

**DYNAMIC RESPONSE AND PERMANENT DISPLACEMENT  
ANALYSIS OF AKKÖPRÜ DAM**

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

**BY**

**DENİZ ÜLGEN**

**IN PARTIAL FULLFILLMENT OF THE REQUIREMENTS FOR THE  
DEGREE OF**

**MASTER OF SCIENCE**

**IN**

**THE DEPARTMENT OF CIVIL ENGINEERING**

**JANUARY 2004**

Approval of the Graduate School of Natural and Applied Sciences.

---

Prof. Dr. Canan ÖZGEN  
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

---

Prof. Dr. Erdal ÇOKÇA  
Head of Department

This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

---

Prof. Dr. M. Yener ÖZKAN  
Supervisor

Examining Committee Members

Prof. Dr. Altay BİRAN

Prof. Dr. M. Yener ÖZKAN

Prof. Dr. Orhan EROL

Asst. Prof. Dr. K. Önder ÇETİN

Gülru S. YILDIZ (M.S.)

## **ABSTRACT**

### **DYNAMIC RESPONSE AND PERMANENT DISPLACEMENT ANALYSIS OF AKKÖPRÜ DAM**

ÜLGEN, Deniz

M.S., Department of Civil Engineering

Supervisor: Prof. Dr. M. Yener ÖZKAN

January 2004, 115 pages

In this study, dynamic response of Akköprü Dam under earthquake motions is analyzed and the permanent displacements are evaluated. Initially, the critical slip surface of the dam and the corresponding yield acceleration are determined by using the computer program SLOPE. Then, by employing the finite element program SAP2000, static analyses are performed to obtain the mean effective stresses which are used in the determination of dynamic material properties of the dam. Four different scenario earthquakes having a magnitude of 7 are used in the dynamic analyses. Two of those scenarios are taken from European Strong Motion Database and the others are generated by XS artificial earthquake generation program prepared by Erdik (1992). Dynamic analyses of the dam are carried out by the finite element program TELDYN. Permanent displacements of the critical slip surface are calculated by utilizing the Newmark method. Consequently, for an earthquake having a magnitude of  $M=7$  and a peak ground acceleration of  $0.20g$ , the maximum permanent displacement of the dam is found to be 15.90 cm. Furthermore, the permanent displacements of the dam are calculated under base motions having different peak ground acceleration values and it is observed that the rate of increase

in the amount of permanent displacements is greater than the increase in the amount of peak ground accelerations.

Keywords: Embankment Dams, Rockfill Dams, Dam Safety, Earthquakes, Soil Dynamics, Dynamic Analysis, Finite Element Method, Permanent Displacement

## ÖZ

### AKKÖPRÜ BARAJININ DEPREM TEPKİLERİ VE KALICI DEFORMASYON ANALİZİ

ÜLGEN, Deniz

Yüksek Lisans, İnşaat Mühendisliği Bölümü

Tez Yöneticisi: Prof. Dr. M. Yener ÖZKAN

Ocak 2004, 115 sayfa

Bu çalışmada, Akköprü Barajının deprem hareketleri altındaki dinamik tepkilerinin analizi yapılmış ve kalıcı deformasyonları değerlendirilmiştir. İlk olarak, kritik kayma yüzeyi ve ona karşılık gelen kayma ivmesi bilgisayar programı SLOPE kullanılarak bulunmuştur. Bundan sonra, barajın dinamik malzeme özelliklerinin belirlenmesinde gerekli olan ortalama efektif gerilmelerini bulmak için sonlu elemanlar programı SAP2000 ile statik analiz yapılmıştır. Dinamik analizlerde 7 büyüklüğünde farklı dört senaryo deprem kullanılmıştır. Senaryo depremlerden ikisi Avrupa Kuvvetli Yer Hareketleri Veritabanından alınmış ve diğerleri de Erdik(1992) tarafından hazırlanan yapay deprem hareketi üreten XS programı ile oluşturulmuştur. Barajın dinamik analizleri sonlu elemanlar programı TELDYN kullanılarak yapılmıştır. Kritik kayma yüzeyindeki kalıcı deformasyonlar Newmark metodundan yararlanılarak hesaplanmıştır. Sonuç olarak 0.20g maksimum yer ivmesine sahip 7 büyüklüğündeki bir deprem için barajın maksimum kalıcı deformasyonu 15.90 cm olarak bulunmuştur. Bununla beraber, farklı maksimum yer ivme değerlerine sahip deprem hareketleri altında barajın kalıcı deformasyonları hesaplanmış ve kalıcı

deformasyon miktarındaki artış oranının ivme miktarındaki artış oranından fazla olduđu gözlenmiştir.

Anahtar Kelimeler: Dolgu Barajlar, Kaya Dolgu Barajlar, Baraj Emniyeti, Depremler, Zemin Dinamiđi, Dinamik Analiz, Sonlu Elemanlar Metodu, Kalıcı Deformasyon

**Dedicated to MY FAMILY**

## **ACKNOWLEDGEMENTS**

I would like to express my gratitude to my thesis supervisor Prof. Dr. M. Yener ÖZKAN for his wise guidance in preparation of this thesis and teaching me soil dynamics. I would also like to thank him for his general help and friendly supports.

Dr. Ramazan DEMİRTAŞ, Vehbi BİLGİ and Mehmet ÖZYAZICIOĞLU are also deeply acknowledged for their valuable supports and comments throughout the study.

I would also like to thank to Asst. Prof. Dr. K. Önder ÇETİN for his help and teaching me geotechnical earthquake engineering.

Also, I would like to thank to Burçin TUNA and Hasan MERCAN for their help in computer related works, especially in Matlab and programming languages.

Finally, I would like to thank to my dear family for their continuous support and friendship throughout the study.

## TABLE OF CONTENTS

ABSTRACT .....	iii
ÖZ.....	v
ACKNOWLEDGEMENTS.....	viii
TABLE OF CONTENTS.....	ix
LIST OF TABLES.....	xii
LIST OF FIGURES.....	xiv
LIST OF SYMBOLS .....	xvii
CHAPTER I.....	1
1. INTRODUCTION.....	1
1.1 General .....	1
1.2 Aim of the Study.....	4
CHAPTER II .....	5
2. STATIC AND DYNAMIC ANALYSES OF EMBANKMENT DAMS.....	5
2.1 Static Analysis of Embankment Dams.....	5
2.2 Design Ground Motion.....	6
2.2.1 Estimation of Peak Ground Acceleration .....	8
2.2.2 Estimation of the Predominant Period .....	10
2.2.3 Estimation of Duration.....	10
2.3 Dynamic Analysis of Embankment Dams .....	12
2.3.1 Pseudostatic Method of Analysis.....	12
2.3.2 Shear Beam Method .....	15
2.3.3 Finite element method .....	18

2.4 Equivalent Linear Model .....	19
2.5 Permanent Displacement Analysis.....	25
<b>CHAPTER III.....</b>	<b>31</b>
<b>3. AKKÖPRÜ DAM AND SEISMICITY OF THE SITE .....</b>	<b>31</b>
<b>3.1 General Information About Dalaman - Akköprü Dam .....</b>	<b>31</b>
3.1.1 Characteristics of the Project.....	31
3.1.2 Geology and the Foundation .....	34
3.1.3 The Dam Embankment .....	35
3.2 Seismicity of Akköprü Dam Site.....	38
3.3 Design Earthquake Ground Motion .....	39
3.3.1 Probabilistic Assessment .....	39
3.3.2 Deterministic Assessment .....	45
3.3.2.1 Acceleration – Attenuation Approach.....	45
3.4 Design Peak Ground Acceleration.....	46
3.5 Estimating Input Ground Motion.....	46
<b>CHAPTER IV .....</b>	<b>50</b>
<b>4. ANALYSES OF AKKÖPRÜ DAM.....</b>	<b>50</b>
4.1 Analyses Procedure for Akköprü Dam .....	50
4.2 Pseudo-Static Analyses of Akköprü Dam.....	51
4.3 Static Analyses of Akköprü Dam.....	52
4.4 Dynamic Material Properties of Akköprü Dam.....	59
4.5 Determination of Input Ground Motions to be Used in the Dynamic Analysis.....	63
4.6 Dynamic Analysis of Akköprü Dam .....	65
4.7 Permanent Displacement Analysis of Akköprü Dam.....	66

<b>CHAPTER V.....</b>	<b>68</b>
<b>5. RESULTS OF THE DYNAMIC ANALYSES OF AKKÖPRÜ DAM.....</b>	<b>68</b>
<b>5.1 Seismic Behavior of Akköprü Dam .....</b>	<b>68</b>
<b>5.2 Results of Permanent Displacements Analysis.....</b>	<b>69</b>
<b>CHAPTER VI.....</b>	<b>77</b>
<b>6. CONCLUSIONS .....</b>	<b>77</b>
<b>REFERENCES .....</b>	<b>80</b>
<b>APPENDICES.....</b>	<b>88</b>
<b>A. EARTHQUAKES WITH <math>M \geq 4</math> OCCURED DURING         THE PERIOD 1900-2002.....</b>	<b>88</b>
<b>B. DESCRIPTION OF THE COMPUTER         PROGRAM TELDYN.....</b>	<b>107</b>
<b>C. DESCRIPTION OF THE COMPUTER         PROGRAM SHAKE91.....</b>	<b>109</b>
<b>D. COMPUTER PROGRAM ACCHIS.....</b>	<b>111</b>
<b>E. COMPUTER PROGRAM PDISP.....</b>	<b>114</b>

## LIST OF TABLES

### TABLE

2.1	Typical Earthquake Durations at Epicentral Distances Less than 10 km.....	12
2.2	Pseudostatic Analyses of Dams with Slope Failures During Earthquakes.....	15
3.1	Properties of Dam Embankment Material.....	38
3.2	Expected Peak Ground Acceleration Values at Akköprü Dam Site.....	42
3.3	Probability of Exceedence of PGA at Akköprü Dam Site.....	42
3.4	Expected Peak Ground Acceleration Values at Dam Site.....	46
3.5	Actual Seismic Records Taken from European Strong Motion Database....	47
4.1	Factor of Safeties of Trial Slip Circles for Upstream and Downstream Slopes.....	54
4.2	Factor of Safeties of Critical Slip Circles for Upstream and Downstream Slopes.....	54
4.3	Average Void Ratio and Poisson's Ratio of Construction Materials.....	60
5.1	Results of Permanent Displacement Analyses.....	75

A.1 List of Earthquakes with Magnitudes Greater Than  $M \geq 4$  Occured in the  
Region Bounded by 35.00 - 38.00 N 27.00 - 30.00  
During the Period 1900 – 2002.....90

## LIST OF FIGURES

### FIGURES

2.1	Variation of Predominant Period at Rock Outcrops with Magnitude and Distance.....	10
2.2	Illustration of Bracketed Duration for 0.05g Threshold Acceleration.....	11
2.3	Forces Acting on the Potential Circular Sliding Mass in Pseudostatic Method.....	13
2.4	Earth Dam, Showing Stresses Acting on an Element of Thickness, dz.....	17
2.5	Hysteric Loop Indicating Shear Stress Relationship for Soils Subjected to Cyclic Loading.....	21
2.6	Backbone Curve Showing Typical Variation of $G_{sec}$ with Shear Strain.....	22
2.7	The Variation of the Modulus Ratio with Shear Strain.....	22
2.8	Modulus Degradation and Damping Curves for Cohesionless Soils.....	23
2.9	Modulus Degradation and Damping Curves for Cohesive Soils.....	24
2.10	Sliding Block Resting on an Inclined Plane.....	27
2.11	Forces Acting on the Sliding Block.....	27

2.12	Integration of Effective Acceleration Time History to Determine Velocities and Displacements.....	30
3.1	Photo of Akköprü Dam Taken at the End of Year 2003.....	32
3.2	Plan of View Akköprü Dam.....	36
3.3	Cross Section of Akköprü Dam.....	37
3.4	Tectonic Map of the Earthquake Zone Surrounding the Akköprü Dam Site.....	40
3.5	Macroseismic Map of 1957 Fethiye Earthquake.....	41
3.6	Macroseismic Map of 1961 Aegean-Mediterranean Earthquake.....	41
3.7	Regional Peak Ground Acceleration Graph of Akköprü Dam.....	43
3.8	Equipotential Acceleration Curves of Akköprü Dam with 20% Probability of Exceedence for 100 Year Economic Life.....	44
3.9	Variation of Predominant Period at Rock Outcrops with Magnitude and Distance.....	47
3.10	Acceleration Time Histories of Design Ground Motions.....	48
3.11	Acceleration Response Spectra of Design Ground Motions.....	49
4.1	Trial Slip Surfaces for Upstream Slope.....	53
4.2	Trial Slip Surfaces for Downstream Slope.....	53

4.3	Critical Slip Surface Determined in Pseudostatic Analysis.....	55
4.4	Finite Element Mesh of Akköprü Dam Used in Static and Dynamic Analyses.....	57
4.5	Saturated and Wet Zones Assumed for the Total Stress Analysis.....	58
4.6	Shear Modulus and Damping Curves for Clay and Filter Material.....	61
4.7	Shear Modulus and Damping Curves for Rockfill Material.....	62
4.8	Shear Wave Velocity Profile of Alluvium Layer.....	63
4.9	Acceleration Response Spectra at the Base of Dam Body.....	64
5.1	Variation of Fundamental Period with the Peak Ground Acceleration.....	70
5.2	Location of Nodal Point Numbers Taken for Acceleration Response Spectra Outputs.....	71
5.3	Maximum Acceleration Distribution Throughout the Dam Axis.....	72
5.4	Acceleration Response Spectra Along the Centerline of Akköprü Dam for Synthetic Record 1.....	73
5.5	Example of Permanent Displacement Analysis.....	74
5.6	Variation of Permanent Displacement with the Peak Ground Acceleration.....	76
A.1	Earthquakes with $M \geq 4$ Occur in Southwestern Anatolia During the Period 1900-2002.....	89

## LIST OF SYMBOLS

$\Delta\sigma_x$	: Change at the normal stress in x-direction
$\Delta\varepsilon_x$	: Change at the normal strain in x-direction
$\Delta\sigma_{xy}$	: Change at the shear stress on xy plane
$\Delta\varepsilon_{xy}$	: Change at the shear strain on xy plane
$\Delta\sigma_y$	: Change at the normal stress in y-direction
$\Delta\varepsilon_y$	: Change at the normal strain in y-direction
$\sigma'_1, \sigma'_2, \sigma'_3$	: Principal stresses
$\sigma'_m$	: Mean effective stress
$\alpha_0, \alpha_1, \alpha_2$	: Coefficients for attenuation relationship
$\beta_0, \beta_1, \beta_2$	: Coefficients for attenuation relationship
$\beta_n$	: $N_{th}$ root of a period relation
$\Delta W$	: Dissipated energy
$\sigma_y$	: Vertical earth pressure
$\mu$	: Damping ratio
$\rho$	: Density
$\gamma$	: Shear strain
$\varepsilon$	: Standard error term
[C]	: Global damping matrix
[K]	: Global stiffness matrix
[M]	: Global mass matrix
A	: Constant
$a_{ave}$	: Average acceleration
$a_b$	: Acceleration of inclined plane
$A_n, B_n$	: Constants

$a_{rel}$	: Relative acceleration
$a_y$	: Yield acceleration
$B$	: Constant
$b_1, b_2, b_3$	: Coefficients for attenuation relationship
$b_4, b_5, b_6$	: Coefficients for attenuation relationship
$b_7, b_v$	: Coefficients for attenuation relationship
$d$	: Closest horizontal distance to the fault
$e$	: Void ratio
$E$	: Young's modulus
$E, F$	: Moment arms
$F_b$	: Total force acting on sliding block
$F_e$	: Force acting on an element
$FS$	: Factor of safety
$g$	: Gravitational acceleration
$G$	: Shear modulus
$G_B, G_C$	: Coefficients for attenuation relationship
$G_{max}$	: Maximum shear modulus
$G_{sec}$	: Secant shear modulus
$h$	: Fictitious depth
$h$	: Height of the dam
$J_0$	: Bessel function
$K$	: Bulk modulus
$K_2$	: Parameter relating $G_{max}$ and $\sigma'_m$
$K_g$	: Parameter used for determining maximum shear modulus
$k_h$	: Horizontal seismic coefficient
$l$	: Length of sliding surface
$M$	: Moment magnitude
$m_b$	: Mass of sliding block
$m_e$	: Mass of element
$M_s$	: Surface wave magnitude
$M_w$	: Moment magnitude
$ng$	: Exponent used for determining maximum shear modulus

$P_a$	: Atmospheric pressure
$r$	: Closest distance to the source
$R$	: Radius of sliding surface
$s$	: Shear strength
$t$	: Time
$u$	: Displacement
$V_A$	: Fictitious shear wave velocity
$V_s$	: Shear wave velocity
$W$	: Maximum strain energy
$W$	: Weight
$w_n$	: $N_{th}$ mode of natural circular frequency
$Y$	: Ground motion parameter

# CHAPTER I

## INTRODUCTION

### 1.1 General

Water control and assured water availability of appropriate quality become essential requirements for continuing economic and social development. To satisfy this, a large number of dams were built all over the world for hydropower generation, flood control and irrigation. Most of these dams have been located in highly seismic areas. Many engineers earlier thought that embankment dams had been inherently safe against earthquakes, but then, especially after the failure of the Lower San Fernando Dam during San Fernando earthquake 1971, the safety evaluation of the dams had become a major concern of the engineers.

Results of the studies on the performance of the embankment dams subjected to strong ground shaking indicate that any well-built dam on a firm foundation can withstand moderate ground shaking with peak acceleration of about 0.2g, with no detrimental effects. Moreover, dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking ranging from 0.35g to 0.8g from a magnitude 8.5 earthquake with no apparent damage. In case of dams constructed of cohesionless soils, a primary cause of damage or failure is the build-up of pore water pressures in the embankment and the possible loss of strength due to high pressures (Seed et al. 1978).

There are several ways in which an earthquake may cause failure of an embankment dam. The most commonly noticed effects are settlement and cracking, and that may be dangerous depending on their magnitude and location. Sherard (1967) listed the possible failures of an embankment dam due to earthquake shaking as follows:

- 1) Disruption of the dam by major fault movement in foundation.
- 2) Slope failures induced by ground motions.
- 3) Piping failure through cracks induced by ground motions.
- 4) Sliding of dam on weak foundations materials.
- 5) Loss of freeboard due to differential tectonic ground movements, slope failures or soil compaction.
- 6) Overtopping of dam due to seiches in reservoir.
- 7) Overtopping of dam due to slides or rockfalls into reservoir.
- 8) Failure of spillway or outlet works.

Considering the potentially harmful effects of earthquakes on embankment dams Seed (1979) offered the possible defensive measures as follows:

- 1) Allow ample freeboard to allow for settlement, slumping or fault movements.
- 2) Use wide transition zones of material not vulnerable to cracking.
- 3) Use chimney drains near the central portion of the embankment

- 4) Provide ample drainage zones to allow for possible flow of water through cracks.
- 5) Use wide core zones of plastic materials not vulnerable to cracking.
- 6) Use a well-graded filter zone upstream of the core to serve as a crack stopper.
- 7) Provide crest details that will prevent erosion in the event of overtopping.
- 8) Flare the embankment core at abutment contacts.
- 9) Locate the core to minimize the degree of saturation of materials.
- 10) Stabilize slopes around the reservoir rim to prevent slides into the reservoir.
- 11) Provide special details if there is danger of fault movement in the foundation.

Until circa 1960, the standard method of evaluating the seismic stability of embankment dams had been the pseudostatic method of analyses which is based on determining the safety factor of the potential sliding mass, but it gives no information about the anticipated displacements. In order to estimate the displacements Newmark (1965) proposed the method of analysis for evaluating the permanent displacements of embankment dams subjected to strong ground motions. Then, with the advent of dynamic analysis procedures, the performance of embankment dams under earthquake loading have been successfully evaluated by using the finite element method which is extensively used today for design and analysis of the embankment dams.

## 1.2 Aim of the Study

The aim of this study is to evaluate the response of Dalaman-Akköprü Dam under earthquake loading with the finite element solutions. As a first step, the potential sliding mass of the embankment is determined by using the pseudostatic method. Then, static analysis is performed to obtain the mean effective stresses for the assessment of the dynamic material properties which represent the nonlinear behavior of the embankment dam. After this, dynamic analysis is carried out by employing finite element method with the estimated scenario earthquakes. Finally, the permanent displacements of the dam under the selected design motions having different values of peak ground accelerations are determined by using the acceleration time histories obtained in the dynamic analysis. In order to calculate the permanent displacements Newmark's method is used in the analysis.

Chapter 2 reviews the literature and previous studies related with the evaluation of response of embankment dams. Chapter 3 gives general information about Akköprü Dam and the seismicity of the site and explains how the design ground motion is estimated. Chapter 4 describes the steps of the analysis procedures, how the potential sliding mass is determined, how the static and dynamic analysis are performed and how the permanent displacements of the potential sliding mass are calculated. Chapter 5 discusses the results of the analysis and evaluates the dynamic behavior of the dam. Chapter 6 includes the conclusions of the study.

## CHAPTER II

### STATIC AND DYNAMIC ANALYSES OF EMBANKMENT DAMS

#### 2.1 Static Analysis of Embankment Dams

In order to assess the stresses displacements in earth dams under static conditions, finite element method can be used by performing the analysis in a number of steps, or increments. Use of incremental analyses procedures provides a convenient means of representing changes in geometry during construction of the embankment, changes in loading during filling of the reservoir and nonlinear stress-strain behavior of the embankment materials.

Incremental analyses follow the stages of construction and loading as they occur in the embankment step by step. At each stage of the analysis the stress-strain properties of the embankment materials are adjusted in accordance with the calculated values of the stresses in the elements to model the nonlinear stress-strain behavior.

Some of the most common finite element programs used in static analysis of embankment dams are FEADAM, ISBILD, LSBILD and TELSTA which is a significantly enhanced version of the others. All these programs are designed for plain strain conditions and incremental analyses are used to approximate the nonlinear behavior of the materials. In each increment of the analysis the stress-strain behavior of the soil is treated as being linear, and the relationship between stress and strain is assumed to be governed by Hooke's law. The relationship can be expressed in terms of Young's modulus and bulk modulus as:

$$\begin{pmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\sigma_{xy} \end{pmatrix} = \frac{3K}{9K - E} \begin{pmatrix} 3K+E & 3K-E & 0 \\ 3K-E & 3K+E & 0 \\ 0 & 0 & E \end{pmatrix} \begin{pmatrix} \Delta\epsilon_x \\ \Delta\epsilon_y \\ \Delta\epsilon_{xy} \end{pmatrix} \quad (2.1)$$

where  $\Delta\sigma$  is the stress,  $K$  is the bulk modulus,  $E$  is the Young's modulus and  $\Delta\epsilon$  is the strain.

In case of analyses to establish pre-earthquake stresses in an embankment dam, the deformations are not of interest. A linear finite element program SAP2000 may be used to find the static stresses throughout the dam body. Since the program does not consider the non-linearity of the soil elements of the dam body, so it will not give the actual state of stresses within the dam body. Therefore, the iterative procedure which involves a sequence of calculations is carried out in order to obtain the actual state of stresses within the dam body. Calculations are related with the relationship between the elastic modulus of deformation value and the stress value. In this study, the following equation (2.2) is used suggested by Baba (1982) :

$$E = A \cdot \sigma_y^B \quad (2.2)$$

where  $E$  is the elastic modulus of deformation,  $\sigma_y$  is the vertical stress,  $A$  and  $B$  are the constants for core, filter and fill materials of the dam.

## 2.2 Design Ground Motion

The aim of earthquake resistant design is to produce a structure that can resist to a certain level of shaking described by a design ground motion. Estimation of the design ground motion is an essential step of the dynamic analysis of a dam to determine the acceleration-time history applied at the base of the embankment-foundation system. In order to select a proper design ground motion for the project following steps should be investigated:

- i) Seismicity of the site, seismic risk and past earthquakes (historical seismicity).
- ii) Surface topography, characteristics of the foundation soil and main rock below the foundation soil and their location.
- iii) Type and dynamic material properties of the soil above the main rock.
- iv) Ground motion characteristics at the main rock caused by the earthquake.

The level of shaking is mostly described in terms of design ground motion parameters that reflect the characteristics of the design ground motion. The parameters most commonly used to specify the design ground motion are peak horizontal acceleration, predominant period and the duration of the earthquake. All of these parameters are determined in accordance with the estimation of the magnitude of the maximum credible earthquake which mostly depends on the historical seismicity of the site.

In case of knowledge of the parameters mentioned above, time history of the ground motion can be estimated by using several methods. It can be estimated by using mathematical and statistical methods or calibrating the ground motion parameters of an actual or synthetic earthquake according to estimated design ground motion parameters. Since the second approach is simple, it is conventionally used by the earthquake engineers. In this study, two artificial earthquakes are generated by using the computer XS artificial earthquake generation program and two actual earthquakes are used to determine the design ground motion. Furthermore, the computer program NONLIN (Nonlinear Dynamic Time History Analyses of Single Degree of Freedom Systems) is used to obtain the desired ground motion parameters reflecting the characteristics of the design ground motion.

### 2.2.1 Estimation of Peak Ground Acceleration

For the assessment of peak ground acceleration a number of predictive relationships in other words attenuation relationships have been developed. These relationships involve the earthquake magnitude, type of faulting, distance to fault and site conditions. They are developed by regression analyses of recorded strong ground motion databases. When using any predictive relationship, it is very important how parameters such as M (magnitude) and R (distance to fault) defined and use of them in a consistent manner. It is also important to recognize that different predictive relationships are usually obtained from different data sets.

Attenuation relationships used in this study are given below. I.M. Idriss (1991) suggested the following empirical relationship:

$$\ln(Y) = [\alpha_0 + \exp(\alpha_1 + \alpha_2 M)] + [\beta_0 - \exp(\beta_1 + \beta_2 M)] \ln(R + 20) + 0.2 F + \varepsilon \quad (2.3)$$

$$\text{For } M \leq 6 \quad \alpha_0 = -0.150, \alpha_1 = 2.261, \alpha_2 = -0.083, \beta_0 = 0, \beta_1 = 1.602, \\ \beta_2 = -0.142 \text{ and } \varepsilon = 1.39 - 0.14M$$

$$\text{For } M > 6 \quad \alpha_0 = -0.050, \alpha_1 = 3.477, \alpha_2 = -0.284, \beta_0 = 0, \beta_1 = 2.475, \\ \beta_2 = -0.286 \text{ and } \varepsilon = 1.39 - 0.14M$$

- Ln : natural logarithm  
exp : exponential function  
Y : ground motion parameter (peak horizontal acceleration, a in g's, and pseudo absolute spectral acceleration, at 5% damping)  
M : local magnitude,  $M_L$ , for  $M \leq 6$  and surface wave magnitude,  $M_S$  for  $M > 6$ . Thus, in essence, M represents moment magnitude,  $M_w$ .  
R : closest distance to the source in km; however, for  $M \leq 6$  the hypocentral distance is used.  
F : style of faulting factor; F=0 for a strike slip fault; F=1 for a reverse fault and F=0.5 for an oblique source.

$\varepsilon$  : standard error term.

Boore et al(1993). proposed the following relationship:

$$\text{Log}(Y) = b_1 + b_2(M-6) + b_3(M-6)^2 + b_4r + b_5\log r + b_6 G_B + b_7G_C \quad (2.4)$$

where  $r = (d^2 + h^2)^{1/2}$ ,  $d$  is the closest distance to the surface projection of the fault in kilometers,  $Y$  is the peak ground acceleration in  $g$ ,  $M$  is the moment magnitude, for randomly-oriented horizontal component (or geometric mean)  $b_1 = -0.105$ ,  $b_2 = 0.229$ ,  $b_3 = 0$ ,  $b_4 = 0$ ,  $b_5 = -0.778$ ,  $b_6 = 0.162$ ,  $b_7 = 0.251$ ,  $h=5.57$  and  $\sigma = 0.230$  (for geometrical mean  $\sigma = 0.208$ ) and for larger horizontal component  $b_1 = -0.038$ ,  $b_2 = 0.216$ ,  $b_3 = 0$ ,  $b_4 = 0$ ,  $b_5 = -0.777$ ,  $b_6 = 0.158$ ,  $b_7 = 0.254$ ,  $h=5.48$  and  $\sigma = 0.205$  and

For site class A,  $V_{s,30} > 750$  m/sec,  $G_B = 0$ ,  $G_C = 0$

For site class B,  $360 < V_{s,30} < 750$ m/sec,  $G_B = 1$ ,  $G_C = 0$

For site class C,  $180 < V_{s,30} < 360$ m/sec,  $G_B = 0$ ,  $G_C = 1$

Kalkan (2001) estimated the following attenuation relationship for Turkey sites by using the same general form of the equation proposed by Boore et al. (1997).

$$\text{Ln}(Y) = b_1 + b_2(M-6) + b_3(M-6)^2 + b_5\ln r + b_v(\ln V_s / V_A) \quad (2.5)$$

where  $r = (d^2 + h^2)^{1/2}$ ,  $Y$  is the peak ground acceleration,  $M$  is the moment magnitude,  $d$  is the closest horizontal distance from the station to a site of interest in km,  $V_A$  is the fictitious shear wave velocity,  $V_s$  is shear wave velocity in m/sec, and

$b_1 = -0.682$ ,  $b_2 = 0.253$ ,  $b_3 = 0.036$ ,  $b_5 = -0.562$ ,  $b_v = -0.297$ ,  $V_A = 1381$  m/s,  
 $h = 4.48$  and  $\sigma = 0.562$

### 2.2.2 Estimation of the Predominant Period

Predominant period is a single parameter that provides a useful representation of the frequency content of the motion. It is defined as the period of vibration corresponding to the maximum value of the Fourier amplitude spectrum. In order to estimate the predominant period of the design ground motion at rock outcrops Seed et al. (1969) have developed idealized curves indicating the variation of predominant period at rock outcrop with magnitude and distance as seen in Figure 2.1.

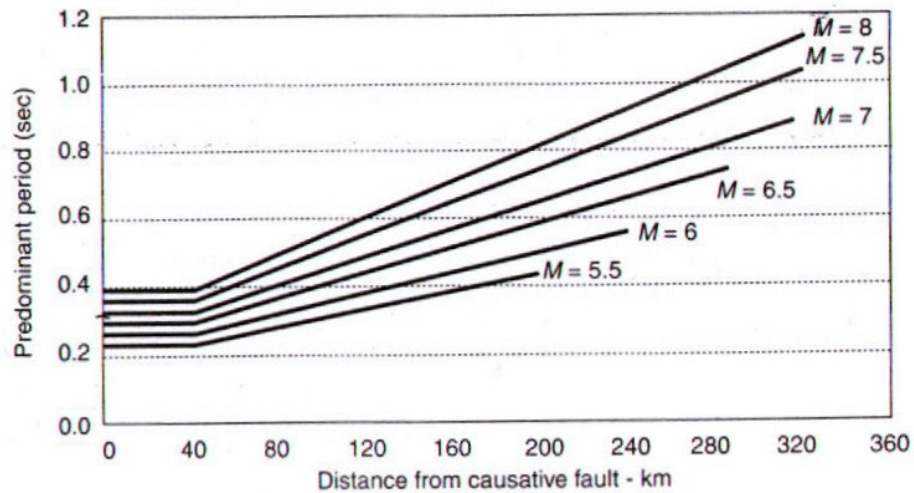


Figure 2.1 Variation of Predominant Period at Rock Outcrops with Magnitude and Distance (After Seed et al., 1969).

### 2.2.3 Estimation of Duration

The duration of a strong ground motion can have a strong influence on earthquake damage. It is related to the time required for release of accumulated strain energy by rupture along the fault. As the length or area of rupture increases, the time required for rupture increases. A motion of short duration may not produce enough load reversals for damaging response to build up in a structure, even if the amplitude

of motion is high. On the other hand a motion with moderate amplitude but long duration can produce load reversals to cause substantial damage.

An earthquake accelerogram generally contains all accelerations from the time the earthquake begins until the time the motion has returned to the level of background noise. For engineering purposes, only the strong motion portion of the accelerogram is of interest. Different approaches have been taken to the problem of evaluating the duration of strong motion in an accelerogram. The bracketed duration (Bolt 1969) is defined as the time between the first and last exceedances of a threshold acceleration (usually 0.05g). Based on a threshold acceleration of 0.05g, the bracketed durations can be obtained from the accelerogram as shown in Figure 2.2.

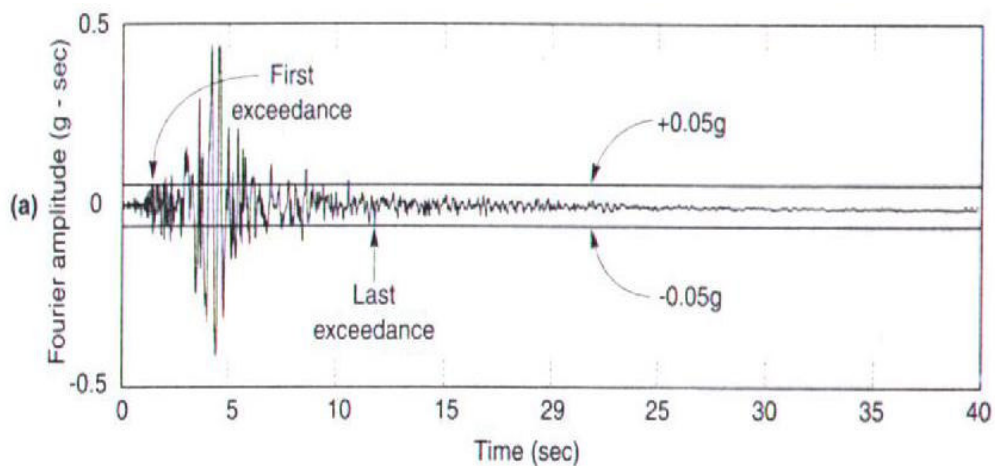


Figure 2.2 Illustration of Bracketed Duration for 0.05g Threshold Acceleration (Kramer 1996)

The duration of strong motion has been investigated by interpretation of accelerograms from earthquakes of different magnitudes. By using a 0.05g threshold acceleration, Chang and Krinitszky (1977) estimated the bracketed durations for soil and rock sites at short (less than 10 km) epicentral distances shown in Table 2.1. Another definition of duration (Trifunac and Brady, 1975b) is based on the time interval between the points at which 5% and 95% of the total energy has

been recorded. Boore (1983) has taken the duration to equal to corner period. The rate of change of cumulative root-mean-square (rms) acceleration has also been used as the basis for evaluation of strong-motion duration (McCann and Shah, 1979). Among these definitions, the bracketed duration is most commonly used for earthquake engineering purposes because it implicitly reflects the strength of shaking (Kramer 1996).

Table 2.1 Typical Earthquake Durations at Epicentral Distances less than 10 km

Magnitude	Duration (sec)	
	Rock Sites	Soil Sites
5.0	4	8
5.5	6	12
6.0	8	16
6.5	11	23
7.0	16	32
7.5	22	45
8.0	31	62
8.5	43	86

## 2.3 Dynamic Analysis of Embankment Dams

### 2.3.1 Pseudostatic Method of Analysis

The seismic safety of earth structures has been analyzed for almost seventy years by using the method of pseudostatic analysis. In this method the same principle is used as in the static slope stability analyses and the overturning and resisting forces on the critical slope surface are taken into consideration. The earthquake effects are represented by an earthquake static horizontal force determined by a seismic coefficient  $k$  and the weight of the sliding mass as seen in Figure 2.3. Equivalent horizontal force can be formulated as follows:

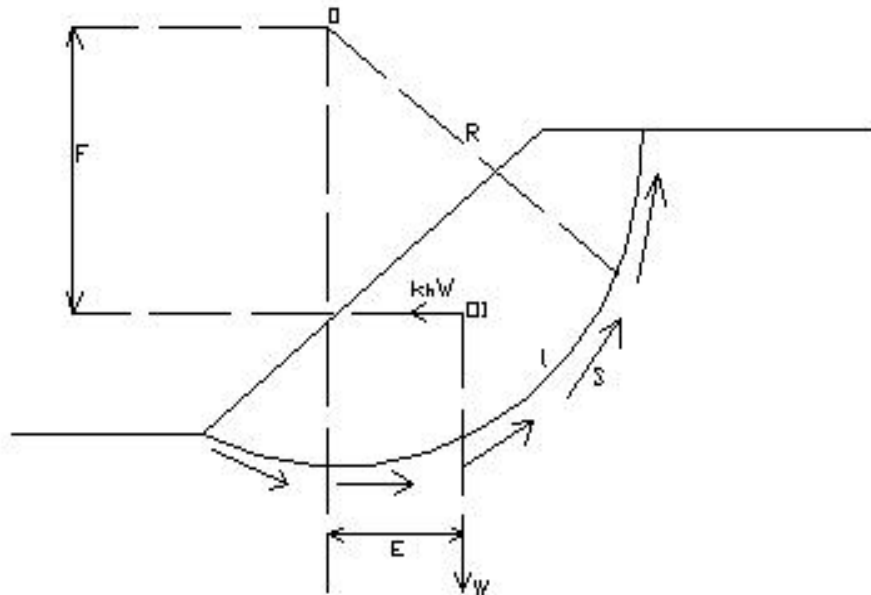


Figure 2.3 Forces Acting on the Potential Circular Sliding Mass in Pseudostatic Method

$$F_{eq} = k_h * W \quad (2.6)$$

By using a simple moment equilibrium analysis the factor of safety can be defined as the ratio resists rotation of a critical slip surface about the center of the sliding surface to the moment that is driving the rotation. For a circular sliding surface as seen in Figure 2.3, the factor of safety can be formulated as follows:

$$FS = \frac{\text{Resisting Moments}}{\text{Overturning moments}} = \frac{s * l * R}{E * W + k_h * F * W} \quad (2.7)$$

where as the shear strength, weight and seismic coefficient are denoted by  $s$ ,  $W$  and  $k_h$ , respectively.  $O$  is the center of the sliding circle and  $O1$  is the gravity center of

the sliding mass; E and F are the moment arms; and R and l are radius and length of the sliding surface respectively.

Equation (2.6) is based on the simplifying assumptions that the horizontal acceleration  $k_h g$  acts permanently on the slope material and in one direction only. Therefore, the concept in conveys of earthquake effects on slopes is very inaccurate to say the least. Theoretically, a value of  $FS = 1$  would mean a slide, but in reality a slope may remain stable in spite of  $FS$  being smaller than unity and it may fail at a value of  $FS > 1$  depending on the character of the slope forming material (Terzaghi 1950). This statement clearly indicates that a slope may be unstable even if the  $FS$  is greater than 1. Related to this case some of the major slope failures of dams analyzed with pseudostatic method are listed in Table 2.2.

As seen from the cases in Table 2.2, in pseudostatic analyses the most important difficulty is the selection of an appropriate seismic coefficient. The seismic coefficient mostly depends on the seismicity activity of the region, type of the dam embankment material and the importance of the project. For the selection of seismic coefficient Seed (1979) listed pseudo design criteria for 14 dams in 10 seismically active countries; 12 required minimum factor of safety of 1.0 to 1.5 with pseudostatic coefficients of 0.10 to 0.12. Marcuson (1981) suggested that appropriate pseudostatic coefficients for dams should correspond to one-third to one half of the maximum acceleration, including amplification and deamplification effects, to which the dam is subjected. Seed (1979) also indicated that deformations of earth dams constructed of ductile soils with crest accelerations less than 0.75g would be acceptably small for pseudostatic factors of safety of at least 1.15 with  $k_h = 0.10$  ( $M = 6.5$ ) to  $k_h = 0.15$  ( $M = 8.25$ ). This criteria would allow the use of pseudostatic accelerations as small as 13 to 20 % of the peak crest acceleration (Kramer 1996).

As a conclusion, difficulties in the selection of seismic coefficients and in the evaluation of safety factor have reduced use of pseudostatic method for seismic slope stability analyses. Hence, today finite element method has become reliably possible to investigate the behavior of embankment dams under earthquake loading.

Table 2.2 Pseudostatic Analyses of Dams with Slope Failures During Earthquakes (Seed 1979)

Dam	Seismic coefficient	Computed factor of safety	Effect of earthquake
Sheffield Dam	0.1	1.2	Complete failure
Lower San Fernando Dam	0.15	1.3	Upstream slope failure
Upper San Fernando Dam	0.15	≈2 to 2.5	Downstream shell including crest slipped about 6ft downstream
Tailings Dam (Japan)	0.2	≈1.3	Failure of dam with release of tailings

### 2.3.2 Shear Beam Method

Dynamic response analysis of the two dimensional earth dams can be greatly simplified by the shear beam analysis. It allows two dimensional problem to be modeled as equivalent one dimensional problem by using the following assumptions:

- i) The dam is infinitely long and it consists of homogeneous and linearly elastic material.
- ii) The dam moves only at horizontal direction. Only horizontal shear deformations are considered
- iii) Shear stresses or shear strains are uniform across horizontal planes.

iv) The dam consists of thin horizontal slices. These slices are connected to each other with elastic springs and viscous damping mechanisms.

v) Influence of the reservoir is neglected.

Based on the assumptions above the dam body can be illustrated as in Figure 2.4 and the dynamic equilibrium can be represented as follows:

$$\frac{\partial^2 u}{\partial t^2} = \frac{G}{\rho} \left[ \frac{\partial^2 u}{\partial z^2} + \frac{1}{z} \frac{\partial u}{\partial z} \right] \quad (2.8)$$

where  $u$  is the horizontal displacement at a given depth  $z$ ,  $t$  is the time,  $G$  is the shear modulus and  $\rho$  is the mass density of the construction material. Using the boundary conditions  $u=0$  at  $z=h$  and  $\partial u / \partial z=0$  at  $z=0$  the solution of the differential equation is obtained as follows:

$$u(z,t) = \sum_{n=1}^{\infty} [A_n \sin w_n t + B_n \cos w_n t] J_0 \left( \beta_n \frac{y}{h} \right) \quad (2.9)$$

where  $h$  is the height of the dam,  $A_n$  and  $B_n$  are the constants,  $J_0$  is the Bessel function of first kind of zero and  $\beta_n$  is the  $n$ -th root of period relation for the modes ( $\beta_1=2.404$ ,  $\beta_2=5.520$ ,  $\beta_3=8.654$ ,  $\beta_4=11.792$ ,  $\beta_5= 4.931$  etc.),  $w_n$  is the  $n$ -th mode of natural circular frequency which may be written as:

$$w_n = \frac{\beta_n}{h} V_s \quad (2.10)$$

where  $V_s$  is the shear wave velocity. Substituting  $w_n = 2\pi/T$  and  $B_n$  values into Equation 2.10, natural periods for two modes of vibration are obtained as:

$$T_1 = 2.612 h / V_s \quad (2.11)$$

$$T_2 = 1.138 h / V_s \quad (2.12)$$

These equations give better results when the ratio of width of the base of the dam to the height of the dam is almost three.

Consequently, in the shear beam method earthquake induced accelerations and deformations acting only in horizontal direction can be determined. It is fundamentally different from the equivalent linear and non-linear models in that it restricts particle movement to the horizontal plane. Even though the shear beam analysis gives general idea about the dynamic behavior of an earth dam, due to the restrictions and assumptions explained above it is far from reflecting actual results.

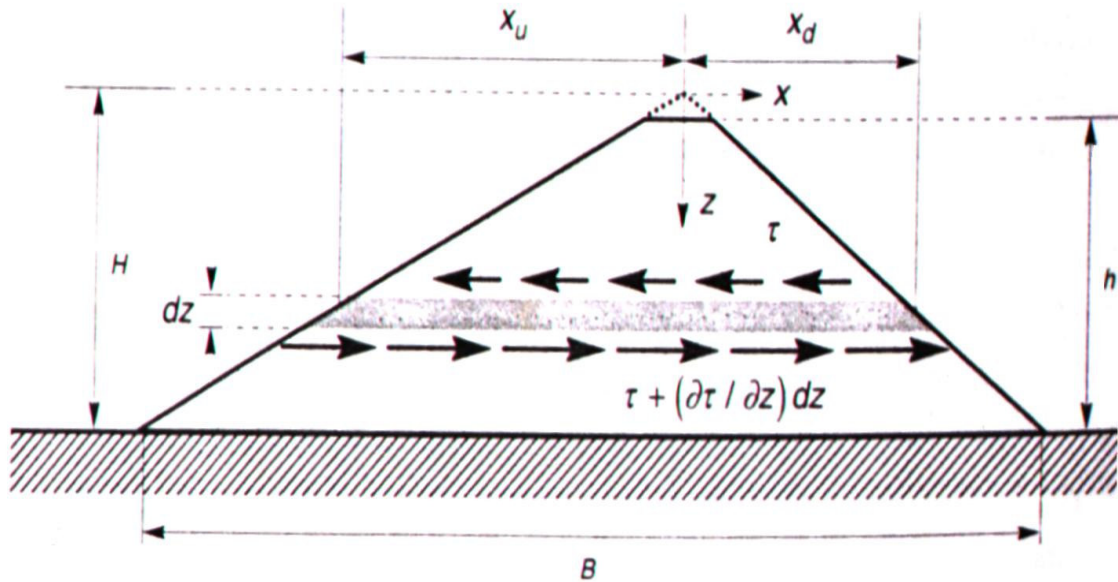


Figure 2.4 Earth Dam, Showing Stresses Acting on an Element of Thickness,  $dz$

### 2.3.3 Finite element method

Finite element method treats a continuum as an assemblage of finite elements which are defined by nodal points and assumes that the response of the continuum equivalent to the response of the nodal points. Elements are connected with each other at the nodal points and they simulate the material behavior of the zones. It is one of the most powerful method for evaluating the response of embankment dams under earthquake loading. It is possible to obtain actual results with this method by considering the nonlinear stress-strain behavior of the construction materials. Comparing with the other methods, advantages of finite element method can be given as follows:

i) Time dependent stress-strain behavior of any element or region of the dam body can be evaluated.

ii) Effects of the embankment interaction and foundation characteristics can be simulated.

iii) Irregular geometry and complex boundary conditions can be taken into account.

iv) Nonlinear behavior of the soil can be analyzed and permanent dynamic deformations can be calculated.

In the case of a response analysis, it is necessary to solve the equation of motion which represents the dynamic equilibrium of the all elements. The equation of motion for dynamic finite element method can be given as:

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = -[M]\ddot{Y} \quad (2.13)$$

where  $U$  is the displacement vector and  $Y$  is the time history of the base motion,  $M$  is the mass matrix,  $C$  is the damping matrix and  $K$  is the stiffness matrix.

There are several methods used for the solution of the Equation 2.13. These methods can be written as:

- i) Direct Integration
- ii) Modal Superposition
- iii) Fourier Analysis

The most common method used for evaluating the behavior of non-linear systems under cyclic loading is the direct integration method. The other methods; modal superposition and Fourier analysis are only valid for the evaluation of the linear-elastic systems.

The finite element method can be used for the solution of the two dimensional and three dimensional dynamic response problems. In the case of earth structures, usually plane strain and two dimensional analysis of transverse (along the river valley, normal to dam axis) sections are used. There are several computer programs available involving the assumption of plain strain conditions. Some of the computer programs used for the evaluation of the response of embankment dams are QUAD-4 (Idriss et al, 1973), LUSH (Lysmer et al, 1974), FLUSH (Lysmer et al, 1975) and TELDYN (Pyke et al, 1984). Among them, an effective one is TELDYN which uses equivalent linear method, provides compliant base and determines the excess pore water pressures.

## **2.4 Equivalent Linear Model**

In case of earthquake response analyses, properties of soils under dynamic conditions are nonlinear and hysteretic. The equivalent linear model has been most widely used for the representation of this nonlinear behavior in embankment and foundation soils. Shear stress-shear strain relationships for soils subjected to cyclic loading are nonlinear and exhibit hysteresis loop of the type as shown in Figure 2.5. As can be seen from the Figure 2.5 the average (equivalent linear) shear modulus can be represented by the secant modulus drawn through the ends of the hysteresis loops.

As the cyclic shear strain amplitude increases, the average shear modulus decreases and the hysteretic damping, as indicated by the area enclosed by the shear strain curve increases.

Instead of hysteretic damping, equivalent damping ratio can be described by using the following formula :

$$\mu = \frac{1}{4\pi} \left[ \frac{\Delta W}{W} \right] \quad (2.14)$$

where  $\Delta W$  is the dissipated energy represented by the area of hysteresis loop,  $W$  is the maximum strain energy represented by the area of the OAB triangle.

As explained above secant shear modulus of the soil element varies with cyclic shear strain amplitude. At low strains, the secant shear modulus ( $G_{sec}$ ) is high, but it decreases as the shear strain amplitude increases. This variation of  $G_{sec}$  with cyclic shear strain can be shown with a curve called backbone as seen in Figure 2.6. In the figure tangent modulus at the origin represents the largest value of shear modulus ( $G_{max}$ ),  $\gamma_c$  is the current strain and  $G_{sec}$  (hereafter instead of  $G_{sec}$  only  $G$  is used for notation) is the corresponding shear modulus. From the backbone curve the modulus reduction curve indicating the variation of  $G / G_{max}$  ratio with shear strain can be determined as shown in Figure 2.7.

In the early years of geotechnical earthquake engineering, Seed and Idriss (1970) suggested modulus reduction curves and damping curves for coarse and fine-grained soils. Then, Seed (1984) and the other researches such as Hardin and Drnevich (1972), Vucetic and Dobry (1991), Ishibashi (1992) modified the curves according to mean effective stress and plasticity index. The curves commonly used for cohesionless soils proposed by Seed (1984) and for cohesive soils proposed by Vucetic and Dobry (1991) can be seen in Figure 2.8 and 2.9 respectively.

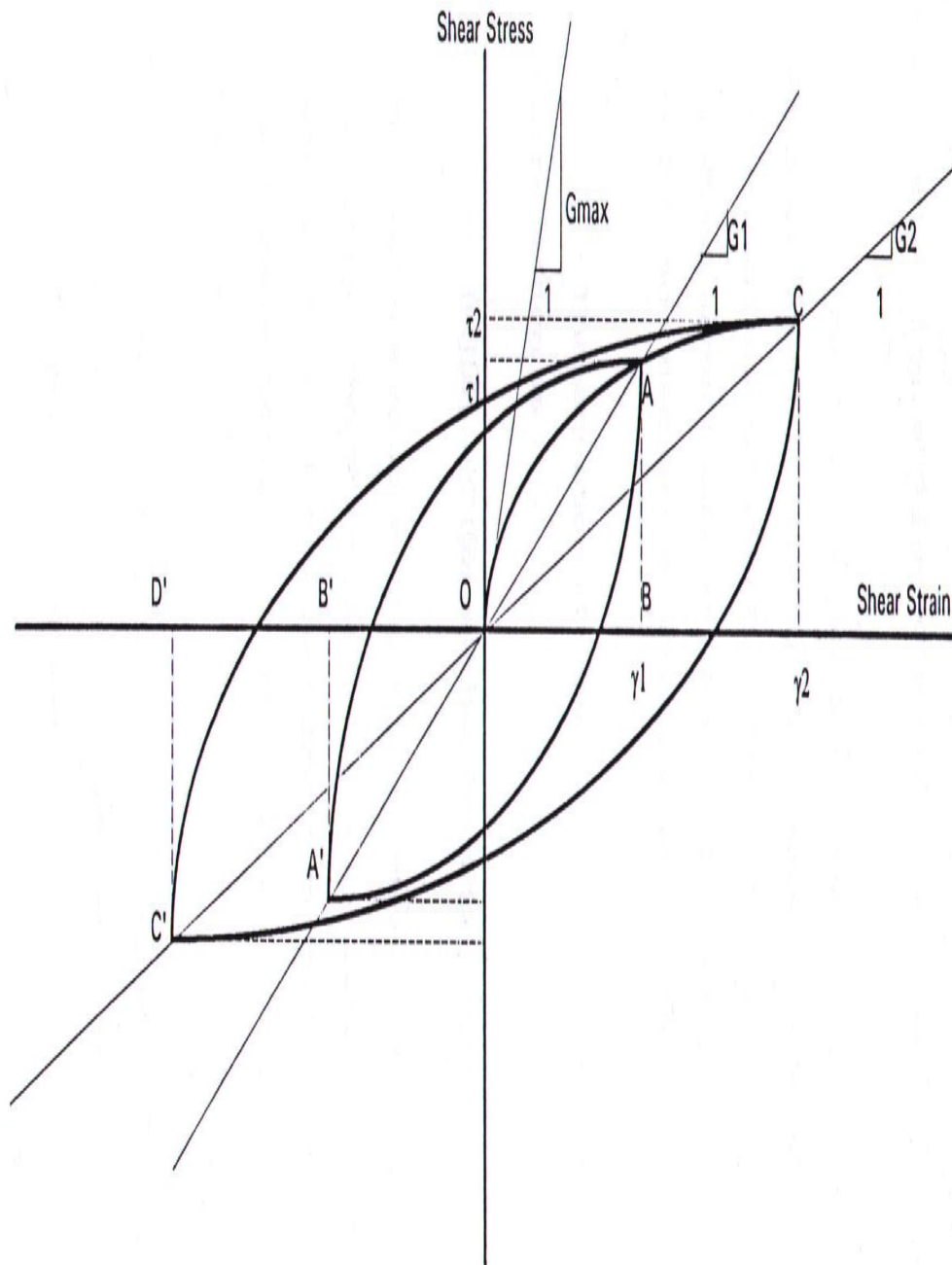


Figure 2.5 Hysteric Loop Indicating Shear Stress Relationship for Soils Subjected to Cyclic Loading

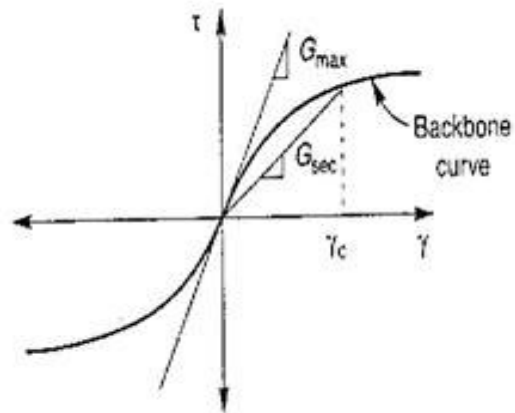


Figure 2.6 Backbone Curve Showing Typical Variation of  $G_{sec}$  with Shear Strain

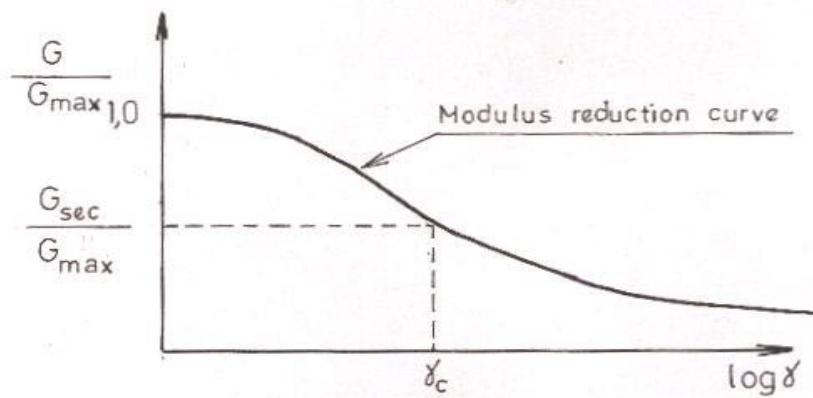


Figure 2.7 The Variation of the Modulus Ratio with Shear Strain

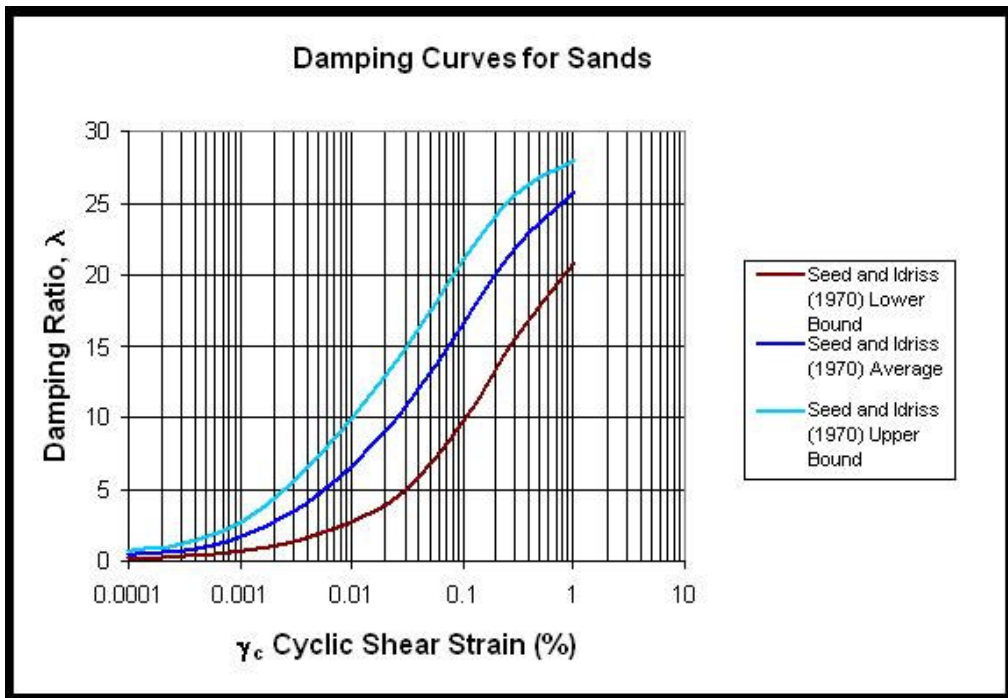
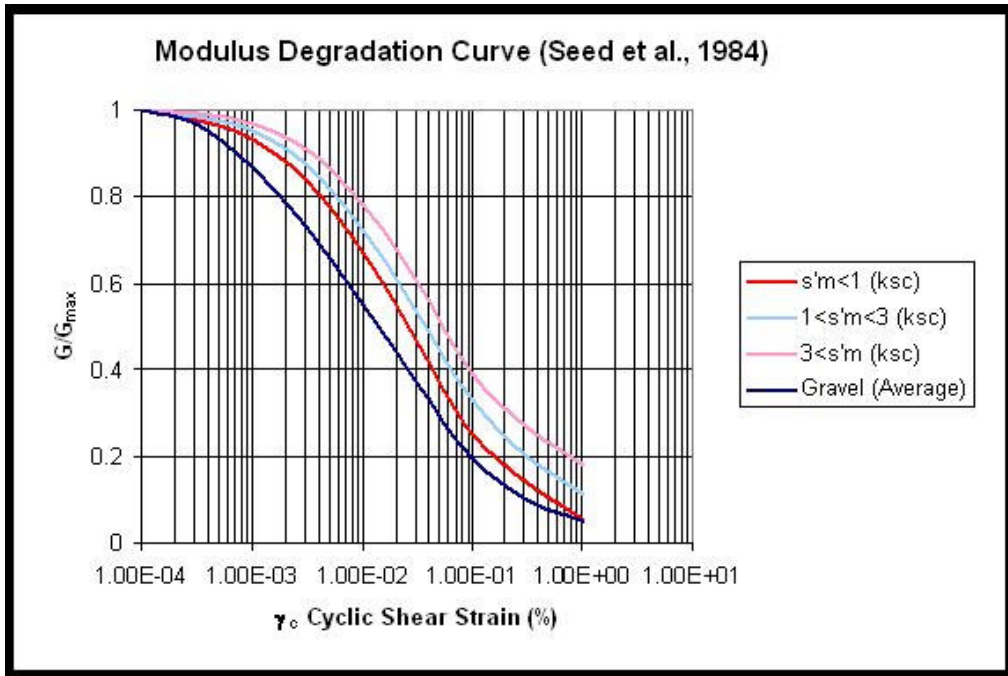


Figure 2.8 Modulus Degradation and Damping Curves for Cohesionless Soils (Seed et al. 1984)

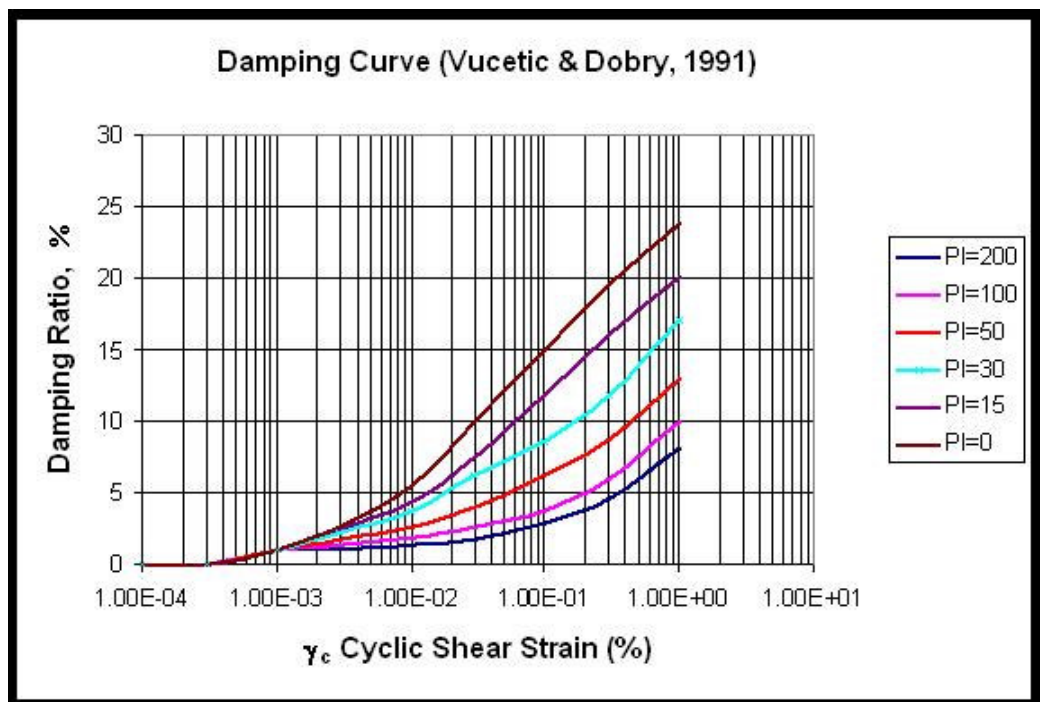
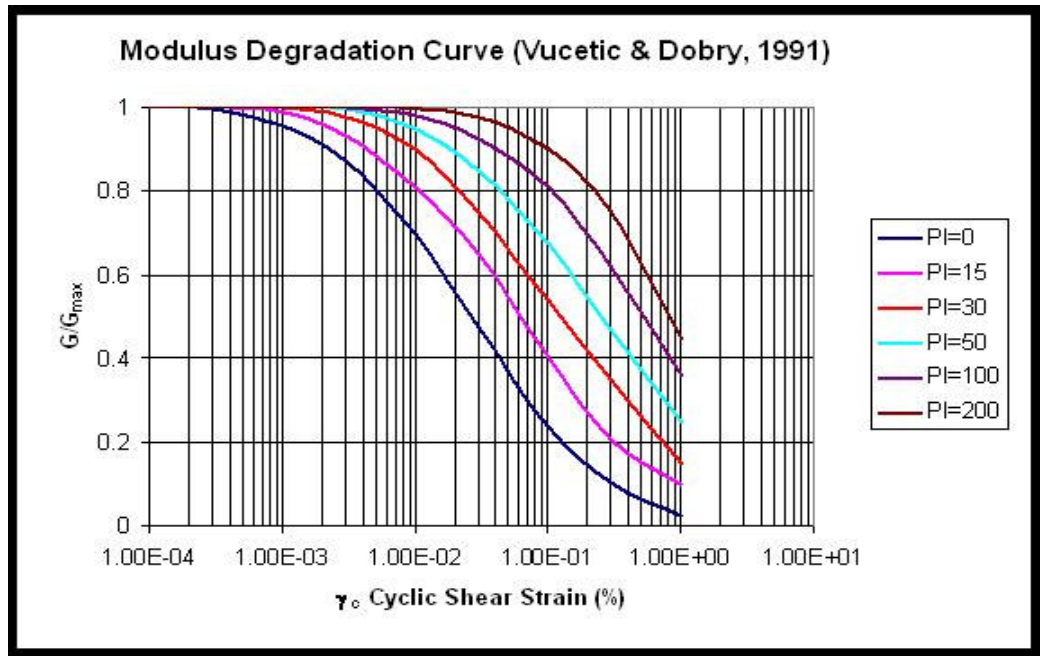


Figure 2.9 Modulus Degradation and Damping Curves for Cohesive Soils (Vucetic & Dobry 1991)

As seen from the modulus reduction curves that show the decrease of shear modulus at larger strains, the stiffness of the soil is characterized by the maximum shear modulus  $G_{\max}$ . Using the shear wave velocities measured in seismic field tests or in laboratory tests at low strains,  $G_{\max}$  can be computed from the following formula:

$$G_{\max} = \rho V_s^2 \quad (2.15)$$

where  $\rho$  is the mass density and  $V_s$  is the shear wave velocity of soil.

The maximum shear modulus  $G_{\max}$  can be determined in several different ways. It can be estimated from the empirical relationships between  $G_{\max}$  and various parameters obtained in laboratory tests or in-situ tests such as SPT and CPT. For instance, a common relationship used for the determination of  $G_{\max}$  for sands is:

$$G_{\max} = 1000 K_{2,\max} (\sigma'_m)^{0.5} \quad (2.16a)$$

$$G_{\max} = 220 K_{2,\max} (\sigma'_m)^{0.5} \quad (2.16b)$$

in which  $K_{2,\max}$  is a coefficient that varied from about 30 to 80 for clean sands and from about 80 to 180 for gravelly soils,  $\sigma'_m$  is the mean effective stress (Seed et al.,1984).

## 2.5 Permanent Displacement Analysis

As explained before, the pseudostatic method of analysis, like all equilibrium methods, is limited because it provides only a safety factor of the slope, but no information regarding seismically induced permanent deformations. Newmark (1965) developed a displacement-based method to predict these seismically induced permanent deformations. In this approach, the mass of soil located above the critical failure surface soil is represented as a rigid block resting on an inclined plane as shown in Figure 2.10. When the block is subjected to acceleration caused by the ground motion which is greater than the yield acceleration, the driving forces may exceed the resisting forces. Thus, the block

slides along the inclined plane. The resisting and the driving forces acting on the sliding block are illustrated in Figure 2.11.

Determination of the yield acceleration is the most critical step of the analysis. The yield acceleration  $a_y$  is the minimum pseudostatic acceleration required to cause the block to move relative to sliding plane. It can be obtained by using the following equation:

$$a_y = k_h \cdot g \quad (2.17)$$

where  $k_h$  is the horizontal seismic coefficient calculated in pseudostatic analysis which is explained in Section 2.3.1.

When a block on an inclined plane is subjected to accelerations greater than the yield acceleration, the block will move relative to plane. Thus, the relative acceleration occurring the displacement can be written as follows:

$$a_{rel}(t) = a_b(t) - a_y \quad (2.18)$$

where  $a_b(t)$  is the acceleration of inclined plane.

Thus, by computing an acceleration at which the inertia forces become sufficiently high to cause yielding to begin and integrating the effective acceleration on the sliding mass in excess of this yield acceleration as a function of time (Figure 2.12), the velocities and ultimately displacements of the slide mass could be evaluated (Seed et al.,1979).

The time history acceleration of the inclined plane,  $a_b(t)$ , can be considered as the average time history acceleration of the sliding mass. In order to determine the average time history acceleration,  $a_{ave}$ , following steps should be carried out:

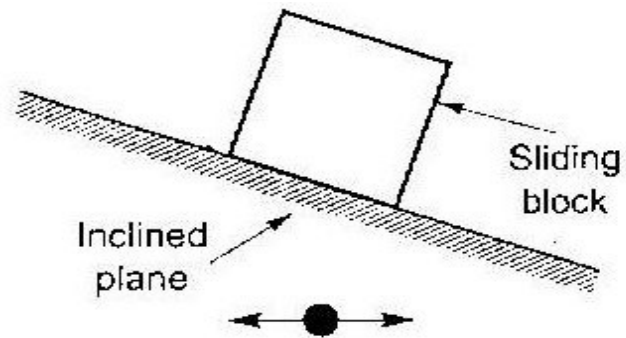


Figure 2.10 Sliding Block Resting on an Inclined Plane

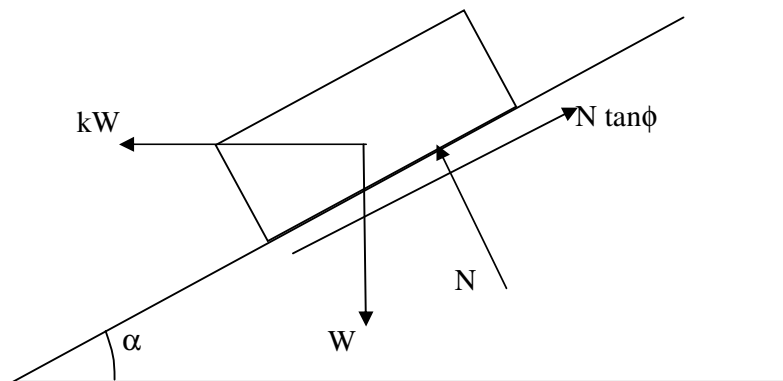


Figure 2.11 Forces Acting on the Sliding Block

i) Sliding mass is divided into finite elements or finite strips.

ii) The average time history of acceleration is calculated for each element by using the dynamic finite element analysis.

iii) The time history of force on an element is obtained by multiplying the acceleration of each element with its mass:

$$F_e(t) = m_e \cdot a_e(t) \quad (2.19)$$

where  $m_e$  is the mass of an element and  $a_e(t)$  is the time history of acceleration of an element.

iv) Total force acting on the sliding mass can be calculated by summing the forces acting on elements:

$$F_b(t) = \sum F_e(t) = \sum m_e \cdot a_e(t) \quad (2.20)$$

v) In the last step, the average time history acceleration of the sliding mass is determined by dividing total force by total mass of the sliding mass:

$$a_{ave} = \frac{F_b(t)}{m_b} = \frac{\sum m_e \cdot a_e(t)}{\sum m_e} \quad (2.21)$$

Consequently, as explained before, by integrating twice average time history of acceleration, permanent displacement of the slope can be calculated.

Makdisi and Seed (1978) developed the Newmark's permanent displacement method by using the sliding block analyses and average accelerations computed by the procedure of Chopra (1966). In this approach, knowing the

fundamental period of embankment and the yield acceleration of the slope, simple charts can be used to estimate earthquake-induced permanent displacements. Furthermore, Lemos and Coelho (1991) and Tika Vassilikos et al. (1993) have both suggested methods that can incorporate a rate dependent friction angle into the Newmark analysis to account for time varying shear strengths due to earthquake loading. Although a number of modified permanent displacement methods have been proposed, today Newmark (1965) type of analysis is widely used by the geotechnical engineers.

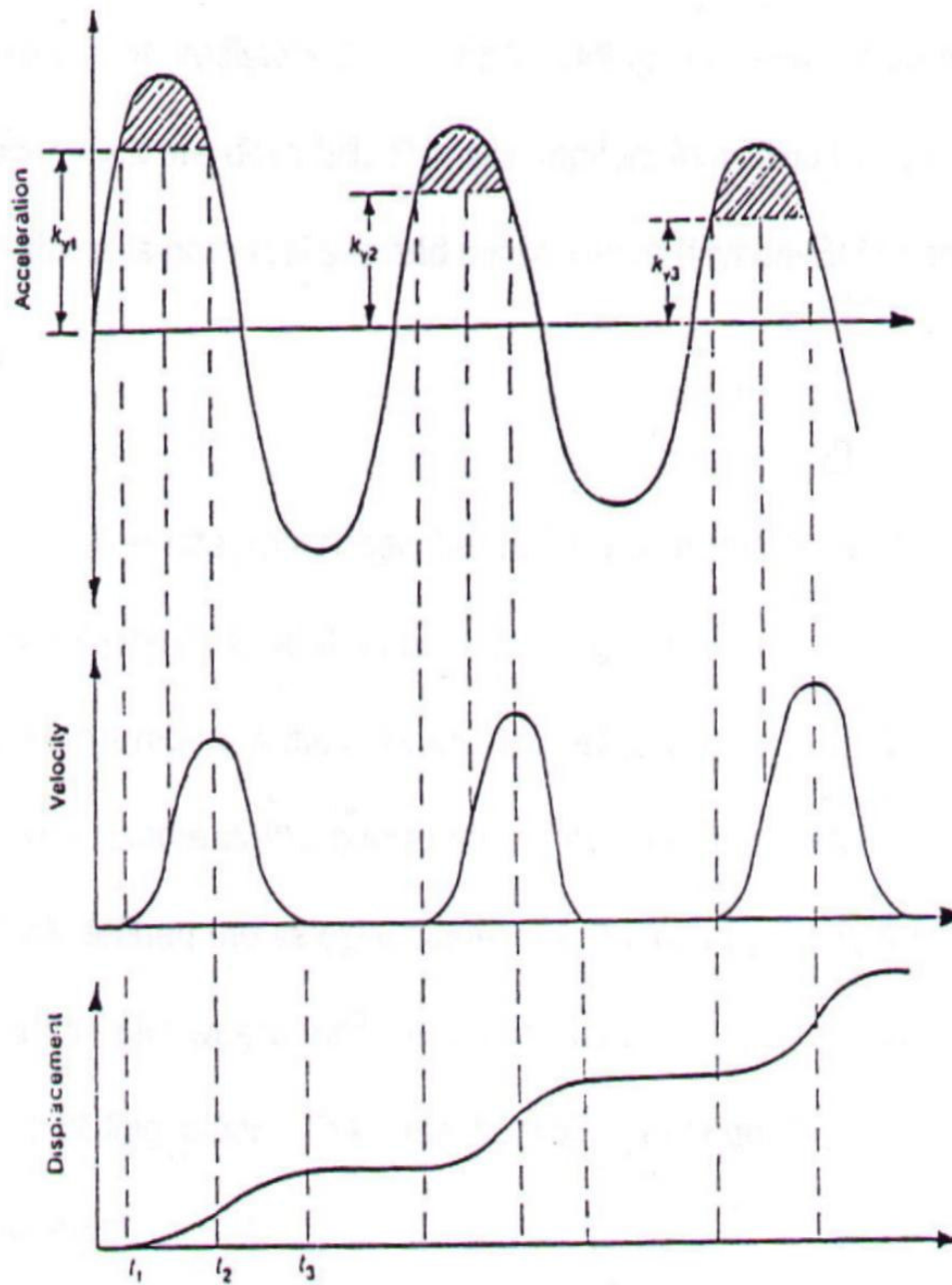


Figure 2.12 Integration of Effective Acceleration Time-History to Determine Velocities and Displacements (Seed et al., 1979)

## CHAPTER III

### AKKÖPRÜ DAM AND SEISMICITY OF THE SITE

#### 3.1 General Information About Dalaman - Akköprü Dam

Dalaman-Akköprü Dam is a rockfill dam located at Dalaman Basin in the southwest of Aegean Region, being built on the Dalaman River, 24 km east of the Köyceğiz Town in Muğla. Construction of the dam was started in 1995 and it is planned to be completed in 2005. The main purposes of this project are the hydropower generation, irrigation and flood control. The general view of Akköprü Dam is given in Figure 3.1. (DSİ, 2003)

##### 3.1.1 Characteristics of the Project

###### Dam Reservoir Data

Rainfall area	: 5132.60 km <sup>2</sup>
Annual average of water	: 1627.40 hm <sup>3</sup>
Regulated water	: 849.70 hm <sup>3</sup>
Regulation	: 52 %
Minimum water level	: 173.50 m
Normal water level	: 200.00 m
Maximum water level	: 204.00 m (In flood)
Minimum water level volume	: 195.81 hm <sup>3</sup>
Normal water level volume	: 384.50 hm <sup>3</sup>
Maximum water level volume	: 419.20 hm <sup>3</sup>
Maximum reservoir area	: 8.92 km <sup>2</sup>



Figure 3.1 Photo of Akköprü Dam Taken at the end of year 2003

### **Dam Embankment Data**

Type	: Rock-fill with clay core
Crest elevation	: 207.50 m
Crest length	: 688.70 m
Crest width	: 12.00 m
Height from thalweg	: 112.50 m
Height from foundation	: 162.50 m
Total body volume	: 12.50 hm <sup>3</sup>

### **Spillway Data**

Type	: Radial gate, 4 gates
Dimensions of the gate	: 11.60 x 15.71 m
Spillway top elevation	: 188.50 m
Spillway design flood	: 5730.00 m <sup>3</sup> / s
Discharge channel length	: 445.37 m

### **Hydroelectric Power Plant (Powerhouse)**

# of Units	: 2 Units
Turbine type	: Vertical axis Francis
Unit capacity	: 57.5 MW
Installed capacity	: 115 MW
Rated head	: 101 m
Design discharge	: 150 m <sup>3</sup> / s
# of cycles of the unit	: 250 rpm
Annual energy production	: 343.5 GWh

### **Diversion Tunnels**

Tunnel diameter	: 6.50 m
Length - Derivation tunnel 1	: 766.62 m
Length - Derivation tunnel 2	: 808.44 m

### **Bottom Outlet**

Type	: Circular 2 pieces
Length	: 315.50 m (for both tunnels)
# of Danger valve type and dimension	: Slide valve, 1 piece, 180 cm x 180 cm (for both tunnels)
# of Adjusting valve, type and dimension	: Conic valve, 1 piece, 200 cm (for both tunnels)
Bottom outlet capacity	: 145.44 m <sup>3</sup> / s

### **Energy (Power) Tunnel**

Tunnel diameter	: 6.50 m
Length	: 532.00 m
Vertical shaft length	: 72.00 m

### **Penstock**

Pipe diameter	: 6.45 – 3.23 m
Length	: 204.00 m

### **3.1.2 Geology and the Foundation**

The base rock at the Akköprü Dam site is peridotite-serpentine. This unit is generally impermeable. Some parts are semi-pervious and 60 m deep grout curtain

beneath the foundation dam upstream and downstream cofferdam axis were constructed by slurry trench method.

Alluvium thickness of the dam site is 38 m. Alluvium is permeable and it was excavated prior to the construction of the dam.

At right abutment 37.20 m thick terrace deposits were observed and excavated.

The base rock at the reservoir area is composed of autochthonous Aktaş limestone and Gökseki flysch formation. Allochthonous Cehennem limestone and Demirli complex serie (melange) overlay autochthon units. Aktaş limestone and Cehennem limestone formations are permeable. The karstification base and discharge points in Cehennem limestone are located over maximum water level. So, no big problems related to leakage and seepage are expected from this unit (Şekercioğlu et al., 1999).

Aktaş limestone is very permeable and groundwater level elevation is located around 116 m below the Dalaman river. In order to prevent leakage from Aktaş limestone , concrete blanket is planned to be constructed.

### **3.1.3 The Dam Embankment**

Akköprü Dam is a rockfill dam having a large clay core, sand and gravel filter zones. The plan view and the maximum cross section of the embankment can be seen in Figure 3.2 and 3.3 respectively. The height of the dam is 162.50 m from the lowest foundation level and 112.50 m from the thalweg. Its crest is 688.70 meters long and 12.00 meters wide. The total dam body volume is 12.30 hm<sup>3</sup>. Downstream slope of the dam is 1(V) / 2(H) and upstream slope of the dam is 1 (V) / 2.5 (H).

The main properties of the materials used at the construction of the dam body as provided by State Hydraulic Works (DSİ) are given in Table 3.1.

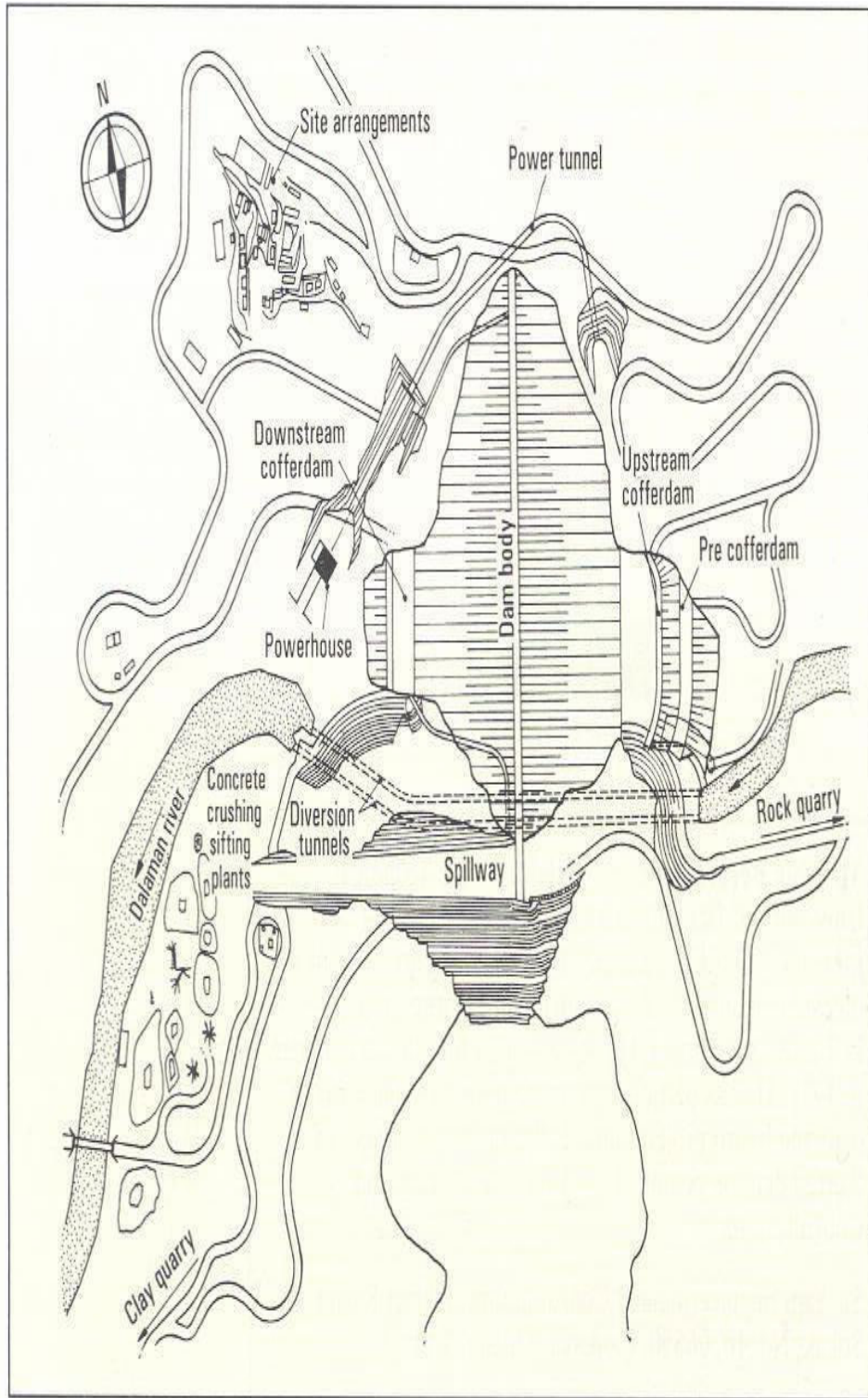


Figure 3.2 Plan View of Akköprü Dam

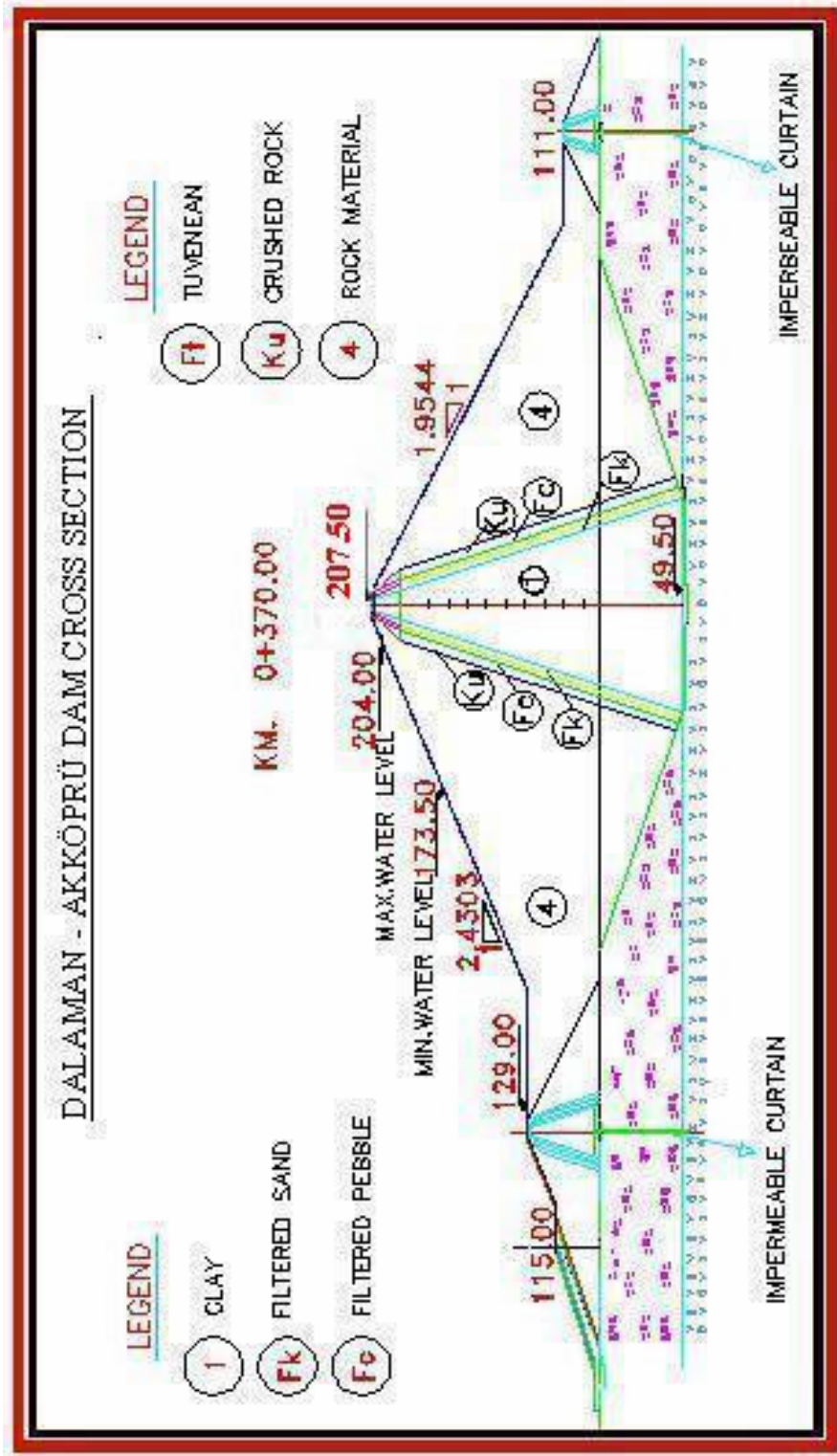


Figure 3.3 Cross Section of Akköprü Dam

Table 3.1 Properties of Dam Embankment Material

<b>Material Type</b>	<b>Dry Density  kN/m<sup>3</sup></b>	<b>Wet Density  kN/m<sup>3</sup></b>	<b>Saturated Density  KN/m<sup>3</sup></b>	<b>Angle of Friction  (<math>\phi</math>) (<math>^{\circ}</math>)</b>	<b>Cohesion Undrained  (C)  kN/m<sup>2</sup></b>
Core Material Clay (CL)	16.5	19.7	20.4	20	40
Grade Filter Material	17.5	18.5	21.5	33	0
Fill material Rock	19.5	20.0	21.0	43	0
Subbase material Alluvium	16.6	17.4	20.2	33	0
Base material Rock	23.0	23.0	23.0	25	20

### 3.2 Seismicity of Akköprü Dam Site

Akköprü Dam is located at southwest of Aegean Region and Eastern Mediterranean Sea. The important normal faults in Turkey exist in the West Anatolia. These faults form the graben system and indicate the deformation continuity occurring in the Aegean-Western Anatolia plate.

There is a wide earthquake zone near the Aegean Sea and Eastern Mediterranean Sea Earthquakes of both shallow focal depth ( $50 \text{ km} < h$ ) and mid focal depth ( $50 \text{ km} < h < 200 \text{ km}$ ) (Alptekin 1978). Investigations made by State Hydraulic Works (DSİ) in this earthquake zone indicate that there are four main regions:

- 1<sup>st</sup> region includes the west and southwest of the Datça Peninsula.
- 2<sup>nd</sup> region includes Köyceğiz, Ula, Çameli, Fethiye and Dalaman

- 3<sup>rd</sup> region includes Acı Lake, Burdur Lake and Söke
- 4<sup>th</sup> region includes Aydın, Denizli and Burdur.

Tectonic map of the earthquake zone including four regions stated above investigated by DSİ is shown in Figure 3.4. Furthermore macro seismic maps of Fethiye Earthquake, Aegean – Mediterranean Earthquake are given in Figure 3.5 and 3.6, respectively.

It is observed that 770 earthquakes with magnitudes greater than 4.0 have occurred in the region bounded by 35.00° - 38.00° N and 27.00° - 30.00° E during the period 1900 – 2002. Epicenter map of the earthquakes with magnitudes greater than 4.0 and a list of the earthquakes with their occurring dates, epicenter locations, focal depths and magnitudes are given in Appendix A.

### **3.3 Design Earthquake Ground Motion**

The procedures employed and the results are obtained in the assessment of engineering parameters of ground motion at the site are presented in this section. First the results of probabilistic studies performed earlier are reviewed. Next the deterministic studies carried out to estimate the peak ground motion parameters are summarized.

#### **3.3.1 Probabilistic Assessment**

Yaralıoğlu et al. (1982) had studied the seismicity of the region around Akköprü dam site. In this work to find the acceleration-risk values an investigation region is selected between the geographical coordinates 36°00 - 38°00 N and 27°00 - 32°00 E and a tectonic map of the region is prepared as seen in Figure 3.4 (Yaralıoğlu et al., 1982). On this map major sismotectonic features of the region are idealized as a line and three area sources as seen in Figure 3.4.

TEKTONİK HARİTASI - MAP OF TECTONIC

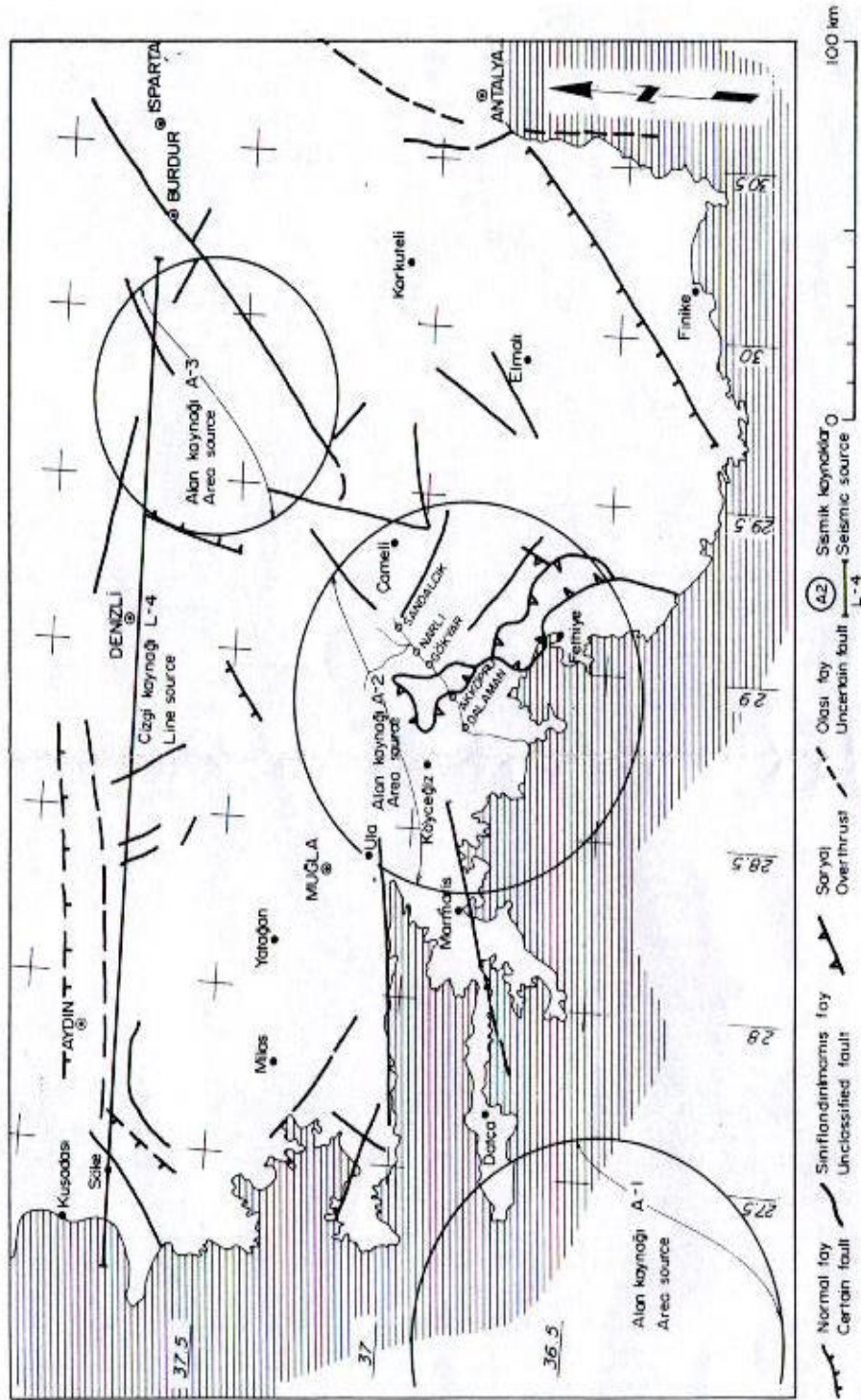


Figure 3.4 Tectonic Map of the Earthquake Zone Surrounding the Akköprü Dam Site (Yaralıoğlu et al., 1982)

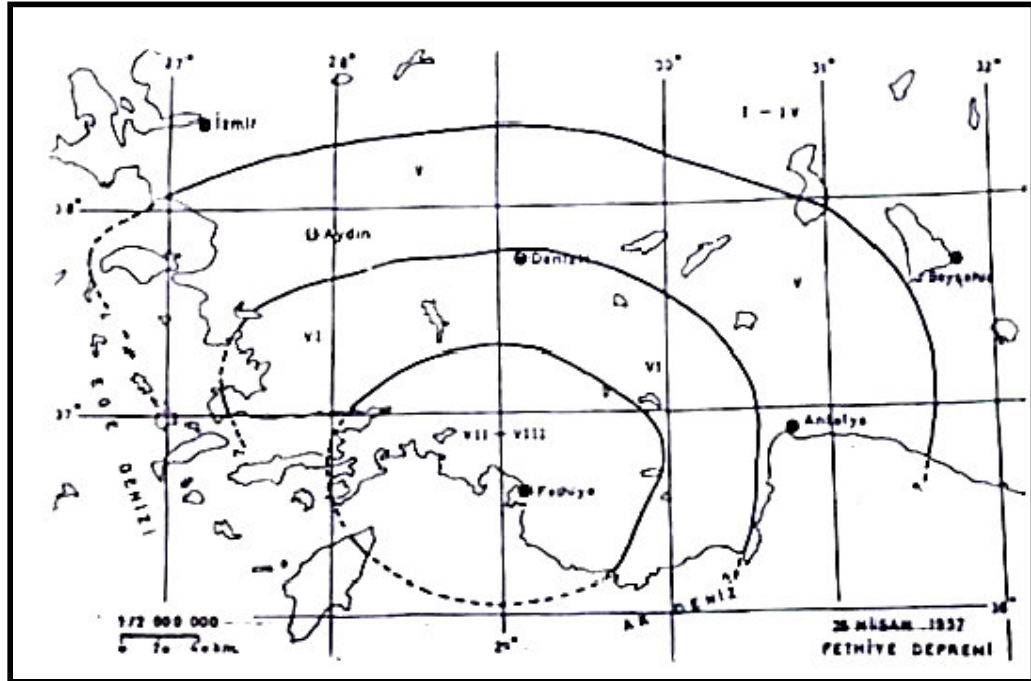


Figure 3.5 Macroseismic map of 1957 Fethiye Earthquake ( Ergin et al., 1967)

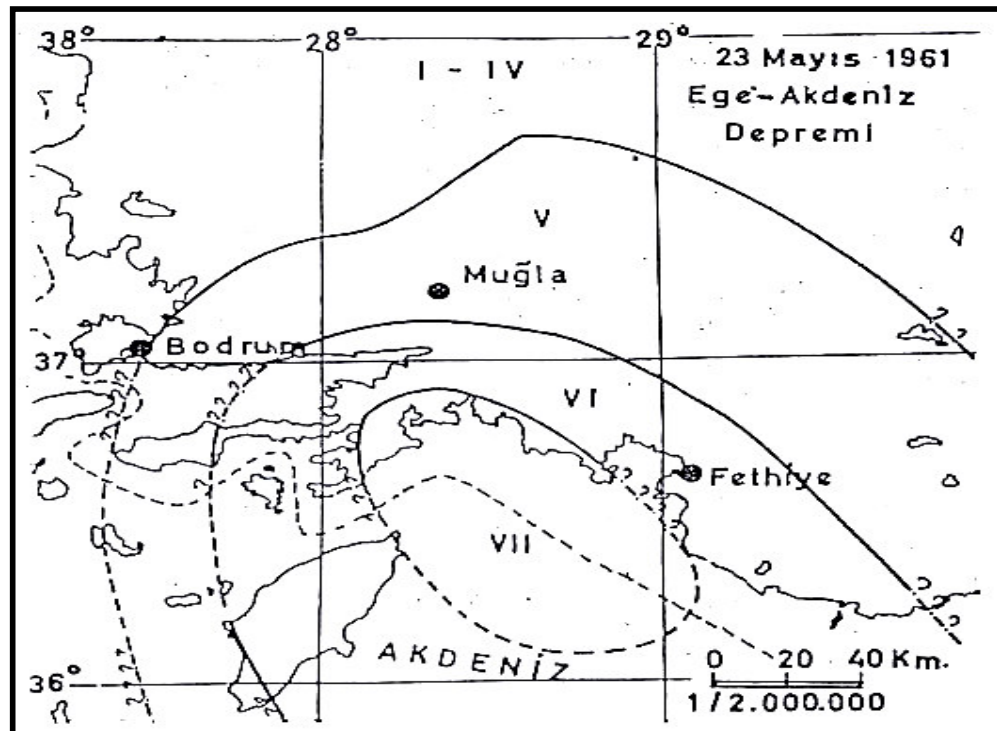


Figure 3.6 Macroseismic map of 1961 Aegean –Mediterranean Earthquake ( Ergin et al., 1967)

Based on this seismotectonic model Yaralıoğlu et al. (1982) had performed a Cornell-type seismic risk analysis in terms of peak ground acceleration. The attenuation relationships developed by Esteva (1970) had been employed in the analysis which did not take into account the effects of the local site conditions at recording stations.

Summary of this results of seismic risk analysis by Yaralıoğlu et al. (1982) are given in Table 3.2, 3.3 & Figure 3.8, 3.9.

Table 3.2 Expected Peak Ground Acceleration Values at Akköprü Dam Site

<b>Probability of exceedence</b>	<u>0.05</u>	<u>0.10</u>	<u>0.20</u>
<b>Economic Life</b>	<b>Peak Ground Acceleration cm/sn<sup>2</sup></b>		
1 year	72.5	48.7	43.5
50 years	296	253.7	200.5
100 years	352.2	294.7	249.7
1000 years	666.1	511.7	389.7

Table 3.3 Probability of exceedence of PGA at Akköprü Dam Site

<b>PGA(cm/s<sup>2</sup>)</b>	10	50	100	150	200	250	300	400	500	1000
<b>Economic Life</b>	<b>Probability of exceedence of Peak Ground Acceleration (PGA) %</b>									
1 year	85	7.7	1.95	0.87	0.43	0.22	0.09	0.014	0.08	0.0014
50 years	100	98	69	35	19	10	4.5	0.7	0.42	0.08
100 years	100	99.9	86	58	35	10	8.5	1.45	0.83	0.15
1000 years	100	100	100	99.9	98	88	60	13	7.	1.64

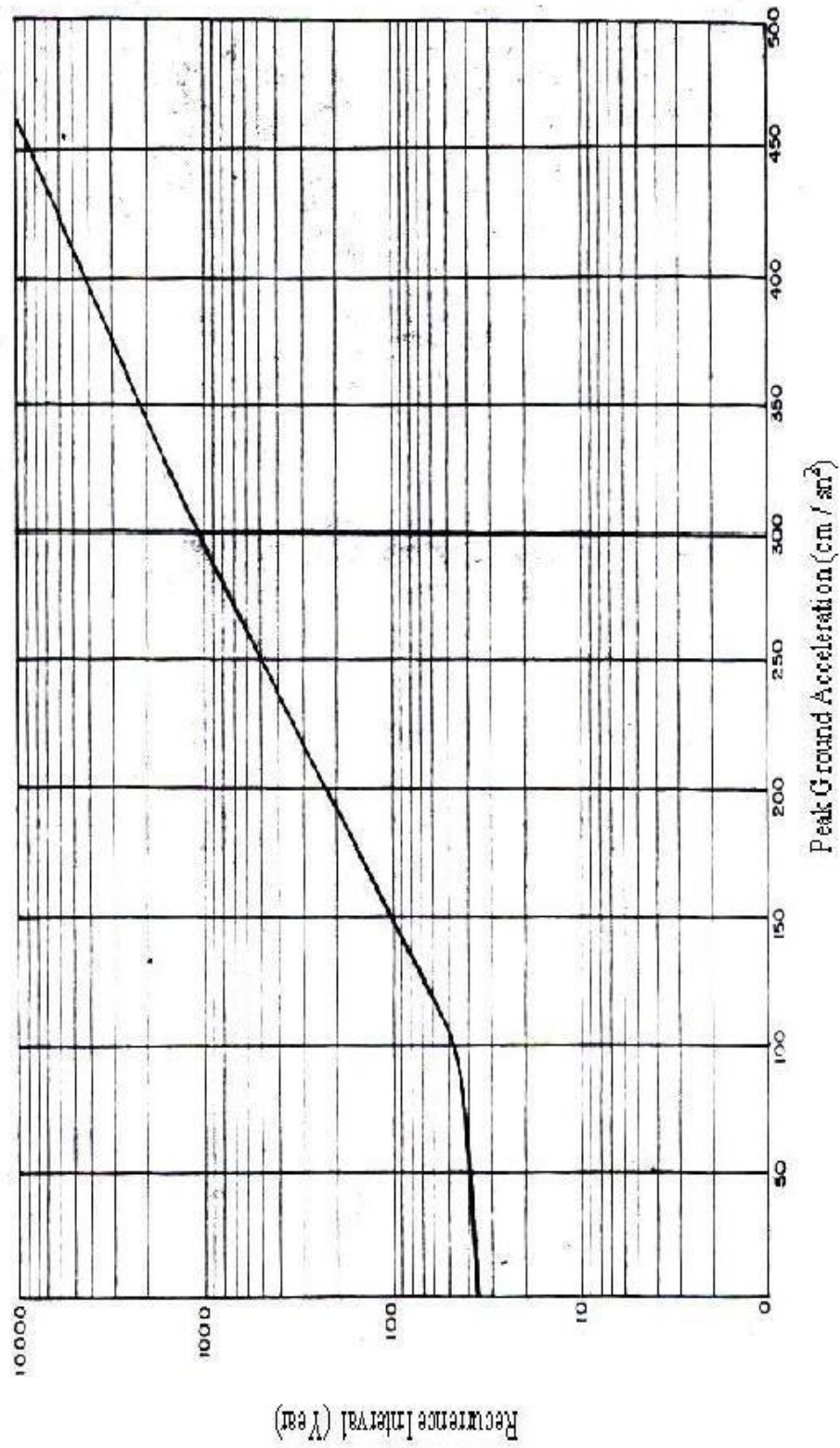


Figure 3.7 Regional Peak Ground Acceleration Graph of Akköprü Dam

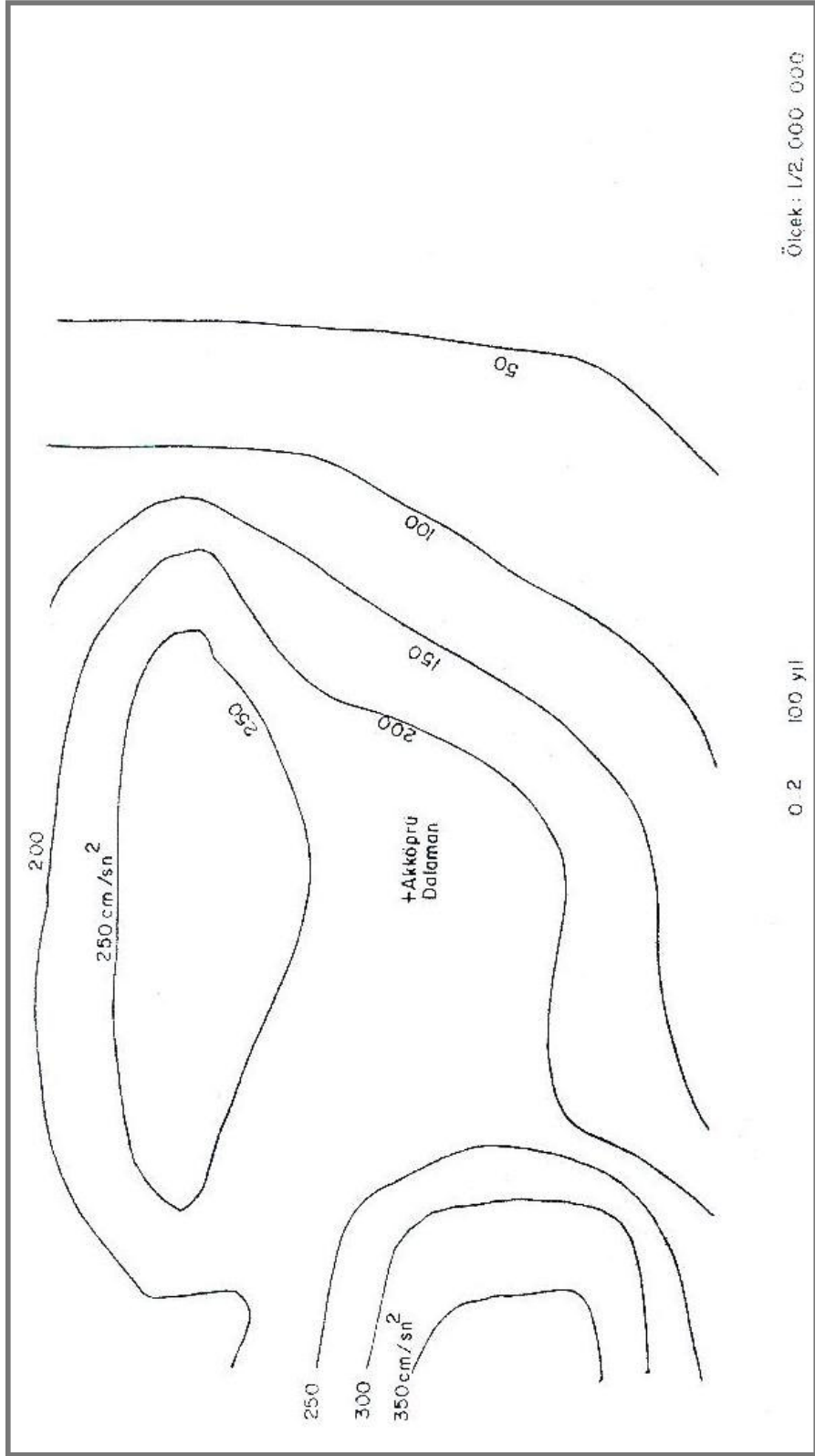


Figure 3.8 Equipotential Acceleration Curves of Akköprü Dam with 20 % Probability of Exceedence for 100 Year Economic Life

### **3.3.2 Deterministic Assessment**

The seismicity of the region had been fully investigated in the probability studies by Yaralıoğlu et al. (1982). The map in Figure 3.4 indicates the main tectonic features of the region under consideration. The largest event for this period was 25.04.1957 Fethiye Earthquake which has a magnitude of about 7. The details of the seismicity may be found in the references cited.

From the results of probabilistic studies and the seismotectonic features of the region, it is concluded that a large magnitude event occurring in fault near Marmaris would govern the design earthquake ground motions at the site of Dalaman Akköprü Dam.

For an assessment of design earthquake motion parameters, an earthquake having a large magnitude of  $M=7$  is considered. The distance to the causative fault is assumed to be 28 km.

#### **3.3.2.1 Acceleration – Attenuation Approach**

Estimation of ground motion parameters for a certain location at a given distance from the epicenter or the causative fault of an earthquake are made by the use of empirical relationships. Attenuation relationships that express ground motion parameters as a function of magnitude, distance, type of faulting and site conditions. The most commonly used ground motion parameter is PGA (Peak Ground Acceleration). In order to estimate the PGA value at Akköprü Dam site Idriss et al. (1991), Boore et al.(1993) and Kalkan (2001) attenuation relationships explained in Chapter II are used . Magnitude and the distances are selected as stated above. Local soil conditions and type of faulting are also necessary. The geology of the dam site generally consists of rock formations.

The results of the acceleration attenuation approach of the deterministic study are tabulated as follows in Table 3.4.

Table 3.4 Expected Peak Ground Acceleration Values at Dam Site

	<b>PGA for <math>M_w=7.0</math> and <math>R=28</math> km</b>
<b>Idriss (1991)</b>	0.160g
<b>Boore (1993)</b>	0.112g
<b>Kalkan(2001)</b>	0.086g

### 3.4 Design Peak Ground Acceleration

In the probabilistic assessment peak ground acceleration was determined as approximately 0.25g with the probability of 20 % exceedence in 100 years which is equivalent to occurrence of once in 450 years. In the deterministic assessment from attenuation relationships by Idriss (1991), Boore (1993) and Kalkan (2001) the peak ground accelerations were calculated as 0.160g, 0.112g and 0.086g respectively. According to these results design peak ground acceleration was selected as 0.20g.

### 3.5 Estimating Input Ground Motion

There are no actual earthquake motion records available at the project site. Hence, in order to make dynamic analyses of Akköprü Dam, two actual earthquakes taken from European Strong Motion Database were used and given in Table 3.5 and two synthetic earthquake motions were generated by XS artificial motion generation program. Based on the historical seismic events at the dam site the maximum probable earthquake magnitude of  $M_w=7$  was estimated for the analyses. The most important parameters of the earthquake hazard motion are the peak ground acceleration, predominant period (frequency content) and the duration of the earthquake. As mentioned before, the most critical fault is at a distance of 28 km from the Akköprü Dam Site. Thus, predominant period is found to be as about 0.30 sec from the suggested relationships by Seed et al. (1969) given in Figure 3.9.

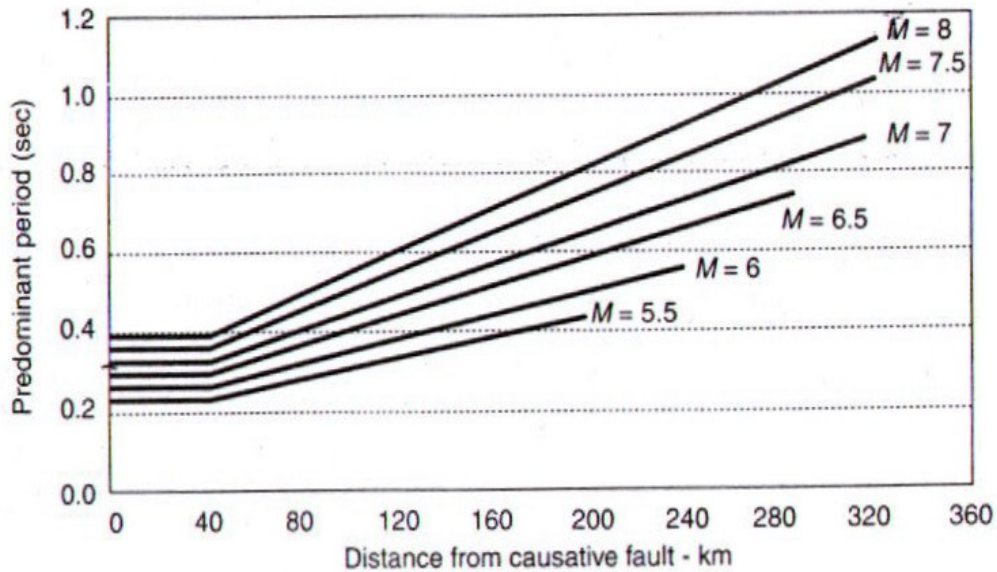


Figure 3.9 Variation of predominant period at rock outcrops with magnitude and distance. (After Seed et al., 1969)

Seed et al. (1964) proposed 20 seconds for the duration of strong ground motions having a magnitude of 7 and Arias (1970) suggested the duration to be about 15 seconds. Based on these estimations, the duration of the strong ground motion was selected between 15-20 seconds.

After selecting the most important parameters of the earthquake ground motion, actual earthquakes and the artificial earthquakes were scaled to 0.15g, 0.20g and 0.25g respectively. Furthermore, to reflect the assumed situation predominant periods and the durations were calibrated by using the computer program NONLIN (Nonlinear Dynamic Time History Analyses of Single Degree of Freedom Systems). The acceleration time histories and the acceleration response spectra of the design ground motions used in the analyses are given in Figure 3.10 and 3.11.

Table 3.5 Actual Seismic Records Taken from European Strong-Motion Database

Event Type	Event Name	Date	Ms	Mw	D(km)	Soil
Normal	Compano Lucano(Italy)	23.11.1980	6.87	6.9	14	Rock
Thrust	Montenegro(Yugoslavia)	15.04.1979	7.03	7	29	Rock

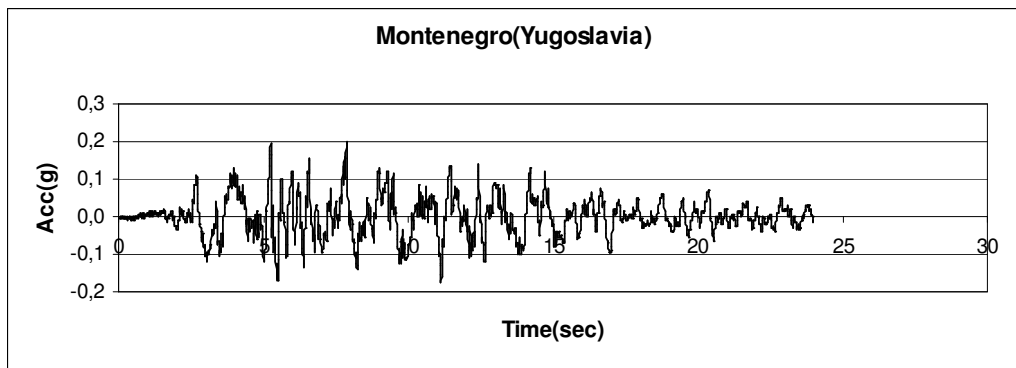
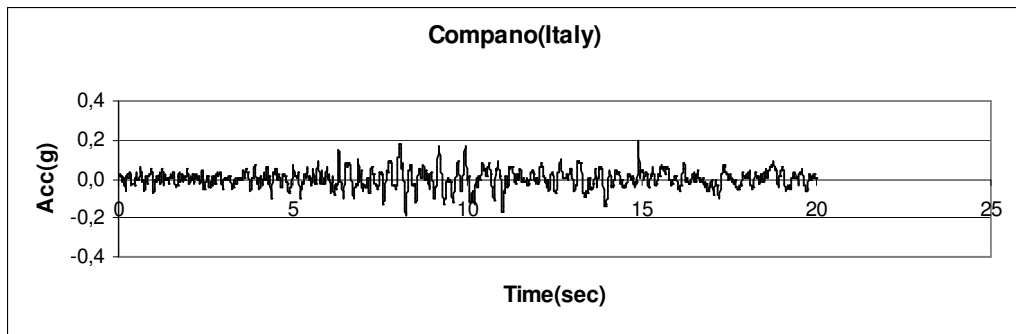
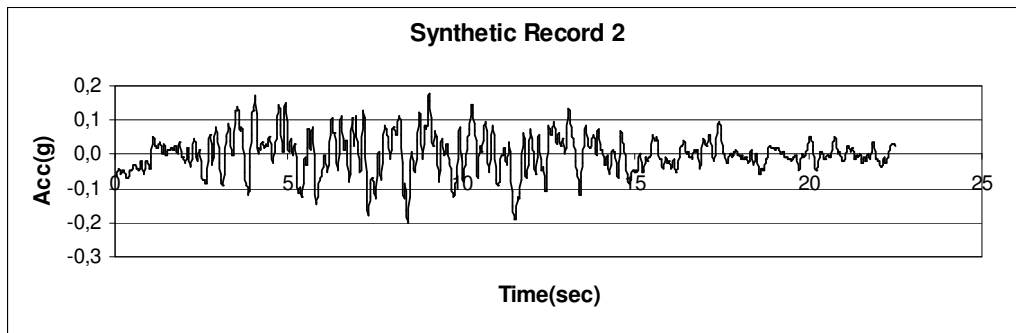
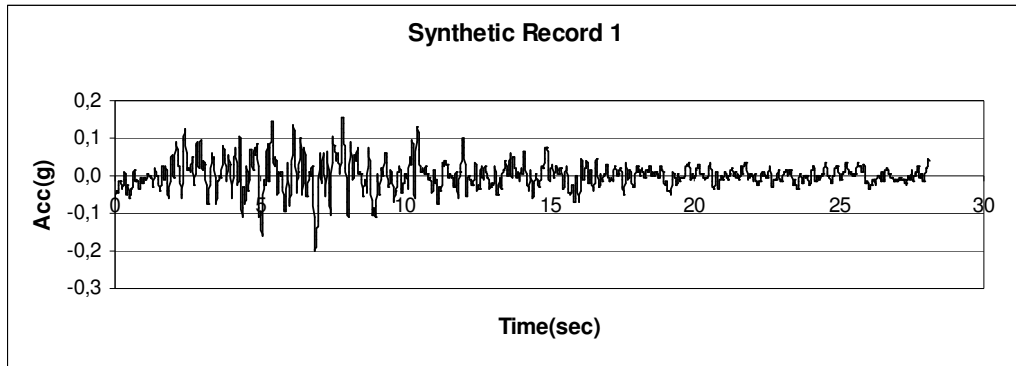


Fig 3.10 Acceleration Time Histories of Design Ground Motions

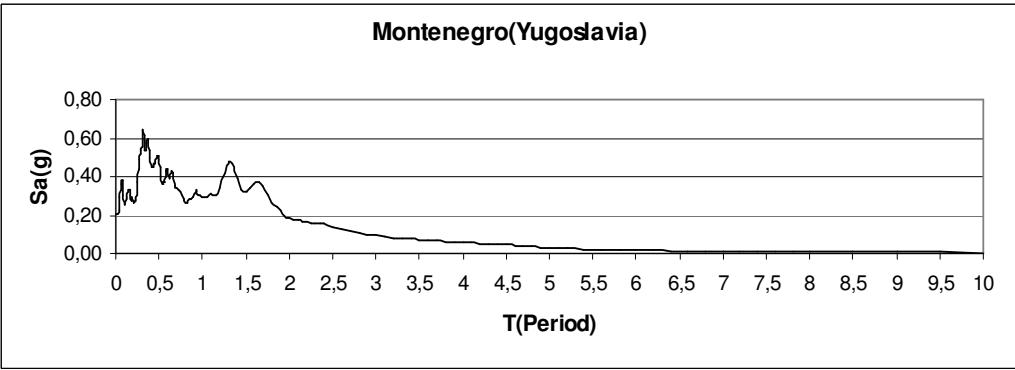
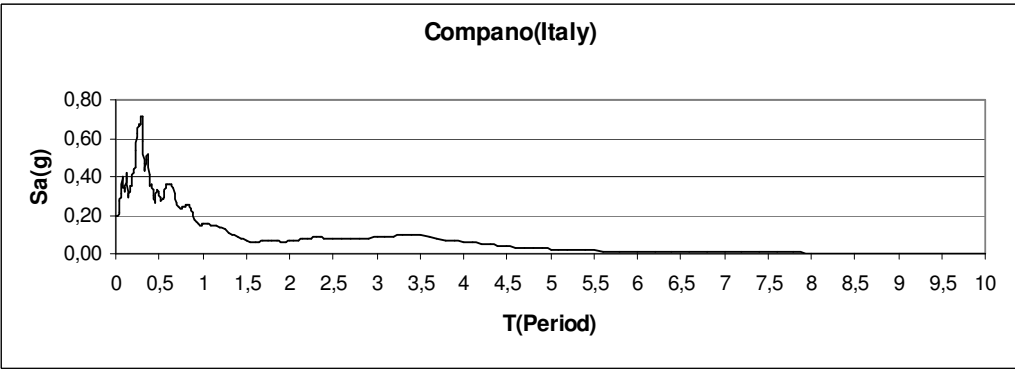
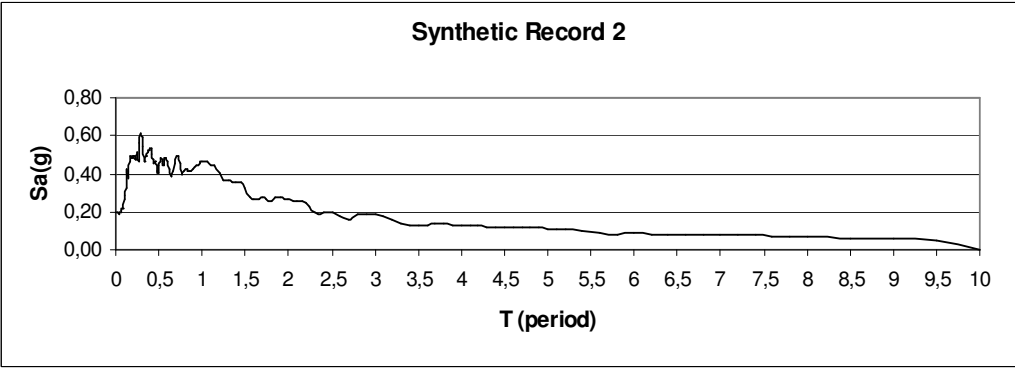
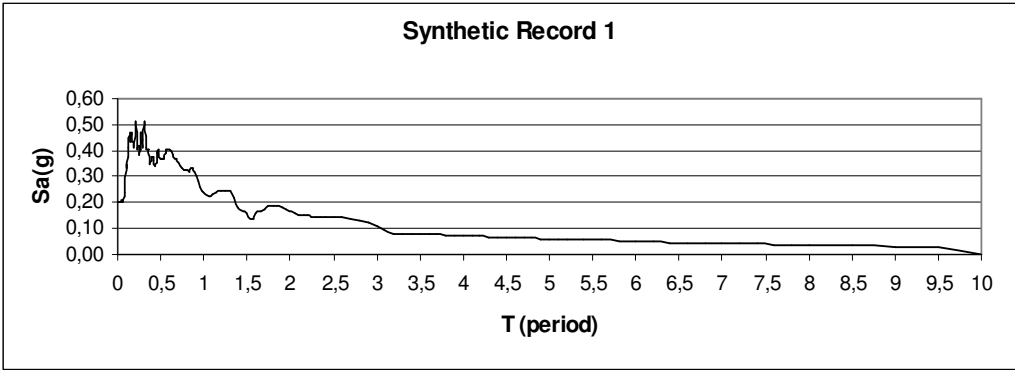


Figure 3.11 Acceleration Response Spectra of Design Ground Motions

## CHAPTER IV

### ANALYSES OF AKKÖPRÜ DAM

#### 4.1 Analyses Procedure for Akköprü Dam

In the previous chapter, necessary data for the analyses of dam and input ground motions were investigated. The largest cross section of the dam is used in the analyses considering that is the most critical one. Earthquake resistant design of Akköprü Dam was made by State Hydraulic Works based on pseudo-static design method. Presently, no doubt that the finite element method has been one of the most powerful tools for evaluation the dynamic response of fill dams under earthquake loading. Therefore, in this study finite element method was chosen for the analyses of Akköprü Dam. The procedure of the analyses was as follows:

- 1) Pseudo-static analysis was performed by using the computer program SLOPE in order to determine the most critical slip surface and yield acceleration  $a_y$  where the factor of safety becomes unity.

- 2) Two synthetic earthquake records produced by using the computer program XS and two actual earthquake records taken from European Strong Motion Database were used in the dynamic analyses of the dam.

- 3) Design ground motions were determined with the estimated ground motion parameters by using the computer program NONLIN.

4) Mean effective stresses needed for the dynamic analyses were determined by performing static analyses with the finite element program SAP 2000.

5) Dynamic material properties of the fill materials were estimated for use in dynamic analysis.

6) Input ground motion at the base of the dam was obtained for use in TELDYN by using the computer program SHAKE91.

7) Dynamic analyses of the dam were performed by using the finite element computer program TELDYN designed for equivalent linear and plane strain conditions. Material properties and strong ground motions determined in previous steps are given as input data to the program in order to obtain the acceleration time histories of the required points at the slip surface.

8) The average acceleration of the sliding block determined in Step 1 was calculated using the nodal time history of accelerations in the sliding block.

9) The permanent displacements of the sliding block were calculated by using the Newmark's permanent displacement method.

#### **4.2 Pseudo-Static Analyses of Akköprü Dam**

The general practice for evaluating the seismic stability of all earth dams had been the use of pseudo-static analyses procedures. In the conventional method of stability analyses the dynamic effects are not considered in the pseudo-static approach. In this study pseudo-static method of analyses has been applied in a non-conventional manner. In this approach, yield acceleration of the critical slip surface is obtained that makes the factor of safety unity. This value is then used for estimating the permanent displacement of the embankment dam by using the Newmark's method of analysis.

In order to determine the most critical slip surface and the corresponding yield acceleration the computer program SLOPE (Borin 1983) was employed by using the Simplified Bishop Method.

For the purpose of evaluating the permanent deformations, the slip surfaces and the corresponding yield accelerations are investigated for both upstream and downstream slopes of the dam embankment. Trial slip surfaces for upstream and downstream slopes are shown in Figure 4.1 and 4.2 respectively. As seen from the figures critical slip surfaces passed through the intersection of crest and filter zone. The factor of safeties of the slip surfaces are shown in Table 4.1 and 4.2. According to this results from Table 4.1 the slip surfaces No 4 is chosen as the most critical one for the upstream part and No 1 for the downstream part. As shown in Table 4.2, yield acceleration for the upstream part is 0.24g and for the downstream part, 0.32g. Consequently, the critical slip surface located is determined as indicated in Figure 4.3.

### **4.3 Static Analyses of Akköprü Dam**

In order to find the dynamic material properties of Akköprü Dam the initial static stresses existing in the embankment were calculated before the earthquake. To determine the stresses the static analyses was performed by using the finite element program SAP2000 for the empty reservoir. The dam body is idealized into 2036 elements and 2099 nodes. Finite element mesh is prepared for both static and dynamic analysis as given in Figure 4.4.

The highest frequency that can be passed through a finite element mesh is a function of the mesh size and the size of the largest elements should be small compared to the wave length of the waves propagating through the model. The minimum vertical element size is calculated by using the following equation:

$$\Delta y = 1 / 8 * V_s / f_{\max} \quad (\text{Pyke, 1984}) \quad (4.1)$$

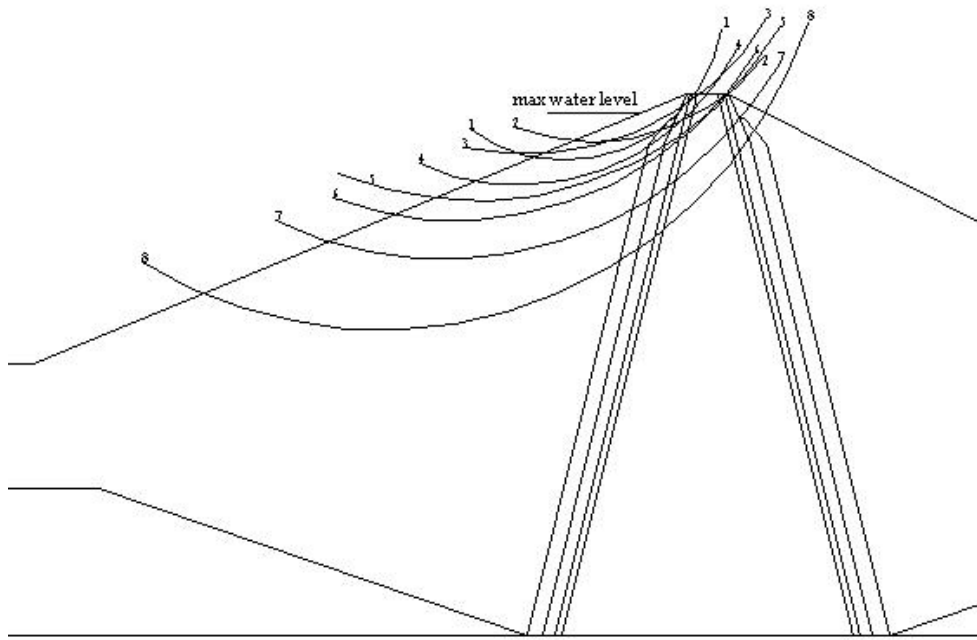


Figure 4.1 Trial Slip Surfaces for Upstream Slope

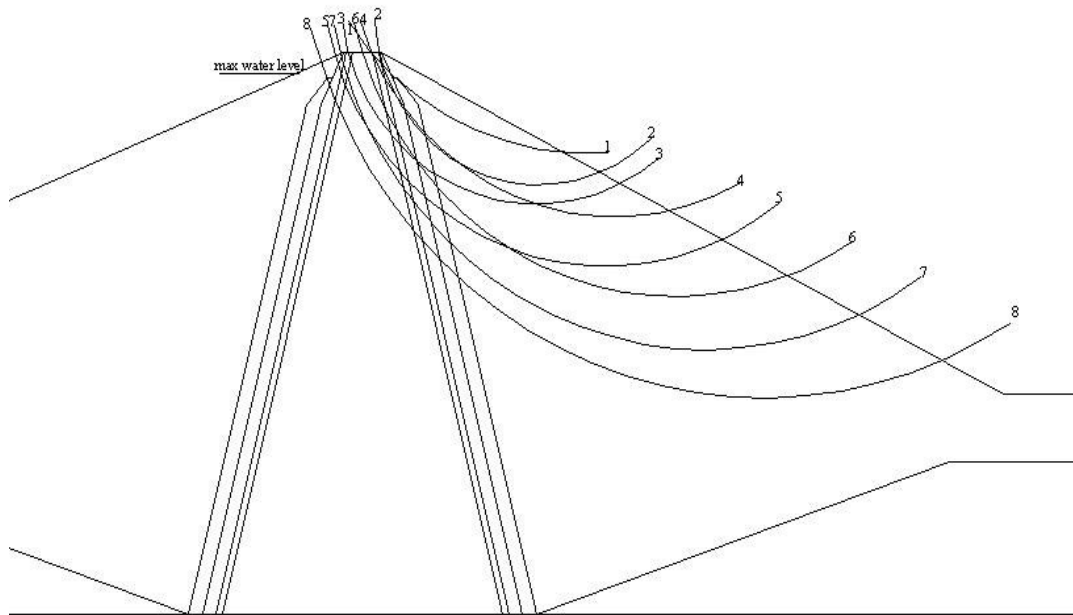


Figure 4.2 Trial Slip Surfaces for Downstream Slope

Table 4.1 Factor of Safeties of Trial Slip Circles for Upstream and Downstream Slopes

<b>UPSTREAM</b>		<b>DOWNSTREAM</b>	
Trial Slip Circles	Static Factor of Safety	Trial Slip Circles	Static Factor of Safety
1	2.396	1	1.902
2	2.409	2	2.473
3	2.416	3	2.394
4	2.393	4	2.215
5	2.399	5	2.241
6	2.441	6	2.248
7	2.647	7	2.346
8	2.624	8	2.338

Table 4.2 Factors of Safeties of the Critical Slip Circles for Upstream and Downstream Slopes

<b>UPSTREAM</b>		<b>DOWNSTREAM</b>	
Slip Circle 4		Slip Circle 1	
Critical Acceleration (g)	Factor of Safety	Critical Acceleration (g)	Factor of Safety
0	2.393	0	1.902
0.10	1.616	0.10	1.518
0.12	1.512	0.14	1.398
0.14	1.419	0.18	1.292
0.16	1.335	0.22	1.199
0.18	1.259	0.26	1.115
0.20	1.190	0.30	1.05
0.22	1.128	0.32	1.003
0.24	1.03		

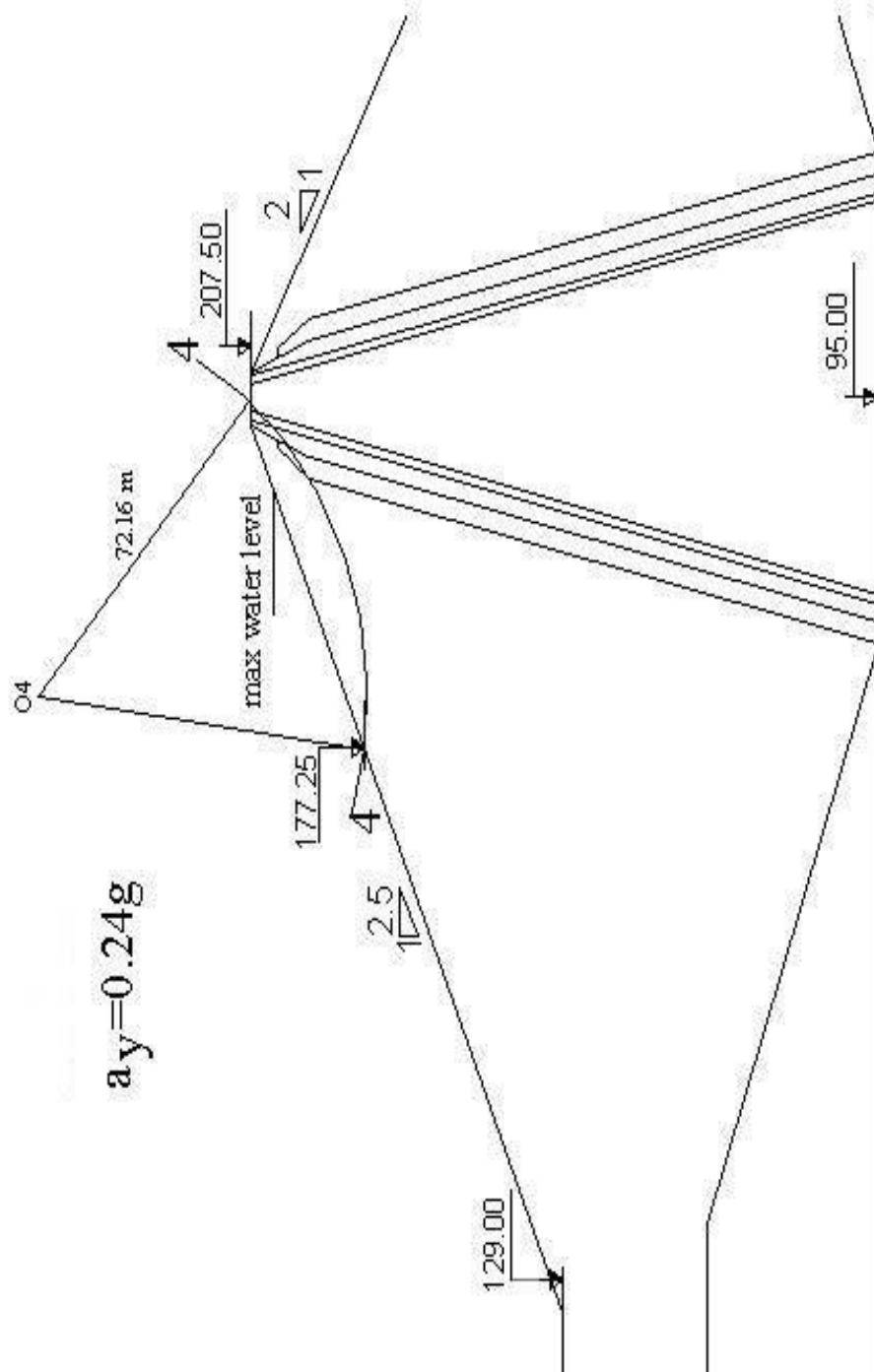


Figure 4.3 Critical Slip Surface Determined in Pseudostatic Analysis

where  $V_s$  is the shear wave velocity in the element and  $f_{\max}$  is highest frequency desired in the analyses. In this study  $f_{\max}$  is taken as 10 herzt and by using the above Equation (4.1) the maximum height of the element is found to be 5 m.

The stress – strain relationship in the case of soils is generally non-linear as mentioned in Chapter II. Hence, the calculation of the stresses become complicated and the problem becomes non-linear in nature. As known, SAP2000 is a non-linear program but it does not consider the material non-linearity. Due to this, iterative procedure is performed to calculate the elastic modulus of the materials by using the Baba's equation explained in Section 2.1. So, the stresses in the elements should be determined in each iteration. As expected the element stresses in the embankment increase from up to the bottom parts and from outer to inner parts. In order to get actual stresses, the dam body is divided into 2036 zones, in other words each element is defined as a different material. The process is repeated until the difference between elastic modulus values from two consecutive iterations is less than 10 % (Sharma, H. D. 1974).

Four iterations were carried out to arrive satisfactory elastic modulus values for the fill materials of the dam when the dam reservoir is empty. In order to determine the effective stresses existing in the dam embankment total stress analysis was performed. It has two steps. In the first step the total stresses within the elements were calculated considering the hydrostatic pressure. Then, in the second step, effective stresses were calculated by subtracting the pore pressures in the saturated zone. For simplicity, based on the phreatic line of the dam the upstream part of the dam with clay core is assumed as saturated and the downstream part of the dam is assumed as wet. The reservoir case, saturated and wet zones are shown in Figure 4.5.

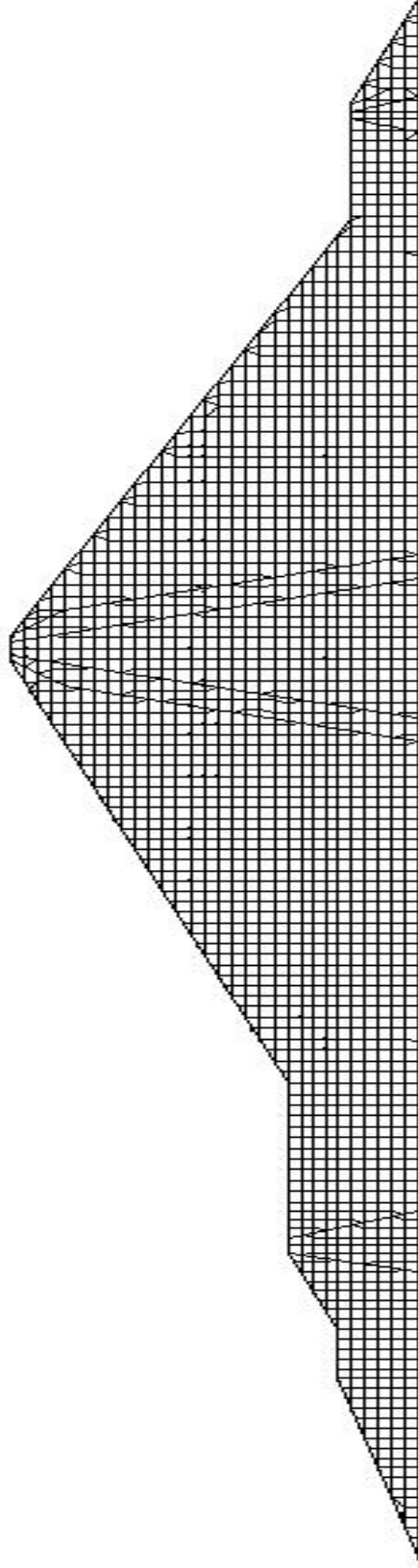
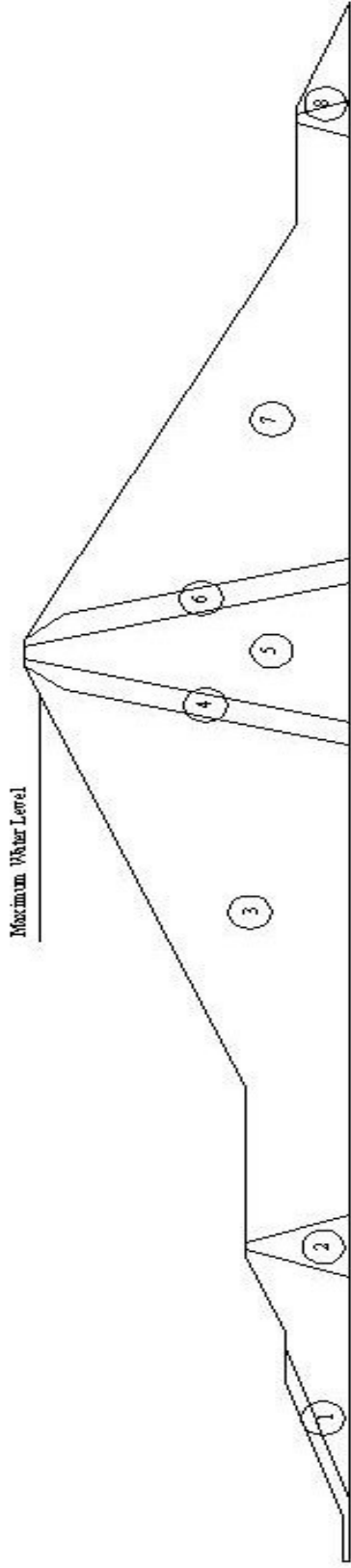


Figure 4.4. Finite Element Mesh of Akköprü Dam Used in Static and Dynamic Analyses



Saturated Unit Weights are used for 1, 2, 3, 4, 5, 6 Zones  
 Wet Unit Weights are used for 7 and 8 Zones

Figure 4.5 Saturated and Wet Zones Assumed for the Total Stress Analysis

#### 4.4 Dynamic Material Properties of Akköprü Dam

The behavior of soils subjected to dynamic loading is governed by the cyclic properties of soil which is mostly known as dynamic soil properties. Hence, determination of the dynamic material properties is a critical task in the evaluation of the response of dams under cycling loading. A wide variety of field and laboratory techniques are available to measure the dynamic material properties at low and high strains as explained in Section 2.4. For many problems dominated by the wave propagation effects, only low levels of strain are induced in the soil, for the others such as those involving the stability of masses of soils large strains are induced in the soil.

The shear moduli are obtained from the ratio  $G/G_{\max}$  where  $G_{\max}$  is the shear moduli at low strain levels. The maximum shear modulus values for clay, filter, and rockfill can be calculated by using the following empirical relationships:

$$G_{\max}(\text{clay}) = 395 \frac{(2.97-e)^2}{(1+e)} \sigma'_m{}^{0.69} \text{ (kg/cm}^2\text{)} \quad (4.2)$$

$$G_{\max}(\text{filter}) = 360 \frac{(2.97-e)^2}{(1+e)} \sigma'_m{}^{0.5} \text{ (kg/cm}^2\text{)} \quad (4.3)$$

$$G_{\max}(\text{rockfill}) = 440^* \frac{(2.97-e)^2}{(1+e)} \sigma'_m{}^{0.55} \text{ (kg/cm}^2\text{)} \quad (4.4)$$

where  $\sigma'_m$  is the mean effective stress  $\sigma'_m = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$  and  $e$  is the void ratio. The equation for filter material is suggested by Richart(1970). Other two equations for clay core and rockfill are suggested by Baba(1982). The void ratio values are taken from Table 4.3.

Table 4.3 Average Void Ratio and Poisson's Ratio of Construction Materials

Material	Average Void Ratio	Poisson's Ratio
Clay (CL)	0.41	0.42
Filter	0.47	0.38
Rockfill	0.42	0.30

The finite element program TELDYN requires curves for the variation of the shear modulus and damping curves with cyclic shear strain ( $10^{-4}$ ,  $10^{-3}$ ,  $10^{-2}$ ,  $10^{-1}$ , 1, 10 in percent). The values for the clay core were obtained from Vucetic and Dobry (1991) and the values for the filter material were obtained from Seed and Idriss (1984) as shown in Figure 4.6. The curves for rockfill material are defined below with the Equation 4.5 and 4.6 respectively and the corresponding curves are illustrated in Figure 4.7.

$$G/G_{\max}(\text{rock}) = \frac{1.56 \times 10^{-3}}{1.56 \times 10^{-3} + \gamma} \quad (4.5)$$

$$\mu(\text{rock}) = 0.23 \frac{\gamma}{1.56 \times 10^{-3} + \gamma} + 0.05 \quad (4.6)$$

where  $\gamma$  is the cyclic shear strain.

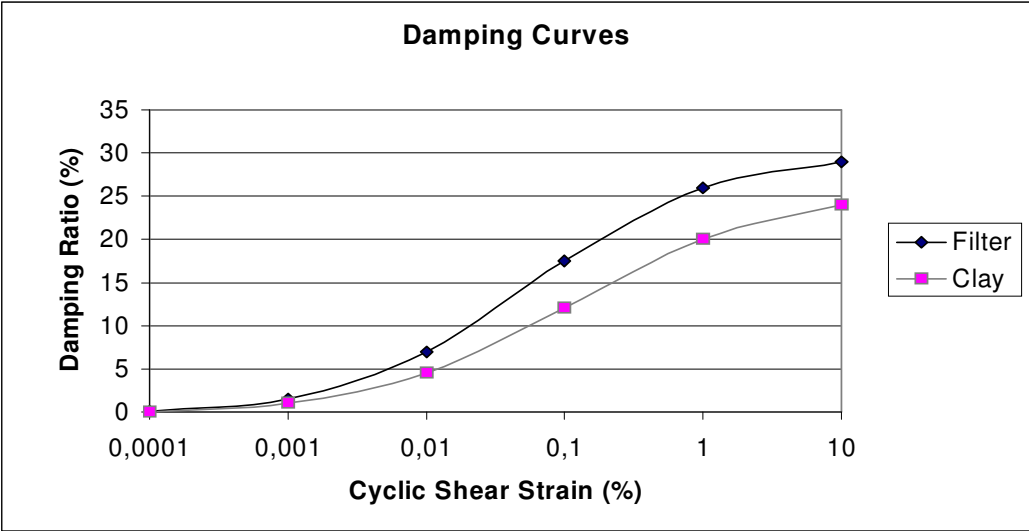
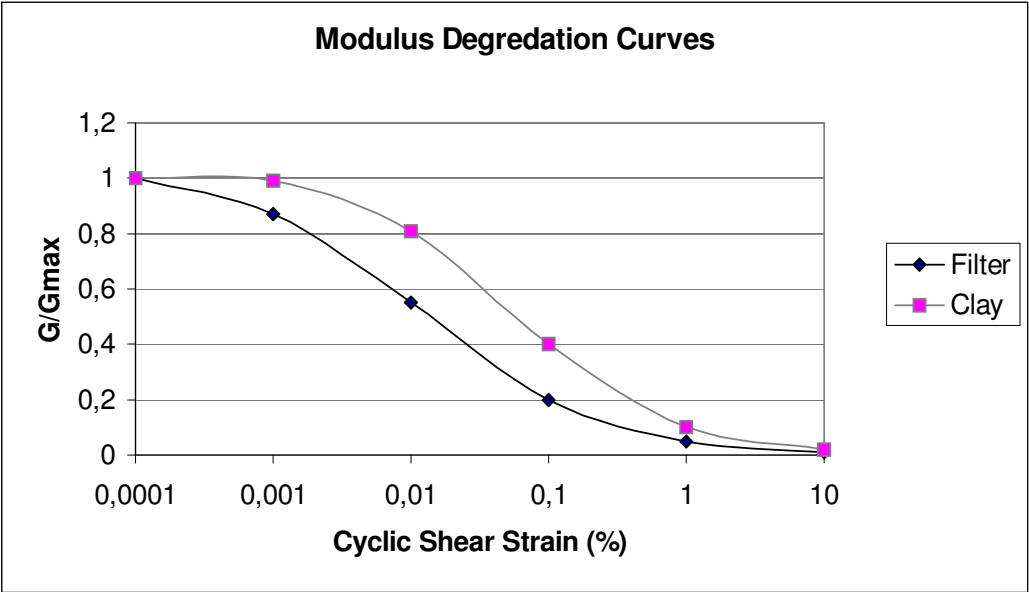


Figure 4.6 Shear Modulus and Damping Curves for Clay (Vucetic & Dobry, 1991) and Filter Material (Seed et al., 1984)

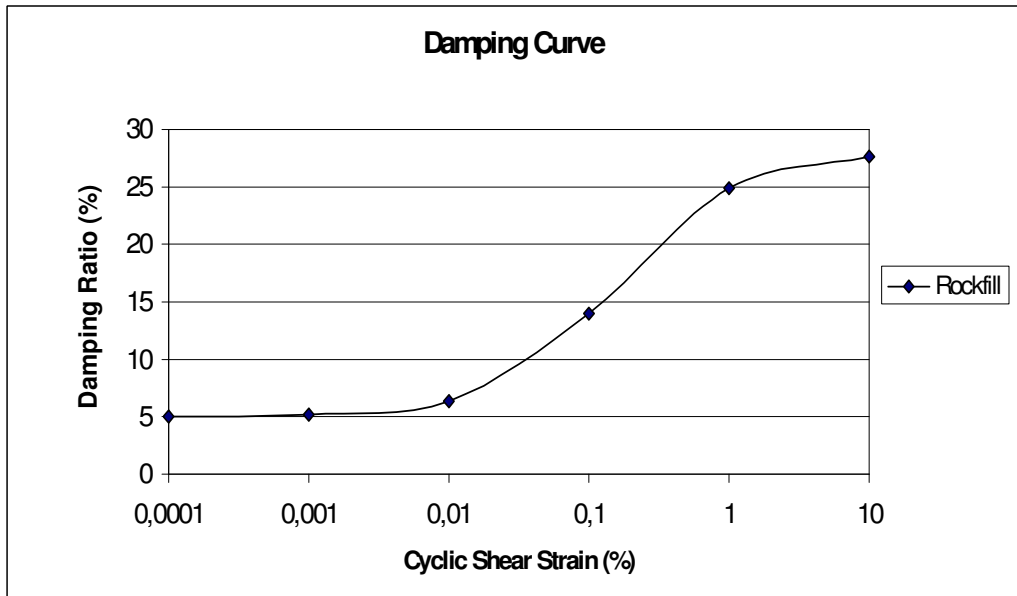
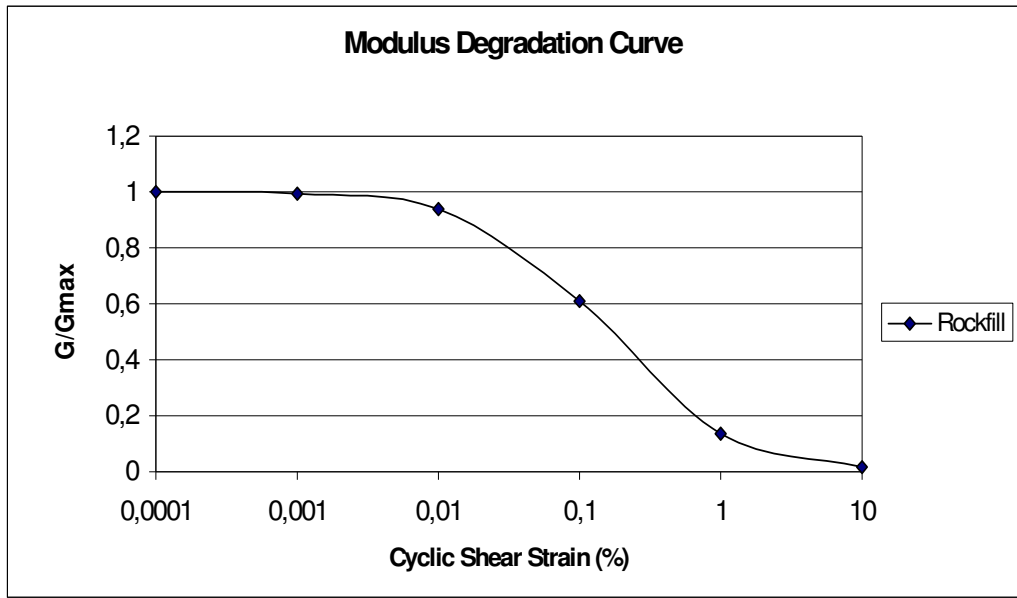


Figure 4.7 Shear Modulus and Damping Curves for Rockfill Material (Baba, 1982)

#### 4.5 Determination of Input Ground Motions to be Used in the Dynamic Analysis

Due to the computational difficulties to handle the alluvium layer in the finite element mesh in program TELDYN, designed ground motions assessed in Chapter 3 are carried from the bedrock outcrop to the base of dam body by using the computer program SHAKE91. In this program, the soil profile is idealized as system of homogeneous, viscoelastic sublayers of infinite horizontal extent and the response of the system is calculated considering vertically propagating shear waves. An equivalent linear procedure is used to account for the nonlinearity of the soil using an iterative procedure to obtain values for modulus and damping that are compatible with the equivalent uniform strain induced in each sublayer. Starting with the maximum shear modulus for each sublayer and a low value of damping, strain compatible properties are obtained in 5 to 8 iterations for most soil profiles. In this study, the soil profile of the alluvium layer divided into 10 sublayers and the shear wave velocities were estimated for each sublayer as shown in Figure 4.8. More detailed information regarding SHAKE91 is given in Appendix B.

Consequently, using SHAKE91, the input ground motions were assessed at the base of the dam body. The calculated response spectra for the input ground motions at the base of the dam are illustrated in Figure 4.9. A comparison of Figure 4.9 with Figure 3.11 shows that the predominant period of the ground motion remains constant and the peak ground acceleration increases and at the base of the dam.

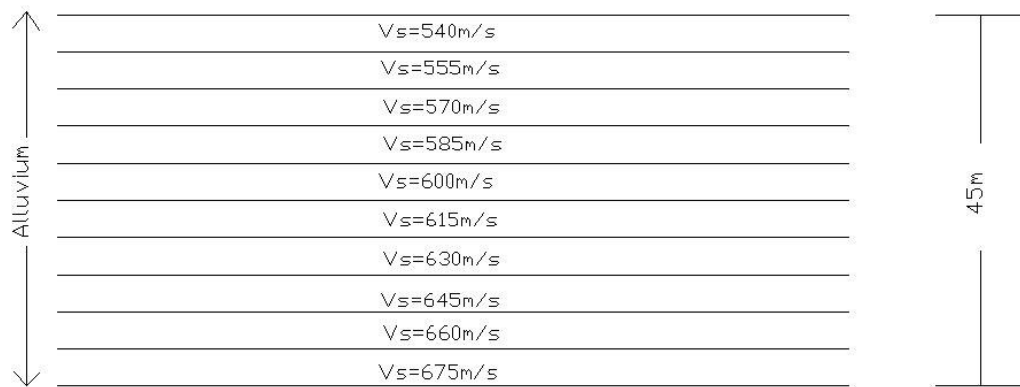


Figure 4.8 Shear Wave Velocity Profile of Alluvium Layer

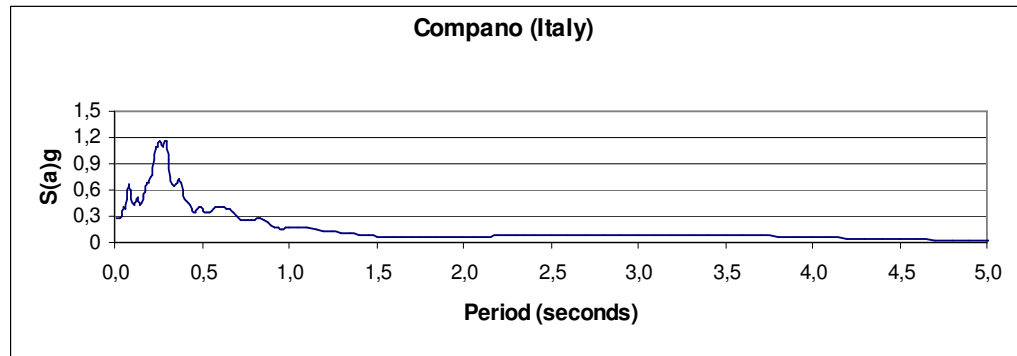
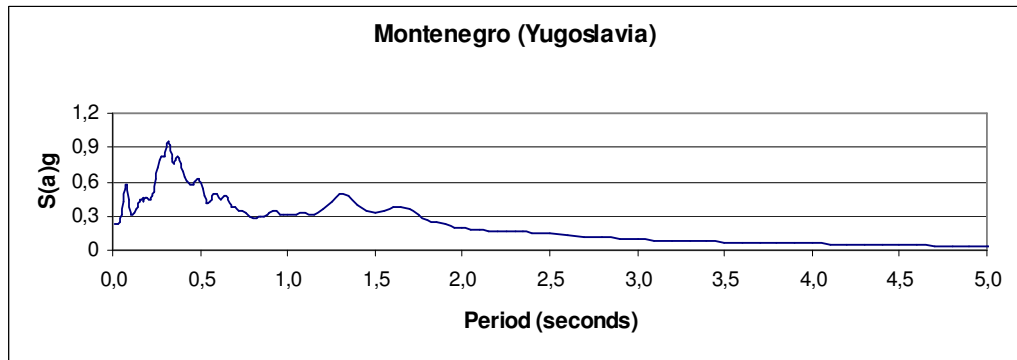
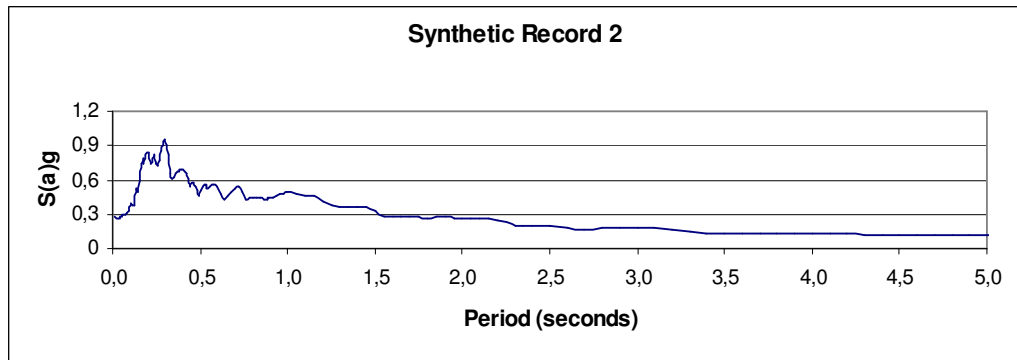
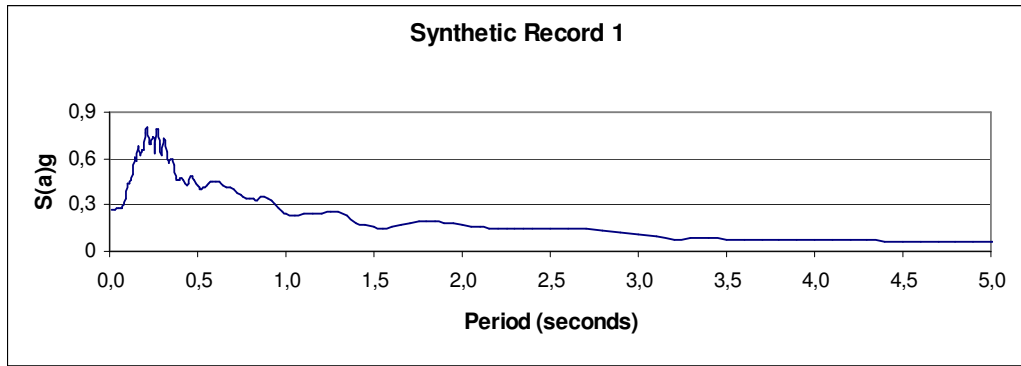


Figure 4.9 Acceleration Response Spectra at the Base of Dam Body

#### **4.6 Dynamic Analysis of Akköprü Dam**

As mentioned before, the plane strain finite element program TELDYN (Pyke, 1984) was used in the analysis of Akköprü Dam under earthquake shaking. This program is specifically designed for the analysis of response of soils to vertically propagating motions caused by ground motions. Moreover, the program employs equivalent linear method which involves an iterative computation in order to obtain shear moduli and damping ratios in each element that are compatible with the average level of shear strain induced by shaking. More detailed information regarding the program TELDYN is given in Appendix C.

In the dynamic analysis of Akköprü Dam, the dam embankment was idealized as a two dimensional mesh which was prepared for static analysis prior to dynamic analyses. The largest cross section of the embankment was subdivided into quadrilateral and triangular elements as shown in Figure 4.3. There are 2099 nodes and 2036 elements in the mesh including three different material zones namely, clay, filter and rockfill. The maximum element size is determined as 5 m by selecting a maximum frequency of 10 hertz for the analysis as explained in Section 4.3.

The soil properties that influence wave propagation, shear modulus, damping ratio, poisson's ratio and density, are used to identify the construction materials of the dam embankment (Kramer 1996). As known, shear modulus and damping ratio values are varying with the cyclic shear strain. In order to represent this nonlinear behavior of the construction material equivalent linear method is applied in the analysis. The equivalencing process of the program involves the following two steps:

- 1) In the first step, shear moduli and damping values are evaluated for each element that are compatible with the cyclic shear strain. Modulus reduction and damping curves are used for the assessment of strain dependent dynamic parameters, which as explained in Section 4.4 before.

2) In the second step of the process, the acceleration time history estimated in Section 4.5 is divided into a number of segments and applied from the base of the dam body. Thus, the shear stress and shear strain time history in each element are obtained in time segments similar to the acceleration time history. The program is then set up to iterate within each segment and to obtain strain compatible values of the shear modulus and damping ratio. It is necessary to specify initial estimates of shear modulus and damping ratio to be used in first iteration of the first segment. The final values computed in the previous segment are used on the first iteration of subsequent segments. The iteration within each segment ceases when acceptable convergence is reached. A maximum difference of about 10 % might be considered to be acceptable. Three to five iterations are normally sufficient to achieve this level of convergence. The average shear strain used in the determination of the shear modulus and damping values is taken to be equal to 0.65 times the maximum shear strain for each segment.

At the end of the analysis fundamental period of the dam, maximum accelerations and shear stresses at nodal points are obtained. Furthermore, it is possible to determine the acceleration, shear stress and shear strain time histories for any node in the model.

#### **4.7 Permanent Displacement Analysis of Akköprü Dam**

In this study, permanent displacement of the potential sliding mass obtained in Section 4.2 was determined by using the Newmark(1965) method. There were 4 different input ground motions applied from the base and each ground motion is scaled to 0.25g, 0.20g and 0.15g respectively. According to these PGA values corresponding displacements were calculated in the analysis.

Two computer programs, ACCHIS(Aksar,2000) and PDISP, were used for the calculation of permanent displacement of the critical slip surface. ACCHIS calculates the time history of average acceleration for a certain slip surface by using

the horizontal nodal accelerations obtained at the end of dynamic finite element analysis of the dam. The output of ACCHIS was stored in a file and entered to the program PDISP as an input file. PDISP calculated the corresponding permanent displacement for the slip surface by double integrating the difference between the applied and yield accelerations. The codes of the programs are given in Appendix D and E respectively.

## CHAPTER V

### RESULTS OF THE DYNAMIC ANALYSES OF AKKÖPRÜ DAM

#### 5.1 Seismic Behavior of Akköprü Dam

The fundamental period of Akköprü dam for small strain vibrations can be estimated from semi-empirical relationship derived by Öner(1984). The equation is given as follows:

$$T_1=0.0408e^{-0.555m} h^{0.75} [1+4/(l/h)^2]^{-0.5} \quad (5.1)$$

where  $h$  is the height of the dam in meters,  $m$  is an arbitrary index for material type( $m=2$  for rockfill dams) and  $l$  is the length of the crest in meters. By using Equation 5.1 with  $h=112.5$  m and  $l=688.70$  m the fundamental period of Akköprü Dam can be calculated as 0.44 sec. This gives an idea about the fundamental period of the dam.

In this study, fundamental period of the dam is determined by using the finite element program TELDYN. The peak ground acceleration of input ground motion was scaled to 0.01g and the fundamental period was found to be 0.66 sec for small strain vibrations. As seen from Figure 5.1 the fundamental period of the dam increases with increasing peak ground acceleration. For 0.20g peak ground acceleration the period increases to 0.78 sec due to nonlinear behavior of the dam during stronger shaking. This is somewhat expected because as the intensity of the ground motion increases, the stiffness of the material decreases and accordingly the natural period of the dam increases.

The maximum acceleration and the acceleration time histories at nodal points along the vertical centerline of the dam body were obtained from the dynamic analyses. The nodal points along the vertical centerline of the dam body are shown in Figure 5.2. The variations of maximum accelerations are plotted for each motion as illustrated in Figures 5.3. It can be seen from these figures that the distribution of the maximum acceleration shows a significant increase towards the top part of the dam body. Furthermore, the acceleration response spectra are determined at four nodal points (3, 235, 739, 2039), which are located at the crest, two-third of dam height, one-third of dam height and base of the embankment as shown in Figure 5.4. As observed in Figure 5.4, predominant period of the earthquake motion remains constant until one-third of dam height, however after this level it increases. Moreover, the amplification of the acceleration increases along the dam height. These findings indicate that the crest of the dam vibrates more severely compared to the bottom part of the dam as mentioned by Okamoto (1973). Therefore, in the design process more attention should be given to the crest region.

## **5.2 Results of Permanent Displacements Analysis**

Permanent displacement analyses of Akköprü Dam are carried out for 4 ground motions by using the Newmark's (1965) procedure. Each ground motion was scaled to 0.25g, 0.20g and 0.15g respectively. Hence, 12 different displacements were calculated in the analyses. As an example, the permanent displacement analysis for the Synthetic Record 1 with 0.20g peak ground acceleration is given in Figure 5.5.

Permanent displacement values for each ground motion were tabulated in Table 5.1. As seen from the results shown in Table 5.1, maximum permanent displacements are calculated as 34.54 cm, 15.90 cm and 4.80 cm for 0.25g, 0.20g and 0.15g peak ground acceleration values, respectively. The tolerable upper limit for the permanent displacement of the dam embankments has been arbitrarily suggested as 100 cm by Hynes-Griffin and Franklin (1984). They stated that it could be tolerated in most dams and in unusual cases, where a dam could not tolerate this displacement

due to small freeboard or vulnerability of critical design features to small displacements, the dam should be reevaluated by other method (Özkan, 1998).

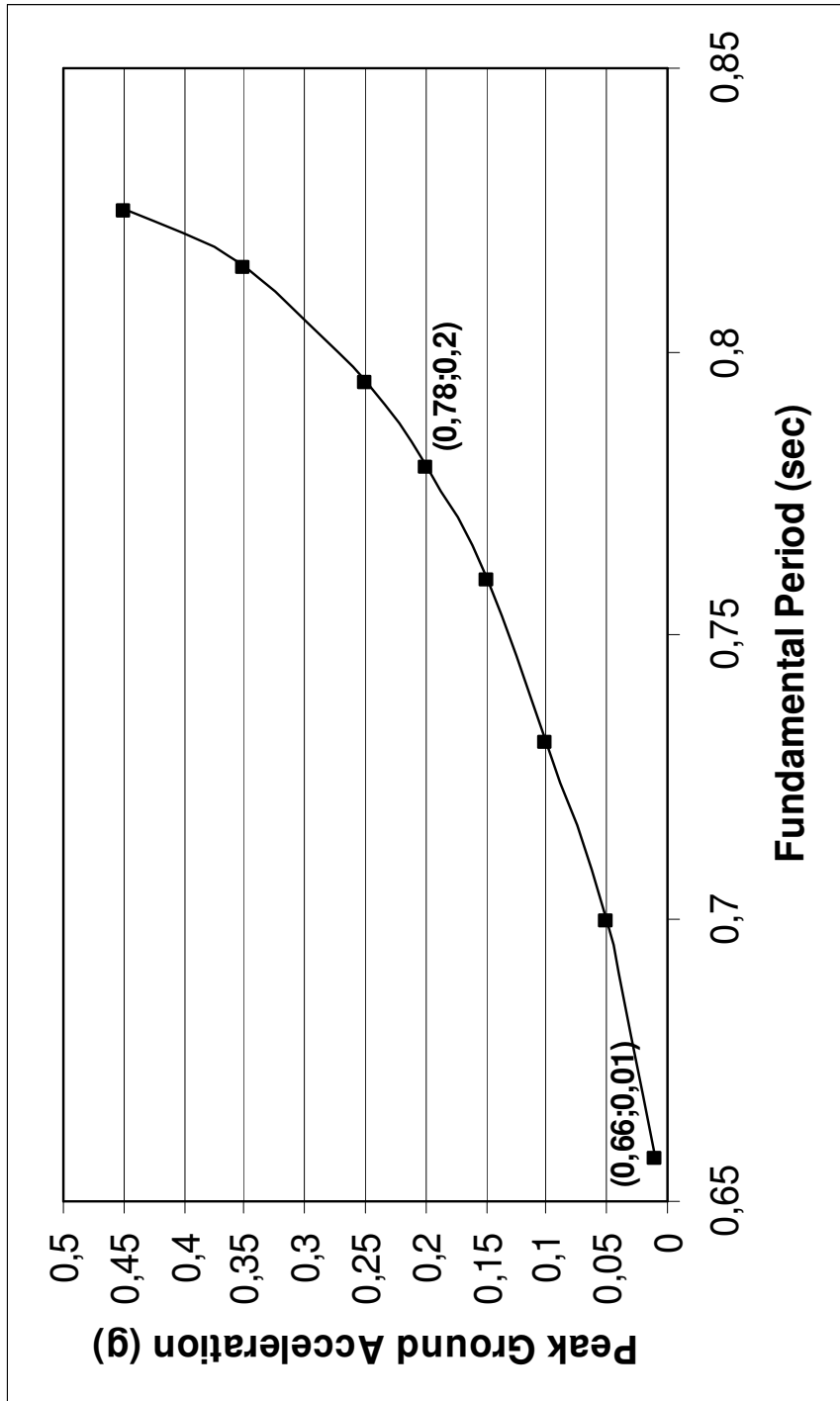


Figure 5.1 Variation of Fundamental Period with the Peak Ground Acceleration

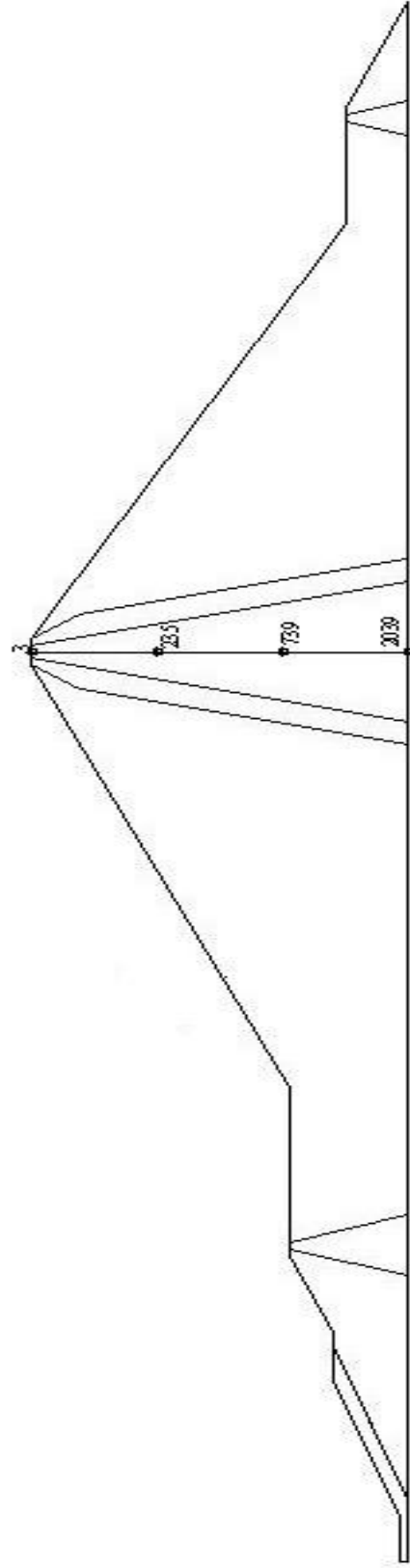


Figure 5.2 Location of Nodal Point Numbers Taken for Acceleration Response Spectra Outputs

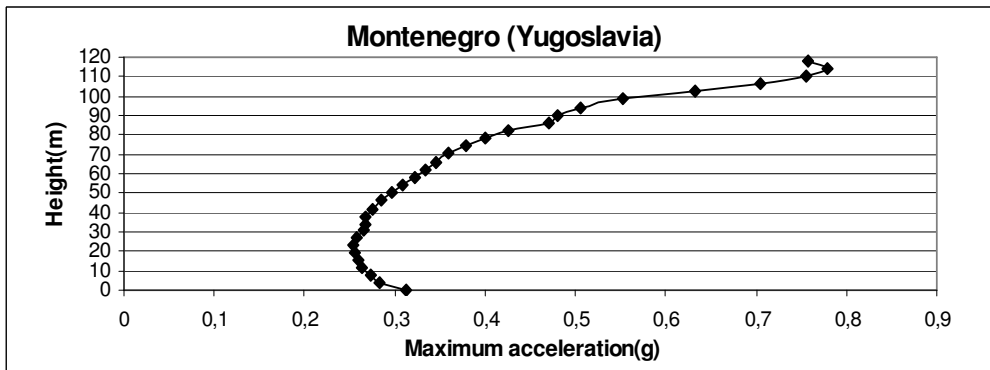
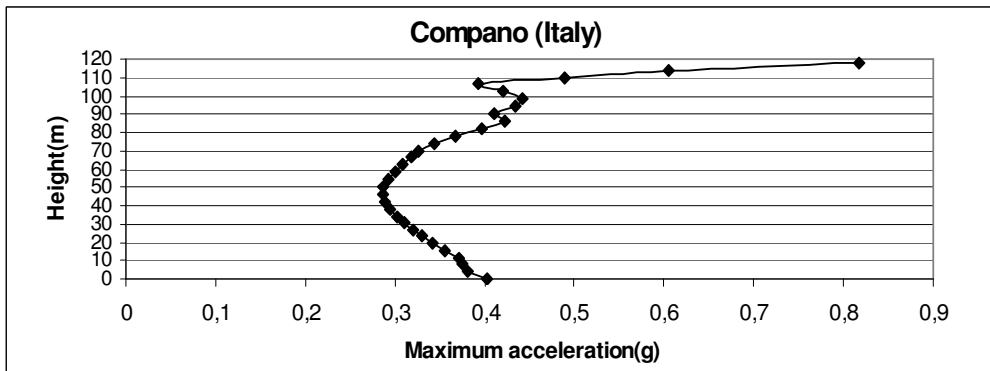
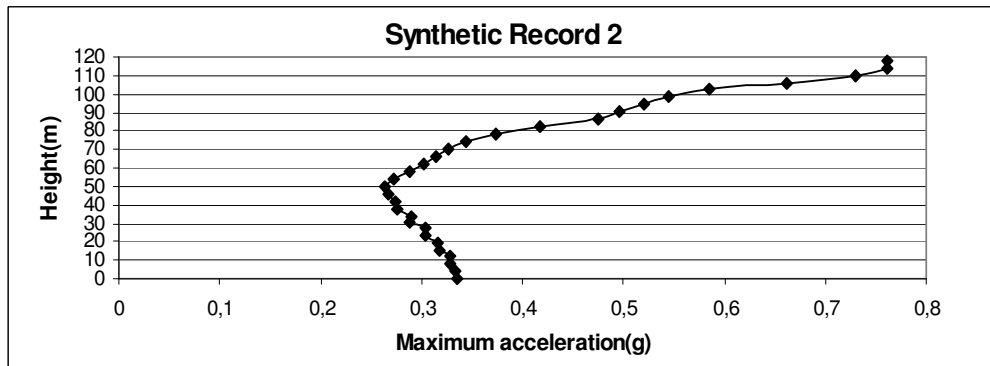
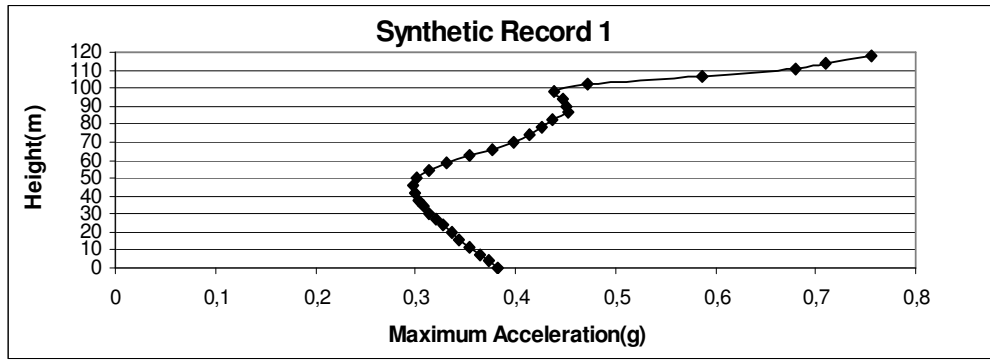


Figure 5.3 Maximum Acceleration Distributions throughout the Dam Axis

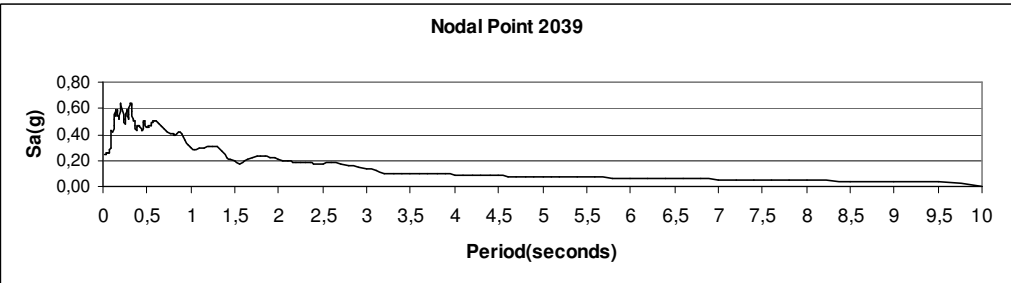
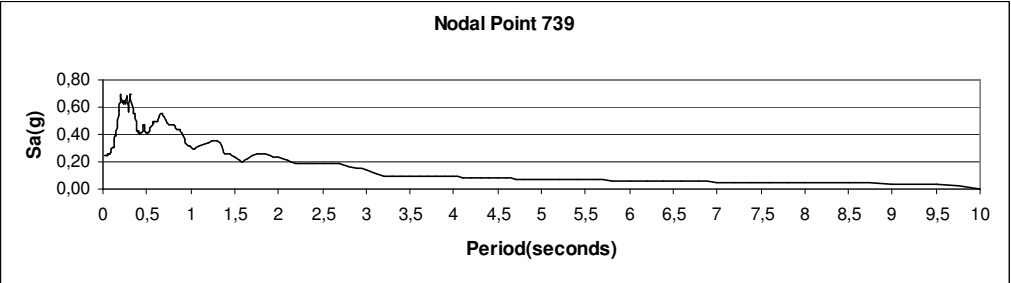
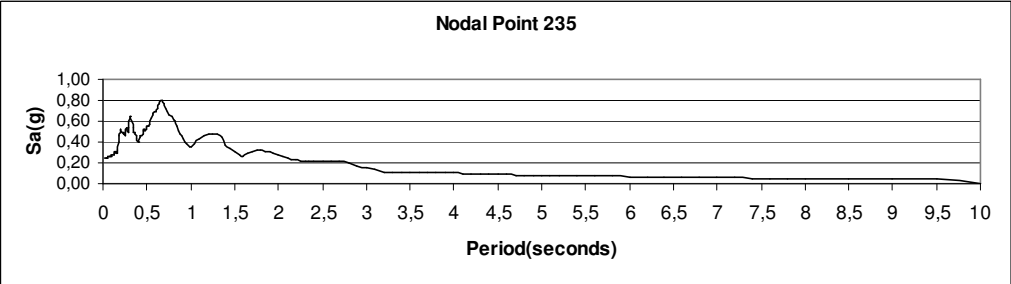
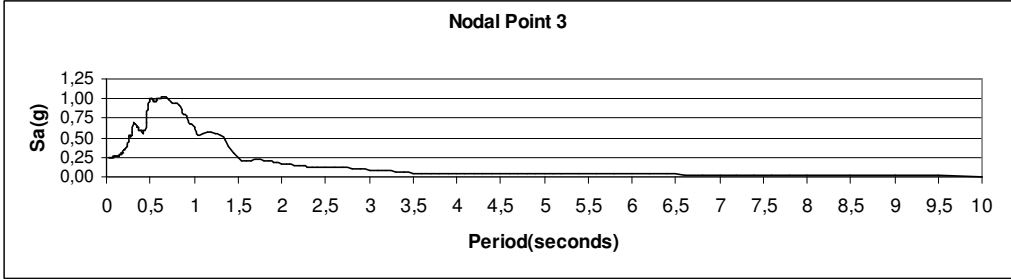


Figure 5.4 Acceleration Response Spectra Along the Centerline of Akköprü Dam for Synthetic Record 1

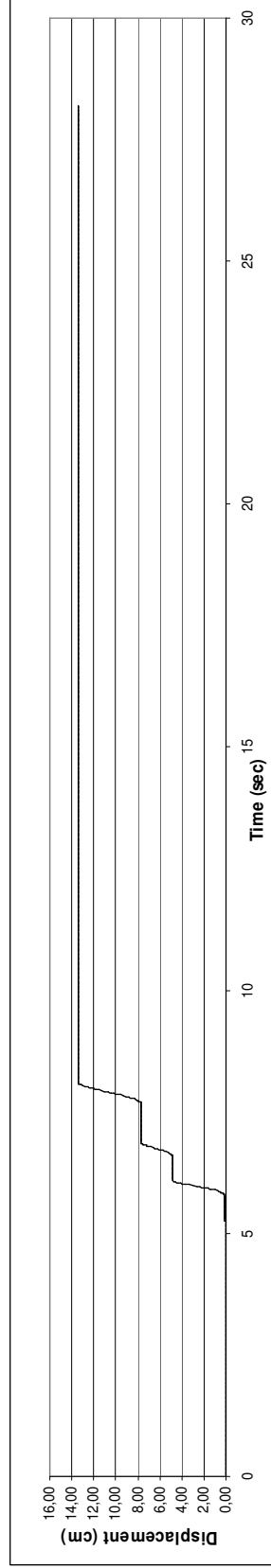
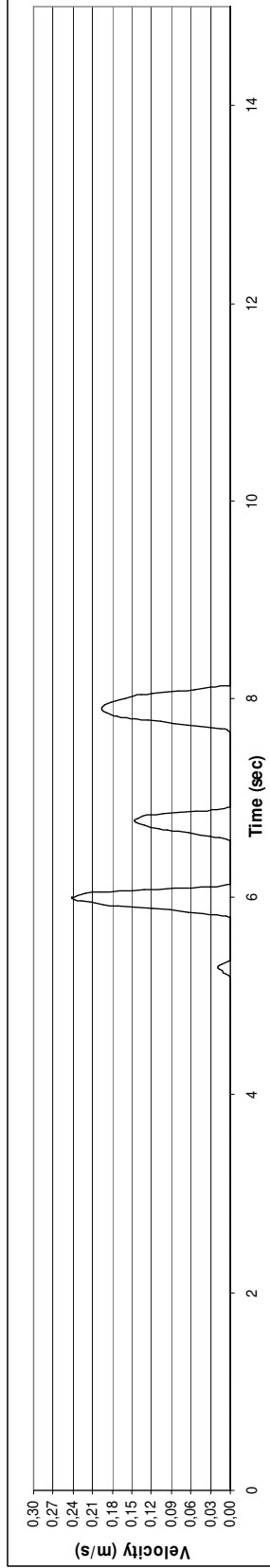
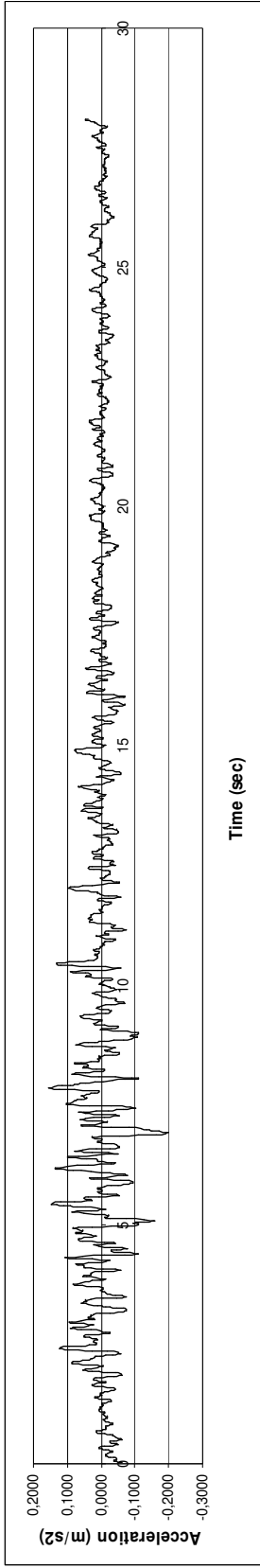


Figure 5.5 Example of Permanent Displacement Analysis (Synthetic Record 1 with 0.02 PGA)

Table 5.1 Results of the Permanent Displacement Analyses

PGA	Permanent Displacements (cm)			
	Synthetic Record 1	Synthetic Record 2	Montenegro (Yugoslavia)	Compano (Italy)
0.25g	31.44	33.42	34.54	29.18
0.20g	13.45	15.10	15.90	12.57
0.15g	3.80	4.50	4.80	3.34

There is not a definite case in the literature regarding the tolerable damage of a rockfill dam occurring due to an earthquake event. Omachi and Kuwano (1994) listed some examples of rockfill dams suffering the strong ground motions. In April 1984, 37 m high rockfill dam suffered some cracks having 30 to 40 cm width, after an earthquake of  $M=6.2$ . In another case, in September 1985 after the earthquake with  $M=8.1$ , La Villita 60 m high rockfill Dam, Mexico, suffered cracks on the crest and the settlement is estimated as 40 cm in the rockfill zone. Furthermore, the 57 m high rockfill Sürgü Dam, Turkey, was subjected to longitudinal cracks on the dam crest and simultaneous settlement on the upstream part of the dam during the 5 May 1986 Malatya Earthquake of  $M=5.8$ . The maximum crack width on the dam crest was 20 cm and upstream part of the crest settled about 15 cm (Özkan,1998).

In this study, for the conservative case, an earthquake having a magnitude of  $M=7$  and a peak ground acceleration 0.20g, the permanent displacement of the sliding block on the upstream part of the dam is calculated as 15.90 cm. Considering the events mentioned above and the suggestion of Hyness-Griffin and Franklin(1984), it can be said that the dam can stand this amount of displacement.

Figure 5.6 depicts the variation of permanent displacements of Akköprü Dam with peak ground accelerations. As can be seen from this figure, the rate increase in the amount of displacement is greater than the increase in the amount of peak ground acceleration.

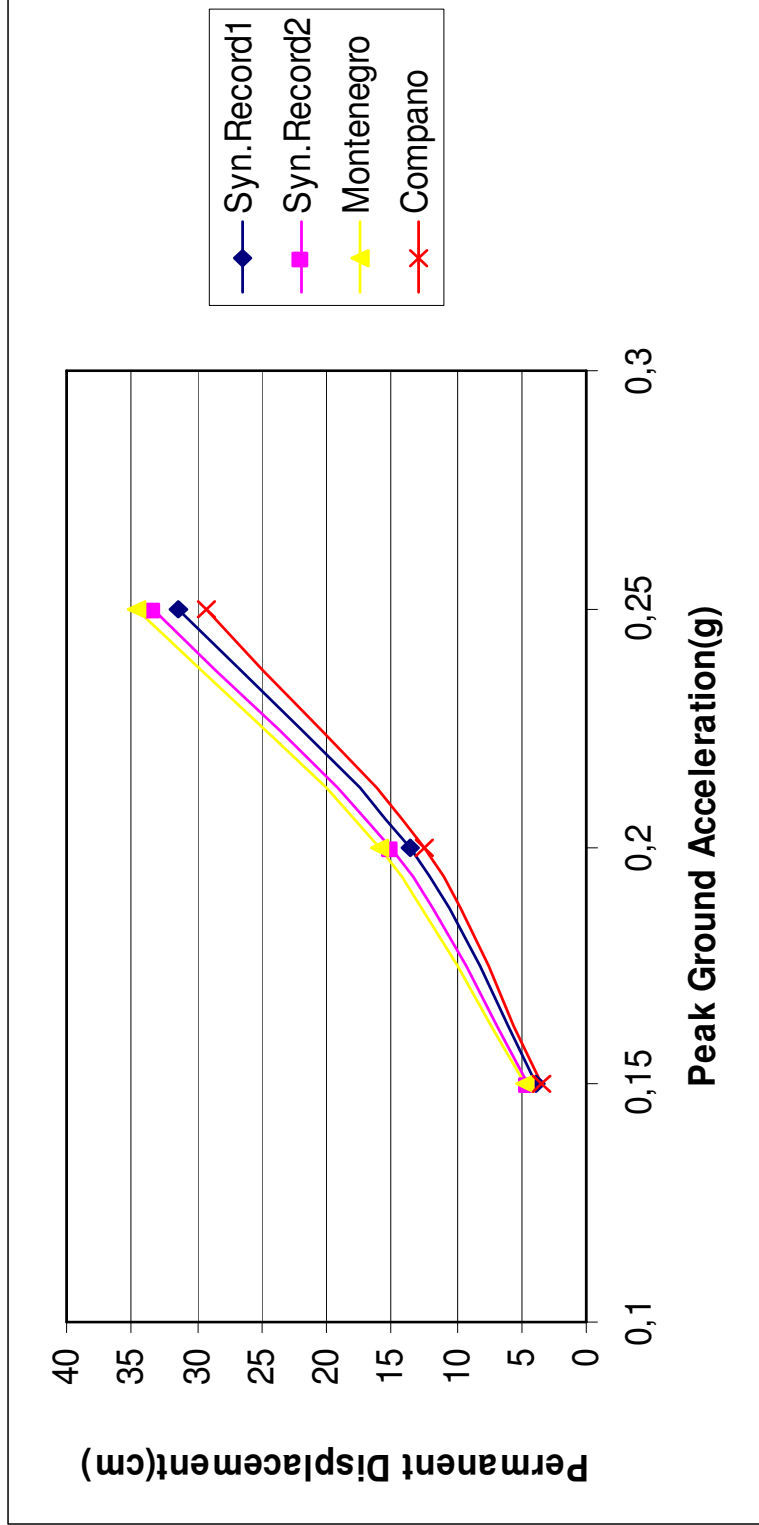


Figure 5.6 Variation of Permanent Displacement with the Peak Ground Acceleration

## CHAPTER VI

### CONCLUSIONS

For many years, the standard method of evaluating the safety of embankment dams against sliding during earthquakes has been the pseudostatic method of analysis. In this method, the dynamic effect of the earthquake is replaced by an inertial static force acting permanently on the potential sliding mass and the stability of this sliding mass is represented by a factor of safety of unity or more. Since pseudostatic is a very simple and straightforward method, many of the dams designed by using this method as in the case of Akköprü Dam, nevertheless this method gives no idea about the anticipated displacements.

Finite element method is one of the most powerful technique to evaluate the stability and behavior of embankment dams under earthquake loading. By considering the nonlinear behavior of the construction material under earthquake loading, acceleration, stress-strain time histories and displacements can be calculated at any point throughout the dam body. The utility of the finite element method has to be considered with respect to the determination of the correct representative values of dynamic soil properties, which vary with cyclic shear strain during an earthquake.

Estimation of the ground motion for the dynamic analyses of the embankment dam requires the determination of proper ground motion parameters namely duration, predominant period and the peak ground acceleration. In this study, different values of peak ground acceleration were used to observe its effects on the behavior of the embankment dam.

According to the results obtained from the dynamic analyses of Akköprü Dam and some other results in the literature, the maximum ground acceleration increases significantly at the upper part of the dam and reaches to its maximum value at the crest. Thus, due to greater acceleration at the upper part of the dam, crest region is more susceptible to seismic excitations. Therefore, the potential sliding surface is generally observed at the upper part of the dam and cracks usually develop near the crest after the earthquake.

Acceleration time histories obtained in the finite element analysis can be used for the calculation of permanent displacement. If the dam embankment or foundation is not subjected to liquefaction, Newmark's analysis can be employed to find the permanent displacement occurring in the potential of sliding mass. Even though there is not a well defined upper limit for the displacements of the potential sliding mass, in most dams, a value of 100cm displacement which is suggested by Hynes- Griffin and Franklin, can be used to evaluate the results of the Newmark's analysis.

The most critical task in the dynamic response analysis of the embankment dams is the assessment of the dynamic properties of the filling material. The properties of the soils under dynamic loading conditions are nonlinear. During the earthquake, the stiffness of the soil and the earthquake energy dissipation within the soil are represented by shear modulus reduction and damping ratio curves respectively. As the intensity of earthquake motion increases, the stiffness of the soil decreases and accordingly damping ratio and natural period of the embankment dam increases.

The amount of maximum permanent displacement of Akköprü Dam is calculated as 15.90 cm for an earthquake having a magnitude of 7 and a peak ground acceleration of 0.20g. Considering the tolerable limit of 100 cm estimated by Hynes-Griffin and Franklin, it is concluded that the Akköprü Dam can stand such amount of displacement, safely.

The change in the amount of permanent displacement is evaluated for different values of peak ground acceleration. Maximum expected permanent displacements of the potential sliding mass are obtained as 34.54 cm, 15.90 cm and 5.80 cm for 0.25g, 0.20g and 0.15g peak ground accelerations, respectively. From the results, it can be observed that the rate increase in the amount of permanent displacement is greater than the increase of the peak ground acceleration. Thus, it can be concluded that estimation of the peak ground acceleration in the design process is very important for determining the displacements obtained by the Newmark's method.

## REFERENCES

- Abdel-Gaffar, Ahmed M., Scott, Rnald F., 1979. "Shear Moduli and Damping Factors of Earth Dam", Journal of Geotechnical Engineering Division, ASCE, Vol. 105, No. GT12, pp. 1405-1426.
- Aksar, Ü.D., 2001. "A Study on Response of Güldürcek Dam During 6, June, 2000 Çankırı Earthquake" M.S. Thesis, Middle East Technical University, Ankara.
- Ambraseys, N., Smit, P. Berardi, R., Rinaldis, D., Cotton, F. and Berge-Thierry C., 2000. "European Strong Motion Database", European Council, Environmental and Climate Research Program.
- Arias, A., 1970. "A Measure of Earthquake Intensity", in R.J. Hansen, ed. Seismic Design for Nuclear Power Plants, MIT Press, Cambridge, Massachusetts, pp. 438-483.
- Bakır, S., 1987. "Behaviour of Embankment Dams Under Earthquake Loading and A Case Study on Sürgü Dam", M.S. Thesis, Middle East Technical University, Ankara.
- Bilgi, V., 1990. "Toprak ve Kaya Dolgu Barajların Projelendirme Kriterleri", The General Directorate of State Hydraulic Works, Ankara.
- Bolt, B. A., 1969. "Duration of Strong Motion", Proceedings of the 4<sup>th</sup> World Conference on Earthquake Engineering, Santiago, Chile, pp. 1304-1315.

- Boore, D.M., 1983. "Stochastic Simulation of High-Frequency Ground Motions based on Seismological Models of the Radiated Spectra", Bulletin of the Seismological Society of America, Vol. 73, pp. 1865-1884
- Boore, D.M., Joyner, W.B., and Fumal, T. E., 1993. "Estimation of Response Spectra and Peak Accelerations from Western North American Earthquakes: An Interim Report", U.S. Geological Survey, OFR 93-509.
- Boore, D.M., Joyner, W.B., and Fumal, T. E., 1997. "Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western north American Earthquakes: A summary of Recent Work", Seismological Research Letters, Vol. 68, No. 1, pp. 128-153.
- Borin, D.L., 1983. "SLOPE, Slope Stability Analysis Program", Geosolve.
- Chang, F. K., and Krinitzsky, E. L., 1977. "Duration, Spectral Content and Predominant Period of Strong Motion Earthquake Records from Western United States", Miscellaneous Paper 5-73-1, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi.
- Charney, F.A., 1995. "NONLIN Version 5.30, Nonlinear Dynamic Time History Analysis of Single Degree of Freedom Systems", Advanced Structural Concepts, Inc.
- Chopra, A.K., 1966. "Earthquake Effects on Dams", Ph. D. dissertation, University of California, Berkeley.
- De Alba, P.A., Seed, H.B., Retamal, E., Seed, R.B., 1988. "Analyses of Dam Failures in 1985 Chilean Earthquake", Journal of Geotechnical Engineering Division, ASCE, Vol. 114, No. 12, pp. 1415 - 1435.

- Demirtaş, R., 2003. Private Conversation, The General Directorate of Disaster Affairs, Ankara.
- Elgamal, A.-W.M., Scott, R.F., Succarieh, M.F., and Yan, L., 1990. “La Villita Dam Response During Five Earthquakes Including Permanent Deformation”, Journal of Geotechnical Engineering Division, ASCE, Vol. 116, No. 10, pp.1443-1463.
- Erdik, M., 1992. “XS, Artificial Earthquake Data Generating Program”, Boğaziçi Üniversitesi Kandilli Rasathanesi.
- Esteva, L., 1970. “Seismic Risk and Seismic Design Decisions” in R.J. Hansen, ed., Seismic Design of Nuclear Power Plants, MIT Press, Cambridge, Massachusetts.
- Hardin, B.O., and Drnevich, V.P., 1972. “Shear Modulus and Damping in Soils: Design Equations and Curves”, Journal of Soil Mechanics and Foundations Divisions, ASCE, Vol.98, No. SM7, pp. 667-692
- Hyness-Griffin, M.E., Franklin, A.G., 1984. “Rationalizing the Seismic Coefficient Method”, Miscellaneous Paper GL-84-13, US Army Corps of Engineers, Washington, DC 20314.
- Idriss, I.M., Lysmer, J., Hwang, R.N., and Seed, H.B., 1973. “QUAD-4: A Computer Program for Evaluating the Seismic Response of Soil Structures by Variable Damping Finite Element Procedures”, Report No. EERC 73-16, Earthquake Engineering Research Center, University of California, Berkeley.
- Idriss, I.M., 1991. “Selection of Earthquake Ground Motions at Rock Sites”, Report Prepared for the Structures Div., Building and Fire Research Lab., NIST.

- Idriss, I.M. and Sun, J.I. 1992. "SHAKE91, A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits", User's Guide, University of California, Davis, 13pp.
- Ishibashi, I., 1992. Discussion to "Effect of Soil Plasticity on Cyclic Response", by M. Vucetic and R. Dobry, Journal of Geotechnical Engineering, ASCE, Vol. 118, No. 5, pp. 830-832.
- Kalkan, E., 2001. ".Attenuation Relationships Based on Strong Motion Data Recorded in Turkey", M.S. Thesis, Middle East Technical University, Ankara, Turkey.
- Kramer, S., 1996. Geotechnical Earthquake Engineering, Prentice Hall, Upper Saddle River, New Jersey.
- Lemos, L.J.L., and Coelho, P.A.L.F. 1991. "Displacements of Slopes under Earthquake Loading", Proceedings, 2<sup>nd</sup> International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. 2, pp. 1051-1056.
- Lysmer, J., Udaka, T., Seed, H.B., and Hwang, R.N., 1974. "LUSH: A Computer Program for Complex Response Analysis of Soil-Structure Systems", Report No. EERC 74-4, Earthquake Engineering Research Center, University of California, Berkeley.
- Lysmer, J., Udaka, T., Tsai, C.F. and Seed, H.B., 1975. "FLUSH: A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems", Report No. EERC 75-30, Earthquake Engineering Research Center, University of California, Berkeley.

- Makdisi, F.I., Seed, H.B., 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations", *Journal of Geotechnical Engineering Division, ASCE*, Vol. 104, GT7, pp. 849-867.
- Marcuson, W. F., 1981. "Moderator's report for session on Earth Dams and Stability of Slopes under Dynamic Loads", *Proceedings, International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri*, Vol. 3, p. 1175.
- McCann, M.W., and Shah, H.C., 1979. "Determining Strong Motion Durations of Earthquakes", *Bulletin of the Seismological Society of America*, Vol. 69, No. 4, pp. 1253-1265.
- Newmark, N.M., 1965. "Effects of Earthquakes on Dams and Embankments", *Geotechnique*, Vol.5, No.2.
- Okamoto, A., 1973. *Introduction to Earthquake Engineering*, University of Tokyo Press.
- Öner, M., Yılmaz, Ç., Erdik, M. and Ertuğrul, M.E., 1979. "Earthquake Analyses for Atatürk Dam Embankment", *METU/EERI Report No. 79-07*.
- Özkan, M.Y., 1998. "Review of Considerations on Seismic Safety of Embankment and Earth and Rock-Fill Dams", *Soil Dynamics and Earthquake Engineering*, Vol. 17, pp. 439-458.
- Priscu, R., Popovici, A., Stermatiu, D., Stere, C., 1985. *Earthquake Engineering for Large Dams*, Editura Acedemici and John Wiley & Sons, Ltd., Romania.
- Pyke, R., 1984. "TELDYN, A Computer Program for Plane Strain, Dynamic Finite Element Analyses of Soils and Simple Structures, Users Manual", TAGA, Engineering Software Services.

- Richart, F. E., Hall, J.R., and Woods, R.D., 1970. *Vibration of Soils and Foundations*, Prentice Hall, New Jersey.
- Seed H.B., and Idriss, I.M., 1969. "Influence of Soil Conditions on Ground Motions During Earthquakes", *Journal of Soil Mechanics and Foundations Division*, ASCE, Vol. 95, No. SM1, pp. 99-137
- Seed H.B., Lee, K.L., and Idriss, I.M. 1969. "Analysis of Sheffield Dam Failure", *Journal of Soil Mechanics and Foundations Divisions*, ASCE, Vol.95, No. SM6, pp. 1453-1490.
- Seed H.B., and Idriss, I.M., 1970. "Soil Moduli and Damping Factors for Dynamic Response Analysis", Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley, California.
- Seed, H.B., Makdisi, F.I., and DeAlba, P., 1978. "Performance of Earth Dams During Earthquakes", *Journal of Geotechnical Engineering Division*, ASCE, Vol. 104, No GT7, Proceedings Paper 13870.
- Seed, H.B., 1979. "Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams" *Geotechnique*, Vol.29, No.3, pp. 215-263.
- Seed, H.B., Idriss, I.M., Lee, K.L., Makdisi, F.I., 1975. "Dynamic Analysis of the Slide in the Lower San Fernando Dam During the Earthquake of February 9,1971", *Journal of Geotechnical Engineering Division*, ASCE, Vol. 114, No. 12, pp. 1415 - 1435.
- Seed, H.B., Wong, R.T., Idriss, I.M., and Tokimatsu, K., 1984. "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils", *Journal of Geotechnical Engineering*, ASCE, Vol.112, No.11, pp. 1016-1032

- Şekercioğlu, E., Özgüler, E., 1999. “Geotechnical Investigations on Leakage Problem of Akköprü Dam, Turkey”, International Commission on Large Dams, Antalya.
- Singh, B., Varshney, R.S., Verghese, B.G., Goel, M.C. and Singh R.P., 1995. Engineering for Embankment Dams, A.A. Balkema, Rotterdam, Brookfield.
- Sherard, J.L., 1967. “Earthquake Considerations in Earth Dam Design”, Journal of the Soil Mechanics and Foundations Divisions, ASCE, Vol. 93, No. SM4, pp. 377-401,
- State Hydraulic Works, 1982. “Dalaman, Akköprü, Gökyar, Narlı, Sandalcık ve Beşkonak Baraj Yerleri Deprem Risk Analiz Raporu”, Ministry of Power and Natural Resources, Ankara, Turkey.
- Terzaghi, K., 1950. Mechanism of Landslides, The Geological Survey of America, Engineering Geology (Berkey) Volume.
- Tika-Vassilikos, T.E., Sarma, S.K., and Ambraseys, N., 1993. “Seismic Displacements on Shear Surfaces in Cohesive Soils”, Earthquake Engineering and Structural Dynamics, Vol. 22, pp. 709-721.
- Trifunac, M.D., and Brady, A.G., 1975b. “A Study of the Duration of Strong Earthquake Ground Motion”, Bulletin of the Seismological Society of America, Vol. 61, pp. 581-626.
- Tunçer, M., 1995. “Earthquake Behaviour and Permanent Displacement Analysis of Sürgü Dam: A Reevaluation”, M.S. Thesis, Middle East Technical University, Ankara, Turkey.

Vucetic, M., and Dobry, R., 1991. "Effect of Soil Plasticity on Cyclic Response",  
Journal of Geotechnical ;Engineering, ASCE, Vol.117, No. 1, pp. 89-107.

Wilson, E.L., and Habibullah, A., 1992. SAP90-A Series of Computer Programs for  
the Finite Element Analysis of Structures-Structural Analyses Users  
Manual", Computers and Structures, Inc., Berkeley, California.

## **APPENDIX A**

### **EARTHQUAKES WITH $M \geq 4$ OCCURRED DURING THE PERIOD 1900-2002**

Here the epicenter map of the earthquakes with magnitudes greater than 4.0 and a list of the earthquakes with their occurring dates, epicenter locations, focal depths and magnitudes are given.

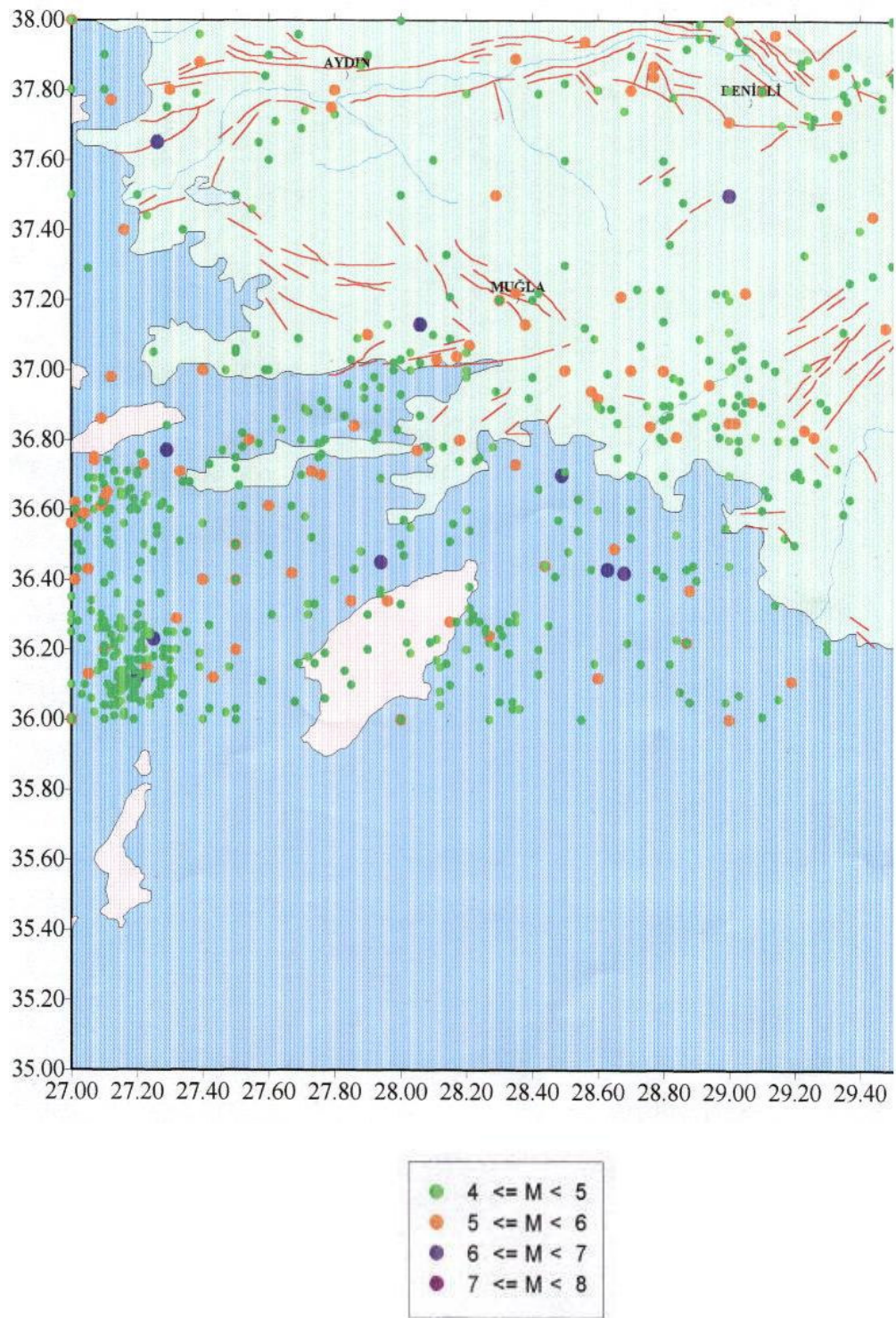


Figure A.1 Earthquakes with  $M \geq 4$  Occur in Southwestern Anatolia During the Period 1900-2002

Table A.1 List of Earthquakes with Magnitudes Greater Than  $M \geq 4$  Occured in the Region Bounded by 35.00 - 38.00 N 27.00 - 30.00 E During the Period 1900 - 2002

<b>LIST OF EARTHQUAKES WITH MAGNITUDES GREATER THAN <math>M \geq 4.0</math>                      OCCURED IN THE REGION                      BOUNDED BY 35.00 - 38.00 N 27.00 – 30.00 E DURING THE PERIOD                      1900 – 2002</b>				
DATE	LATITUDE	LONGITUDE	DEPTH	MB
09.20.00	37,8	29,1	0	5,2
02.23.01	37,9	27,9	15	4,8
06.21.02	37,8	29,1	15	4,6
01.01.04	37,8	29,1	20	5,0
08.11.04	37,7	26,9	6	6,2
08.15.04	37,4	26,6	5	4,7
03.08.08	37,8	27,8	15	5,2
08.07.10	37,8	28,7	30	5,4
02.08.14	37,8	27	7	4,6
06.04.17	37	26	0	4,9
06.13.17	36	28	15	4,8
03.17.18	36	28	15	5,0
03.17.18	36	28	15	5,6
07.16.18	36,08	26,99	70	6,1
09.23.18	36	28	150	5,7
11.13.18	37,8	27,3	35	5,3
11.25.18	36,4	27,5	10	5,1
11.28.18	36,4	27,5	12	5,0
11.29.18	36,4	27,5	15	4,6
04.05.19	36,6	26,7	15	5,3
07.18.19	36	28	15	5,3
07.20.19	36	28	15	5,0
08.24.19	36,84	27,86	10	5,5
05.01.20	37	28,7	30	5,2
07.02.20	37,5	29	15	4,7
07.04.20	37,5	29	15	5,2
07.04.20	37,5	29	15	5,3
09.28.20	37,89	28,35	10	5,7
01.26.21	36	28	15	4,6
01.27.21	36	28	15	5,5
05.22.21	37	28,7	32	5,2
06.03.22	36,49	28,65	30	5,1
08.17.22	36	28	15	5,2
11.20.22	37,5	29	28	5,1

12.06.22	37,5	29	15	5,3
11.27.23	36	26	15	5,0
03.17.25	37,2	26,2	15	5,2
07.29.25	37,5	27,5	15	4,7
02.08.26	37,1	27,9	7	5,3
02.18.26	36,7	27,2	5	4,9
03.16.26	37,5	29	15	6,2
03.18.26	36	29	12	4,9
03.19.26	36	29	15	4,7
03.21.26	36	29	15	5,2
03.28.26	36	29	15	5,0
03.31.26	36	29	15	5,1
04.01.26	36	29	14	5,2
04.20.26	36	29	15	5,0
05.02.26	36	26	15	4,7
05.22.26	36	26	15	5,0
06.06.26	36	26	15	4,7
06.26.26	36,75	26,98	10	7,4
06.26.26	36,5	27,5	15	5,4
06.27.26	36,7	27,76	10	5,4
07.05.26	36,52	26,69	140	6,2
12.07.26	36	27	15	5,1
12.08.26	36	27	15	5,2
12.12.26	36	27	15	4,7
05.20.27	37,5	27,5	15	4,7
04.10.28	37,4	26,1	15	5,0
03.27.29	36,63	26,68	106	5,2
03.27.29	36,4	26,54	80	6,2
11.11.29	36,68	26,21	15	5,7
08.22.30	36,2	27,5	120	5,6
08.09.32	36,71	27,73	110	5,3
12.07.32	36,71	27,33	60	5,2
04.23.33	36,77	27,29	30	6,3
05.15.33	36,35	26,8	10	5,1
07.24.33	37,8	29	15	4,7
08.17.33	37,36	28,82	60	4,7
04.28.36	36,43	27,05	160	5,6
05.01.38	37,2	28,4	15	4,7
03.13.39	36	29	15	5,2
07.24.39	37,2	28,3	15	5,0
05.23.41	37,07	28,21	40	6,0
05.23.41	37,2	28,4	10	4,7
05.23.41	37,13	28,38	40	5,4
05.23.41	37,22	28,35	48	5,3
06.07.41	37,2	28,3	5	4,7
06.23.41	37,2	28,3	7	5,1

07.13.41	37,66	26,09	60	5,9
09.21.41	37,5	28,29	70	5,3
10.14.41	37,2	28,4	15	4,8
12.13.41	37,13	28,06	30	6,4
06.21.42	36,12	27,2	40	6,3
01.11.43	37	27,4	19	5,3
10.16.43	36,45	27,94	120	6,5
11.15.43	36,81	28,84	83	5,3
11.20.43	36,44	28,44	40	5,7
01.05.44	36,4	27,4	150	5,7
01.05.44	36,42	27,67	70	5,7
01.08.44	36,8	27,7	15	4,7
03.25.44	37,8	26,8	8	4,7
05.27.44	36,23	27,25	40	6,2
12.21.45	37,9	29	4	4,9
04.12.46	36,24	26,69	80	5,6
01.13.51	36	26	0	4,8
11.05.51	36	29	0	5,3
04.09.52	37,8	26,9	10	4,7
06.09.52	36,83	27,64	20	5,0
09.15.52	37,6	27,6	32	4,8
10.22.52	36,86	27,09	107	5,3
01.12.53	36	28	0	5,0
03.20.53	36,48	28,51	160	4,6
04.17.53	37,6	27,6	16	4,7
06.13.53	36	26,92	40	4,7
12.05.53	36,5	26,8	15	4,8
01.02.54	36,98	27,12	140	5,4
05.01.54	37,8	27	0	5,0
05.01.54	37,88	27,39	30	5,2
05.01.54	37,77	27,12	40	5,3
05.01.54	37,75	27,29	60	5,0
06.17.54	36,52	27,73	60	4,7
08.20.54	37,5	27	15	4,7
09.04.54	36,63	27,1	160	5,7
05.31.55	36	26	15	4,9
07.16.55	37,65	27,26	40	6,6
07.18.55	37,9	27,1	33	4,5
08.28.55	37,4	27,16	20	5,3
11.10.55	37,44	27,23	10	4,7
11.11.55	37,54	26,97	10	4,9
11.11.55	37,5	27,2	18	4,5
05.05.56	36,99	28,63	40	4,9
05.05.56	37	28,2	0	4,7
06.27.56	37,8	27,1	26	4,3
07.08.56	36,9	26	15	4,7

07.09.56	36,68	26,05	40	4,9
07.09.56	36,46	26	60	4,8
07.09.56	36,54	26,32	40	4,7
07.09.56	36,72	26,12	10	4,7
07.09.56	36,45	26,09	60	5,0
07.10.56	36,69	26,06	20	4,4
07.10.56	36,72	26,52	10	4,8
07.10.56	36,77	26,23	30	5,3
02.05.57	36,37	28,88	60	5,3
02.09.57	36,75	26,44	40	5,2
04.24.57	36,43	28,63	80	6,6
04.25.57	36,42	28,68	80	6,9
04.25.57	36,16	28,73	80	4,4
04.25.57	36,44	28,89	30	4,4
04.25.57	36,12	28,6	10	5,2
04.26.57	36,22	28,87	50	5,9
04.26.57	36,41	28,8	10	4,9
03.04.58	36,34	27,85	120	5,3
04.24.58	36,76	26,55	10	5,0
05.09.58	36,61	27,6	67	5,3
05.23.58	36,5	28	13	4,4
05.27.58	36,8	26,76	160	5,2
06.30.58	36,29	27,32	100	5,4
09.04.58	36,56	26,72	40	5,2
09.04.58	36,4	27,01	140	5,1
09.04.58	37,9	27,6	20	4,6
12.09.58	36,56	28,16	50	4,7
01.06.59	36,85	29,16	20	4,7
01.06.59	36,66	29,11	30	5,0
01.06.59	36,8	29,2	0	4,6
01.07.59	36,71	29,21	40	5,0
01.09.59	36,55	28,99	40	4,5
01.11.59	36,64	29,12	50	4,9
01.11.59	36,6	29,1	0	4,7
01.14.59	36,05	28,99	60	4,5
01.20.59	36,7	28,7	0	5,0
01.26.59	36,83	29,23	70	5,5
01.26.59	36,7	29	0	4,7
04.08.59	36,57	26,8	160	4,9
04.25.59	36,94	28,58	30	5,9
04.25.59	36,92	28,6	40	5,4
04.30.59	36,22	26,68	100	5,0
05.20.59	36,74	26,67	70	5,0
06.09.59	36,81	29,08	20	4,9
06.10.59	36,8	29,15	50	4,7
07.12.59	36,03	26,28	80	5,4

11.15.59	36,8	26	80	5,0
12.08.59	36,91	29,07	70	5,2
01.09.60	36,85	29	70	5,1
01.17.60	37,02	29,11	40	4,5
01.19.60	36,9	28,8	0	4,5
01.19.60	36,7	29	60	4,7
01.20.60	36,7	28,8	0	4,6
01.26.60	36,85	29,02	60	5,3
01.26.60	36,89	28,61	30	4,8
01.26.60	36,93	29,03	50	4,6
01.26.60	36,84	28,96	10	4,6
01.27.60	36,6	28,7	5	4,6
01.28.60	36,6	29,1	8	4,4
01.29.60	36,88	29,04	60	4,7
04.08.60	36,2	28,2	0	4,5
04.10.60	37,73	27,8	40	5,0
04.12.60	37,69	27,7	40	4,8
05.02.60	36,95	26,83	20	4,8
05.15.60	37,2	26,7	0	4,7
11.03.60	36,1	26,1	15	4,7
02.23.61	36,75	27,07	80	5,2
02.23.61	36,6	27,2	15	4,9
02.23.61	36,73	27,22	40	5,1
02.27.61	36,56	27	70	5,2
02.27.61	36,59	27,02	60	5,2
03.13.61	36,21	26,43	10	5,0
05.23.61	36,7	28,49	70	6,5
05.25.61	36,72	26,66	60	5,0
06.21.61	37,87	28,77	60	5,5
08.16.61	36	26	0	5,0
04.16.62	36,15	27,23	140	5,5
04.28.62	36,03	26,87	50	6,0
04.28.62	36,09	26,88	50	5,6
02.16.63	36,47	28,01	40	4,7
03.11.63	37,96	29,14	40	5,6
05.23.63	36,01	29,1	80	4,5
07.08.63	36,48	27,88	80	4,9
07.26.63	36,84	28,76	80	5,2
08.21.63	36,3	26,7	15	4,7
09.29.63	36,44	29	60	5,0
03.31.64	36,43	28,78	57	4,8
05.13.64	36,28	28,21	82	4,5
06.08.64	36,26	28,26	62	4,6
07.18.64	36,13	26,01	99	5,0
09.28.64	36,7	29,2	63	4,4
10.13.64	36,94	28,29	76	4,5

01.07.65	36,5	26,85	35	5,0
01.09.65	36	27,4	63	4,4
04.09.65	36,8	26,5	33	4,7
04.29.65	37,14	26,89	8	4,7
05.01.65	37,18	26,91	0	4,6
05.07.65	36,74	26,86	162	4,5
06.10.65	36,44	26,64	142	4,7
06.13.65	37,85	29,32	33	5,1
06.17.65	37,6	28,8	33	4,7
06.17.65	37,77	29,36	37	4,7
07.12.65	37,62	29,35	50	4,6
11.08.65	36	27	0	4,2
11.28.65	36,12	27,43	73	5,7
12.02.65	37,61	29,32	38	4,7
12.07.65	36,25	27	0	4,4
12.08.65	37,3	28,5	0	4,5
12.22.65	37,1	28,1	0	4,5
02.08.66	36,23	28,11	79	4,5
03.08.66	36,5	26,6	0	4,3
03.27.66	37	26,9	0	4,2
03.27.66	37	26,9	0	4,2
03.27.66	37	26,9	0	4,4
03.28.66	37	26,9	0	4,2
03.28.66	37	26,9	0	4,2
03.29.66	37	26,9	0	4,3
03.29.66	37	26,9	0	4,4
04.07.66	36,8	26,7	26	4,2
05.04.66	37,74	27,71	37	4,8
05.07.66	37,75	27,79	9	5,0
07.03.66	36,1	26,5	0	4,2
07.17.66	37,05	27,25	0	4,2
08.18.66	36,22	26,35	133	4,4
09.06.66	36,66	26,63	158	4,4
09.10.66	36,53	26,9	146	4,1
10.13.66	36,14	27,83	7	4,5
11.21.66	36,2	29,3	0	4,5
04.04.67	36,68	29,27	24	4,9
05.22.67	36,59	29,35	54	4,6
06.01.67	36,81	29,26	43	5,0
06.02.67	36,89	29,3	0	4,3
06.07.67	37	26,9	0	4,3
06.18.67	36,78	29,32	35	4,9
07.25.67	37,8	28,6	75	4,4
07.25.67	37,9	28,7	101	4,5
08.09.67	36,98	28,4	64	4,8
08.28.67	36,73	26,74	169	4,5

09.05.67	36,72	29,33	24	4,5
10.11.67	36,07	27,12	34	4,6
10.26.67	37,22	29,05	46	5,0
11.13.67	37,78	28,83	34	4,6
12.05.67	36,53	26,85	137	4,6
01.13.68	36,3	27,9	71	4,4
02.07.68	36,65	26,74	153	5,0
02.20.68	36,4	26,2	0	4,1
02.20.68	36,15	27,39	64	4,9
03.30.68	36,7	27,7	64	4,3
07.04.68	36,77	29,03	94	4,4
09.05.68	36,3	26,7	1	4,4
10.06.68	36,96	26,38	17	4,7
10.10.68	36,5	29,2	0	4,5
10.27.68	36,2	27,1	0	4,4
10.28.68	36,75	27,75	0	4,2
10.31.68	36,62	27,01	2	5,1
10.31.68	36,5	27,5	0	4,2
10.31.68	36,75	27,5	0	4,2
11.04.68	36,44	26,98	35	4,6
11.11.68	36,61	27,1	33	4,5
11.12.68	36,74	27,11	26	4,7
11.12.68	36,64	27,16	24	4,6
11.12.68	37,6	28,5	0	4,3
11.16.68	36,6	27,18	0	4,1
11.26.68	36,2	27,3	41	4,3
12.04.68	36,34	26,98	43	4,4
12.04.68	36,48	27,03	36	4,4
12.04.68	36,5	27,02	32	4,7
12.04.68	36,67	27,1	0	4,1
12.04.68	36,71	27,2	58	4,1
12.05.68	36,6	26,92	31	5,4
12.05.68	36,4	27,2	0	4,1
12.21.68	36,6	27,07	30	4,5
12.21.68	36,6	27,3	0	4,2
12.21.68	36,56	27,4	30	4,2
01.14.69	36,11	29,19	22	5,6
02.19.69	37,79	28,2	0	4,2
02.26.69	36,66	27,18	33	4,4
03.23.69	37,9	27,6	0	4,9
03.24.69	36,6	28,6	0	4,6
04.21.69	36,22	28,27	11	4,6
04.24.69	36,35	28,73	53	4,7
04.26.69	36,71	28,5	13	4,3
04.27.69	36,54	28,21	33	4,7

05.23.69	36,1	27	0	4,3
08.03.69	37	29	57	4,1
09.06.69	36,73	28,35	72	5,0
09.22.69	36,57	28,01	86	4,6
12.21.69	36,66	28,42	69	4,6
01.26.70	37	28,5	0	5,1
02.18.70	36,3	27	0	4,7
02.18.70	36,41	27,12	11	4,6
02.18.70	36,68	27,14	10	4,6
02.20.70	36,4	27,2	33	4,9
02.20.70	36,55	27,26	20	4,6
02.24.70	36,37	27,94	102	4,2
03.01.70	36,9	29,1	0	4,5
03.02.70	36,8	28,8	49	4,6
03.27.70	36,03	28,36	0	4,5
03.28.70	37,2	29	0	4,3
04.17.70	37,09	26,92	32	4,7
04.19.70	36	27,1	0	4,5
04.24.70	36,75	28,66	34	4,6
06.28.70	36,2	27,1	0	4,5
09.28.70	37,09	28,59	24	4,6
10.19.70	37,01	29,01	11	4,6
10.24.70	36,86	28,8	28	4,1
11.21.70	36,88	28,92	19	4,5
12.28.70	37,06	29,02	7	4,4
12.28.70	37,09	28,91	23	4,4
12.29.70	36,03	28,34	26	4,6
12.30.70	36,05	28,34	36	4,2
12.30.70	36,96	28,94	23	5,1
12.31.70	37,11	29	38	4,4
01.02.71	37,07	29,04	7	4,4
01.16.71	36,63	26,9	157	4,2
02.20.71	37,82	29,39	36	4,5
03.18.71	36,32	26,98	141	4,4
09.03.71	36,81	28,79	0	4,6
10.16.71	36,63	28,54	61	4,8
11.12.71	36,61	27,09	23	5,1
01.20.72	36,64	27,15	16	4,6
01.20.72	36,64	27,23	34	4,8
03.25.72	36,67	27,51	55	4,4
03.31.72	36,68	27,36	0	4,1
03.31.72	36,62	27,09	18	4,4
08.29.72	37	29,14	0	4,4
10.23.72	37,78	26,32	28	4,4
12.24.72	36,19	27,77	35	4,3
01.13.73	36,3	26,78	54	4,2

09.12.73	36,56	26,99	157	4,3
02.05.74	36,74	26,86	157	4,7
03.12.74	36,76	26,4	45	4,8
05.09.74	36,62	27,22	26	4,4
05.12.74	36,71	26,89	149	4,2
05.16.74	36,11	27,27	32	4,1
07.09.74	36,57	28,48	49	4,9
05.31.75	36,74	28,23	34	4,1
06.02.75	36,47	26,52	31	4,7
09.23.75	36,6	26,76	158	4,6
11.12.75	36,28	28,15	65	5,3
12.08.75	36,43	27,86	5	4,3
01.10.76	36,8	27,92	31	4,6
02.10.76	36,82	27,93	39	4,7
04.06.76	36,62	27,28	151	4,5
05.01.76	36,95	27,94	31	4,3
08.15.76	37,84	28,77	11	5,3
08.17.76	36,74	27,07	160	5,0
08.18.76	36,48	26,95	148	4,3
08.18.76	36,73	27,42	157	4,7
08.19.76	37,71	29	20	5,0
09.12.76	36,59	26,96	154	4,2
12.24.76	36,1	26,77	159	4,3
03.08.77	36,54	28,54	65	4,1
03.28.77	36,82	27,52	36	4,8
07.12.77	36,62	26,97	157	4,4
07.14.77	36,16	27,69	9	4,3
10.27.77	37,87	27,88	16	4,7
01.11.78	37,48	28,86	5	4,9
03.13.78	37,81	26,88	5	4,7
04.03.78	37,22	28,99	0	4,9
06.17.78	37,54	28,81	0	4,8
10.25.78	36,69	27,07	168	4,3
10.29.78	36,04	27,24	9	4,1
11.28.78	36,04	26,39	115	5,0
02.28.79	36,63	27,05	136	4,3
06.22.79	36,8	29,03	11	4,1
08.23.79	37,99	28,91	10	4,2
09.01.79	36,43	26,31	10	4,2
09.03.79	36,05	27,68	89	4,1
10.21.79	36,8	27,77	10	4,4
10.26.79	36,91	27,76	10	4,6
11.06.79	36,83	27,99	10	4,6
11.09.79	36,89	27,93	5	4,4
02.18.80	36,98	27,92	7	4,6
02.19.80	36,95	27,89	4	4,4

04.11.80	36,96	27,84	0	4,6
04.15.80	36,68	27,34	0	4,2
04.29.80	37,07	28,73	32	4,4
06.11.80	36,06	27,77	108	4,3
10.04.80	37	28,8	26	5,0
11.11.80	36,9	28,82	0	4,9
11.11.80	36,9	28,75	1	4,1
01.03.81	36,9	28,6	0	4,8
04.27.81	36,05	28,88	37	4,6
05.11.81	36,78	28,08	22	4,7
06.08.81	36,22	28,87	68	4,1
11.16.81	36,64	26,82	161	4,7
11.19.81	36,33	27,72	41	4,3
12.21.81	36,58	26,81	2	4,6
01.09.82	37,92	28,87	3	4,6
01.24.82	36,61	27,52	146	4,5
04.18.82	36,65	27,11	155	5,0
05.04.82	36,44	26,78	10	4,6
05.05.82	36,35	27	0	4,3
05.11.82	36,93	28,86	12	4,2
05.25.82	36,87	27,83	10	4,4
06.07.82	36,98	27,92	10	4,7
06.12.82	36,92	27,89	10	4,3
06.12.82	36,92	27,89	8	4,1
06.12.82	36,89	27,78	10	4,4
06.28.82	37	27,98	1	4,3
06.29.82	37	28,03	10	4,5
08.30.82	36,76	27,21	10	4,3
02.12.83	36,78	27,52	38	4,5
02.28.83	36,3	27,72	107	4,5
03.22.83	37,01	29,24	10	4,4
03.24.83	37,11	29,35	10	4,6
04.14.83	36,57	27,03	10	4,7
04.23.83	36,24	26,43	136	4,4
06.14.83	36,44	28,44	94	4,1
06.25.83	37,79	29,35	10	4,1
08.04.83	37,84	27,59	0	4,7
09.27.83	36,72	26,93	160	5,4
11.18.83	36,87	28,82	15	4,5
11.21.83	36,26	27,1	24	4,5
11.21.83	36,42	26,92	43	4,6
01.31.84	37,03	28	15	4,9
02.05.84	37,21	28,67	30	5,0
02.06.84	37,09	28,15	26	4,9
03.14.84	37,13	27,96	10	4,6
03.21.84	37,79	28,42	33	4,2

03.25.84	37,74	28,68	0	4,5
04.21.84	36,06	27,24	49	4,7
04.23.84	37,85	26,87	29	4,6
04.23.84	37,83	26,87	27	4,8
04.26.84	37,22	28,42	9	4,3
05.04.84	37,89	29,24	22	4,7
06.07.84	37,23	28,72	10	4,3
06.20.84	36,69	27,05	166	4,7
07.19.84	36,13	27,3	55	4,5
09.23.84	36,52	26,49	155	4,4
11.15.84	36,3	27,61	5	4,3
11.18.84	37,95	28,95	13	4,1
12.16.84	36,35	26,82	147	4,2
02.16.85	36,52	26,52	142	4,5
02.17.85	36,61	27,67	128	4,7
02.25.85	36,43	26,7	157	4,2
04.10.85	36,77	27,46	5	4,5
04.10.85	36,8	27,54	20	5,0
04.23.85	36,29	26,95	137	4,5
05.20.85	36,16	28,82	51	4,8
06.05.85	36,2	27,9	44	4,3
08.23.85	37,23	28,79	11	4,5
09.11.85	36,43	28,87	58	4,6
11.18.85	36,03	27,42	10	4,2
11.24.85	37,65	27,57	23	4,5
12.03.85	36,64	26,89	156	4,6
12.06.85	36,97	28,85	9	4,6
12.23.85	36,88	26,58	39	4,2
12.23.85	36,81	26,62	25	4,9
02.21.86	36,38	26,52	146	4,8
03.19.86	37,55	26,93	19	4,7
03.19.86	37,61	26,93	5	4,6
05.03.86	36,93	28	119	4,2
07.29.86	36,69	27,94	109	4,1
08.11.86	36,2	26,83	128	4,1
08.17.86	36,53	26,96	154	4,2
10.20.86	37,79	27,38	3	4,5
12.31.86	36,19	27,1	42	4,7
01.06.87	36,19	28,03	7	4,3
01.06.87	36,15	28,12	16	4,4
01.07.87	36,17	28,11	35	4,2
01.31.87	36,16	28,14	17	4,3
02.01.87	36,22	28,02	10	4,1
02.01.87	36,08	28,12	25	4,4
04.04.87	36,92	28,39	20	4,6
04.08.87	36,35	26,05	133	4,1

04.08.87	36,09	27,12	46	4,4
04.17.87	36,07	27,13	42	4,3
05.01.87	36,07	27,34	39	4,7
05.02.87	36,03	27,33	24	4,1
05.07.87	36,63	26,75	153	4,7
06.11.87	36,34	26,53	147	4,4
06.19.87	36,8	28,18	85	5,0
07.11.87	36,64	26,9	161	4,5
07.18.87	36,15	28,24	51	4,2
08.31.87	36,58	27,71	113	4,1
09.03.87	36,65	27,17	162	4,2
09.15.87	37,85	26,95	1	4,6
10.05.87	36,24	28,27	27	5,1
10.06.87	36,21	28,29	16	4,6
10.06.87	36,25	28,26	36	4,5
10.06.87	36,28	28,24	20	4,5
10.06.87	36,28	28,33	14	4,7
10.09.87	36,28	28,35	10	4,4
10.09.87	36,24	28,31	8	4,6
10.12.87	36,26	28,3	10	4,6
10.25.87	36,3	28,35	24	4,6
10.25.87	36,28	28,17	8	4,2
10.25.87	36,38	28,21	18	4,5
10.27.87	36,19	28,33	6	4,6
11.26.87	36,06	29,15	14	4,5
12.12.87	36,78	28,28	78	4,3
12.26.87	36,89	27,71	35	4,7
12.30.87	36,88	27,72	30	4,8
01.30.88	36,29	28,22	1	4,5
05.24.88	36,04	28,12	7	4,2
06.02.88	36,28	26,73	130	4,1
08.15.88	37,89	29,24	11	4,7
08.24.88	36,59	26,29	26	5,0
09.11.88	36,26	26,35	124	4,6
09.27.88	36,02	27,16	50	4,2
10.26.88	37,96	27,69	19	4,8
10.29.88	36,1	28,15	4	4,7
12.30.88	36,07	27,27	31	4,5
01.05.89	37,09	27,69	10	4,6
01.08.89	37	27,6	8	4,4
02.19.89	36,98	28,2	1	4,8
02.24.89	37,73	29,33	11	5,0
02.24.89	37,72	29,26	19	4,4
02.24.89	37,73	29,24	23	4,5
03.08.89	36,33	27,74	96	4,5
04.27.89	37,04	28,17	12	5,3

04.28.89	37,03	28,11	17	5,1
04.28.89	37,05	28,03	6	4,1
06.24.89	37,02	28,06	21	4,3
07.19.89	36,14	27,19	49	4,3
08.16.89	36,08	28,85	58	4,2
11.01.89	36,47	26,99	142	4,6
11.24.89	36,71	26,61	11	4,6
12.07.89	36,68	26,95	159	4,2
12.18.89	37,87	29,22	7	4,5
12.22.89	36,91	26,42	10	4,6
12.26.89	36,41	26,79	142	4,3
12.31.89	36,14	27,08	10	4,6
01.08.90	36,08	27,15	26	4,3
01.10.90	36,04	27,1	26	4,6
01.13.90	36,1	27,15	28	4,7
01.13.90	36,09	27,16	23	4,7
01.13.90	36,16	27,1	6	4,8
01.13.90	36,12	27,17	1	4,7
01.13.90	36	27,19	12	4,4
01.14.90	36,04	27,14	23	4,6
01.14.90	36,12	27,14	16	4,4
01.14.90	36,13	27,14	13	4,7
01.14.90	36,04	27,12	37	4,1
01.17.90	36,05	27,14	42	4,5
01.25.90	36,07	27,26	22	4,6
01.25.90	36,09	27,27	23	4,6
01.28.90	36,06	27,18	24	4,4
01.28.90	36,09	27,14	10	4,7
02.08.90	36,66	27,07	159	4,6
02.12.90	36,2	27,1	7	5,0
02.19.90	36,1	27,18	20	4,4
02.19.90	36,11	27,18	37	4,7
02.19.90	36,19	27,14	40	4,7
02.20.90	36,16	27,09	20	4,3
02.21.90	36,05	27,21	54	4,4
02.23.90	36,05	27,15	20	4,8
04.14.90	36,1	27,11	35	4,7
04.20.90	36,18	27,13	22	4,7
04.22.90	36,53	26,88	149	4,6
05.07.90	36,27	27,14	10	4,5
05.15.90	36,1	27,18	11	4,5
05.25.90	36,89	28,64	10	4,2
05.26.90	36,85	28,66	12	4,1
06.13.90	36,53	26,98	10	4,8
06.26.90	37,33	29,23	19	4,2
08.28.90	36,27	27,22	41	4,6

09.01.90	37,02	27,98	7	4,5
11.23.90	36,35	26,85	131	4,1
01.20.91	36,32	26,4	144	4,1
01.21.91	36,63	26,23	10	4,7
01.25.91	36,98	28,83	2	4,3
06.02.91	36,41	28,47	14	4,2
07.18.91	36,09	27,19	108	4,7
07.30.91	36,32	28,21	48	4,3
04.07.92	36,7	26,35	19	4,8
04.23.92	37,33	28,14	31	4,1
05.15.92	37,71	27,62	4	4,3
06.01.92	36,05	28,3	89	4,2
09.30.92	37,25	29,37	10	4,1
11.06.92	37,84	26,79	10	4,1
11.10.92	37,03	27,85	10	4,7
12.25.92	37,21	28,15	3	4,5
01.14.93	37,2	28,3	22	4,6
01.18.93	37,82	28,5	42	4,3
03.05.93	36,84	27,29	10	4,3
03.05.93	36,43	26,68	148	4,2
04.15.93	36,3	26,61	128	4,5
04.19.93	36,36	27,94	34	4,3
04.24.93	36,46	26,8	157	4,2
08.26.93	36,77	28,05	36	5,2
12.02.93	36,78	26,71	10	4,2
03.28.94	36	28,27	5	4,2
04.03.94	37,22	28,96	5	4,2
11.09.94	36,98	29,06	24	4,2
11.13.94	36,91	29,05	10	4,6
11.13.94	36,91	28,98	26	4,5
11.13.94	37,03	28,94	10	4,9
01.22.95	36,9	29,02	16	4,3
01.25.95	36,22	29,3	51	4,4
02.12.95	36,4	26,86	147	4,1
02.28.95	36,33	29,14	55	4,2
03.07.95	36,76	27,76	13	4,9
07.04.95	36,87	28,06	41	4,1
08.03.95	37,4	27,34	33	4,2
08.22.95	36,61	26,71	167	5,0
09.21.95	36,48	26,79	147	4,7
09.28.95	36,63	29,37	22	4,4
10.30.95	36,33	28	43	4,2
10.30.95	36,51	28,15	18	4,2
11.30.95	36,54	27,11	134	4,8
01.02.96	36,19	27,39	38	4,2
02.03.96	37,51	26,86	11	4,5

04.02.96	37,84	26,98	11	5,2
04.04.96	37,89	26,88	12	4,1
04.12.96	36,59	27,04	162	5,2
04.26.96	36,34	27,96	72	5,1
04.29.96	37,06	27,5	1	4,2
04.29.96	37	27,47	10	4,2
04.29.96	37	27,59	10	4,2
05.20.96	37,46	27,55	9	4,2
05.21.96	36,52	26,71	151	4,2
07.20.96	36,13	27,05	27	5,6
07.20.96	36,14	27,16	42	4,3
07.20.96	36,22	27,13	54	4,2
07.20.96	36,17	27,23	54	4,1
07.20.96	36,26	27,15	38	4,6
07.20.96	36,12	27,31	48	4,1
07.20.96	36,17	27,21	17	4,8
07.20.96	36,1	27,17	37	4,3
07.20.96	36,01	27,16	39	4,3
07.21.96	36,11	27,2	5	4,1
07.21.96	36,23	27,15	5	4,2
07.21.96	36,1	27,3	35	4,2
07.21.96	36,13	27,21	32	4,4
07.22.96	36,06	27,19	36	4,1
07.22.96	36,08	27,12	25	4,7
07.22.96	36,38	26,98	31	4,2
07.22.96	36,2	27,28	40	4,2
07.22.96	36,27	27,42	10	4,2
07.22.96	36,36	27,27	5	4,4
07.22.96	36,17	27,18	32	4,1
07.22.96	36,08	27,19	24	4,1
07.22.96	36	27,5	38	4,4
07.22.96	36,26	27,2	25	4,2
07.23.96	36,17	27,3	68	4,3
07.23.96	36,15	27,18	5	4,2
07.23.96	36,65	27,15	36	4,4
07.23.96	36,25	27,29	54	4,2
07.23.96	36,2	27,22	26	4,6
07.24.96	36,63	27,19	17	4,6
07.24.96	36,71	27,17	44	4,2
07.24.96	36,61	27,27	59	4,2
07.24.96	36,22	27,21	37	4,2
07.24.96	36,2	27,2	9	4,2
07.24.96	36,2	27,32	9	4,2
07.24.96	36,68	27,15	25	4,5
07.24.96	36,26	27,08	1	4,2
07.25.96	36,26	27,16	5	4,2

07.25.96	36,24	27,24	5	4,2
07.26.96	36,27	27,21	58	4,2
07.26.96	36,27	27,1	8	4,2
07.26.96	36,13	27,15	32	4,3
07.28.96	36,1	27,2	38	4,3
07.28.96	36,36	27,14	1	4,2
07.29.96	36,71	27,13	2	4,2
07.30.96	36,21	27,21	10	4,2
07.30.96	36,25	27,23	5	4,2
07.30.96	36,1	27,28	7	4,2
07.31.96	36,13	27,23	36	4,3
07.31.96	36,14	27,15	5	4,2
07.31.96	36,1	27,04	5	4,2
08.01.96	36,17	27,17	5	4,2
08.01.96	36,29	27,15	5	4,2
08.03.96	36,31	26,83	5	4,2
08.06.96	36,31	27,06	5	4,2
08.08.96	36,19	27,1	5	4,2
08.10.96	36,3	27,09	10	4,2
08.11.96	36,21	27,13	27	4,1
08.14.96	36,28	27,09	5	4,2
08.18.96	36,24	27,22	27	4,4
08.19.96	36,61	27,19	5	4,2
08.19.96	36,6	27,01	5	4,2
08.20.96	36,25	27,24	31	4,1
08.23.96	36,55	27,05	20	4,2
08.31.96	36,39	26,98	5	4,2
08.31.96	36,43	27,02	5	4,2
09.02.96	36,3	27,2	10	4,2
09.10.96	36,25	27,35	6	4,2
09.22.96	36,08	27,14	14	4,2
09.27.96	36,39	27,1	53	4,2
11.03.96	36,21	27,1	24	4,2
11.06.96	36,28	27,1	34	4,4
11.06.96	36,23	27,03	5	4,2
11.06.96	36,26	27,11	42	4,2
11.07.96	36,86	27,62	5	4,2
11.28.96	37,02	26,65	28	4,2
12.10.96	36,79	27,57	27	4,2
12.10.96	37,14	26,88	7	4,2
12.12.96	36,63	26,73	165	4,2
12.15.96	36,62	26,9	161	4,2
06.02.97	36,04	27,14	5	4,2
06.04.97	36,6	26,4	28	4,1
07.27.97	36,18	27,72	5	4,2
08.08.97	36,51	27,11	5	4,3

08.21.97	36,45	27,21	17	4,2
08.27.97	36,5	27,12	32	4,4
08.29.97	36,51	27,33	5	4,2
09.25.97	36,31	26,6	162	4,2
10.18.97	36,5	27,22	5	4,2
10.24.97	36,46	26,57	155	4,2
12.27.97	36,56	27,16	17	4,3
02.13.98	36,27	28,45	77	4,8
02.24.98	36,3	28,22	31	4,5
03.10.98	36,04	26,03	143	4,4
07.09.98	37,87	26,79	17	4,9
10.03.98	36,13	28,42	70	4,1
10.17.98	36,67	26,7	159	4,2
10.26.98	36,81	28,97	5	4,1
10.30.98	36,17	27,2	11	4,2
11.01.98	36,34	27,06	5	4,1
11.03.98	36,28	27,15	8	4,1
11.19.98	36,59	27,11	6	4,3
11.25.98	36,64	26,35	9	4,3
12.03.98	36,36	26,76	151	4,1
03.03.99	37,29	27,05	3	4,1
03.29.99	37,14	28,8	38	4,2
04.08.99	36,49	26,68	155	4,2
07.26.99	36,64	27,15	153,7	4,3
08.02.99	36,52	29,17	16	4,4
10.05.99	36,75	28,24	16,7	4,8
10.05.99	36,78	28,13	6,3	4,2
12.26.99	36,55	28,03	89	4,1
02.28.00	36,1	26,52	128	4,4
02.28.00	37,05	27,5	10	4,3
04.21.00	37,7	29,25	6,8	4,9
04.27.00	37,88	29,22	10	4,1
08.10.00	37,26	28,96	29	4,1
09.16.00	36,72	27,5	33	4,5
10.04.00	37,92	29,05	8	4,7
10.04.00	37,95	28,91	5	4,3
10.18.00	36,22	28,69	10	4,4
01.04.01	37,66	26,86	11	4,1
02.18.01	37,98	26,66	6	4,2
03.01.01	37,84	26,81	10	4,1
04.06.01	36,17	26,91	151	4,4
04.09.01	36,43	28,58	79	4,2
06.12.01	37,59	26,84	11	4,2
06.30.01	37,28	26,83	27	4,2
10.08.01	36,74	28,18	88	4,2
07.30.02	37,7	29,16	11	4,6

## **APPENDIX B**

### **DESCRIPTION OF THE COMPUTER PROGRAM TELDYN**

TELDYN is a computer program specifically designed for analysis of the response of soils to vertically propagating motions caused by earthquakes. An equivalent linear procedure is used to account for the nonlinearity of the soil. Furthermore, it is possible for the user to divide the input acceleration history into segments and the shear moduli and the damping ratios are then set to be compatible with the average shear strains within each segment. An option is also provided in TELDYN whereby the excess pore pressures in saturated elements can be computed at the end of each segment and the shear moduli can then be determined in the next segment as a function of the reduced mean effective stress.

Here the basic steps of input and output of the program are given:

#### **A. Input Data**

- 1) Nodes
- 2) Elements
- 3) Boundary Conditions
  - a) Compliant boundary
  - b) Viscous boundary
  - c) Mixed boundary
- 4) Material Properties
  - a) Modulus Reduction Curves of Construction Materials
  - b) Damping Curves of Construction Materials
  - c) Pore Pressure Generation Curves
  - d) Specific Values of Material Parameters

e) Saturated Elements

5) Input Motion

a) Data about Horizontal Input Motion

b) Data about Vertical Input Motion

B. Output Options

1) Print Options

2) Restart Options

## APPENDIX C

### DESCRIPTION OF THE COMPUTER PROGRAM SHAKE91

The soil profile is idealized as a system of homogeneous, visco-elastic sublayers of infinite horizontal extent. The response of this system is calculated considering vertically propagating shear waves. An equivalent linear procedure is used to account for the nonlinearity of the soil using an iterative procedure to obtain values for modulus and damping that are compatible with the equivalent uniform strain induced in each sublayer. Thus, at the outset, a set of properties (shear modulus, damping and total unit weight) is assigned to each sublayer of the soil deposit. The analysis is conducted using these properties and the shear strains induced in each sublayer is calculated. The shear modulus and the damping ratio for each sublayer are then modified based on the applicable relationship relating these two properties to shear strain. The analysis is repeated until strain-compatible modulus and damping values are arrived at. Starting with the maximum shear modulus for each sublayer and a low value of damping, essentially (ie, difference less than one percent) strain compatible properties are obtained in 5 to 8 iterations for most soil profiles.

The following assumptions are incorporated in the analysis:

- 1) Each sublayer,  $m$ , is completely defined by its shear modulus,  $G_m$ , damping ratio,  $\mu_m$ , total unit weight,  $\gamma_{tm}$  (or corresponding mass density,  $\rho_m$ ) and thickness,  $h_m$ ; these properties are independent of frequency.
- 2) The responses in the soil profile are caused by the upward propagation of shear waves from the underlying rock half-space.

- 3) The shear waves are specified as acceleration ordinates at equally spaced time intervals. (Cyclic repetition of the acceleration time history is implied in the solution).
- 4) The strain dependence of the shear modulus and damping in each sublayer is accounted for by an equivalent linear procedure based on an equivalent uniform strain computed in that sublayer. The ratio of this equivalent uniform strain divided by the calculated maximum strain is specified by the user and is assumed to be the same for all sublayers.

The options incorporated into SHAKE91 are as follows:

- 1) Dynamic soil properties
- 2) Data for soil profile
- 3) Input motion
- 4) Assignment of object motion to the top of a specified sublayer
- 5) Number of iterations specified and ratio of uniform strain to maximum strain
- 6) Sublayers at top of which peak accelerations and time histories are computed and saved
- 7) Sublayer at top of which time history of shear stress or strain is computed and saved
- 8) Save time history of object motion
- 9) Compute response spectrum
- 10) Compute amplification spectrum
- 11) Compute Fourier amplitudes

## **APPENDIX D**

### **COMPUTER PROGRAM ACCHIS**

In order to determine the time history of average acceleration of the critical slip surface a computer program called as ACCHIS has been written by using the FORTRAN programming language.

The input file of the program includes the elements with their nodes and masses and the acceleration time histories at the nodal points of the sliding surface which are computed by using the finite element program TELDYN in this study.

```

dimension nelco(4,4),xmass(4),ac(4,2200),avac(4,2200)
*,force(4,2200),forcet(2200),acav(2200)
character*4 a
open(1,file='akkxs1')
open(2,file='akkxs1acc')
read(1,100) nel,acr,dt,nt
print*, nel,acr,dt,nt
do 10 k=1,nel
read(1,150) (nelco(k,i),i=1,4),xmass(k)
print*,(nelco(k,i),i=1,4),xmass(k)
10 continue
j=0
do 20 kk=1,nel
25 rewind 1
j=j+1
do 21 l=1,nt
21 ac(j,l)=0
continue
30 read(1,200) a
if (a.ne.'node') goto 30
backspace 1
read(1,250) nn
if (nn.ne.nelco(kk,j)) goto 30
read(1,300) (ac(j,jj),jj=1,nt)
if (j.lt.4) goto 25
continue
do 35 ii=1,nt
avac(kk,ii)=(ac(1,ii)+ac(2,ii)+ac(3,ii)+ac(4,ii))/4
35 continue
j=0
rewind 1

```

```

20  continue
    do 40 kj=1,nel
        do 40 kl=1,nt
            force(kj,kl)=avac(kj,kl)*xmass(kj)
40  continue
        do 50 km=1,nt
            forcet(km)=0
            do 50 kn=1,nel
                forcet(km)=forcet(km)+force(kn,km)
50  continue
            totmass=0
            do 55 jk=1,nel
                totmass=totmass+xmass(jk)
55  continue
            do 60 jl=1,nt
                acav(jl)=forcet(jl)/totmass
                write(2,350) acav(jl)
60  continue
100 format(i5,2f5.0,i5)
150 format(4i5,f4.2)
200 format(a4)
250 format(4x,i2)
300 format(8f9.4)
350 format(f9.6)

    stop
    end

```

## **APPENDIX E**

### **COMPUTER PROGRAM PDISP**

In order to calculate the permanent displacement from a given average time history of the sliding mass, the computer program PDISP is written in MATLAB. The steps of the program was previously defined in Section 2.5. The input file obtained from program ACCHIS is AccRecord1 and the output files are vel and disp.

Here the program written for Synthetic Record 1 is given as an example.

```

a=load('AccRecord1.txt');
plot(a);
figure

sum=0;
a=a;
for i=1:2200
if((a(i)<=.24) && (sum<=0))
velocity(i)=0;
t=i;
elseif (a(i)>.24) || (sum>0)
    a(i)=a(i)-.24;
    y=t+1;
    for x=y:i
        vel(i)=trapz(a(y:i))*0.025;
        sum=vel(i);
    end
end
end
end
for k=1:2200
    if(vel(k)<0)
        vel(k)=0;
    end
end
vel=vel;
plot(velocity)
for z=1:2200
    disp(z)=trapz(vel(1:z))*0.025;
    sum=disp(z);
end
figure
plot (disp)

```