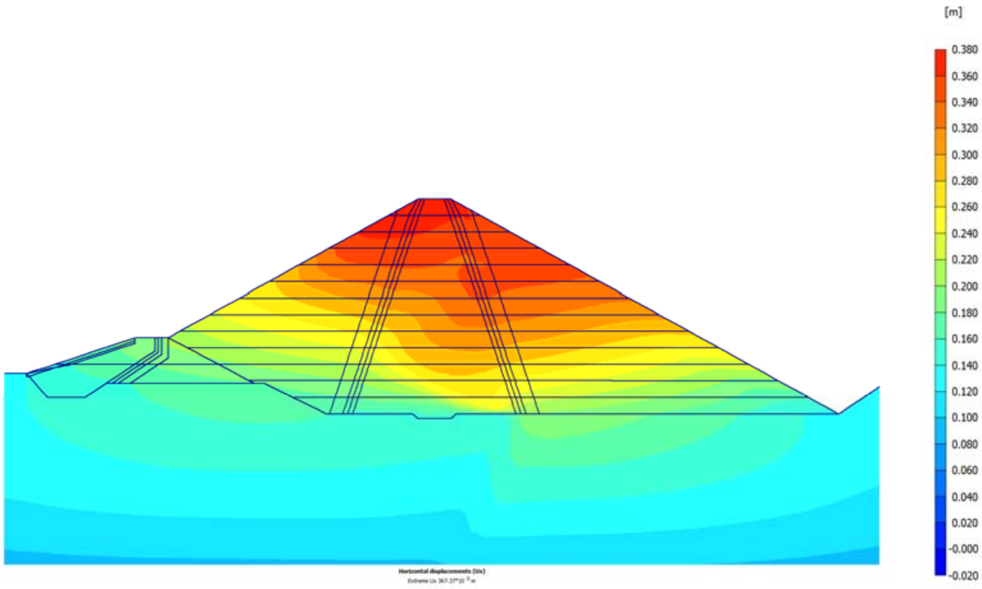

Figure 36: Horizontal displacement occurred at seismic analysis just after construction phase; k=0.1

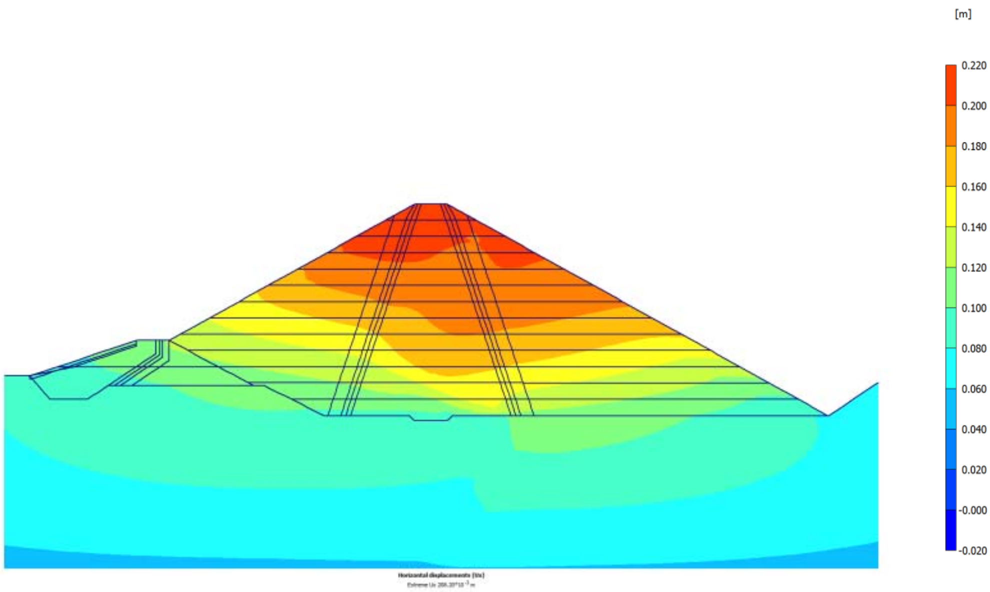

Figure 37: Horizontal displacement occurred at seismic analysis just after construction phase; k=0.06

The maximum displacements come out to be 36.74 cm and 20.82 cm, respectively, which occurred inside the dam body. Since the material properties change at each end of material surface, the maximum displacement occurs at the surface of sand-clay intersection plane.

5.2.2 Just After Full Reservoir

As it is discussed in the previous part, the most critical stage during an earthquake will be the phase of dam which the reservoir is full that is after water is reached to maximum level.

After water fills the reservoir, dam body becomes more rigid to coming earthquakes. Since the water mass on the upstream face supports the dam body, it lets body to move comparatively less than the first case.

Below figures are representing deformations at the end of seismic analyses for 2% and 10% probability of exceedence.

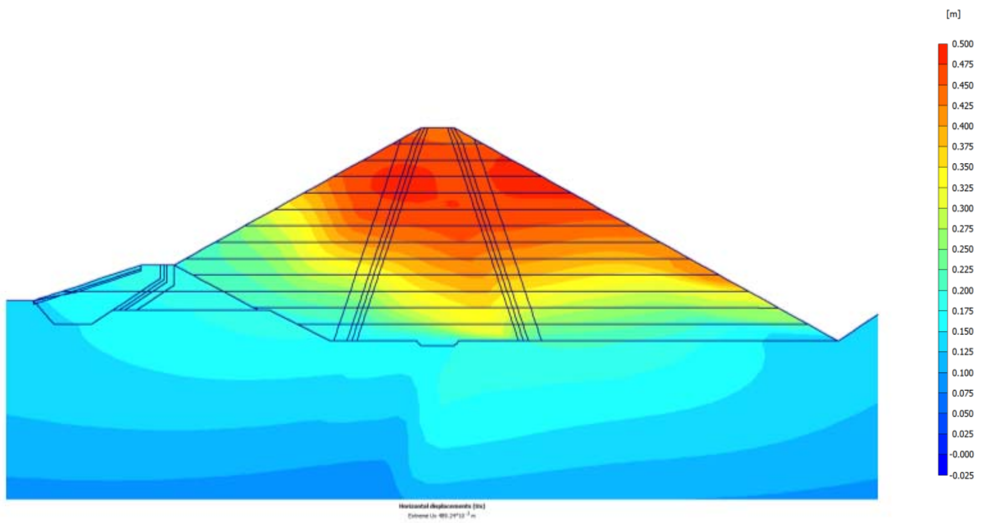

Figure 38: Horizontal displacement occurred at seismic analysis just after full reservoir phase; k=0.1

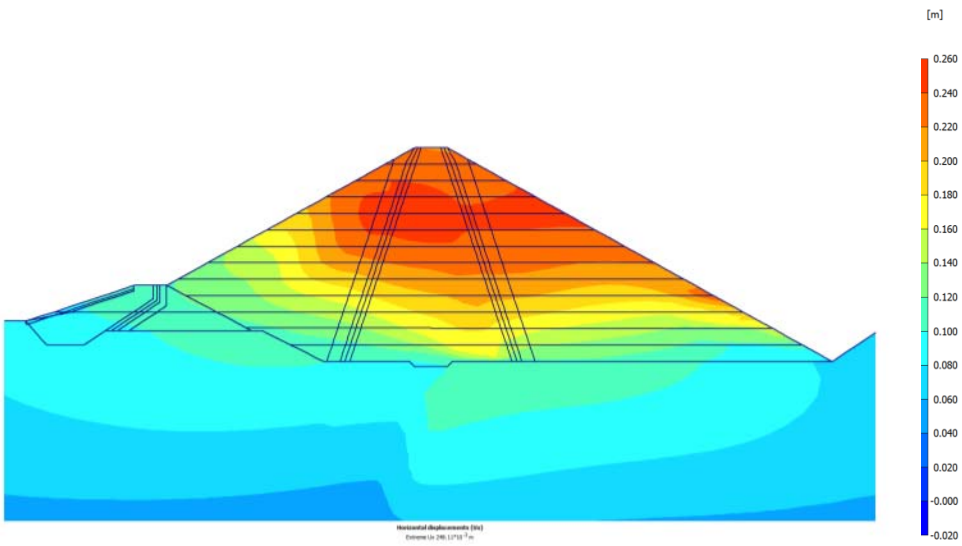

Figure 39: Horizontal displacement occurred at seismic analysis just after full reservoir phase; k=0.06

For the case of full reservoir, the dam body deflects mostly from top part. This change in deflection behavior occurs due to water existence. The maximum deflections are 48.92 cm and 24.81 cm, respectively, for 2% and 10% probability of exceedence in 50 years period.

5.2.3 Longterm Period

In longterm period, clay material parameters changes and turns out to be sandy clay. Because of this reason, behavior of dam also changes and differs from previous part.

The behavior of dam under the same conditions for earthquake is shown below figures.

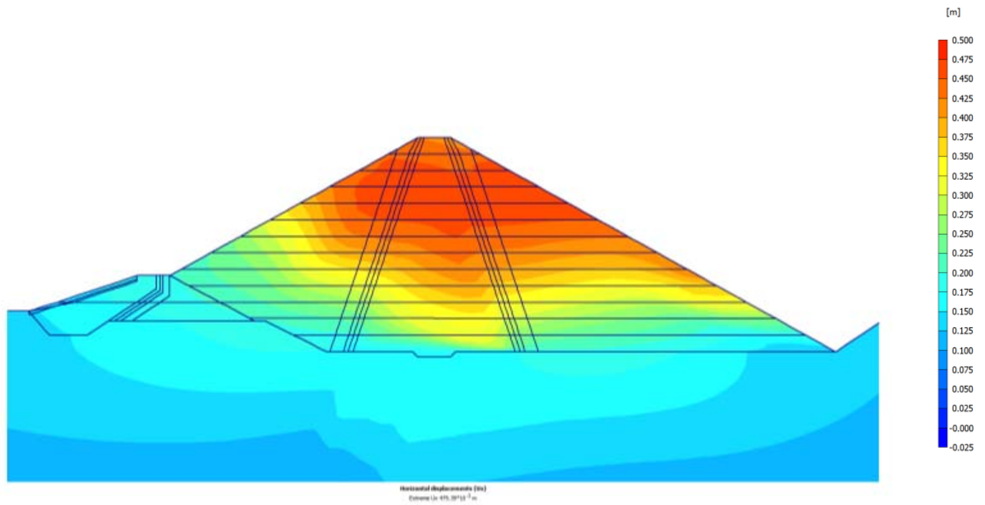

Figure 40: Horizontal displacement occurred at seismic analysis in longterm period; k=0.1

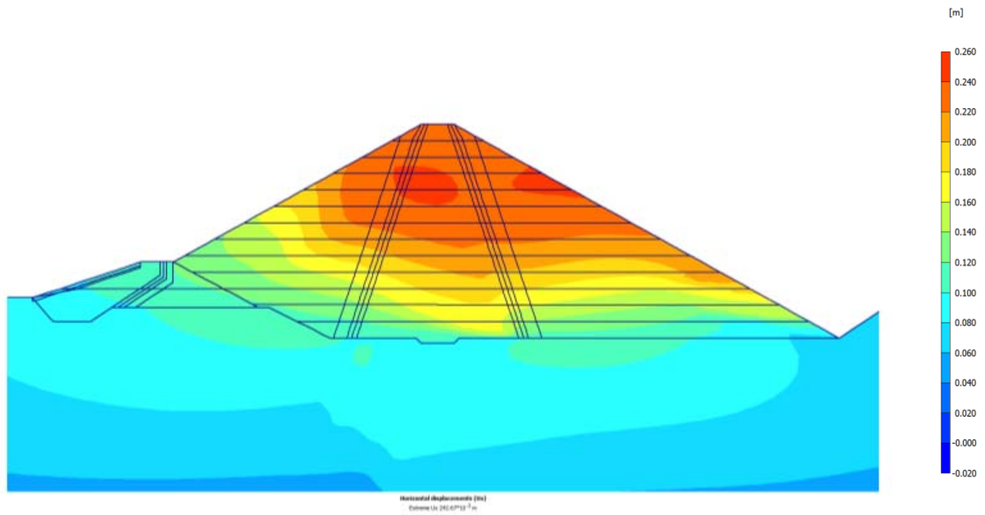

Figure 41: Horizontal displacement occurred at seismic analysis in longterm period; k=0.06

The maximum deformation in the longterm phase will be 47.54 cm and 24.27 cm for 2% probability and 10% probability of exceedence, respectively.

The comparison of the seismic results is given in Table13.

Table 13: Comparison of the seismic results

	Maximum Horizontal Deformations (m)	
	Probability of Exceedence in 50 years	
	2%	10%
Just After Construction	0.367	0.208
Reservoir is Full	0.489	0.248
Longterm Period	0.475	0.243

After Phi/c reduction analysis of the phases i) just after construction finishes, ii) just after reservoir is full and iii) longterm period, the factor safety values are given in Table 14.

Table 14: Factor of Safety values from Phi/c reduction analysis

	Factor of Safety	
	Probability of Exceedence in 50 years	
	2%	10%
Just After Construction	1.259	1.402
Reservoir is Full	1.135	1.249
Longterm Period	1.119	1.260

As it can be seen from Table 14, the most critical phase of the Bahçelik Dam analysis is found as longterm period which has a slight difference with the full reservoir phase for the case of 2% probability of exceedence in 50 years, whereas, full reservoir phase is the most critical one for 10% probability of exceedence in 50 years. Since the latter values are very close to each other, it can be said that the dam is critical at longterm phases.

Since for seismic analysis, required factor of safety is mostly 1.1, the Bahçelik Dam is safe for the used parameters and analysis procedure.

5.3 Seepage Analysis

Seepage Analysis is performed by using PlaxFlow software which is designed for only flow through a soil mass.

Flow analysis is performed by using coefficients of permeability given in Table 9 which are typical values from the literature for the materials, since there was no laboratory or field permeability measurements in these materials. It is assumed that water level in the upstream face shall level up in three stages which is almost realistic. The results according to flow analysis are given below.

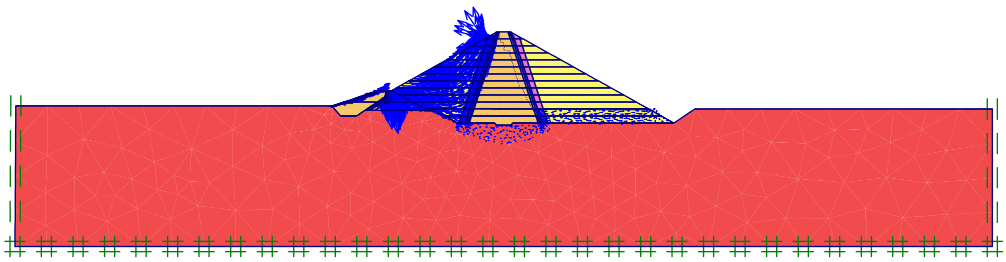

Figure 42: Flow field at full reservoir

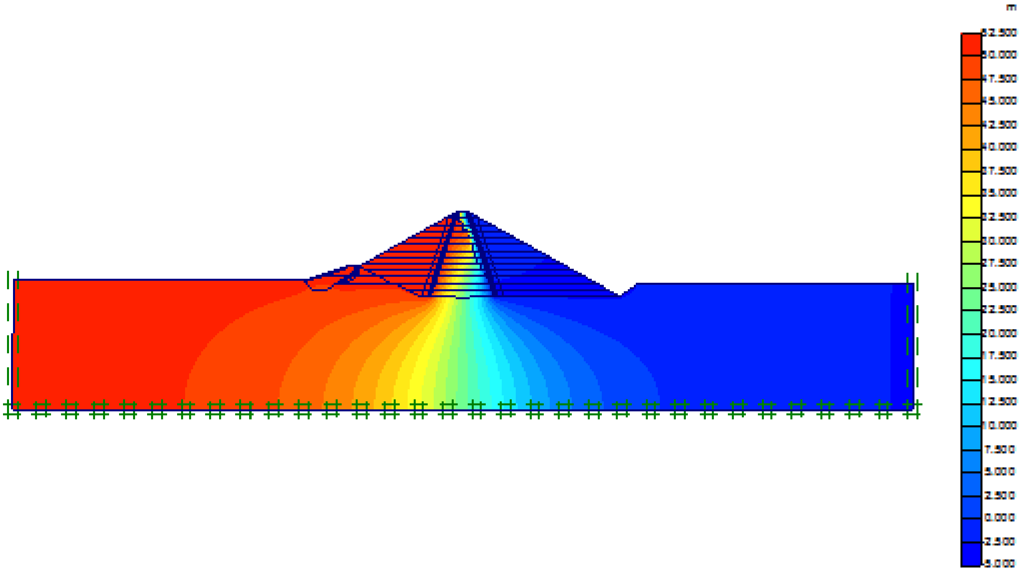

Figure 43: Active water head at full reservoir

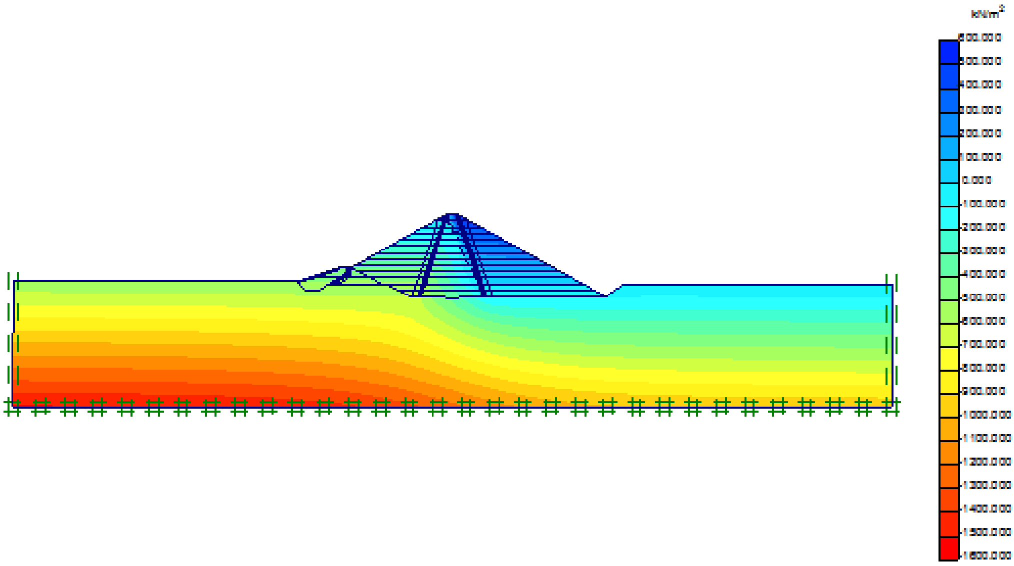

Figure 44: Active pore water pressure at full reservoir

According to results, the mean discharge at tail of the dam is calculated as 1.16E-6 m^3/s/m which equals to 0.1 m^3/day/m water.

If the value shall be compared with another real rockfill dam case with similar geometric and material properties, it can be the Kinda Dam. Typical seepage histogram is given below. According to the histogram, the maximum seepage quantity is recorded as 6 l/s which isequal to 519 m^3/day. However, this value is the total value overall length of the dam. If the dam crest length is 625 m (real value), the flow rate is calculated as 0.83 m^3/day/m (Kutzner, 1997).

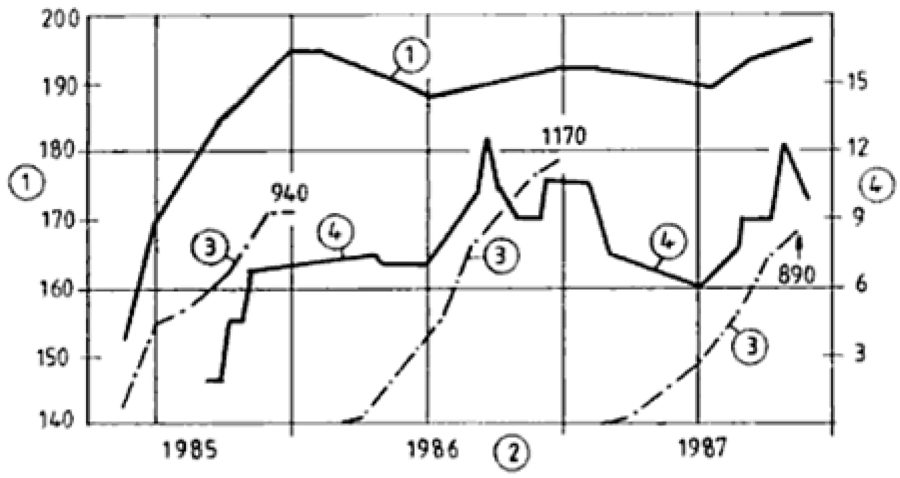

Figure 45: Typical seepage histogram of Kinda Dam (1-Reservoir water level (m a.s.l.), 2-Years of operation, 3-Precipitation (total in mm), 4-Seepage quantity (l/s))

5.4 Pore Pressure Results

Pore pressure controls inside the dam body are performed by using inclinometers that are installed during construction. In Part 3.3.1 inclinometer locations were explained.

The below figures show the active pore pressures in several piezometers.

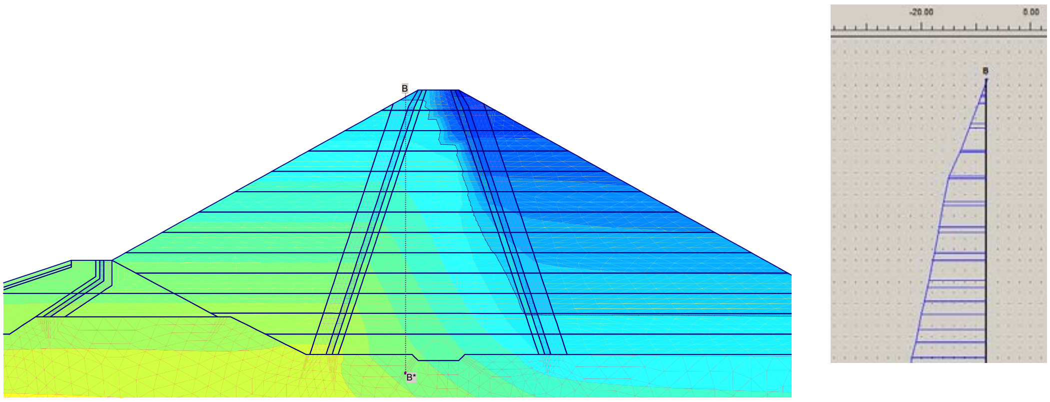

Figure 46: Active pore pressure of No:14 piezometer

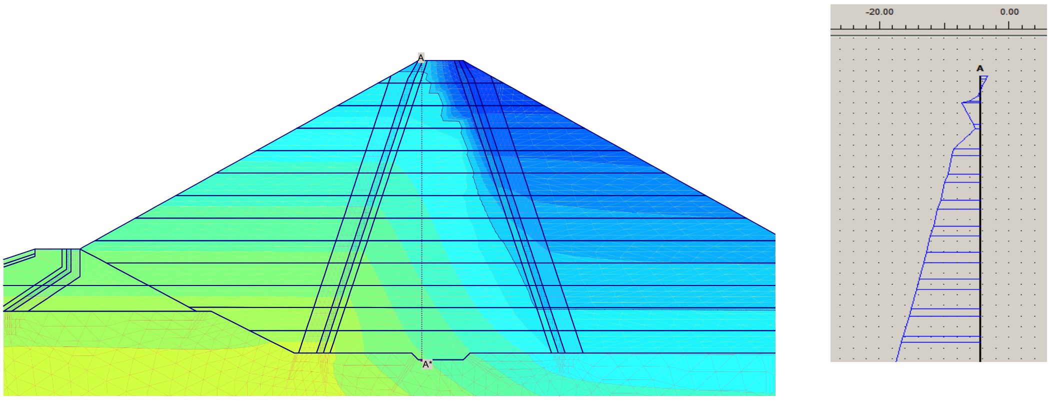

Figure 47: Active pore pressure of No:24 piezometer

In order to compare above figures and the values of pore pressures Table 15 shall be referred. The values show that the pore pressures values calculated by finite element method and measured values in the dam are comparable.

Table 15: Comparison of active pore pressure values

Piezometer No →	No:14	No:24
Computer Modelling (10^1kPa)	~19	~11
Real Dam (10^1kPa)	20.28	8.72

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary

Deformation behavior of a rockfill dam with a clay core is studied in this thesis. Bahçelik Dam which is constructed between 1996 and 2005 near Kayseri Province in Turkey has been chosen as a real case study for this purpose.

Bahçelik Dam is a rockfill dam with a clay core inside and it is 65 m high. The dam stands on Zamantı River and accumulates 216 hm^3 water volumes in normal water level.

Analyses are performed for mainly to understand the deformation behavior of the rockfill dam by using 2D finite element modeling software. The dam model is constructed in 2D plane strain modeling by using elastic-plastic Mohr-Coulomb material model. Deformation behavior of Bahçelik Dam has been evaluated for several cases: i) end of construction, ii) after reservoir is full and iii) after a longtime period. Since the data observed from DSİ do not include any information about the material used for the dam, the material parameters are defined after a series of back analyses. In order to find reasonable material parameters, real case deformation readings taken from actual dam and the deformation data resulting from analyses are compared. Maximum deformation values obtained from the actual and computer model dam are compared and material parameters are adjusted until a better agreement is obtained.

In addition to deformation behavior analyses, factor of safety evaluation for all cases including seismic activity and the behavior of the dam for seepage are also performed.

6.2 Conclusion

For vertical deformations, end of construction settlement is computed, however the measured data of vertical deformations for end of construction are not available for comparison (since the zero time of instruments are after end-of-construction). In this study, for the reservoir full condition, maximum upward movements of about 40 cm have been calculated by the simple elastic plastic Mohr Coulomb material model. In reality, some small upward movements are expected for rockfill dams without impervious upstream face (such as asphalt or concrete). This is because reservoir water acts as an uplifting force causing unloading behavior in the rockfill material, and some vertical upward movements (heave or relaxation) could be observed. It should be noted that in Figure 22 and in the tables given in Appendix D there have been some reported upward movements (up to values of 0.25 m) in Bahcelik dam, especially in the monuments with numbers 1-6 located in the upstream face of the dam. A simple material model cannot take into account the increased stiffness of the rockfill material in the unloading stress path condition therefore could give larger upward movements than expected in real dam. Therefore, in relation to vertical deformations, it is concluded in this study that, for the reservoir full condition, if there is no impervious material at the upstream face of the dam, the vertical movement behavior of the dam cannot be precisely calculated by a simple material model in PLAXIS.

As for the horizontal deformations, comparison of measured and computed horizontal deformations are given in Table 16. When the measured and computed horizontal deformations are compared, it can be seen that the top part of the actual dam deflects less than that of the computer model. This can be due to some operator/reading error in the measured values, or it could be because of the time difference of installation of instruments at the middle and upper part of the dam. According to Hunter and Fell (2003) the typical horizontal displacement in rockfill dams shall be less than 0.2% of the dam height. In this case, the horizontal displacement measurements and computer modeling results are within these approximate values.

Table 16: Comparison of maximum horizontal displacement readings taken from actual dam and computer modeling

Readings from Actual Dam		Readings from Computer Modeling	
Distance from Centerline	Movement (m)	Distance from Centerline	Movement (m)
10 m	0.185	10 m	0.363
40 m	0.381	40 m	0.331
70 m	0.096	70 m	0.241

It can be seen in Table 16 that the computer model, using simple material model, gave about twice the measured horizontal deformations. It should be noted that a recent study by Unsever (2007), using hardening soil model for the rockfill material, concluded that the calculated and measured horizontal deformations in rockfill dams could be within 0.5 to 2 times each other, and this order of magnitude estimation is still considered to be successful. In the current study, a simpler material model (elastic plastic Mohr Coulomb model) is used for the rockfill instead of more parameter-demanding material models, because the former requires less number of input material model parameters to be entered into the analyses. Therefore, in conclusion, an analysis by using a simple material model (after a careful parameter back-analysis) can be considered reasonably successful and the results obtained could be valid and adequate for preliminary evaluation purposes.

Other reasons for the discrepancy in the measured and calculated values in this study could be due to (1) the set goal of only capturing the maximum deformation value measured at a point rather than capturing the deformation behavior throughout the dam, (2) using simple material model for all soils, (3) nonuniform compacting and different material properties in real dam, (4) the possible 3D arching effect in reality due to valley shape which cannot be captured in 2D plane strain analysis in this study (5) inaccuracy in measured deformations and/or inaccuracy in our estimate of the start time of zero deformation reading etc.The behavior of horizontal deformation with distance from centerline is given in Figure 48 (also see Appendix C).

The Bahçelik Dam is also investigated for the dynamic performance. In order to define seismic parameters, NEHRP method is used and then pseudo-static analysis is performed by using PLAXIS Software. The deformation behavior and the factor of safety in dynamic performance are shown in Table 17. The values in the table are results for only seismic activity; the deformations do not include the values from static analysis. According to the results, the dam is safe for all cases since the factor safety is larger than 1.1 which is acceptable

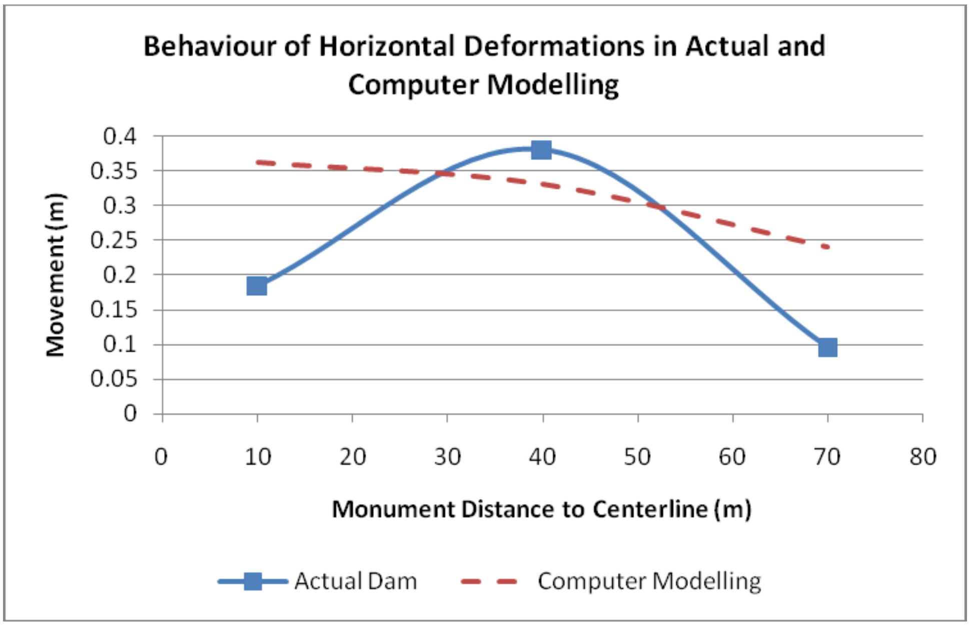

Figure 48: Horizontal displacement behaviors for computer modeling and real case

Table 17: Dynamic performance results

	Maximum Horizontal Deformations (m) / The Factor Of Safety	
	Probability of Exceedence in 50 years	
	2% (2475 years)	10% (475 years)
End of construction	0.367/ 1.259	0.208/ 1.402
Reservoir is Full	0.489 / 1.135	0.248 / 1.249
Longterm Period	0.475 / 1.119	0.243 / 1.260

According to permeability analysis which is done by using PlaxFlow Software the mean discharge is calculated as 0.1 m^3/day/m water.

Recalling back the initial objectives stated at the beginning of this study: the deformations obtained from finite element modelling of a rockfill dam with real measured values are compared. The validity, accuracy and adequeacy of the simple material model is checked. It is concluded that, although it has limitations, a simple elastic plastic Mohr Coulomb material model could predict horizontal deformations within 0.5 to 2 times measured values in clay cored rockfill dams. Pore pressures within the dam body could be predicted quite accurately as long as reasonable values are used for the permeability of rockfill and clay-core materials. Seismic stability and deformations of Bahcelik dam is evaluated and its safety is checked. It should be noted that, the results of such a finite element analyses with simple material model should be used with caution, and only in the preliminary evaluation stage of a project.

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APPENDIX A

STRESS AND STRAIN DIAGRAMS

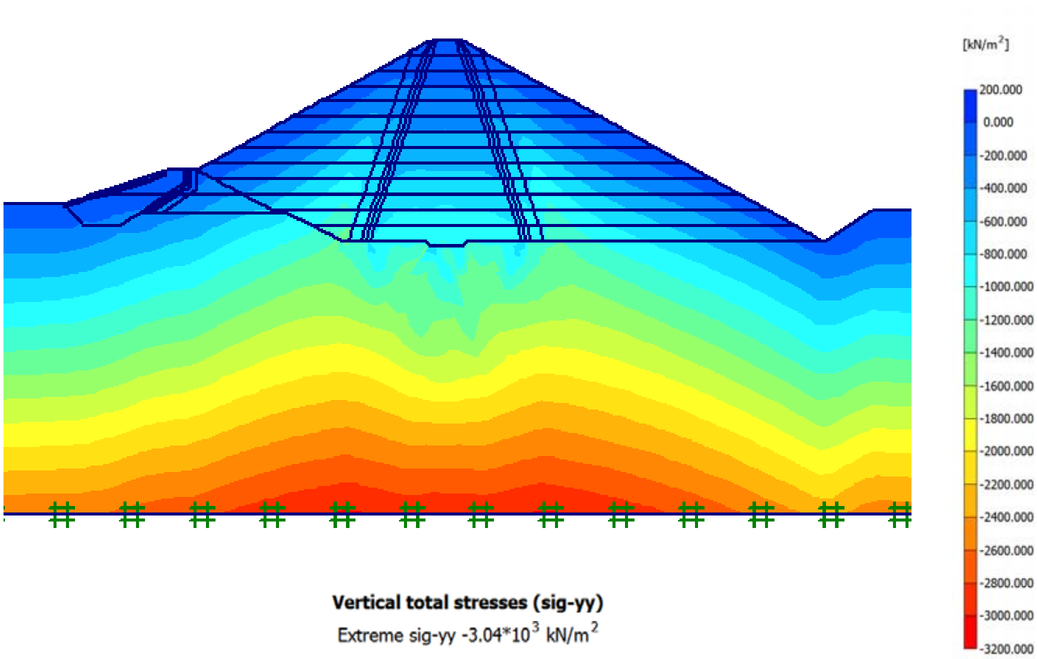

Figure A 1 Vertical total stresses at just after end of construction

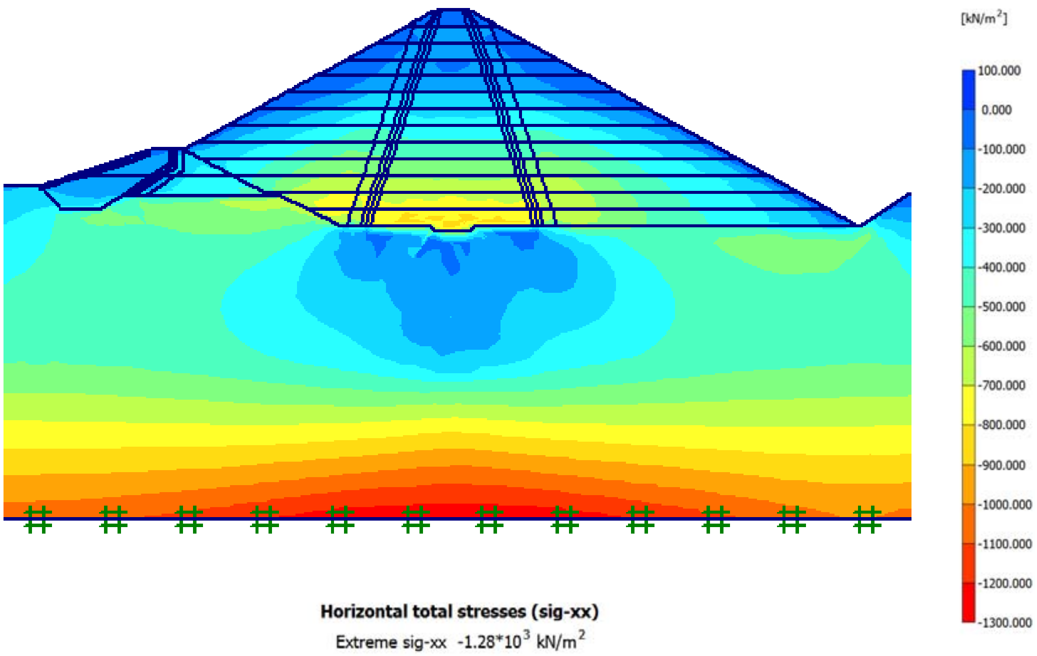

Figure A 2 Horizontal total stresses at just after end of construction

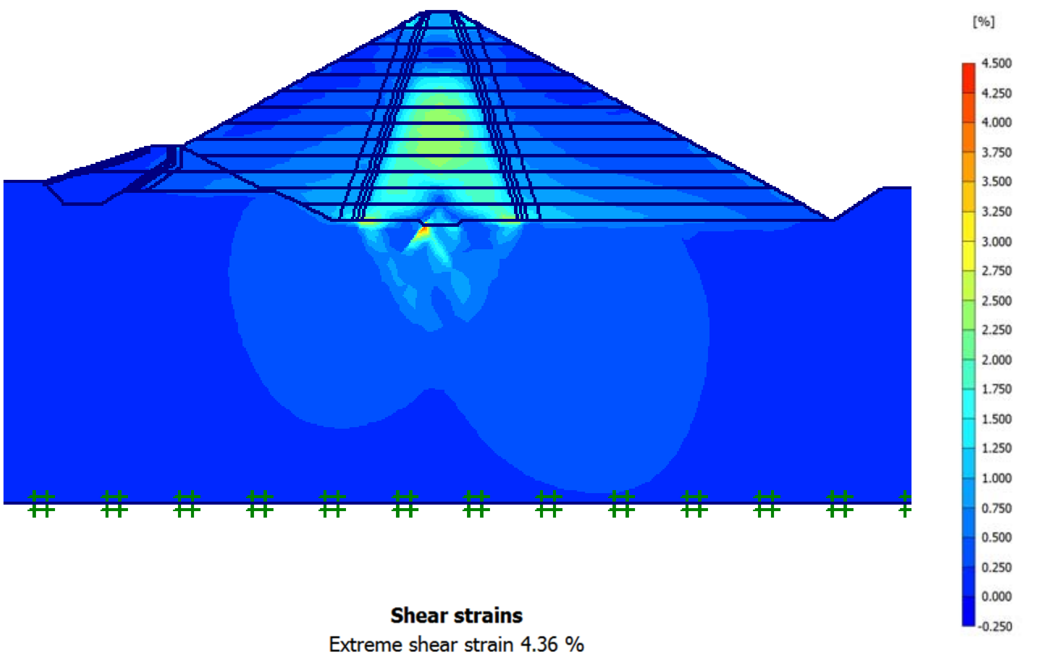

Figure A 3 Shear strain at just after end of construction

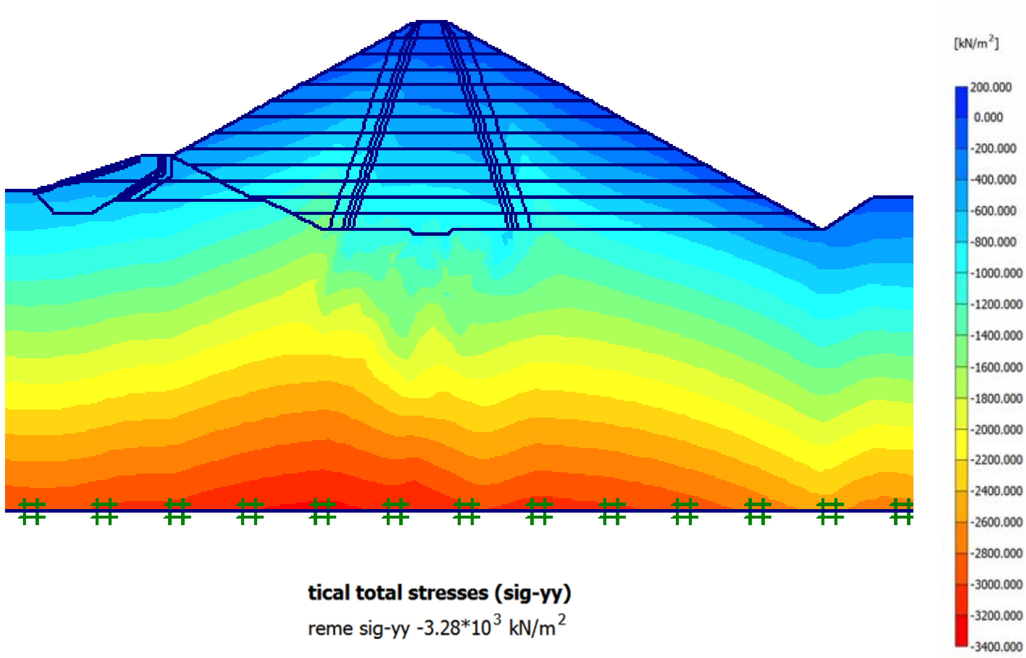

Figure A 4 Vertical total stresses at just after full reservoir

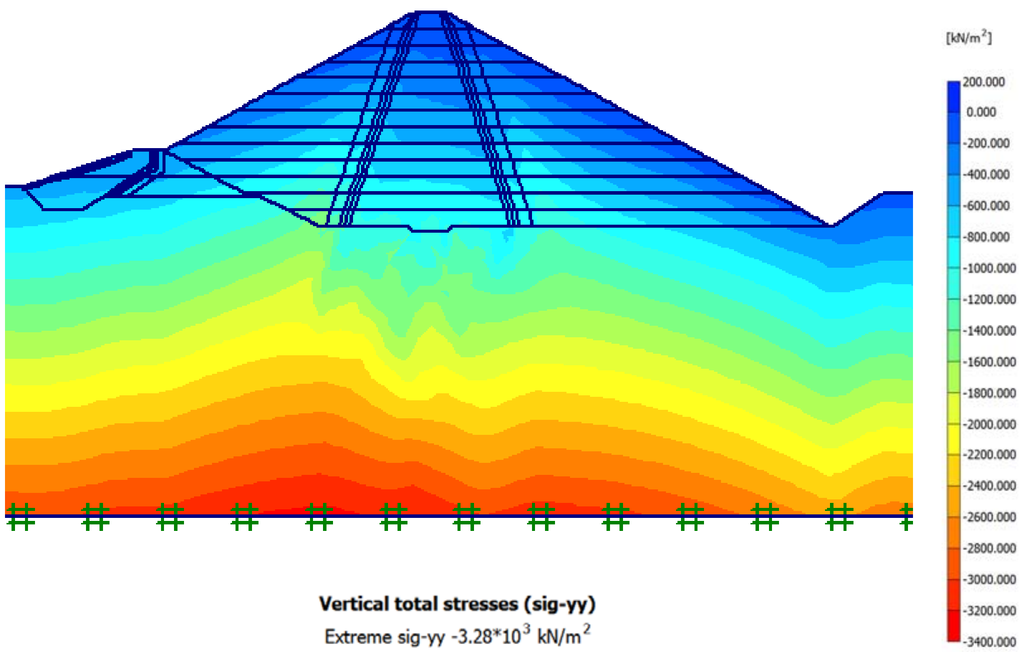

Figure A 5 Horizontal total stresses at just after full reservoir

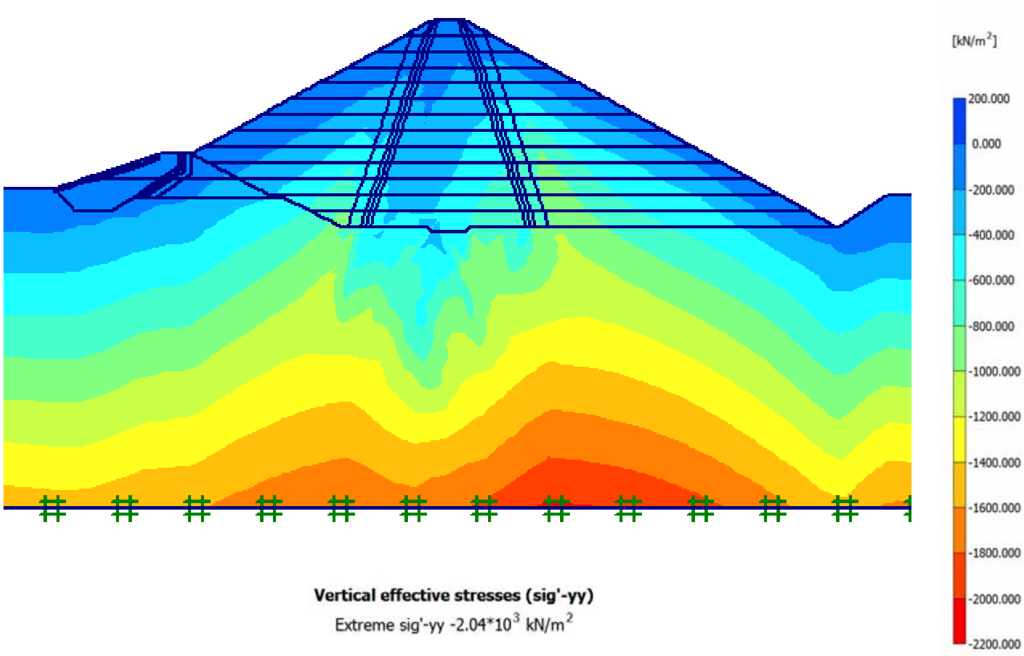

Figure A 6 Vertical effective stresses at just after full reservoir

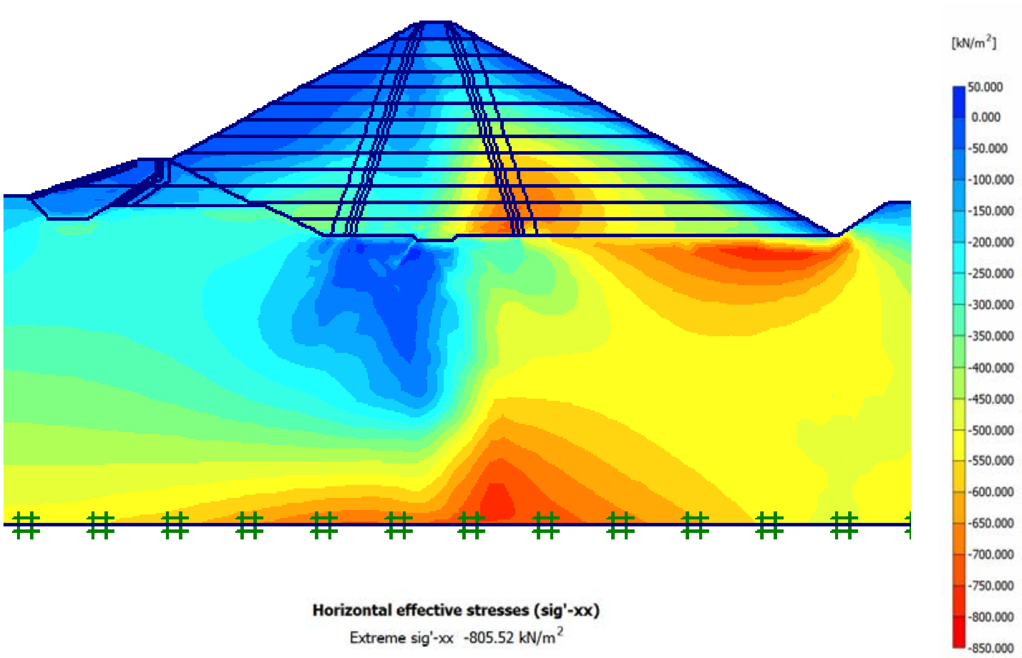

Figure A 7 Horizontal effective stresses at just after full reservoir

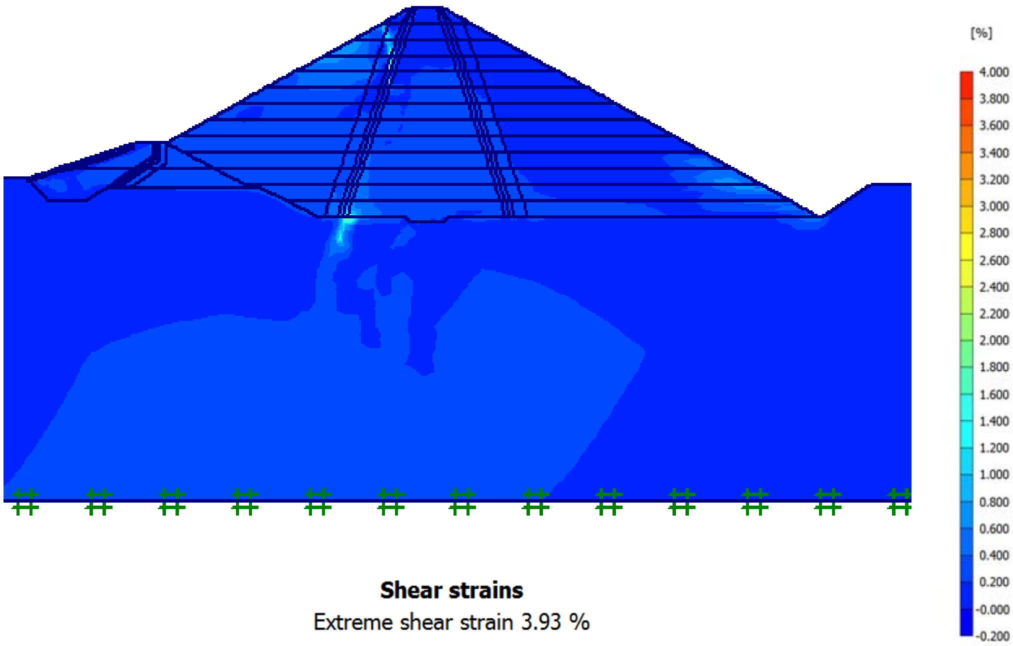

Figure A 8 Shear strain at just after full reservoir

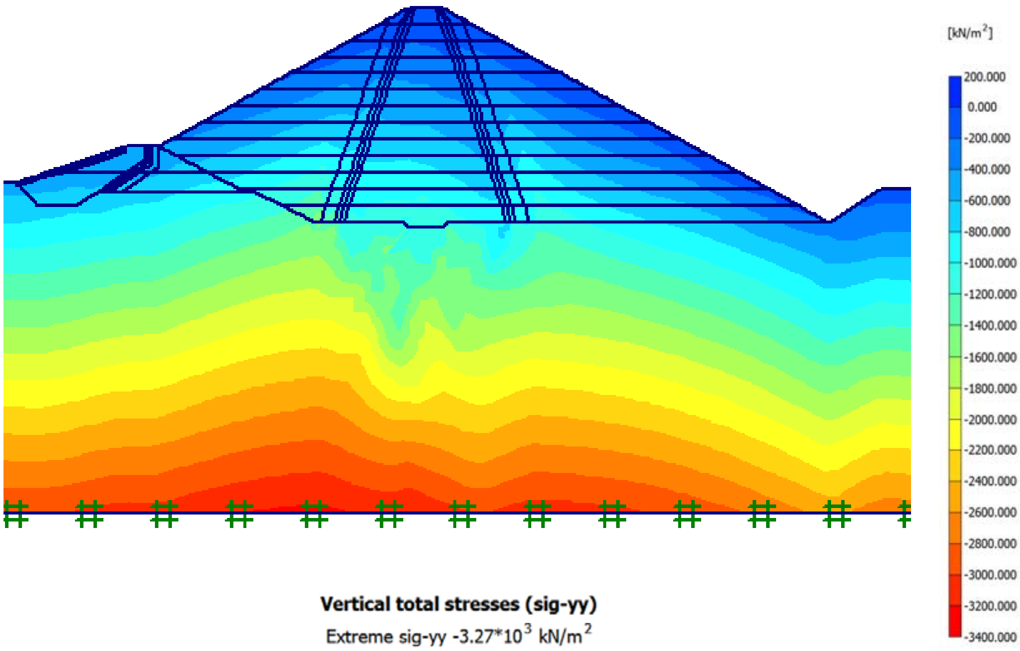

Figure A 9 Vertical total stresses at longterm period

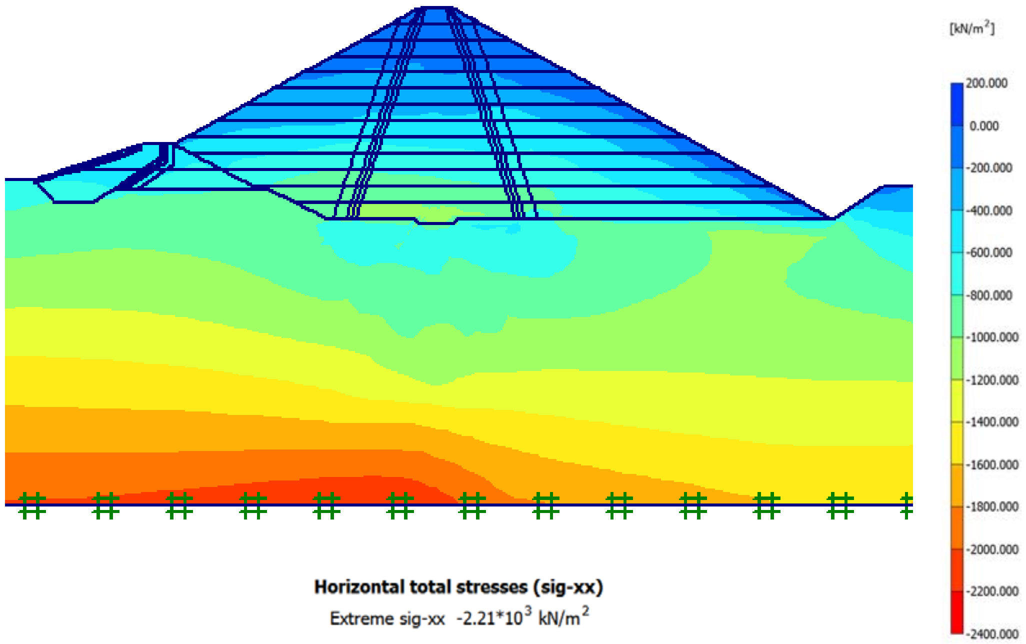

Figure A 10 Horizontal total stresses at longterm period

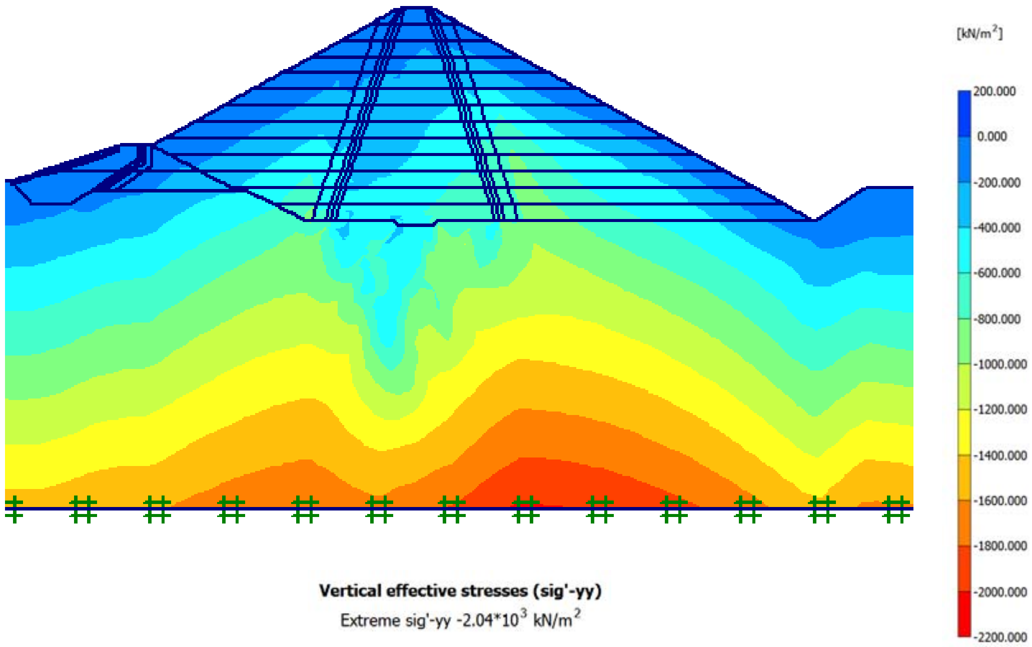

Figure A 11 Vertical effective stresses at longterm period

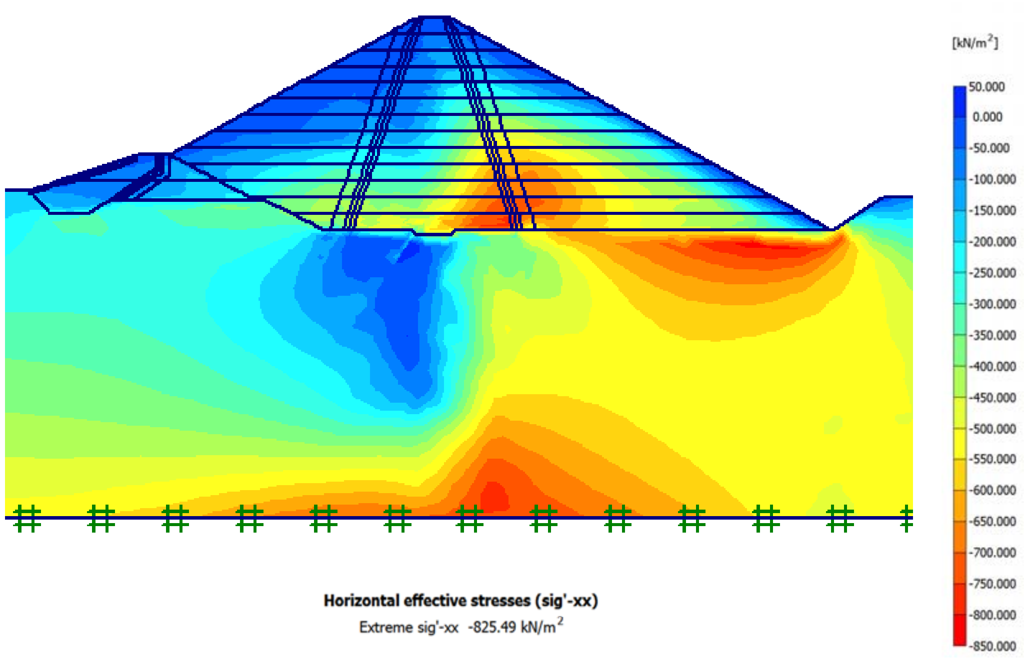

Figure A 12 Horizontal effective stresses at longterm period

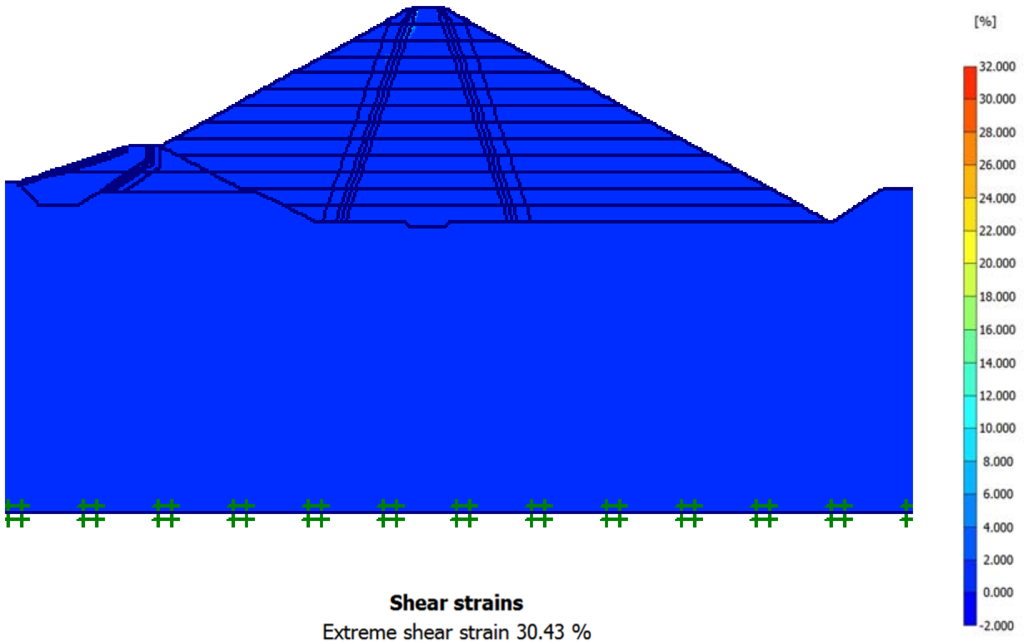

Figure A 13 Shear strain at longterm period

APPENDIX B

STRESS AND STRAIN DIAGRAMS

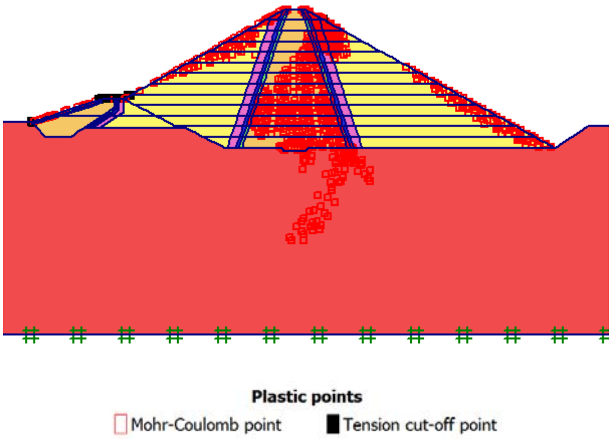

Figure B 1 Plastic points at k=0.06 just after end of construction

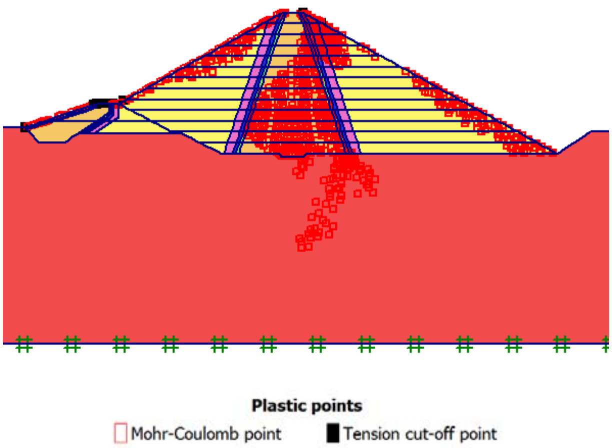

Figure B 2 Plastic points at k=0.1 at just after end of construction

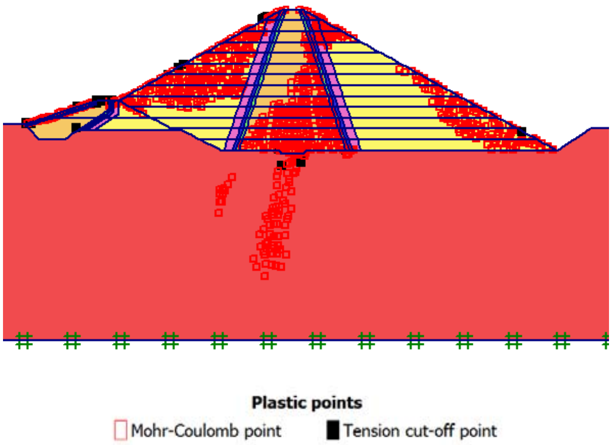

Figure B 3 Plastic points at k=0.06 at just after full reservoir

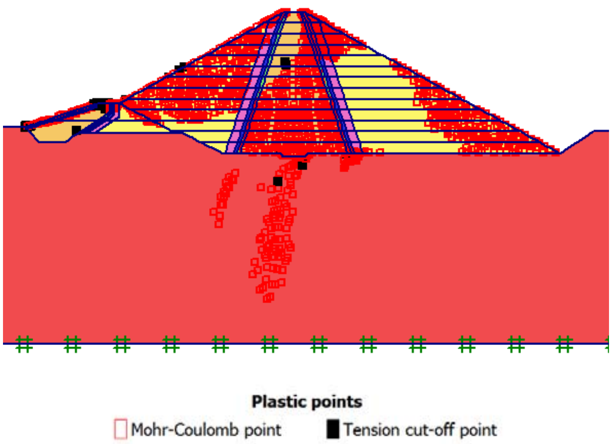

Figure B 4 Plastic points at k=0.1 at just after full reservoir

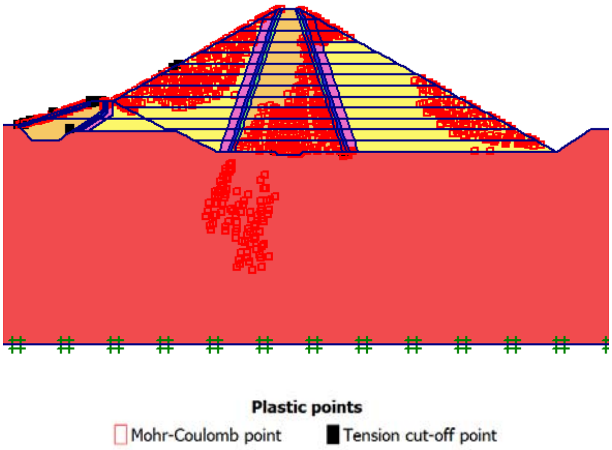

Figure B 5 Plastic points at k=0.06 at long term period

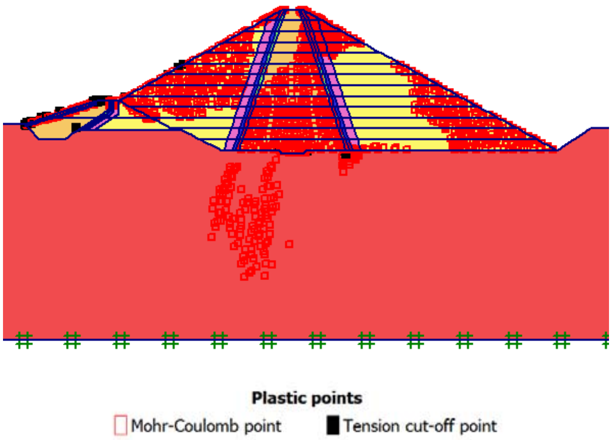

Figure B 6 Plastic points at k=0.1 at long term period

APPENDIX C

THE PROCEDURE OF DISPLACEMENT CALCULATION

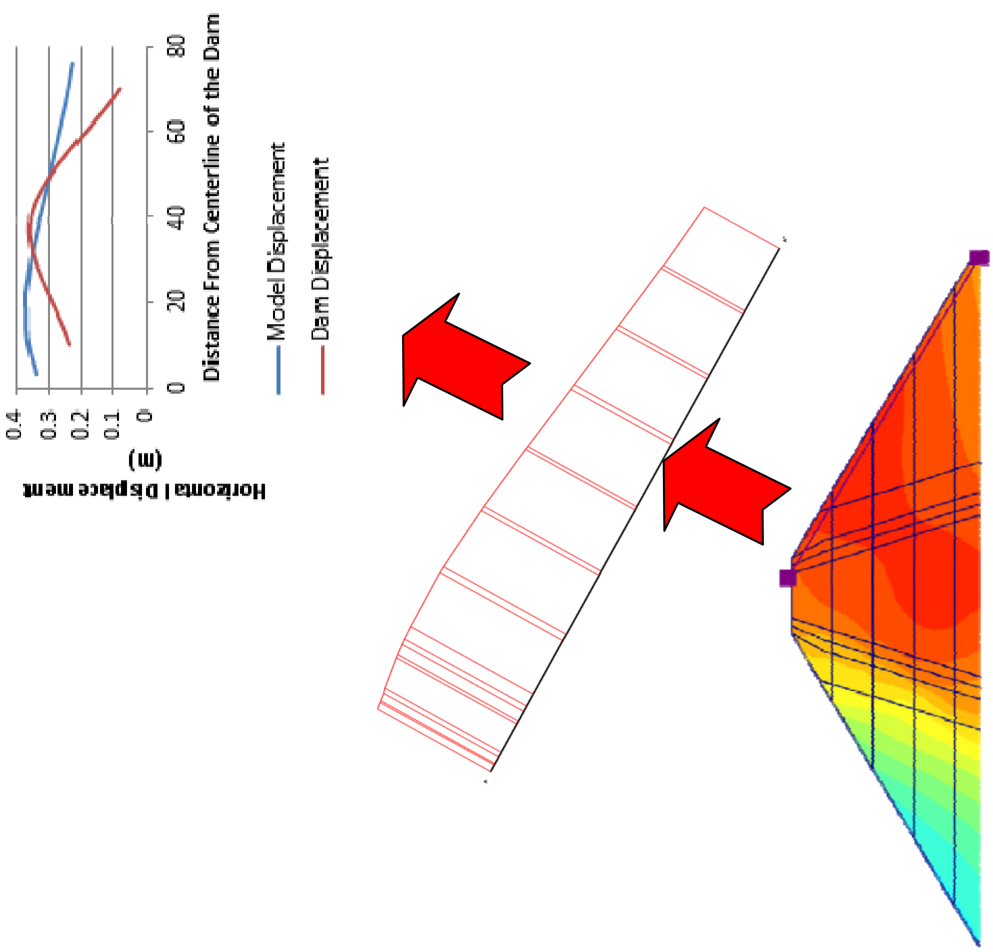

Figure C1 Comparison of horizontal deformation readings from computer modelling and real dam (KM 0+110)

APPENDIX D

DATA OF BAHÇELİK DAM

Table D 1: Field Data No:1

BAHÇELİK DAM SURFACE MONUMENTS							
LAKE WATER LEVEL :	1489.000				DSİ XII. REGION DIRECTORATE		
DATE OF MEASUREMENT :	04.12.2008				BAHÇELİK DAM		
MEASURED BY :	Ali KOCAOĞLU						
NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT
		FIRST	LAST	QUANTITY AND DIRECTION	FIRST	LAST	QUANTITY AND DIRECTION
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.885	-0.203	1501.371	1501.122	-0.249
2	0+110	10.079	9.885	-0.194	1501.156	1500.879	-0.277
3	0+150	10.036	9.830	-0.206	1501.676	1501.400	-0.276
4	0+200	10.047	9.873	-0.174	1501.553	1501.281	-0.272
5	0+260	10.039	9.886	-0.153	1501.181	1500.901	-0.280
6	0+330	10.020	9.970	-0.050	1501.630	1501.524	-0.106
7	0+060	10.311	10.610	0.299	1501.658	1501.495	-0.163
8	0+110	10.277	10.540	0.263	1501.665	1501.470	-0.195
9	0+150	10.255	10.500	0.245	1501.936	1501.737	-0.199
10	0+200	10.180	10.353	0.173	1501.972	1501.760	-0.212
11	0+260	10.147	10.205	0.058	1501.902	1501.707	-0.195
12	0+330	10.098	10.085	-0.013	1502.243	1502.125	-0.118

Table D 1 continue

13	0+060	40.309	40.707	0.398	1485.035	1484.962	-0.073
14	0+110	40.271	40.663	0.392	1484.673	1484.587	-0.086
15	0+150	40.249	40.608	0.359	1484.355	1484.258	-0.097
16	0+200	40.205	40.500	0.295	1484.378	1484.248	-0.130
17	0+260	40.153	40.360	0.207	1484.861	1484.750	-0.111
18	0+330	40.086	40.162	0.076	1485.780	1485.743	-0.037
19	0+110	70.235	70.330	0.095	1468.462	1468.403	-0.059
20	0+150	70.181	70.292	0.111	1468.570	1468.473	-0.097
21	0+200	70.154	70.160	0.006	1468.673	1468.576	-0.097
22	0+260	70.111	70.170	0.059	1469.100	1469.052	-0.048

BAHÇELİK DAM SURFACE MONUMENTS

LAKE WATER LEVEL : 1488.098 — DSİ XII. REGION DIRECTORATE

DATE OF MEASUREMENT : 14.10.2008 — BAHÇELİK DAM

MEASURED BY : Ali KOCAOĞLU

NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT
		İLK	SON	QUANTITY AND DIRECTION	FIRST	LAST	QUANTITY AND DIRECTION
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.847	-0.241	1501.371	1501.179	-0.192
2	0+110	10.079	9.906	-0.173	1501.156	1500.925	-0.231
3	0+150	10.036	9.855	-0.181	1501.676	1501.470	-0.206
4	0+200	10.047	9.943	-0.104	1501.553	1501.322	-0.231
5	0+260	10.039	9.993	-0.046	1501.181	1500.947	-0.234
6	0+330	10.020	10.000	-0.020	1501.630	1501.564	-0.066
7	0+060	10.311	10.550	0.239	1501.658	1501.531	-0.127
8	0+110	10.277	10.490	0.213	1501.665	1501.506	-0.159
9	0+150	10.255	10.445	0.190	1501.936	1501.773	-0.163
10	0+200	10.180	10.300	0.120	1501.972	1501.796	-0.176
11	0+260	10.147	10.169	0.022	1501.902	1501.743	-0.159

Table D 1 continue

12	0+330	10.098	10.056	-0.042	1502.243	1502.162	-0.081
13	0+060	40.309	40.646	0.337	1485.035	1484.994	-0.041
14	0+110	40.271	40.600	0.329	1484.673	1484.622	-0.051
15	0+150	40.249	40.546	0.297	1484.355	1484.292	-0.063
16	0+200	40.205	40.466	0.261	1484.378	1484.286	-0.092
17	0+260	40.153	10.315	-29.838	1484.861	1484.808	-0.053
18	0+330	40.086	40.136	0.050	1485.780	1485.777	-0.003
19	0+110	70.235	70.326	0.091	1468.462	1468.432	-0.030
20	0+150	70.181	70.226	0.045	1468.570	1468.517	-0.053
21	0+200	70.154	70.195	0.041	1468.673	1468.614	-0.059
22	0+260	70.111	70.124	0.013	1469.100	1469.058	-0.042

BAHÇELİK DAM SURFACE MONUMENTS

LAKE WATER LEVEL : 1490.810 **DSİ XII. REGION DIRECTORATE**

DATE OF MEASUREMENT: 14,08,2008 **BAHÇELİK DAM**

MEASURED BY: Ali KOCAOĞLU

NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT QUANTITY AND DIRECTION	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT QUANTITY AND DIRECTION
		İLK	SON		FIRST	LAST	
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.846	-0.242	1501.371	1501.138	-0.233
2	0+110	10.079	9.900	-0.179	1501.156	1500.884	-0.272
3	0+150	10.036	9.850	-0.186	1501.676	1501.393	-0.283
4	0+200	10.047	9.925	-0.122	1501.553	1501.269	-0.284
5	0+260	10.039	9.964	-0.075	1501.181	1500.883	-0.298
6	0+330	10.020	9.965	-0.055	1501.630	1501.485	-0.145
7	0+060	10.311	10.532	0.221	1501.658	1501.510	-0.148
8	0+110	10.277	10.465	0.188	1501.665	1501.489	-0.176
9	0+150	10.255	10.440	0.185	1501.936	1501.758	-0.178
10	0+200	10.180	10.307	0.127	1501.972	1501.779	-0.193

Table D 1 continue

11	0+260	10.147	10.187	0.040	1501.902	1501.726	-0.176
12	0+330	10.098	10.081	-0.017	1502.243	1502.135	-0.108
13	0+060	40.309	40.610	0.301	1485.035	1484.973	-0.062
14	0+110	40.271	40.581	0.310	1484.673	1484.602	-0.071
15	0+150	40.249	40.537	0.288	1484.355	1484.275	-0.080
16	0+200	40.205	40.564	0.359	1484.378	1484.292	-0.086
17	0+260	40.153	40.337	0.184	1484.861	1484.789	-0.072
18	0+330	40.086	40.161	0.075	1485.780	1485.753	-0.027
19	0+110	70.235	70.276	0.041	1468.462	1468.434	-0.028
20	0+150	70.181	70.308	0.127	1468.570	1468.520	-0.050
21	0+200	70.154	70.197	0.043	1468.673	1468.623	-0.050
22	0+260	70.111	70.148	0.037	1469.100	1469.066	-0.034

BAHÇELİK DAM SURFACE MONUMENTS

LAKE WATER LEVEL : 1496.960 DSİ XII. REGION DIRECTORATE

DATE OF MEASUREMENT : 22.04.2009 BAHÇELİK DAM

MEASURED BY : Ali KOCAOĞLU

NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT
		İLK	SON	QUANTITY AND DIRECTION	FIRST	LAST	QUANTITY AND DIRECTION
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.882	-0.206	1501.371	1501.110	0.261
2	0+110	10.079	9.984	-0.095	1501.156	1500.877	0.279
3	0+150	10.036	9.843	-0.193	1501.676	1501.402	0.274
4	0+200	10.047	9.887	-0.160	1501.553	1501.283	0.270
5	0+260	10.039	9.910	-0.129	1501.181	1500.897	0.284
6	0+330	10.020	9.900	-0.120	1501.630	1501.515	0.115
7	0+060	10.311	10.565	0.254	1501.658	1501.495	0.163
8	0+110	10.277	10.508	0.231	1501.665	1501.479	0.186
9	0+150	10.255	10.481	0.226	1501.936	1501.747	0.189

Table D 1 continue

10	0+200	10.180	10.338	0.158	1501.972	1501.767	0.205
11	0+260	10.147	10.206	0.059	1501.902	1501.714	0.188
12	0+330	10.098	10.088	-0.010	1502.243	1502.134	0.109
13	0+060	40.309	40.657	0.348	1485.035	1484.970	0.065
14	0+110	40.271	40.629	0.358	1484.673	1484.593	0.080
15	0+150	40.249	40.586	0.337	1484.355	1484.264	0.091
16	0+200	40.205	40.634	0.429	1484.378	1484.253	0.125
17	0+260	40.153	40.360	0.207	1484.861	1484.791	0.070
18	0+330	40.086	40.166	0.080	1485.780	1485.752	0.028
19	0+110	70.235	70.344	0.109	1468.462	1468.408	0.054
20	0+150	70.181	70.364	0.183	1468.570	1468.484	0.086
21	0+200	70.154	70.269	0.115	1468.673	1468.598	0.075
22	0+260	70.111	70.207	0.096	1469.100	1469.033	0.067

BAHÇELİK DAM SURFACE MONUMENTS

LAKE WATER LEVE : 1497.640 — DSİ XII. REGION DIRECTORATE

DATE OF MEASUREMENT : 17.06.2009 — BAHÇELİK DAM

MEASURED BY: Ali KOCAOĞLU

NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT
		İLK	SON	QUANTITY AND DIRECTION	FIRST	LAST	QUANTITY AND DIRECTION
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.795	-0.293	1501.371	1501.054	0.317
2	0+110	10.079	9.840	-0.239	1501.156	1500.810	0.346
3	0+150	10.036	9.810	-0.226	1501.676	1501.346	0.330
4	0+200	10.047	9.894	-0.153	1501.553	1501.222	0.331
5	0+260	10.039	9.935	-0.104	1501.181	1500.833	0.348
6	0+330	10.020	9.967	-0.053	1501.630	1501.472	0.158
7	0+060	10.311	10.604	0.293	1501.658	1501.470	0.188
8	0+110	10.277	10.554	0.277	1501.665	1501.453	0.212

Table D 1 continue

9	0+150	10.255	10.514	0.259	1501.936	1501.722	0.214
10	0+200	10.180	10.365	0.185	1501.972	1501.744	0.228
11	0+260	10.147	10.223	0.076	1501.902	1501.690	0.212
12	0+330	10.098	10.072	-0.026	1502.243	1502.087	0.156
13	0+060	40.309	40.700	0.391	1485.035	1484.950	0.085
14	0+110	40.271	40.668	0.397	1484.673	1484.572	0.101
15	0+150	40.249	40.630	0.381	1484.355	1484.240	0.115
16	0+200	40.205	40.661	0.456	1484.378	1484.230	0.148
17	0+260	40.153	40.375	0.222	1484.861	1484.770	0.091
18	0+330	40.086	40.165	0.079	1485.780	1485.747	0.033
19	0+110	70.235	70.404	0.169	1468.462	1468.391	0.071
20	0+150	70.181	70.388	0.207	1468.570	1468.467	0.103
21	0+200	70.154	70.231	0.077	1468.673	1468.610	0.063
22	0+260	70.111	70.165	0.054	1469.100	1469.056	0.044

BAHÇELİK DAM SURFACE MONUMENTS

LAKE WATER LEVEL : 1496.540 DSİ XII. REGION DIRECTORATE

DATE OF MEASUREMENT: 06.08.2009 BAHÇELİK DAM

MEASURED BY : Ali KOCAOĞLU

NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT
		İLK	SON	QUANTITY AND DIRECTION	FIRST	LAST	QUANTITY AND DIRECTION
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.850	-0.238	1501.371	1501.090	0.281
2	0+110	10.079	9.870	-0.209	1501.156	1500.844	0.312
3	0+150	10.036	9.817	-0.219	1501.676	1501.370	0.306
4	0+200	10.047	9.872	-0.175	1501.553	1501.251	0.302
5	0+260	10.039	9.872	-0.167	1501.181	1500.869	0.312
6	0+330	10.020	9.867	-0.153	1501.630	1501.512	0.118
7	0+060	10.311	10.582	0.271	1501.658	1501.469	0.189

Table D 1 continue

8	0+110	10.277	10.525	0.248	1501.665	1501.448	0.217
9	0+150	10.255	10.491	0.236	1501.936	1501.714	0.222
10	0+200	10.180	10.345	0.165	1501.972	1501.735	0.237
11	0+260	10.147	10.203	0.056	1501.902	1501.683	0.219
12	0+330	10.098	10.073	-0.025	1502.243	1502.110	0.133
13	0+060	40.309	40.676	0.367	1485.035	1484.948	0.087
14	0+110	40.271	40.645	0.374	1484.673	1484.563	0.110
15	0+150	40.249	40.595	0.346	1484.355	1484.232	0.123
16	0+200	40.205	40.581	0.376	1484.378	1484.251	0.127
17	0+260	40.153	40.358	0.205	1484.861	1484.760	0.101
18	0+330	40.086	40.150	0.064	1485.780	1485.737	0.043
19	0+110	70.235	70.291	0.056	1468.462	1468.410	0.052
20	0+150	70.181	70.276	0.095	1468.570	1468.500	0.070
21	0+200	70.154	70.226	0.072	1468.673	1468.605	0.068
22	0+260	70.111	70.155	0.044	1469.100	1469.050	0.050

BAHÇELİK DAM SURFACE MONUMENTS

LAKE WATER LEVEL : 1494.300 **DSİ XII. REGION DIRECTORATE**

DATE OF MEASUREMENT: 28.09.2009 **BAHÇELİK DAM**

MEASURED BY : Ali KOCAOĞLU

NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT
		İLK	SON	QUANTITY AND DIRECTION	FIRST	LAST	QUANTITY AND DIRECTION
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.799	-0.289	1501.371	1501.091	0.280
2	0+110	10.079	9.839	-0.240	1501.156	1500.851	0.305
3	0+150	10.036	9.801	-0.235	1501.676	1501.375	0.301
4	0+200	10.047	9.876	-0.171	1501.553	1501.259	0.294
5	0+260	10.039	9.929	-0.110	1501.181	1500.872	0.309
6	0+330	10.020	9.971	-0.049	1501.630	1501.512	0.118

Table D 1 continue

7	0+060	10.311	10.599	0.288	1501.658	1501.472	0.186
8	0+110	10.277	10.539	0.262	1501.665	1501.454	0.211
9	0+150	10.255	10.496	0.241	1501.936	1501.722	0.214
10	0+200	10.180	10.359	0.179	1501.972	1501.742	0.230
11	0+260	10.147	10.211	0.064	1501.902	1501.691	0.211
12	0+330	10.098	10.069	-0.029	1502.243	1502.120	0.123
13	0+060	40.309	40.695	0.386	1485.035	1484.958	0.077
14	0+110	40.271	40.661	0.390	1484.673	1484.574	0.099
15	0+150	40.249	40.609	0.360	1484.355	1484.242	0.113
16	0+200	40.205	40.585	0.380	1484.378	1484.236	0.142
17	0+260	40.153	40.364	0.211	1484.861	1484.771	0.090
18	0+330	40.086	40.150	0.064	1485.780	1485.752	0.028
19	0+110	70.235	70.325	0.090	1468.462	1468.406	0.056
20	0+150	70.181	70.315	0.134	1468.570	1468.479	0.091
21	0+200	70.154	70.253	0.099	1468.673	1468.585	0.088
22	0+260	70.111	70.176	0.065	1469.100	1469.009	0.091

BAHÇELİK DAM SURFACE MONUMENTS

LAKE WATER LEVEL : 1492.850 DSİ XII. REGION DIRECTORATE

DATE OF MEASUREMENT : 18.11.2009 BAHÇELİK DAM

MEASURED BY : Ali KOCAOĞLU

NO	STATION KM	DISTANCE TO CENTER OF THE DAM (m)		MONUMENTS HORIZONTAL MOVEMENT	ELEVATION (m)		MONUMENTS VERTICAL MOVEMENT
		İLK	SON	QUANTITY AND DIRECTION	FISRT	LAST	QUANTITY AND DIRECTION
1	2	3	4	(5)=4-3	6	7	(8)=6-7
1	0+060	10.088	9.780	-0.308	1501.371	1501.094	0.277
2	0+110	10.079	9.890	-0.189	1501.156	1500.846	0.310
3	0+150	10.036	9.860	-0.176	1501.676	1501.376	0.300
4	0+200	10.047	9.890	-0.157	1501.553	1501.252	0.301
5	0+260	10.039	9.830	-0.209	1501.181	1500.861	0.320

Table D 1 continue

6	0+330	10.020	9.930	-0.090	1501.630	1501.498	0.132
7	0+060	10.311	10.510	0.199	1501.658	1501.472	0.186
8	0+110	10.277	10.480	0.203	1501.665	1501.460	0.205
9	0+150	10.255	10.440	0.185	1501.936	1501.726	0.210
10	0+200	10.180	10.300	0.120	1501.972	1501.754	0.218
11	0+260	10.147	10.170	0.023	1501.902	1501.692	0.210
12	0+330	10.098	10.062	-0.036	1502.243	1502.123	0.120
13	0+060	40.309	40.600	0.291	1485.035	1484.953	0.082
14	0+110	40.271	40.580	0.309	1484.673	1484.574	0.099
15	0+150	40.249	40.530	0.281	1484.355	1484.250	0.105
16	0+200	40.205	40.436	0.231	1484.378	1484.277	0.101
17	0+260	40.153	40.330	0.177	1484.861	1484.777	0.084
18	0+330	40.086	40.130	0.044	1485.780	1485.754	0.026
19	0+110	70.235	70.220	-0.015	1468.462	1468.426	0.036
20	0+150	70.181	70.190	0.009	1468.570	1468.513	0.057
21	0+200	70.154	70.160	0.006	1468.673	1468.618	0.055
22	0+260	70.111	70.100	0.065	1469.100	1469.062	0.038

PIEZOMETER REASINGS

LAKE WATER LEVEL **1491.69**

DOWNSTREAM WATER LEVEL **:**

ZAMANTI PROJECT

DATE OF MEASUREMENT **15.12.2009**

BAHÇELİK DAM

MEASURED BY **Adnan SEYHAN**

NO.	MANOMETRE READINGS FORWARD	BACKWARD	AVERAGE	TIP CONSTANT (m)	AVERAGE PRESSURE AT TIP (m)	PİYEZO. TIPELEVATION (m)	PİYEZOMET. WATER LEVEL INSIDE (m)	NOT
1	2	3	4(*)	5	6(*)	7	8(*)	
T-1	0.00	0.00	0.00	34.81	34.81	1438.47	1473.28	Clogged
T-2	0.00	0.00	0.00	36.92	36.92	1436.36	1473.28	No backward
1	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No pressure

Table D 1 continue

2	0.02	0.02	0.02	20.28	20.48	1453.00	1473.48	
3	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No pressure
4	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No pressure
5	0.02	0.02	0.02	20.28	20.48	1453.00	1473.48	
6	0.04	0.04	0.04	20.28	20.68	1453.00	1473.68	
7	0.03	0.03	0.03	0.28	0.58	1473.00	1473.58	
8	0.02	0.02	0.02	0.28	0.48	1473.00	1473.48	No pressure
9	0.02	0.02	0.02	0.28	0.48	1473.00	1473.48	
10	0.02	0.02	0.02	0.28	0.48	1473.00	1473.48	
11	0.00	0.00	0.00	-9.72	-9.72	1483.00	1473.28	No pressure
12	0.02	0.02	0.02	-9.72	-9.52	1483.00	1473.48	
13	0.02	0.02	0.02	-9.72	-9.52	1483.00	1473.48	
14	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backward
15	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backward
16	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No pressure
17	0.02	0.02	0.02	20.28	20.48	1453.00	1473.48	
18	0.04	0.04	0.04	20.28	20.68	1453.00	1473.68	
19	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backward
20	0.00	0.00	0.00	0.28	0.28	1473.00	1473.28	No backward
21	0.02	0.02	0.02	0.28	0.48	1473.00	1473.48	
22	0.00	0.00	0.00	0.28	0.28	1473.00	1473.28	No pressure
23	0.02	0.02	0.02	0.28	0.48	1473.00	1473.48	
24	0.10	0.10	0.10	-9.72	-8.72	1483.00	1474.28	
25	0.02	0.02	0.02	-9.72	-9.52	1483.00	1473.48	
26	0.00	0.00	0.00	-9.72	-9.72	1483.00	1473.28	No backward
27	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backward
28	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backwardk
29	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backward
30	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backward
31	0.00	0.00	0.00	20.28	20.28	1453.00	1473.28	No backward
32	0.02	0.02	0.02	20.28	20.48	1453.00	1473.48	
33	0.02	0.02	0.02	0.28	0.48	1473.00	1473.48	
34	0.03	0.03	0.03	0.28	0.58	1473.00	1473.58	
35	0.00	0.00	0.00	0.28	0.28	1473.00	1473.28	No pressure
36	0.00	0.00	0.00	0.28	0.28	1473.00	1473.28	No backward
37	0.00	0.00	0.00	-9.72	-9.72	1483.00	1473.28	No backward
38	0.00	0.00	0.00	-9.72	-9.72	1483.00	1473.28	No backward

Table D 1 continue

39	0.20	0.20	0.20	-9.72	-7.72	1483.00	1475.28	

4(*) –Average of Forward-Backward readings.

6(*) –Summation of Average and Tip constant.
8(*) –Metric value of piezometer water pressure which is the sum of pressure (6) and elevatıon of waterpiezometer (7).

NOT : Manometer elevation at measurement room is 1473.28 m.