

ENGINEERING GEOLOGICAL AND GEOTECHNICAL SITE
CHARACTERIZATION AND DETERMINATION OF THE SEISMIC
HAZARDS OF UPPER PLIOCENE AND QUATERNARY DEPOSITS
SITUATED TOWARDS THE WEST OF ANKARA

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

BY

MUSTAFA KEREM KOÇKAR

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF DOCTOR OF PHILOSOPHY
IN
GEOLOGICAL ENGINEERING

JANUARY 2006

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ABSTRACT

ENGINEERING GEOLOGICAL AND GEOTECHNICAL SITE CHARACTERIZATION AND DETERMINATION OF THE SEISMIC HAZARDS OF UPPER PLIOCENE AND QUATERNARY DEPOSITS SITUATED TOWARDS THE WEST OF ANKARA

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January 2006, 401 pages

The purpose of this study is to assess the engineering geological and geotechnical characteristics and to perform seismic hazard studies of the Upper Pliocene and Quaternary deposits located towards the west of Ankara. Based on a general engineering geological and seismic characterization of the site, site classification systems are assigned for seismic hazard assessment studies. The objective of the research is to determine the regional and local seismic soil conditions, predominant periods and ground amplifications, and to idealize the soil profile of the sites by the aid of surface geophysical methods. These studies are combined and integrated with the geotechnical database from a variety of in-situ and laboratory studies that are compiled from present and previous studies regarding the project area and then transferred to an analytical environment for creating relevant information for our site. Then, engineering geological and geotechnical seismic characterization along with seismic zoning map preparation is accomplished. Finally, based on a general

engineering geological and geotechnical site characterization, site classification systems are assigned to account for site effects in seismic hazard assessments along with the assessment of mitigation and remediation of seismic hazards.

Keywords: Quaternary Sediments, Site Characterization, Seismic Hazard Assessments, Site Amplifications, Ankara

ÖZ

ANKARA’NIN BATISINDAKİ GEÇ PLİYOSEN VE KUVATERNER ZEMİNLERİN MÜHENDİSLİK JEOLJİSİ VE JEOTEKNİK KARAKTERİZASYONUNUN YAPILMASI VE SİSMİK TEHLİKE DEĞERLENDİRMELERİNİN BELİRLENMESİ

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Ocak 2006, 401 sayfa

Bu çalışmanın amacı Ankara’nın batısındaki, özellikle Geç Pliyosen ve Kuvaterner zeminlerin mühendislik jeolojisi ve jeoteknik karakterizasyonunun ve sismik tehlike değerlendirmelerinin yapılmasıdır. Mühendislik jeolojisi ve jeoteknik zemin karakterizasyonu çalışmalarına bağlı olarak proje alanındaki zemin sınıflandırma sistemleri, sismik risk değerlendirmeleri için belirlenmiştir. Bu amaç doğrultusunda proje alanındaki bölgesel ve yerel sismik zemin özellikleri, hakim titreşim periyotları, zemin büyütme değerleri ve ideal zemin profilleri yüzey jeofiziği metodları yardımıyla belirlenmiştir. Bu çalışmalar proje alanında bu zamana kadar yapılmış ve yapılmakta olan jeoteknik ve mühendislik jeolojisi kriterlerine göre arazi ve laboratuvar çalışmaları sonucunda hazırlanan zemin sınıflaması haritaları ve zemin profilleri ile karşılaştırılarak zemin karakterizasyonlarının yapılmasına ve sismik zonlama haritalarının hazırlanmasına yardımcı olmuştur. Sonuç olarak, proje alanı için jeoteknik zemin sınıflandırma sistemleri, zeminlerin yer etkileri de

gözönüne alınarak sismik tehlike değerdendirmeleri yapılmış ve ne tür önlemler alınabileceğı hakkında gerekli öneriler verilmiştir.

Anahtar Kelimeler: Kuvaterner Sedimanlar, Zemin Karakterizasyonu, Sismik Tehlike Değerdendirmeleri, Zemin Büyütmeleri, Ankara

This thesis is dedicated to my son who is due to be born

ACKNOWLEDGMENTS

I would like to express my appreciation to many people who have contributed to this dissertation. Their help and support was a significant benefit to this study.

The years spent working towards the completion of this dissertation was challenging, stimulating and exciting. Even though the reasons to embark in this academic experience were personal, many people were part of it and deserve recognition. First of all I must acknowledge and express my respectful gratitude to my academic advisor, Prof. Dr. Haluk Akgün who was always there from the beginning of my career to the very end providing advice, friendship, and support; his trust in my work and his unique ability to guide, made it possible. His sharing knowledge and insight, and his guidance are always appreciated.

I would like to express my gratitude and acknowledgement to Assoc. Prof. Dr. Ellen M. Rathje from The University of Texas at Austin for her support and guidance throughout the course of this study, particularly when I was in Austin, TX for a period of almost 11 months. I would like to take the opportunity to thank her for sharing her insights on many aspects of this study and on Geotechnical Earthquake Engineering.

I would like to express my sincere gratitude to the members of my dissertation committee, namely, Prof. Dr. Vedat Doyuran, Prof. Dr. Asuman G. Türkmenoğlu and Prof. Dr. Erdal Çokça, who were generous in giving their time and provided very invaluable guidance and comments regarding this dissertation research. Particularly, I would like to thank Prof. Dr. Vedat Doyuran again. His guidance is one of the reasons for me to get into Applied Geology. I would also like to

acknowledge Prof. Dr. Ali Koyigit for his encouragement during the dissertation and valuable comments regarding the geology and tectonics of Ankara.

I am grateful to Assist. Prof. Dr. Piril Önen not only for contributing to the dissertation but also for her encouragement, constructive criticisms and advice throughout the past several years of my academic life. Additionally, I would like to express my gratitude to Dr. Uğur Kuran for his enthusiasm throughout the dissertation and valuable assistance regarding the information he has supplied on historical earthquakes and on site investigation studies. Thanks are due to Mr. Mehmet Altıntaş and Prof. Dr. Polat Gülkan for his encouragement and assistance during the field testing studies. Without their advice and support, this work would have been more difficult. I would like to acknowledge Prof. Dr. Philippe Rosset for providing to use and modify his software of SPECRATIO.

I would like to acknowledge the financial support provided by the Middle East Technical University Research Fund Project (BAP-2004-03-09-01). I am very grateful for this support.

I would also like to express my gratitude to several organizations including General Directorate of Disaster Affairs, Earthquake Research Department (AFET) who provided the assistance and conducted the short-period noise recordings of microtremor measurements; and ESER Technical Boring and Trading Inc. who provided the assistance and conducted the surface geophysical testing field studies of seismic refraction and resistivity measurement under the sponsorship of Middle East Technical University Research Fund Project (BAP). Additionally, I would like to thank several governmental organizations and private companies including, Turkish General Staff and particularly General Directorate of State Hydraulic Works (DSI), General Directorate of Mineral Research and Exploration, General Directorate of Railways, Harbors and Airports Construction Railroad (DLH), AKTÜRK Construction Industry and Trading Co., TOKER Drilling and Construction Co., TEKAR Technical Research Limited Co., Yüksel Project Int., Temelsu International Engineering Services, GÜRİŞ Construction and Engineering

Co. Inc. and ESER Technical Boring and Trading Inc. who provided geotechnical reports, data sets and support for this study.

I would like to thank the Turkish General Staff, particularly Land, Air and Gendarmerie Forces to give permission and supply assistance during the in-situ seismic site characterization studies along the military area in the Ankara basin.

I would like to thank to Mr. Özgür Aktürk (METU); Mr. Mete Mirzaoglu, Bekir Tüzel and Murat Nurlu (ERD); Mustafa Özveren and Nihat Kerkütlü (ESER Technical Boring and Trading Inc.); Kıvanç Okalp (METU), Baran Bilçen and Sibel Yanikömeroğlu (AKTÜRK), Mehmet Gökçeer (TOKER) and Cem Özbey (UT, Austin) for their contribution and assistance during the study. Their contributions are gratefully acknowledged.

I would like to express my appreciation to the many others who are not mentioned here but who have contributed and assisted this dissertation. Their contributions and assistances are greatly appreciated.

The author also would like to thank his friends and colleagues Mr. Özgür Aktürk, Mr. Cem Özbey, Mr. Ali Sarı, Mr. Kıvanç Okalp, Mr. Evrim Sopacı, and many others for their encouragement and friendship throughout the evolutionary stages of this study.

Finally I would like to express my deep thanks to my wife Benat for her patience and encouragement and to my parents who inspired and believed in me as I progressed through my studies. Their belief in me has always been an inspiration and has encouraged me throughout my studies.

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CHAPTER 1

INTRODUCTION

1.1. Introduction

Every damaging earthquake reaffirms the importance of seismic hazard and seismic risk assessment for identifying and mitigating against the consequences of an earthquake. Although some progress in the area of seismic prediction has been made, earthquakes cannot be accurately predicted in time, magnitude, or location. Even if an accurate prediction were possible, the earthquake occurrence and consequent damage potential could not be prevented. Seismic hazard and risk cannot be eliminated, but it can be effectively analyzed and possibly reduced by combining the available regional seismic hazard estimation studies with recent technological developments. Hence, sufficient engineering geological, geotechnical and seismic data must be available to perform independent analyses and thus a large number of factors are required to describe these earthquake hazards. The various seismic hazard evaluations have been combined with these site specific engineering geological, geotechnical and geophysical information to enable the development of seismic zonation studies that summarize the potential for the respective hazards within the specified area.

Earthquake hazard zonation has become an important consideration for planners and engineers as many governmental agencies attempt to take a pro-active role in emergency response planning. By knowing the type and extent of anticipated damage, the agencies hope to control additional development in damage-prone areas and to promote the retrofitting of critical structures. Since there is likely to be significant direct and indirect costs associated with the

retrofitting of structures or with restricting development in certain areas, it is imperative that the hazard zonation be as specific as possible.

Seismic hazard zonation provides an environment which is ideal for conducting earthquake hazard analyses where the different geotechnical and geological hazards associated with seismic events can be taken into account. Common geotechnical hazards that occur during earthquakes particularly include ground motion amplification, soil liquefaction, landslide and surface fault rupture. This research study focuses on a methodology in evaluating engineering geological and geotechnical seismic site characterization, and then evaluating local site effects in terms of liquefaction and soil amplification. The methodology combines the spatial distribution and uncertainty associated with geotechnical parameters used in the analytical operations. For instance, a well-established liquefaction evaluation method which is the liquefaction potential index is applied to reflect the severity of the hazard at the ground surface and the ambient noise measurements to estimate ground motion responses are applied in the urban environments through using different approaches to account for local site effects and to establish the seismic zonation map. The seismic hazard methodologies have been employed to determine the earthquake hazard potential of the research area with an areal extent of about 300 km² which is situated towards the western part of the Ankara basin.

1.2. Purpose and Scope

The purpose of this research is to assess the engineering geological and geotechnical site characteristics and to perform seismic hazard studies of the Upper Pliocene to Pleistocene fluvial and Quaternary alluvial and terrace deposits situated towards the west of Ankara. In this regard, an extensive in-situ geophysical investigation program was conducted by the METU, Department of Geological Engineering research team in the western part of the Ankara basin along the sedimentary deposits for a period of almost five months (i.e., during July 27, 2004 through December 25, 2004). The geophysical field investigation portion of this research, namely microtremor measurements, and seismic refraction and electrical

sounding was assisted by the General Directorate of Disaster Affairs, Earthquake Research Department and ESER Technical Boring and Trading Inc., respectively. Then, these field investigation studies have been integrated with the geotechnical database that was compiled from a variety of previously performed in-situ and laboratory studies to create relevant geotechnical information for the research site.

This research mainly focused on the development of a methodology to integrate the various components necessary for a regional multi-hazard seismic risk analysis that included consideration of hazards due to local site effects such as soil amplification and liquefaction potential. These tasks have been fulfilled through the development of an engineering geological, geotechnical and seismic data database that were obtained from invasive (e.g., boreholes) and non-invasive (e.g., geophysical) explorations at the study area. The engineering geological and geotechnical site characterization studies (i.e., in-situ SPT results along with the geotechnical laboratory testing studies) have been compiled; and seismic site characterization (i.e., surface P and S wave velocity, electrical sounding and microtremor measurement) studies have been performed, particularly in the Quaternary alluvial and terrace sediments of the Ankara basin. Other complementary data sources regarding topographical, geological and hydrogeological maps, deep borings and laboratory test results and satellite images were also included in the database. In particular, surface wave velocities were measured to aid in the development of a regional V_s model for the site classes. Short-period noise recordings of microtremor measurements were conducted to study the seismic response and to estimate the fundamental resonance periods and amplification factors of the site. By using all of these studies, a seismic hazard assessment map (i.e., liquefaction susceptibility, liquefaction potential, site period, site amplification and site class maps) was developed. Finally, the consequences of the seismic hazards were investigated and mitigation and remediation recommendations were presented. The methodology for the integrated analysis presented in this dissertation is designed for seismic hazard and risk analysis on a regional scale which is intended to give general estimates of potential damage distributions and to indicate areas that require further detailed investigation. Since the models chosen to be included in the information analysis are suitable to be used

with regional spatially-distributed data, a hazard model for each of the hazard components listed above was developed and implemented in the GIS environment. Since local soil conditions play a major role in the modeling of the various seismic hazards; a treatment of the soil parameter uncertainty was included to account for the large variations in the amount and type of spatial geologic data that are available for the region.

The work, research and methodology reported in this dissertation, namely a synthesis of the geophysical and geotechnical data for geological and geotechnical site characterization along with site effect considerations in regards to seismic risk evaluations of the foundation soils situated towards the west of Ankara is novel and no such detailed geotechnical and seismic characterization in this extent has been previously performed, particularly on Quaternary alluvial sediments in this region. On a regional scale, few important field studies that were conducted mainly for investigating the geotechnical properties of the Upper Pliocene to Pleistocene fluvial red clastics that are so-called “Ankara Clay Formation” in regards to foundation soils (Ordemir et al., 1965; Lohnes, 1974; Birand, 1978; Kasapoğlu, 1980; Erol et al., 1980; Kiper, 1983; Çokça, 2000) and for seismicity (Tabban, 1976; Ergünay, 1978) might also be listed as previous studies even though their purposes were different than seismic hazard assessments.

Even though the methodology reported in this dissertation is not only important for Ankara but is important for Turkey in its entirety, especially for regions situated in or close to the vicinity of earthquake prone areas, studies of such character and detail that have been performed in Turkey are very limited in number. If the detailed geological and geotechnical characterization work presented through this dissertation were to be considered as a “whole”, it seems to represent a “prototype” research work that needs to be applied to a wide range of areas in Turkey, especially those regions situated in or close to the vicinity of earthquake prone areas. Hence, the methodology reported through this dissertation will form a starting point as well as a systematic study for geological and geotechnical characterization studies of similar nature. As a sum, this dissertation that involves the engineering geological, geotechnical and seismic characterization of the foundation soils situated towards the west of Ankara comprises a novel and

scientific study in regards to developing a methodology to protect the safety and welfare of the public along with providing information and technology for future work of similar characteristics.

1.3. Location and Climate

The research area lies within the Ankara basin located towards the west of Ankara in an approximately ENE-WSW-trending, 25-30 km long and 10-15 km wide structural depression (or basin) that is drained in the east-west direction through the present-day course of the Ankara River. The major part of the Ankara basin constitutes Upper Pliocene to Pleistocene fluvial red clastics and Quaternary terrace and alluvial deposits that are formed in and near a fault-bounded depression as a result of fault-controlled continental sedimentation (Koçyiğit, 1991). On a regional scale, the study area is in the Ankara region which is about 150 km wide and 250 km long; and includes numerous earthquake centers related to the intermediate areas between the northern and southern Anatolian ranges and to the faults which form the boundaries between these areas and the ranges. The research area that is situated within the western part of the city center of the Ankara province along with the main municipalities and city districts is shown in Figure 1.1.

Continental climate prevails in the study area. Mean annual precipitation for the years 1975 through 2004 according to the records of the General Directorate of State Meteorological Works of the Ankara Meteorological Stations is 382 mm (Figure 1.2). Mean annual temperature is 11.7 °C. The mean temperature is maximum in June and August (23.1°C and 23 °C, respectively) and minimum in January and February (0 and 1.2 °C, respectively) (Figure 1.3). The annual mean evapotranspiration is 1316.9 mm.

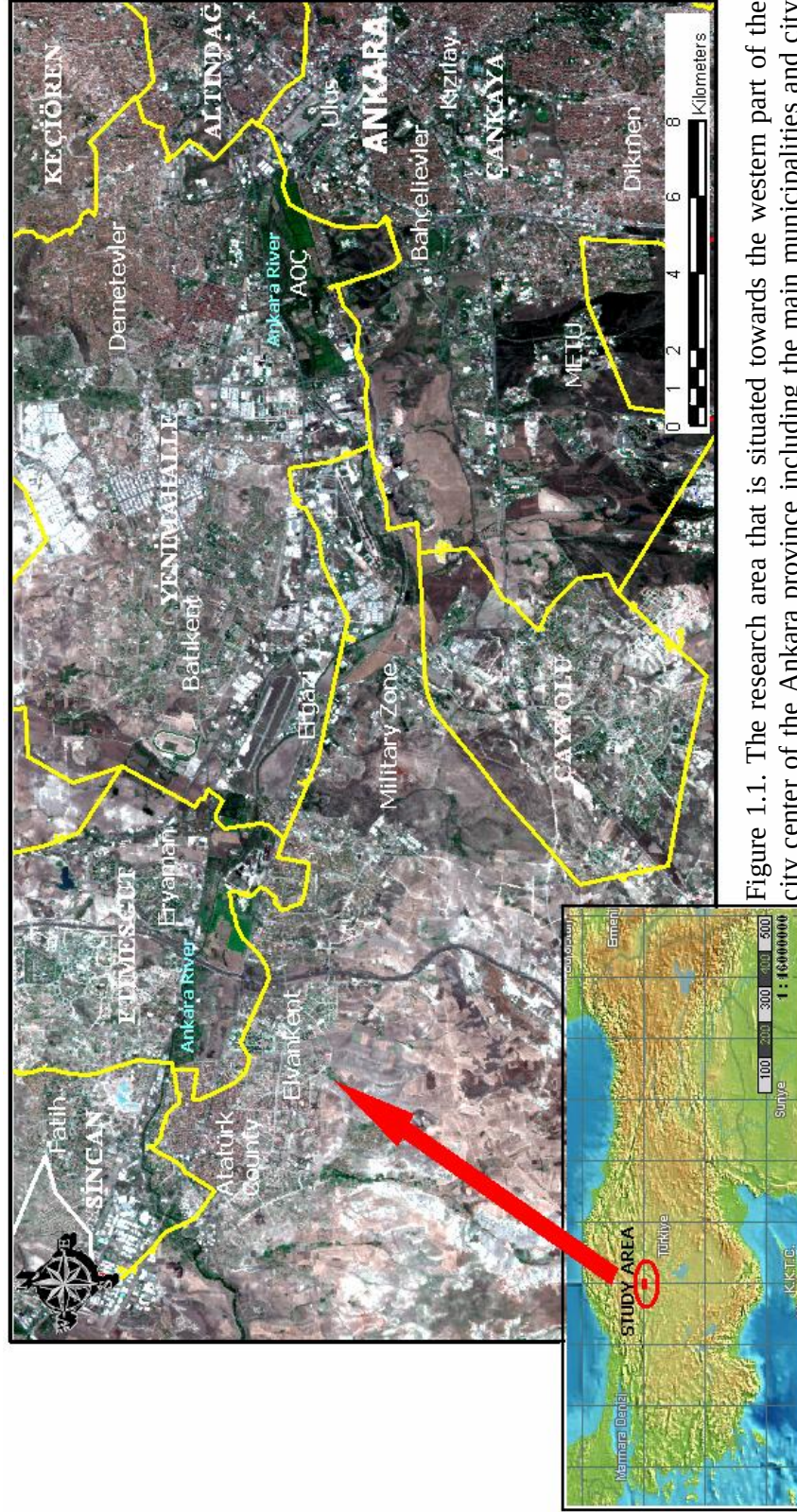


Figure 1.1. The research area that is situated towards the western part of the city center of the Ankara province including the main municipalities and city districts

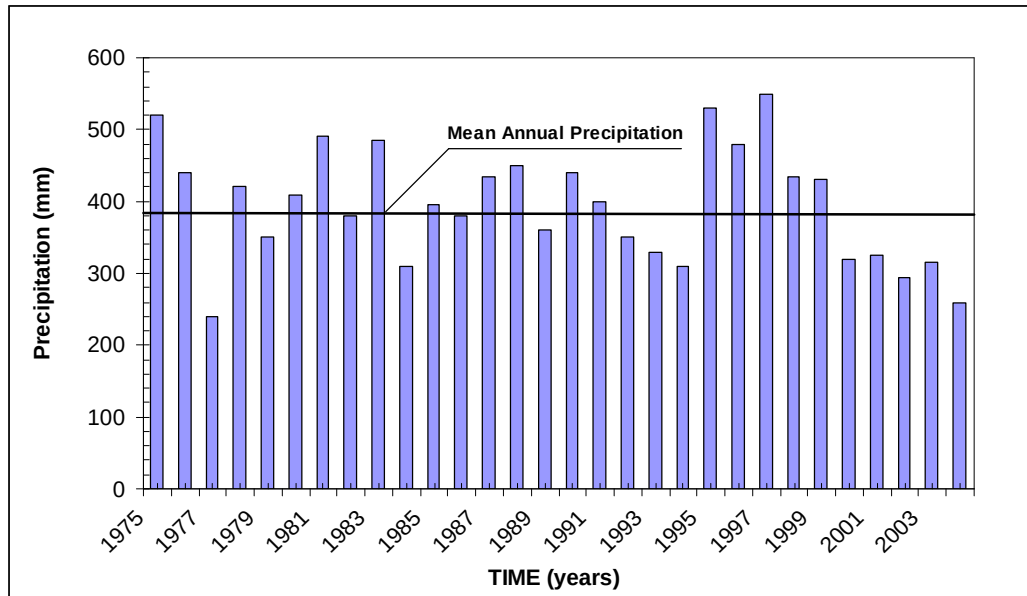


Figure 1.2. Annual precipitation totals for Ankara between 1975 and 2004.

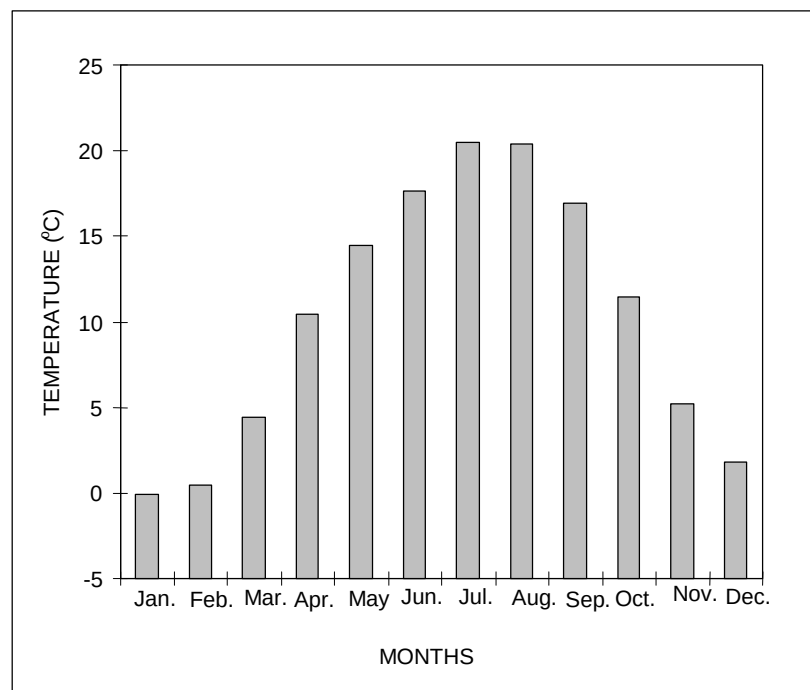


Figure 1.3. Mean monthly temperatures for Ankara between 1975 and 2004

1.4. Organization of the Dissertation

This dissertation documents the assessment of the geotechnical characteristics and the seismic hazard studies related to Pliocene and Quaternary deposits which are located towards the west of Ankara.

Chapter One presents an introduction while Chapter Two provides an overview of the regional geologic, tectonic and seismological setting of the study area and discusses the general character and distribution of the major geologic units, particularly Upper Pliocene to Pleistocene fluvial and Quaternary terrace and alluvial deposits in the Ankara basin that have been investigated in regards to their geological, geomorphological and neotectonic characteristics for seismic hazard assessments.

Chapter Three addresses the importance of a regional seismic hazard analysis to quantify the potential damages and losses in the Ankara region due to future earthquakes. An evaluation of the seismic source characteristics; historical and recent seismicity, and attenuation of ground motion intensity of the Ankara region considered for the assessment of earthquake hazards have been performed. Finally, a discussion of the deterministic seismic hazard scenarios that were conducted for the Ankara region and the probabilistic seismic hazard analysis that have been compiled from the regional scale comprehensive studies to be used for the determination of bedrock peak ground motion for the Ankara region are presented.

Chapter Four discusses the methodologies that were employed for the development of an engineering geological, geotechnical and seismic site characterization study. The integration of a variety of databases coupled with invasive and non-invasive field testing results obtained in this dissertation study to identify and map geotechnical hazards utilizing geographic information technology were performed. Hence, the various seismic hazard evaluations have been combined with site specific engineering geological, geotechnical and geophysical information to enable the development of detailed maps that summarize the potential for the respective hazards, and then the in-situ test methods utilized for

the engineering geological and geotechnical characterization of the western part of Ankara basin are presented.

Chapter Five presents the regional geotechnical and seismic site characterizations results based on information assembled from databases provided by a number of engineering geological and geotechnical seismic site characterization studies in order to identify the relevant parameters and factors, and to determine the variations in local site conditions according to design code of IBC2003 (ICC, 2003). In particular, surface wave velocities were measured, and then correlated with other characterization results to aid in the development of a regional V_s model and to construct seismic zonation maps for assessing site conditions to be used in predicting the site characteristics of the Ankara basin. Other supplementary site characterization methods were also employed for idealizing the local geologic conditions for site characterizations.

The general concepts of liquefaction of soils during earthquakes are considered in Chapter Six. The first part focuses on preliminary geological site evaluation and identifies the liquefaction susceptible zones based on Quaternary geological maps. Then more quantitative evaluation methods, namely, standard penetration and shear wave velocity simplified procedures for evaluating the liquefaction resistance of soil, and the concept of liquefaction potential index and its evaluation methods are explained. Finally, the primary products of these liquefaction hazard evaluation studies showing the liquefaction susceptibility and liquefaction potential zonation maps that might be applied to properly assess and mitigate the potential risk from future liquefaction hazards in the study area are presented.

Chapter Seven focuses on seismic hazards due to the effect of local site conditions near the earth's surface. The amplification of ground motion due to local site conditions (such as ground motion resonance and amplification, etc.) that play an important role in increasing the seismic damage are discussed. Hence, the ambient noise measurements, namely the Reference Site (S/R) and the Horizontal-to-Vertical Component (H/V) spectral ratios were performed to obtain ground motion responses and to estimate the local site conditions of amplitudes and resonances. Particularly, the H/V spectral ratio of the Nakamura technique that was

employed provided satisfactory estimates of the site response of soft soil deposits since it is well adapted to the urban environment for assessing site effects and for determining the fundamental frequencies and establishing seismic zonation.

Finally, a summary, conclusion and recommendations of this research effort are presented in Chapter Eight.

CHAPTER 2

REGIONAL GEOLOGY AND ITS SIGNIFICANCE FOR EVALUATING EARTHQUAKE HAZARDS

2.1. Introduction

The purpose of this chapter is to provide an overview of the regional geologic setting of the study area and to indicate the general character and distribution of the major geologic units. More detailed descriptions of the lithologies and engineering geological properties of the neotectonic units, especially Quaternary deposits in the Ankara basin have been investigated and compiled in regards to their geological, geomorphological and neotectonic characteristics in an account for hazard assessments.

The dissertation area lies within the Ankara basin located towards the west of Ankara that is an approximately ENE-WSW-trending, 25-30 km long and 10-15 km wide structural depression (or basin) that is drained in the east-west direction through the present-day course of the Ankara River. The major part of the Ankara basin constitutes Upper Pliocene to Pleistocene fluvial red clastics (Yalınca Formation) and Quaternary terrace and alluvial deposits that are formed in and near fault-bounded depression as a result of fault-controlled continental sedimentation (Figure 2.1).

The geologic, tectonic and geomorphologic characteristics of the Ankara Region have been studied by various researchers since the eighteenth century. Particularly, geological characteristics of this region have been studied by Chaput (1931), Salamon-Calvi (1940), Salamon-Calvi and Kleinsorge (1940), Chaput (1947), Lahn (1949), Bailey and McCallien (1950), Erol (1954), (1955), (1956), Erentöz (1975), Kasapoğlu (1980), Akyürek et al. (1984), (1996), Tokay et al.

(1988), Koçyiğit and Türkmenoğlu (1991); tectonical characteristics of this region have been studied by Bailey and McCallien (1953), Erol (1961), Ketin (1959), Koçyiğit (1987, 1989, 1991, 2003) geomorphological characteristics of Ankara region has been studied by Salamon-Calvi (1936), İlyüz (1940), Pfannenstiel (1940), Chaput (1947), Lahn (1948), Erol (1964, 1973, 1980, 1993) and Erol et al. (1980).

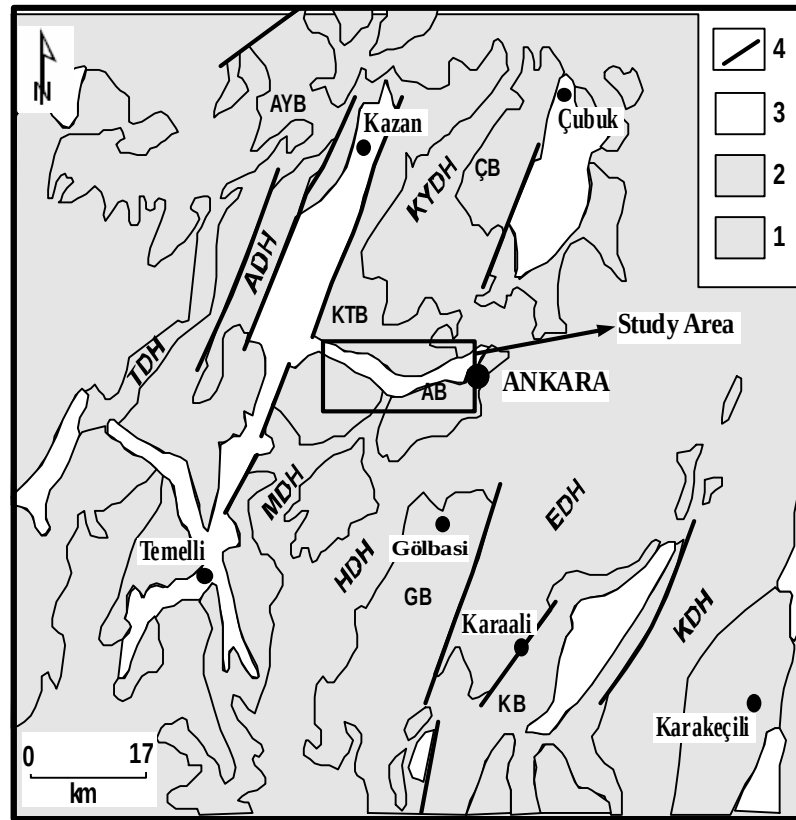


Figure 2.1. Simplified geological map that presents the location of the Ankara basin and its surrounding regions (1- Quaternary alluvial sediments, 2- Upper Miocene -Pliocene continental basin deposits and volcanics, 3-Pre-Upper Miocene basement rocks, and 4- Basin margin fault. Key to abbreviations: AB-Ankara Basin, AYB-Ayaş Basin, BB-Beyşehir Basin, ÇB-Çubuk Basin, GB-Gölbaşı Basin, KB-Karaali Basin, KKB-Karakeçili Basin, KTB-Kazan-Temelli Basin, ADH-Abdülselemdağ Highland, EDH-Elmadağ Highland, HDH-Hacılaradağ Highland, KDH-Küredağ Highland, KYDH-Karyağdıdağ Highland, MDH-Meşedağ Highland, and TDH-Torludağ Highland.) (after Koçyiğit and Türkmenoğlu, 1991).

The lithological characteristics of the Ankara Region are highly complex. The rock units cropping out in the region range from Triassic to Quaternary in age. The older rock units in the Ankara region are highly deformed pre-Upper Miocene basement rocks and Upper Miocene-Lower Pliocene rocks. They are encountered unconformably by the Triassic schists and greywackes with carbonate blocks, Liassic clastics and Upper Jurassic-Lower Cretaceous limestones and the Upper Miocene-Lower Pliocene volcanics, fluvial-lacustrine sedimentary rocks from bottom to top (Koçyiğit and Türkmenoğlu, 1991). Based on the nature of the tectonic regime and type of deformation, these rock units are defined as paleotectonic units. The younger stratigraphic rock units in this region are encountered unconformably by the Upper Pliocene to Pleistocene fluvial red clastics and Quaternary alluvial and terrace deposits. These exposed rock units in study area are also defined as neotectonic units depending on the tectonic regime that is still active and continuing at present (Figure 2.2).

The scope of this study is to mainly investigate the engineering geological, seismic and geotechnical site characteristics, and to perform site classification of the Upper Pliocene to Pleistocene fluvial red clastics and Quaternary alluvial and terrace deposits located towards the west of the center of the city of Ankara which is situated towards the west of the Ankara Plain. In this respect, the Upper Pliocene to Pleistocene fluvial red clastics and Quaternary alluvial and terrace deposits are studied in more detail in regards to their geologic, geomorphologic, paleogeographic and neotectonic characteristics.

2.2. Paleotectonic Units

Paleotectonic units delineating areas in the western part of the Ankara basin are the Triassic schists and greywackes with carbonate blocks in the south-east, Liassic clastics and Upper Jurassic-Lower Cretaceous limestones in the south, and Upper Miocene-Lower Pliocene volcanics and fluvial-lacustrine sedimentary rocks in the north, east, north-east and in the south-west, respectively, which delineates the periphery of the study area.

The Ankara group of Triassic age consists of the rock units forming the lowermost part of the sequence in the region, which is divided into several formations of which some crop out in and around the study area. They consist mainly of dark brown graywacke, black shale and diverse sized carbonate blocks. Graywackes are massive to medium bedded, highly deformed, intensely crushed and sheared. The shear planes are marked by offsetted calcite and quartz veins. Shales are dark blue to black in colour, laminated and show both vertical and lateral gradations with the graywackes (Figure.2.3). The rock units that constitute the Ankara Group comprise the limestone blocks of various dimensions, dissimilar age and facies (Akyürek et al., 1996) which are of Carboniferous, Permo-Carboniferous and Permian ages (Erol, 1956). They float in a matrix composed of graywacke and shale. They are massive, medium to thick bedded and laminated, in places. The Liassic clastics unconformably overlie the Ankara Group. These sedimentary sequences grade upward into Upper Jurassic-Lower Cretaceous limestones that crop out in the study area (Ketin, 1959 and Akyürek et al., 1984) (Figure 2.4).

Upper Miocene-Lower Pliocene volcanics and fluvial-lacustrine sedimentary rocks lie uncomfortably on the erosional surface of pre-Miocene basement rocks and consist of various volcanic and continental sedimentary facies, including mostly unsorted basal conglomerates, andesitic pyroclastic rocks, fluvial sandstone and red conglomerate, coal-bearing lacustrine shale, marl, limestone, evaporite, andesitic to dacitic and basaltic lavas and sills (Figure 2.5). All contacts between these lithofacies are gradational (Figure 2.3) (Koçyiğit, 1991). In the northern part of the study area, mostly Miocene Tekke volcanics and Mamak formation are exposed that composed of andesite, trachyandesite, basalt, agglomerate, tuff and agglomerate, tuff, andesite, respectively. As an example of the Lower Pliocene volcanics (Bozdağ Basalt) that crop out in the study area was shown in Figure 2.6. The rock units that are predominantly composed of sedimentary rocks are deposited contemporaneously with the Miocene volcanics and display intertonguing with the mentioned units (Akyürek et al., 1996). The sedimentation was accompanied by calc-alkaline volcanic activity (Karayigit, 1983; Pasquare et al., 1988 as quoted by Koçyiğit, 1991).

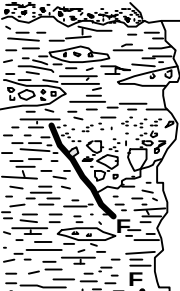
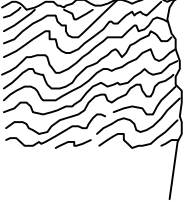
AGE	UNIT	THICKNESS (m)	LITHOLOGY	DESCRIPTION	DEPOSITIONAL SETTING
Quaternary				Alluvial deposits	
Neotectonic					
	YALINCAK FORMATION (Late Pliocene to Pleistocene)				
		75-100		Red siltstone, mudstone and shale alternation with carbonate concentration Growth fault Scour and fill or channel are common features	Flood-plain
		5-10		Wedge to trough cross-bedded conglomerate and sandstone	Braided-plain
		2-15		Debris flow conglomerate with carbonate concretion	
		1-3		White porous limestone	Alluvial fan
		1-35		Debris flow conglomerate	
Paleotectonic	BASEMENT			Pre-Late Miocene Basement Rocks	

Figure 2.3. Generalized stratigraphic columnar section of the Ankara basin (after Koçyiğit and Türkmenoğlu, 1991)



Figure 2.4. View of the Upper Jurassic-Lower Cretaceous limestone outcrops in the study area (southern side of the TCDD Long Rail Welding Factory, Güvercinlik Region).



Figure 2.5. View of an Upper Miocene fluvial-lacustrine sedimentary rock (Saraycık County from the southern Sincan) consisting of continental sedimentary facies (Hançili Formation; sandstone, siltstone, marl, clayey limestone tuff, gypsum and bituminous shale alternations).



Figure 2.6. View of the Lower Pliocene volcanics (Bozdağ Basalt) that crop out in the study area (northern side of the TRT Low-frequency radio station, Batıkent region).

2.3. Neotectonic Units

Most geological study, however, are compiled for purposes other than estimating sediment characteristics and soil behavior in an account for hazard assessments (Borchert, 1997). For example, most geological maps, particularly for regional mapping purposes, differentiate rock units in considerable detail, but only simply differentiate young, unconsolidated sedimentary deposits of most concern for estimates of sediment characteristics and soil behavior. Therefore, in this part of the section, geologic units are especially defined and mapped on the basis of geologic and genetic criteria such as geomorphic expression, inferred depositional environment and sediment characteristics.

The geologic stratification of Upper Pliocene to Pleistocene fluvial red clastics (Yalıncağ Formation) and particularly Quaternary alluvial and terrace deposits are of outstanding importance for this study since their geological, geomorphological and various other factors influence the physical properties of soils they have formed (i.e., sediment characteristics and soil behavior for hazard

assessments). Additionally, the sedimentary units are formed in and near the fault-bounded depression as a result of fault-controlled continental sedimentation in the Ankara basin (i.e., they are deposited under the control of the Neotectonic regime). Regarding the aim of this study, basin fill types of these sedimentary units which are located towards the west of Ankara will be emphasized in more detail in regards to their geological, geomorphological and neotectonic characteristics.

The basin fill types of Upper Pliocene to Pleistocene sedimentary units of about 200 km² area are widely exposed and cover the major part of the study area that is situated in the western part of the Ankara basin. These fluvial sediments showing a continental origin have accumulated in and near fault-bounded basins around the Sincan, Etimesgut, Batıkent, Demetevler, Yenimahalle, Atatürk Orman Çiftliği (AOÇ), Bahçelievler, Anıttepe, Emek, Yenışehir, METU, Balgat, Yüzüncüyıl, Beytepe counties and regions of the study area (Figure.2.2).

2.3.1. Upper Pliocene to Pleistocene Fluvial Red Clastics

Upper Pliocene to Pleistocene fluvial sedimentary units that are so-called “Ankara Clay Formation” generally reflect the source of their basement type. As a result of denudational and depositional process, decomposed and disintegrated parent rock units having different compositions and origin, and formed in different locations from surrounding environments were transported and then deposited into this widely exposed fluvial sedimentary basin. In addition, fine lacustrine interlayers are also encountered within these units (Erol, 1973; Erol et al., 1980). Calcareous concretions occur near the surface of the fault-bounded basins due to surface drying and wetting activities. From this point of view, these deposits possess highly heterogeneous structure and appearance, which contain various sizes of silt, sand and gravel particles in the forms of layers and lenses, in respect to their rock pieces, grain size distributions and colors. For this reason, the sediment characteristics and their physical appearances change from one location to another. For example, they vary from yellowish grey to brown in color (Kasapoğlu, 1980) and possess arenaceous or sandy appearance in the vicinity of graywacke units

(Erol et al, 1980); dark red in color and earthy or soil like appearance in the vicinity of limestones units; and pinkish purple in color and pebbly or gravelly appearance in the vicinity of volcanic units (Erol et al, 1980 and Kasapoğlu, 1980). The surfaces of the fragments are polished and glossy in nature (Ordemir et al., 1965).

Stratigraphic characteristics of the Upper Pliocene to Pleistocene fluvial sedimentary units (Yalıncağ formation) consist mainly of the unsorted loose debris flow conglomerate, braid plain conglomerate to sandstone and clay-bearing finer clastics of flood plain origin from bottom to top (Koçyiğit and Türkmenoğlu, 1991) (Figure 2.3). In general, the grain size of the basin fill becomes finer in the direction from south to north towards the center of deposition. These sedimentary units lie unconformably on the irregular erosional surface of the highly deformed and steeply dipping older basement rocks, and are overlain by Quaternary alluvial and terrace deposits or, are rarely thrust over by older rocks (Koçyiğit, 1991).

The lowermost lithofacies of these units is basal conglomerate. It is an unsorted debris flow conglomerate composed of subrounded to angular pebbles of dissimilar origin, age and facies, which are set in a finer sandy matrix depending on the type of the basement rocks. On the top of the basal conglomerate, lensoidal and porous caliche-like carbonate occurs. It grades upward into a second level of debris flow conglomerate with rare carbonate concretion. Debris flow conglomerates are succeeded conformably by gray-yellow-reddish wedge to trough cross-bedded conglomerate and sandstone package (Figure 2.3). The upper half of the units consists of finer clastics of flood plain origin. It begins on the top of the underlying trough cross-bedded conglomerates and continues upward with the red shale, siltstone and clay-bearing mudstone alternation (Figure 2.7). This level is dominated markedly by the white carbonate concretions to laminations, lensoidal channel conglomerates and normal to reverse type of growth faults (Koçyiğit and Türkmenoğlu, 1991) (Figure 2.3).

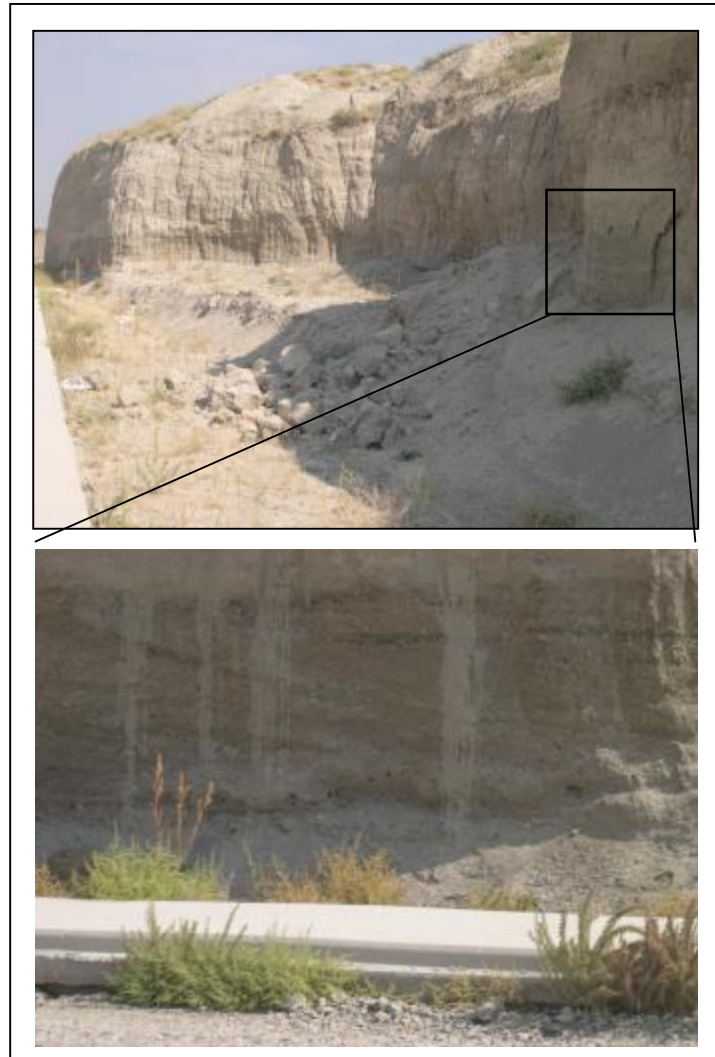


Figure 2.7. View of the Upper Pliocene to Pleistocene fluvial sedimentary deposits that overly the trough cross-bedded conglomerates (lower figure) and continues upward with the red shale, siltstone and clay-bearing mudstone alternation (type locality at the western side of the Ring Road in Elvankent County, Etimesgut).

In summary, the sedimentary clastics filling the Ankara basin were deposited in three different depositional settings of a fluvial system; these are the alluvial fan, braid plain and flood plain. Red clastics filling the Ankara basin are fluvial in nature and are mostly horizontal or gently tilted. The distinctive facies of these sedimentary clastics are the growth-faulted, lensoidal stream channel

conglomerates that strongly suggest that the Upper Pliocene to Pleistocene fluvial clastics were also deposited in a tectonically active depositional regime. Their thicknesses range from a few meters to 200 m based on their stratigraphic position (Erol, 1954 and Tabban, 1976).

2.3.2. Quaternary alluvial and terrace deposits

In the study area, Quaternary alluvial fill and terrace sediments were deposited by flood waters of a 105 km² area in the fault-bounded Ankara basin throughout the flood plains of east-west direction of Ankara River and its tributaries namely Büyükçayır, Pınarbaşı, Beşpınar, Kutuğun, Ergazi, Macun, Altıncioğlu, Yalınca közü, Kara and İncesu Brooks from east to west (Figures 2.2 and 2.8).



Figure 2.8. View of different sections from recent courses of stream beds of the Ankara River (a- AOÇ section in Yenimahalle and b- Ergazi section in Etimesgut).

Basin fill types of recent flood plain sediments and their terraces, and the remaining part of older Quaternary alluvium have generally been classified according to their geological and geomorphological characteristics. Although, there is not enough evidence to assess the stratigraphic age of older alluvial sediments

(particularly coarser terrace deposits present at the margin of the flood water plain), Quaternary alluvial fills are considered as recent and terraces as older sediments on the basis of their morphology, even though they might be confused with Upper Pliocene sediments. Generally, the terrace deposits which are mostly formed by erosional setting are separated with a thin gravelly layer overlying older basement units. When these units comprise coarse grained Upper Pliocene sediments (i.e., pebbly, gravelly, etc.), it is difficult to differentiate the gravelly Quaternary sediments from the Upper Pliocene sediments, especially at higher terraces because of insufficient Quaternary sediment accumulations. As a result, some of the coarser terrace sediments that are the remnants of the Upper Pliocene deposits are very similar to the Quaternary sediments. However, at lower terraces, there are old alluviums of the Quaternary which are different from the grey colored silts and gravels of the Upper Pliocene. These terraces might be considered as Quaternary and also described as old alluvium. These points have also been mentioned by several other researchers (Chaput, 1931 and 1936; Ilgüz, 1940; Pfannenstiel, 1941; Erol, 1954; 1973; 1993; Erol et al., 1980 and Koçyiğit, 1991). Furthermore, regarding the depositional setting of the fluvial clastics, larger sediment particles have naturally settled near the edges of the basin, though some thin sand and gravel layers and lenses are encountered in clay deposits in the middle of the flood plain due to sediments which have been disorderly and irregularly deposited by flood waters. Due to these reasons, the stratigraphic characters of the Quaternary sediments (especially Early Quaternary) as well as the Upper Pliocene sediments could not be completely differentiated to date, and then defined as a single geologic unit on the geological mapping of Ankara (Figure 2.2). Therefore, further investigation of such deposits should be performed through correlation with various other studies (i.e., engineering geological, seismic studies, age dating, paleontology and paleoclimate, etc.) as well as the geologic and geomorphological investigations to stress the importance of the study.

As will be explained in more detail below, Quaternary deposits are differentiated as older terrace deposits (Pleistocene) that are present at the margins and younger alluvium (Holocene) that is present at the stream beds of the Ankara Basin (Erol, 1954 and 1993; Erol et al., 1980; Koçyiğit, 1991).

2.3.2.1. Quaternary terrace deposits

The outcrop of these deposits are well-exposed at different elevations as several step-like river terraces along the margins of fault-bounded depressions of the Ankara basin and mark an active tectonic (relative) uplift in the Ankara region (Koçyiğit, 1991) (Figure 2.2). These were under the influence of the periodic pluvial fluctuations (climatic changes) and continuous subsidence at the central parts of the basins (Erol, 1961 and 1980). The sediment characteristics and physical properties of terrace deposits are not homogenous as like Upper Pliocene deposits because of their heterogeneous content including different sizes of silty, sandy and gravely particles in the forms of layers and lenses (Figure.2.3). Therefore, these deposits, especially at higher terraces, which are unclear or uncertain, are confused with the Upper Pliocene formations along the fault-bounded margins of the basin. Whereas at lower terraces these old alluviums are separated from the grey colored silts and gravels of the Upper Pliocene deposits and consented as Quaternary. As far as that were mentioned at previously above, step-like terraces and their coarser sediments might be arranged in order geomorphologically according to their different elevations that are relatively higher elevated in surrounding environment of the younger alluvium deposits. These are Early Pleistocene, Middle Pleistocene, Upper Pleistocene terraces and their coarser sediments that are from 90 to 70 or more, 50 m, and 20 m or less, respectively (Chaput, 1936; Erol, 1954; 1966 and Erol et al., 1980). Briefly, it can be concluded that the lowest terraces that are grey colored without carbonate concretions of gravels and different from the Upper Pliocene deposits be taken as Quaternary or even end of Quaternary [Upper Pleistocene: started at 126.000 years ago, International Commission on Stratigraphy (ICS), 2004]. Nevertheless, stratigraphic and geomorphologic determinations of these deposits still have to be strengthened with the other studies that were given above regarding the importance or detail of the study on terrace deposits. The estimated thickness of these sediments generally varies from 5 to 10 m in the study area (DSİ, 1975 and This study).

2.3.2.2. Quaternary alluvial deposits

The Quaternary alluvial sediments that are rather thick and Holocene in age, has been deposited by flood waters along the both side of the recent river beds. They disconformably cover the sediments on the pre-Quaternary rocks. These sediments have not been in place long enough to show any appreciable effect of soil forming factors. There is no suspect that they belong to the very last period of the Quaternary because the rivers of our time are still depositing this material along their valleys at present (Figure.2.2). The general view of the Quaternary alluvial deposits from the different sections in Ankara basin was shown in Figure 2.9.

These are normally consolidated soft deposits and their parent materials are varied like Upper Pliocene to Pleistocene deposits, but relatively more homogeneous, depending on the nature of the rock and soils in the areas drained by the streams (Sürgeç, 1976, as reported by Lohnes, 1974). Groundwater level in the inner basin is close to the surface in general, though it varies within the alluvium, depending upon the soil characteristics of depositional environment and proximity of the major course of the recent stream beds. It is gradually deeper beneath the terrace and older deposits bordering the basin. It ranges between 2.3 to 6 m (DSİ, 1975) and 5.5 m (by this study) as an average, respectively. Generally, the thickness and width of the recent alluvial deposits observed along the Ankara River and their major tributaries relatively range between 5 m and 45 m and from 0.2 to 3 km, respectively (İlgüz, 1940; Erol, 1973; DSİ, 1975; Tabban, 1976, Kasapoğlu, 1980 and This Study).



Figure 2.9. General view of the Quaternary alluvial deposits from different sections of the Ankara basin (a- Ergazi-Şaşmaz Section in Etimesgut; b-Çayyolu Section; c- Atatürk Conventional Center-Hipodrom, Dışkapı Section).

These relatively thick recent deposits consist of both coarse-grained marginal and fine-grained axial depositional systems (Figure 2.3). The coarse-grained depositional system is composed of terrace and alluvial fan sediments deposited by debris flows and braided rivers that comprise unsorted, loose and different size of sediments such as gravel, sand and silt. They are the lowest and the

youngest terraces in the fluvial basins and correlated to the younger alluviums (Erol, 1980). The axial depositional system comprises fine-grained alluvial plain sediments such as sand, silt, and clay (Koçyiğit, 1991) towards to the recently deposited stream beds. In addition, black to dark coloured and organic material rich silty, clayey, and muddy sediments of swamp origin are also encountered in or nearby recent stream beds, and generally have a thickness of 4 to 5 m in the study area [i.e., Etimesgut Sugarcane Factory property, AOÇ property, old Hipodrum location (Figure 2.9c), etc.] (Chaput, 1936).

2.4. Paleogeography

The Triassic schists of the basement and the overlying greywackes crop out especially at the southern and southeastern rim of the Ankara basin. It is possible that they have been yielded by the erosion of a formation lying above the schists at the Northwest of the basin. At the beginning of Jurassic, some synclines were formed around Ankara Anticline (Erol, 1954 and 1968).

A reddish flysch has developed above the basement which has been uplifted as a result of Liassic movements. This can be observed around Yakacık and Balkuyumcu and at northern parts of Bağlum. Marls and limestones have accumulated in depressions formed on the rocks of the basement (Ketin, 1966). The basement has been drawnd seas at the Middle and the Upper Jurassic (Erol, 1954).

Deep sea formations observed near Ankara have developed between Middle Jurassic and Middle Cretaceous crop out within the Karyağdı Mountain and Haymana. Ankara Anticline has emerged and has supplied materials in Upper Cretaceous and then the sea became deeper, as a result of the transgression that has lasted until Lutetian (Erol, 1954).

In Miocene times, lakes have developed on eroded surfaces and volcanic lavas and tuffs extending north and northwest side of the basin have been introduced into lacustrine sediments. The continental formations Elma, İdris, Mire Mountains at the East and Northeast side of the Ankara Basin have been eroded and the products of erosion have accumulated at the slopes of these mountains and

in these lakes (Erol, 1954 and 1973). At the end of the Miocene, northern parts of the region have been uplifted again and Early Pliocene (Pontian) river sediments have accumulated around the eroded mountains. Nevertheless, several lakes survived in southern parts but have finally disappeared as the areas have been affected by the last Epirogenic movements in the Middle Pliocene (Ordemir et al., 1977) and then lakes have been filled by fluvial sediments in the Upper Pliocene that contain sand, silt, gravel and clay size particles transported from the surrounding old formations by flood waters, mostly andesites, graywackes and limestones.

There are different views regarding the sedimentary formations of the Ankara basin during the Neotectonic period whether they are depositional or erosional.

Considering the first view, erosional setting might have occurred and started at the end of Upper Pliocene and continued in the Early Quaternary-Pleistocene. Erol (1954), (1962) and (1966), have taken into account the average elevation of the surface basin sediments which are 850 m at the present time and argued that the Ankara basins were alluviated up to elevations 1000 to 1050 meters and subsequently uplifted and eroded several times which resulted in a desiccation and preconsolidation pressure that is equivalent to between 150 and 180 meters thick sediment. Additionally, recent surface drying and wetting activities have formed the most recent preloading processes leading to the occurrence of the calcareous concretions near the surface since the beginning of the Quaternary (Kasapoğlu, 1980).

However, according to the view of others, the existence of this source of overburden pressure has not been well supported by sound geological evidence or reasoning (Lohnes 1974, as reported by Sürgel, 1976). They support that the Pleistocene terraces are the result of a cut and fill sequence and as such their present surfaces are depositional rather than erosional and that the soils which lie beneath these surfaces have not experienced any overburden pressure greater than that which exists at present (Sürgel, 1976). Therefore, it can be pointed out that the alluvium deposited by the flood waters has not been in place long enough to show

any appreciable effect of soil forming factors as such the recently developed fluvial sediments that are formed in Ankara basin due to their depositional character.

In this dissertation, the different views regarding whether depositional or erosional character and process of the Ankara basin will also be examined in detail by using various supportive studies, such as engineering geological, geotechnical and geophysical studies to interpret the sediment characteristics and soil behavior of these fluvial deposits of the Ankara basin in an account for seismic hazard assessments.

2.5. Structural Geology of the Ankara Region

The study area is situated within the Ankara basin located towards the west of Ankara that cut and cross by the east-west direction of the streams delineated between the highlands and plateaus. The area mostly constitutes fluvial and alluvial deposits that are formed in and near fault-bounded depression as a result of fault-controlled continental sedimentation. As a general, the study area is in Ankara that is N-S trending, about 150 km wide and 250 km long, includes numerous earthquake centers related to the intermediate areas between the northern and southern Anatolian ranges (Lahn, 1949 and Koçyiğit, 1991) and to the faults which form the boundaries between these areas and the ranges. It lies between the North Anatolian Fault System (NAFS) to the north and northwestern part of Anatolia to the South. It is a broad and structurally triangular area outlined by the Kesikköprü and the Seyfe Fault Zone in the east, the Salt Lake Fault Zone in the southeast, the İnönü-Eskişehir Fault Zone in the west-south-west, and the North Anatolian Fault System in the north (Figure 2.10).

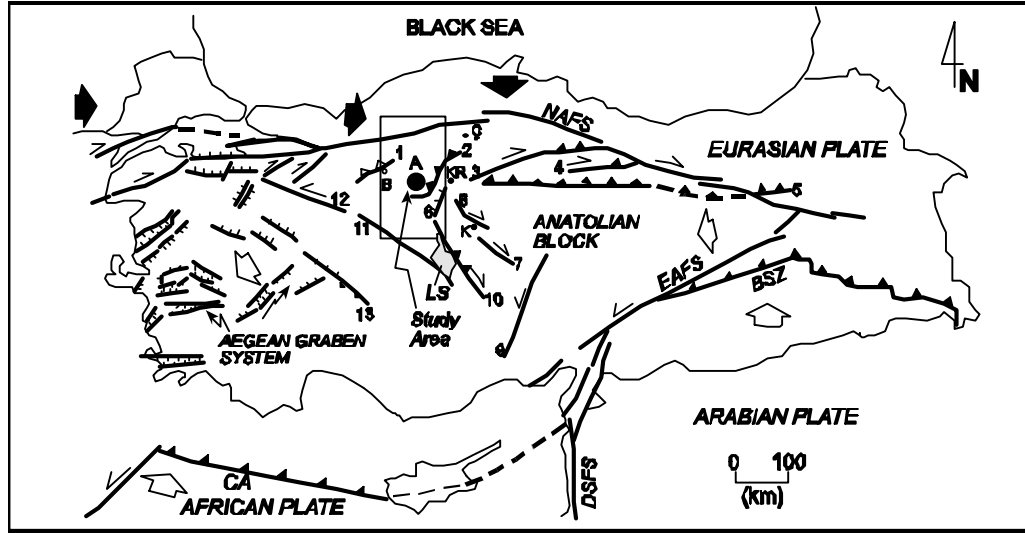


Figure 2.10. Simplified neotectonic map showing the general outline of the study area and some of the major neotectonic structures of Turkey. A-Ankara; B-Beypazarı; LS-Salt Lake; BSZ-Bitlis Suture Zone; CA-Cyprian arc; DSFS-Dead Sea Fault System; EAFS-East Anatolian Fault System; NAFS-North Anatolian Fault System; 1-Beypazarı-Çayırhan faulted monocline; 2-Elmadağ imbricate thrust zone; 3-Kırıkkale-Erbaa fault; 4-Almus Fault; 5-Ankara-Erzincan Suture; 6-Seyfe Fault Zone; 7-Salanda Fault Zone; 8- Kesikköprü Fault Zone; 9-Ecemiş Fault Zone; 10-Salt Lake Fault Zone; 11-Eskişehir Fault Zone; 12-İnegöl fault; 13-Akşehir-Simav Fault Zone. Note that the black arrows show the orientations of the maximum compressive stresses along the NAFS; white arrows show the sense of the plate motions and half arrows show the relative sense of movements on the faults (after Koçyiğit, 1991).

Based on the nature of tectonic regime, the exposed rocks units in the study area can be differentiated as paleotectonic and neotectonic rock units. Regarding the orogenic phases of the Ankara Region, Pre-Alpine and Alpine Orogenic Movements had been encountered in the paleotectonic regime from Visean (Middle Mississippian in Carboniferous) to Oligocene (Erol, 1954 and 1961). After that, the Alpine orogenic movements have gradually started to weaken in the Middle Pliocene during the epirogenic movements and then Neotectonic period have been started at the end of Miocene. During the neotectonic regime, Attic, Rhadonic and post-Villafranchian Orogenic Movements (end of Upper Pleistocene) have been experienced since the Upper Miocene (Erol, 1961 and 1980). Quaternary is

represented by the alluvial fill of the depressions which were formed by rejuvenation at the Upper Pliocene that are a proof of residual crustal deformations inherited from Alpine Orogeny (Lahn, 1949; Erol, 1956).

The Ankara Region is located at the junction of the Intra-Pontide, Izmir-Ankara and Intra-Tauride sutures (Şengör and Yılmaz, 1981; Şengör, 1984). Therefore, the Ankara region displays a very complicated structural pattern, in which strike-slip faults with both thrust and dip-slip components, and open to south-southeastward slightly overturned folds of the neotectonic regime, are superimposed on the tight to overturned fold-and low-angle thrust-dominated contractional structures of the earlier collisional tectonic regime (Koçyiğit, 1991) (Figure 2.10). The fault characteristics of the Ankara represent a intermediate zone of deformation in between the right lateral strike-slip fault systems namely North Anatolian Fault System in the north and the oblique-slip normal fault systems in the south (i.e., Salt Lake Fault Zone, Seyfe Fault Zone, etc.) that are characterizing a transition of compressional and extensional neotectonic regime from north to south (Koçyiğit, 2003).

In the Ankara Region, the geological structures that are folds, normal faults, low-to high-angle thrust faults, strike-slip faults, and fault-parallel depressions are well exposed (Figure 2.11). Based upon their age and origin, these structures are classified into pre-Upper Pliocene structures and Upper Pliocene-Quaternary structures. The first group is dominated by N-trending thrust faults, southeast-vergent, thrust faulted monoclines, NE-trending strike-slip fault with thrust component, approximately E-W trending strike-slip faults, and NE-trending folds. Though the first group of structures was inherited from the last phase of the former collisional tectonic regime, these structures were reactivated during the neotectonic regime that has operated since the Upper Miocene. Consequently, the first group of structures indicates that a NW-SE directed contraction prevailed before Upper Pliocene in the Ankara region. The second group of structures comprises the NNE-trending normal faults, NE- and NW-trending sinistral and dextral strike-slip faults, oblique-slip faults, ENE-trending thrust faults with strike-slip component and folds. These Upper Pliocene-Quaternary structures collectively form a well-developed strike-slip fault system that suggests an approximately N-S directed contraction

continuing since Upper Pliocene in the Ankara Region. Thus, these two groups of structures reveal that the contractional stress orientation in the progressive intercontinental deformation in the Ankara region has changed from NW-SE to N-S direction during the neotectonic period since Pliocene (Koçyiğit, 1991).

In the Ankara Region, the seismic events that are associated with the faults in the Ankara basin are seismically active but have rather short extent. Hence, these are capable of producing frequently occurring small earthquakes with magnitudes of 5.0 or less (i.e., they are incapable of producing large destructive seismic events in Ankara). These faults are particularly southeastern segment of the İnönü-Eskişehir Fault Zone (namely Ilıca Fault Zone), Çeltikçi Fault Zone, Kesikköprü Fault Zone, Çubuk Fault Zone, Balaban Fault Zone, Kızılırmak Fault Zone and Kalecik Fault, etc. As a regional scale, whereas, the Ankara Region might be affected from surrounding of large-scale Fault Systems and Fault Zones, especially North Anatolian Fault Systems, the Salt Lake Fault Zone and Seyfe Fault Zone that have capability to produce large destructive earthquakes ($M > 6.0$) and have to be considered as well. The sources of historical earthquakes and recent examples indicate that the significant seismic events had taken place along these fault systems that have affected Ankara and its surroundings seriously. In this respect, these larger Fault Systems and Fault Zones will be briefly mentioned here as a neotectonic point of view. Later, seismicity and seismic source characteristics of these Fault Systems and Zones will be explained in detail for assessment of earthquake hazard. Interpretation of both the neotectonic and the seismologic character of a region provide the essential information towards the assessment of seismic sources. The correlation of seismicity with the neotectonic elements constitutes an important phase of the earthquake hazard assessments (Erdik et al., 1996).

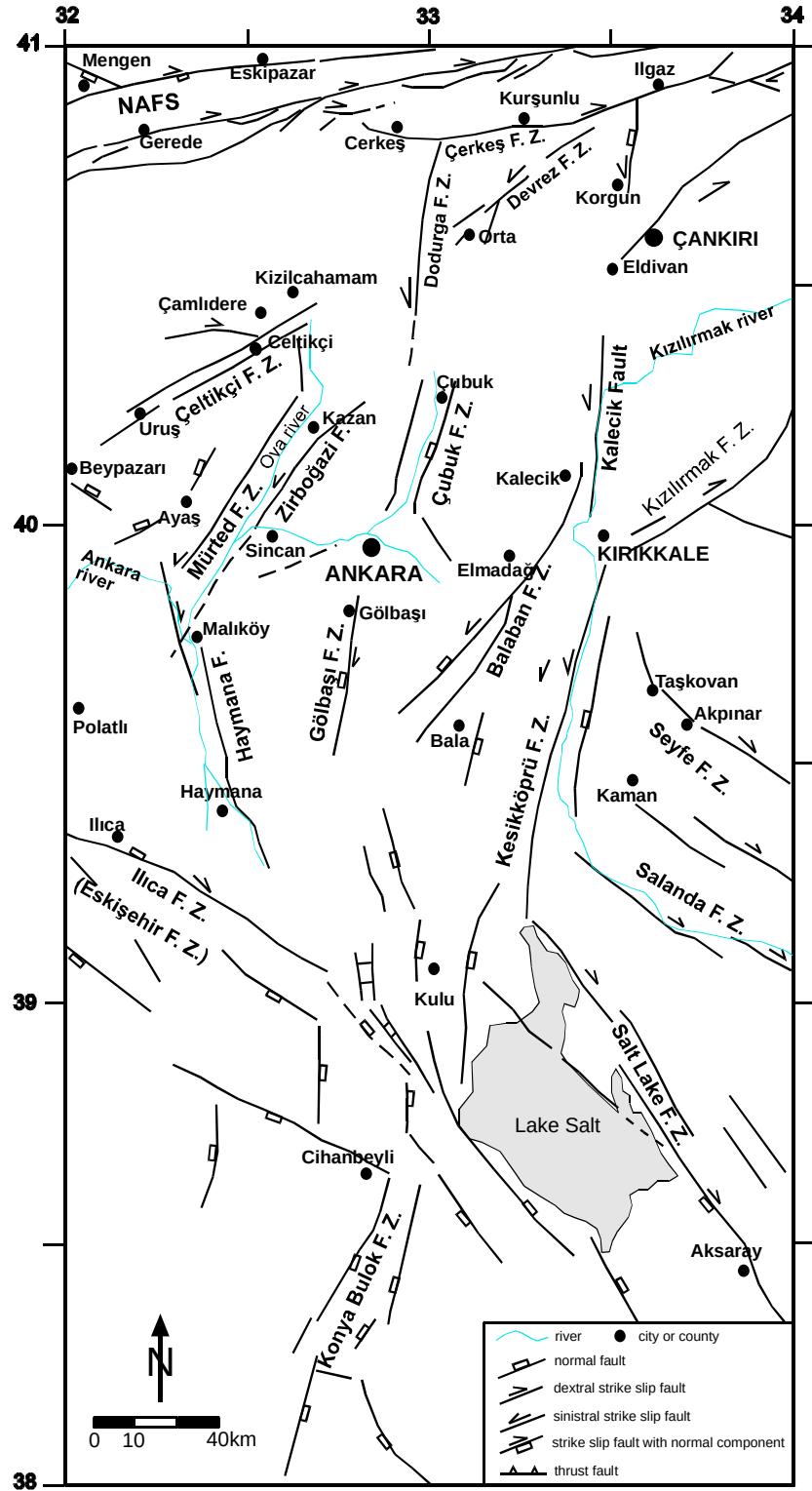


Figure 2.11. Simplified seismotectonic map of the Ankara region and its vicinity (compiled from Koçyiğit, 1991 and 2003; Koçyiğit et al., 2001).

The details of their structural geologic and neotectonic character about these faults are given in Figure 2.10 and 2.11. A discussion of the Fault Systems and Fault Zones follows.

North Anatolian Fault System (NAFS) is an intercontinental transform fault boundary between the Eurasian plate in the north and the Anatolian block in the south. NAFS is a morphologically distinct and seismically active right lateral strike slip fault where countless damaging earthquakes have occurred throughout history which extends for about 1200 km from Karliova to the Gulf of Saros along the Black Sea Mountains of North Anatolia. It takes up the relative motion between the Black Sea and the Anatolian plates, thereby connecting the E. Anatolian convergent zone with the Hellenic Trench through the complex plate boundary zone of the Aegean (Şengör, 1979). The width of the dextral shear zone ranges between a few kilometers and a hundred kilometer. Regarding the faulting associated with historical and recent earthquakes, the transform fault system has periods of seismic activity from historical times (A.D. 29) to present, is characterized by frequently $6 \leq M \leq 8$ earthquakes (Ergin et al., 1967; Ambraseys and Finkel, 1995). Two large destructive events ($M=7.3$) have been occurred on 26.11.1943 and 1.02.1944 along the NAFS, are the major examples that indicate the seismic activity of the central segment of this Fault System (Dewey, 1976).

The NAFS displays two common distribution patterns or geometries along its length: (1) splay-type geometry, and (2) anastomosing-type geometry. The splay type is well developed in both the Erzincan-Çerkeş and the Marmara sections of the NAFS. In the area between Erzincan in the east and Çerkeş in the west, a number of fault zones, fault sets and isolated faults of varying sizes branch as splay structures from the master strand of the NAFS. These structures first trend E-W for some distance, and then bend southward and trend approximately NE-NNE, traversing the Anatolian platelet for several hundreds kilometers, cutting across and deforming it. In the second pattern or geometry of the NAFS, the master strand first bifurcates into several subfault zones, fault sets and isolated faults of varying sizes, and then they rejoin and rebifurcate several times, leaving behind a series of lensoidal highlands (pressure ridges) and lowlands (basins) whose long axes parallel the general trend of the master strand of the NAFS. This pattern is the most

diagnostic characteristic of the Kargı-East Marmara section of the NAFS. Examples of subfault zones, fault sets and isolated faults having anastomosing geometry along the NAFS in its Kargı-East Marmara section are, from E to W, the Ulusu-Gerede-Abant, Tosya, Çerkeş-Kurşunlu, Karadere-Kaynaşlı-Mengen-Eskipazar and the Hendek-Yığılca fault zones (Koçyiğit et al., 1999 and 2001).

Regarding the central part of the NAFS that was particularly considered related with the seismotectonics of Ankara and its surroundings, it bends towards the north at its central part and results in a northward convex arc. In addition, the NAFS has also a thrust component at the center of its northward convex arc, even though it is not recorded seismically (Jackson and Mc Kenzie, 1984) (Figure 2.10). Both the Gerede and İsmetpaşa segments of the NAFS consist of numerous parallel and subparallel faults, namely Devrez, Çerkeş-Kurşunlu, and Dodurga fault zone (06.06.2000 Orta Earthquake of M=5.9 occurred along this fault zone). The Gerede segment of NAFS is marked by a series of fault-parallel sag ponds, pressure ridges and strike-slip basins such as the Yeniçağa fault wedge and Dörtdivan pull-apart basins. In contrast, the İsmetpaşa segment of the NAFZ is dominated by a half-ramp basin, the Çerkeş basin located on its southern block (Koçyiğit, 1991) (Figures 2.10 and 2.11).

Salt Lake Fault Zone (SLFZ) occurs in the southern and southeastern parts of the study area. It is an approximately 100-150 km long and 15-20 km wide intracontinental transcurrent fault zone consisting of several parallel to subparallel fault segments. The Salt Lake Fault Zone displays a step-like fault pattern and also has a thrust or compressional component in places (Şengör et al., 1985). The Salt Lake Fault defines the eastern margin of Salt Lake along which lake terrace deposits of Pliocene-Quaternary age are located at different elevations, suggesting that the bottom of Salt Lake is subsiding (Erol, 1973, 1980). Consequently, the dextral offset drainage system, fault-parallel alignment of hot water springs (such as those at Ayaş, Malıköy, and Haymana) and the seismic activity is characterized by 28.06.1933 and 21.04.1983 M= 4.7 earthquakes at the NW tip of the Salt Lake Fault Zone indicate that the fault set is active (Koçyiğit, 1991; KOERI, 2005).

The Seyfe Fault Zone (SFZ) is about 120-km long and consists a few-km wide right lateral faults with normal components consisting of several parallel to

sub-parallel fault segments from the Kırıkkale at the North-west to Hasanlar village at the South-east. This fault zone controls the southern edge of the Seyfe Lake depression (Koçyiğit, 2003). It consists of both NW-SE and NE-SW trending faults with short lengths that ranges from few hundred meters to 20 km. The Kırşehir earthquake of March 19, 1938 ($M_s=6.8$) with surface faulting occurred on this fault zone indicate that the fault set is active (Arni, 1938). During the earthquake that activated the Akpınar-Taşkovan segment of the Seyfe Fault Zone, approximately 15 km long and few meters to kilometers wide surface faulting zone was developed. Kırşehir earthquake of March 19, 1938 is the biggest earthquake in the Central Anatolia during this century. Briefly, Seyfe Fault Zone is an active and NW-SE tip of the Fault Zone still possesses seismic gap characteristics.

2.6. Geologic and Geomorphologic Significance in Evaluating Earthquake Hazards

The properties of the geologic materials beneath a site can have a major impact on seismic events. In particularly, geologic conditions in Quaternary sediments vary across a region and commonly influence ground response during earthquakes. The damage is often significantly greater on unconsolidated soils than on rock sites when structure is more than a few or even hundred kilometers from the earthquake source (i.e., 1985 Mexico City, 1989 Loma Prieta, and 1999 Kocaeli Earthquakes). Delineating surficial geologic units in order to evaluate areal differences in seismic response from future earthquakes requires knowledge of the surface and subsurface distribution of Quaternary sedimentary deposits in relation to their wave-propagation characteristics. Consideration of these site conditions has become an important part of assessment of seismic hazards.

The geology and geologic mapping commonly emphasize the detailed distribution and character of bedrock units, including lithology, age, and rock structure (i.e., beddings, lineations, folds, and faults). In contrast, areas underlain by relatively uncemented or loose alluvium or other sediment without apparent structural or stratigraphic character commonly are depicted as a single map unit.

Variations in the physical properties of neotectonic units, especially Quaternary alluvial deposits that pertain to the delineation of geologic hazards such as ground shaking or ground failure are not usually distinguished on standard geologic maps (Tinsley and Fumal, 1985). Because urban areas at risk from earthquakes commonly are located on Quaternary alluvial deposits, conventional geologic maps usually are not adequate to obtain physical properties of sediment characteristics and soil behavior for evaluating earthquake hazards. Hence, the geomorphological land classification might be appropriate site classification schemes for evaluating ground motion hazards (Midorikawa et al., 1994). It is also known that the geomorphological land classification correlates with liquefaction susceptibility (e.g. Kotoda et al., 1988).

For this purpose, Quaternary alluvial and terrace deposits of the Ankara basin have been extensively discussed in this chapter in regards to their geological, geomorphological and neotectonic characteristics up to present. Additionally, during the past few decades, other specialized techniques directed specifically at identifying and evaluating earthquake hazards in alluvial deposits have evolved that were mentioned briefly and were put in to practice by Borchardt and others (1975); Lajoie and Helley (1975); Tinsley and Fumal (1985); Fumal and Tinsley (1985); Matsuoka and Midorikawa (1995); Petersen et al. (1997); Park and Elrick (1998); Wills and Silva (1998); Wills et al. (2000) and Holzer et al. (2002). In this respect, the principal sources of these techniques including different maps, descriptive accounts of geological studies (including boring logs), penetration resistance from engineering geological borings, and geophysical methods (i.e., mapping shear-wave velocities, electrical resistance of the units) will be examined in the following chapters.

CHAPTER 3

ASSESSMENT OF EARTHQUAKE HAZARDS IN THE ANKARA REGION

3.1. Introduction

Earthquakes occur in a sudden, repeated, and catastrophic manner which can result in damage to property and in loss of life. These are continuous reminders of the need to further study these natural phenomena that are difficult to predict. This inability to predict the occurrence of such events has led engineers and planners to focus significant efforts on ways to mitigate their potential consequences. Seismologists and geologists have dedicated much of their effort in trying to understand how, where, and when earthquakes will occur. On the other hand, geotechnical engineers and engineering geologists look at earthquakes as a problem to be accounted for in design. The occurrence of earthquakes cannot be predicted with the desired certainty; therefore it is important to develop an understanding of the resultant geotechnical hazards particularly in densely populated areas that threaten safety (Kremidjian, 1992).

A seismic hazard analysis takes into account the history of seismic events, epicentral distance, attenuation relationships, location and type of source (fault or background), and the location of the site in question. This type of analysis provides results of the probability of an earthquake (magnitude and acceleration) occurring at a site. Although, the amplification of ground motion due to the local (site specific) site conditions plays an important role in increasing seismic damage, seismicity and source characteristics are a crucial precursor to any earthquake evaluation (i.e., geotechnical earthquake hazards) that has a direct physical impact on man made structures and people.

3.2. Earthquake Hazards

The geotechnical earthquake hazard occurs at the end of a series of seismic related steps which start at the source and end where the damage occurs, i.e., at the site. The three basic steps in a regional seismic hazard analysis procedure particularly include: (1) identification of earthquake sources and regional seismicity, (2) estimating the attenuation of earthquake motions between the sources and the region, and (3) evaluating the local site effect on ground motion. These are typical key parameters underlying these effects and how they relate to the process of seismic zonation of ground motions (TC4, ISSMFE, 1999).

The goal of a regional seismic hazard analysis is to quantify the potential damages and losses in a region due to future earthquakes. This analysis requires a synthesis of the information given above and an evaluation of the seismicity, seismic source characteristics and attenuation of ground motion intensity of the Ankara region for the assessment of earthquake hazards. Interpretations of these characteristics of a region provide the essential information towards the evaluation of local site conditions on ground motion that constitutes an important phase of the earthquake hazard assessments.

3.2.1. Assessment of Earthquake Sources and Regional Seismicity

Description of the seismicity and the neotectonic regime of a region provide the essential information towards the assessment of seismic source zones. Almost all earthquakes of the Central Anatolia and its vicinity are associated with tectonic elements. The correlation of seismicity with the tectonic elements (seismo-tectonics) constitutes an initial phase of earthquake hazard assessment.

As a general, the Ankara region includes numerous earthquake centers related to the intermediate areas between the northern and southern Anatolian ranges and to the faults which form the boundaries between these areas and the ranges. The region is particularly surrounded by these seismically active Fault

Systems (i.e., North Anatolian Fault System) and Fault Zones (i.e., Salt Lake Fault Zone and Seyfe Fault Zone) that characterize a transition of compressional and extensional neotectonic regime. Hence, the significant seismic events have taken place along these large-scale Fault Systems and Fault Zones that might affect Ankara and its surroundings and thus have to be considered seriously for earthquake hazard estimation. The list of these earthquake events that have been occurred along the major fault systems and zones since 1900 with magnitudes greater than 5.0 (except for the Salt Lake Fault Zone; i.e., less than 4.0) is tabulated in Table 3.1. Some of the major events have occurred along these Fault Systems and Fault Zones (i.e., 26.11.43 Bolu and 01.02.44 Gerede earthquakes along the central part of the NAFS, and 19.03.1938 Taşkovana-Akpınar earthquake along the Seyfe Fault Zone) are indicated in Table 3.1 in bold. Additionally, the seismic events that are associated with the faults that are in or close to Ankara are seismically active but have rather short extent considering the surrounding active fault zones. Hence, these are capable of producing frequently occurring small earthquakes with magnitudes of 5.0 or less (i.e., Ilıca Fault Zone, Çeltikçi Fault Zone, Kesikköprü Fault Zone, Kızılırmak Fault Zone and Kalecik Fault, etc.). However, some of the recent seismic activities in this region, namely the 06.06.2000 Orta ($M_w=5.9$ and two aftershocks of $M_w= 5.2$ and 5.0), 30.12.2004 Çubuk ($M_l=4.6$), and finally 31.07.2005- 09.08.2005 series of Bala (particularly $M_l=5.3$, 4.8 and 4.6) earthquakes (obtained from Kandilli Observatory and Earthquake Research Institute, KOERI) indicate that these fault zones are capable of producing relatively higher (moderate) seismic events that might affect Ankara and have to be considered as well. A list of these earthquake events and particularly their recent seismic activities that have occurred in the Ankara region is tabulated in Table 3.2 (the prominent ones are indicated in bold). Finally, the distribution of epicenters for major earthquakes that occurred in the Ankara region and its surroundings since 1900 with magnitudes greater than 4.0 is shown in Figure 3.1 (seismic data were obtained from KOERI, 2005 and Earthquake Research Department under the General Directorate of Disaster Affairs, ERD, 2005).

Table 3.1. A list of the earthquake events that have occurred along the North Anatolian Fault System, Seyfe Fault Zone and Salt Lake Fault Zone since 1900 with magnitudes greater than 5.0 (except for the Salt Lake Fault Zone). Note that seismic data were obtained from KOERI and ERD (2005).

DATE	TIME (GMT)	LATITUDE	LONGITUDE	DEPTH (km)	MAGNITUDE
Along the Central Part of the North Anatolian Fault System					
05.10.1977	05:34	41.02	33.57	10	5.3
10.12.1966	17:08	41.09	33.56	13	5.2
21.09.1957	20:16	40.75	34.02	40	5.1
07.09.1953	03:59	41.09	33.01	40	6.4
14.08.1951	18:46	41.08	33.18	40	5.1
13.08.1951	18:33	40.88	32.87	10	6.9
13.05.1949	20:14	40.74	32.71	20	5.1
19.12.1947	17:31	40.71	32.82	10	5.1
21.01.1946	11:25	41.05	33.48	60	5.0
26.10.1945	13:56	41.54	33.29	50	6.0
02.03.1945	10:39	41.20	33.40	10	5.6
18.10.1944	12:54	40.89	33.47	10	5.2
10.02.1944	12:05	41.00	32.30	10	5.3
01.02.1944	03:22	41.05	32.20	10	7.3
02.01.1944	10:59	41.00	33.70	0	5.0
26.11.1943	22:20	41.05	33.72	10	7.3
18.11.1936	15:50	41.25	33.33	10	5.2
21.09.1936	11:41	41.21	33.53	20	5.1
03.10.1928	00:57	40.47	33.42	70	5.0
09.06.1919	15:47	40.68	33.89	10	5.0
09.06.1919	07:13	41.16	33.20	10	5.7
09.08.1918	00:39	40.89	33.41	10	5.8
25.06.1910	19:26	41.00	34.00	0	6.2
Along The Seyfe Fault Zone					
21.07.1938	21:56	39.56	33.68	10	5.0
19.03.1938	23:11	39.65	33.87	30	5.0
19.03.1938	10:59	39.44	33.79	10	6.6
09.04.1930	05:07	39.70	34.00	0	5.0
Along The Salt Lake Fault Zone					
07.03.2002	06:12	38.31	33.88	5	4.4
03.03.1985	13:02	39.13	33.17	10	4.3
06.04.1985	04:42	39.55	32.93	5	4.4
21.04.1983	16:18	39.31	33.06	36	4.7
27.04.1973	00:31	38.65	32.92	29	4.6
28.06.1933	11:54	39.30	33.20	0	4.7
16.01.1918	16:32	38.80	32.90	0	5.3

Table 3.2. A list of the earthquake events (i.e., İlica Fault Zone, Çeltikçi Fault Zone, Kesikköprü Fault Zone, Kızılırmak Fault Zone and Kalecik Fault, etc.) and particularly their recent seismic activities that have occurred in the Ankara region. Note that seismic data were obtained from KOERI and ERD (2005).

DATE	TIME (GMT)	LATITUDE	LONGITUDE	DEPTH (km)	MAGNITUDE
17.12.2005	00:15	39.40	33.13	5	4.2
09.08.2005	04:28	40.55	32.96	5	4.7
06.08.2005	09:09	39.34	33.10	5	4.6
01.08.2005	13:22	39.44	33.07	17	4.2
01.08.2005	02:02	39.43	33.07	10	4.0
01.08.2005	00:45	39.41	33.06	7	4.6
31.07.2005	23:41	39.45	33.09	5	4.8
31.07.2005	15:23	39.42	33.10	5	4.2
31.07.2005	15:18	39.42	33.10	5	4.3
31.07.2005	03:45	39.42	33.13	6	4.2
31.07.2005	00:45	39.44	33.09	5	5.3
29.12.2004	22:22	40.40	32.96	32	4.6
08.02.2004	09:27	39.20	32.59	5	4.5
07.02.2004	19:26	39.19	32.65	6	4.1
27.02.2002	21:26	39.95	33.49	8	4.1
05.10.2000	08:38	40.42	33.10	13	4.1
22.08.2000	11:40	40.32	32.13	10	4.3
09.06.2000	11:48	40.39	33.02	7	4.0
09.06.2000	03:14	40.63	32.97	20	5.2
08.06.2000	21:27	40.64	33.01	22	5.0
06.06.2000	02:41	40.72	32.87	10	5.9
30.08.1999	06:51	39.31	32.40	2	4.1
24.08.1999	17:33	39.61	32.62	8	4.7
17.03.1999	20:27	40.26	32.14	5	4.1
22.01.1999	01:25	40.03	32.76	8	4.3
04.04.1995	11:23	40.40	32.77	7	4.0
14.10.1989	21:12	39.74	32.83	10	4.0
06.04.1985	04:42	39.55	32.93	5	4.4
21.04.1983	16:18	39.31	33.06	36	4.7
21.01.1983	21:52	39.40	32.30	10	4.5
04.07.1978	22:39	39.45	33.19	23	4.9
22.09.1975	16:31	40.26	33.34	18	4.1
30.07.1975	16:25	39.45	32.13	2	4.5
18.06.1968	10:09	40.00	33.00	33	4.2
03.06.1961	06:16	39.33	32.64	10	4.3
13.08.1951	18:33	40.88	32.87	10	6.9
28.06.1933	11:54	39.30	33.20	0	4.7
04.10.1928	11:14	40.22	33.67	10	5.7

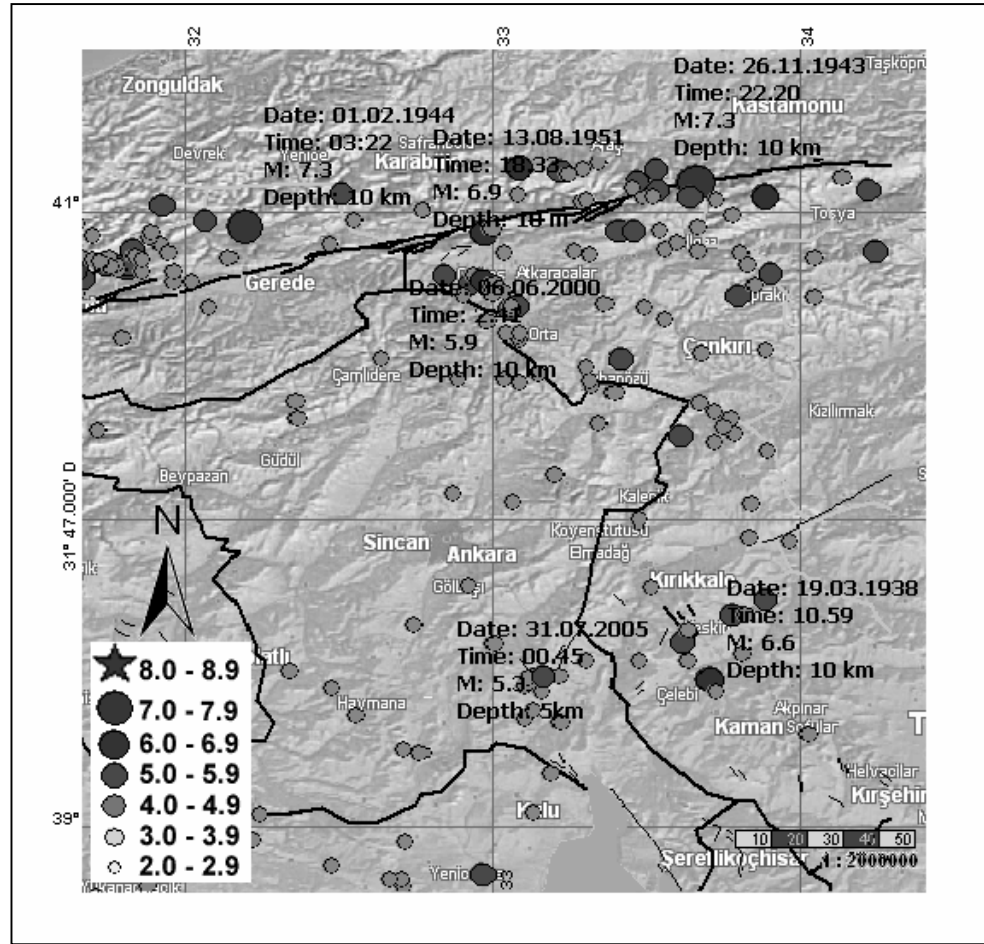


Figure 3.1. The distribution of epicenters for major earthquakes that occurred in the Ankara region and its surroundings since 1900 with magnitudes greater than 4.0 (seismic data were obtained from KOERI, 2005 and ERD, 2005).

3.2.2 Compilation and Interpretation of the Historical (before 1900) and Past Recent (after 1900) Seismic Events along the major earthquake zones.

The majority of the surrounding large-scale Fault Systems and Fault Zones that have a capability to produce moderate to large scale destructive earthquakes in the Ankara region were briefly mentioned in Chapter 2 and in this chapter. Hence, the sources of these recent earthquakes indicate that the significant seismic events that have taken place along these fault systems might affect the Ankara

region and its surroundings. Additionally, for the determination of possible sources of future seismic events of Ankara, the description and evaluation of the source material related to historical and past recent earthquakes, is also very crucial to support the evidences of this hypothesis. In this respect, the historical and past recent database is summarized here from the available and prominent past seismic events.

According to Ambraseys (1970), the NAFS has faced at least four historical fault breaks in the years 967, 1035, 1043 and 1050 and these breaks probably occurred where faulting occurred recently in 1939, 1943 and 1944, respectively on the basis of historical records (Pinar and Lahn 1952). The historical crack growth shows discontinuous eastward migration rather than westward migration which took place between 1939 and 1944 (Kuran, 2005).

One of the most destructive earthquakes, usually referred to as “Bolu-Kastamonu” occurred in 1668 along the NAFS which was felt all over the Anatolia (Engin et al., 1967). It was associated with extensive faulting between Bolu and Kastamonu. Bolu, Gerede, and Kastamonu were severely damaged and the earthquake was felt strongly in Amasya, Niksar, Kayseri and Ankara (Ambraseys and Zapotek, 1968) because a ground rupture of 380 km had developed during this earthquake (Engin et al., 1967). If the 1668 earthquake is comparable to or greater than the 1939 Erzincan earthquake $M=7.9$ (leading to a 360 km long surface rupture), the destruction given by the 1668 event to the structures and the level of casualties would have been much greater (Kuran, 2005). According to Sipahioğlu and Gündoğdu (1982) and Kuran (2005), earthquake activity started on the 3rd of July in this region, and following the main shock, which was felt all over Anatolia, many strong aftershocks took place between July 18 and September 13, 1668.

According to the Calvi, 1940; Soysal et al., 1981; Sipahioğlu and Gündoğdu, 1982; and Ambraseys and Finkel, 1986 and 1995, the damage was caused by a series of earthquakes of intensities VIII-IX occurred in a short period of interval and not by a single shock in the 1668 event. The first shock of 1668 occurred on July 3 and was felt strongly between Bolu and Kastamonu; whereas the second and third shocks of 1668 occurred on August 12 and 15, respectively and caused extensive damage in

Ankara and Beypazarı, known as the Ankara Earthquakes. These historical earthquakes had caused severe damages to the structures and led to loss of many lives in the city of Ankara, Beypazarı and their surroundings (Ambraseys and Finkel, 1995).

Additionally, another historical earthquake, namely 1875/76 Ankara Earthquake was also mentioned with an intensity of VI in the earthquake catalogue of Ergin et al. (1967). However, there is no definite information regarding the seismicity and damage of this earthquake.

Considering the past recent activities, one of the important seismic event is the Kırşehir earthquake of March 19, 1938 ($M_s=6.8$) that occurred with an intensity of IX along the Seyfe Fault Zone. During the earthquake that activated the Akpınar-Taşkovan segment of the Seyfe Fault Zone, an approximately 15 km long and few meters to kilometers wide surface faulting zone was developed. Hence, Kırşehir earthquake of March 19, 1938 is the biggest earthquake in the Central Anatolia during this century that had caused severe damages to the structures in the city center of Ankara.

Along the central segment of the NAFS, two large destructive events have occurred with an intensity of X on 26.11.1943 and 1.02.1944, respectively (Dewey, 1976). These seismic events are the Kastamonu and Gerede earthquakes of $M_s=7.3$ that were associated with faulting between Ilgaz and Ladik; and Gerede- Çerkeş segments of the NAFS. It is important to note that Ankara and Beypazarı were affected from these earthquakes and particularly from the Gerede event that had caused severe damages to the structures and led to loss of lives in the city of Ankara and Beypazarı.

From the information so far collected for the 1943 and 1944 events, it is more likely that the two earthquakes that occurred in 967 and 1035 took place where faulting occurred in 1944. Additionally, although the extent of rupture associated with the 1668 event is not clear, the earthquakes of 12 and 15 August 1668 may have similar origins with these earthquakes, and broken segments of the NAF zone where the 1943 and 1944 segment also ruptured. One should, however, note that since there were several major events during the period of 3 July to 13 September 1668, it seems

probable that one shock may have broken a segment which ruptured in 1943. As a result of this event, Ilgaz, Merzifon and Amasya were destroyed. Note that these settlements were also affected by the 1943 event (Kuran, 2005).

3.2.3 Seismic Hazard Assessments

The aim of this section is to conduct a seismic hazard analysis for Ankara and its neighboring regions, using the recently developed attenuation relationships. After identification of the earthquake sources and the regional seismicity discussed above, the next step in the regional seismic hazard and risk analysis is to determine the bedrock motion in the region regarding the modeling of the earthquake occurrence on each seismic source.

The most common method involves the use of an empirical attenuation relationship. These relationships express a given ground motion parameter in a region as a function of the size and location of an earthquake event. Numerous relationships have been developed since then, typically by applying statistical regression analyses to recorded data. Often these relationships are developed with different functional forms and with different definitions of ground motion, magnitude, distance, and site conditions (Campbell, 1985).

Areas of high seismicity, such as the Marmara region, present opportunities for determining seismic hazard based on the specially compiled catalogs of historical and instrumental analysis of strong motion data. The frequency of high magnitude events and the vast array of seismic instruments in operation provide abundant data for predicting bedrock motion. However, in Central Anatolia such as the Ankara region, seismic activity is less frequent and stations are widely spaced or almost absent. The lack of seismic activity and the scarcity of data for large magnitude events make earthquake hazard estimation in this region uncertain and thus it is difficult to identify the seismic source regionalization. Figure 3.2 shows the National Strong Motion Network of Turkey (ERD, 1976 to 2005). According to the aim of this research and distribution of the

instruments with limited number of accelerometers (i.e analog and digital instruments along with local networks), they are installed particularly along the major Fault Systems and Fault Zones where the big earthquakes occurred or the expected active areas with a distance about 50-80 km to these fault systems (Çeken in ERD, 2000). Among these instruments, only two analog accelerometers are installed in the Ankara region, namely Beypazarı (BEY) and Haymana (HAY). Note that two additional instruments are installed in Ankara, namely at Yalıncak and the ERD Main building, however these velocity measuring instruments are used for a different purpose. Therefore, is it not only difficult to predict the magnitude of a potential earthquake, but also the lack of earthquake information makes the attenuation of the earthquake motions difficult to characterize. The selection process used to identify earthquake magnitudes and bedrock ground motions that could affect the Ankara region are summarized in the paragraph below.

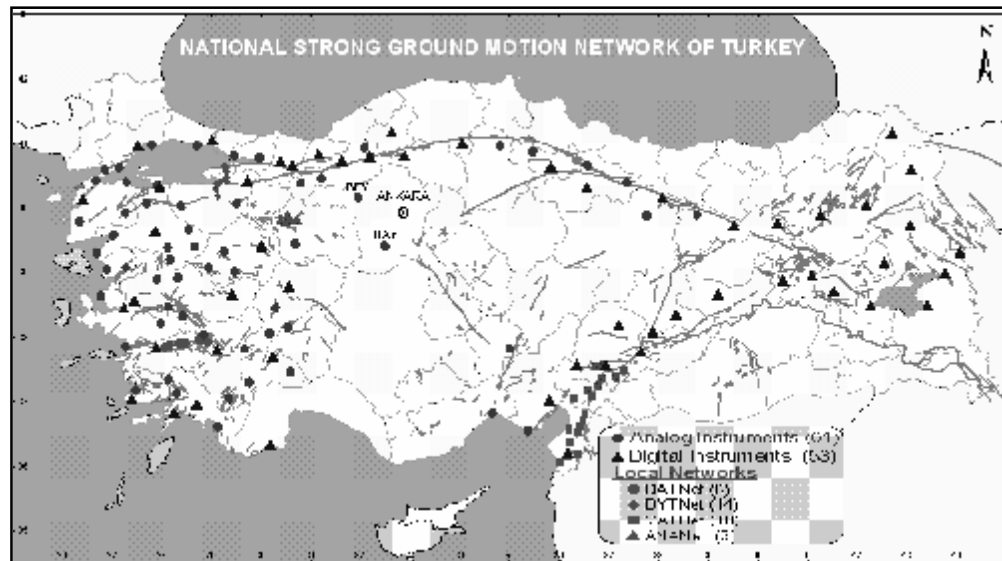


Figure 3.2. The National Strong Motion Network of Turkey (ERD, 1976 to 2005).

Seismic sources are identified by using historical earthquakes and instrumental locations of the earthquakes. According to the information presented above, three earthquake source zones are the most well known, where countless damaging earthquakes have occurred throughout history were identified (NAFS, SLFZ and SFZ). In addition, other seismic zones might be defined to account for other earthquakes; however these are not taken into account due to the low seismic activity capability of these sources (i.e., $M < 6$) and also to delineate zones where no significant earthquake has taken place in this century (i.e., 1875/76 Ankara Earthquake).

It should be noted that seismic hazard analysis requires an appropriate strong-motion attenuation relationship, which describes the propagation and modification of ground motions as a function of earthquake size (Magnitude, M) and the distance (R) between the source and the site of interest. In general, there are two basic approaches in developing design ground motions that are commonly used in practice: deterministic and probabilistic.

In the deterministic approach that is utilized in this dissertation, individual earthquake scenarios (earthquake magnitude and location) are developed for each relevant seismic source and a specified ground motion probability level is selected (by tradition, it either possesses 0 or 1 standard deviation above the median). Based on the seismic source location, the distance to the site is computed. Given the magnitude, distance, and number of standard deviations for the ground motion, the ground motion is then computed for each earthquake scenario, using a ground motion attenuation relation or a numerical simulation method. The largest ground motion from any of the considered scenarios is used for the design ground motion. The approach is “deterministic” in that single values are selected for the scenario parameters (magnitude, distance, and number of standard deviations for the ground motion) for each scenario (Abrahamson, 2003).

The scarcity of the local strong-motion acceleration data in Turkey makes it unavoidable to either define the attenuation on the basis of local intensities or borrow the already developed acceleration attenuation relationships based on foreign data (Erdik et. al., 1985). In many aspects, the Western American Fault

Systems (i.e., San Andreas and Hayward Fault) and North Anatolian fault zones show many similarities. They both are similarly right-lateral, strike-slip faults, at the same time, are transforms (Ketin, 1976). Additionally, the attenuation relationships based on western American strong-motion data satisfactorily agree with the strong-motion data obtained from Anatolian earthquakes (Erdik et. al., 1985 and 1996). Based on the material presented by these explanations, the following attenuation relationships were taken into considering for the determination of peak ground acceleration (PGA) and brief descriptions of these relationships used to assign attenuated rock motion to the Ankara region are as follows.

Considering the suitability of the attenuation models, the moment magnitude M was used as the general magnitude unit because the use of moment magnitude avoids the “saturation” of the more traditional band-limited magnitude measures (i.e., local magnitude- M_L and surface wave magnitude- M_s) at large seismic moments. Therefore, moment magnitude is a better measure of the true size of an earthquake (Kanomori, 1983 and Idriss, 1985) and hence, as described in the models, it is recommended to be used with the attenuation relationships.

The determination of the magnitude for a source zone is a difficult and quite subjective task. In seismic sources with prominent faults and large activity, the maximum magnitude versus fault rupture regression equations (Slemmons, 1977) or the historical maximum magnitudes may be utilized. The general assumption, based on worldwide data is to assume 1/2 to 1/3 of the total fault length as the rupture that will correspond to the maximum magnitude (Mark, 1977) or increase the maximum historical earthquake magnitude by half a magnitude unit to yield the maximum magnitude. These approaches may not necessarily yield coherent maximum magnitude estimation among the seismic sources. Indeed, the methods for determining the earthquake potential of active faults are based on observations related to earthquake size (magnitude or seismic moment), recurrence intervals, fault strain or slip rates, length of surface fault ruptures, maximum or average surface fault displacement, and overall length of the total fault zone. Empirical relationships among these factors make it possible

to estimate the size of past earthquakes, using the evaluation of historical surface rupture parameters. This provides a key for assessing the potential of future earthquakes (Slemmons, 1982).

In this dissertation, several empirical equations are utilized to estimate maximum magnitudes associated with seismic sources encompassing major faults as a function of rupture length and/or displacement (DePolo and Slemmons, 1990 and Wells and Coppersmith, 1994). However, these empirical results should be considered along with the historical earthquakes that have occurred along the fault or fault zone as the maximum credible earthquake (Slemmons, 1977) that is likely to occur on the structure in the future. This method may provide a reasonable assessment for seismic source of these faults.

It has been observed that larger fault zones commonly do not rupture over their entire lengths during individual earthquake events, but rather some part or percentage of the fault length will rupture (e.g., North Anatolian Fault System). After a fault or fault zone has been delineated, some logic or technique needs to be applied to estimate the potential earthquake rupture length. Several techniques have been developed, such as the half-length, fractional-fault-length, and segmentation techniques (Slemmons, 1982).

The “fractional-fault-length” is considered to be the most suitable technique amongst the others because some of the statistical data along with worldwide data are based on the fault ruptures of the major earthquake events along the NAFS. During the short time period of 1939 to 1967, the NAFS ruptured over most of its length, not as a single thoroughgoing event but as a series of individual earthquake segments (DePolo and Slemmons, 1990). The method is used for all fault-slip types, although the original application was for a region where most of the data was for strike-slip faults with historical earthquakes of $6 < M < 9$, in regions of shallow focus earthquakes. These data suggest that the fractional rupture length varies for faults of this type from about; 1/6th of the total fault length for faults with less than about 200 km total length to 1/3rd of the total fault length for faults with lengths of more than 1000 km. The relationships for reverse-slip and normal-slip faults have not been compiled and it has not been

demonstrated that this relationship applies to all fault slip types (Table 3.3). Once the fractional rupture length is assessed, the normal procedure is to use regression equations given below to calculate the maximum credible earthquake magnitudes.

The percent of total fault length (P) ruptured during a maximum earthquake regressed vs. total fault length (L, in km) for strike-slip faults are expressed as ($R^2 = 0.805$, Std. Dev. $= \pm 0.221$):

$$P = 15.76 + 0.0012 (L) \quad (3.1)$$

Then, regression relationships between earthquake magnitude (M) and the length of surface rupture for the surface length (L, in m) for worldwide data of all fault types are stated as:

$$M = 2.062 + 1.068 \log (L) \quad (3.2)$$

Table 3.3. Relations of earthquake magnitude (M) regressed vs. log Length (L, in m) from Slemmons (1982).

	NUMBER	a	b	Std. Dev.	R ²
NORTH AMERICA	23	1.267	1.238	0.290	0.817
REST OF WORLD*	33	2.855	0.899	0.286	0.630
WORLDWIDE DATA*	56	2.062	1.068	0.297	0.722
A, normal fault	15	0.809	1.341	0.318	0.563
B, reverse fault	8	2.021	1.142	0.197	0.882
C, reverse-normal-oblique	4	0.875	1.348	0.143	0.900
D, normal-oblique	9	1.199	1.271	0.273	0.752
E, strike-slip	20	1.404	1.169	0.205	0.879

* A total of 6 disastrous earthquake events ($M = 6.9$ to 7.9) along the NAFS were included in the database during 1939 and 1967.

More recently, Wells and Coppersmith (1994) compiled a source parameter data set of 421 worldwide earthquakes and derived a series of empirical regression relationships that combine moment magnitude (M) with the different rupture scenarios as a regression function of rupture length, rupture width and rupture area. The data included shallow-focus earthquakes of magnitudes greater

than approximately 4.5. Among these data, a total of 15 historical earthquakes were compiled depending on the fault ruptures of the major earthquake events along the NAFS, EAFS and Western Anatolia Fault Zones where the data was for strike-slip, normal and reverse faults with historical earthquakes of $M = 6.5$ to 7.9 . Considering the empirical relationships, ordinary least-squares regression analyses included regression of M and log of surface rupture length as a function of all slip type which are tabulated in Table 3.4. Then, regression equations between earthquake magnitude (M) and the length of surface rupture for the surface length (L , in m) of fault types are given as:

$$M_w = 5.16 + 1.12 \log (L) \quad (\text{for strike-slip fault}) \quad (3.3)$$

$$M_w = 4.86 + 1.32 \log (L) \quad (\text{for normal fault}) \quad (3.4)$$

Table 3.4. Regression analyses include regression of M and log of surface rupture length as a function of all slip type.

Equation*	Slip Type	Number of Events	Coefficients and Errors		Standard Deviation s	Correlation Coefficient r	Magnitude Range
			a(sa)	b(sb)			
$M = a + b * \log (\text{SRL}^*)$	SS	43	5.16(0.13)	1.12(0.08)	0.28	0.91	5.6 to 8.1
	R	19	5.00(0.22)	1.22(0.16)	0.28	0.88	5.4 to 7.4
	N	15	4.86(0.34)	1.32(0.26)	0.34	0.81	5.2 to 7.3
	All	77	5.08(0.10)	1.16(0.07)	0.28	0.89	5.2 to 8.1
$\log (\text{SRL}) = a + b * M$	SS	43	-3.55(0.37)	0.74(0.05)	0.23	0.91	5.6 to 8.1
	R	19	-2.86(0.55)	0.63(0.08)	0.20	0.88	5.4 to 7.4
	N	15	-2.01(0.65)	0.50(0.10)	0.21	0.81	5.2 to 7.3
	All	77	-3.22(0.27)	0.69(0.04)	0.22	0.89	5.2 to 8.1

*SRL-surface rupture length (km); SS-strike slip fault; R-reverse fault; N-normal fault.

Considering the methodologies of DePolo and Slemmons (1990) and Wells and Coppersmith (1994) along with the historical earthquakes that have occurred along the major Fault systems and Fault Zone (i.e., NAFS, SFZ), these relationships were applied to estimate the maximum magnitudes associated with seismic sources encompassing major faults as a function of rupture length. Hence,

as a result, the major active Fault System or Fault Zones (seismic sources) and their corresponding maximum magnitudes (M) surrounding the Ankara region are tabulated in Table 3.5. The results of the maximum magnitude (M) calculated from the empirical equations showed general consistency with quite a few of the compiled data from the NAFS. These empirical results are also compatible with the recorded historical earthquakes along the NAFS. Regarding the largest historical earthquakes that have occurred along the NAFS, the event of the 1939 and 1943 earthquakes of $M=7.9$ and 7.3 and their ground ruptures of about 360 km and 270 km, respectively (note that the 1668 earthquake had about 380km ground rupture) where the recorded magnitudes as a function of rupture length are very high, display quite reasonable agreement with the empirical results. Considering the largest historical earthquakes that have occurred along the SFZ, the Akpınar-Taşkovan event of the 1939 earthquake ($M_s=6.8$) also reasonably compared well with the empirical results. Finally, the earthquake potential of the Ankara region was determined to account for the maximum credible earthquake for all cases to be later used in the attenuation study to assign attenuated rock motion to the Ankara region.

Table 3.5. The major active Fault System or Fault Zones and their corresponding maximum magnitudes (M) surrounding the Ankara region.

Major Fault System and Fault Zones (Seismic Source)	Approximate length of Fault System and Fault Zones	Fault slip type	Maximum magnitude (M) calculated from empirical Eqs.	Maximum assigned magnitude (M)
North Anatolian Fault System	~1200	Strike Slip	8.05 ¹	8.00
			7.98 ²	
Seyfe-Keskin Fault Zone	~138	Strike Slip	7.0	7.0
			6.7	
Salt Lake Fault Zone	~185	Normal Fault	7.2	7.0
			6.8	

Empirical equations of 1- Wells and Coppersmith (1994); 2- DePolo and Slemmons (1990)

In respect to the suitability of the attenuation models depending on the characteristics of the faults in the study area, basically the two attenuation models proposed by Boore et. al. (1997) and Abrahamson and Silva (1997) were considered in predicting the attenuated rock motion for the earthquake hazard assessment study of the Ankara region. These proposed attenuation relationships were developed based specifically on shallow earthquakes in Western America that show many similarities with the North Anatolian Fault Systems since the strong-motion data obtained from the west American earthquakes are quite compatible with those obtained from Anatolian earthquakes.

Attenuation Relationships Developed by Boore et. al. (1997): Boore et. al. (1997) proposed attenuation relationships for random horizontal peak ground acceleration and pseudo-acceleration response spectra for shallow earthquakes in western North America of moment magnitude (M) greater than or equal to 5.3. The equations give ground motion in terms of moment magnitude, distance, and site conditions for strike-slip, reverse-slip, or unspecified faulting mechanisms. Boore et al. (1997) use moment magnitude as a measure of earthquake size and a distance equal to the closest horizontal distance from the station to a point on the earth's surface that lies directly above the rupture (r_{jb}), referred to as the "Joyner-Boore distance (BJF)". Site conditions are represented by the shear velocity averaged over the upper 30 m, and recommended values of average shear wave velocity are given for typical rock and soil sites and for site categories used in the National Earthquake Hazard Reduction Program's (NEHRP) recommended seismic code provisions.

The ground-motion estimation equation is:

$$\ln Y = b_1 + b_2(M - 6) + b_3(M - 6)^2 + b_5 \ln r + b_v \ln \frac{V_s}{V_A} \quad (3.5)$$

where

$$r = \sqrt{r_{jb}^2 + h^2} \quad (3.6)$$

and

$$b_1 = \begin{cases} b_{1SS} & \text{for strike – slip earthquakes;} \\ b_{1RS} & \text{for reverse – slip earthquakes;} \\ b_{1ALL} & \text{if mechanism is not specified.} \end{cases} \quad (3.7)$$

In Eq. (3.5), Y is the ground-motion parameter (peak horizontal acceleration in g); where, the predictor variables are moment magnitude (M), distance (r_{jb} , in km), and average shear-wave velocity to 30 m (V_s , in m/s). Coefficients or entries for zero period to be determined are b_{1SS} , b_{1RS} , b_{1ALL} , b_2 , b_3 , b_5 , b_h , b_v , and V_A , which are equal to -0.313, -0.117, -0.242, 0.527, 0, -0.778, 5.570, -0.371 and 1396, respectively (Boore et al., 1997). Note that h is a fictitious depth that is determined by the regression.

In the Boore et al. (1997) method, the coefficients in the equations for predicting ground motion were determined using a weighted, two-stage regression procedure. In the first stage, the distance and site condition dependence were determined along with a set of amplitude factors, one for each earthquake. In the second stage, the amplitude factors were regressed against magnitude to determine the magnitude dependence.

The mean plus one standard deviation of sigma value of the natural logarithm of the ground-motion value from Eq. (3.5) is $\ln Y + \sigma_{\ln Y}$, where $\sigma_{\ln Y}$ is the square root of the overall variance of the regression, given by

$$\sigma_{\ln Y}^2 = \sigma_r^2 + \sigma_e^2 \quad (3.8)$$

where, σ_e^2 represents the earthquake-to-earthquake component of the variability and is determined in the second stage of the regression, and σ_r^2 represents all other components of variability.

$$\sigma_r^2 = \sigma_1^2 + \sigma_c^2 \quad (3.9)$$

where σ_1^2 is the variance from the first stage of the regression and σ_c^2 represents the correction needed to give the variance corresponding to the randomly-oriented horizontal component.

Note that the coefficients for estimating the peak horizontal accelerations are entered as entries for zero period and the values of the mean plus one standard error terms of σ_1 , σ_c , σ_r , σ_e , and $\sigma_{\ln Y}$ are taken as 0.431, 0.226, 0.486, 0.184 and 0.520, respectively (Boore et al., 1997).

Attenuation Relationships Developed by Abrahamson and Silva (1997):

Using a database of 655 recordings from 58 earthquake main shocks and after shocks with magnitudes greater than 4.5 in the full data set, empirical response spectral attenuation relations are derived for the average horizontal and vertical component for shallow earthquakes in active tectonic regions. One of the new features of this model is the inclusion of a factor to distinguish between ground motions on the hanging wall and footwall of dipping faults. The site response is explicitly allowed to be non-linear with a dependence on the rock peak acceleration level. In this study, empirical models are developed for the attenuation of response spectral values for both the average horizontal and the vertical components applicable to shallow crustal events in active tectonic regions. The site classification is based on the Geomatrix (1993) site class. Geomatrix site class C and D were combined into a single deep soil site category. The Geomatrix A and B classes (rock and shallow soil) were also combined into a single “rock” site category. Although the site classification based on the Geomatrix (1993) has quantitative values, for most of the sites, such quantitative information is not available, so the sites are classified subjectively using the criteria as a guide rather than a strict classification scheme. The closest distance to the rupture plane, r_{rup} was used in this research and the distribution of the data in terms of magnitude and distance space is represented for four periods ($T= 5.0, 1.0, 0.2$, and 0.075 seconds) for the horizontal and vertical components.

In developing the functional form of the regression equation, Abrahamson and Silva (1997) combined features of the regression equations. Random effects

model are used for the regression analysis. The general functional form that is employed in this dissertation is given by:

$$\ln Sa(g) = f_1(M, r_{rup}) + Ff_3(M) + HWf_4(M, r_{rup}) + Sf_5(pga_{rock}) \quad (3.10)$$

where $Sa(g)$ is the spectral acceleration in g, M is moment magnitude, r_{rup} is the closest distance to the rupture plane in km, F is the fault type (1 for reverse, 0.5 for reverse/oblique, and 0 otherwise), HW is the dummy variable for hanging wall sites (1 for sites over the hanging wall, 0 otherwise), and S is a dummy variable for the site class (0 for rock or shallow soil, 1 for deep soil). For the horizontal component, the geometric mean of the two horizontals is used.

The function $f_1(M, r_{rup})$ is the basic functional form of the attenuation for strike-slip events recorded at rock sites. For $f_1(M, r_{rup})$, Abrahamson and Silva (1997) used the following equations:

for $M \leq c_1$

$$f_1(M, r_{rup}) = a_1 + a_2(M - c_1) + a_{12}(8.5 - M)^n + [a_3 + a_{13}(M - c_1)] \ln R \quad (3.11)$$

for $M > c_1$

$$f_1(M, r_{rup}) = a_1 + a_4(M - c_1) + a_{12}(8.5 - M)^n + [a_3 + a_{13}(M - c_1)] \ln R \quad (3.12)$$

where

$$R = \sqrt{r_{rup}^2 + c_4^2} \quad (3.13)$$

The entries for zero period (coefficients for the average horizontal component), namely a_1 , a_2 , c_1 , a_{12} , n , a_3 , a_{13} , a_4 and c_4 that are given in Eqs. (3.11) through (3.13) above are taken as 1.64, 0.512, 6.40, 0, 2, -1.145, 0.170, -0.144 and 5.6, respectively (Abrahamson and Silva, 1997).

The functional form that allows for a magnitude and period dependence of the style of the faulting factor is given by:

$$f_3(M) = \begin{cases} a_5 & \text{for } M \leq 5.8 \\ a_5 + \left(\frac{a_6 - a_5}{c_1 - 5.8} \right) & \text{for } 5.8 < M < c_1 \\ a_6 & \text{for } M \geq c_1 \end{cases} \quad (3.14)$$

where, the entries a_5 and a_6 for zero period are taken as 0.610 and 0.260, respectively (Abrahamson and Silva, 1997).

The magnitude and distance dependence of the functional form, $f_4(M, r_{rup})$, for the hanging wall effect is modeled as separable in magnitude and distance so that:

$$f_4(M, r_{rup}) = f_{HW}(M)f_{HW}(r_{rup}) \quad (3.15)$$

The total standard error is computed by adding the variance of the inter-event (t) and intra-event (s) error terms. The total standard error is then smoothed to fit the form:

$$\sigma_{total}(M) = \begin{cases} b_5 & \text{for } M \leq 5.0 \\ b_5 - b_6(M - 5) & \text{for } 5.0 < M < 7.0 \\ b_5 - 2b_6 & \text{for } M \geq 7.0 \end{cases} \quad (3.16)$$

The final model for estimating the related standard error term entries for zero period, namely b_5 and b_6 are 0.700 and 0.135, respectively (Abrahamson and Silva, 1997).

Abrahamson and Silva (1997) computed the regression using multiple steps. The multiple steps are used to constrain the resulting model to be a smooth function of period for all magnitudes, distances, mechanisms, and site conditions. Following each step, the period dependence of the uncorrelated coefficients was

smoothed using piecewise continuous linear fits on the log period axis. For highly correlated coefficients, one coefficient was smoothed and then the other coefficients were re-estimated.

Based on these relationships presented by these explanations, the maximum horizontal ground accelerations (PGA) for rock in the Ankara region were developed using the Boore et al (1997) and Abrahamson and Silva (1997) attenuation methodologies. A summary of the median PGA results along with median plus one standard error predictions ($PGA + \sigma$) for rock are tabulated in Table 3.6. As noted previously, the deterministic approach of the attenuation relationships traditionally uses at most 1 standard deviation above the median for the ground motion (Abrahamson 2000; 2003).

Examining the developed mean PGA results with one plus standard error predictions, it might be concluded that the attenuation relationship results are successful in predicting the peak horizontal acceleration values in a given interval and particularly show close relationships when comparing the median peak horizontal acceleration values at the given research sites irrespective of the type of source-to-site distance measuring system utilized. Note that Abrahamson and Silva (1997) attenuation relationship is uses the closest distance to the rupture surface, whereas the Boore et al (1997) attenuation relationship uses the BJJ distance method instead of the closest distance.

In relation to the aim of this chapter, deterministic seismic hazard analysis for Ankara and its neighboring regions were conducted using the recently developed attenuation relationships, and then the bedrock peak ground motion to be used for regional seismic hazard study in the region was determined for evaluating the local site effect on ground motion that will be explained in later chapters. However, for the sake of the reliability of this research, it might be compared with the other comprehensive studies that have been generally conducted on a more regional scale containing probabilistic estimates of the maximum MSK intensity, and maximum horizontal peak ground acceleration. The results obtained from research were compared with the results obtained from

some of the important research conducted by Erdik et al. (1985), Gülkan et al. (1993) and Erdik et al. (1996) for the evaluation of seismic hazards.

Table 3.6. Summary of the median peak horizontal acceleration values (PGA) along with the median peak horizontal acceleration + one standard error prediction values (PGA+ σ) for rock based on Boore et al (1997) and Abrahamson and Silva (1997) attenuation relationships to predict the for the Ankara region.

Boore et al. (1997); M and r_{jb}			
Fault Source	r_{jb} (km)-h(km)	M (assigned)	PGA (PGA +σ)
North Anatolian F.S.	77.64-10	8.0	0.124 (0.208)
Seyfe F.Z.	65.76-10	7.0	0.081 (0.142)
Salt Lake F.Z.	70.00- 0	7.2	0.089 (0.150)
Abrahamson and Silva (1997); M and r_{rup}			
Fault Source	r_{rup} (km)	M (assigned)	PGA (PGA+ σ)
North Anatolian F.S.	77-85	8.0	0.126 (0.192)
Seyfe F.Z.	65-70	7.0	0.076 (0.117)
Salt Lake F.Z.	70-75	7.2	0.082 (0.125)

The major aim of the seismic hazard analyses presented in this study is to assess the probability that the ground motion parameter at a site due to the earthquakes from potential seismic sources will exceed a certain value in a given time period. Hence, all possible and relevant deterministic earthquake scenarios as well as all possible ground motion probability levels (a range of the number of standard deviations above the median) are considered.

It should be highly emphasized that in probabilistic studies, the most important argument in reaching a severity of shaking level that is either not too costly or not too rare is: “given this large set of deterministic ground motions, which one should one select?” Both of these conditions must occur to rule out using worst-case ground motions because it has a large impact on the cost of the design and it is so rare that its use is not justified. In the selection process, the

scenarios are ranked in decreasing order of severity of shaking (e.g. decreasing amplitude of the ground motion). The rates of scenarios are then summed up from the most severe shaking to the least severe shaking. The summed rate is called the "hazard" that is the rate at which the ground motion equal or larger to a specified level occurs at the site (Abrahamson, 2003).

Therefore, the acceptable hazard level that was considered to be used during the preparation of the probabilistic research studies is based on the defined regulation of the UBC 1997 (ICBO, 1997) and Global Seismic Hazard Assessment Program (GSHAP, Giardini and Basham, 1993) which specify a hazard level of 0.0021/yr (corresponding to 10% chance of being exceeded in 50 years) to define the ground motion. On the basis of these explanations, a probabilistic seismic hazard analysis map of Turkey by Erdik et al. (1996) and Gülkan et al. (1993), in terms of iso-horizontal PGA (g) contours corresponding to 475 year return period (10% probability of exceedance in 50 years) were considered to be used for this research study. Figures 3.3 and 3.4 display the area of the research site that is projected on the seismic hazard map in terms of iso-horizontal PGA contours corresponding to a 10% probability of exceedance in 50 years that were prepared by Erdik et al. (1996) and Gülkan et al. (1993), respectively.

Regarding these evaluation results, it might be concluded that the median plus one standard deviation ($PGA + \sigma$) results that were developed for the Ankara region through using the Boore et al (1997) and Abrahamson and Silva (1997) attenuation relationships are in close agreement in predicting the iso-horizontal PGA contours for both of the probabilistic seismic hazard maps of Turkey prepared by Erdik et al. (1996) and Gülkan et al. (1993). The research site is situated along the PGA contours of 0.2g in the seismic hazard maps. Consequently, bedrock peak ground motion for the Ankara region was determined to be as 0.2 g that will be used to evaluate the local site effects in the regional seismic hazard study.

It should be noted that the $PGA + \sigma$ (g) values denote maximum horizontal accelerations on competent soil or rock. With soil deposits of soft- and medium-

stiff sands and clays of appreciable depth, the ground accelerations will be different than those indicated on these hazard maps (Erdik et al., 1996). Hence, it should be emphasized that considering the estimated intensity of the bedrock acceleration for the selected site, the site amplification of the soil deposits have to be determined for assessing the local site conditions and in particularly for local scale seismic hazard evaluations because the impact of local site conditions subsequent to the significant influence of strong ground motions on site amplification have to be considered.

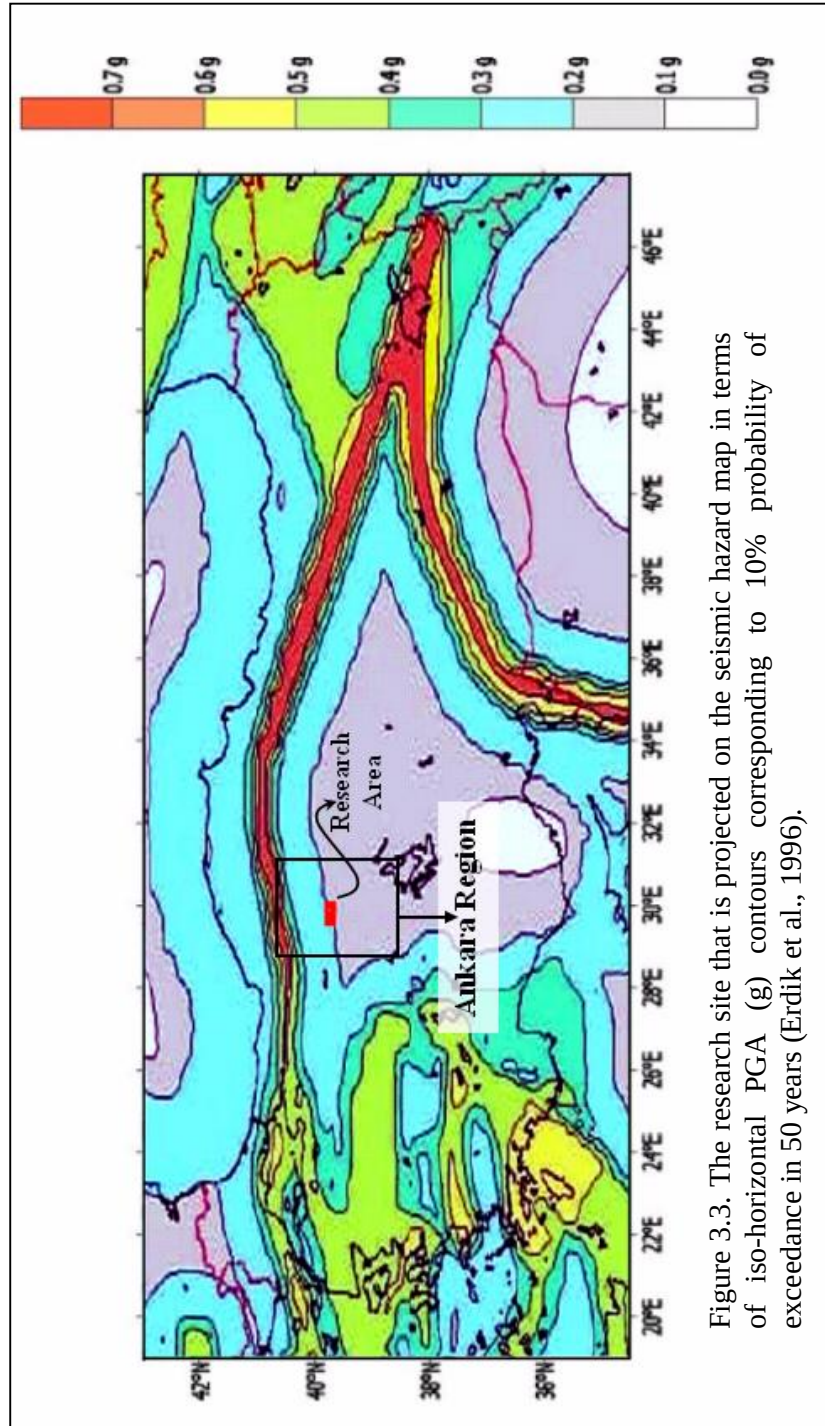
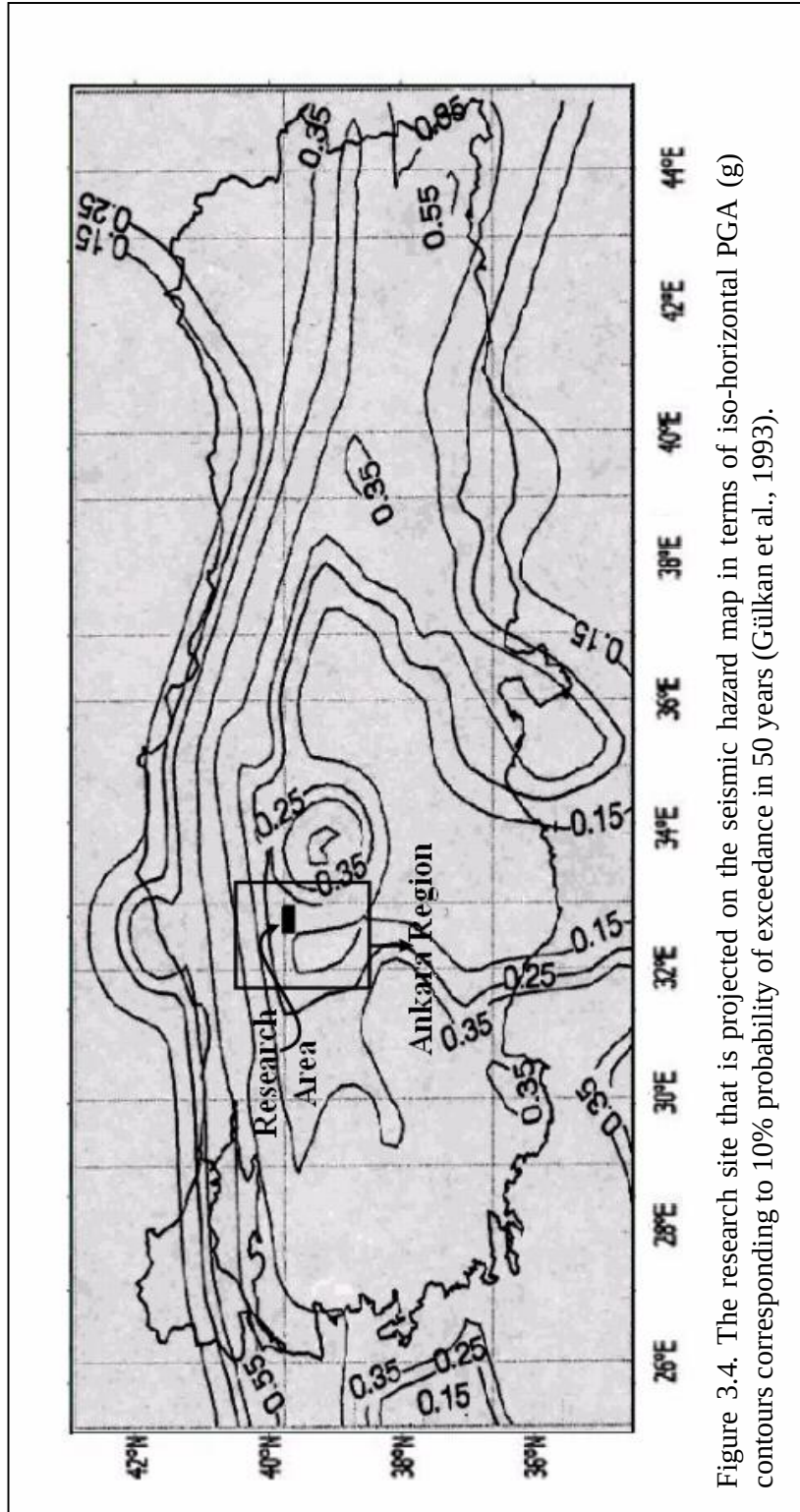


Figure 3.3. The research site that is projected on the seismic hazard map in terms of iso-horizontal PGA (g) contours corresponding to 10% probability of exceedance in 50 years (Erdik et al., 1996).



CHAPTER 4

METHODOLOGY FOR DEVELOPMENT OF AN ENGINEERING GEOLOGICAL, GEOTECHNICAL AND SEISMIC SITE CHARACTERIZATION STUDY

4.1. Introduction

In order to perform seismic hazard assessments, sufficient geological, engineering geological, geotechnical and seismic data must be available to perform independent analyses and thus a large number of factors are required to describe these earthquake hazards. The resulting large databases require an appropriate environment to optimize the evaluation procedures. The objective of this research is to integrate a variety of databases and methods (or analyses procedures) coupled with invasive and non-invasive field testing results obtained in this study in regards to identify and map geotechnical hazards utilizing geographic information technology. The various seismic hazard evaluations have been combined with site specific engineering geological, geotechnical and geophysical information to enable the development of detailed maps that summarize the potential for the respective hazards within the project area.

A traditional engineering geological and geotechnical engineering study typically concentrates on the subsurface characterization of a specific site and the interaction of man made structures with the earth mass; however, multi-disciplinary studies usually expand the focus of the project into other related fields (i.e., geo environmental engineering). For this purpose, the engineering geologist is required to collect all the available information to solve the problem. The sources of information are the subsurface data recovered by invasive (e.g., boreholes) and non-

invasive (e.g., geophysical) explorations, the existing surface structures, and the future surface and subsurface structures planned for the site, if any. The multiple types of information are available in different physical forms and the engineer's expertise and judgment are used to collect this information and make decisions and recommendation about how to proceed with the research project. When the amount of information that can be effectively collected and manipulated is abundant, the use of information and database management systems can aid in the problem solving process of the engineer (Rockaway et al., 1995 and Luna, 1995).

4.1.1. Engineering Geological and Geotechnical Data Source and Data Quality Requirements

Regarding the development of an engineering geological and geotechnical seismic database, the formal process of defining the data sources and requirements is an important task, as in any other database design. Even though the geotechnical data typically encompasses subsurface information, other forms of data are required to complement the geotechnical data and make it useful in a spatial framework. Table 4.1 shows an example list of different data sources for a geotechnical project classified by their category and type (Luna, 1997; Luna and Frost, 1998).

Once the data sources have been identified, one should identify what is required to solve the problem statement. This can be as simple as listing all the requirements and organizing them as a function of their relationships. Engineering geological and geotechnical data also depend on other data and may require corrections of the measured value as a function of depth. For example, the resistance to penetration SPT N-value is dependent on information available in the geotechnical report, such as, drilling, sampling and testing methods. Relationships between these different data types can be established in a database once it is logically organized and structured. This task is usually accomplished early on in the research and may have a major impact on the design and implementation of the

database. Most data sources listed in Table 4.1 are generally used on a routine basis in engineering as a particular level of standardization.

Table 4.1. List of typical geotechnical (or related) data sources (modified from Luna, 1997)

Data Category	Data Source or Type	Content of Data
Geotechnical Subsurface	Geotechnical Reports	Subsurface conditions as they relate to foundation engineering and soil mechanics containing specific engineering recommendations.
	Geologic Reports	Regional geology, geomorphology, or structural geology documented for general usage.
	Boring (Borehole) Logs	Subsurface conditions, stratigraphy, drilling, sampling, field and laboratory testing, etc.
	CPT Logs	Tip resistance, side friction and pore pressure during penetration. Permeability.
	Test Pit Logs	Excavation properties, stratigraphy, and soil descriptions.
	Piezometers and Groundwater Well Logs	Groundwater changes and pore water pressures
Geophysical Subsurface	Geophysical Borehole	cross-hole, up and down hole testing data
	Geophysical Reflection and Refraction, Surface Wave Methods	Stratigraphy, dynamic soil properties.
	Ground Penetrating Radar	Stratigraphy of subsurface and subsurface anomalies
Seismic and Tectonic Records	Earthquake Events	Location, hypocenter, epicenter, magnitude, and acceleration of event.
	Seismic Maps	Locations of faults, background sources and other earthquake source.
Infrastructure and Demographic	Regional DLG and DEM	Spatial data including transportation, topographical, hydrological, political boundaries, etc.
	TIGER Data	Spatial census information with demographics and regional features
	CADD Drawings	Roads, buildings, utilities
Images	High and Low Altitude Aerial Photographs, Satellite Images	Image of surficial features at a specific time (buildings, roads, EQ hazards and damage areas), and surficial geologic and geomorphologic structures
	Photographs	General interest site photographs to aid in the reconnaissance. (damage areas and documentation of hazards may be recorded)

Data quality is essential for the adequate use in any engineering application. Quality has different aspects that are of importance to users and can be used to determine the fitness of the data for their particular use. In geological and geotechnical engineering, it starts in the field during the subsurface investigation, or even before, when the investigation is being planned and specified. ASTM standards

are available for most of the field testing and sampling methods, but they are not always required in practice which makes the standardization and quality of geotechnical data very variable. The advantage of combining the subsurface information with other information in a computer system allows for means of complementing the data where gaps are found (D'Andria et al., 1995 and Rockaway, et al., 1997).

Based on the general introduction and data requirements outlined above, the collection and organization of engineering databases being considered as a part of this research are discussed below.

4.2. Collection and Organization of Subsurface Data

It would be difficult to create an intelligible and useful engineering database that might contain sufficient geological, engineering geological, geotechnical and geophysical data for the seismic hazard studies. The first set of issues concern the resolution and distribution of data, or samples, and their relationship to the variability of the geology (i.e., lithologies and facies) and the engineering geological, geotechnical and geophysical properties (i.e., shear wave velocity, blow count, required laboratory tests, etc.). For this reason, a significant portion of the engineering geological, geotechnical and geophysical data required to complete the research has been both collected from existing public and private sources; and incorporated into our research on Upper Pliocene to Pleistocene fluvial red clastics and Quaternary alluvial and terrace sediments in the Ankara basin.

4.2.1. Construction and Organization of Engineering Geological and Geotechnical Data Compiled from Previous Studies

The type of information being collected from existing public and private sources that is incorporated into the database includes geology, geomorphology and type of sediments, typical soil characteristics (e.g. degree of compaction, density of

sediments, index properties, penetration resistances, shear wave velocities), deep borings for bedrock depth determination, groundwater table location, topography, seismicity, air photos and satellite images. Sources for information of this type range from large scale regional geological surveys to local site investigation studies involving in-situ and laboratory tests on specimens.

The existing engineering geological and geotechnical data available for the research site can vary in many ways given that most investigations have a specific purpose which is not exactly related to providing data for an earthquake hazard assessment study. A site may contain borehole and laboratory data in a scattered, sparse manner or, on the other hand, it may be very abundant and laid out on a grid or another particular spatial distribution. Additionally, the quality of the data has to be examined for its appropriateness in the study, since this may affect the results. The quality of the data will depend on the type of the in-situ test available (i.e., SPT data from boring or geophysical data). In the case of SPT data, no record of energy efficiency measurements has been identified in the data collection phase. However, a more rigorous evaluation of data quality was undertaken to classify boreholes. In the case of the geophysical data, insufficient amount of energy sources are also indicators of weaker data quality. However, for most applications, it is sufficient when combined with a signal enhancement seismograph with an appropriate processing technique. The sources of geotechnical and seismic data generation (e.g., engineer, consultant, technician or driller) are also qualitative indicators of quality and have to be considered in the evaluation of the site and processing of the data set.

Another important condition to make the engineering data acceptable may be the accuracy of the geographic reference point (spatial coordinate system and map projection) used in identifying the location of the in-situ exploration in regards to the GIS database development (i.e., an acceptable standard coordinate system and map projection should be established).

From the spatial distribution point of view, data may exist in local clusters and there are methods available that account for these particular distributions (e.g., kriging), but preferably one would like to find a data set that is reasonably well distributed such as a grid system. The quantity and distribution of data is important in performing spatial analyses with a point source data set. For example, if the total

number of spatially distributed data points is randomly and sparsely operated, then, the data set is not populated to model the spatial variability with a variogram in a geostatistical process (D'Andria et al., 1995).

In the research area, the variability of the geology (lithologies and facies), especially Quaternary alluvial deposits that pertain to the delineation of geologic hazards needed to be extensively investigated. Because urban areas that are at risk from earthquakes are commonly located in Quaternary alluvial deposits, conventional geologic maps are usually not adequate for evaluating earthquake hazards because most standard geologic maps do not, in themselves, contain sufficient data for these purpose, particularly in areas underlain by unconsolidated sedimentary deposits (Tinsley and Fumal, 1985). Most geologic maps, for example, differentiate bedrock units in considerable detail but only crudely differentiate young, unconsolidated deposits (Lajore and Helley, 1975). For this reason, different sources of information in alluvial and terrace deposits have been collected, organized and digitized to enable the mapping of geologic units in the Ankara basin. Principal sources of information and database that were used in this study included topographic maps, geologic maps, geomorphologic maps, satellite images, and descriptions of excavations (i.e., including water wells, deep borings for bedrock elevation, other boreholes and geophysical surveys). Geologic units were defined and mapped on the basis of geologic and genetic criteria such as geomorphic expression, inferred depositional environment, soil-profile development, and grain size. In the absence of information concerning additional geologic factors that commonly influence site response (i.e., depth to basement rocks, alluvium thickness, vertical variations in sediment type, and so on), these maps could provide a reasonably well first approximation of relative ground response.

Consequently, preparation of the maps showing the areal extent of exposed geologic materials and subsurface data regarding the distribution and thicknesses of concealed geologic materials, the main data sources have been used for the study area include 1:25,000 scale topographic quadrangles of İ29-a₁, -a₂, -b₁, -b₂ and some part of the İ29-a₃, -b₄; 1:100,000 scale geologic map of the Ankara- F15 (No: 55) quadrangle prepared by General Directorate of Mineral Research and Exploration-MTA (1995); 1:50,000 scale geological and engineering geological map of Ankara

prepared by Erol et al. (1980); 6 sheets of 1:25,000 scale geomorphological maps of Ankara prepared by Erol et al. (1980); 1:100,000 scale hydrogeological map of the Ankara Hatip Plain prepared by the General Directorate of State Hydraulic Works-DSİ (1975); ASTER Images taken in 2002 by TERRA Satellite Platform, NASA; and various deep boring studies in determining the bedrock elevation and other geologic units performed and/ or compiled by DSİ (1975), Tabban (1976), MTA (1954) and (2002).

The site characterization of the study area is based on information assembled from databases provided by a number of studies in order to identify the relevant parameters and factors, and to determine the local site conditions for predicting seismic site response. The research area with an areal extent of about 250 km² contains more than about 1000 locations explored by either testing and sampling in boreholes or by in-situ tests that were conducted in previous studies towards the west of the city center of Ankara. The area where the database was mainly compiled includes within the communities (or counties) of Sincan, Etimesgut, Batıkent, Atatürk Orman Çiftliği, Emek, Bahçelievler, Maltepe, Ulus, Kızılay, Söğütözü, METU, etc. These existing databases contained about 1089 boreholes and over 2,000 laboratory samples assembled for the purpose of the engineering geological and geotechnical site investigation study. About 949 of the borings contain Standard Penetration Test results, SPT-N values (blows/meter) from penetrometer studies depths equal to or greater than 20 m and about 140 of the borings contain descriptions of cores generally obtained from deep borehole drilling studies, which are included in the database and given as a summary in Table 4.2. The most of the samples (i.e., samples taken for laboratory purpose, disturbed and/or undisturbed samples taken from borings) contained laboratory test results in different depths. Figure 4.1 shows the spatial distribution of the SPT boring data compiled for the study area. Additionally, many of the existing boreholes were not included in the database because their penetration depth was too shallow (i.e., less than 20 m) and/or they were not reliable (or incomplete) to be used in our database for seismic hazard study. There were also previous geophysical studies, namely, surface wave refraction measurements at 55 locations that have been compiled for the research area. However, these previous studies were local studies performed for different

purposes other than those pursued for this research, and the volume of the data was not sufficient to be used for the entire research area. Because of these reasons, the conducted site characterization study research program particularly concentrated on the Quaternary alluvial and terrace sediments in the Ankara basin. The various sources of the existing databases that have been utilized in this study in regards to seismic hazard assessments are summarized in Table 4.2.

Table 4.2. Summary of the existing database used in this research.

BOREHOLE DATA			
Description	Type	No of Data	Description
Ankara Metro 3 rd Stage (BATIKENT-SINCAN)	SPT	113	Geophysical and Geotechnical Reports
Ankara Metro 2 nd Stage-SOGUTOZU	SPT	24	Geotechnical Report
Ankara Metro 2nd Stage (KIZILAY-CAYYOLU)	SPT	80	Geotechnical Report
Ankaray 3 rd Stage 1 st Lap (STATION)	SPT	10	Geotechnical Report
Dikimevi-ASTI Metro Studies	SPT	61	Geotechnical Report
DLH Train Route Studies	SPT	154	Geotechnical Reports
Sincan Municipality Studies	SPT	180	Boring Logs
Etimesgut Municipality Studies	SPT	62	Boring Logs
Others (37 different studies), Eser (45), Toker (120), etc.	SPT	265	Boring Logs, Geotechnical and Geophysical Reports
Sum	SPT	949	
Various Deep Drilling Studies	CORE	23	Boring Logs-
MTA and METU Drilling Studies	CORE	13	Geotechnical Report
DSI Deep Drilling Studies	CORE	104	Boring Logs and Geophysical Report
Sum	CORE	140	
TOTAL	SPT-CORE	1089	

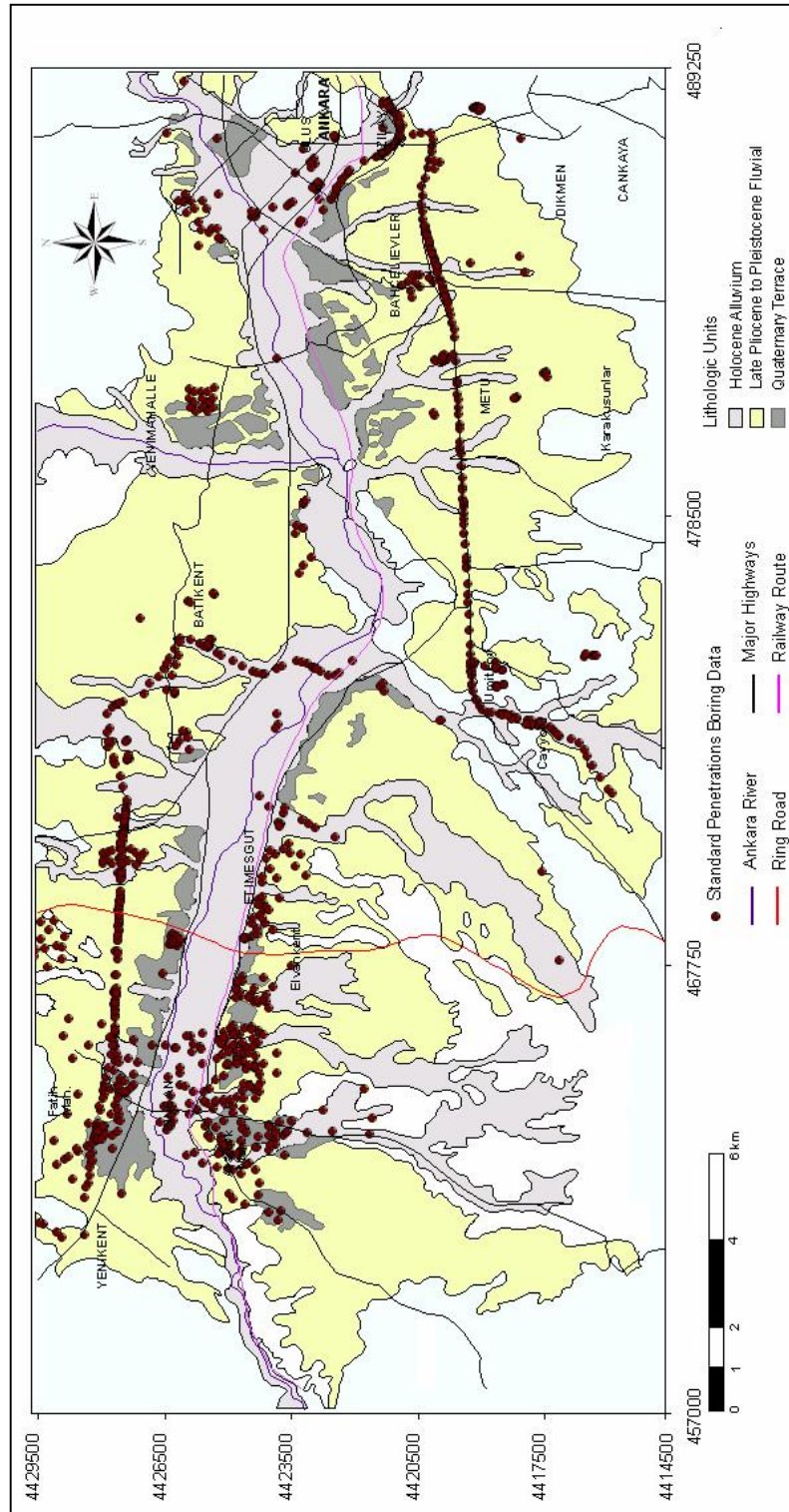


Figure 4.1. The spatial distribution of the SPT boring data compiled from previous studies for the study area in the western part of the Ankara basin.

4.2.2. Construction and Organization of the Database from the Conducted Surface Geophysical Studies Performed in this Research

As mentioned previously, besides the geological, engineering geological and geotechnical data; geophysical data must be supplied as well for building a consistent and thorough database in order to assess the seismic and geotechnical site characterizations and interpret the local site conditions particularly in calculating seismic hazards. This is because the desire to predict site conditions with geophysical testing methods for site characterization and seismic microzonation purposes, has generated considerable effort aimed at clarifying or measuring the soil and rock conditions for ground motion calculations. Besides being an indicator of the potential for site amplification and categorizing geologic units because they are dependent on basic physical properties of the material (i.e., density, porosity, grain size and cementation of sediments). For this reason, in addition to the site characterization study collected from previous existing databases, the surface geophysical site characterization exploration program performed for this research is based on seismic refraction, resistivity surveys and microtremor measurements in an attempt to investigate the engineering geological and seismic properties of the near-surface geologic materials and to identify the local site conditions for predicting seismic site response in the western part of the Ankara basin.

The geophysical testing has been conducted at sites in a spectrum of various geologic materials, the Upper Pliocene to Pleistocene fluvial red clastics and Quaternary alluvial and terrace sediments, which are highly susceptible to seismic hazards. Therefore, near-surface compressional and shear-wave velocities (V_p and V_s) were measured simultaneously by using the seismic refraction surveys at a total of 204 project site locations; and resistivity measurements (vertical electrical sounding, VES) were also performed at 113 of these locations, particularly, in the Quaternary sediments. These surface geophysical testing field studies were conducted through the assistance provided by ESER Technical Boring and Trading Inc. The spatial distribution of the seismic refraction and resistivity measurements performed for the study area were presented in Figure 4.2. The geophysical studies were utilized to complement areas of sparser borehole data, and thus prepare a

consistent and well distributed database for the main purpose of seismic zonation. It needs to be mentioned that in such regional scale geotechnical and seismic characterization studies have not been applied previously in the Ankara basin. As mentioned above, the geophysical characterization study performed with seismic refraction and resistivity surveys were generally conducted on the Pleistocene fluvial red clastics and Quaternary alluvial and terrace sediments in the Ankara basin for calculating seismic hazards that are quite important for the geotechnical and seismic characterization of neotectonically younger sedimentary deposits (i.e., especially Quaternary deposits).

In addition, short-period noise recordings of microtremor measurements were conducted at 352 project site locations on the Upper Pliocene to Quaternary sediments with assistance provided by the General Directorate of Disaster Affairs, Earthquake Research Department in regards to study the seismic response of the Ankara basin and its close vicinity. The spectral ratio between the horizontal and vertical components (H/V ratio) of the microtremor measurements at the ground surface has been used to estimate the fundamental periods and amplification factors of the site. The spatial distribution of the microtremor measurement locations were given by Figure 4.3. The measured frequency values were compared (or correlated) with existing geotechnical and seismic information that were obtained from collected and performed site investigation studies to check the accuracy and hence the reliability of the results. In other words, H/V spectral ratios from the microtremor measurements were performed approximately at the same geographical measurement points as those of the compiled boring penetration data along with the seismic refraction and resistivity surveys on the Upper Pliocene to Quaternary sedimentary deposits in the Ankara basin. It should be noted that a few of H/V spectral ratios of microtremor measurements were also performed at rock to form a “reference site” for estimating the site response for seismic hazard studies.

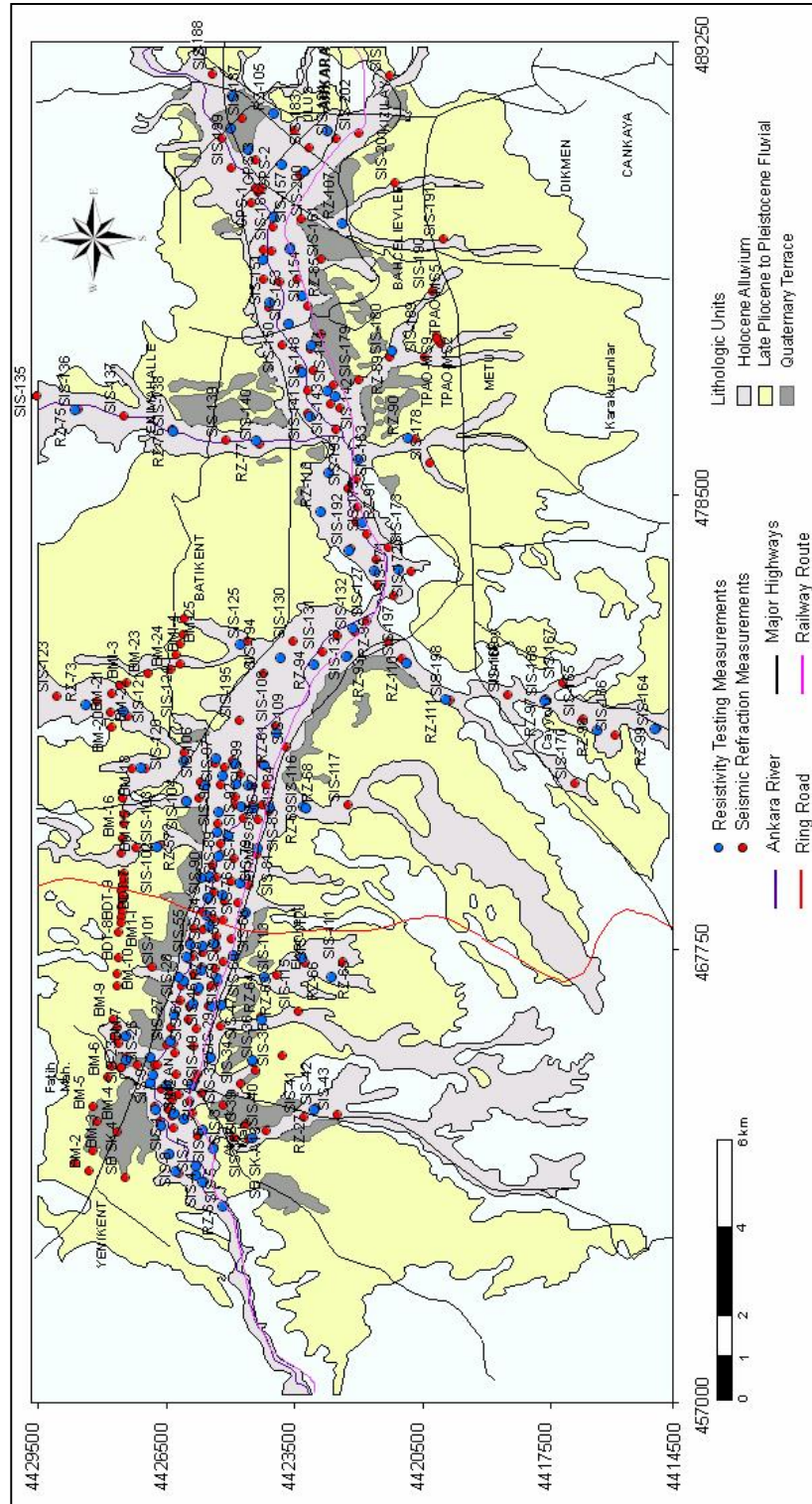


Figure 4.2. The spatial distribution of the seismic refraction and resistivity measurements performed for the study area in the western part of the Ankara basin.

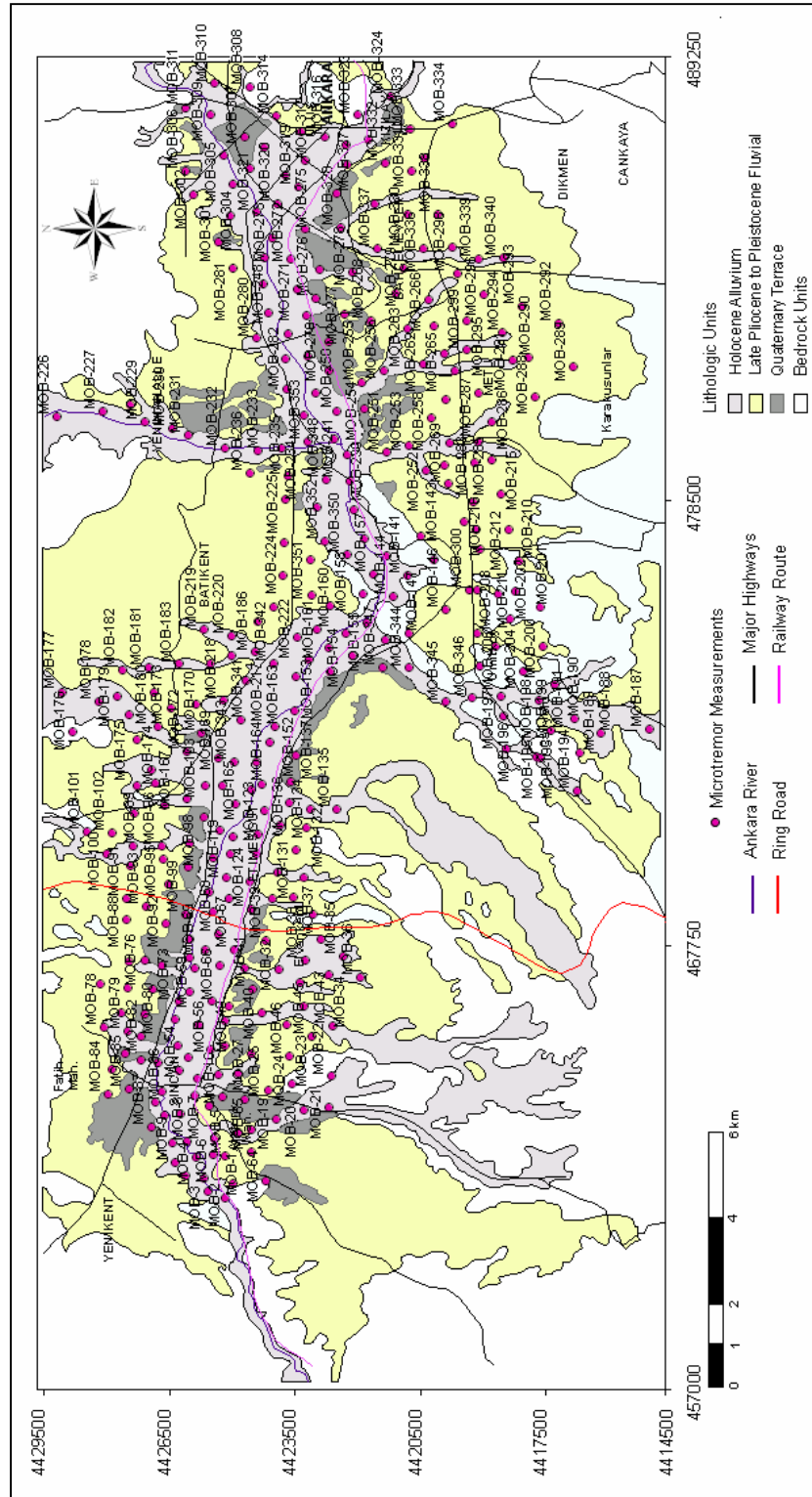


Figure 4.3. The spatial distribution of the microtremor measurements performed for the study area in the western part of the Ankara basin.

4.3. Engineering Geological, Geotechnical and Surface Geophysical In-Situ Methods of Investigations

The method of field investigations, on both a site specific basis as well as a regional basis, relies on the measurement of the geologic features and variability; interpretation, interpolation and extrapolation of the sampled databases obtained from various sources; and assessment of engineering geological and geotechnical parameters of the ground. Arriving at an accurate depiction of the physical world as possible is the desired result. The main objective of this section is to explain and discuss the site investigation methods for identifying and mapping the site characteristics that will later on be used for the assessment of seismic hazards.

Planning and selecting appropriate engineering geological, surface geophysical and geotechnical in-situ investigation is a delicate task that requires knowledge of available methods or techniques. The in-situ test methods utilized for the engineering geological and geotechnical characterization of the western part of Ankara basin are discussed below.

4.3.1. Engineering Geological and Geotechnical In-Situ Testing

4.3.1.1. Introduction

The process of identifying the layers of deposits that underlie a proposed structure and their physical characteristics is the main purpose of the in-situ testing. Generally, the in-situ exploration may be categorized where disturbed or undisturbed samples are collected. On the basis of preliminary borings a decision is made whether to base additional site design information on in-situ tests or to recover undisturbed samples for laboratory tests or in the usual case use a combination (i.e., SPT). In other words, what results at a drill site depends on the drilling technique as well as the sampling and testing procedures being used (Bowles, 1988).

As mentioned previously, the results of the standard penetration test (SPT) at certain locations of the project area has been compiled and used in this research to determine the desired engineering geological and geotechnical design parameters for identifying and mapping the site characters.

4.3.1.2 Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is a widely used dynamic penetration method for in situ characterization of soils. A standard split-barrel sampler assembly for use in SPT is shown in Fig. 4.4 (ASTM D-1586-84). The main objective of using the split-barrel sampler is to obtain the resistance of soil to penetration (N-value), and to obtain representative samples for identification and laboratory tests. The method is applicable to all soil types. It is most often used in granular materials but also in other materials when simple in-place bearing strengths are required. The SPT blow count data are especially applicable to fairly clean medium-to-coarse sands and fine gravels at various water contents and to saturated or nearly saturated cohesive soils. When cohesive soils are not saturated, the penetration resistances may be misleading. Likewise, the engineering behavior of saturated or nearly saturated silty sands may be underestimated by the penetration resistance test (U S Army Corps of Engineers, 2001).

Mainly, field measured SPT blowcount numbers based on standardized equipment and procedures (Seed et al., 1985; Youd and Idriss, 1997 and 2001) as explained below are corrected to account for hammer efficiency, overburden pressure, sampling device, borehole diameter, “short” rod length and other factors. In the standard penetration test, measured $(N_1)_{60}$ blow count values must be normalized to an overburden pressure of approximately 100 kPa (1 atm $\approx$ 2000 psf) and a standard hammer energy ratio or hammer efficiency of 60% is a suitable standard (Skempton, 1986).

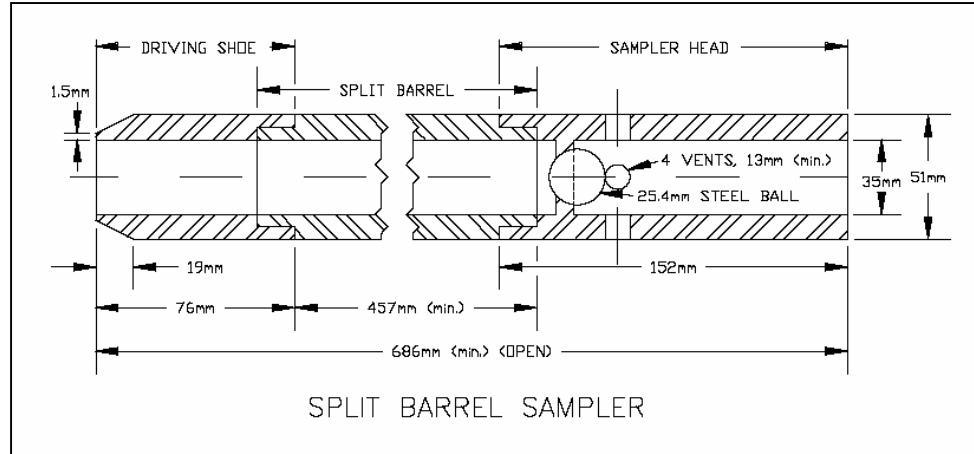


Fig. 4.4. Split-Barrel Sampler Assembly for Use in Standard Penetration Testing. ASTM Specifications (from ASTM D-1586-84).

The standardized penetration resistance $(N_1)_{60}$ is a new “standard” SPT blow count, based on standardized equipment and procedures (Seed et al., 1984 and 1985). The use of other types of hammer (i.e., donut hammers), or other types of mechanisms to raise and drop the hammer (i.e., automatic mechanical “trip” hammers, “free fall” hammers, rope and cathead with three turns of the rope about the cathead, etc.), can impart different levels of energy to the top of the drill stem. These non-standard procedures and equipment require correction of the blow counts in order to develop the standardized blow count. The $(N_1)_{60}$ “standardized” system and procedures, combined with a “typical” rope and cathead system (with two turns of the rope about the cathead) typically deliver approximately 60% of the theoretical “free fall” hammer energy to the drill stem. For other systems, the measured penetration resistances (N , blows/m) should be corrected as

$$N_{60} = N \times ER / 60 \quad (4.1)$$

where ER or energy ratio is the “efficiency” or percent of theoretical free fall energy delivered by the hammer system actually used to the top of the drill stem. This can be measured directly, using a pile analyzer, or can be estimated (for the most

common alternate systems in widespread use) based on correlations and data (Seed and Harder, 1990).

Approximate correction factors ($C_E = ER/60$) to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 4.3. Because of variations in drilling and testing equipment and differences in testing procedures, a rather wide range in the energy correction factor C_E has been observed as noted in the table. Even when procedures are carefully monitored to conform to established standards, such as ASTM D 1586-99, some variation in C_E may occur because of minor variations in testing procedures. Measured energies at a single site indicate that variations in energy ratio between blows or between tests in a single borehole typically vary by as much as 10%. It is recommended that the hammer energy be measured frequently at each site where the SPT is used. Where measurements cannot be made, careful observation and notation of the equipment and procedures are required to estimate a C_E value for use in liquefaction resistance calculations (Youd et al., 2001). Use of good-quality testing equipment and carefully controlled testing procedures conforming to ASTM D 1586-99 will generally yield more consistent energy ratios and C_E with values from the upper parts of the ranges listed in Table 4.3.

Several factors in addition to hammer energy correction influence SPT results, as given in Table 4.3 and Eq. (4.2) incorporates these corrections.

$$(N_1)_{60} = N \times C_N \times C_E \times C_B \times C_R \times C_S \quad (4.2)$$

where N = measured standard penetration resistance; C_N = factor to normalize N to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B = correction factor for borehole diameter; C_R = correction factor for rod length; and C_S = correction for samplers with or without liners.

Table 4.3. Corrections to SPT (Modified from Skempton, 1986 as quoted by Robertson and Wride, 1998 and Youd et al., 2001).

Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P / \sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5–1.0
Energy ratio	Safety hammer	C_E	0.7–1.2
Energy ratio	Automatic-trip Donut- type hammer	C_E	0.8–1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.8
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1–1.3

Because SPT N -values increase with increasing effective overburden stress, an overburden stress correction factor is applied (Seed and Idriss 1982). All N -values may be corrected for overburden effects (N_1) as:

$$N_1 = N \times C_N \quad (4.3)$$

C_N is commonly calculated from the following equation where C_N is taken as (Liao and Whitman, 1986):

$$C_N = (P_a / \sigma'_{vo})^{0.5} \quad (4.4)$$

where C_N normalizes N to an effective overburden pressure (P_a) of approximately 100 kPa (1 atm) and C_N should not exceed a value of 1.7. The effective overburden pressure σ'_{vo} in Eq. (4.4) is the overburden pressure at the time of drilling and testing.

Skempton (1986) suggested and Robertson and Wride (1998) updated correction factors for rod lengths <10 m, borehole diameters outside the recommended interval (65-125 mm), and sampling tubes without liners. Range for

these correction factors are listed in Table 4.3. For liquefaction resistance calculations and rod lengths < 3 m, a C_R of 0.75 should be applied as was done by Seed et al. (1985) in formulating the simplified procedure. Although application of rod-length correction factors listed in Table 4.3 will give more precise $(N_1)_{60}$ values, these corrections may be neglected for liquefaction resistance calculations for rod lengths between 3 and 10 m because rod-length corrections were not applied to SPT test data from these depths in compiling the original liquefaction case history databases. Thus rod-length corrections are implicitly incorporated into the empirical SPT procedure (Youd and Idriss, 2001).

C_S was applied in cases wherein a “nonstandard” (though very common) SPT sampler was used in which the sampler had an internal space for sample liner rings, but the rings were not used. This results in an “indented” interior liner annulus of enlarged diameter, and reduces friction between the sample and the interior of the sampler, resulting in reduced overall penetration resistance (Seed et al., 1984 and 1985). The reduction in penetration resistance is on the order of $\sim 10\%$ in loose soils ($N_1 \leq 5$ blows/ft), and $\sim 30\%$ in very dense soils ($N_1 \geq 30$ blows/ft), so C_S varied from 1.1 to 1.25 which is linearly interpolated in between this range during calculations (Rathje, 2003).

Field SPT measurements, visual examination of samples recovered during the field investigations, results of laboratory testing, and review and interpretation of boring logs, from previous studies comprised the various elements of the liquefaction evaluations performed in regards to the identification of the earthquake hazard potential of the Ankara basin. Details are given in the following chapters.

4.3.2. Geophysical Investigations

4.3.2.1. Introduction

Geophysical investigations which involve study of the subsurface by quantitative physical methods play a preponderant role in near surface

characterization. Both elastic and electromagnetic waves share most of the inherent characteristics in wave phenomena. Geophysical methods can be used to infer engineering design parameters. Hence, strong interrelation exists between the engineering parameters and wave-related measurements. This interrelation highlights the complementary nature of elastic and electromagnetic waves in near-surface characterization. The goal of geophysical surveys is to assess these parameters and their spatial distribution (e.g., stratigraphy, layering, the presence of anomalies, etc.).

4.3.2.2. Seismic waves

Different modes of propagation can be identified by observing the particle motion relative to the propagation direction. There are two major classes of modes of propagation or seismic waves: body waves, which pass through the volume of a material; and, surface waves, that exist only near a boundary.

Body waves, for stress (seismic) waves propagating far from any boundaries in a uniform medium, two fundamental modes of propagation exist: compression waves, also called P-waves, and shear waves, also called S-waves. Since they propagate within the mass or body of the medium, the waves are known as body waves.

Near a typical three-dimensional (3-D) source used in seismic testing, P- and S-wave particle motions are quite complex, with each wave containing “additional near-field” and “far-field” components (Sanchez-Salinero et al., 1986). As the waves propagate away from the source, the additional near-field components decay rapidly, leaving only the far-field components. Nearly all treatments of wave propagation in engineering implicitly ignore the additional near-field components and consider only the far-field components. Typically, a propagation distance of two to four wavelengths is necessary before the far-field component is clearly the dominant component (Bolt, 1976).

The fastest traveling of all seismic waves is the compressional or pressure or primary wave (P-wave). The particle motion of P-waves is extension (dilation) and

compression along the propagating direction. P-waves travel through all media that support seismic waves; air waves or noise in gasses, including the atmosphere, are P-waves. Compressional waves in fluids, e.g. water and air, are commonly referred to as acoustic waves. The second wave type to reach a point through a body is the secondary or transverse or shear wave (S-wave). S-waves travel slightly slower than P-waves in solids. S-waves have particle motion perpendicular to the propagating direction, like the obvious movement of a rope as a displacement speeds along its length. These transverse waves can only transit material that has shear strength. S-waves do not exist in liquids and gasses, as these media have no shear strength. S-waves may be produced by a traction source or by conversion of P-waves at boundaries. The dominant particle displacement is vertical for SV-waves traveling in a horizontal plane. Particle displacements are horizontal for SH-waves traveling in the vertical plane. SH-waves are often generated for S-wave refraction evaluations of engineering sites (U.S. Army Corps of Engineers, 1995).

Elastic body waves passing through homogeneous, isotropic media have well-defined equations of motion. Utilizing these equations, computations for the wave speed may be uniquely determined. Field surveys can readily obtain wave velocities, V_P and V_S usually in meters/second (m/s). A homogeneous, isotropic medium's engineering properties of elastic modulus (E) and shear modulus (G) and either density (p_b) or Poisson's ratio (ν) can be determined, if V_P and V_S are known. Manipulation of equations from Grant and West (1965) yields:

$$n = [(V_P/V_S)^2 - 2] / \{2[(V_P/V_S)^2 - 1]\} \quad (4.5)$$

$$E = p_b V_P^2 (1-2n) (1+n) / (1-n) \quad (4.6)$$

$$G = E / [2(1+n)] \quad (4.7)$$

$$p_b = G/V_S^2 \quad (4.8)$$

Note that these are not independent equations. Knowing two velocities uniquely determines only two unknowns of p_b , ν or E. Shear modulus, G is dependent on two other values.

Two types of surface waves, which exist only at surfaces or interfaces are Love and Rayleigh waves, each of which propagate along the surface of the medium. Traveling only at the boundary, these waves attenuate rapidly with distance from the surface. Surface waves travel slower than body waves. Love waves travel along the surfaces of layered media, and are most often faster than Rayleigh waves. Love waves have particle displacement similar to SH-waves. Rayleigh waves consist of vertical and radial horizontal component, and can exist in an elastic half space. Love waves, on the contrary, consist of a traverse horizontal component alone, and can exist only in a layered half space (Tokimatsu, 1997). Most of the surface waves methods currently available observe Rayleigh waves to which the following discussions are mentioned in brief.

The presence of interfaces alters the particle motion, causing other modes of propagation. In particular, if the medium has an exposed surface, such as the ground surface at a geotechnical site, surface Rayleigh R-waves develop. Rayleigh wave particle motion is a combination of vertical (shear) and horizontal (compression) motions. Near the surface, Rayleigh waves create particle motion that follows a retrograde elliptical pattern. The decay with depth of the vertical and horizontal components of R-wave particle motion is illustrated in Figure 4.5 (Richart et al., 1970). The depth axis is normalized by the Rayleigh wavelength, λ_R . It is representing to see in Figure 4.5 that the horizontal component changes sign at a normalized depth around 0.15. The meaning of this change in sign is that R-wave particle motion changes from a retrograde ellipse to a prograde ellipse in a uniform half-space.

Surface waves are produced by surface impacts, explosions and wave form changes at boundaries. Love and Rayleigh waves are also portions of the surface wave train in earthquakes. These surface waves may carry greater energy content than body waves. These wave types arrive last, following the body waves, but can produce larger horizontal displacements in surface structures. Therefore surface waves may cause more damage from earthquake vibrations. A detailed discussion of surface waves is well presented in the literature that may be found in a number of references (i.e., Woods, 1968; Richart et al., 1970; Achenbach, 1975; Sanchez-Salinero et al., 1986; Tokimatsu, 1997; Stokoe and Santamarina, 2002).

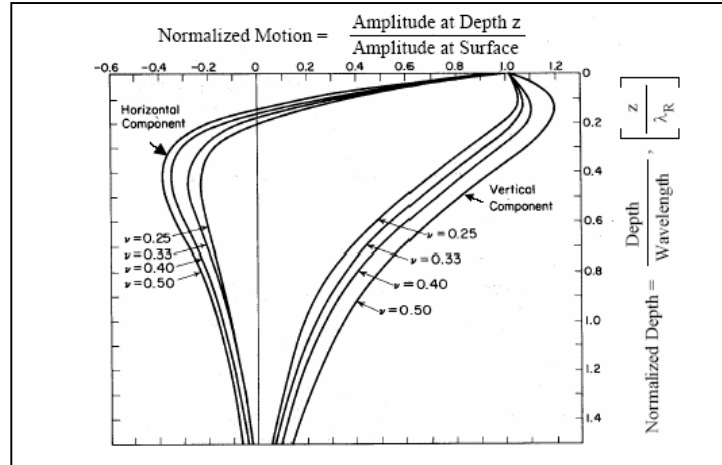


Figure 4.5. Variation in Normalized Particle Motions with Normalized Depth for Rayleigh Waves Propagating Along a Uniform Half Space (Richart et al., 1970)

4.3.2.2.1. Refraction Survey Method

Refraction surveying applied to this study is an established geophysical method for nonintrusively identifying sediment stiffness at depth. Nonintrusive methods eliminate or minimize the time and costs allocated for drilling and in general, effectively cover reasonably large areas. The method is based on the ability to detect the arrival of wave energy that is critically refracted from a higher velocity layer underlying lower-velocity sediment. The test is performed by deploying a linear array of receivers on the ground surface, as shown in Figure 4.6. Refracted arrivals from energy traveling along interfaces below the exposed surface are used to measure the velocities of underlying layers. Although there are several types of seismic refraction methods depending on the survey objectives or targets that can be performed with both shear and compression wave energy, the most common methods are based on the first arrivals of P-waves. Shear wave refraction is more complicated to implement than compression wave refraction because the shear wave is a later wave arrival and may be masked by other wave arrivals in the time record. Therefore, the orientation and polarity of the source must be controlled to ensure that the shear wave is the dominant energy generated and received. It is preferable to

generate and measure horizontally polarized shear (SH) waves to limit the conversion of shear-to-compression waves at soil interfaces which would further complicate the interpretation of wave arrivals (Mediav, 1967).

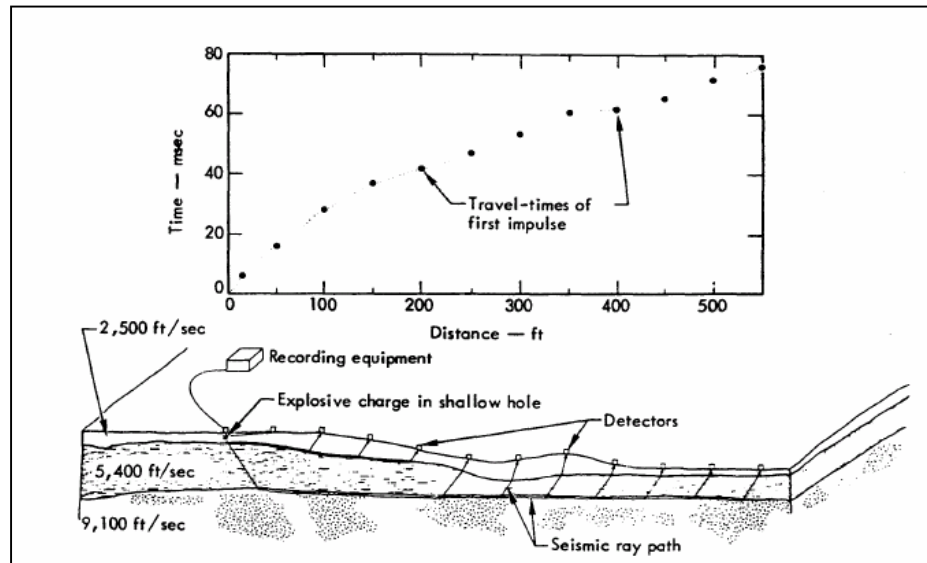


Figure 4.6. Seismic refraction survey testing arrangement (Redpath, 1973)

There are three components in a seismic refraction survey, namely instrumentation, field work and interpretation, which must be well tuned to each other if reliable results are to be obtained. The time data provided by the equipment must have sufficient resolution, the field operations must yield adequate information and the interpretation work must do justice to the field data obtained. A fourth factor affecting the reliability of the results is the positioning of the seismic lines with regard to the geology and the possibilities and limitations of the method (Sjögren, 1984).

The general equipment necessary to carry out a refraction survey includes seismograph and power supply; geophones, geophone cables and geophone extension cables; energy source and associated equipments. Regarding the main principles of seismic instrumentations, the energy required to generate elastic waves

in the ground is produced by an impact, explosives or a mechanical source. The seismic source may be a hammer repetitively striking an aluminum plate or weighted plank, drop weights of varying sizes, a shotgun, a harmonic oscillator, waterborne mechanisms, or explosives. Despite the obvious disadvantages of storage, transportation, and safety, explosives are very good energy sources for refraction work (Institute of Makers of Explosives, 1980 and 1981). Weight-drop and shotgun systems also provide intermediate energy levels (Figure 4.7). However, reference to the energy disturbance for seismic work does not necessarily mean an explosive or shotgun source was used. The type of survey dictates some source parameters. Smaller mass, higher frequency sources are preferable. Higher frequencies give shorter wavelengths and more precision in choosing arrivals and estimating depths. However, sufficient energy needs to be entered to obtain a strong return at the end of the survey line. For instance, in residential or industrial areas perhaps the maximum explosive charge should be limited. The depth of the drilling shot holes for explosives or the shotgun may need to be limited. Therefore, site engineers should be cautious not to exceed requirements of the permits and utility easements and also to choose the alternative sources required (or necessary) for the seismic investigations.

In general, a sledge hammer and striker plates are commonly used for shallow investigations in residential or industrial areas (Figure 4.7). Its basic limitation is in the amount of energy available, but for most applications it is sufficient when combined with a signal enhancement seismograph. Best results are obtained when the striker plate is placed on firm ground and the signal is stacked in the seismograph 5 to 15 times (Mooney, 1981).



Figure 4.7. Field applications of the seismic energy sources, such as sledge hammer and striker plates (a) and weight-drop (b).

The arrivals of the various seismic waves from the impacts are detected by geophones placed in a straight line through the impact points. The geophones that are either accelerometers or velocity transducers transform the mechanical energy into a small electric current, which is transmitted by cables to a recording unit where it is amplified before the signals are recorded directly or the information is stored on magnetic tape. Most geophones are vertical, single-axis sensors to receive the incoming wave from beneath the surface. Some geophones have horizontal-axis response for S-wave, vertical-axis response for P-wave, or surface wave assessments (Figure 4.8). Geophones are chosen for their frequency band response. In seismic-refraction work, low-frequency 20 Hz vertical-motion and 14 Hz horizontal-motion geophones are generally used. Geophone cables come in a variety of lengths with predetermined distances between geophone connections. These cables are designed so that either end may be attached to the seismograph, and the geophone positions are sequentially numbered. Extension cables are similar in design to geophone cables except that no provision is made for connecting geophones. These are used in refraction studies to obtain offsets of the shot point from the first geophone (Crice, 2002).



Figure 4.8. General view of the some of the equipments, particularly geophones, geophone cables and geophone extension cables that are necessary to perform refraction survey in the field, (a). Figure 4.8b shows a close up view of the P (20 Hz) and S wave (14 Hz) geophones.

The equipment that records input geophone voltages in a timed sequence is the seismograph. Current practice uses seismographs that store the channels' signals as digital data in discrete time units. Stacking, inputting, and processing the vast volumes of data and archiving the information for the client virtually require digital seismographs (Sjögren, 1984). The seismograph system may be an elaborate amalgam of equipment to trigger or sense the source, digitize geophone signals, store multichannel data, and provide some level of processing display. These seismographs record the data digitally and are compatible with digital computers. The type of equipment best suited for engineering studies is typically in the middle of this range, a 12- or 24-channel signal-enhancement seismograph (Bullock, 1978 and Haeni, 1988). These seismographs can be used with non-explosive energy sources because they can add the refracted signals from several successive nonexplosive impacts (Figure 4.9). The summation of these signals causes the amplitude of the refracted signal to increase and the random noise to cancel out. The digital measuring equipment for seismic refraction surveying is becoming

increasingly more compact and offers multi-channel recording capability. The main features and components necessary to carry out a seismic refraction survey have previously been briefly indicated in Figure 4.9.



Figure 4.9. A general and close up view of the 12-channel signal-enhancement seismograph (GEOMETRICS, SmartSeis-26325-01) and its' associated equipment (i.e., power supply, integrated extension cables for connecting the geophones and seismic energy source).

The seismic refraction method is generally applicable in situations where the P-wave velocity increases with depth. This is the usual situation in the near surface in a geological site. Though, one drawback of the refraction method is the inability to deal with stiffness or velocity inversions (Nunn and Bostas, 1977). The method depends on the assumption of increasing stiffness with depth, which is not always valid. In other words, the method may not be useful in situations when the seismic velocity does not increase with depth (velocity inversions) and when the velocity and thickness of some layers are such that they do not give rise to seismic first arrivals (the blind zone problem). Velocity inversion problem in the subsurface is one of the most serious limitations that might be creating serious errors in seismic

refraction if it is unrecognized and/ or more sophisticated field and processing procedures, and inverse modeling programs (i.e., SIPT2, SIPQC developed by Scott, 1973; Scott et al., 1993) are not used (Whiteley and Greenhalgh, 1979). A solution to the problem may be obtained from a borehole velocity survey, uphole seismic survey, crosshole seismic survey, or by multiple shooting; bi-directional energy sources.

Theoretically, seismic-refraction methods measure the time it takes for a compressional wave generated by a sound source to travel down through the layers of the earth and back up to detectors placed on the land surface (Figure 4.10). By measuring the travel time of the compressional wave and applying the laws of physics that govern the propagation of compressional waves, the subsurface geology can be inferred. The field data, therefore, will consist of measured distances and seismic travel times. From this time-distance information, velocity variations and depths to individual layers can be calculated and modeled.

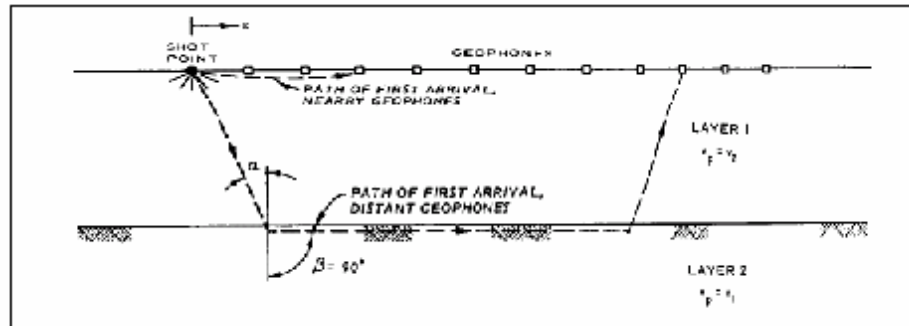


Figure 4.10. Schematic of seismic refraction survey

The foundation of seismic-refraction theory is Snell's Law, which governs the refraction of sound or light waves across the boundary between layers having different velocities. As sound propagates through one layer and encounters another layer having faster seismic velocities, part of the energy is refracted, or bent, and part is reflected back into the first layer (Figure 4.10). When this refracted wave

arrives at the land surface, it activates a geophone and arrival energy is recorded on a seismograph (OYO Corporation, Masuda, 1975).

If a series of geophones is spread out on the ground in a geometric array, arrival times can be plotted against source-to-geophone distances (Figure 4.11), which results in a time-distance plot, or time-distance curve. It can be seen from Figure 4.11 that at any distance less than the crossover distance (X_C), the sound travels directly from the source to the detectors (Parasnis, 1979). This compressional wave travels a known distance in a known time, and the velocity of layer 1 can be directly calculated by $V_1 = x/t$, where V_1 is the velocity of sound in layer 1 and x is the distance a wave travels in layer 1 in time t . Figure 4.11 is a plot of time as a function of distance; consequently, V_1 is also equal to the inverse slope of the first line segment (Redpath, 1973). The intercept time and the crossover distance are directly dependent on the velocity of sound in the two materials and the thickness of the first layer, and therefore can be used to determine the thickness of the first layer (z) (OYO Corporation, Masuda, 1975).

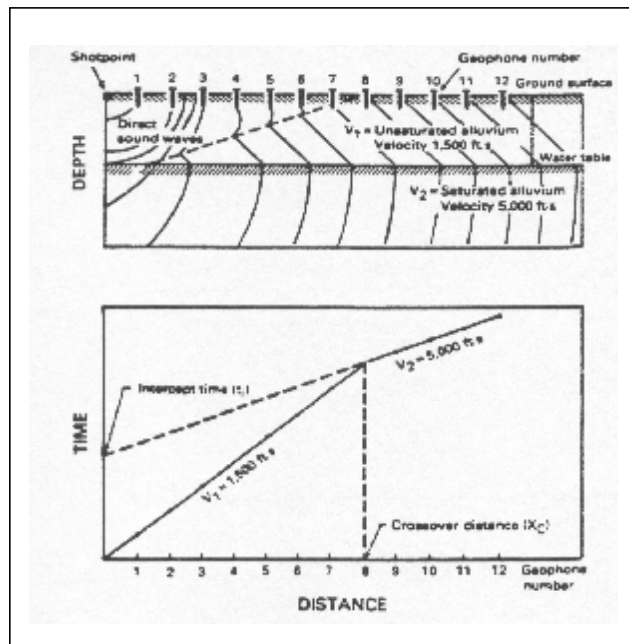


Figure 4.11. Simple two-layer case with plane, parallel boundaries, and corresponding time-versus-distance plot for seismic refraction survey (Redpath, 1973)

Intercept times and crossover distance-depth formulas have been derived in the literature (Grant and West, 1965; Zohdy et. al., 1974; Dobrin, 1976; Telford and others, 1976; Parasnis, 1979; Mooney, 1981). These derivations are straight forward in as much as the total traveltimes of the sound wave is measured, the velocity in each layer is calculated from the time-distance plot, and the ray path geometry is known. The only unknown is the depth to the high-velocity refractor. These interpretation formulas are based on the following assumptions: (1) the boundaries between layers are planes that are either horizontal or dipping at a constant angle, (2) there is no land-surface relief, (3) each layer is homogeneous and isotropic, and (4) the seismic velocity of the layers increases with depth (Haeni, 1988).

For the idealized case shown, the curve representing the direct wave is a straight line with a slope equal to the reciprocal of the velocity of the surface layer V_1 . For those ray paths refracted through the second layer, Figure 4.11 demonstrates that the distance traveled in the surface layer is the same for all geophones.

By consideration of Figure 4.11, two important equations can be identified: one for the intercept time and one for the crossover distance. The expression for travel time in the refracted layer for the case of the plane layer parallel to the surface is given by (Redpath, 1973):

$$T_{sr} = 2D_1 (V_2^2 - V_1^2)^{1/2} / (V_1 V_2) + (1/V_2) D_{sr} \quad (4.9)$$

where T_{sr} = time to travel from source to receiver (beyond the crossover distance); D_{sr} = distance from source to receiver; V_1 or V_2 = velocity of layer 1 or 2; and D_1 = depth to the first, flat lying interface.

By analogy with a straight line whose equation is $y = mx + b$, the intercept time as describe by Dobrin (1976) is the second term in the above equation:

$$T_i = 2D_1 (V_2^2 - V_1^2)^{1/2} / V_1 V_2 \text{ or } D_1 = (T_i/2) (V_1 V_2) / (V_2^2 - V_1^2)^{1/2} \quad (4.10)$$

where T_i = intercept time.

These equations assume knowledge of V_2 which is easily derived from the travel time curve for this case of flat, plane layers only. V_2 is the inverse of the slope of the travel-time curve beyond the crossover distance (see Figure 4.10). Eq. (4.10) can be used for a rudimentary form of interpretation and depth estimation.

The other equation of interest is that for the crossover distance:

$$D_1 = (X_c/2) \{(V_2 - V_1)/(V_2 + V_1)\}^{1/2} \quad (4.11)$$

where X_c = crossover distance; and D_1 = depth to a horizontal refracting interface; and the other variables are defined above.

Eq. (4.11) is most useful for survey design. Note that information about the lower layer is derived from arrivals beyond the crossover distance. Thus, the length of the refraction line must be longer than the X_c indicated by this equation.

Regarding Interpretational Methods, many seismic-refraction interpretation schemes have been developed and published in the literature (Grant and West, 1965; Musgrave, 1967; Scott, 1973; Dobrin, 1976; Palmer, 1980 and 1986) because of the widespread use of seismic-refraction techniques in geologic and engineering studies. They include the standard interpretation approaches, such as Wavefront Construction Methods (intercept-time), the Conventional Reciprocal Method (which is also known as the ABC Method in the U.S., as Hagiwara's Method in Japan, and as the Plus-Minus Method in Europe), Hales' Method, the Generalized Reciprocal Method (GRM) and Ray-tracing Method. Each interpretation scheme has its advantages and, when properly selected and applied, will give satisfactory results. In addition to the standard interpretation approaches, there have been major advances in the acquisition and processing technique. These advances have been driven largely by the spectacular developments in the improved resolution of the instrumentation, data processing methods and computer-based interpretation programs that are available for a wide variety of field problems.

Briefly, the interpretation scheme that has been used successfully in this research study under varying engineering geological and geotechnical field conditions is a computer-modeling procedure based on a delay-time technique modified by Scott et al., 1972 and Scott (1973 and 1977). Scott's program first generates a model of the subsurface using the delay-time technique and then refines the model with a series of iterative ray-tracing procedures (Scott, 1977). The objective of this section is not to attempt to review or summarize the all available interpretation schemes but rather to present the interpretation schemes namely, the Intercept-time Method; Reciprocal or Delay-time Method and Ray-tracing that have been successfully used in our research in the Ankara basin.

Regarding the Time-intercept Methods, the basic equation for the time-intercept method was given in Eqs. (4.9) and (4.10). Eq. (4.11) for the crossover distance can also be solved for depth. This equation can be used to interpret data when, for some reason, the shot initiation time was not recorded (unknown cap delay, etc.).

Regarding the Reciprocal or Delay-time Methods, a specialized interpretation technique which illustrates the versatility of the refraction seismic survey and its direct application to many geological and civil engineering problems is commonly referred to as the Reciprocal (delay-time) Method. The Reciprocal Method is a technique utilizing the amount of calculated travel time required for the wave to traverse the overburden between the surface and the refractor, and is the difference between the hypothetical time which would be measured if the refractor were on the surface and the actual time (Ballard and McLean, 1975). The definition of reciprocal time is the travel time along the refractor from one shotpoint to another shotpoint.

Figure 4.12 illustrates the simple Reciprocal Method. From the figure it can be seen that $T_{AG} + T_{BG} - T_{AB}$ is equal to the sum of the slant times plus a small time corresponding to travel between the two points where the ray paths emerge from the refractor. For flay-lying, near-plane refractors, these times can be converted to a distance by the Eq. (4.12) summarized in U.S. Army Corps of Engineers (1995):

$$Z_G \approx (2V_1/\cos \alpha)(T_{AB} + T_{BG} - T_{AB}) \quad (4.12)$$

where Z_G = distance to the refractor from the geophone G; T_{AB} = travel time from shotpoint A to shotpoint B; T_{AG} = travel time from shotpoint A to geophone G; T_{BG} = travel time from shotpoint B to geophone G; V_1 = velocity of the upper layer and $\cos \alpha$ is given by $(V_{22} - V_{12})^{1/2} / V_2$

Note that the calculation of $\cos \alpha$ requires the value of V_2 (which can not be measured directly from the travel time curves). A satisfactory approximation is given by:

$$V_2 = (2V_{2u}V_{2d}) / (V_{2u} + V_{2d}) \cos d \quad (4.13)$$

where V_2 = approximation to the velocity of the lower medium; V_{2u} = apparent velocity of the lower medium measured up-dip; V_{2d} = apparent velocity of the lower medium measured down-dip; and δ = is the approximate angle of dip of the section, $\delta = (1/2) (\sin^{-1}(V_1/V_{2d}) - \sin^{-1}(V_1/V_{2u}))$.

The angle δ is an approximation of the dip for the whole line. Often the cosine of δ is approximated as 1.0, thus implying low dips. It should be stressed that the primary assumption in the use of intercept-time methods is that “the layer boundary is planar”.

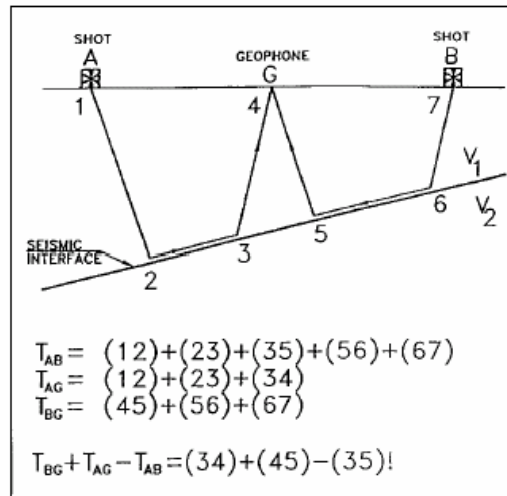


Figure 4.12. Development of simple reciprocal-time method equations

Ray-tracing Methods. Ray-tracing programs usually derive some first approximation of a model based on one of the methods described above. A calculation of the expected arrival time at a geophone based on the starting model is then calculated. This calculation becomes quite involved as the complexity of the model increases. As there is no closed form solution for the calculations, iterative methods of generating ray paths are used and convergence must sometimes be forced as the models become more complex. After the model times have been calculated for the arrivals at the geophones, some form of model adjustment is made which will cause the calculated times to become closer to the observed times. Once the adjustment is made, the process starts over again with calculation of travel times based on the adjusted model. This process is a form of geophysical inversion, for instance production of a geophysical model which accounts for the observations by calculation of the responses of a model and adjustment of that model (Scott, 1973).

Successful geophysical inversions have several general properties. The number of observations is generally several times larger than the number of parameters to be determined (that is, the number of shots and observed travel times is far larger than the number of velocities, layers, and inflection points on the layers). The geophysical model is substantially similar to the geological model being measured (i.e. the approximately flat-lying, low-dip geophysical model is not forced on a geological model of vertical bedding with significant horizontal velocity changes between geophones). Ray-tracing programs using the appropriate approximations necessary for computation on personal computers are available and are stiff competition for programs based on generalized reciprocal methods (U.S. Army Corps of Engineers, 1995).

4.3.2.2.2 Shear Wave Velocity Measurements from Refraction Survey

An important element in establishing seismic design criteria and characterization for an engineering site is the measurement of seismic shear wave velocity (V_s). Shear wave velocity, together with other physical properties of earth materials, can be used to determine their dynamic properties and hence the seismic

response of the ground to theoretical loads caused by local earthquakes. Shear wave velocity measurements are a much more diagnostic tool for engineering properties than P-wave velocities. They are dependent on the shear strength of the material and give a lot more about the character of the material in-situ. Knowing the velocities of the P and S waves and the density of the material, the elastic properties that relate the magnitude of the strain response to the applied stress might be calculated satisfactorily.

The first arrival of seismic energy from a finite energy source is always the compressional wave which is followed by shear wave and other more complex wave types. All seismic energy is composed of elements of compressional and shear wave energy in proportions that are dependent upon the characteristics of the seismic energy source. Because the shear wave travels more slowly than the compressional wave, the time of first shear wave arrival, which is necessary to determine V_s , can be the most formidable obstacle lay in the difficulty in distinguishing the initial motion of the shear waves from the compressional wave (Kanai, 1983). When a compressional wave and shear wave strikes an interface between zones of differing seismic velocities, no less than twelve new wave types are produced. For instance, shallow seismic refraction investigations of in-situ materials showed that the energy from the horizontally polarized shear wave (SH-wave) is usually immersed in a high noise level. This noise presumably comes from side reflections and refractions of the earlier P-wave which, because of the short distances involved, is not widely separated in time from the SH-wave (Whitcomb, 1966; Mediav, 1967). In other words, a compressional wave will produce a refracted compressional and shear wave, a reflected compressional and shear wave and boundary waves of the Rayleigh and Love wave type. A shear wave produces similar wave types. The arrival times of these events add to the confusion of following the initial arrival of shear wave energy.

It is therefore important in obtaining V_s measurements by Impulse Methods to use controlled energy sources that are strong in the development of shear wave energy and weak in the development of compressional wave energy, together with seismic detectors that are particularly responsive to shear wave energy or transverse particle motion. Even with these precautions, the identification of shear wave energy

may be difficult owing to confusion or interference caused by other slow traveling body and surface or boundary waves that have complex wave forms associated with particle motions transverse to the direction of wave travel (Schwarz and Musser, 1972).

There is one unique characteristic of shear wave energy that sets it apart from compressional wave energy and can be helpful in establishing the clear and accurate identification of first shear wave arrival time (McDonal, 1958). This is the fact that the polarity of first shear wave arrival will reverse with the direction of shear wave energy input from a reversible directional energy source whereas the polarity of compressional wave energy will remain the same (Schwarz and Musser, 1972). Thus the two can be readily identified as shown in Figure 4.13. It is therefore important in making V_s determinations that, the energy source is rich in the development of shear wave energy and weak in the development for compressional wave energy, the direction of shear wave energy input should be reversible, and that directional seismic detectors be employed and oriented on an axis transverse to the direction of wave travel and in a plane parallel to the axis of energy input.

The satisfactory results are generally obtained with the Horizontal Traction Method introduced by Kobayashi (1959) and discussed by Warrick (1974). Use of this source overcomes the usual problems in identification of S-wave arrivals due to interference by P-waves as it produces a high proportion of shear to P-wave energy. Since the energy will be limited, signal enhancement seismographs provide the best data as well. It is observed that some spacing will not produce any reasonable arrivals, so the recognition will be easier with multi-channel instruments. Then, multiple element geophone arrays are generally preferred and then employed. Explosives make a poor source of seismic energy for V_s measurements because a large amount of V_p energy is generated and the direction of energy input cannot be reversed (White and Sengbush, 1963). A directional impulse type energy sources and the Horizontal Traction Method such as a sledge hammer striking steel plates or plank are preferred because they generate energy along a single axis that can be oriented transverse to the direction of wave travel and can be easily reversed. This will generally produce refraction data to effective depths of from 20 to 40 meters in

areas that are reasonably free from seismic background noise (Schwarz and Musser, 1972).

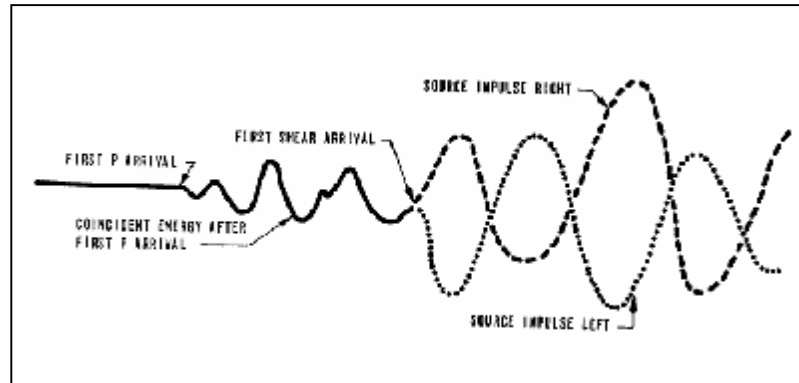


Figure 4.13. Shear waves reverse polarity when the source polarity is reversed. Two superimposed traces of seismic energy produced by reversible directional energy source are used to identify the time of first arrival (modified from Schwarz and Musser, 1972).

The Horizontal Traction Technique which deserves mentioning is a reverse impact, reverse polarity method. It requires a signal enhancement seismograph with a polarity switch on the geophone. Using the traction method, the board should be stroke a few times on one end to stack the data in the seismograph memory. Then the polarity of the geophone should be reversed and the other end of the plank or steel plates should be stroke the same number of times. The V_p waves, and some other extraneous waves, will have the same polarity regardless of the hammer blow direction and tend to cancel out when the geophone polarity is reversed. The shear waves, which reversed polarity at the same time the geophone polarity was reversed, will enhance (Crice, 1980 and 2002). The arrival of first shear wave energy is identified by superimposing the records and observing the time at which the phase of energy arrival reverses as shown in Figure 4.13. A typical travel time curve and interpreted seismic velocity profile developed by this method is shown in Figure 4.14. Both V_p and V_s first arrivals are shown.

The surface refraction method of V_s measurement, the extremely effective mechanism to generate a clean shear wave is simply a wooden plank weighted down

with a vehicle. By hitting the end of the plank with a hammer, a shearing stress is applied to the ground. The shear wave propagates in the direction perpendicular to the plank towards the geophone. Geophones are available with different sensitive axes, usually horizontal or vertical. The geophone is oriented parallel to the plank, in the same axis as the particle motion (Crice, 2002). It will be quite sensitive to the shear waves, and relatively insensitive to any compressive waves. In this figure, horizontal geophone is used but shear waves can be oriented in any direction depending on the polarization of the source. Briefly, this same vehicle on a plank source can be used with multiple geophones that are employed with the sensitive axis of the sensors in a line on a surface and the standard refraction methods and formulas can be used to analyze the data.

Consequently, the seismic refraction method offers a rapid, inexpensive, and accurate method for subsurface exploration and site characterization for an engineering site. Effective measurement of shear wave velocities can be carried on as a routine practice in site investigations. The method has a further advantage that drill holes are not required and a seismic velocity profile produced that can be useful in determining the geologic structure beneath the site. In horizontally or sub-horizontally stratified media, this technique will usually yield V_s values that are somewhat higher than the true average owing to anisotropic seismic characteristics of the formation. Recommendations have been made regarding the desirability of using refraction surveys in conjunction with a boring study. This way, the quality of information gathered from boring investigation will be enhanced if refraction surveys are carried out first and if the results are used to guide the boring operations. Even if some of the refraction data appears complex, ambiguous or just plain peculiar, refraction surveys will have designated areas in which boring methods will provide more useful information. For example, without being combined with borehole surveys, surface shear wave refraction surveys can not accurately predict layered alluvial materials having hidden or unconsolidated layers or lenses, or any other situation where there is likely to be a low velocity layer sandwiched between high velocity layers (i.e. a velocity inversion) as mentioned previously. In that case, alternative seismic borehole surveys methods might be conducted satisfactorily to overcome this issue (i.e., cross hole and down hole

seismic surveys) depending on the purpose of a study because of the high level of expense. In brief, as more use is made of shear wave velocities, the values will become more meaningful to the geological engineering and geotechnical community, and their diagnostic nature will enhance the results of the study when the values are combined with other in-situ tests and boring logs.

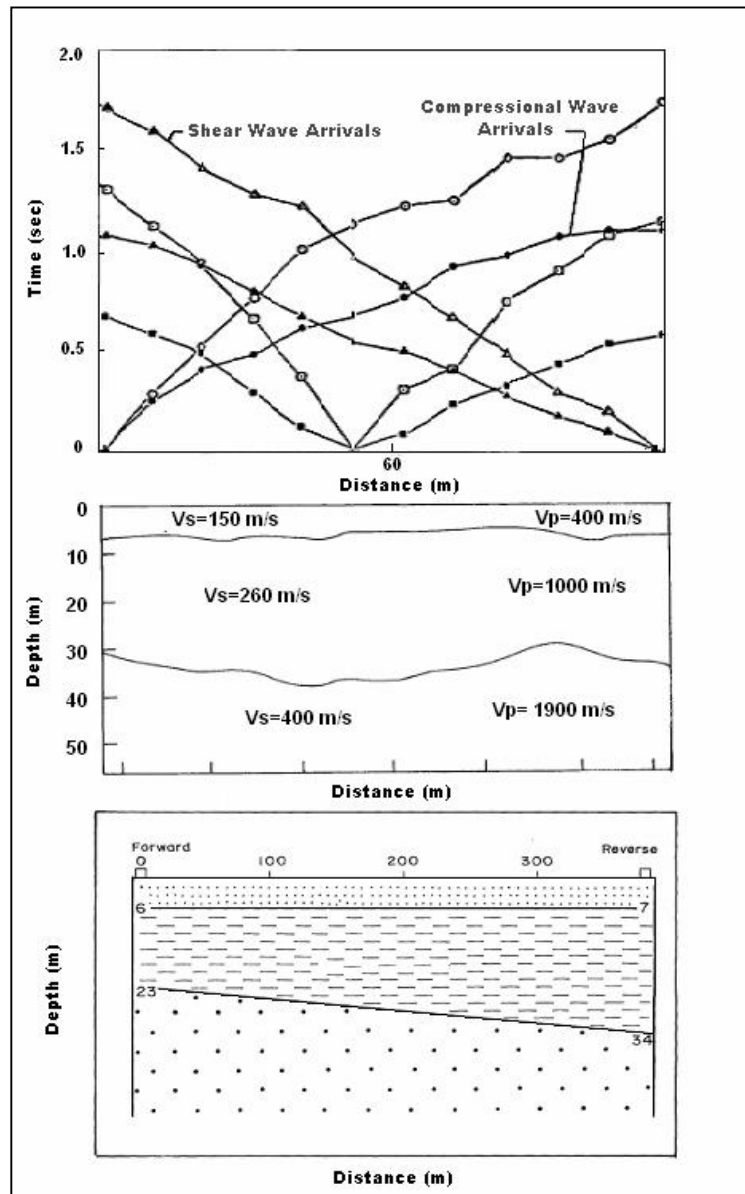


Figure 4.14. An example data processing result of compressional and shear wave velocity profiles and travel time curves from surface refraction survey (travel time curves, depth and ground models, respectively, processed by SIPQC software).

As far as the appropriate instrumentation, field work and interpretation methods of the above mentioned seismic refraction surveys are considered, the site characterization exploration program has been conducted in the western part of the Ankara basin to investigate the engineering geological and geotechnical seismic properties of the near surface geologic materials for predicting the site response. Hence, near-surface compressional and shear-wave velocities (V_p and V_s) were measured simultaneously along the 204 profiles running perpendicular or parallel to the main course of the Ankara River in a spectrum of various sedimentary materials, especially on Quaternary alluvial and terrace sediments in the western part of the Ankara basin (Figure 4.2).

During the field work, seismic refraction measurements were generally performed as long geophone separation profiles ranging from 60 m to 120 m (that resulted in 0-20 m and 0-40 m depth penetrations, respectively) depending on the suitability of the site since occasionally there was lack of space to extend the survey lines especially in the highly urbanized portions in the study area. Survey lines were tried to be extended much longer than the estimated distance of the survey target in order to cover longer penetration depths in the project area. Regarding the instrumentation of the field work, a SmartSeis 12-channel signal-enhancement seismograph was used (GEOMETRICS, 2000). It possesses a 16-bit precision and a 100-dB dynamic range, is a high performance exploration seismograph, applicable to reflection, refraction, borehole and other specialized seismic surveys (Figure 4.9). This seismograph that adds the refracted signals from several successive non-explosive impacts can be used with non-explosive energy sources. The summation of these signals causes the amplitude of the refracted signal to increase and the random noise to cancel out. The digital measuring equipment for seismic refraction surveying is compact and offers multi-channel recording capability. Data processing techniques increasingly employ automated analysis. Considering the suitability of the field work and seismograph, bi-directional energy sources and multiple shooting methods were used along with sledge hammer and striker aluminum plate; and the drop weights for the field studies that are sufficient with these methodologies when combined with the 12-channel signal enhancement seismograph (Figure 4.7). Throughout the seismic-refraction study, Sensor type low-frequency 20 Hz vertical-

motion of P-wave and 14 Hz horizontal-motion of S-wave geophones were used (Figure 4.8).

Regarding data processing techniques, the interpretation scheme that has been used successfully in this research is the Reciprocal time (delay-time) technique that was processed by the aid of inverse modeling programs of SIPQC software (Scott, 1973; Scott et al., 1993). This program generates a model of the subsurface using the delay-time technique and then refines the model with a series of iterative ray-tracing procedures (Scott, 1977). Finally, the data processing results of compressional and shear wave velocity profiles, namely, travel time curves, depth and ground models have been developed, respectively by SIPQC software for the research area of the Ankara basin (Figure 4.14).

4.3.2.3. Electrical Method

4.3.2.3.1 Introduction

Electrical geophysical prospecting methods detect the surface effects produced by electric current flow in the ground, and then analyze the information of geo-electrical difference between layers. Using electrical methods, one may measure potentials, currents, and electromagnetic fields which occur naturally or are introduced artificially in the ground. Furthermore, the measurements can be made in a variety of ways to determine a variety of results. There is a much greater variety of electrical techniques available than in the other prospecting methods, where only a single field of force or anomalous property is used (Telford et al. 1976). Basically, however, it is the enormous variation in electrical resistivity found in different rocks and minerals which makes these techniques possible.

There are several of electrical prospecting methods and many ways of classifying them. In general, the most commonly used techniques are the (direct current) resistivity method, self-potential method, induced polarization method, etc. Among them, the resistivity method is the most widely used method for solving

geological, geotechnical and geo-environmental problems. Furthermore, it can be used in various stages of a geological engineering and civil engineering projects varying from reconnaissance through site investigation to maintenance. In this section of this chapter, the resistivity method that is applied to our research will be introduced as an electrical prospecting method.

4.3.2.3.2 Resistivity Methods

Surface electrical resistivity surveying is based on the principle that the distribution of electrical potential in the ground around a current-carrying electrode depends on the electrical resistivities and distribution of the surrounding soils and rocks. Basically, field equipment for the measurements consists mainly of the resistivity meter (usually consisting of a power supply, a transmitter and a voltage receiver), electrodes, and cables.

The resistivity of soils and rocks is governed primarily by the amount of pore water, its resistivity, and the arrangement of the pores. To the extent that differences of lithology are accompanied by differences of resistivity, resistivity surveys can be useful in detecting bodies of anomalous materials or in estimating the depths of bedrock surfaces. In some cases, especially alluvium strata, within the depth of interest, proved to be more electrically differentiated than elastically due to their heterogeneous nature (Zohdy, 1965). In coarse granular soils, the groundwater surface is generally marked by an abrupt change in water saturation and thus by a change of resistivity. In fine-grained soils, however, there may be no such resistivity change coinciding with a piezometric surface. There are wide ranges in resistivity for any particular soil or rock type, and resistivity values cannot be directly interpreted in terms of soil type or lithology. Commonly, however, zones of distinctive resistivity can be associated with specific soil or rock units on the basis of local field or boring hole information, and resistivity surveys can be used profitably to extend field investigations into areas with very limited or nonexistent data. Also, resistivity surveys may be used as a reconnaissance method that can be further

investigated by complementary geophysical methods and/or drill holes (U.S. Army Corps of Engineers, 1995).

In the resistivity surveys, the most appropriate method should be selected from vertical sounding, horizontal profiling, 2-D profiling and other available methods taking into consideration the scope and the stage of the survey. In the selection of the survey method, topographical and geological conditions, cost and operational efficiency should also be taken into consideration. Figure 4.15 shows a schematic diagram of the measurements involved in the resistivity method. The method is based on transmitting current into the ground through electrodes C1 and C2, and measuring the electrical potential with electrodes P1 and P2 to determine the electrical resistivity of the subsurface of the ground (SEG Japan, 2000).

Considering the purpose of our study in an account for the geological conditions, cost and operational suitability, Vertical Sounding was deemed the most appropriate survey method when compared to the other 1-D survey methods. In the Vertical Sounding Method, resistivity measurements are repeated using a set of electrodes with the same midpoint but with different intervals. Since a larger electrode interval corresponds to a deeper penetration depth, a one-dimensional (1-D) resistivity model at the measurement location is obtained with the Vertical Sounding Method.

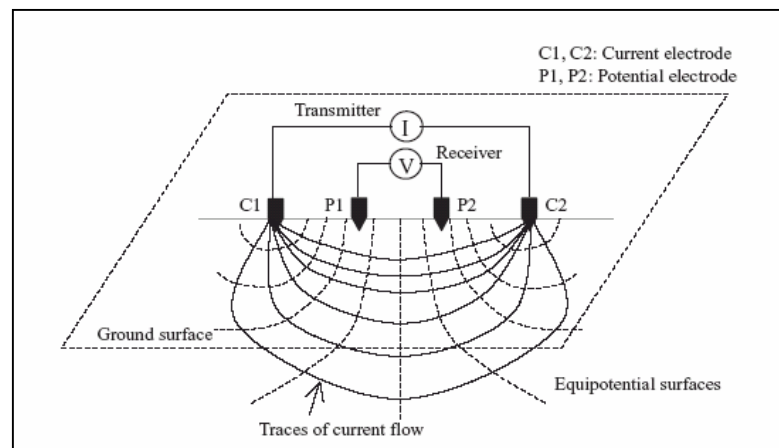
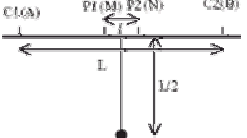


Figure 4.15. A schematic diagram of measurements involved in the resistivity method (SEG Japan, 2000). Note that equipotentials and current lines for a pair of current electrodes C1 and C2 on a homogeneous half-space

The optimum electrode configuration, namely Schlumberger array of vertical sounding was selected by considering the configurations of the four electrodes, geological and surface conditions along with the desired output (Table 4.4). The Wenner array (that might be used for Schlumberger array Vertical Sounding) and the dipole-dipole array are suitable for horizontal profiling. These electrode configurations for horizontal profiling are commonly employed for 2-D profiling (Takahashi, 2004).

The maximum electrode spacing is determined from the target depth and the electrode configuration used. A survey line should be much longer than the estimated depth of the survey target in order to cover the investigated area not only horizontally but also vertically. Typically, maximum electrode spacing of three or more times the depth of interest is necessary to assure that sufficient data have been obtained. The minimum spacing is determined from the required spatial resolution. In horizontal profiling and 2-D profiling, requirements for both depth of investigation and resolution cannot be satisfied simultaneously. Therefore, they are usually determined bearing in mind the survey objectives, cost and operational efficiency.

Table 4.4. Electrode (array) configuration factors for various electrode arrays in resistivity measurements (After Takahashi, 2004)

Array type		Image point of apparent resistivity	Electrode configuration factor
4-electrodes	Wenner		$2\pi a$
	Dipole-dipole		$2\pi a(1+n)(n+2)a$
	Schlumberger		$\pi(L^2 - L^2)/4L \{L > 5L/2\}$

As a theoretically, data from resistivity surveys are customarily presented and interpreted in the form of values of apparent resistivity ρ_a . An equation giving the apparent resistivity in terms of applied current, distribution of potential, and arrangement of electrodes can be arrived at through an examination of the potential distribution due to a single current electrode (Van Nostrand and Cook, 1966). The effect of an electrode pair can be found by superposition. If a single point electrode, located on the boundary of a semi-infinite, electrically homogeneous medium, which represents a fictitious homogeneous earth is to be considered if the electrode carries a current I , measured in amperes (a), the potential at any point in the medium or on the boundary is given by (Keller and Frischknecht, 1966)

$$U = r \frac{I}{2\rho r} \quad (4.14)$$

where, U = potential, in volts (V); ρ = resistivity of the medium, in ohm-meters; and r = distance from the electrode, in meters

For an electrode pair with current I at electrode $C1$, and $-I$ at electrode $C2$ (Figure 4.15), the potential at a point is given by the algebraic sum of the individual contributions:

$$U = \frac{rI}{2\rho r_{C1}} - \frac{rI}{2\rho r_{C2}} = \frac{rI}{2\rho} \left[\frac{1}{r_{C1}} - \frac{1}{r_{C2}} \right] \quad (4.15)$$

where r_{C1} and r_{C2} = distances from the point to electrodes $C1$ and $C2$

In addition to current electrodes $P1$ and $P2$, Figure 4.15 shows a pair of electrodes $P1$ and $P2$, which carry no current, but between which the potential difference V may be measured. Following the previous equation, the potential difference V may be expressed as

$$V = U_{P1} - U_{P2} = \frac{rI}{2\rho} \left[\frac{1}{C1P1} - \frac{1}{C2P1} + \frac{1}{C2P2} - \frac{1}{C1P2} \right] \quad (4.16)$$

where U_{P1} and U_{P2} = potentials at P1 and P2; and $\overline{C1P1}$ = distance between electrodes C1 and P1,... etc.

These distances are always the actual distances between the respective electrodes, whether or not they lie on a line. The quantity inside the brackets is a function only of the various electrode spacings. The quantity is denoted as $1/K$, which allows rewriting the equation as

$$V = \frac{rI}{2p} \frac{1}{K} \quad (4.17)$$

where K = array geometric factor and Eq. (4.17) can be solved for ρ to obtain

$$r = 2p K \frac{V}{I} \quad (4.18)$$

The resistivity of the medium can be found from the measured values of V , I , and K , the geometric factor. K is a function only of the geometry of the electrode arrangement.

Wherever these measurements are made over a real heterogeneous earth, as distinguished from the fictitious homogeneous half-space, the symbol ρ is replaced by ρ_a for apparent resistivity. The resistivity surveying problem is, reduced to its essence, the use of apparent resistivity values from field observations at various locations and with various electrode configurations to estimate the true resistivities of the several earth materials present at a site and to locate their boundaries spatially below the surface of the site (Zohdy, 1974). An electrode array with constant spacing is used to investigate lateral changes in apparent resistivity reflecting lateral geologic variability or localized anomalous features.

As mentioned previously, the Schlumberger array is the most convenient Vertical Sounding Method that is considered as an appropriate method for the purpose of this research. Figure 4.16 shows a general view of the Schlumberger

array Vertical Sounding Method performed in the field. It should be noted that this method is more flexible, less time consuming and suitable for being correlated with the complementary geophysical and engineering geological studies that will be mentioned below.

For the Schlumberger array, in the limit as l approaches zero, the quantity V/l approaches the value of the potential gradient at the midpoint of the array. In practice, the sensitivity of the instruments limits the ratio of L to l and usually keeps it within the limits of about 3 to 30. Therefore, it is a typical practice to use a finite electrode spacing and Eq. (4.16) to compute the geometric factor (Keller and Frischknecht 1966) that was shown in Table 4.4. The apparent resistivity is:

$$r_a = \rho \left[\frac{(L/2)^2}{l} - \frac{l}{4} \right] \frac{V}{I} = \rho \left[\frac{L^2 - l^2}{4l} \right] \frac{V}{I} \quad (L > 5l/2) \quad (4.19)$$

Vertical electrical sounding (VES) surveys with the Schlumberger array are performed with a fixed center point. An initial spacing L (the distance from the center of the array to either of the current electrodes) is chosen, and the current electrodes are moved outward with the potential electrodes fixed. The spacing l must never be larger than $0.4L$ or the potential gradient assumption is no longer valid (Cook and Gray, 1961; Van Nostrand and Cook, 1966). Potential electrode spacing is therefore determined by the minimum value of L . As L is increased, the sensitivity of the potential measurement decreases; therefore, at some point, if L becomes large enough, it will be necessary to increase the potential electrode spacing (Van Nostrand and Cook, 1966). For VES, it is common to create an apparent resistivity versus electrode spacing curve by drawing apparent resistivity values against the corresponding electrode spacing on a full logarithmic section paper. Also, it is possible to estimate resistivity and depths of boundaries of subsurface formations from the VES curve. Data processing was previously conducted by the graphical analysis method using master curves. However, it is now commonly performed by 1-D inversion on a computer program (i.e., WinSev-6, IPES-5t). To avoid misinterpretation of the processed resistivity image, it is important to understand

restrictions and limitations of the inverse method. It is recommended to refer to additional or other information (i.e., geology, boring logs, etc.) regarding the survey site. It is interesting to note that the interpretation of the Schlumberger array curve is much closer to the boring data instead of the other array methods (Zohdy, 1965).



Figure 4.16. A general view of the Schlumberger array Vertical Sounding Method performed in the field.

Some typical ranges of resistivity values for man made materials and natural minerals and rocks, might be found in the literature (van Blaricon 1980; Telford et al. 1976; Zohdy, 1974 Zohdy et. al, 1969; Imai, 1972; Keller and Frischknecht 1966). The ranges of values shown are those commonly encountered but do not represent extreme values. It may be inferred from the values presented that the user would expect to find in a typical resistivity survey low resistivities for the soil layers, with underlying bedrock producing higher resistivities. For instance, lower resistivity generally indicates higher clay content or higher water content (i.e., the porosity multiplied by water saturation) (Takahashi, 2004). Usually, this will be the case, but the particular conditions of a site may change the resistivity relationships. For instance, coarse sand or gravel, if it is dry, may have a resistivity like that of igneous rocks, while a layer of weathered rock may be more conductive than the soil overlying it (Zohdy, 1974). In any attempt to interpret resistivities in terms of soil types or lithology, consideration should be given to the various factors that affect resistivity. Furthermore, the value of the specific resistivity of each layer is a

reference datum to identify various characters of this layer. For example, it serves for detection of aquifers, estimation of the depth of weathering of the bedrock, evaluation of porosity in the layers, etc. Some typical ranges of resistivity values of particular interest to this research are given by Figure 4.17 (Imai, 1972). This figure shows the value range of specific resistivity of some rocks and soils, for dry and wet rock/soil conditions.

Apart from the useful explanations, however, the electrical resistivity method has some inherent limitations that affect the resolution and accuracy that may be expected from it. Like all methods using measurements of a potential field, the value of a measurement obtained at any location represents a weighted average of the effects produced over a large volume of material, with the nearby portions contributing most heavily (Van Nostrand and Cook, 1966). This tends to produce smooth curves, which do not tend themselves to high resolution for interpretations. There is another feature common to all potential field geophysical methods; a particular distribution of potential at the ground surface does not generally have a unique interpretation. While these limitations should be recognized, the non-uniqueness or ambiguity of the resistivity method is scarcely less than with the other geophysical methods. For these reasons, it is always advisable to use several complementary geophysical testing (i.e., seismic field testing, particularly P waves) along with engineering geological testing (i.e., boring logs) in an integrated exploration program rather than relying on one single exploration method.

Regarding the appropriate instrumentation, field work and interpretation methods of resistivity survey in relation to the site characterization studies of the near-surface geologic materials of the western part of the Ankara basin, the electrical resistivity survey using vertical electrical sounding (VES) that has proved to be cost-effective and operationally suitable for the investigation of horizontally stratified structures, has been conducted almost concurrently with the seismic refraction surveying (at or nearby locations) at 113 stations, distributed along the 113 profiles perpendicular or parallel to the main course of the Ankara River in a spectrum of various sedimentary materials, especially in Quaternary alluvial and terrace sediments (Figure 4.2). During the field operations, resistivity results were measured by OYO McOhm 2115A type electrical resistivity instrument and its associated

equipments (Figure 4.18). The Schlumberger array vertical sounding method that is more flexible, less time consuming and suitable for being correlated with complementary seismic and engineering geological studies is considered as an appropriate field method for this research. The maximum electrode ($\overline{C1C2/2}$) spacing of the Schlumberger array, generally long electrode separation ranged from 100 m to 175 m depending on the suitability of the site because occasionally there was lack of space to extend the survey line in the highly urbanized portion of the research site. Survey lines were tried to be extended much longer than the estimated length of the survey target in order to cover a longer depth of the investigated area not only horizontally but also vertically in order to obtain sufficient amount of data. Consequently, the vertical sounding data was processed by 1-D inversion on computer programs of WinSev-6 and IPES-5t to determine the true resistivity and depth penetration values of the lithological layers for being utilized in the site characterization studies of the Ankara basin. In the interpretation of the theory, there are some limitations and disadvantages that were discussed previously. Hence, to avoid misinterpretation of the processed geoelectrical resistivity measurements, it is always advisable to use several complementary geophysical and engineering geological testing in an integrated exploration program which will be mentioned in Chapter 4.

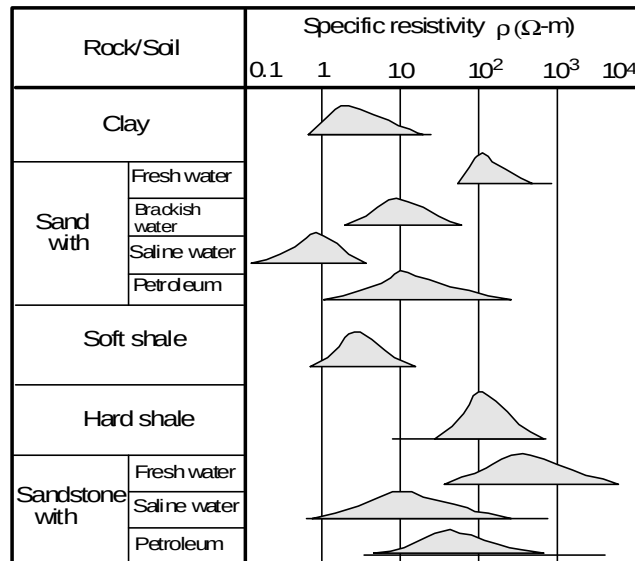


Figure 4.17. Typical specific resistivities of rock and soils (After Imai, 1972)

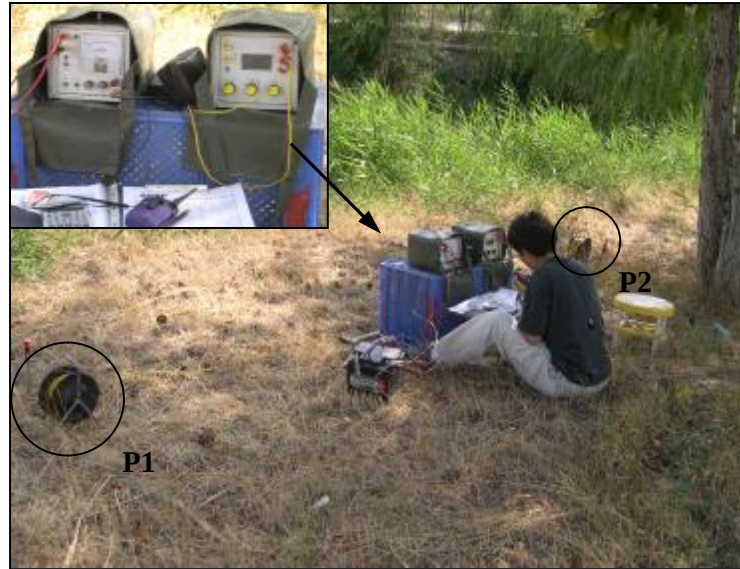


Figure 4.18. General and close up view of the OYO McOhm 2115A type electrical resistivity instrument and its associated equipments from the field measurements at the research site.

4.3.2.4. Microtremor Measurements

4.3.2.4.1. Introduction

Several studies have documented that near-surface sedimentary deposits can significantly amplify seismic waves and thereby increase the level of damage experienced during an earthquake (e.g., Singh et al., 1988; Borchert et al., 1989). Recent destructive earthquakes have clearly shown that near-surface geological and topographical conditions that can generate significant amplification and spatial variations of the earthquake ground motion play a major role in the level of ground shaking and in gathering information on soft soil response. It is therefore highly desirable to develop methods for identifying and characterizing regions prone to this type of site amplification. There are several methods to determine a subsurface structural model over a wide area, but these explorations are still economically and technically difficult to make, especially in urbanized areas. On the other hand, the

microtremor method introduced by Kanai (1957), being a relatively easy and economically attractive method for urban areas, involves using ambient seismic noise to evaluate the sediment-amplification potential. It is utilized to estimate site response and assess the effect of soil conditions under earthquake excitations in the field of earthquake engineering (Lermo, 1994; Gaull, 1995).

4.3.2.4.2. Techniques Used for the Analysis of Microtremors

As a general, the surface layers are normally exposed to tremor by natural forces (storm, sea waves, etc.) and artificial forces (plant, automobile, train, etc.). Sea waves induce a tremor of a relatively long period (2-3 sec. or more), so-called a microseisms, while the storm and artificial forces induce a tremor of a short period, so-called a microtremor (Nakamura, 1989). In particular, long-period microtremors, or microseisms, recorded with high magnification instruments (Richter, 1958), are related to deeper soil structure extending down to the basement rock. They permit a rough estimate of site effects and that the amplitude of spectral ratio of microtremors at a soft soil site, relative to a firm rock site, correlates well with thickness of sediments. Thus, simultaneous observation of long-period microtremors can be a practical tool in earthquake engineering for the evaluation of wave amplification due to deep soil deposits (Kagami et al., 1982 and 1986). Short-period microtremors are very low-amplitude oscillations of the ground surface to use records of ambient seismic noise. They are combination of various types of waves, from many natural sources such as traffic, industrial machinery, small magnitude earth tremors, and movement of objects triggered by wind, all kinds of waves can be observed, depending also on the type of seismograph and on the method of analysis (Kanai, 1966). The seismograph which has more than about a thousand times magnification, records the ground motions continuously in an ordinary place. Usually, the maximum amplitudes of the motions, from 0.1 to 1 micron, and the ranges of their periods are from 0.1 sec to 1 up to 2 seconds (Kobayashi, 1991). The short-period microtremors are related to shallow subsurface structures several tens of meters thick (Kanai, 1983).

The application of short-period microtremors to estimate site effects has been investigated for many years by Kanai and Tanaka (1961); Kanai (1983); Kobayashi et al. (1986). They assume that the microtremor horizontal motions at short-periods consist mainly of shear waves, and that the spectra of the horizontal motions reflect the transfer function of the ground at site. And also, they consider it possible to estimate dominant period and amplification level of soft sediments by measuring directly the dominant period of microtremors and its maximum amplitude in microns. This approach, with some variants, has been used to characterize site effects in a wide variety of seismic environments (e.g., Kanai and Tanaka, 1954; Kobayashi et al., 1986; Lermo et al., 1988; Field et al., 1990; Finn, 1991). However, there is still some discussion about the applicability of short-period microtremors to determine fundamental frequency of soft soil sites, mainly because of difficulties in discriminating source effects from pure site effects in noise recordings (Lermo and Chavez-Garcia, 1999) and discrepancies between noise and earthquake recordings (Udwadia and Trifunac, 1973; Aki, 1988). Some typically oriental subtleties allowing a sound use of microtremors could probably not be perceived, somewhat blunt, rationality (Bard, 1999).

In spite of the problems related to their interpretation, microtremor measurements provide a very interesting approach to site effect evaluation by virtue of the simplicity of the analysis, rapidity of field operations requiring only several minutes of recording at each station and low cost noise measurements that are consistent with strong motion observations. Especially, the ability of the H/V spectral ratio is to provide satisfactory estimates of the site response of soft deposits.

In the microtremor measurements, the standard equipment generally used comprises a three-component velocity or acceleration seismometer (velocimeter or acceleromeler) with amplifier and PC as a data recorder. Natural periods of sensors used are based on the period dependency (sensors are generally 1.0 s or longer than 1 s. especially for short period of range) of microtremors considering the purpose of the study. As mentioned before, the short-period microtremors are very low-amplitude oscillations of the ground surface to use records of ambient seismic noise, whereas long-period microtremors recorded with high magnification instruments, which is related to deeper soil structure down to a depth of the basement rock.

Microtremors have been used basically in four different ways in relation to site conditions: direct interpretation of Spectral (Fourier) amplitudes or power spectral density (e.g., Kanai and Tanaka, 1954; Katz, 1976); computation of spectral ratios relative to a firm site reference station (Borcherdt, 1970; Ohta et al., 1978; Kagami et al., 1982 and 1986; Singh et al., 1988; Field and Jakob, 1990; Chavez-Garcia et al., 1990; Seo and Samano, 1992; Field et al., 1995, Lachet et al., 1996; Ibs-von Seth and Wohlenberg, 1999); computation of spectral ratios between horizontal components of motion relative to the vertical component obtained at the same site (Lermo and Chavez-Garcia, 1992 and 1994; Field and Jacob, 1993 and 1995; Lachet and Bard, 1994; Tokimatsu, 1997; Bard, 1998; Konno and Omachi, 1998; Nakamura, 1989, 1997 and 2000), and finally velocity structure inversion through array recordings. The first three techniques imply an interpretation of the nature of microtremors and have proven to be useful in some cases. A common feature of these techniques assumes that site effects are due to a single soft soil layer overlying an elastic half-space. Thus, site effect is defined by a resonant period and an amplification level given by the impedance ratio between the layer and the half-space. In order to evaluate the ability of each technique to predict dominant period and amplification level, we must have an independent estimate of site effects. This was taken to be the spectral ratios of the intense wave part of weak or strong motion records obtained at the same sites. The first three techniques have a common feature in that they directly provide some information on the site response, while the fourth one is basically a geophysical exploration technique, leading only indirectly to site response estimates. This assumption has been studied by several authors (Aki, 1957; Akamatsu, 1961; Nogoshi and Igarashi, 1970, 1971; Udawadia and Trifunac, 1973; Irikura and Kawanaka, 1980; Horike, 1985; Matsushima and Okada, 1990; Tokimatsu and Miyadera, 1992; Milana et al., 1996; Chouet et al., 1998; Tokimatsu, 1997; Tokimatsu et al., 1998; Konno and Omachi, 1999; Arai and Tokimatsu, 1998 and 2000; Kudo, 2002). They showed the large proportion of surface waves in noise, and proposed a few techniques (Spatial Auto-correlation method, or f-k analysis, or even H/V ratio) to process noise recordings based on the assumption that they consist mainly of surface waves. The good success of these specific processing techniques in providing velocity profiles is indeed an indirect proof that the surface

wave assumption is probably to be preferred. We will not consider array measurements of microtremors, which require more elaborate field operations and more sophisticated processing.

Regarding the purpose of the research and the methodology that was conducted in the study area of the Ankara basin, spectral ratios with respect to a reference site will be briefly reviewed, especially as the H/V spectral ratio method of microtremors, which have been used to provide information on the soil conditions and site response is extensively mentioned here.

Spectral Ratios Relative to a Reference Site: the spectral ratio technique is very popular for the analysis of weak or strong motion records, where it is easy to identify a common window in both the reference and the soft soil seismograms. Usually, the intense S-wave part of the seismogram is extracted with a tapering window and the ratio of smoothed amplitude Fourier spectra is taken to be the transfer function between the soft soil site and the reference station (site) (Lermo and Chavez-Garcia, 1994). The reference site is generally chosen as a rock or very stiff site. This technique has been applied to microtremor measurements by Kagami et al. (1982 and 1986), Gaull et al., (1995), Field (1996), Lachet et al. (1996), Ibs-von Seth and Wohlenberg (1999) with good results and by Seo (1992), Seo and Samano (1992), and Field et al. (1995) with negative results. In computing spectral ratios, it is not required that the spectrum of the excitation be flat. In contrast, a necessary assumption is that the motion recorded at the reference station is representative of the excitation arriving at the interface substratum/sediments, under the soft soil site. Before supporting optimistic conclusions, a great care is to be attached to the measurements and processing.

The application of the spectral ratio technique to microtremor records presents a basic problem where it is very difficult to identify a common wave train for the two stations involved. There are two possible solutions to this problem. One is to select data windows using absolute time, without regard to the appearance of the seismogram. The second is to assume that an average of many windows for a given site is representative of its motion at any time. Before supporting reliable conclusions, a great care is to be attached to the measurements and processing. For any experiment, at least one, and preferably several sites should be selected for

continuous measurements throughout the whole duration and the interstation distance should be adapted to the frequency band of interest. For any experiment, it is certainly safer to select a few sites where the microtremor spectral ratios are to be checked against earthquake spectral ratios. The microtremor result may then be used for a calibrated interpolation of the earthquake results (Bard, 1999).

In addition, to estimate either the corresponding amplification, or the bandwidth over which amplification is to be expected for actual earthquakes, the installation of a few temporary stations in order to obtain several earthquake recordings is preferable or at least for a local calibration of noise results. Besides, it is probably better to use spectral ratios with respect to a reference site for amplification estimates if it is limited to the low frequency range only, and/or the distance from the site to the reference station does not exceed 1 to 2 km (Seekins et al., 1996).

Spectral Ratios of Horizontal Relative to Vertical Components: a microtremor observed at the ground surface is not always steady, but usually shows daily, weekly and seasonal changes. This is because the tremor is easily affected by surrounding noise sources. As a result, frequency components of horizontal and vertical microtremors indicate effects not only on local site conditions but also on such noise sources. Thus, local site effects are hardly detected with a sufficient accuracy from the horizontal or vertical motion of the microtremor alone. However, when a ratio of the frequency components between the horizontal and vertical motions is considered, it usually shows a steady spectrum, neglecting any contribution from deep sources, which is called the H/V spectral ratio (Nakamura, 1989). Due to its steady nature, the H/V spectral ratio has become often used for analyses of microtremor data for site effect studies. It may be supposed that the vertical component of motion is not amplified by the soft soil layer according to H/V spectral ratio (some related evidence is discussed by Campillo et al., 1988).

H/V spectral ratio (also referred to as the Nakamura technique) relies on the interpretation of microtremors as the ellipticity of Rayleigh waves propagating in a single layer over a half-space because of the predominance of Rayleigh waves in the vertical component. This ellipticity is frequency dependent and exhibits a sharp peak around the fundamental frequency for sites displaying a high enough impedance

contrast between surface and deep materials. This peak is related with a vanishing of the vertical component, corresponding to a reversal of the rotation sense of the fundamental Rayleigh wave, from counterclockwise at low frequency, to clockwise at intermediate frequency (Figure 4.19). The vertical component of ambient noise keeps the characteristics of source to sediments surface ground, is relatively influenced by Rayleigh wave on the sediments and can therefore be used to remove both of the source and the Rayleigh wave effects from the horizontal components.

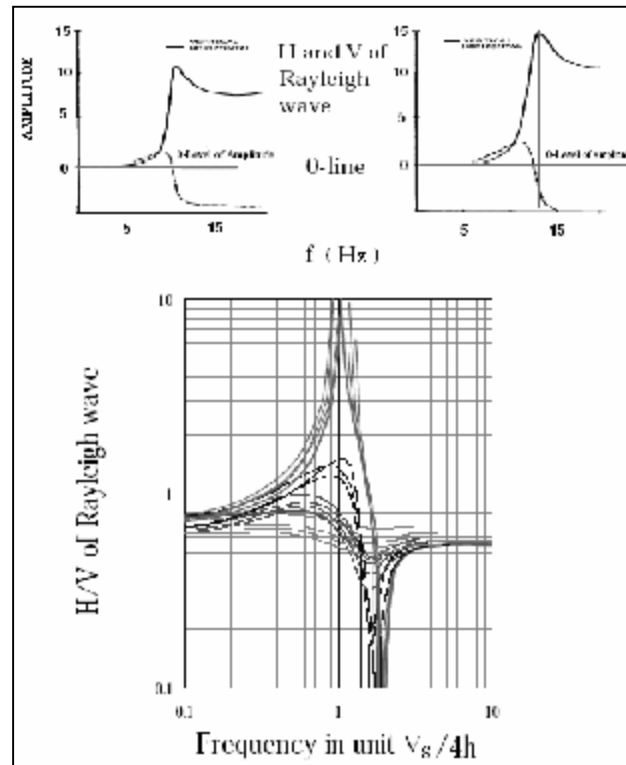


Figure 4.19. An example of ellipticity for Rayleigh waves in a stratified half-space, displaying the H/V ratio as a function of frequency for single layered (top) and two layered ground (bottom). The infinite peaks correspond to a vanishing of the vertical component, while the sharp troughs correspond to a vanishing of the horizontal component (modified from Nakamura, 2000).

It can be clearly seen from this figure that, the energy of Rayleigh wave does not appear on the peak of H/V of Rayleigh wave. It can be easily seen that there is no energy around the peak frequency of H/V and amplitude is almost zero for

horizontal and zero for vertical components of Rayleigh waves. On the other hand, Rayleigh wave energy gets its maximum on later frequencies at minimum group velocity of Rayleigh wave and this is nearly equal to trough frequency which is almost two times of the H/V peak frequency (Nakamura, 2000). In the Nakamura technique, for different wave-lengths, H/V and phase velocity of Rayleigh waves are calculated for two layer models for various impedance ratios (varying between 1.2-4.5) and Poisson's ratio (varying between 0.25-0.49) according to Ohta (1962) both in sedimentary and basement layers is drawn in Figure 4.19 (bottom one) to show the relations between impedance, peak and frequency. Note that frequency in this figure is normalized with $V_s/4h$, where V_s is S-wave velocity and h is depth of sediment.

As theoretically, site effects due to surface geology are generally expressed as the spectral ratio $S_1(\omega)$ between the horizontal component of earthquake recordings at the surface of the soft layer (H_s) and the ones at the ideally horizontal outcropping bedrock (H_B) (Figure 4.20):

$$S_1(\omega) = \frac{H_s(\omega)}{H_B(\omega)} \quad \dots\dots\dots(4.20)$$

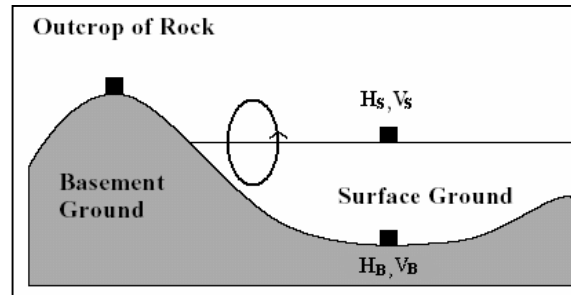


Figure 4.20. Simple model assumed by Nakamura (1989) to interpret microtremor measurements (H_B and V_B are denoted as spectra of horizontal and vertical motion in the basement under the basin, outcropped basin. H_s and V_s are denoted as spectra of horizontal and vertical directions of motion at the surface, Rayleigh waves).

The instrumental method consists of recording the ambient background noise caused by industrial activities and urban traffic, then calculating the spectral ratio of

the horizontal and vertical components of the recorded signal. The technique is based on the following assumption that:

- microtremors are composed of several waves, but essentially Rayleigh waves propagating in a soft surface layer overlying a stiff substratum;
- the effect of the Rayleigh waves (E_{RW}) on the noise motion is included in the vertical spectrum at the surface (V_S), but not at the base of the layer (V_B):

$$E_{RW}(w) = \frac{V_S(w)}{V_B(w)} \quad (4.21)$$

- the vertical component of microtremor motion is not amplified by the soft soil layer;
- the effect of the Rayleigh waves on microtremor motion is equivalent for the vertical and horizontal components;
- for a wide frequency range (0.2-20 Hz), the spectral ratio of the horizontal and vertical components of motion at the bottom of the layer is close to unity:

$$\frac{H_B(w)}{V_B(w)} = 1 \quad (4.22)$$

In these conditions, the spectral ratio between the horizontal and vertical components of the background noise recorded at the surface of a soft layer enables the effects of the Rayleigh waves (E_{RW}) to be eliminated, conserving only the effects resulting from the geological structure of the site. Therefore, this spectral ratio will be called the H/V spectral ratio given as:

$$S_2(w) = \frac{S_1(w)}{E_{RW}(w)} = \frac{H_S(w)}{V_S(w)} \quad (4.23)$$

Many theoretical (Lachet and Bard, 1994) and experimental (Lermo and Chavez-Garcia, 1994; Tevez-Costa et al., 1996; and Seekins et al., 1996) studies have shown that the spectral ratio obtained in this manner enables an adequate determination of the site fundamental frequency. However, the Nakamura method does not seem to be able to provide all the information required for a reliable estimation of the amplification of surface ground motion. One should not forget that this technique is based on various assumptions, as yet not totally verified, concerning the nature of the incident background noise (Bour et al. 1998).

Several attempts to obtain “noise synthetics” with numerical simulation have been performed in recent years, assuming the noise is due to surface sources randomly distributed in space and time. Their main interest is that they simultaneously account for body and surface waves. Their main shortcoming is that their ability to capture the actual noise wavefield is very uneasy to check. The results were generally given for 1D horizontally stratified media. Field and Jacob (1993), Lermo and Chavez-Garcia (1993), Wakamatsu and Yasui (1996) proposed a frequency domain approach with root mean square summation to account for source randomness, while Lachet and Bard (1994 and 1995), Cornou (1998), Tokeshi and Sugimura (1998) and Volant et al. (1998) followed a time-domain approach. Additionally, some numerical analysis for 2D (semi-circular) and 3D (semi-spherical) structures, with a severe limitation (i.e. in the presence of significant lateral heterogeneity) were presented by Dravinski et al. (1996), Lavace et al. (2001), and Coutel and Mora (1998) using pseudo-spectral techniques and randomly distributed surface sources. As a result, for 1D layered media, they reached the now usual conclusions (agreement for the fundamental frequency, failure on amplification estimates), and less optimistic conclusions for 2D media in that even the fundamental frequency may not be retrieved by the H/V ratio on valley edges.

Figure 4.21 displays schematically how we calculate the numerical simulation H/V ratio from synthetic seismograms. The example shown is generated with the multiple source model to obtain a three components synthetic noise recording (Lachet and Bard, 1994). The best way is to probably construct a complex signal $x(t) + iy(t)$ (where x and y stand for the two orthogonal horizontal components), and to compute its complex Fourier transform. The fast Fourier

transform (FFT) of the three components are calculated (two smoothed horizontal spectra and the vertical one), and then smooth the spectra obtained to avoid spurious peaks linked with sharp troughs on the vertical component spectrum. It has important consequences on the spectral shape and may strongly affect the determination of the peak frequency (Bard, 1999). A simple remedial solution is to adapt the width of the smoothing triangle window to the frequency of interest, i.e., to have a constant bandwidth on a logarithmic frequency scale. The advantage is that the smoothing width is a function of the frequency, so it can be kept constant for all the cases. Then one of the two smoothed horizontal spectra and the vertical one are taken, followed by calculating their ratio (Lachet and Bard, 1994).

There are alternative options and various implementation techniques to process data during the numerical simulation, in particular using either averaging spectra or spectral ratios (Bouden-Romdhane and Mechler, 1998), choosing either arithmetic or geometric averages (Field, 1994, Lachet and Bard, 1994), and applying methods of smoothing the spectra (Lachet and Bard, 1994; Konno and Omachi, 1998; and Mucciarelli, 1998). These solutions give very similar results in general; however, they are based on theoretical assumptions and are still being developed.

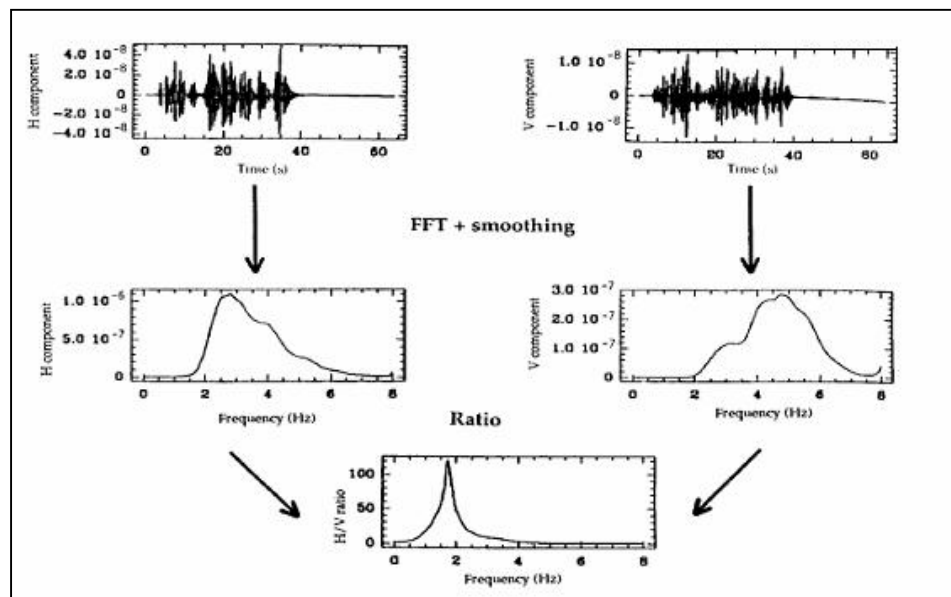


Figure 4.21. Example procedure used to calculate the H/V ratio from synthetic seismogram (Lachet and Bard, 1994)

Furthermore, since these techniques are based on various assumptions, as yet not totally verified, the spectral responses obtained can be inspected and/or can not be used alone (Bour et al, 1998; Rosset, 2004). Therefore, to validate the experimental technique, H/V spectral ratios were compared with the transfer functions obtained from a one-dimensional (1-D) numerical simulation (SHAKE 91), which consists of the response analysis of horizontally layered soils under seismic solicitation, with a linear equivalent soil behavior (Schnabel et al., 1972 and Idriss and Sun, 1992). A comparison of the results obtained by the experimental and numerical methods showed that the fundamental frequencies are in good agreement. Although the amplification obtained by the H/V ratio is generally higher than that obtained by the numerical modeling, the fundamental frequency obtained by the two methods is very similar. The presence of a harmonic resonance frequency on the transfer functions is calculated by SHAKE, which is absent in the H/V spectral ratios (Bour et al., 1998). Briefly, such a methodology is able to give spectral and temporal features of dynamic soft soil response, especially in urban areas (Rosset et al., 2004). It can be concluded that the numerical and experimental approaches should be combined so as to better constrain the microzonation of a given region, in particular those of weak seismic activity and/or high levels of ambient noise.

Dealing with the data recording and processing techniques in the field, the important recommendations described by Duval (1994), Nakamura (1996), Mucciarelli et al (1997), Mucciarelli (1998) and Bard (1999) as based on their general experience are as follows:

- avoid accelerometers and prefer velocimeters: the resolution power of present accelerometers is too low at low frequencies,
- prefer 5 s or 10 s velocimeters, for analysis at periods longer than 1 s (however, note that some authors claim that a 4.5 Hz seismometer provides reliable results down to 0.5 Hz),
- avoid recordings close to loads with heavy vehicles which cause strong and rather long transients. For instance, light traffic does not affect the H/V ratio,

while heavy traffic does affect it. The proximity to stationary machines should also be avoided.

Despite the fuzzy and inconsistent results rarely obtained due to their theoretical point of view that has not been totally verified, the theoretical background experimental results prove that H/V ratio does successfully point out the fundamental resonance frequency of soft soils. The estimate the fundamental resonance frequency of soil is easier on the H/V ratio than on absolute spectra or site to reference spectral ratios, especially with the improvements allowed by the phase processing for the determination of frequency where the horizontal and vertical noise spectra is the minimum in sites with low impedance contrast used extensively for urban mapping, especially in the short period range and in some cases as a engineering geological and geotechnical tool. Therefore, it is well adapted in the urban environments to be of great interest for site effects and to determine the fundamental frequency and establish the seismic microzonation. In other words, microtremor measurements are very useful in seismic hazard assessment, as they can be used to obtain both the dominant period of motion at a soft soil site with good reliability and a rough estimate of amplification level if geological conditions are relatively simple.

Finally, it can be concluded that experimental approaches obtained by microtremor measurements should be used in conjunction with other site investigation or with available geological and geophysical information as well as complementary numerical simulation analysis (if possible) to obtain determine much more reliable and comprehensive information on soft soil response, and thus identifying and characterizing the site amplification and fundamental frequency in urban areas.

Considering the purpose of the research and the methodology that was conducted in the study area, short-period noise recordings of microtremor measurements were conducted at 355 project site locations on the Upper Pliocene to Pleistocene Fluvial and Quaternary Alluvial sediments to study the seismic response in the western part of the Ankara basin (Figure 4.3). The spectral ratios relative to a firm site reference station (S/R ratio), and the spectral ratio between the horizontal

and vertical components (H/V ratio) of the microtremor measurements at the ground surface has been used to estimate the fundamental periods and amplification factors of the site that provided satisfactory estimates of the site response of soft deposits. The Nakamura's technique (H/V ratio), which also gave an estimate of amplification of seismic waves (when the local geology is relatively simple) was utilized to process the data considering the various assumptions of ambient background noise that was discussed in detail previously.

During the microtremor measurements, the standard equipment utilized comprised an Akashi JEP-6A3, three component built-in acceleration seismometer with V243FA elements (Figure 4.22) and a DATAMARK LS-8000 WD with 24 bit resolution A/D type measuring instrument as a data recorder (Figure 4.23). Natural periods of sensors used were based on the period dependency (sensors are generally 1.0 s or longer than 1 s. especially for short period of range) of microtremors. During the recording of the measurements, the sampling interval rate was 100 Hz and the duration of each sample recording was 180 s. Microtremor recordings were attempted to be spaced approximately 500 m depending on the suitability of the particular location in the study area.



Figure 4.22. General view of microtremor measurements and a close-up view of the Akashi JEP-6A3 three component built-in acceleration seismometer (small figure) during the field investigation.



Figure 4.23. General view of the DATAMARK LS-8000 WD with 24 bit resolution A/D type data recorder and its associated equipment used for microtremor measurements in the field investigation.

The numerical simulations of H/V and S/R ratio from synthetic seismograms that is generated with the multiple source models to obtain a three component synthetic noise recording were processed. And then, its complex Fourier transform (FFT) was computed from these complex signals for smoothing the spectra to avoid spurious peaks linked with sharp troughs on the vertical component spectrum. Note that the spectral shape may strongly affect the determination of the peak frequency. A simple remedial solution is to adopt the width of the smoothing triangle window to have a constant bandwidth on a logarithmic frequency scale for the frequency of interest that can supply to be kept constant for all the cases. During processing of data for each of the measurement point that was recorded from the field study, 20 s quite time interval (particularly noiseless) was chosen to process microtremor data due to filtering the noise. Then, Fast Fourier Transform (FTT) procedure was applied on each window and the obtained Fourier spectrum is smoothed. The Fourier spectrum was obtained by applying the hanning window to waveform data (2048 points; 20 sec.) after period or frequency analysis with FFT. During the analyses of the data for smoothing, band-width (bw) was chosen as 0.4 Hz to catch

all possible peaks in the acceleration spectrum that is the most suitable one when compared to the other smoothing results from 0.1 to 0.8 Hz. Additionally, the spectra were also smoothed by applying a 0.1 Hz low and 10 Hz high Butterworth bandpass filter to eliminate the spectral contribution of high frequency spikes which were present in the time series. One of the examples regarding the waveform of the unprocessed microtremor data are presented in Figure 4.24. Finally, the processed FFT results of the horizontal and vertical acceleration spectrum (Fourier spectra), and H/V spectral ratio spectrum according to the Nakamura technique and spectral ratios relative to a firm site reference station (S/R ratio) were developed, and then one of the samples from the research site was given in Figures 4.25 and 4.26. The measured frequency values were compared (or correlated) with existing geotechnical and seismic information, and ground response analyses software (if possible) that were obtained from collected and performed site investigation studies to check the accuracy and hence the reliability of the results and validating of the experimental technique.

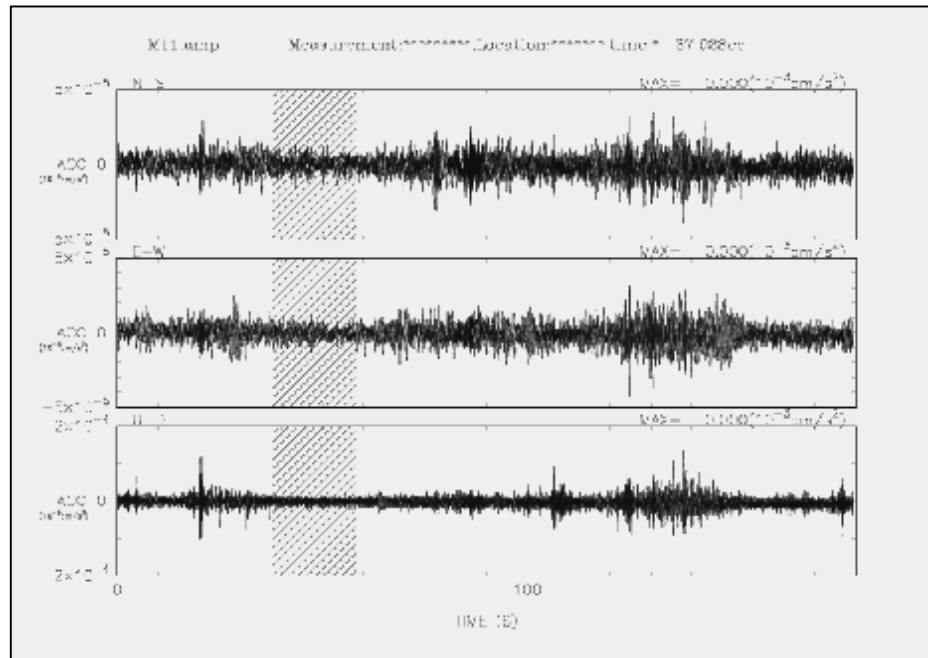


Figure 4.24. An example of the waveform from the unprocessed microtremor data from microtremor measurements in the field operations of this study (measurement of data point MOB-11).

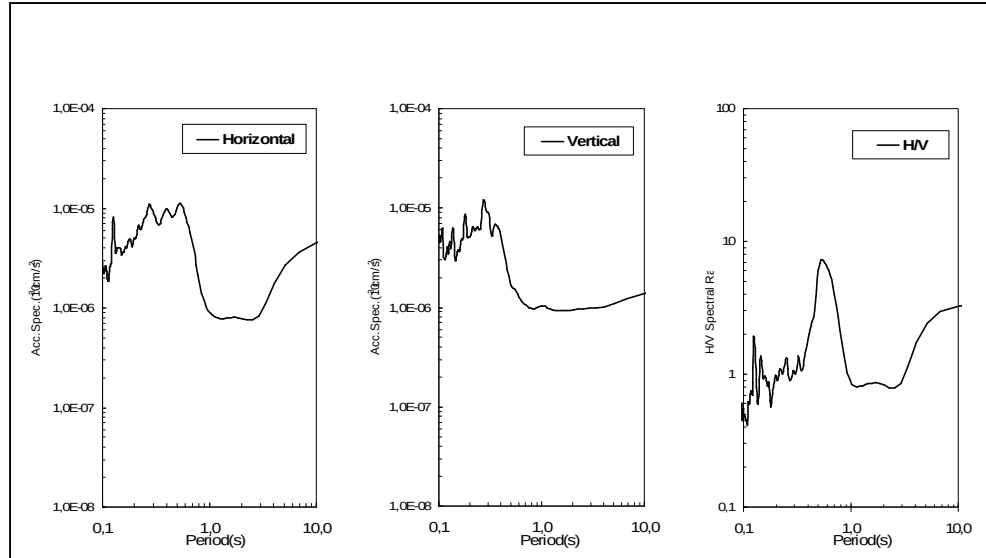


Figure 4.25. An example for the horizontal and vertical (H/V) acceleration spectrum, and H/V spectral ratio spectrum that were developed for the research site (processing of data point MOB-11).

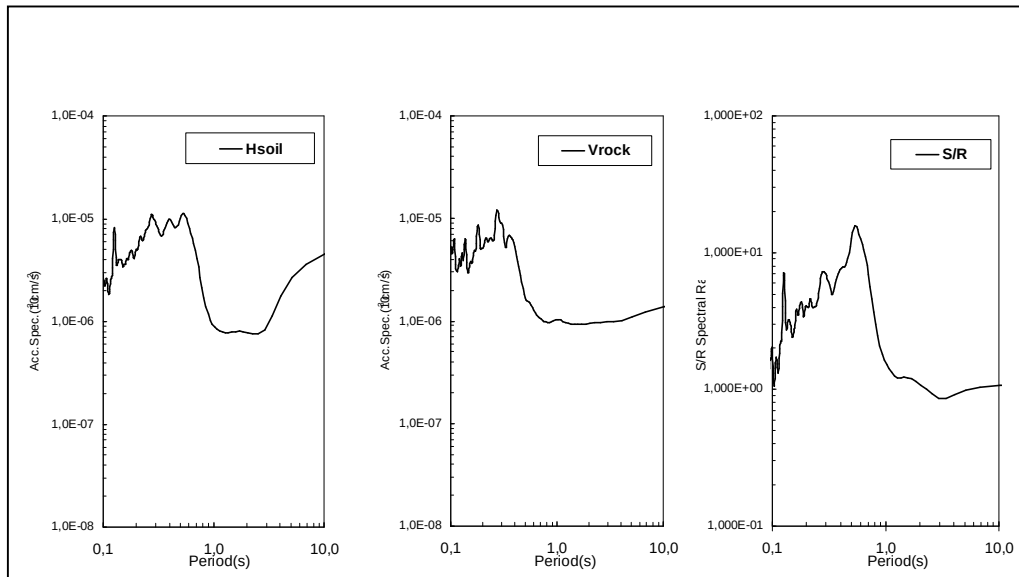


Figure 4.26. An example for the Ratios Relative to a Reference Site (S/R) acceleration spectrum, and S/R spectral ratio spectrum that were developed for the research site (processing of data point MOB-11).

CHAPTER 5

ENGINEERING GEOLOGICAL AND GEOTECHNICAL SITE CHARACTERIZATION AND SEISMIC ZONATION

5.1. Introduction

Site characterizations are based on information assembled from databases provided by a number of engineering geological and geotechnical seismic site characterization studies in order to identify the relevant parameters and factors, and to determine the variations in local site conditions in seismic hazard assessments. Hence, site characterizations and seismic zonations entail obtaining the field measurements of dynamic characteristics and properties such as shear wave velocity, penetration blow count, etc. These approaches require extensive field testing. Therefore, presented herein is a method for identifying soil profiles in regards to site characterization that merges in-situ measurements of dynamic properties with geologic information and especially surface seismic methods. Engineering geological, geotechnical and seismic investigations were performed in the project area to classify and characterize especially the Quaternary soil deposits according to design code of IBC2003 (ICC, 2003). GIS was used to offer a platform for merging these various data types and for assessing the spatial extent of soils to characterize and classify the site for seismic zonation and seismic hazard determinations.

5.2. General Procedure for Determining Code-Based Site Characterizations and Site Classifications

The distribution of damage caused by earthquake ground shaking commonly reflects areal differences in local geologic and site conditions. Damage on thick deposits of unconsolidated sediments, for example, is frequently observed to be much greater than that on bedrock at nearby locations. The consideration of site conditions has become an important part of assessment of seismic shaking hazard. In mapping geographic variations in shaking response, parameters are needed that can be used to predict these variations in ground motion due to differences in site geology. Recent studies indicate that shear-wave velocity is a critical factor in determining the amplitude of ground motion (Joyner and Fumal, 1985; Boore et al., 1993; Anderson et al., 1996) and thus might be a useful parameter to characterize local geologic conditions for seismic zonation studies. As an alternative, in characterization of a geologically complex region, especially in younger deposits, relations between shear-wave velocity and several other physical properties of geologic materials (i.e., standard penetration resistance, undrained shear strength, or void ratio) have been identified, which can be mapped more readily on a regional scale (Fumal, 1978; and Fumal and Tinsley, 1985). By using this concept of assigning shear wave velocities to the mapped geologic units that were developed by Tinsley and Fumal (1985), and modified by Park and Elrick (1998), the engineering geological and geotechnical parameters that show useful correlations with shear-wave velocity are texture (relative grain-size distribution) and standard penetration resistance for unconsolidated sedimentary deposits, and lithology for bedrock materials. These correlations can be applied to a particular area by using data concerning the areal distribution, physical properties, and thickness of the geologic units present to estimate and map shear-wave velocity that is useful for dynamic engineering geological design and building code enforcement.

Seismic ground response characteristics, defined generally as “local site conditions”, are suitably reflected in seismic code provisions. The selection of appropriate elastic response spectra according to soil categories and seismic intensity is the simplest way to account for site effects both for engineering projects and for general purpose seismic zonation studies. Modern seismic codes (IBC2000 and 2003, UBC97, NEHRP) which have all been introduced in the last decades, especially after the recent strong earthquakes in America, Europe, Japan produced numerous invaluable data. These codes have incorporated the most important experimental and theoretical results with the necessary adjustments and simplifications for purely practical reasons. “The Existing Model Code Groups” formed the International Code Council (ICC) with express purpose of developing a single set of construction codes for the entire United States, leading to the combination of UBC (Uniform Building Code) and NEHRP (National Seismic Hazard Reduction Program) into the IBC2000 and 2003 (International Building Code International Code Council, ICC). The most recent design code, IBC2003 is a common code all of the U.S. and also acceptable internationally since it is indeed a significant forward step towards harmonization and a major scientific breakthrough. IBC 2003 reflects the basic knowledge and technology of the present time, having an acceptable level of accuracy, compatible among others with the tools used for the seismic design of structures.

The 1998 Turkish Earthquake Code also defines local site classes to identify the site conditions which is one of the best procedures used for site classification in earthquake codes. In fact, one can think that local soil conditions can be easily identified and reflected to design, if proper and appropriate experimental and in-situ data are used for evaluating the soil conditions as specified in the Turkish Code. However, strong motion records obtained in the near field during past earthquakes have displayed that earthquake characteristics can change significantly from one point to the other (Chaves-Garcia et al., 1996; Field & Hough, 1997; Hartzell et al., 2000; Borchardt, 2002). Thus, especially in case of high rise buildings and critical

structures, detailed investigation of local soil conditions and determination of site specific design parameters must be obligatory in evaluating the seismic forces controlling the structural response. From this perspective, seismic zonation or microzonation studies may seem relatively inadequate because evaluating the design parameters, particularly by using shear wave velocity results for each lot in the selected area is not the objective. Moreover, converting the scale from 1:1,000,000 to 1:5,000 in an earthquake hazard study along with the geological and geotechnical data will not provide enough resolution for a comprehensive assessment in such a scale (Ansal et al., 2004). Hence, the IBC code is the best alternative to determine the effects of soil layers on design earthquake characteristics through utilizing the equivalent shear wave velocity that is defined as the weighted average of shear wave velocities of soil and rock layers in the upper 30 meters.

In the code-based site characterization, the main improvement is that amplification factors of spectral values are varying with the seismic intensity; lower shaking intensity earthquakes introduce higher amplification factors due to the more linear elastic soil behavior, contrary to higher intensities where soils are exhibiting non-linear behavior resulting in a decrease of peak spectral values. Additionally, a more accurate soil categorization is introduced based on a better description of soil profiles using standard geotechnical parameters (i.e., soil type, cementation, plasticity index, undrained shear strength S_u) and average V_s values. In IBC2003 and other codes of the same family, special attention is given to near field conditions introducing higher amplification factors for the same earthquake magnitude. Also for soil layers of small thickness presenting high impedance contrast, the new version of codes attribute higher amplification factors which is compatible with observations and theory (Pitilakis, 2004).

The amplification of ground motions at a nearly level site is significantly affected by the natural period of a site ($T_n = 4H/V_s$; where T_n = natural period, H = soil depth, and V_s = shear wave velocity; i.e., both dynamic stiffness and soil depth

are also important). Other important seismic site response factors are the impedance ratio between surficial and underlying deposits, the material damping of the surficial deposits, and how these seismic site response characteristics vary as a function of the intensity of the ground motion, as well as other factors. To account partially for these factors, a site classification system should include a measure of the dynamic stiffness of the site and a measure of the depth of the deposit (Rodriguez-Marek, 2001).

Wave propagation theory suggests that ground motion amplitude should depend on the density and shear wave velocity of near-surface media (e.g., Bullen, 1965; Aki and Richards, 1980). Density has relatively little variation with depth, and so shear wave velocity is the logical choice for representing site conditions. Two methods have been proposed for representing depth dependent velocity profiles with a single representative value. The first takes the velocity over the depth range corresponding to one-quarter wavelength of the period of interest (Joyner et al., 1981), which produces frequency-dependent values. Fumal and Tinsley (1985) developed 1-Hz V_s maps by relating quarter-wavelength velocities inferred from boreholes to geologic units.

A practical problem with the quarter wavelength V_s parameter is that the associated depths are often deeper than can economically be reached with boreholes. In the actual case, engineering site investigations are usually limited to the uppermost 30 m of material since that is the typical depth of borings, and it has become the standard depth for classifying site characteristics. Therefore, the 30-m V_s parameter, intensity-dependent, short- and long-period amplification factors depending on the average shear-wave velocity measured over the upper 30 m of a site, was proposed to overcome this difficulty and has found widespread use in practice. Based on empirical studies, Borchardt and Glassmoyer (1994), and Borchardt (1994) recommended 30-m average V_s as a means of classifying sites for building codes, and similar site categories were selected for the seismic design provisions for new buildings (Martin, 1994 and Martin and Dobry, 1994). And then,

these studies, along with the work of Seed et al. (1991) and Dobry et al. (1994), has been incorporated into the site classification studies (ICBO, 1997). Consequently, recent seismic code provisions such as UBC 97, NEHRP, IBC 2000 and IBC 2003 adopted the site classification using average shear-wave velocity over the upper 30-m of a site as the sole parameter for site classification establishing site classes for characterizing the subsurface.

In the IBC 2003 site characterization, the site classification system is an attempt to capture the primary factors affecting seismic site response while minimizing the amount of data required for site characterization. Site categorization schemes have generally found distinct levels of amplification at sites with different geologic and geotechnical characteristics that have been used include surface geology, average shear wave velocity in the upper 30 m, geotechnical data, including sediment stiffness, depth, and material type, and depth to basement rock.

In the International Building Code System, three options are mainly suggested for characterizing the local site conditions in terms of mean shear wave velocity for purposes of estimating ground response or amplification factors. The first option allows the site conditions to be characterized by classification of the site using a description of the physical properties of the near-surface materials. This classification in turn allows the mean shear wave velocity for the corresponding simplified site class to be assigned to the site. The second and third options afford more quantitative characterizations in terms of either inferred or measured estimates of mean shear wave velocity to a depth of 30 m (100 ft). These estimates of shear wave velocity permit more accurate classification of the sites using the shear wave velocity criteria (Borcherdt, 1994). Direct measurement technique provides the most accurate characterization of a site for purposes of estimating amplification factors.

The site specified by IBC 2003 as defined in Table 5.1 is identical to the provisions of IBC 2000, and practically distinguishes soil profiles in five main categories where the special conditions “E” and “F” that correspond to very loose or

liquefiable material are also prescribed, respectively. Where the soil shear wave velocity, $\overline{V_s}$, is not known, site class shall be determined, as permitted in Table 5.2, from standard penetration resistance, $\overline{N}$, or from soil undrained shear strength, $\overline{S_u}$, calculated in accordance with Equations 5.1, 5.2, 5.3 and 5.4. Where site-specific data are not available to a depth of 30 m, appropriate soil properties are permitted to be estimated by the registered design professional preparing the soils report based on known geologic conditions (ICC, 2003). Nevertheless, it should be noted that the above approach may be a particular simplification which might potentially lead to erroneous results especially in cases of deep soil formations or abrupt stiffness change between the soil layer at a depth of 30 m and the bedrock lying deeper.

When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official determines that Site Class -E or -F soil is likely to be present at the site (ICC, 2003).

As mentioned above, the advantage of such a classification is that the three parameters that are used for soil identification (shear wave velocity, SPT N values and undrained strength) is relatively easy to be measured, but on the other hand the soil stiffness is determined by the V_s values of the 30 m uppermost layers only. The $\overline{V_s}$, $\overline{N}$ and $\overline{S_u}$ are computed according to the following equations. Note that the notations presented below apply to the upper 30 m of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designed by a number that ranges from 1 to n at the bottom where there is a total of n distinct layers in the upper 30 m. The symbol, i , then refers to any one of the layers between 1 and n .

Table 5.1. Site Class Definitions (ICC, 2003)

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 30 m (100 feet),		
		Soil shear wave velocity, V_s (m/s)	Standard penetration resistance, N	Soil undrained shear strength, S_u (kPa)
A	Hard rock	$V_s > 1,500$	N/A	N/A
B	Rock	$7,600 < V_s \leq 1,500$	N/A	N/A
C	Very dense soil and soft rock	$360 < V_s \leq 760$	$N > 50$	$S_u \geq 100$
D	Stiff soil profile	$180 \leq V_s \leq 360$	$15 \leq N \leq 50$	$50 \leq S_u \leq 100$
E	Soft soil profile	$V_s < 180$	$N < 15$	$S_u < 50$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w > 40\%$, and 3. Undrained shear strength $S_u < 25$ kPa		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1 foot = 304.8 mm, 1 square foot=0.0929 m², 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

Table 5.2. Site Classification (ICC, 2003)

SITE CLASS	$\overline{V_s}$	$\overline{N}$ or $\overline{N}_{ch}$	$\overline{S_u}$
E	< 180 m/s	< 15	< 50 kPa
D	180 to 360 m/s	15 to 50	50 to 100 kPa
C	360 to 760 ft/s	> 50	> 100 kPa

For SI: 1 foot per second = 304.8 mm per second, 1 pound per square foot = 0.0479 kN/m².

If the S_u method is used and the N_{ch} and S_u criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

$$\overline{V_s} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{V_{si}}} \quad (5.1)$$

$$\sum_{i=1}^n d_i = 30 \text{ m (100 ft)}$$

where

V_{si} = The shear wave velocity in meter per second (ft/s)

d_i = The thickness of any layer between 0 and 30 m (100 ft)

$$\overline{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (5.2)$$

$$\overline{N_{ch}} = \frac{\sum_{i=1}^n d_s}{\sum_{i=1}^m \frac{d_i}{N_i}} \quad (5.3)$$

where

$$\sum_{i=1}^m d_i = d_s$$

Use only d_i and N_i for cohesionless soils.

d_s = The total thickness of cohesionless soil layers in the top 30 m (100 ft)

Note that N_i is the Standard Penetration Resistance (ASTM D 1586-84), not exceeded 100 blows/ft (0.3 m) as directly measured in the field without corrections.

$$\overline{S_u} = \frac{d_c}{\sum_{i=1}^k \frac{d_i}{S_{ui}}} \quad (5.4)$$

where

$$\sum_{i=1}^k d_i = d_c$$

S_u = The undrained shear strength in kPa (psf), not to exceed 240 kPa (5000 psf), ASTM D 2166-91 or D 2850-87.

d_c = Total thickness (100- d_s) (For SI:30- d_s) of cohesive soil layers in the top 30 m (100 feet).

PI = The plasticity index, ASTM D 4318.

w = The moisture content in percent, ASTM D 2216.

Note that the shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer, highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C. The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m, surficial shear wave velocity measurements are permitted to be extrapolated to assess $\overline{V_s}$. The rock categories, Site Classes A and B, shall not be used if there is more than 3 m (10 ft) of soil between the rock surface and the bottom of the spread footing or mat foundation (ICC, 2003).

For sites with ground conditions matching the two special subsoil classes Site Class E and Class F special studies for the definition of the seismic action are required. For these classes and particularly for F the possibility of soil failure under

the seismic action requiring site-specific evaluation must be considered. Further sub-division of this classification is permitted to better conform to special soil conditions.

Regarding the site coefficients, amplification factors for five discernable soil classes that incorporate four major site conditions are developed. The site coefficients are based on in-situ data collected and the response of each site to strong ground motions. The three criteria specified to distinguish the site classes are physical properties, mean shear wave velocity to 30 m and minimum thickness. The minimum thickness criteria was selected to ensure that sufficient material is present for resonant amplification to occur in the period band of engineering interest (>0.1 s). These criteria define classes with distinguishable amplification characteristics (Borcherdt, 1991 and 1994; Borcherdt et. al., 1991) that also are useful for mapping purposes. IBC adjusts the expected spectral acceleration at a reference rock site based on the actual site conditions to increase or decrease expected ground motions.

The IBC provisions outline a recommended procedure for determining design spectra based on seismic hazard maps of spectral acceleration and site coefficients determined by local site conditions. In general, the average horizontal spectral amplification increases with decreasing mean shear wave velocity is distinctly less for short-period motion than for intermediate-, long- or mid-period motion. This important observation suggests that site response can best be characterized by two factors, one for the short-period component of motion and the one for the other period bands. This important result is most apparent for sites underlain by soft soils. It implies that average horizontal response characteristics at the sites can be summarized by amplification factors expressed as continuous functions of mean shear wave velocity (Borcherdt, 1994 and Dobry et. al, 2000). The seismic hazard maps identifying the maximum considered earthquake ground motion for short- and mid- or long-period motions provide empirical estimates of the short-period (F_a) and the mid- or long-period (F_v) site-specific amplification factors and the site coefficients, which are given in Tables 5.3 and 5.4 .

Table 5.3. Values of Site Coefficient F_a as a Function of Site Class and Mapped Spectral Response Acceleration at Short Periods (S_s)^a (ICC, 2003)

SITE CLASS	Mapped Spectral Response Acceleration At Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s .

b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values, except that for structures with periods of vibration equal to or less than 0.5 second, values of F_a for liquefiable soils are permitted to be taken equal to the values for the site class determined without regard to liquefaction.

Table 5.4. Values of Site Coefficient F_v as a Function of Site Class and Mapped Spectral Response Acceleration at 1- Second Period (S_1)^a (ICC, 2003)

SITE CLASS	Mapped Spectral Response Acceleration At 1- Second Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_1 .

b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values, except that for structures with periods of vibration equal to or less than 0.5 second, values of F_v for liquefiable soils are permitted to be taken equal to the values for the site class determined without regard to liquefaction.

5.3. Seismic Zoning Study for the Ankara Basin Based on Geological and Geotechnical Site Conditions

The seismic damages are controlled basically by three interacting factor groups; earthquake source and path characteristics, local geological and

geotechnical site conditions, structural design and construction features. Seismic zonation can be considered as the assessment of the first two groups of factors. In general terms, it is the process for estimating the response of soil layers under seismic excitations and thus the variation of earthquake characteristics on the ground surface. Seismic zonation is the initial phase of earthquake risk mitigation and requires multi-disciplinary approach with major contributions from geology, engineering geology, seismology, geotechnical and structural engineering. The key issue affecting the applicability and the feasibility of any microzonation study is the usability and reliability of the parameters selected for seismic zoning (Ansal et al., 2004).

Seismic zoning maps are generally prepared in small scales such as 1:1,000,000 or less and are independent of geological and geotechnical site conditions. However, seismic microzonation requires for instance 1:5,000 or even 1:1,000 scale studies that take into consideration both the earthquake source and regional geological and geotechnical site conditions in order to be utilized for urban and land-use planning. Thus, detailed seismological, geological, engineering geological and geotechnical studies are highly preferable or recommended to pursue seismic zonation assessments.

A seismic zonation study consists of three stages: (1) estimation of the regional seismic hazard, (2) determination of the local geological and local geotechnical site conditions (3) assessment of the expected ground response and ground motion parameters on the ground surface.

There may be various differences among the adopted procedures with respect to these three stages (i.e., Slemmons, 1982; Cluff et al., 1982; Finn, 1991; Marcellini et al., 1995; Lachet et al., 1996; Doroudian et al., 1996; Fah et al., 1997; Bour et al., 1998; Ansal, 2002, etc.). These differences mostly arise from different intentions that produced zonation maps and different levels of accuracy achieved based on the available input data in terms of local geological and geotechnical site conditions. A first preference may be to produce zonation maps to be used mainly

for city and land-use planning. A second preference is to use the zonation maps to estimate the possible earthquake characteristics for the assessment of structural vulnerability in an earthquake scenario study. A third preference may be to provide input for the earthquake design codes. The study presented herein is more directed towards the first preference considering a seismic zonation study performed to determine especially the local geological, engineering geological and geotechnical site conditions of the Upper Pliocene to Pleistocene fluvial red clastics and Quaternary alluvial and terrace sediments lying towards the western part of the Ankara basin.

5.3.1. Geological and Geotechnical Site Conditions

In this study, the seismic zonation study performed to determine the local geological, engineering geological and geotechnical site conditions would start with the assessment of the local geological formations and with the mapping of the surface geology based on available sources of information and database that were compiled from studies including topographic maps, geologic maps, geomorphologic maps, satellite images, and descriptive accounts of excavations (including water wells, deep borings for bedrock elevations, other boreholes and geophysical surveys). Geologic units were defined and mapped on the basis of geologic and genetic criteria such as geologic and geomorphic expression, inferred depositional environment, and soil-profile development with the purpose of determining the boundaries and the characteristics of the geological formations. A 1:25000 scale geological map for the Ankara basin is presented in Figure 2.2 in Chapter 2. All the related geologic and geomorphologic features have been digitized using careful coordinate control under the Geographic Information System, ArcGIS. This map indicates the geological formations and their variation within the Ankara basin. It is important to point out that the previously prepared detailed geologic maps showed

all aspects of geology, including subunits of major formations that are recognized by their distinctive lithology or mode of origin, however the details of surficial or Quaternary geology for assessing the effects of local soil conditions that have major influences on seismic hazards are not included with these geologic maps. Although areas underlain by unconsolidated sedimentary deposits are most susceptible to ground motion amplification hazards, geologic maps ordinarily contain little data about the physical properties of these deposits if they are differentiated at all (Fumal, 1978; Wills et al., 2000). It should be noted that the site investigations also demonstrated that the existing geological units are not homogenous and significant changes in their properties could be observed from one point to another, even in the same formation. Hence, geologic nomenclature based on age and environment of deposition are more important rather than geologic formation names. Therefore, considering the geological units as the only criteria in seismic zonation is not adequate for evaluating earthquake hazards (Tinsley and Fumal, 1985). The main purpose of the geology map may be regarded as the basic information to commence the detailed site investigations and to control the reliability of the results obtained by site characterizations and site response analyses.

As mentioned previously, besides an extensive body of geologic information, engineering geological and geotechnical seismic information must be supplied as well to evaluate areal differences and to characterize the local site conditions to particularly determine seismic hazards. Bedrock and alluvial deposits are differentiated according to the physical properties that control ground response; maps of these properties are prepared by analyzing existing geologic maps, the geomorphology of surficial units, and geotechnical data obtained from boreholes and seismic testing. The average shear wave velocities and also standard penetration results in the upper 30 m of near-surface geologic units must be estimated to characterize the geological units for evaluating local site conditions. Using shear wave velocities and standard penetration results for classifying site conditions rather than geological units, even though the determination of these

results requires extensive field investigations. Therefore, particularly accurate depictions of near-surface shear-wave velocity useful for predicting ground-motion parameters take into account the thickness of the Quaternary deposits, vertical variations in sediment type, and the correlation of shear-wave velocity with standard penetration resistance of different sediments.

During site characterization for seismic zonation, the purpose was to determine the site conditions for specific locations as accurate as possible in order to reliably assess the site response characteristics. Therefore, for the identification of the local soil conditions the conducted data as well as the available existing data were taken into account as was stated previously. Furthermore, data were available from different sources for the project area, with varying degree of reliability and quality of the derived data. Thus the compiled information was dealt with great care and selectivity in carrying out the zonation procedure. Finally, a GIS methodology was used as a tool for unifying the various data types and for assessing the spatial extent of soils as related to the characterization and classification of the site. For this reason, the site characterization in the highly urbanized portions of the Ankara basin was conducted by adopting reasonably well distributed data set as much as possibly close to a grid system. A representative soil profile was determined based on all available geophysical and borehole data along with in-situ and laboratory test results. There were basically two objectives behind this approach (1) to utilize all the available data in order to have more comprehensive and reliable information about the soil profile; (2) to eliminate or to minimize the possibility of misrepresenting the distances between boreholes or site investigation points during GIS mapping.

The objective of the site characterization was to determine each representative soil profile with respect to a site classification criterion. Engineering geological, geotechnical seismic investigations and geological studies were used to classify the site conditions in the Ankara basin according to design code of the IBC 2003.

5.3.1.1. Site Conditions Based on Shear-Wave Velocity Characteristics of the Geologic Units of the Ankara Basin

Various studies have shown that local geologic conditions play an important role in the strength of earthquake shaking and the intensity of damage. The desire to predict ground response in relation to earthquake motions, for both building design and seismic microzonation purposes, has generated considerable effort aimed at clarifying the relationship between ground motion and local geology (Fumal, 1978). Recent studies have shown the importance of the upper 30 m of the earth's surface on strong motions from an earthquake, and also the empirical correlation between the ground motion during earthquakes and the average shear-wave velocity. This correlation has led to several efforts to predict site amplification factors for future earthquakes (Borcherdt, 1994; Joyner and Fumal, 1985; Boore et al., 1993). Shear-wave velocity (V_s) is an appropriate measure of rock or soil conditions for ground motion calculations because it directly affects ground motion amplification. V_s is proportional to the impedance of the material and particle velocity is inversely proportional to the square root of the impedance. As a seismic wave passes through material of decreasing impedance and V_s , the resistance to motion decreases and the particle velocity and, therefore, the amplitude of the seismic wave increases. Besides being a direct indicator of the potential for amplification of shear waves, V_s is effective in categorizing geologic units for calculating shaking because it is dependent on basic physical properties of the material, such as density, porosity, cementation of sediments and hardness and fracture spacing of rock (Fumal, 1978). Hence, in recent years, shear-wave velocity has become the primary measure of the "quality of the foundation soil" for determining the strength of earthquake ground shaking (Wills and Silva, 1998).

Initially, the study started with a classification scheme based on a simple geologic map of the Ankara basin. This map briefly subdivided the surface geology into three units: (1) Quaternary alluvial and terrace deposits, (2) Upper Pliocene to

Pleistocene fluvial deposits, and (3) Pre-Upper Miocene age basement rocks and Upper Miocene-Lower Pliocene rocks. The sedimentary units of Upper Pliocene to Pleistocene and especially Quaternary units in the western part of the Ankara Basin were subdivided on the basis of additional geologic, morphogeologic, geophysical and other studies in regards to better characterize the local sites.

As noted previously, surface wave measurements of S-waves (and P-waves as well) have been performed by using seismic refraction surveying at 204 locations in the project site in a spectrum of various sedimentary materials, especially in Quaternary alluvial and terrace sediments and in Upper Pliocene to Pleistocene fluvial red clastics, which are highly susceptible to seismic hazards. Additionally, the field study performed in this research was compared with 55 seismic wave measurements (S and P-waves) that were performed previously for other studies in the project area (i.e., Ankara Metro 3rd Stage Seismic Investigation, 2001; Mydonose Showland Seismic Investigation, 2002; Sincan Municipality Disaster Planning Project Seismic Investigation, 2002 and Kuran et al., 1978). Seismic travel times were measured in each site using a seismic refraction technique to provide profiles of near-surface compressional and shear wave velocities. The histogram of depth distributions for all velocity profiles used in this study are given in Figure 5.1. Additionally, the general distribution of site characterization data according to geological settings and average shear wave velocities are shown in Figure 5.2.

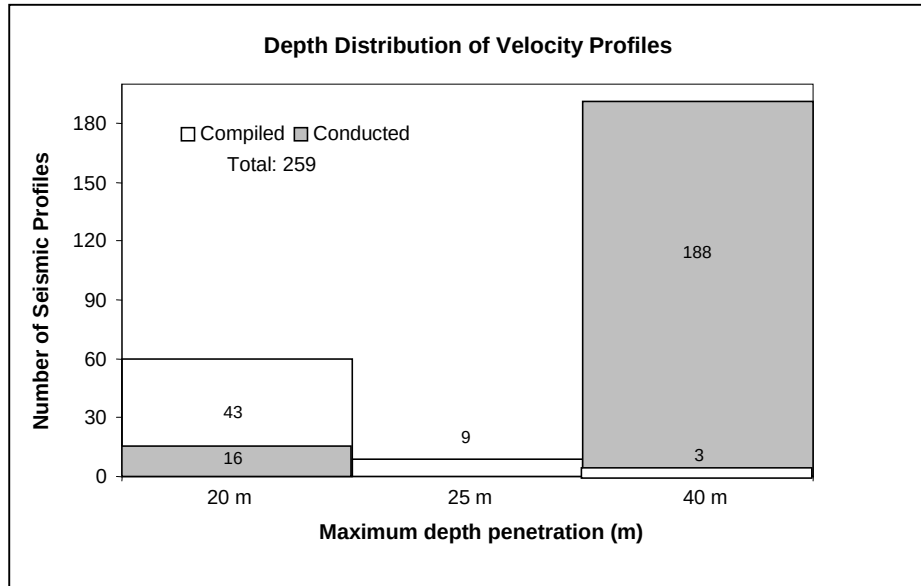


Figure 5.1. Histogram of depth distributions for all velocity profiles

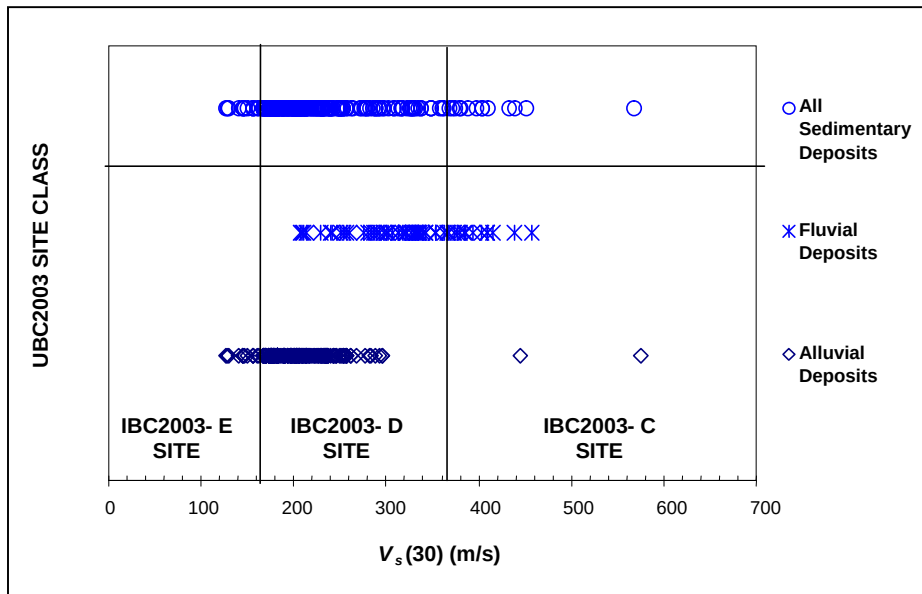


Figure 5.2. The general distribution of site characterization data according to geological setting and average shear wave velocities

The average shear-wave velocity of the top 30 m of the ground [$\overline{V_s}(30)$], which is computed by dividing 30 m by the travel time from the surface down to a depth of 30 m] is an important parameter in predicting the potential to amplify seismic shaking used in classifying sites in recent building codes (e.g., Dobry et al., 2000; BSSC, 2001) and in estimating losses. Hence, velocity profiles should be complete to a depth of 30 m as explained previously. In this dissertation, a total of 188 seismic profiles extended to at least 30 m, while only 16 (6.2 % of the entire data and 7.8% of the conducted data of the 204 seismic measurement) did not penetrate deeper than 20 m (Figure 5.1) due to the difficulty in performing surface wave measurements in residential or industrial areas (i.e., main reasons being, lack of space to extend geophone cables in the local settlement area, excess permanent noise from industrial facilities that created an energy source disturbance, etc.). Additionally, 52 (nearly 94.6% of the compiled data and 20% of the entire data) of the compiled seismic profiles had also not extended down to 30 m due to similar difficulties (Figure 5.1). Profiles (compiled or performed) less than 20 m deep have not been included in the seismic zoning calculations. If the data for measurements between 20 m and 30 m are to be used, some methods for extrapolating the velocities down to 30 m depth have been proposed. Among the extrapolation methods (i.e., linear and probability-based) that were stated by Boore (2004), both “averaged velocity” model and “assuming constant velocity model” were selected and applied to obtain confident extrapolation results for the shear wave velocity data that did not reach a depth of 30 m. According to Boore (2004), the percent of sites misclassified by these methods is generally less than 10% especially for shallower depth penetration, since velocity generally increases with depth and falls to negligible values for velocity models extending to at least 25 m. It should be noted that these methods are highly suitable for the data that were obtained from the Ankara basin.

In both methods, a key quantity is the time-averaged velocity $\overline{V}_s(d)$ to a depth d . Generally d is the depth to the bottom of the velocity model, which is not necessarily the depth to the bottom of the borehole or the depth of the deepest measurement in a borehole if the velocity model was determined from borehole logging. The time-averaged velocity is computed from Equation (5.5):

$$\overline{V}_s(d) = d/tt(d) \quad (5.5)$$

where the travel time $tt(d)$ to depth d is given by

$$tt(d) = \int_0^d \frac{dz}{V_s(z)} \quad (5.6)$$

In Equation (5.6), $V_s(z)$ is the depth-dependent velocity model. Site classes are determined by $\overline{V}_s(30)$, as indicated in Table 5.1. The purpose of this note is to investigate ways of approximating $\overline{V}_s(30)$ if $d < 30$ m (Boore, 2004).

In the “Assuming Constant Velocity” model; if the velocity model is available only to depth d , an assumption about the velocity between d and 30 m can be used to compute an estimate of $\overline{V}_s(30)$ using the following equation:

$$\overline{V}_s(30) = 30/d/tt(d) + (30 - d)/V_{eff}, \quad (5.7)$$

where V_{eff} is the assumed effective velocity from depth d to 30 m. The simplest assumption is that V_{eff} equals the velocity at the bottom of the velocity model:

$$V_{eff} = V_s(d) \quad (5.8)$$

Consequently, an average velocity for the upper 30 m was then computed using these extrapolation methods for the compiled and conducted data that did not reach a depth of 30 m (Figure 5.1.). In most cases, the final site classification based on both of the extrapolation methods are consistent with each other (deviation amongst them is less than about 5%) and in most cases these results matched with the preliminary site classification results where the average shear wave velocities were calculated prior to the extrapolation of the data (for a depth of 20 m to 25 m). It should be noted that none of the methods are likely to give an exact correct value of $\overline{V_s}(30)$ for a specific site. The procedures are inherently statistical, and over many sites the values should be correct on the average; the procedures only make sense when the data from many testing locations are being used in a statistical way, such as a regression analysis, and $\overline{V_s}(30)$ or site class for any particular site is not important (Boore et. al, 1993 and Boore, 2004).

In order to perform a site evaluation, the database has to be generalized from shear-wave velocity measurements at individual sites to broad shear-wave velocity classes that include sedimentary mapping units. Interpretation of the distribution of $V_s(30)$ results were included for each unit. According to the conducted in-situ testing database, summary of the results that describe the characteristics of generalized sedimentary geologic units and their IBC site classes based on $V_s(30)$ are given in Table 5.5. The histograms showing the brief summary of the data distribution considering the IBC2003 site classes for each geologic unit are also given in Figures 5.3 and 5.4. It can be seen from Table 5.5 that the density and distribution of the conducted seismic characterization studies have mainly concentrated on Quaternary alluvial and terrace sediments because the data locations of the compiled database generally concentrated on Upper Pliocene to Pleistocene fluvial deposits as previously discussed in detail in Chapter 4.

Table 5.5. Description of the characteristics of generalized geologic units and their IBC site classes based on $V_s(30)$ data.

Geologic Unit	# of Data	Percentage (%)	$V_s(30)$	$\pm$ Std	SITE CLASS (IBC 2003)
Quaternary alluvial and terrace deposits	0	-	-	-	CLASS B
	2	0,92	504	91	CLASS C ¹
	134	61,75	221	27	CLASS D
	81	37,33	169	13	CLASS E
Upper Pliocene to Pleistocene fluvial deposits	0	-	-	-	CLASS B
	14	33,33	392	27	CLASS C
	28	66,67	319	23	CLASS D
	0	-	-	-	CLASS E
Pre-Lower Pliocene basement rocks	No data ²				CLASS B
					CLASS C
					CLASS D
					CLASS E

Note 1. 2C sites in Quaternary are not consistent with the other results due to artificial fill (pre-emplaced fill materials) at those locations, and so were not included in the interpretation of the site characterization,

Note 2. In-situ tests have not been conducted for these geologic units in the research area since bedrock units were out of the scope of seismic hazard study.

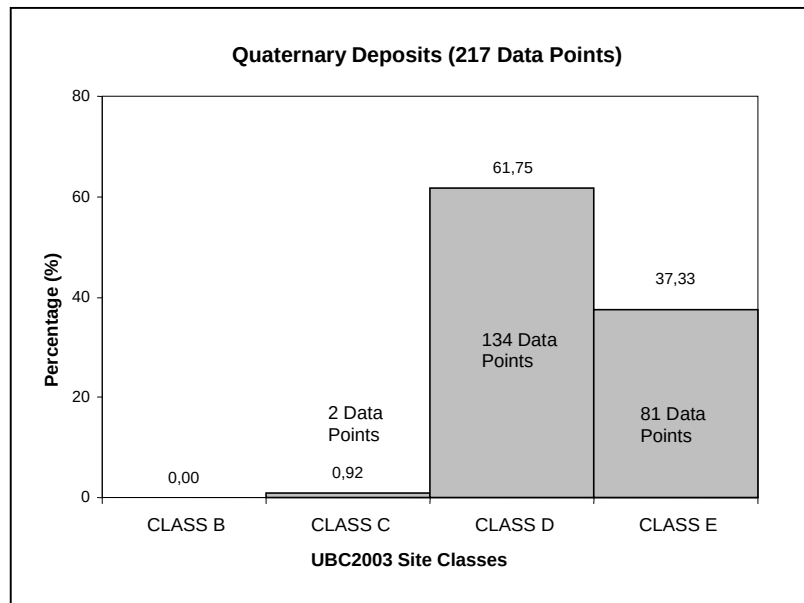


Figure 5.3. Data distribution considering the IBC site classes for Quaternary alluvial and terrace deposits based on $V_s(30)$ data.

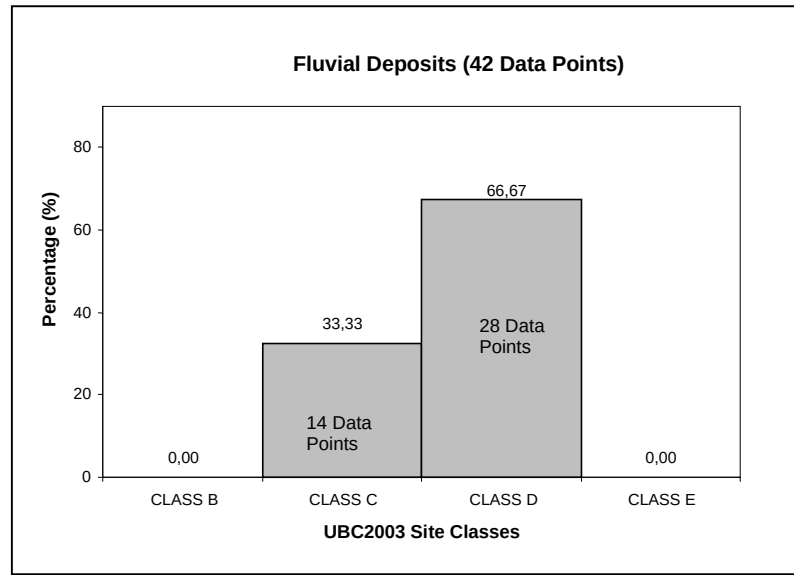


Figure 5.4. Data distribution considering the IBC site classes for Upper Pliocene to Pleistocene fluvial deposits based on $V_s(30)$ data.

During the interpretation of the $V_s(30)$ data, it was obvious that the velocities for each geologic unit were somewhat variable. In particular, Quaternary deposits presented variable velocity ranges that overlapped with the existing shear-wave velocity categories of the younger fluvial deposits. In addition, fluvial deposits had ranges of $V_s(30)$ that crossed boundaries between the existing shear-wave velocity categories of older geologic units. A major contributor to this crossing was attributed to the variability of sample textures which is an indicator of the range of depositional environments included within each geological unit. For instance, the Holocene and Pleistocene alluvial fan deposits include levee, channel, and flood-basin facies. Specifically, all of the sandy clays, silt loams, and gravelly sediments encountered were Upper Pleistocene or older fluvial deposits. In addition, post-depositional changes have generally resulted in a wider range of physical properties within the Upper Pleistocene textural groups. These environments cause deposition of a variety of soil textures that can influence V_s .

Since some sedimentary geologic units did not exactly fit into the code-based shear-wave velocity categories mentioned above, these units might be differentiated within two different categories. The units such as Upper Pliocene to Pleistocene fluvial deposits and Quaternary deposits have $V_s(30)$ distributions that cross the boundary between IBC2003 site categories D and C, and E and D, respectively (Figure 5.2 and Table 5.5). Considering the geological and morphological character of these sedimentary units, it might be that they also include different depositional or erosional characteristic themselves at the time of setting. Hence, it can be pointed out that site characterization depending on the $V_s(30)$ results is an appropriate measure of rock or soil conditions and may give valuable evidences to define the local site conditions, which might be helpful in differentiating the characteristics of the generalized sedimentary mapping units into the more detailed sub-classes that are consistent with geological age and depositional character.

5.3.1.1.1 Site Conditions of Quaternary Alluvial and Terrace Deposits

In these deposits, average shear wave velocity results were calculated for a total of 215 $V_s(30)$ testing locations in Quaternary deposits described either as younger, Holocene or undifferentiated Quaternary alluvium, or located within Quaternary alluvium, or older Quaternary deposits in a general geological map. As mentioned previously, since two (2) of the measurements leading to IBC site-C class are erroneously above the expected range of $V_s(30)$ for Quaternary alluvial sediments because artificial fill materials (i.e., railway fill materials, granular embankment fill materials, etc.) were previously emplaced at those locations, these two measurements have not been taken into account in the site characterization. In general, the older alluvium (Pleistocene or terrace deposits) has not been differentiated on the geological map or included in the profiles that were described

as having bedrock in the first 30 m. Naturally, most of the testing points with younger (Holocene) alluvium at the surface generally include older alluvium within the upper 30 meters or terrace deposits that are at a relatively higher elevation than the surrounding environment of the younger alluvium. Because Holocene alluvium is rarely more than 30 m thick and the transition from Holocene to Pleistocene or terrace deposits may not be reliably differentiated by using only surface seismic methods without any other supportive site characterization methods (i.e., boring logs, resistivity measurements), an attempt was made to subdivide this unit by the thickness of the young alluvium that will be mentioned later. The histogram given in Figure 5.5 shows the distribution of site classes for Quaternary alluvial and terrace deposits based on $V_s(30)$. The mean shear wave velocity for Quaternary geologic units is about 202 m/s but data can have variable velocity characteristics, which is ranging at the boundary between IBC site categories E and D that were mentioned previously. Regarding the distribution of the data from Figure 5.5, it may be stated that the $V_s(30)$ results for the Quaternary units are ranging from about 125 m/s to 292 m/s that appear to be relatively variable, as also evident from the standard deviation of the mean shear wave velocity.

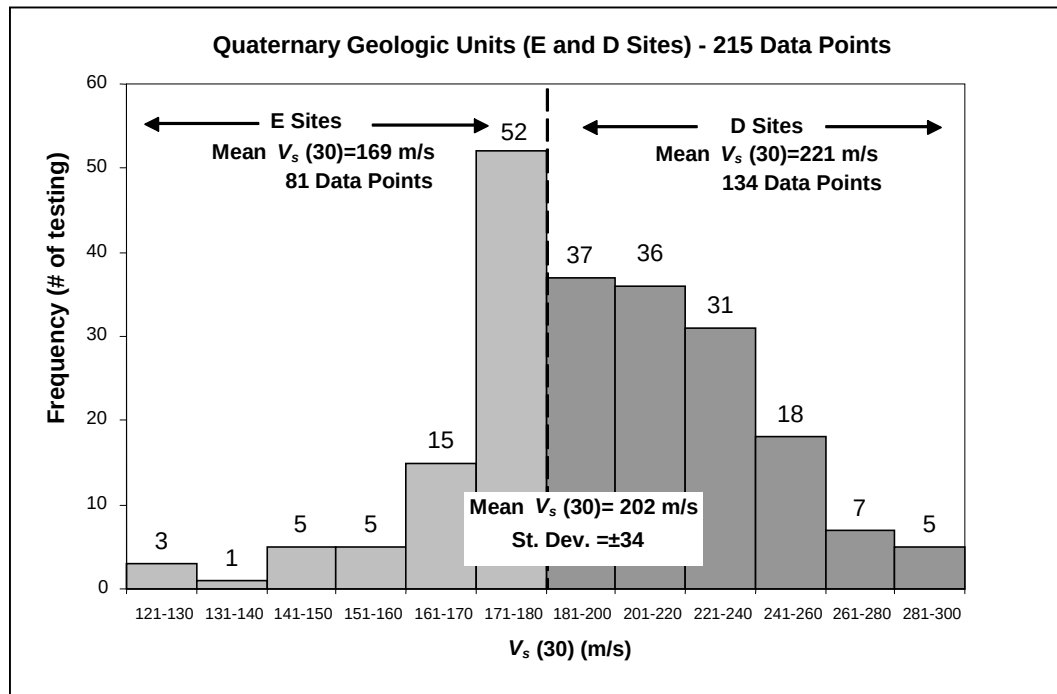


Figure 5.5. Histogram of $V_s(30)$ for distribution of site classes for Quaternary alluvial and terrace deposits (Note: The two (2) data points of C-site class were not taken into account as explained within the text).

There are some possible reasons for this variability in $V_s(30)$ results. Shear wave velocity in unconsolidated deposits is dependent on material properties including grain size, depth, density, and ground water levels (Fumal, 1978; Wills and Silva, 2000; Holzer et al., 2005). In addition, the details of lithologic descriptions of Quaternary alluvium for assessing the effects of local soil conditions are not compiled in most of the geologic maps. For instance, some areas shown on a geologic map as alluvium may have only a thin layer of younger alluvium underlain by different material that could range from fine-grained older alluvium to bedrock. Hence, the resulting variation in V_s may be substantial. Regarding the thickness of alluvium, it is important that $V_s(30)$ profiles with alluvium at the surface, including different material underlying the alluvium within the first 30 m in the most of the conducted and collected testing profiles, ranged between 20 m and 5 m. In addition,

the alluvium is gradually thinner towards to the edges of the alluvial basin (i.e., the northern side of the Tandoğan district in Sincan and Kurtuluş Recreation Field in Kolej) and tributaries of stream beds (i.e., Büyükçayır Creek stream bed along Saraycık County), ranging between 5 and 2 m in the form of a surface cover material. Therefore, it is naturally observed that $V_s(30)$ results gradually increase when the thickness of the alluvium profile that has different material underlying it decreases within the first 30 m (i.e., composite profile). However, considering the alluvium as one unit for the purpose of regional studies, as it commonly appears on geologic maps, and ignoring the thickness of the alluvium, is a useful first approximation.

Another reason is the variability of the V_s profiles with grain size for the two composite profiles, namely, fine-grained and coarse-grained alluvium. These composite profiles were constructed using only those layers classified by using the unified soil classification system. Layers of fine-grained soil (e.g., clay, silt, and silty sand) were grouped into one profile while layers of coarse-grained soil (e.g., sands and gravels) were grouped into the other profile (Fumal, 1978, and Wills and Silva, 1998). Therefore, there might be some interpretation methods in which soil descriptions are made by these kinds of studies. In the Ankara basin, resistivity profiles and boring logs at the particular testing locations complemented the soils descriptions.

The following methodology by Wills and Silva (1998) was applied for interpreting the soils descriptions of younger alluvial sediments in Ankara basin. The V_s measurements obtained through seismic surveying were generally averaged over several meters. Thin layers of contrasting lithologies have had only slight effect on the overall average. In preparing composite profiles, a layer of uniform V_s was assigned to either coarse or fine alluvium depending on the predominant grain size within the layer. For example, a layer containing alternating gravelly sand and clay was grouped with the coarse alluvium if the total thickness of gravelly sand was greater. It was grouped with the fine alluvium if the thickness of clay was

greater. In general, the fine alluvium profile leads to a relatively lower mean shear-wave velocity, slower increase of V_s with depth, and less variability. The differences attributable to grain size show that, while grouping all younger alluvium into a single unit may be appropriate as a first approximation, further distinctions should be made. For the Ankara basin, however, grain size characteristics and alluvial thickness have been generally described on geologic maps and laboratory studies for the upper surface of about 5 meters in the literature. These are important disadvantages before performing such a kind of site evaluation (i.e., due to the lack of geotechnical data down to a depth of 30 m).

According to the variable velocity characteristics of the Quaternary lithological units given above, these units were differentiated within three different units, and then characterized. They were classified as younger (Holocene or undifferentiated) and older Quaternary (terrace deposits or Pleistocene) at first. It should be stated that the reasons related with the depositional environment to affect the variability of the V_s profiles that were stated above however, is not sufficiently enough in that geologic and geomorphologic criteria has to be considered as well to differentiate the younger and especially older Quaternary terrace deposits that were explained in detail in Chapter 2. During the classification of the Quaternary alluvium as a younger and older deposit, boring logs, resistivity measurements, geologic and geomorphologic maps, and detailed field geological studies are the important steps to interpret the site conditions of these deposits.

Furthermore, younger alluvium deposits that can also have the distinct variability of velocity characteristics (Figure 5.5) commonly represent the results of deposition in environments similar to those present nearby today. Therefore, mapping of these Quaternary geologic units generally classify those units by environment of deposition that has a great deal of control over the sediment characteristics and physical properties (i.e., grain size, density, porosity, and cementation), and these factors can correlate with V_s . Regarding the evaluation of these variability characteristics, younger alluvium deposits (Holocene or

undifferentiated) that cover the major part of the Quaternary alluvium were classified as two separate IBC site categories of E and D according to their variability of V_s profiles with thickness of the alluvium and with grain size by creating two composite profiles, namely, fine-grained and coarse-grained alluvium.

Consequently, the results of the measurements of $V_s(30)$ in Quaternary alluvium deposits in the Ankara basin that were differentiated allows a corresponding degree of refinement in regrouping into three units with significantly different seismic wave velocity characteristics. These three different units of younger alluvium of E-Site, younger alluvium of D-Site and older Quaternary deposits (Terrace) are summarized in Table 5.6. The statistical results (i.e., mean, standard deviation, range and sample size) of $V_s(30)$ for Quaternary deposits and each three site categories within these deposit are also shown in Table 5.6.

Table 5.6. Summary of the statistical results of $V_s(30)$ for Quaternary deposit and each three site categories within these deposits.

Geologic Unit	No. of Data	Percentage (%)	$V_s(30)$ (m/s)			SITE CLASS (IBC 2003)	General Descriptions
			Mean	Std. Dev. ($\pm$)	Range		
Quaternary Deposits	215	100	202	34	125-292	D & E	Holocene-Upper Pleist
Younger Alluvium (E-Site)	81	37,67	169	13	125-180	E	Holocene
Younger Alluvium (D-Site)	119	55,35	217	26	181-292	D	Holocene
Older Quaternary or Terrace Deposits	15	6,98	251	24	218-284	D	Upper Pleistocene

Regarding the Younger Alluvium of E-Site, $V_s(30)$ was calculated for a total of 81 profiles that were described as Holocene or undifferentiated Quaternary alluvium. Of the 215 measured $V_s(30)$ values, 37,67% of the data fell within this unit of the Quaternary alluvium. The mean shear wave velocity was about 169 m/s

and the variability of $V_s(30)$ values were quite narrow (Std. Dev. = ± 13 m/s) with a range from 125 to 180 m/sec. This unit showed the lowest velocity unit that would be expected for a predominantly soft, uniform, normally consolidated fine-grained material (i.e., clay and silty clay with fine sand) which most probably indicated that the depth of the alluvium is relatively thick within the first 30 m. The composite profile for the Alluvium of E-Site unit showed low-velocity materials at the surface, with a gradual increase in velocity to about 30 m. The histogram given by Figure 5.6 shows the distribution of site classes for the Younger Alluvium of E-Site unit based on $V_s(30)$ data.

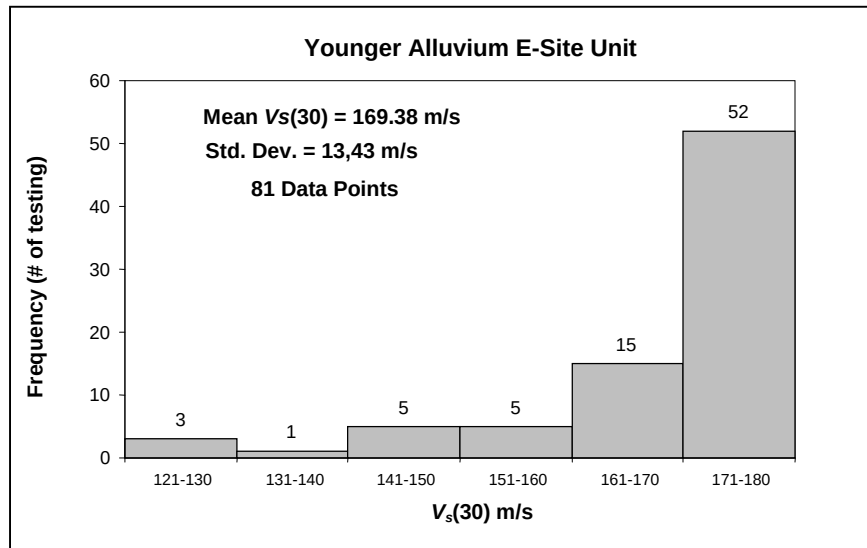


Figure 5.6. Histogram of $V_s(30)$ for distribution of site classes for the Younger Alluvium E-Site unit of the Quaternary Alluvium.

Considering the Younger Alluvium of D-Site unit, $V_s(30)$ was calculated for 119 profiles that was also described as Holocene or undifferentiated Quaternary alluvium. Of the 215 measured $V_s(30)$ values, 55,35% of the data fell within this unit in the Quaternary alluvium. The distribution of $V_s(30)$ for this unit showed a peak between 195 and 225 m/s with several values well above the $V_s(30)$ range of

this category (Figure 5.7). The mean shear wave velocity was about 217 m/s and the $V_s(30)$ values range from 181 to 292 m/s showing somewhat similar characteristics, however leading to relatively higher velocities with respect to the Holocene Alluvium of the E-Site unit. The main reasons for higher velocities attained might be that the Younger Alluvium of the D-Site unit might be expected to be relatively less uniform and might possess relatively coarse-grained material as well as fine grained material when compared to the E-Site unit. The outliers in the distribution are sites where the alluvial unit at the surface is relatively less uniform (i.e., intercalated with underlying units having different material contents or layers) or thinner layers where the $V_s(30)$ value might reflect the velocity of the underlying material. The higher velocity of the underlying units, which are either older Quaternary units or Upper Pliocene to Pleistocene fluvial deposits, or even Pre-Lower Pliocene basement rocks lead to slightly higher $V_s(30)$ values and higher standard deviation results for this unit. The composite profile for the Alluvium of the D-Site unit is similar to that of the Alluvium of the E-Site unit showing low-velocity materials at the surface and a gradual increase in velocity to about 30 m. The histogram given in Figure 5.7 shows the distribution of site classes for the Holocene Alluvium of the D-Site unit based on $V_s(30)$ data.

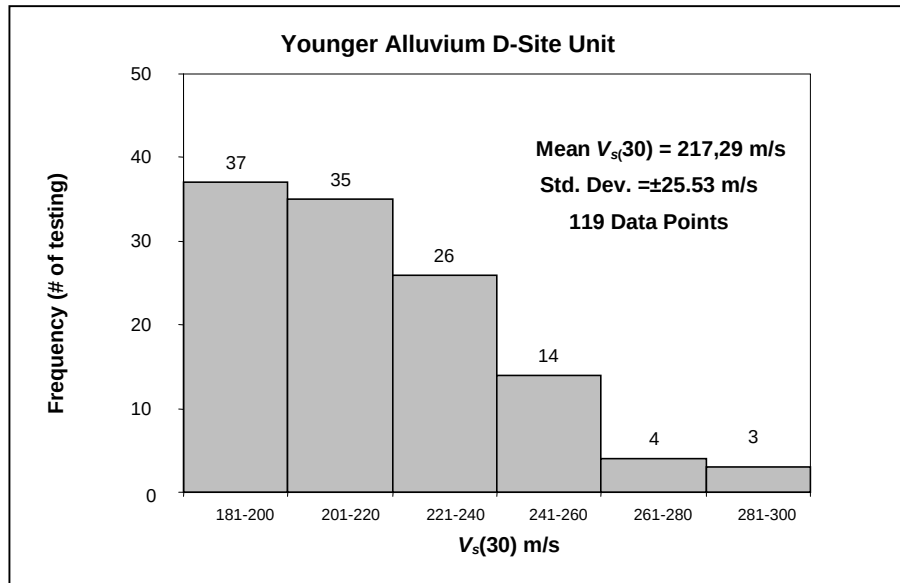


Figure 5.7. Histogram of $V_s(30)$ for distribution of site classes for the Younger Alluvium D-Site unit of the Quaternary Alluvium.

Considering the Older Quaternary or Terrace Deposits, $V_s(30)$ was calculated for 15 profiles that were described as Upper Pleistocene alluvium. Of the 215 measured $V_s(30)$ values, only about 6.98% of the data fell within this unit of the Quaternary alluvium since the in-situ seismic testing study in the project area was mainly conducted on Holocene age Quaternary alluvial deposits that are more susceptible to seismic hazards. However, the compiled data gave invaluable information related to the local site conditions and depositional environment of these deposits. The mean shear wave velocity was about 251 m/s and the $V_s(30)$ values ranged from 218 to 284 m/s, skewed toward the high end of the range for the category which showed the highest velocity in the entire Quaternary units. Since this unit outcrops at relatively higher elevations surrounding the younger alluvium and is intercalated with older fluvial deposits, it is not as homogenous as the younger alluvium deposits, and is occasionally confused with the Upper Pliocene to Pleistocene fluvial deposits, especially at higher terrace elevations. The average

velocities are apparently influenced from the underlying stiffer fluvial deposits that have as high as 410 m/s velocities most likely due to the thickness of the softer alluviums that are relatively thinner. The histogram given in Figure 5.8 shows the distribution of site classes for Quaternary Terrace deposits based on $V_s(30)$ data.

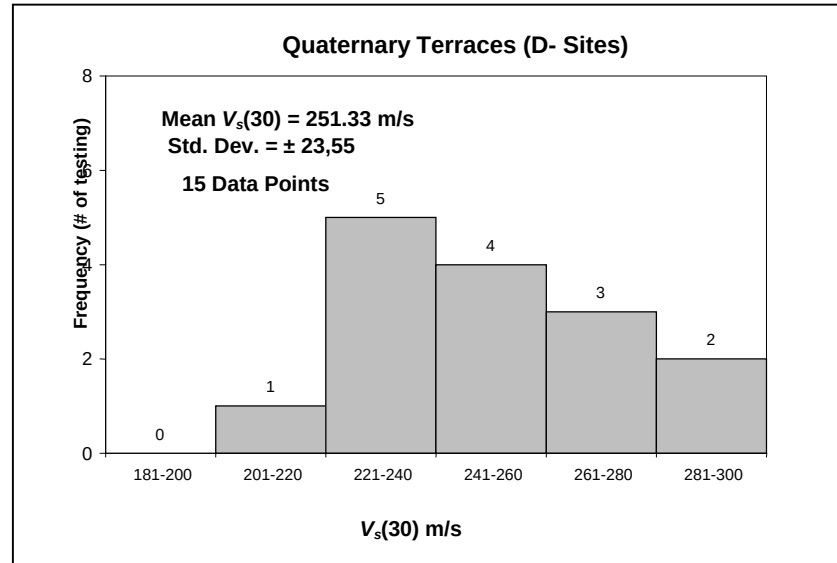


Figure 5.8. Histogram of $V_s(30)$ for distribution of site classes for Terrace Deposits (D-Site) in Quaternary Deposits.

5.3.1.1.2 Site Conditions of Upper Pliocene to Pleistocene Fluvial Red Clastics

Regarding these deposits, which are described as a undifferentiated single unit on the general geologic map, the average shear wave velocity results were calculated at 42 testing points. These basin fill types of sedimentary units are widely exposed and cover the major part of the study area which is situated towards the western part of the Ankara basin. At most of the testing locations where these fluvial deposits were observed at the surface, their thickness was determined to be more than 30 m in general where bedrock units were not encountered in the first 30

m. They are somewhat cemented, deformed and uplifted (situated in higher elevations) as compared to the surrounding environment consisting of alluvial terrace or younger Quaternary deposits. However, they were observed to be intercalated with these units at outlets of the basin. Because of this reason, relatively younger and partly consolidated fluvial deposits may sometimes get confused with the Quaternary terrace deposits, especially at higher terrace elevations. Hence, these deposits have generally been included in and mapped as the fluvial deposits in literature, which has proven to be one of the main difficulties during the site characterization study of the Ankara basin. These sediments have not been differentiated and mapped in as great a detail as the other units. The histogram given in Figure 5.9 shows the distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits based on $V_s(30)$. The mean shear wave velocity results for these units is about 343 m/s and the variability (Std. Dev. = ± 42 m/s) of the data is noticeably high with a range from about 250 m/s to 451 m/s. A possible reason for the variability might be because these older sediments encompass a wider range of textures, such as sandy clay and gravel that are found in the Holocene deposits. In addition, post-depositional conditions might have been more variable within the older deposits, resulting in a wider range of physical properties within the textural groups. Therefore, these fluvial deposits are mostly distributed at the boundary between Site-D and Site-C according to the IBC2003 building code. This poses a problem for mapping shear-wave velocity groups using geologic units and for assigning site classes (i.e., whether the fluvial deposit be grouped with soil profile site-C or classified as site-D for conservatism). However, it can be observed from Figure 5.9 that the variability of the V_s profiles in the fluvial deposits has two distinct peaks or characteristics within the two different IBC site categories of D and C. Consequently, the results of the measurements of $V_s(30)$ in Upper Pliocene to Pleistocene Fluvial Deposits in Ankara basin led to a decision to classify them as relatively less dense or stiff Fluvial Deposits of D-Site (Pleistocene?) and stiffer or denser Fluvial Deposits of D-Site (Upper Pliocene?; Table 5.7). Note that the

number of $V_s(30)$ data measurements using in these deposits are relatively less when compared with those of the Quaternary deposits because the seismic testing study was mainly aimed at Holocene age Quaternary alluvial deposits that are more susceptible to seismic hazards. In addition, the site characterization database gathered from the data collected from previous investigations contain sufficient amount of data on Upper Pliocene to Pleistocene fluvial deposits that enable the characterization of the local site conditions and the correlations of the results with the $V_s(30)$ measurements for the fluvial deposits. The $V_s(30)$ measurements and their correlation with previous investigations give invaluable information regarding the local site conditions and depositional environment of these fluvial deposits.

Table 5.7. Summary of the statistical results of $V_s(30)$ for Upper Pliocene to Pleistocene Fluvial Deposits and two site categories within these deposits.

Geologic Unit	No. of Data	Percentage (%)	$V_s(30)$ (m/s)			SITE CLASS (IBC 2003)	General Descriptions
			Mean	Std. Dev. ($\pm$)	Range		
L. Pliocene to Pleistocene Fluvial Deposits	42	100	343	42	250-451	D-C	Upper Pliocene to Pleistocene
Fluvial Deposits of D-Site	28	66,67	319	23	250-358	D	L. Pleistocene?
Fluvial Deposits of C-Site	14	33,33	392	27	361-451	C	Upper Pliocene?

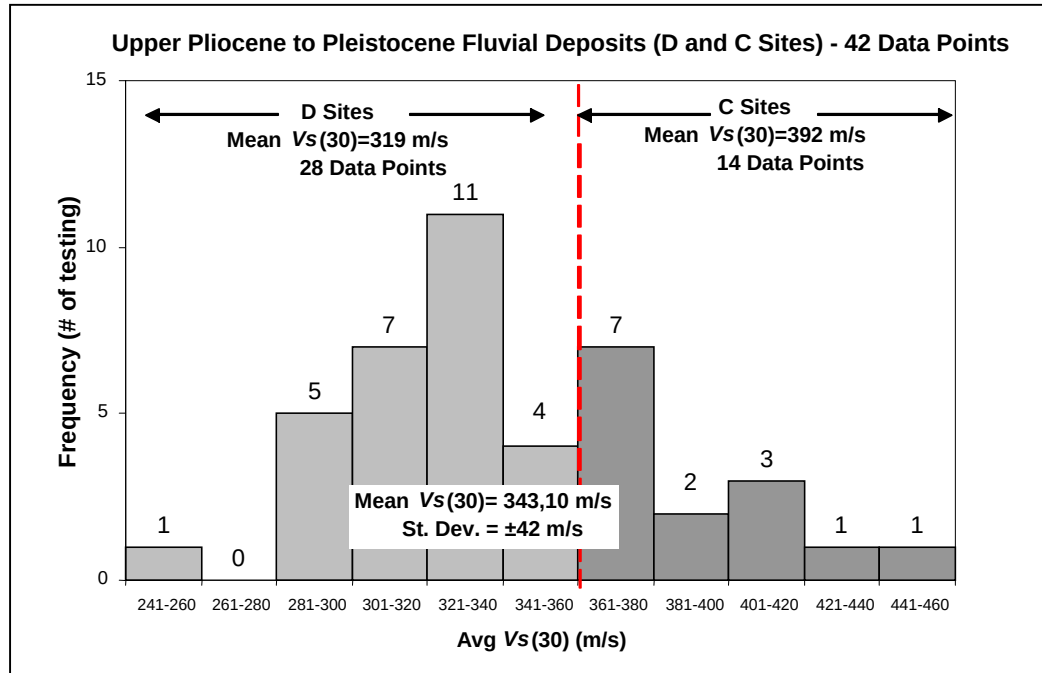


Figure 5.9. Histogram of $V_s(30)$ for distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits.

$V_s(30)$ was calculated for 28 profiles in the Fluvial Deposits of the D-Site unit that appears to be younger fluvial sedimentary deposits (i.e., Early Pleistocene) since this unit consists of locally consolidated and poorly indurated fluvial deposits and the measurements of $V_s(30)$ results are considerably lower when compared to the other fluvial unit. This unit may sometimes be confused with the Quaternary terrace deposits having similar soil texture. Of the 42 measured $V_s(30)$ values in the fluvial deposits, two-thirds of the data fell within this unit. This unit was generally encountered to be more than 30 m thick at the test locations. The distribution of $V_s(30)$ in this unit was generally uniform and the variability (Std. Dev. = ± 23 m/s) of the data was relatively narrow with respect to other units. The profile of the data showed a gradual increase in $V_s(30)$ from about 220 m/s at the surface to over 540

m/s at a depth of 30 m. The mean shear wave velocity was about 319 m/s and the $V_s(30)$ values ranged from 250 to 358 m/s showing somewhat closer values with relatively higher velocities with respect to the Quaternary terrace deposits. The histogram given by Figure 5.10 shows the distribution of site classes for the Fluvial Deposits of D-Site based on $V_s(30)$ data.

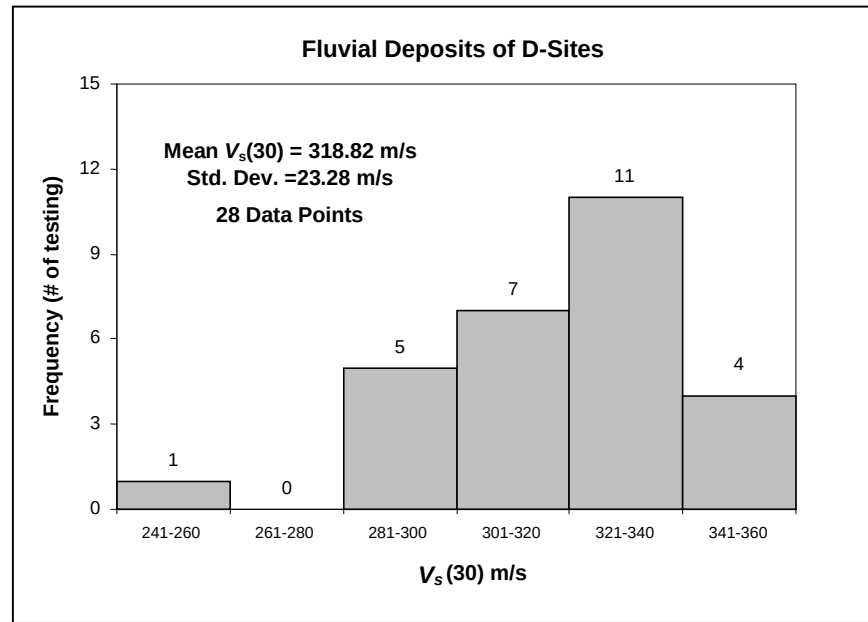


Figure 5.10. Histogram of $V_s(30)$ for distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits of D-Site.

In the Fluvial Deposits of C-Site, $V_s(30)$ was calculated for 14 profiles that can be also described as relatively older sedimentary fluvial deposits (i.e., Upper to Middle Pliocene ?) because this unit consists of stiffer, consolidated and cemented fluvial deposits, and the measurements of the $V_s(30)$ results are the highest when compared with the other characterized units of the Ankara basin. Of the 42 measured $V_s(30)$ values, one-thirds of the data fell within this unit of the fluvial

deposits. At some testing points, these fluvial deposits were observed to be more than 30 m thick, but considerable amount of stiffer deposit or bedrock units were encountered in the composite profile of the first 30 m as well. Therefore this composite profile for unit of Site Class C shows a relatively rapid increase in velocity from about 300 m/s at the surface to about 830 m/s at 30 m. With only 14 measured profiles, it is less likely to be representative of the entire unit but still gives a general idea related to the local site conditions and the depositional environment. As mentioned previously, other site characterization data (i.e., standard penetration results) obtained from previous investigations in regards to these fluvial deposits will be used as well to characterize the local site conditions and will be correlated with the $V_s(30)$ measurements for this unit. Note that the seismic testing studies were mainly conducted on the younger alluvial deposits in the research area since they are highly susceptible to seismic hazards. The distribution of the $V_s(30)$ results were somewhat variable for this unit (Std. Dev. = ± 27 m/s) as compared to the other units. The mean shear wave velocity was about 392 m/s and the $V_s(30)$ values ranged from 361 to 451 m/sec showing the highest velocities with respect to the entire sedimentary units in the Ankara basin. The histogram given below in Figure 5.11 shows the distribution of site classes for the Fluvial Deposits of C-Site unit based on $V_s(30)$ data.

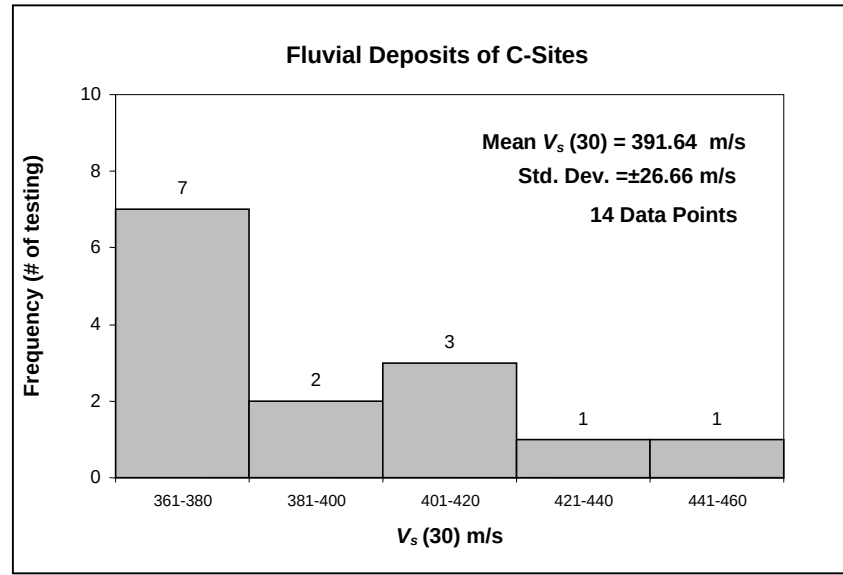


Figure 5.11. Histogram of $V_s(30)$ for distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits of C-Site.

5.3.1.1.3. Shear-Wave Velocity Regional Map Characteristics of Geological Units Susceptible to Seismic Hazards in the Ankara Basin

Shear wave velocities have been synthesized and aggregated in each of the units defined for sedimentary deposits that can be used to construct a map of estimated mean velocities for assessing local site characteristics to be used in predicting the site response of the Ankara basin. The range of velocities for a given map unit is dependent on the variety of materials which have been included in the unit. The sediments have been differentiated in the depositional environments on the basis of relatively well-preserved original geologic and geomorphic expressions and the field studies that were performed. Deposits in each of these environments show characteristic physical properties such as texture, density and depth of soil deposits. In addition to these physical and genetic criteria, the age of the deposits has been used as an important factor in forming a geologic map of these

sedimentary units. In order to prepare a map of the mean shear-wave velocities, correlations were established between seismic velocity and other more readily obtainable physical properties of the geologic materials. These correlations may be applied to a particular area by using data related to the areal distribution, physical properties, and thickness of the geologic units present to estimate and map shear-wave velocities.

Therefore, a site condition map was developed by grouping geologic units with similar physical properties into categories that are expected to have similar shear wave velocity characteristics, in addition, composite $V_s(30)$ profiles for these geologically defined units show that most units do have relatively distinct shear wave velocity properties. Then, the $V_s(30)$ results for each testing point were contoured to provide a map of mean shear-wave velocities for the Ankara basin. It is anticipated that this study will prove to be quite useful in preparing regional seismic-hazards maps in which the V_s characteristics can be used as part of a site-conditions term. Consequently, there appears to be a reasonable agreement with respect to surface geology and the regional seismic map of $V_s(30)$ prepared in regards to the site classes specified in the IBC 2003 (Figure 5.12). Softer sites E and D were mostly situated in the areas that were classified as Quaternary alluvial and terrace deposits while stiffer sites D and C were mostly situated in areas that were classified as Upper Pliocene to Pleistocene fluvial formations.

In order to define site conditions with respect to a code-based site classification through reflecting the shear-wave velocity data, the test location of all $V_s(30)$ measurement points were plotted on the map of the Ankara basin by using a grid system and a GIS query. If the geological units have been appropriately grouped, the distribution of $V_s(30)$ for each unit were observed to be distinct with a few overlaps.

During the regional site characterizations, the results obtained were mapped using GIS techniques by applying linear interpolation among the points, thus enabling a smooth transition of the selected parameters. Soft transition boundaries

were preferred to show the variation of the mapped parameters. Better defined sharp boundaries were avoided as much as possible due to the accuracy of the study and in addition to allow some flexibility to the urban planners and to avoid misinterpretation by the end users that may consider the clear boundaries as accurate estimations of the different zones.

It is important to note that these results were based on representative site profiles of $V_s(30)$ that were estimated by using the available conducted and collected data based on the Code Based (IBC2003) site classification methods and the site classification map might be one of the best methods for assessing the local site conditions to be solely used for regional design purposes and not for individual structural design purposes. The regional map that was digitized and generalized from the 1:25,000 scale map was clearly not appropriate for this type of site-specific use. For example, especially in case of high rise, high priority buildings and industrial facilities, detailed investigation of local soil conditions and determination of site specific design parameters must be obligatory in evaluating the seismic forces controlling the structural response (General Directorate of Disaster Affairs, MERM, 2004). Although, it is believed that the map is a very useful tool in assessing regional seismic hazards, its use for site-specific evaluations is discouraged since site-specific soil investigations are required to determine local site conditions for each building area to be used in the structural design. From this perspective, the seismic zonation performed by this study or even a microzonation study may seem relatively inadequate because evaluating the design parameters for each lot in the selected area is not the objective herein.

Additionally it should be noted that since performing shear-wave velocity measurements in bedrock was out of the scope of this research, bedrock related data are not indicated on the seismic zonation map of $V_s(30)$.

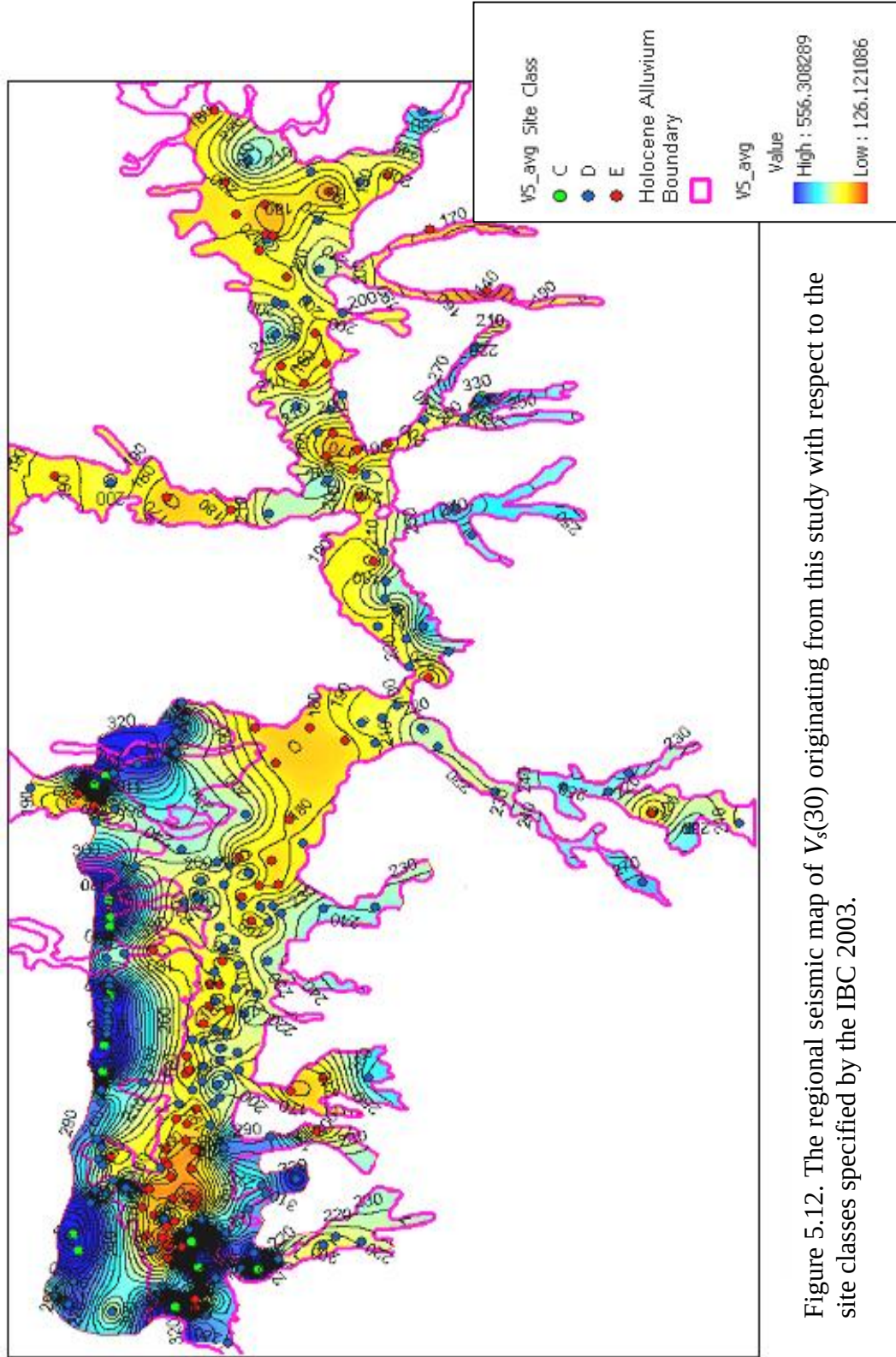


Figure 5.12. The regional seismic map of $V_s(30)$ originating from this study with respect to the site classes specified by the IBC 2003.

5.3.1.2. Site Conditions Based on Standard Penetration Results of the Sedimentary Geological Units of the Ankara Basin

The shear-wave velocity is an important factor in determining the amplitude of ground motion because it directly affects ground motion amplification and thus might be a useful parameter in characterizing the local geologic conditions particularly in assessing seismic hazards. Nevertheless, as an alternative, where the soil average shear wave velocity is not known and/or when the characterization of a geologically complex region is to be pursued especially in younger deposits, the physical properties of geologic materials for classifying the sites can be determined from average standard penetration resistance results in regards to code-base site characterizations. Then, the average shear wave velocities as well as the standard penetration results of the upper 30 m of near-surface geologic units must be estimated to characterize the geological units for evaluating local site conditions for seismic zoning. Ideally, more accurate soil categorizations are performed based on a better description of soil profiles using standard geotechnical parameters (i.e., standard penetration results, density, plasticity index, void ratio and grain size) along with the average $V_s(30)$ values. This way, useful correlations between shear-wave velocity and standard penetration resistance data may be obtained in the project area for a more thorough site classification for building codes that is far more useful in regards to engineering design.

In this section, our objective is to identify the local site geology and construct a map of the site characterization by using standard penetration results along with the in-situ seismic wave velocity measurements as previously mentioned in Section 5.3.1.1. These penetration testing results were compiled from the boring studies performed in the area during previous geotechnical investigations in the Ankara basin. During the standard penetration based site characterization study, a similar site classification scheme that was outlined in the seismic wave velocity measurement section was used to assemble a consistent and well distributed site

classification results and also to perform useful correlations between each test to idealize the site character. Regarding this classification scheme, geologic units were briefly subdivided into the three units, and then the average penetration results of these statistically distinct units were investigated probabilistically.

A total of 949 borehole data have been compiled from a variety of reliable, subsurface engineering studies previously performed by a variety of credible engineering firms for site characterization of the Ankara basin. The standard penetration tests performed in borings sunk down to depths of between greater than 20 m but less than 30 m were linearly extrapolated to depths of 30 m. The general distribution of site characterization data according to geological settings and average standard penetration results are shown in Figure 5.13.

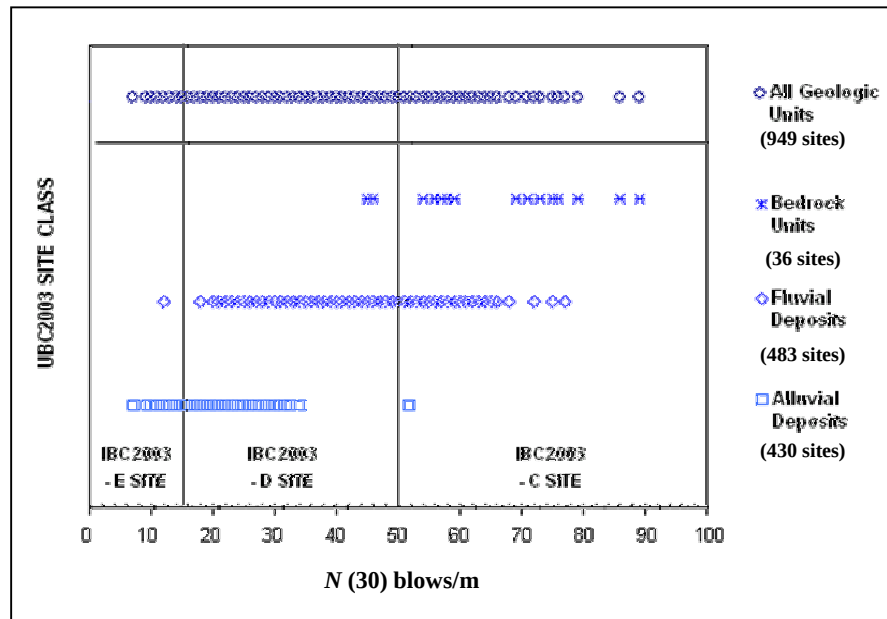


Figure 5.13 The general distribution of site characterization data according to geological settings and average standard penetration results.

However, the compiled boring data is not well distributed as the conducted seismic testing studies over the project area and includes a spectrum of different geologic materials with a relatively higher amount of data in Upper Pliocene to Pleistocene fluvial red clastics and a lesser amount of data on Quaternary alluvial and terrace sediments. There was a lack of data particularly in Holocene alluvial deposits that are dominantly distributed around the Sincan County. Hence, for the effectiveness of this research project, the surface wave measurements have been conducted primarily in the Quaternary alluvial and terrace sediments in the Ankara basin to develop a consistent and well distributed database.

In this characterization study, standard penetration data for borings sunk down to at least 20 m were taken into consideration. In other words, some of the borings did not penetrate down to 30 m most probably due to adverse ground conditions, shallow bedrock conditions, etc. Regarding the compiled boring profiles from various studies, about two-thirds of the profiles extended to at least 30 m, while about one-thirds of them penetrate less than 20 m. Linear interpolation has been applied for the shallow borings that are less than 20 m in order to extrapolate them to a depth of 30 m and hence, to obtain $N(30)$ plots for the study area.

A summary of the $N(30)$ results obtained in the sedimentary geological units and their IBC site classes based on $N(30)$ are given in Table 5.8. The histograms presenting a summary of the data distribution considering the IBC2003 site classes for each related geological unit are also given in Figures 5.14 and 5.15. It can be observed from Table 5.8 that the density and distribution of the compiled site characterization database were mainly collected in Upper Pliocene to Pleistocene fluvial deposits and relatively of fewer amounts and unevenly distributed in the Quaternary alluvial and terrace sediments.

Table 5.8. Description of the characteristics of generalized geologic units and their IBC2003 site classes based on $N(30)$ data. 949 data points.

Geologic Unit	# of Data Points	Percentage (%)	$N(30)$	$\pm$ Std	SITE CLASS (IBC 2003)
Quaternary alluvial and terrace deposits	0	-	-	-	CLASS B
	1	0,23	52,0	-	CLASS C ¹
	323	75,12	25,4	6,2	CLASS D
	106	24,65	12,7	1,8	CLASS E
Upper Pliocene to Pleistocene fluvial deposits	0	-	-	-	CLASS B
	78	16,15	55,8	5,8	CLASS C
	404	83,64	38,5	6,4	CLASS D
	1	0,21	12,0	-	CLASS E ²
Pre-Lower ³ Pliocene basement rocks	16	44,44	N/A	N/A	CLASS B
	18	50,00	69,6	11,1	CLASS C
	2	5,56	45,5	1,1	CLASS D
	0	-	-	-	CLASS E

Note 1. 1C site in Quaternary was erroneously encountered due to the presence of artificial fill (pre-emplaced fill materials) at this location, therefore the data point was omitted;

Note 2. 1E site in Fluvial deposits is not consistent with the remainder of the data, therefore, the data point was omitted;

Note 3. The compiled in-situ tests results for bedrock units are presented above for showing the consistency and general trend of the data, it should be noted that these units are out of the scope of seismic hazard study; N/A = Not applicable.

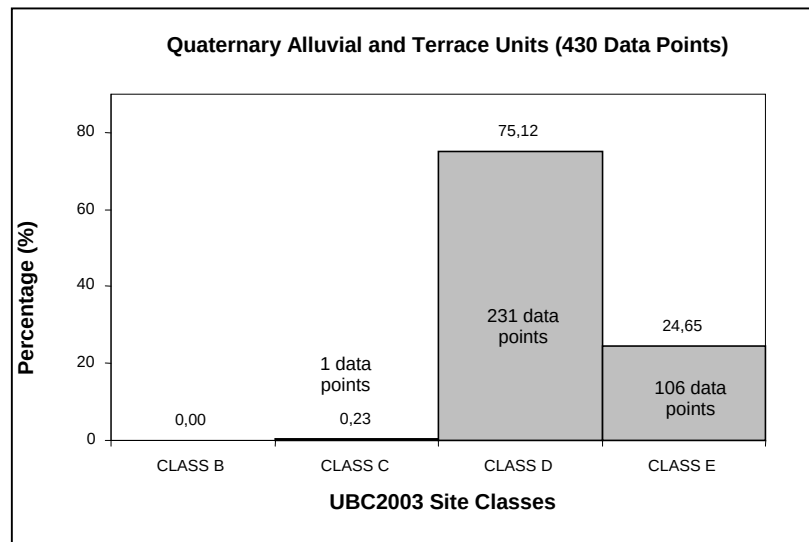


Figure 5.14. Data distribution considering the IBC site classes for Quaternary alluvial and terrace deposits based on $N(30)$ data.

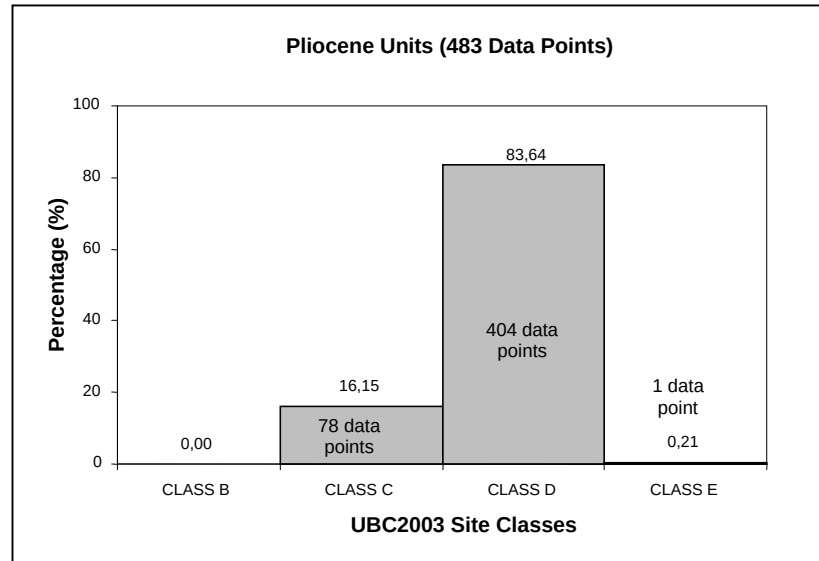


Figure 5.15. Data distribution considering the IBC site classes for Upper Pliocene to Pleistocene fluvial deposits based on $N(30)$ data.

During the interpretation of the $N(30)$ data, it was observed that the distribution of the penetration results and hence, the site classes for some geologic units were relatively variable was also observed for the $V_s(30)$ results of site classes, especially in Quaternary alluvial and fluvial deposits of D-Sites, whereas the distribution of $N(30)$ was more uniform for Quaternary deposits of E-Sites that had a relatively narrow range of average penetration results. Therefore, the Quaternary alluvial and fluvial deposits had quite variable ranges of $N(30)$ that crossed boundaries between the existing standard penetration categories of younger and older sedimentary or bedrock units. One of the main reason for this crossing was the variability of sample textures which is an indication of the range of depositional environments represented by the sediment included within each sedimentary unit. The depositional environments have generally resulted in a wider range of physical soil properties causing deposition of a variety of soil textures that can influence standard penetration results as well as shear wave velocity results as discussed in Section 5.3.1.1.

As a result, some sedimentary units did not exactly fit into the defined code-based penetration result categories, and then these units might be differentiated within different site categories. For example, the units such as Upper Pliocene to Pleistocene fluvial deposits and Quaternary alluvial deposits which have a $N(30)$ distribution that crosses the boundary between IBC2003 site categories D and C, and E and D, respectively. Finally, it might be concluded that site characterization depending on the average standard penetration results is a favorable tool as like the shear wave velocity results in measuring geologic conditions; and they give valuable outputs to idealize the local site conditions, which might be suitable to classify the characteristics of the general sedimentary mapping units into the more detailed sub-classes that are consistent with the age and depositional character of geologic units.

Regarding the bedrock units, it should be noted that these lithologies, that are out of the scope of seismic hazard study will only be mentioned briefly herein. These units have a highly variable distribution of $N(30)$ values (Table 5.8). In general, the assignment of standard penetration classes to the bedrock geological units were not uniform. For example, relatively older bedrock rocks (i.e., Mesozoic to older) were generally classified as Class B in the literature, however, as would be expected, these rocks are more deformed and fractured in tectonically active areas as compared to those in inactive areas. Therefore, they might be classified as weaker or softer geologic units in between the site classes of B and C. In the western part of Ankara, these extensive geologic units (Triassic greywackes, Upper Jurassic-Lower Cretaceous limestones) tend to be regionally variable both in lithology and amount of deformation, and so these units are mapped slightly differently on the geological maps. According to the penetration results, few boring results collected from these units (particularly at 20 sites, two (2) of them are B-Site Class) indicate that they are moderately fractured and weathered rock units, and so they are classified as in between the site class of B and C. It was previously stated that Site Class B should be either measured on site or estimated by an engineering

geologist for competent rock with moderate fracturing and weathering (ICC, 2003). Softer and more fractured and weathered rock was classified on site through the standard penetration results as Site Class C according to IBC2003.

A similar lithological variability is also observed in Lower Pliocene to older Cenozoic basement rocks (Upper Miocene-Lower Pliocene volcanics, fluvial-lacustrine sedimentary rocks that were discussed in Chapter 2). Generally, volcanic rocks tend to be hard, but extensively fractured in which most were classified as B Site and sometimes C Site according to the degree of fracturing and weathering. The sedimentary rocks, relatively medium to soft rock units, have highly variable velocity characteristics, and so are sometimes confused with the relatively older, stiffer, consolidated and cemented Upper Pliocene fluvial deposits. The classifications by $N(30)$ is somewhat easier, however, because they have highly variable geologic character based on depositional characteristics, age and texture, all of which may correlate with penetration results. $N(30)$ values for these units generally are between 54 and 89 blows/0.3 meter, which fits into the C class.

The younger Tertiary sedimentary units are classified according to the penetration results as medium to soft rock with C-Site Class (at particularly 14 sites where 4 of them were composite profiles) and rarely soft rock as D-Site (particularly at 2 sites where both of them were composite profiles). It is important to note that composite profile is less likely to be representative of the unit as a entire unit for depths less than 30 m. As an observation, exceptions to this general scheme that based on the lithology of an individual formation were made when a sedimentary unit was known to be notably coarse-grained; in which case it was classified C Class. As a result, it can be concluded that it is difficult to generalize these lithological rock units and to assign site classes for the entire area based on the physical character and textural information. Hence, during determination of the code-based site classes, standard penetration and/or shear wave velocity results that should be used are one of the most appropriate measures of rock and soil conditions to determine the variations in local site character and to idealize the local geology.

5.3.1.2.1 Site Conditions of Quaternary Alluvial and Terrace Deposits

Regarding these deposits, the average standard penetration results were calculated for 429 $N(30)$ testing points in the Quaternary deposits which are generally described as younger, Holocene or undifferentiated Quaternary alluvium units and older Quaternary terrace units with similar geological boundaries in site characterization scheme that was discussed in the Section 5.3.1.1.1. As mentioned before, Holocene alluviums that cover the major part of the alluvium is generally less than 30 m thick and the terrace deposits are situated at relatively higher elevations in the surrounding environments of the younger alluviums. They can generally be observed from the boring logs, most of the testing points with younger alluviums at the surface generally include older alluviums (Pleistocene or terrace) within the upper 30 meters. Hence, the transition from Holocene to terrace deposits might be differentiated in profiles and boring logs to determine the thickness of the young alluvium and also by using any other supportive site characterization methods. The histogram given in Figure 5.16 shows the distribution of site classes for all Quaternary alluvial and terrace deposits based on $N(30)$. It should be noted that one pair of data (IBC Site-C class) that was inconsistent (i.e., above of the expected range of $N(30)$ for Quaternary alluvial sediments) was not taken into account during the interpretation of the site characterization. The mean standard penetration result for Quaternary geologic unit was about 23 blows/0.3m. The $N(30)$ results for Quaternary geologic units ranged from about 7 to 48 blows/0.3m that appeared to be highly variable (Std. Dev. was about ± 7.6 blows/0.3m) and ranged between the border between IBC2003 site categories of E and D.

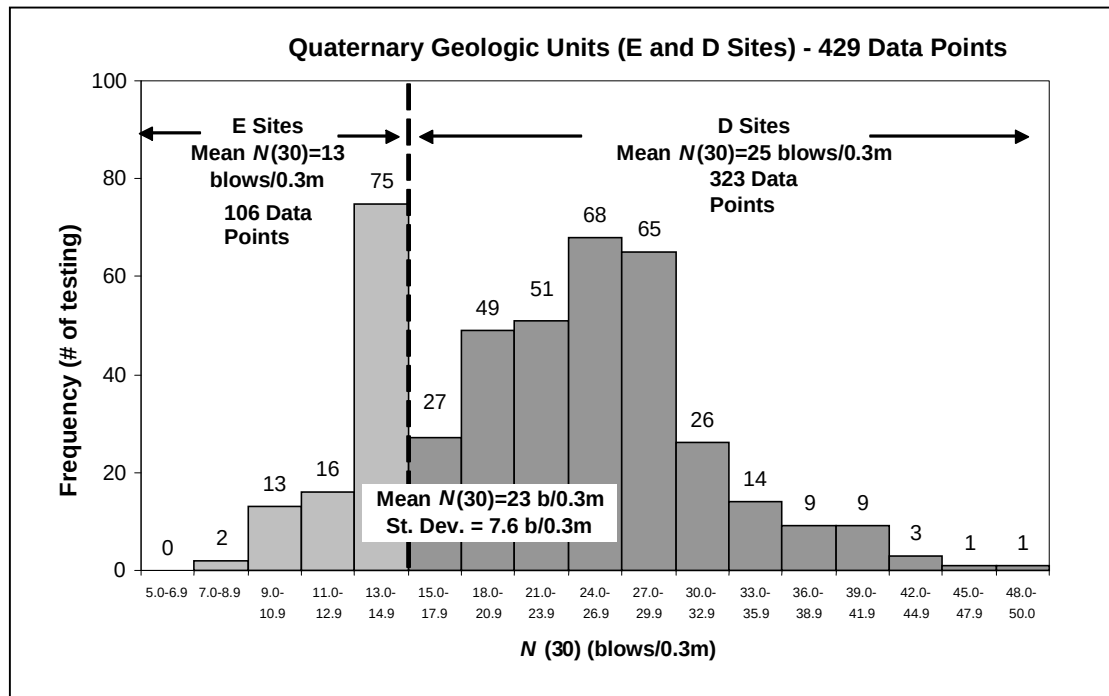


Figure 5.16. Histogram of $N(30)$ for determining site classes for Quaternary alluvial and terrace deposits (Note: one pair of data falling in the C-sites class was not taken into account as explained in text).

The variability of the $N(30)$ data in the unconsolidated deposits might be due to the variation of the material properties including grain size, depth, density, and ground water levels, and also due to lack of differentiation of the local soil conditions of the Quaternary alluvium in the geologic map. These observations were extensively discussed during the assessment of the shear wave velocity characteristics in Section 5.3.1.1, and seems to control the consistency and reliability of the general site characterization results through interpreting the boring profiles and their characteristics of standard penetration results. Additionally, some methodologies used for shear-wave velocity measurements were applied together with the standard penetration results as well to assess the soils descriptions/characterization of younger alluvial sediments in the Ankara basin.

Regarding the variability of the penetration characteristics of Quaternary sedimentary units, they were classified within three different units, and then characterized with respect to these three units in the Ankara basin. Therefore, Quaternary deposits were initially classified as younger (Holocene or undifferentiated) and older (terrace deposits or Pleistocene) Quaternary. The younger alluvium deposits that may also show a distinct variability of penetration results and display two different peaks (Figure 5.16) however, commonly represent the results of deposition in environments similar to those present nearby today. Therefore, the environment of deposition that has a great deal of control over the sediment characteristics and physical properties, and these factors may also correlate well with the standard penetration results and also shear wave velocity results (Section 5.3.1.1). Regarding the evaluation of these variability characteristics, younger Holocene alluvium deposits were classified as two different IBC2003 site categories of E and D according to their variability of $N(30)$ profiles with thickness and depositional texture of the alluvium.

Consequently, Quaternary alluvium deposits in the Ankara basin were differentiated as three different units of younger alluvium of E-Site, younger alluvium of D-Site and older Quaternary deposits according to results of the measurements of $N(30)$ that are summarized in Table 5.9. The statistical results (i.e., mean, standard variation, range and sample size) of $N(30)$ for Quaternary deposits and each three site categories within these deposit are also given in Table 5.8.

Table 5.9. Summary of the statistical results of $N(30)$ for Quaternary deposit and each three site categories within these deposits.

Geologic Unit	No. of Data	Percentage (%)	$N(30)$ (blows/0.3m)			SITE CLASS (IBC 2003)	General Descriptions
			Mean	Std. Dev.(±)	Range		
Quaternary Deposits	429	100	23	7.6	7-48	D & E	Holocene-Upper Pleist
Younger Alluvium (E-Site)	106	24,71	13	1,8	7-14	E	Holocene
Younger Alluvium (D-Site)	231	53,85	23	4,4	15-34	D	Holocene
Older Quaternary or Terrace Deposits	92	21,44	31	6,4	18-48	D	Upper Pleistocene

For the Younger Alluvium of E-Site, $N(30)$ was calculated for 106 profiles that was described as Holocene alluvium. Of the 429 measured $N(30)$ values, 24,71% of the data fell within this unit of the Quaternary alluvium. The mean standard penetration value was about 13 blows/0.3m. The variability of the $N(30)$ values were very narrow (Std. Dev. = ± 1.8 blow/0.3m) with a range from 7 to 14 blows/0.3m which represented the unit with the lowest penetration characteristics of the entire Quaternary unit domain. As indicated by the penetration results, this unit appears to be composed of relatively soft, uniform, normally consolidated fine-grained materials (i.e., clay and silty clay with fine sand). In addition, the alluvium appears to be relatively thick within the first 30 m. The physical characteristics of this geological unit obtained through the penetration results show consistency with those estimated from shear wave velocity measurements. The histogram given in Figure 5.17 displays the distribution of site classes for Younger Alluvium of E-Site based on $N(30)$ data.

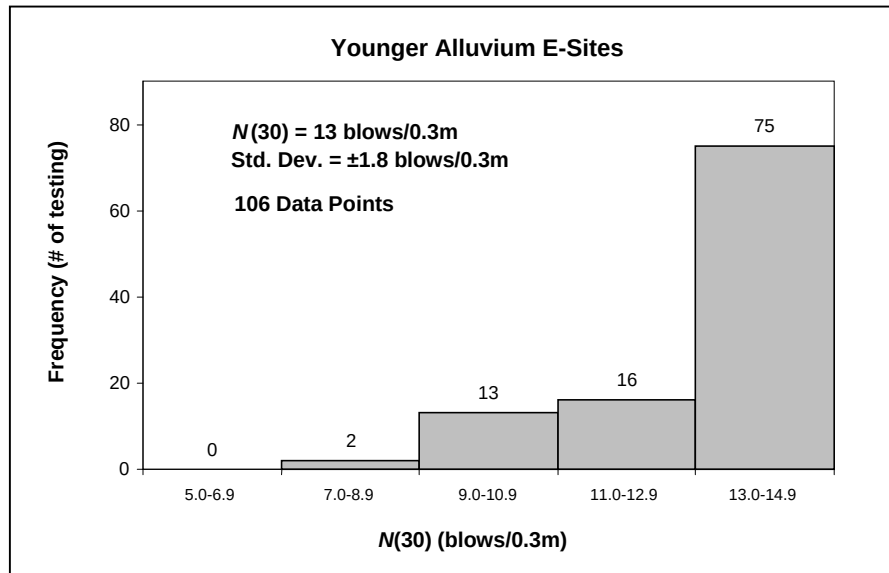


Figure 5.17. Histogram of $N(30)$ data for distribution of site classes for Younger Alluvium E-Site in Quaternary Alluvium.

For the Younger Alluvium of D-Site, $N(30)$ was calculated at a total of 231 profiles that were also described as Holocene or undifferentiated Quaternary alluvium. Of the 429 measured $N(30)$ values, 53.85% of the data fell within this unit of the Quaternary alluvium (Table 5.9). The mean standard penetration value was about 23 m/s and the $N(30)$ values ranged from 15 to 34 blow/0.3m showing somewhat similar characteristics, however relatively higher penetration results with respect to the Holocene Alluvium of E-Site was obtained most likely due to the relatively less uniform and coarse-grained material as well as the fine grained material content as compared to the Site-E. The distribution of the $N(30)$ results may be considered as variable (Std. Dev. = ± 4.4 blows/0.3m) which separate the unit from the Site-E unit. The depositional setting and thickness of the alluvial units based on the penetration results showed similar characteristics to that based on the shear wave velocity measurements that were discussed in Section 5.3.1.1.1. The histogram given in Figure 5.18 shows the distribution of site classes for Holocene Alluvium of D-Site based on $N(30)$ data.

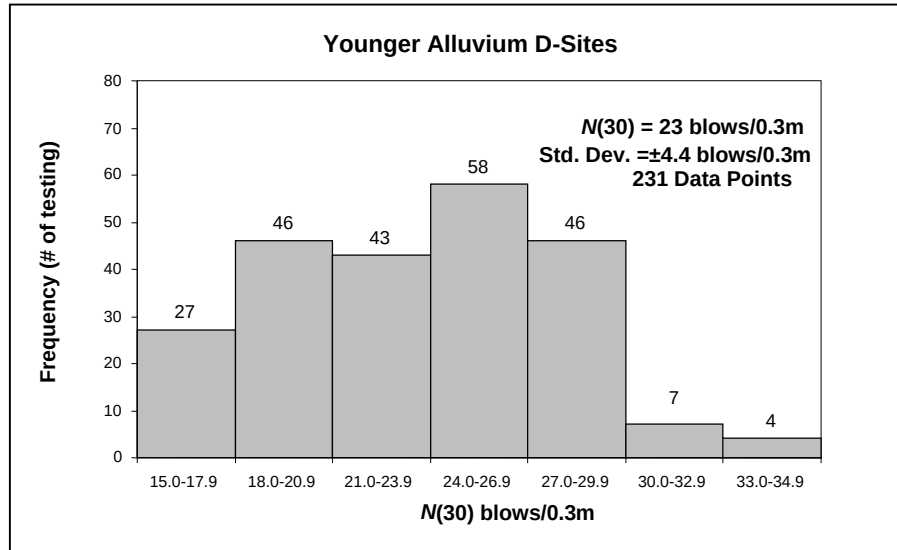


Figure 5.18. Histogram of $N(30)$ data for distribution of site classes for Younger Alluvium D-Site in Quaternary Alluvium.

For the Older Quaternary (terrace) Deposits, $N(30)$ was assessed at a total of 92 boring locations that were described as Upper Pleistocene alluvium. Of the 429 measured $N(30)$ values, 21.44% of the data fell within this unit of the Quaternary alluvium. On the contrary to the in-situ seismic testing study that was mainly conducted on Holocene Quaternary alluvial deposits (i.e., since it was considered to be more susceptible to seismic hazards), standard penetration data were generally compiled from geotechnical investigations related to Upper Pliocene to Pleistocene fluvial deposits in an attempt to characterize the local site conditions. Since the older Quaternary units are situated at relatively higher elevations than the surrounding environment of the younger alluvium that is mostly intercalated with older fluvial deposits that were not differentiated as separate units in the standard geologic maps of the Ankara basin, they may get confused with the Upper Pliocene to Pleistocene fluvial deposits. Therefore, local geological information along with the mapping of the surface geology of this unit was based on available sources of information and databases that were compiled from geomorphological maps,

satellite images and boring profiles. When compared with the conducted shear wave velocity measurements, since the compiled data was quite excessive in volume, it gave invaluable information regarding the local site conditions of the Quaternary terrace deposit that were correlated with the $V_s(30)$ measurements to aid in classifying the site conditions for seismic zoning. The distribution of the $N(30)$ results were highly variable (Std. Dev.= ± 4.4 blows/0.3m) which demonstrated the variability of the depositional and textural characteristics which show similarities to the characteristics of the older fluvial deposits discussed above.

The mean standard penetration results was about 31 blows/0.3m and the $N(30)$ values ranged from 18 to 48 blow/0.3m, skewed toward the high end of the range for the category, showing the highest penetration results of the entire Quaternary units. The histogram given by Figure 5.19 shows the distribution of site classes for Quaternary Terrace deposits based on $N(30)$ data.

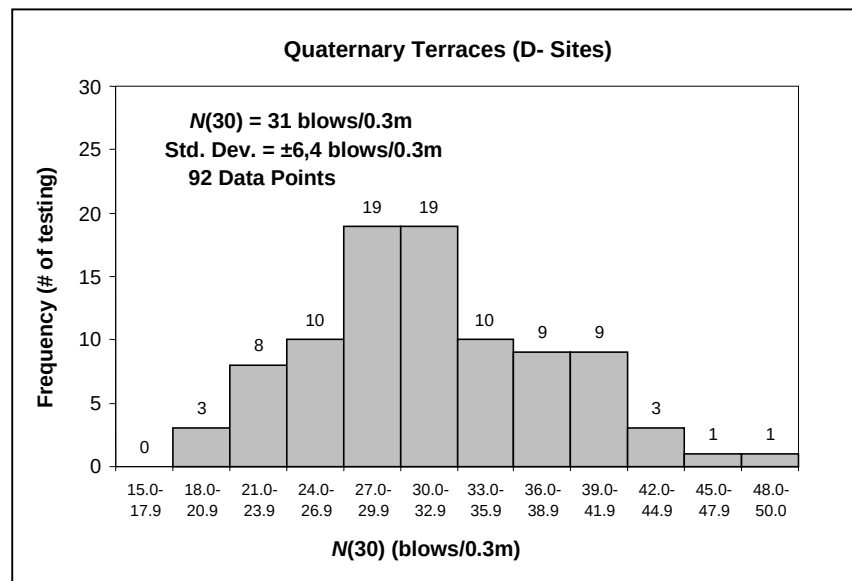


Figure 5.19. Histogram of $N(30)$ data for distribution of site classes for Terrace Deposits (D-Site) in Quaternary Deposits.

5.3.1.2.2 Site Conditions of Upper Pliocene to Pleistocene Fluvial Red Clastics

These deposits which are generally intercalated with Quaternary older terrace deposits that constitute a single mapping unit on the general geologic map were investigated at a total of 482 test locations. These basin fill types of sedimentary units are widely exposed and cover the major part of the study area which is situated in the western part of the Ankara basin. Since they are particularly cemented, deformed and uplifted as compared to the surrounding environment composed of alluvial terrace or younger Quaternary deposits, their average standard penetrations give relatively stiffer results owing to their general depositional characteristics that were extensively discussed in Section 5.3.1.1.2.

The histogram given in Figure 5.20 shows the distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits based on $N(30)$ data. The mean standard penetration results for these units was about 41 blows/0.3m and the variability (Std. Dev. = ± 9.0 blows/0.3m) of the data was noticeably high that ranged from about 25 to 77 blows/0.3m. These fluvial deposits were classified to be situated at the boundary between Site-D and Site-C IBC Codes. It might be observed Figure 5.20 that the variability of penetration results in the fluvial deposits has distinct characteristics within the two different IBC site categories of D and C considering the measurements of the average penetration results was observed for the average shear wave velocities in Section 5.3.1.1.2. These site categories gave relatively less dense or stiffer results for the Fluvial Deposits of D-Site (Pleistocene?) and stiffer or denser results for the Fluvial Deposits of D-Site (Upper Pliocene?) as summarized in Table 5.10. Note that the results obtained at one of the testing sites (IBC Site-E Class) that was inconsistent and above the expected range of the $N(30)$ for the fluvial sediments was not taken into account during the interpretation of the site characterization.

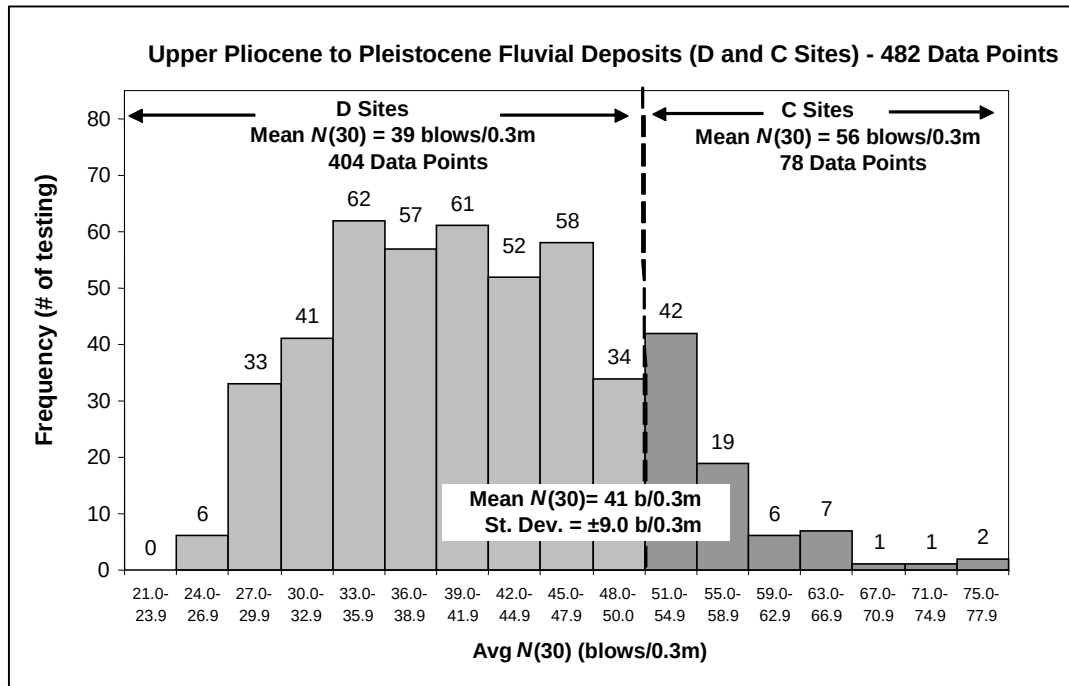


Figure 5.20. Histogram of $N(30)$ data for distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits (Note: One pair of data of E-site Class was not taken into account as explained in text).

Table 5.10. Summary of the statistical results of $N(30)$ for Upper Pliocene to Pleistocene Fluvial Deposits and two site categories within these deposits.

Geologic Unit	No. of Data	Percentage (%)	$N(30)$ (blow/0.3m)			SITE CLASS (IBC 2003)	General Descriptions
			Mean	Std. Dev. ($\pm$)	Range		
L. Pliocene to Pleistocene Fluvial Deposits	482	100	41	9,0	25-77	D-C	Upper Pliocene to Pleistocene
Fluvial Deposits of D-Site	404	83,82	39	6,4	25-50	D	L. Pleistocene?
Fluvial Deposits of C-Site	78	16,18	56	5,8	51-77	C	Upper Pliocene?

In the Fluvial Deposits of D-Site, $N(30)$ was calculated for a total of 404 profiles. This unit might be described as younger fluvial sedimentary deposits,

particularly Early Pleistocene because it is a locally consolidated and poorly indurated fluvial deposit that appears to be relatively younger and possesses lower $N(30)$ measurements. This unit occasionally gets confused with the Quaternary terrace deposits that were discussed in Section 5.3.1.1.2. Of the 482 measured $N(30)$ values, 83.82% of the data fell within this unit of the fluvial deposits. The distributions of $N(30)$ were variable (Std. Dev. = ± 6.4 blows/0.3m) showing similar characteristics to the younger terrace deposits because of the similar depositional and textural characteristics. Although, depending on the age of deposition, the mean standard penetration result was about 39 blows/0.3m and the $N(30)$ values ranged from 25 to 50 blows/0.3m showing somewhat closer penetration values to those of the Quaternary terrace deposits. The histogram given in Figure 5.21 shows the distribution of site classes for the Fluvial Deposits of D-Site based on $N(30)$ data.

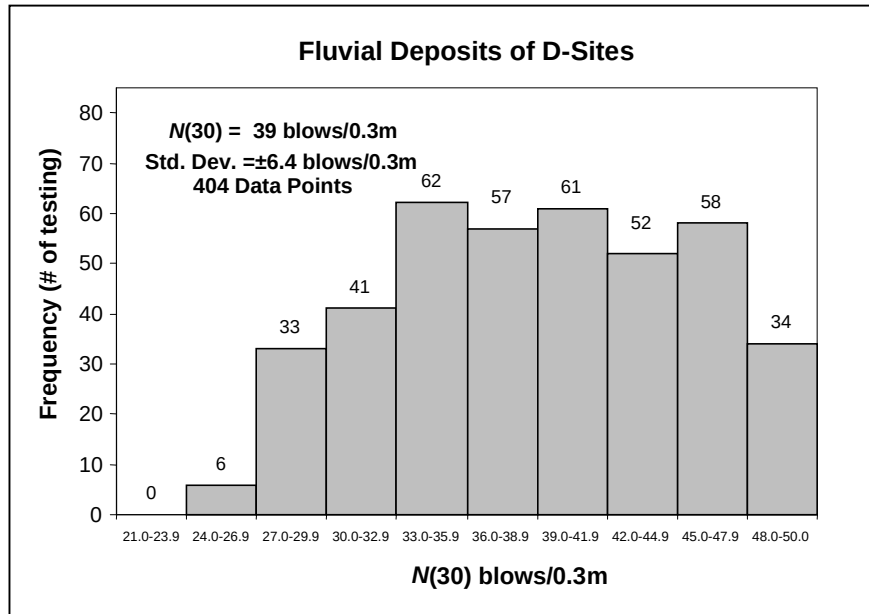


Figure 5.21. Histogram of $N(30)$ data for distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits of D-Site.

In the Fluvial Deposits of C-Site, $N(30)$ was calculated for a total of 78 profiles that may also be described as relatively older sedimentary fluvial deposits (i.e., Upper to Middle Pliocene?) because this unit consists of relatively stiffer, consolidated and cemented fluvial deposits, and the $N(30)$ results were the highest when compared with the other units characterized for the Ankara basin. Of the 482 measured $N(30)$ values, 16.18% of the data fell within this unit in the fluvial deposits. The general depositional characteristics of this unit were presented in Section 5.3.1.1.2. The distribution of the $N(30)$ results were variable as a distinct unit and the standard deviation was ± 5.8 blows/0.3m. The mean standard penetration and range was about 56 blows/0.3m and 51 to 77 blows/0.3m, respectively, showing the highest penetration results of the entire sedimentary units in the Ankara basin. The histogram given by Figure 5.22 shows the distribution of site classes for the Fluvial Deposits of C-Site based on $N(30)$ data.

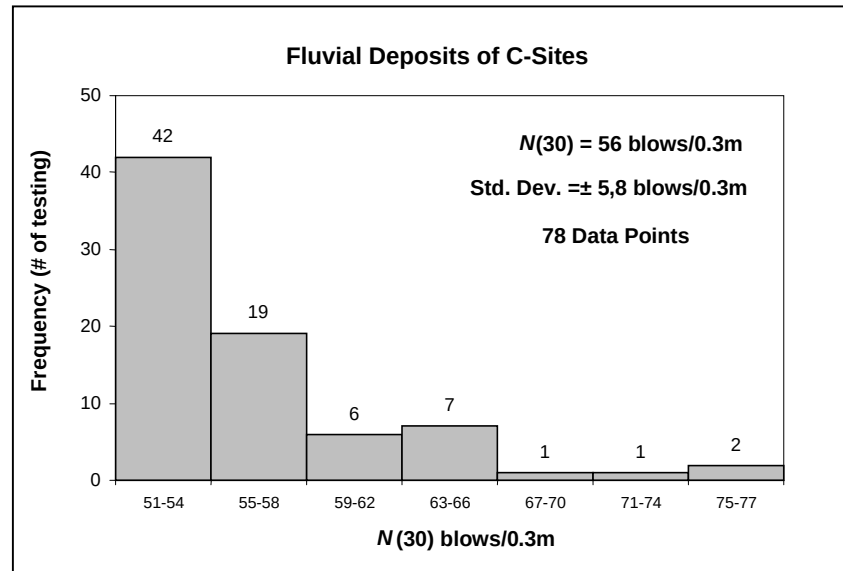


Figure 5.22. Histogram of $N(30)$ data for distribution of site classes for Upper Pliocene to Pleistocene Fluvial Deposits of C-Site.

5.3.1.2.3. Standard Penetration Resistance Regional Map of Geological Units Susceptible to Seismic Hazards in the Ankara Basin

As was performed for the shear wave velocities, average standard penetration results have been synthesized and aggregated in of the each sedimentary unit that might be used to construct a map of the compiled mean standard penetration results for assessing site conditions to use in predicting the site characteristics of the Ankara basin. The range of penetration results for a given map unit is dependent on the variety of materials which have been included in the units. These sedimentary units have generally been classified in the depositional environments on the basis of geological and geomorphological maps, boring profiles and field studies that were performed.

Consequently, a map of general site conditions was developed by classifying geologic units with similar physical properties into categories that are expected to have similar standard penetration characteristics and composite $N(30)$ profiles that were compiled for these geologically defined units indicated that most units possessed relatively different standard penetration characteristics. Then, the assigned $N(30)$ results for each testing point were contoured by interpolation to provide a zonation map of the mean penetration results of the Ankara basin. It was observed that these results might be useful in preparing regional seismic hazard maps, however the compiled data that was larger in volume was naturally not as orderly distributed (i.e., at some locations it appeared to distribute as a cluster along the line of the field investigation, whereas at other locations it was distributed in a scattered or sparse manner as discussed in Section 5.3.1.2.) in the project area as the conducted seismic testing. Therefore, during the interpolation, the data did not always give nice smooth contour plots and soft transition boundaries to show the variation of the mapped parameters for site conditions of seismic zonation map, especially on Holocene Alluvial deposits. The regional seismic map of $N(30)$ data with respect to the site classes specified in the IBC 2003 is shown in Figure 5.23. In

a general sense, there appears to be consistency with respect to surface geology and the seismic testing results. In ideal case, however, for more accurate soil categorization, both the standard penetration results and the average shear wave velocities of near-surface geologic units must be estimated together that show useful correlations amongst each other in order to characterize the geological units for evaluating local site conditions. Regarding this issue, the assemblage of site characterization database, namely, standard penetration results with other site characterization indices to evaluate the effects of using shear-wave velocity correlations will be conducted in the next section for pursuing a general site characterization in regards to seismic zonation. This study might prove to be highly useful for the seismic hazard assessment of the Ankara basin.

It should be noted that during the preparation of site classification map of penetration results, similar GIS techniques were applied in the site characterization scheme as briefly mentioned in Section 5.3.1.1.3. The results were estimated based on the compiled $N(30)$ data and the accuracy of this site classification map can be suitable for assessing the local site conditions to be solely used for regional design purposes and not for individual structural design purposes.

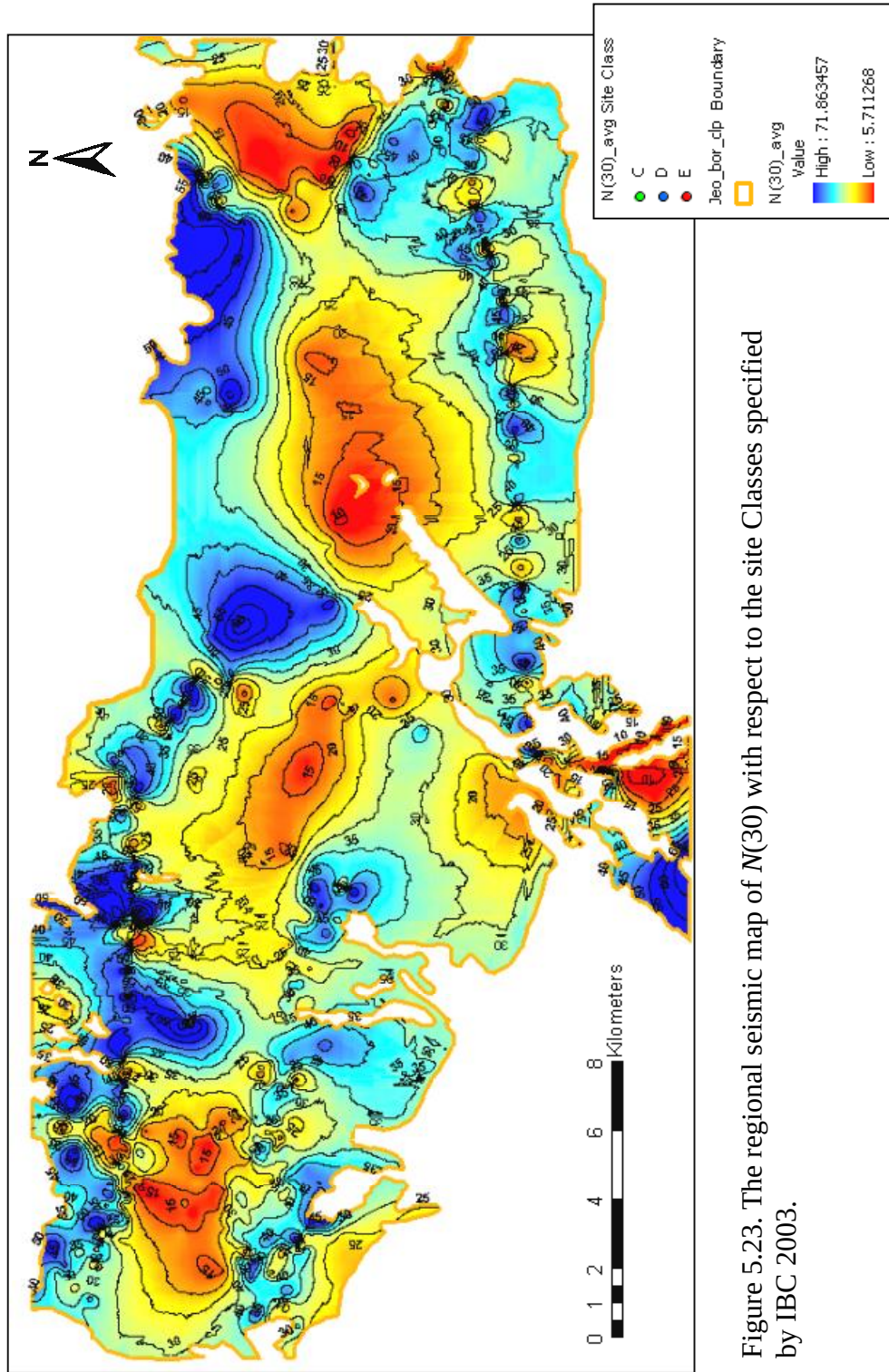


Figure 5.23. The regional seismic map of $N(30)$ with respect to the site Classes specified by IBC 2003.

5.3.1.3. Standard Penetration Test-Shear Wave Velocity Correlation for Regional Site Characterization Map of the Ankara Basin

A study was conducted to evaluate the effects of using shear-wave velocity correlations that will be used to construct a map for assessing site conditions to be used in predicting the site characteristics of the Ankara basin. Since few decades, many empirical equations have been proposed for estimating the shear wave velocity from soil indices, so as to avoid the problems of in-situ measurements and also to investigate the physical relationships between soil indices and shear wave velocity. An investigation to systematize empirical equations for the shear wave velocity of soils was applied in terms of the most common site characteristic indexes that are particularly significant parameters of penetration results (i.e., Terzaghi and Peck, 1967; Murphy, 1972; Ohta et al, 1978; Fumal and Tinsley, 1985; and Sykora, 1987) and depth. The proposal of so many empirical relations between the standard penetration result and the shear wave velocity might demonstrate that the shear wave velocity might not be described satisfactorily in terms of the standard penetration value alone. Hence, the adopted indices of geology and soil type practically available might be the other important factors to evaluate the shear-wave velocity besides the standard penetration results as well.

Actually, the success of this sensitivity study depends in a large part on the selection of the shear-wave velocity correlation and the more abundant reliable index measurements, especially standard penetration results. The first step of the study was to determine if there were any obvious trends within the data. The shear-wave velocities were selected according to depositional setting and material type as a function of depth. The second step of the study was to select a shear-wave velocity correlation that would incorporate the available data. Consequently the empirical equations including as many characteristic indexes as possible were tried to assemble a consistent and well distributed database for improving their accuracy and then, these studies were compared with the other worldwide proven studies

from various investigators documented in the published papers (i.e., Ohta and Goto, 1978, Piratheepan and Andrus, 2001 and Andrus et. al, 1994). The constructed empirical formulas for estimating the shear wave velocity measurements were used to idealize the site character for seismic zonation map. Note that the indexes used were generally based on standard penetration results, depth, textural and geological descriptions in this research study.

Regarding a recommended procedure for correlation that was outlined above, one of the acceptable methodology proposed by Ohta and Goto (1978) and modified by (Seed et al., 1985) was used to construct empirical formulas for estimating the shear wave velocity systematically in terms of the soil indexes. This method is a relatively more advanced approach compared to the others in the sense of the difficulty of processing data, particularly if some of the important indexes are non-metric. However, when all the variates are metric, a simple multivariate analysis is applicable with ease. In general, it is practically available for constructing an empirical equation of which the variates are metric and non-metric.

From the other methodologies that were proposed to be applied by Piratheepan and Andrus (2001), and Andrus et al. (2001), the compiled data pairs of V_s and soil indexes of SPT blow count have been compiled from measurements made by various studies that are all from Holocene age soil deposits with FC < 40 %. The available soil information is the soil type, fines content (FC), plasticity, coefficient of uniformity, and age corresponding to the V_s measurements. Therefore, these soil properties are considered in the development of the regression analyses depending on the uncorrected and corrected index measurements provide various plotted data pairs and exhibit a uniform scatter to confirm the success of the regression equations to provide good fits of the compiled data for Holocene soils.

On the basis of correlation measurements in soil deposits in Japan, Ohta and Goto (1976) developed the relationship. Assuming rod energy in Japanese SPT tests is 67 percent of the theoretical free-fall energy (Seed et al., 1985), and then these relationships can be presented through the following equation:

$$V_s = 93.1(N_j)^{0.249} (E)(F) \quad (5.9)$$

where V_s is in m/s, N_j = blow count measured in Japanese practice ($\approx 1.1 N_{60}$; standard penetration test delivering 60 percent of the theoretical free-fall energy to the drill rods; Seed et al., 1985), E = a factor depending on the age of the deposit, and F = a factor depending on the soil type. The best-fit values of E are 1 for Holocene soils and 1.448 for Pleistocene soils. The best-fit values of F are given in Table 5.11. Note that the correlation coefficient of the best fit equation (R^2) is 0.787 and the probable error (SDE) is 24.0%.

Table 5.11. F Factors for Various Soil Types (Seed et al., 1985 modified from Ohta and Goto, 1976).

Soil Type	Factor F
clay	1.00
fine sand	1.056
medium sand	1.013
coarse sand	1.039
sandy gravel	1.069
gravel	1.221

On the basis of correlation measurements in soil deposits, Andrus et. al (2001) also developed some relationships. Grouping the data through FC and considering arbitrary combinations of the soil properties, different regression equations were derived. Listed below are the three regression equations that were chosen from a total of 34 different regression equations that seem most useful and also most appropriate for this particular research. Note that listed below are the coefficient of determination (R^2) and standard deviation error (SDE, m/s) associated with each equation.

$$V_s = 95.5(N_{60})^{0.226} \quad \text{for FC} < 10 \%, R^2 = 0.688, \text{SDE} = 17.5 \quad (5.10)$$

$$V_s = 103.4(N_{60})^{0.205} \quad \text{for } 10\% \leq \text{FC} \leq 35\%, R^2 = 0.878, \text{SDE} = 11.7 \quad (5.11)$$

$$V_s = 101.8(N_{60})^{0.205} \quad \text{for } 0 \leq \text{FC} \leq 40\%, R^2 = 0.719, \text{SDE} = 16.7 \quad (5.12)$$

where V_s is in m/s and N_{60} = blows/0.3m measured in standard penetration test delivering 60 percent of the theoretical free-fall energy to the drill rods. It should be noted that the factor E is taken for Holocene soils to be equal to 1.0 in the above equations.

The objective of this section is to develop improved regression equations between shear wave velocity and site characterization indices, particularly penetration resistance. Besides the other correlation measurement studies, empirical formulas were also derived from this research to estimate the effects of using shear-wave velocity correlations, and then comparing these studies with the other studies mentioned above in regards to predicting the site characteristics of the Ankara basin. Therefore, 123 sets of available data of V_s and SPT blow counts conducted at or near the same locations have been studied as part of these correlation measurements. Regarding the general variability of the characteristics of the sedimentary units considering the amount of the suitable data pairs to the success of this correlation study, the data were briefly classified within two different sedimentary environments as younger Holocene alluvial sediments and older Upper Pliocene to Pleistocene Fluvial deposits.

Among the 123 obtained V_s -SPT data sets, 74 fell within the Quaternary Holocene fluvial sedimentary unit possessing site categories of E and D, whereas the other 49 of the data fell within the Upper Pliocene to Pleistocene Fluvial sedimentary unit of site categories D and C. The general characteristics and statistical results for the data pairs of this sedimentary unit are summarized in Table 5.12. It can be observed from Table 5.12 that about 8.9% (11 data points) of the

correlated $V_s(30)$ and $N(30)$ data pairs were not consistent at some points because some of the data used in this research, particularly penetration measurements were compiled from the various investigations and these particular data might have been in error. Of the 74 measured data pairs of Holocene Alluvium, 10.8% (8 data points) of them, whereas of the 49 measured data pairs of Upper Pliocene to Pleistocene Fluvial Deposits, 6.1% (3 data points) of them had inconsistencies within the general database. The details of these results and their inconsistencies in the $V_s(30)$ and $N(30)$ data pair correlation are tabulated in Table 5.13.

Considering the success of the correlation measurements that depends on the selection of the abundant and reliable $V_s(30)$ and $N(30)$ data pairs with other characteristic indices and their consistencies mentioned above, it was necessary to make a preliminary adjustment of the data before starting the regression analysis. Initially, 8.9% of the data (11 data points) having inconsistencies that are indicated above were eliminated from the regression analysis. About 9.8% of the data (12 data points) with an average $N(30)$ of greater than 50 blows/0.3m (C-Site Class) were rejected from the Upper Pliocene to Pleistocene Fluvial sedimentary unit database because of their poor accuracy. The $V_s(30)$ and $N(30)$ data pairs of E and D site categories in Quaternary Holocene fluvial units were utilized to evaluate the correlation measurements together because the geological and morphological characteristics of these sedimentary units had the same depositional characteristics at the time of setting. Hence, there were sufficient amount of reliable and consistent data pairs that might be used for the regression analysis.

Table 5.12. Summary of the general characteristics and statistical results of the $V_s(30)$ & $N(30)$ data pairs of the sedimentary units used in correlation measurements.

Geologic Unit		No. of Data	Percent. Data (%)	avg. $V_s(30)$ (m/s)			Sites do not agree for V_s (%)	No. of Data	Percent. Data (%)	avg. $N(30)$ (b/0.3m)			Sites do not agree for $N(30)$ (%)
				Mean	Std. Dev.	Range				Mean	Std. Dev.	Range	
ASSIGNED IBC2003 SITE CLASS	E-Site	35	28,5	169	13	125-180	6	31	25,2	13	1	10-14	2
	D-Site	39	31,7	217	26	181-292	2	43	35,0	22	4	16-29	6
Holocene Alluvium (Total)		74	60,2	202	34	125-303	8 (6,5)	74	60,2	18	5	10-29	8 (6,5)
ASSIGNED IBC2003 SITE CLASS	D-Site	37	30,1	301	39	208-358	2	36	29,3	40	8	25-50	1
	C-Site	12	9,8	388	22	363-433	1	13	10,6	52	2	51-55	2
L. Pliocene to Pleistocene Fluvial Deposits (Total)		49	39,8	322	52	208-433	3 (2,4)	49	39,8	43	9	25-55	3 (2,4)
All Units (Total)		123	100				11 (8,9)	123	100				11 (8,9)

Table 5.13. The details of the results and inconsistencies of $V_s(30)$ and $N(30)$ data pairs.

Testing Info	Geologic Unit	Research Info	avg V_s (30 m)	Site Class	Testing Info	Research Info	avg $N_{(30)}$ (30 m)	Site Class	Info
SIS-18	Q. Alluvium	BAP	165	E	EKA-31	Sincan Municipality	18	D	6,5 % of the entire data; 10,7 % of the Q. Alluvium
SIS-113	Q. Alluvium	BAP	166	E	EKA-44	Etimesgut Municipality	18	D	
SIS-183	Q. Alluvium	BAP	154	E	HIPOD-5	Eser Study	17	D	
SIS-13	Q. Alluvium	BAP	138	E	EKA-20	Sincan Municipality	16	D	
SIS-32	Q. Alluvium	BAP	168	E	EKA-31	Sincan Municipality	18	D	
SIS-33	Q. Alluvium	BAP	172	E	EKA-49	Sincan Municipality	16	D	
BM-8	Q. Alluvium	Ank. Metro 3. Stage	209	D	BM-8	Ank. Metro 3. Stage	14	E	2,4 % of the entire data; 6,1 % of the L. Plio. to Pleist.
SIS-132	Q. Alluvium	BAP	198	D	S-127	DLH Study	14	E	
BMD-1	L. Plio. to Pleist.	Ank. Metro 3. Stage	316	D	BMD-1	Ank. Metro 3. Stage	53	C	
BM-25	L. Plio. to Pleist.	Ank. Metro 3. Stage	320	D	BM-25	Ank. Metro 3. Stage	51	C	
SIS-38	L. Plio. to Pleist.	BAP	433	C	EKA-41	Sincan Municipality	39	D	

The $V_s(30)$ and $N(30)$ data pairs with other characteristic indices were analyzed to derive regression equations for younger Holocene alluvial and older Upper Pliocene to Pleistocene fluvial sediments which may be presented by the following relationships:

$$V_s = 59.94(N_{60})^{0.428} \text{ for younger alluvial deposits, } R^2 = 0.64, \text{ SDE} = \pm 18.9 \quad (5.13)$$

$$V_s = 37.05(N_{60})^{0.572} \text{ for older fluvial deposits, } R^2 = 0.73, \text{ SDE} = \pm 17.1 \quad (5.14)$$

where V_s is in m/sec, N_{60} = blows/0.3 meter measured in standard penetration test delivering 60 percent of the theoretical free-fall energy to the drill rods. Note that the computations presented above were performed for a depth of 30 m.

These relationships are based on the particular age of the deposits for estimating shear wave velocities from standard penetration tests. These relationships are quite useful because they have been generated based on the actual data sets representing local site conditions of our research area other than studies that were based on a different database from various investigations under similar site conditions. The correlation coefficients of the best fit equations (R^2) are 0.64 and 0.73 and probable errors (SDE) are $\pm 18.9\%$ and $\pm 17.1\%$, respectively. Regarding the derivation of the regression equations from this study, correlation measurements for V_s and SPT- N_{60} data pairs of regression results developed for Holocene alluvial and older Upper Pliocene sediments are tabulated in Figures 5.24 and 5.25, respectively.

Additionally, comparisons between measured shear wave velocities and estimated shear wave velocities using Equations (5.13) and (5.14) are also presented in Figures 5.26 and 5.27, respectively. The plotted data exhibit a uniform scatter about the best fit line, confirming the success of the regression equations in providing good fits for the conducted data for the Holocene alluvial and older Upper Pliocene to Pleistocene fluvial sediments.

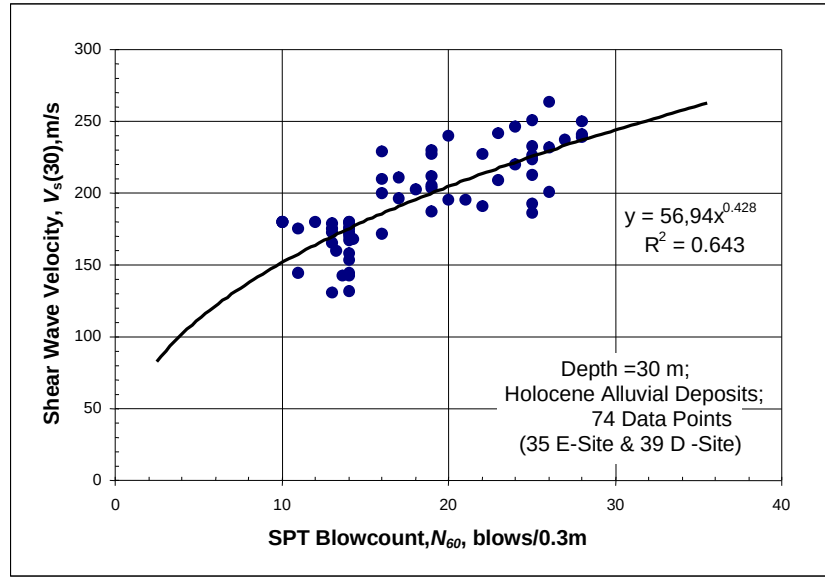


Figure 5.24. V_s and SPT- N_{60} regression equation developed for this study for Holocene alluvial sediments

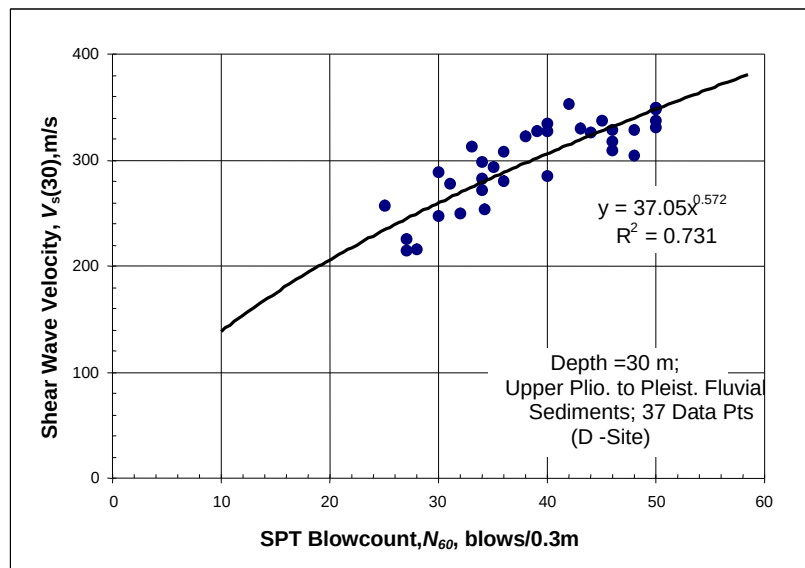
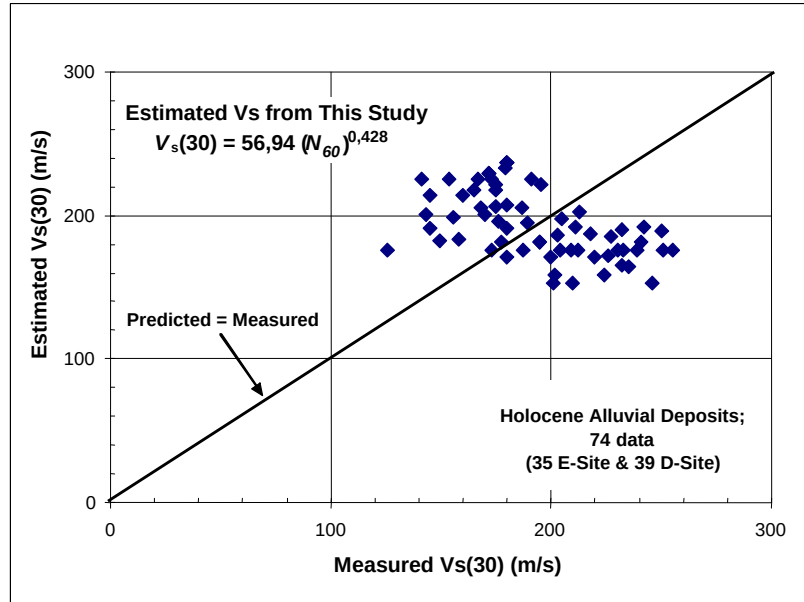
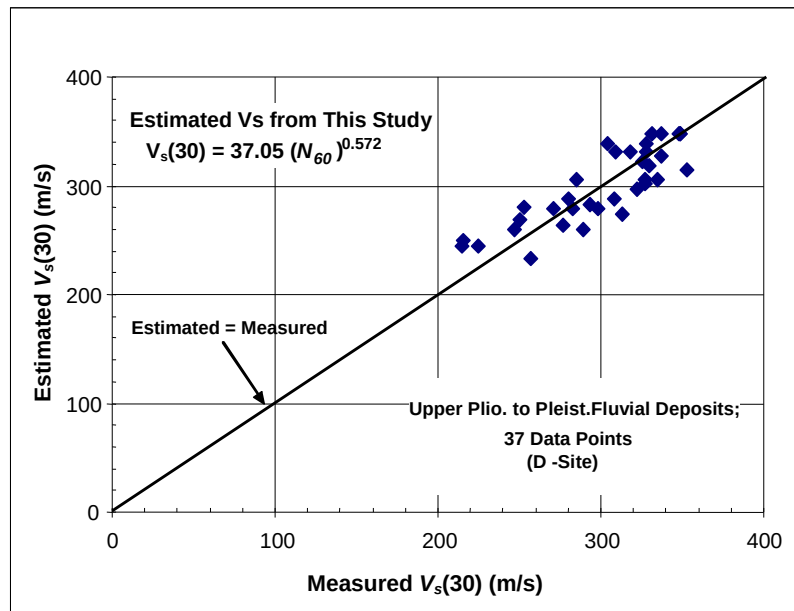


Figure 5.25. V_s and SPT- N_{60} regression equation developed for this study for older Upper Pliocene to Pleistocene fluvial sediments.



Figures 5.26. Comparison of measured and predicted $V_s(30)$ data pairs as a function of standard penetration blow count for Holocene Alluvial sediments.



Figures 5.27. Comparison of measured and predicted $V_s(30)$ data pairs as a function of standard penetration blow count for Upper Pliocene to Pleistocene fluvial sediments.

The result of the regression equations (5.13) and (5.14) developed for Holocene alluvial and older Upper Pliocene sediments are plotted in Figures 5.28 and 5.29 along with correlation of previously mentioned regression equations (through Eqns. (5.9) to (5.12)) based on different databases from various investigations.

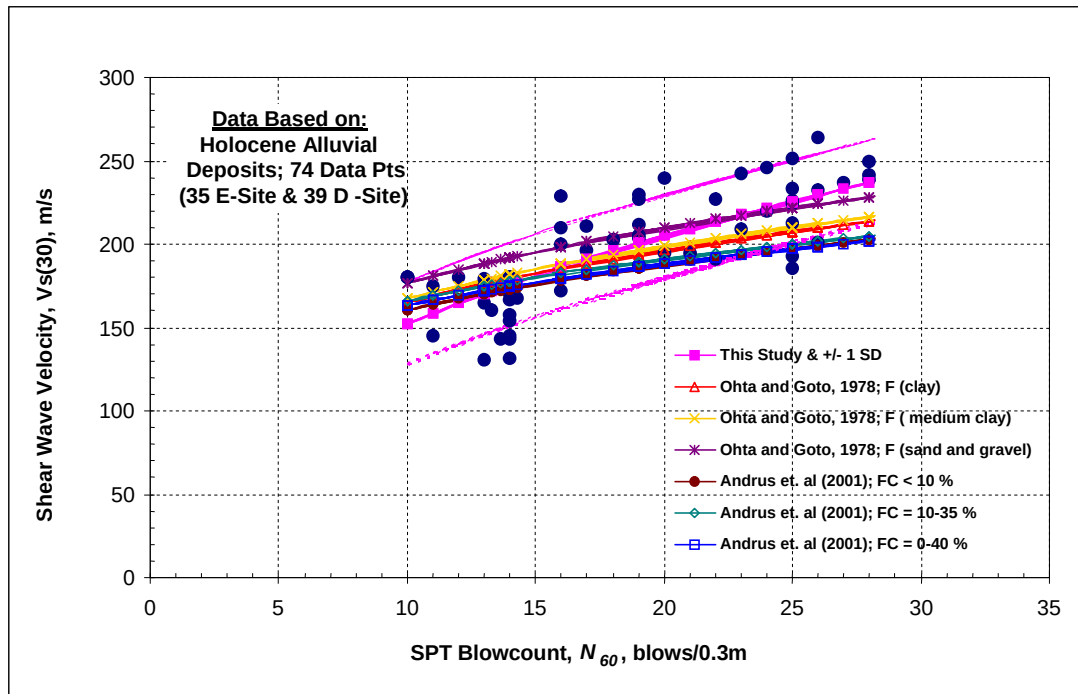


Figure 5.28. Comparison of the regression equation developed for this study with the Ohta and Goto (1978) and Andrus et. al (2001) regression equations along with the compiled data from the Holocene Alluvial sediments.

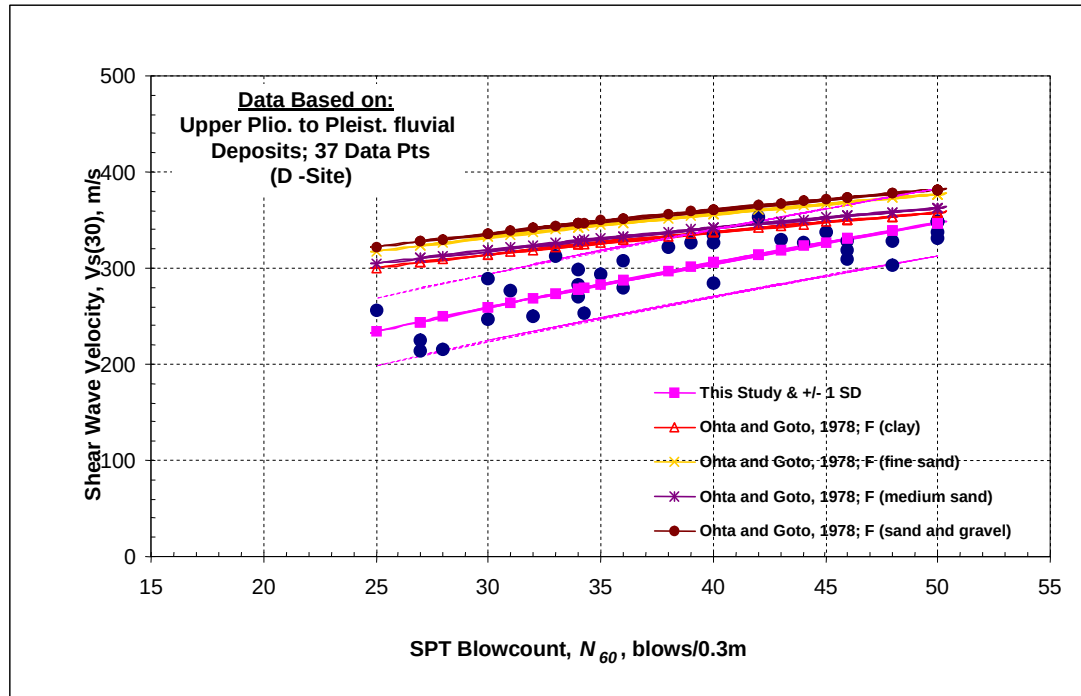


Figure 5.29. Comparison of the regression equation developed for this study with the Ohta and Goto (1978) regression equations along with the compiled data from Upper Pliocene to Pleistocene fluvial sediments.

It is observed from the plotted data of Figures 5.28 and 5.29 that the estimated results that were derived from Equations (5.13) and (5.14) of this study compare well with the measured V_s results and provides the best estimates of V_s for both the Holocene alluvial and Upper Pliocene to Pleistocene Fluvial sediments. This could be due to the generated regression results that are based on the actual data sets representing local site conditions of our research area.

It appears from the plotted data that the equations obtained from Ohta and Goto (1976) for finer sediments (through clay to medium sand) are relatively consistent but overestimate the measured V_s results for $N_{60} < 13$ blows/0.3m in Holocene Alluvial and $N_{60} < 40$ blows/0.3m in Upper Pliocene to Pleistocene Fluvial sediments. The equations obtained for coarser sediments (i.e., for medium

sand to sandy gravel) overestimate the measured V_s results for $N_{60} < 20$ blows/0.3m in Holocene Alluvial and all N_{60} results, particularly in Upper Pliocene to Pleistocene Fluvial sediments. Regarding the V_s averages from the plotted data of Ohta and Goto (1976), they were overestimated up to about 4% (for clay) and up to 8% (for medium sand) in the Holocene alluvial unit, whereas the overestimation was about 11% (for clay) and up to 16% (for medium sand) in the Upper Pliocene to Pleistocene Fluvial sediments.

It was observed from the plotted data that the equations obtained from Andrus et. al (2001) for the $0 \leq FC \leq 40$ % and $FC < 10\%$ curves (Eqns. (5.10) and (5.12)) showed similar trends with those of Ohta and Goto (1976) and overestimated the measured V_s results for $N_{60} < 15$ blows/0.3m for all equations (Eqns. (5.10) to (5.12)) in the Holocene Alluvial sediments. As mentioned before, there was not enough amount of data points to determine the regression equations for Upper Pliocene to Pleistocene Fluvial sediments. Regarding the V_s averages from the plotted data of Andrus et. al (2001), they were underestimated of up to about 4% (for $FC < 10\%$) to 7% (for $0 \leq FC \leq 40$ %) in the Holocene alluvial unit.

Considering the regression results from Ohta and Goto (1976), and Andrus et al. (2001) reported above, most of the data that were used to construct the regression results were from depths less than 10 m and most of the data pertained to Holocene sediments. The regression equations from these various studies (Figures 5.28 and 5.29) might have also been influenced by some of the important indices, particularly soil type and age of deposits in addition to the shear-wave velocity and the standard penetration results. The study of Andrus et. al (2001) that is summarized by Equations (5.10), (5.11) and (5.12) shows that the soil type may have an influence on both the regression coefficient and exponents, whereas the study from Ohta and Goto (1976) shows that the soil type directly influences the regression coefficient as well as the age of deposits. These are the primary factors to consider in pursuing a comparison.

As a result, in considering the appropriate regression equations that are relatively consistent with the depositional setting and textural characteristics of the Ankara basin and compare well with the measured V_s results, Ohta and Goto (1978) regression equations of clay and medium sand for Holocene Alluvium and Upper Pliocene to Pleistocene Fluvial sediments, respectively; Andrus et. al (2001) regression equation of FC=0-40% (Eq. 5.12) for Holocene Alluvium; and the regression equations of (5.13) and (5.14) that were derived from this study based on the compiled data were taken into consideration to estimate the shear wave velocities. Finally, these empirical formulas for estimating the shear wave velocity measurements as well as the measured shear wave velocity results obtained from this research (Section 5.3.1.1.3.) were assembled to prepare a consistent and well distributed database and to characterize the local site conditions for seismic zoning which is highly important for seismic hazard assessments of the Ankara basin.

5.3.1.3.1. Regional Shear Wave Velocity Seismic Zonation Map from Estimated and Measured V_s Studies for Seismic Hazard Assessment of the Ankara Basin

As explained under the methodologies of Sections 5.3.1.1.3 and 5.3.1.2.3, the average shear wave velocity results assembled from the estimated and measured V_s studies have been synthesized and aggregated for both Holocene alluvial and Upper Pliocene to Pleistocene Fluvial sediments to construct an average V_s map for assessing site conditions to be utilized in predicting the site character of the Ankara basin. The sedimentary units have generally been classified according to their depositional environments taking into account their physical properties such as texture, density and thickness. The physical properties of the soil deposits along with their age have been used as primary factors in mapping of these sedimentary units.

Consequently, the measured and estimated average $V_s(30)$ results for each testing point were contoured to provide a consistent and well distributed database along with a mean shear-wave velocity map of the Ankara basin. It is anticipated that this study will be quite useful in preparing regional seismic hazard maps in which V_s characteristics can be used as part of a site-condition term. The regional seismic maps of $V_s(30)$ with respect to the site classes specified in the IBC 2003 according to this study based on the actual data, according to Ohta and Goto, 1978 and according to Andrus et. al, 2001) complemented by the measured $V_s(30)$ studies conducted in Section 5.3.1.1.3 are given in Figures 5.30, 5.31 and 5.32, respectively. Considering Sections 5.3.1.1.3 and 5.3.1.2.3, it can be inferred from these results that there clearly appears to be more consistency with respect to surface geology and more accurate soil categorization with seismic surveying. Finally, the regional site classification map of the Ankara Basin in regards to the site classes as specified by the IBC 2003 were prepared based on measured and estimated average $V_s(30)$ measurements resulting from this study to characterize the local site conditions for seismic zonation studies (Figure 5.33).

It should be noted that during the preparation of site classification map with respect to penetration results, similar GIS methodologies to those mentioned in Section 5.3.1.1.3 were applied in the site characterization scheme. These results were estimated depending on the available estimated and measured average $V_s(30)$ data and the accuracy of this site classification map can be suitable for assessing the local site conditions to be solely used for regional design purposes as mentioned previously. Although, the regional map is clearly not appropriate for site-specific use, it is believed that the map is a very important tool in regional seismic hazard assessments.

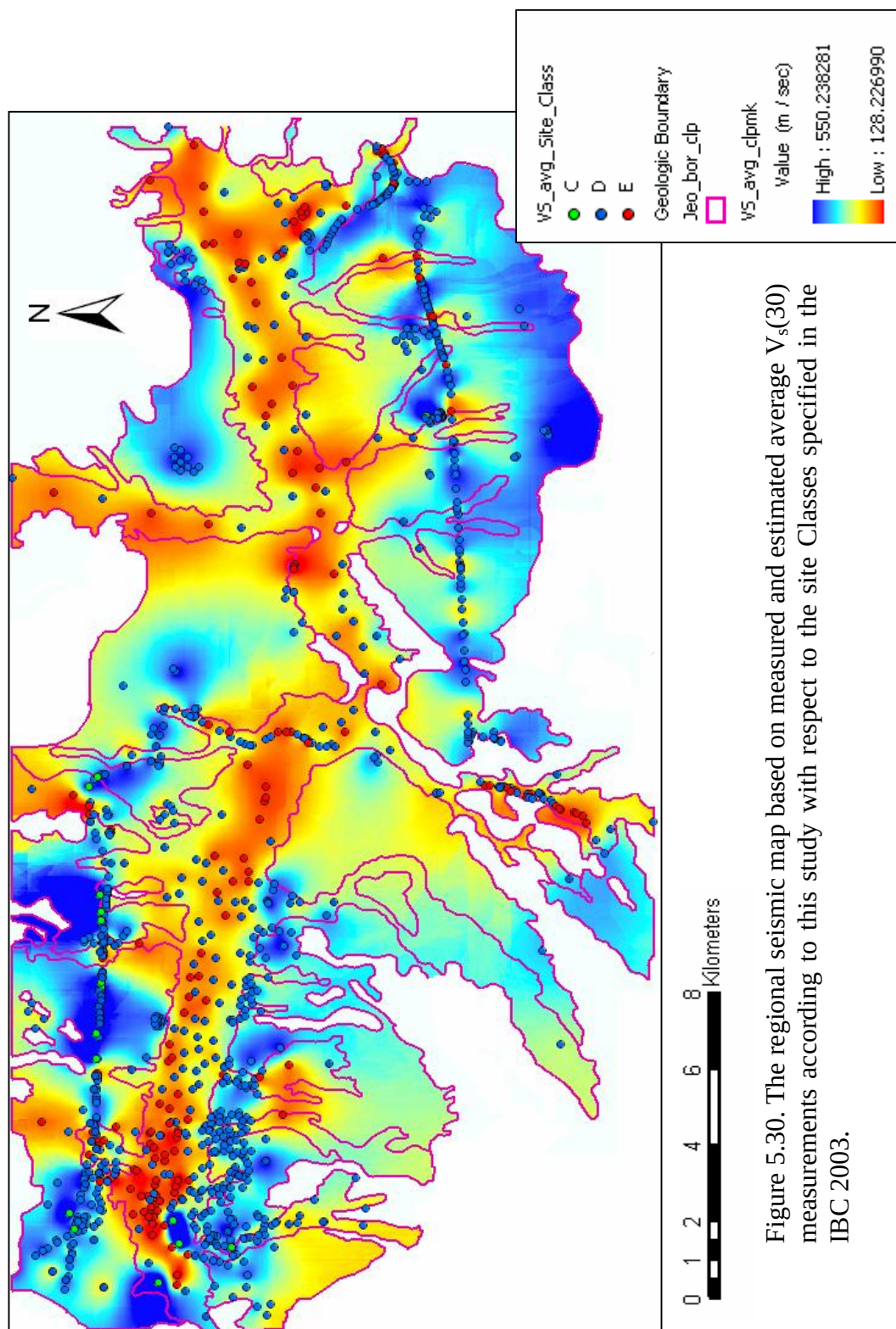


Figure 5.30. The regional seismic map based on measured and estimated average $V_s(30)$ measurements according to this study with respect to the site Classes specified in the IBC 2003.

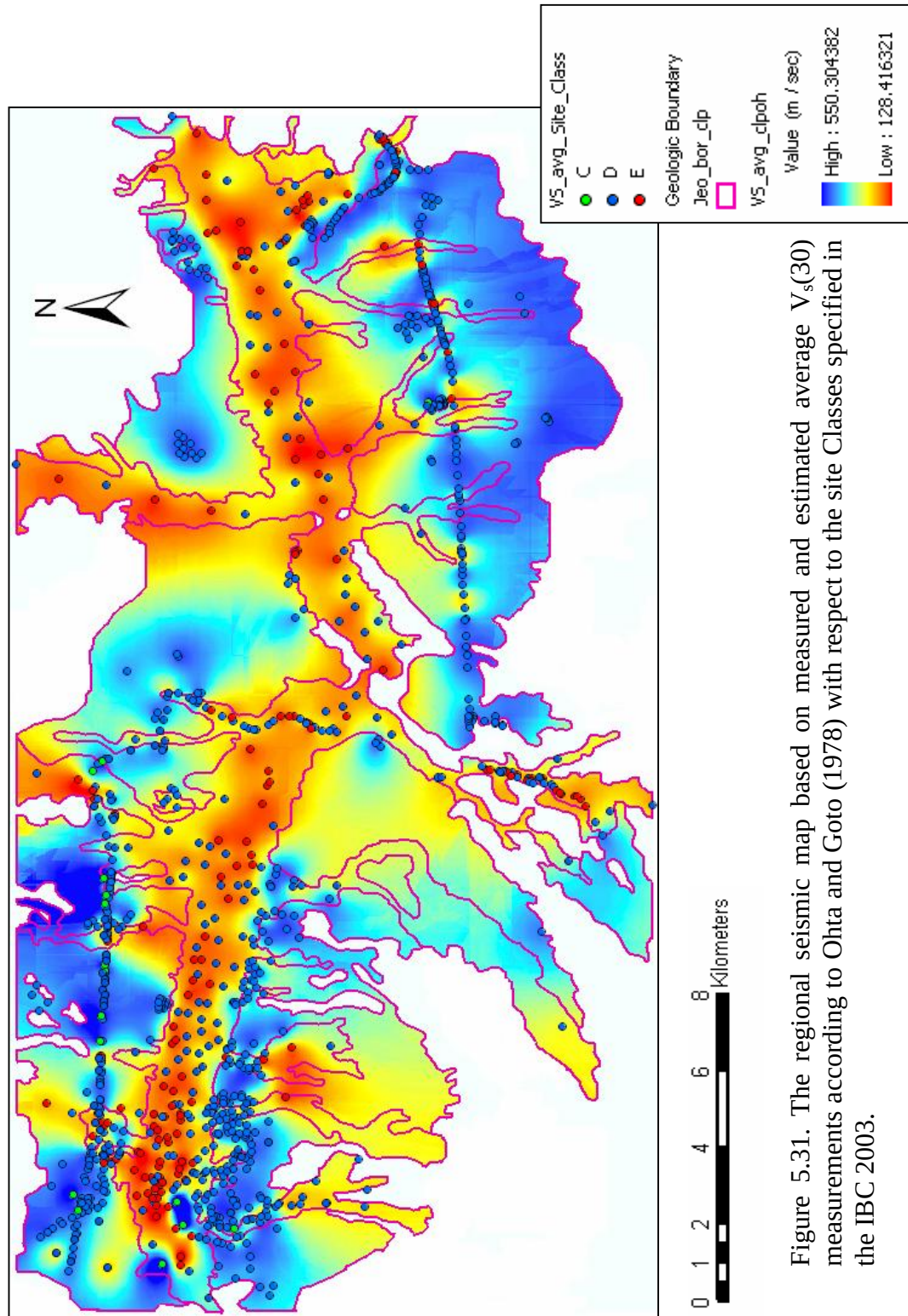


Figure 5.31. The regional seismic map based on measured and estimated average $V_s(30)$ measurements according to Ohta and Goto (1978) with respect to the site Classes specified in the IBC 2003.

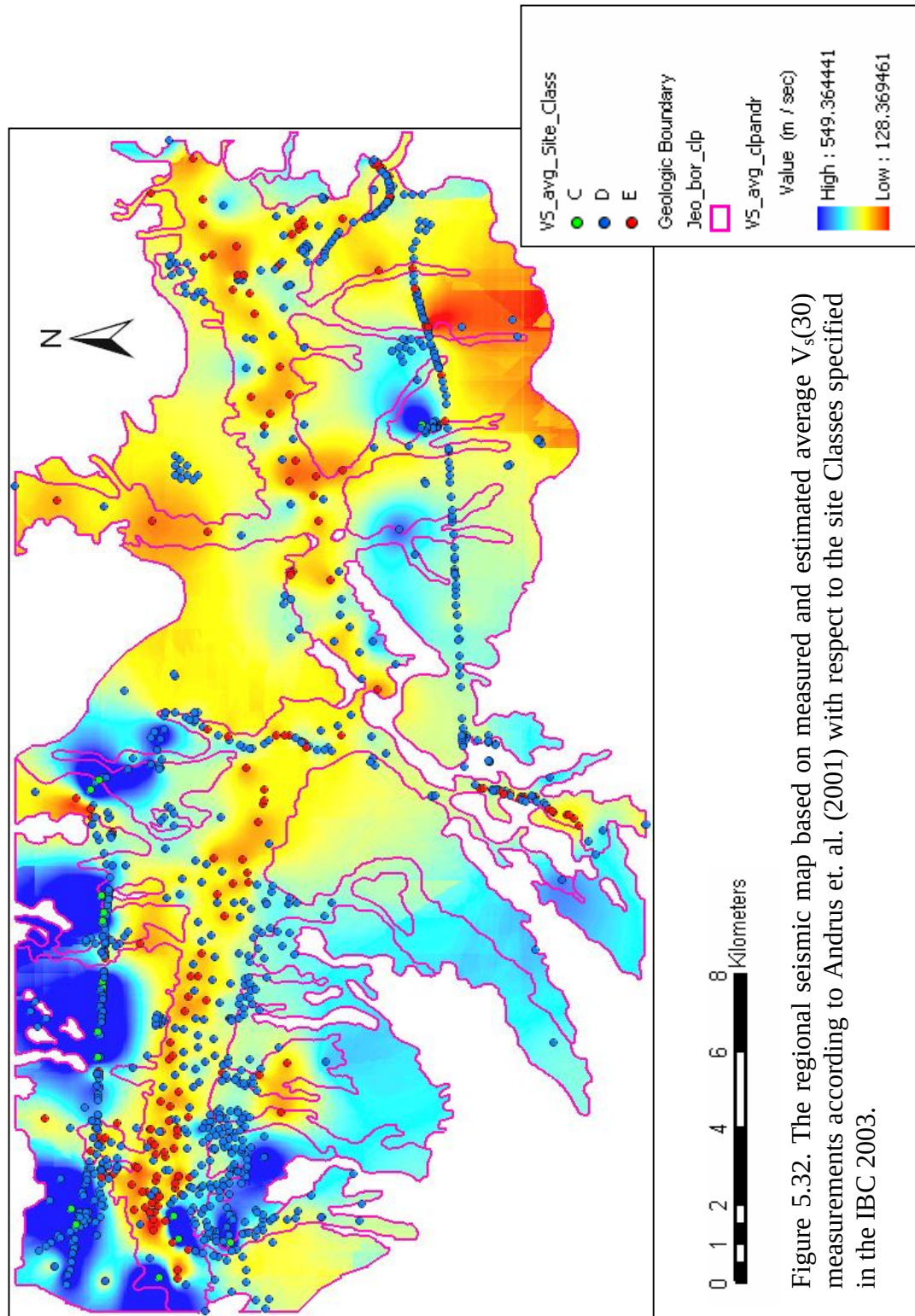


Figure 5.32. The regional seismic map based on measured and estimated average $V_s(30)$ measurements according to Andrus et. al. (2001) with respect to the site Classes specified in the IBC 2003.

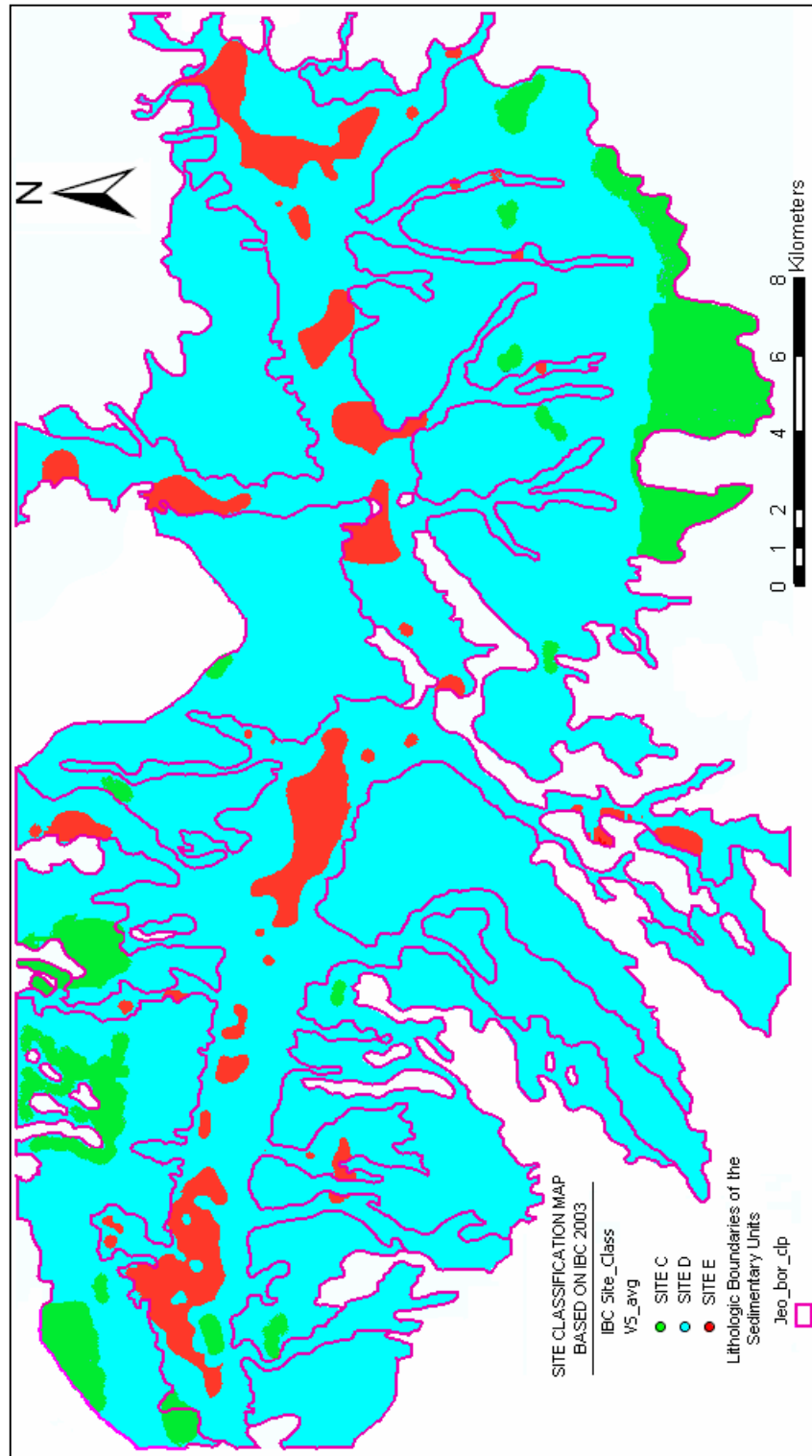


Figure 5.33. Regional site classification zonation map of the Ankara Basin in regards to the site classes as specified by IBC 2003 based on measured and estimated average $V_s(30)$ measurements performed in this study.

5.3.1.4. Other Supplementary Site Characterization methods

The methodologies that have been mentioned in this chapter are critical for conducting site characterization and thus for idealizing the local geologic conditions for seismic zonation. In addition to these methodologies for the characterization of a geologically complex region and evaluation of the variability of its units (particularly in younger deposits), other supportive site characterization studies (i.e., resistivity measurements, P-wave velocity measurements) may be utilized for a more thorough interpretation of the site conditions, especially in sedimentary deposits. By using supplementary characterization methods, site characterization of shear-wave velocity and standard penetration results for unconsolidated sedimentary deposits may be correlated with supplementary studies that can be applied to a particular area for providing assistance in better characterizing the physical properties of the geologic materials (i.e., texture, soil types), ground water conditions and thickness of the younger lithologic units.

5.3.1.4.1. Site Conditions Based on Resistivity Measurements of Sedimentary Units in the Ankara Basin

An electrical resistivity survey using vertical electrical sounding (VES) has been conducted concordant with the seismic refraction survey study (at the same or close by locations) at 113 stations, perpendicular to or parallel to the main course of the Ankara River in a spectrum of various sedimentary materials, especially in Quaternary alluvial and terrace sediments, with the main purpose of determining the depth to the water table and for identifying the depth to bedrock. The Schlumberger array vertical sounding method that is more flexible, less time consuming and suitable for being correlated with the complementary seismic and engineering geological studies was considered to be an appropriate method to be employed in

the field. The maximum electrode spacing of the Schlumberger array ($\overline{C1C2}/2$), generally long electrode separation, ranged from 100 m to 150 m depending on the suitability of the site since at times there was not enough space to extend the survey line due to the highly urbanized research areas of the project site. The survey lines were tried to be extended much longer than the estimated depth of the survey target in order to cover a longer distance not only horizontally but also vertically in the investigated area so that sufficient data could be obtained for site characterization. Data processing techniques were performed by 1-D inversion on computer programs of WinSev-6 and IPES-5t to determine the true resistivity and depth penetration values of the lithologic layers. To avoid misinterpretation of the processed geoelectrical resistivity image, geology, seismic refraction profiles and boring log data (if situated at the test location or nearby) were used to check the data at the survey site.

During the resistivity surveys, the value of the specific resistivity of each layer, in conjunction with the seismic refraction profiles, might be useful in identifying various characters of these layers, particularly younger alluvium sediments. Particularly, these methods are best suited to get a better understanding regarding the depth to the water table, the thickness of saturated zone, evaluation of porosity of the layers and determining the depth to bedrock. However, resistivity values cannot be directly interpreted in terms of soil type or lithology because a particular distribution of potential at the ground surface does not generally have a unique interpretation for all field geophysical methods. For these reasons, it is always advisable to use several complementary geophysical testing (i.e., seismic field testing) and engineering geological studies (i.e., boring logs) in an integrated exploration program rather than relying on a single exploration method.

In the alluvial units, the variability of lithology is accompanied by differences of resistivity. In some cases within the depth of interest, these units may prove to be more electrically differentiated than elastically due to their heterogeneous nature. For instance, in coarse granular soils, the groundwater

surface is generally marked by an abrupt change in water saturation and thus by a change of resistivity, whereas in fine-grained soils, there may be no such resistivity change coinciding with a piezometric surface (Zohdy, 1965).

Considering the resistivity measurement of alluvial sediments in the research area, it was often found that layers of a given lithologic composition have a more variable resistivity at shallow depth than at greater depths (especially below the water table). The shallow layers vary in their resistivity according to lithologic composition, degree of weathering, location in a wet or dry environment (e.g. near or away from a river or an irrigation canal). The layers below the water table, although they also vary in resistivity according to lithologic composition, are more consistent in their electrical properties because they are under permanent saturation. These variations in the resistivity of the alluvial layers are important in interpreting electrical soundings.

van Blaricon (1980), Telford et al. (1976), Zohdy (1974), Zohdy et. al. (1969), Imai (1972), Keller and Frischknecht (1966) present typical ranges of resistivity values for a wide spectrum of geomaterials. By interpretation of the processed data obtained from the research site as well as the previous studies and available complementary studies (i.e., geology, seismic field testing and boring logs), interpretation of the electrical soundings in terms of soil types or lithology suggests the general ranges presented in Table 5.14 for the true resistivity ranges in alluvial sediments in the project site. It should be noted that the ranges of the values shown are those commonly encountered but do not represent extreme values that were explained in detail in Chapter 4.

Table 5.14. General range of resistivity values obtained from geoelectrical soundings encountered in the research site in alluvial sediments

Material type	Resistivity, in ohm-m
clay to silty clay ¹	~2-15
silty or clayey sand	~15-25
sand	~25-50
sandy gravel to gravel	~31-75
soft to hard bedrock	~44-130

1-It should be note that the encountered clay saturated with brackish water is about <5 ohm-m and brackish to fresh water is about 5-10 ohm-m in general.

In the characterization of a geologically complex region, particularly in younger alluvial deposits, physical properties of geologic materials have been identified, which might be mapped more readily on a regional scale. For this reason, complementary studies such as the resistivity surveys might be used to assist the primary site characterization methods in order to better characterize the physical properties of especially unconsolidated deposits. Hence, resistivity measurements at individual stations distributed along the profiles led to the construction of iso-resistivity maps in order to interpret the general resistivity character of the electrical soundings as related to site evaluation.

The importance of an iso-resistivity map should be emphasized as being a primary factor that has contributed to the success of these surveys. This is because surface heterogeneity and lateral resistivity variations alone may, under certain special distributions, yield sounding curves similar to those resulting from vertical resistivity discontinuities or at least distort the shape of sounding curves. Therefore, by orienting the lines of electrical soundings in the proper direction, the effects of severe lateral variations were minimized. In this way the space assumed for the theoretical sounding curves was approached as much as possible and more meaningful, reliable, and smoother curves were obtained. Furthermore, changes in

the character of the sounding curves from the minimum-type to the maximum-type became readily explainable in terms of location on the resistivity map. However, it should be noted that the one of the disadvantages about iso-resistivity map that were also stated by Zohdy (1965 and 1974) should be emphasized. On constructing such a map, one should realize that the measured apparent resistivity is a function of the azimuth of the resistivity profile with respect to the anomalous body. Therefore, for a given locality, in spite of using the same electrode configuration, spacing, and station density, somewhat different apparent resistivity maps will be obtained with differently oriented profiles Zohdy (1974).

The complete sets of electrical profiles and their resistivity results of the lithologic layers were determined at 113 stations in the research area by electrical resistivity survey using vertical electrical sounding method. In general, engineering site investigations studies in this research were usually limited to the uppermost 30 m of material since that thickness is the typical depth of borings and seismic testing penetrations; it has become the standard depth for classifying site characteristics widespread use in practice. Towards this objective, the iso-resistivity contour maps have been prepared for the Holocene alluvium of the Ankara basin to classify the basin horizontally as a function of various depths of 3 m, 5 m, 10 m, 15 m and 30 m. Then, these contour maps that showed the general resistivity characteristics of the geologic materials for each assigned depth (Figures 5.34) and for each particular depth (Figures 5.35-5.38) were presented. It should be noted that during the preparation of the iso-resistivity contour maps, similar GIS techniques to those summarized in Section 5.3.1.1.3. for site characterization were used.

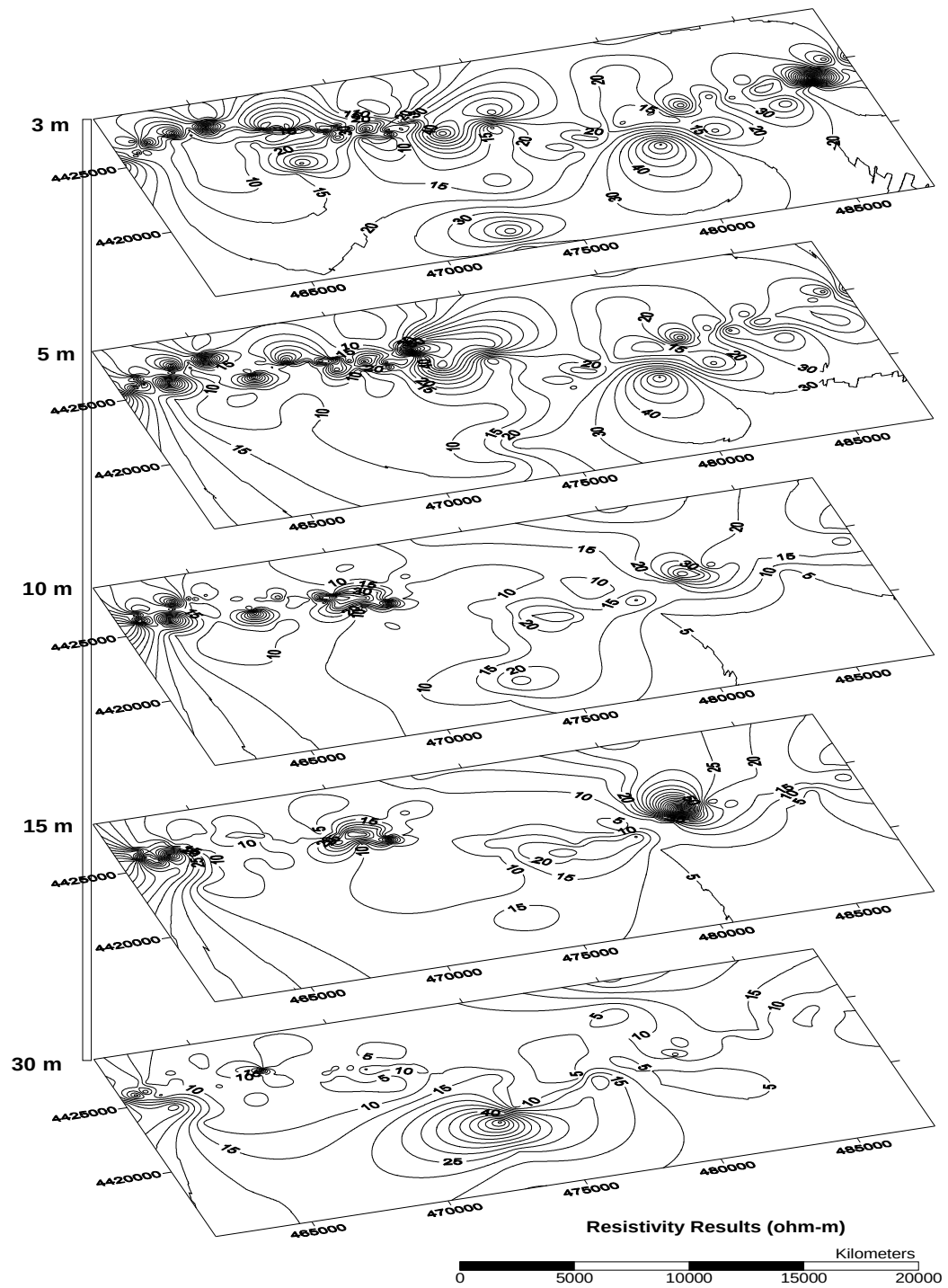


Figure 5.34. The variability of the isoresistivity contour maps at depths of 3m, 5 m, 10 m, 15 m and 30m in Holocene alluvium at the Ankara basin.

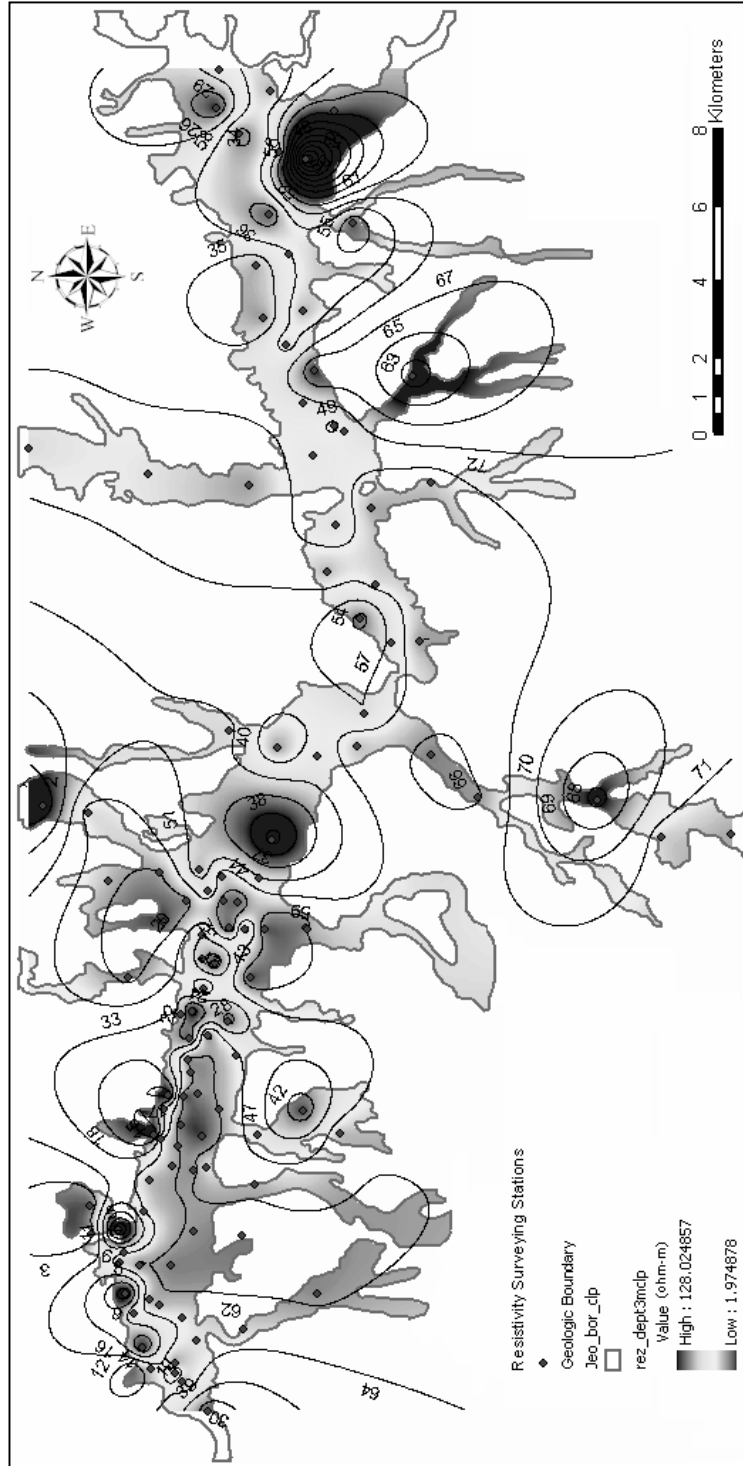


Figure 5.35. The isoresistivity contour map at a depth of 3 m in Holocene alluvium of the Ankara basin.



Figure 5.36. The isoresistivity contour map at a depth of 5 m in Holocene alluvium of the Ankara basin.

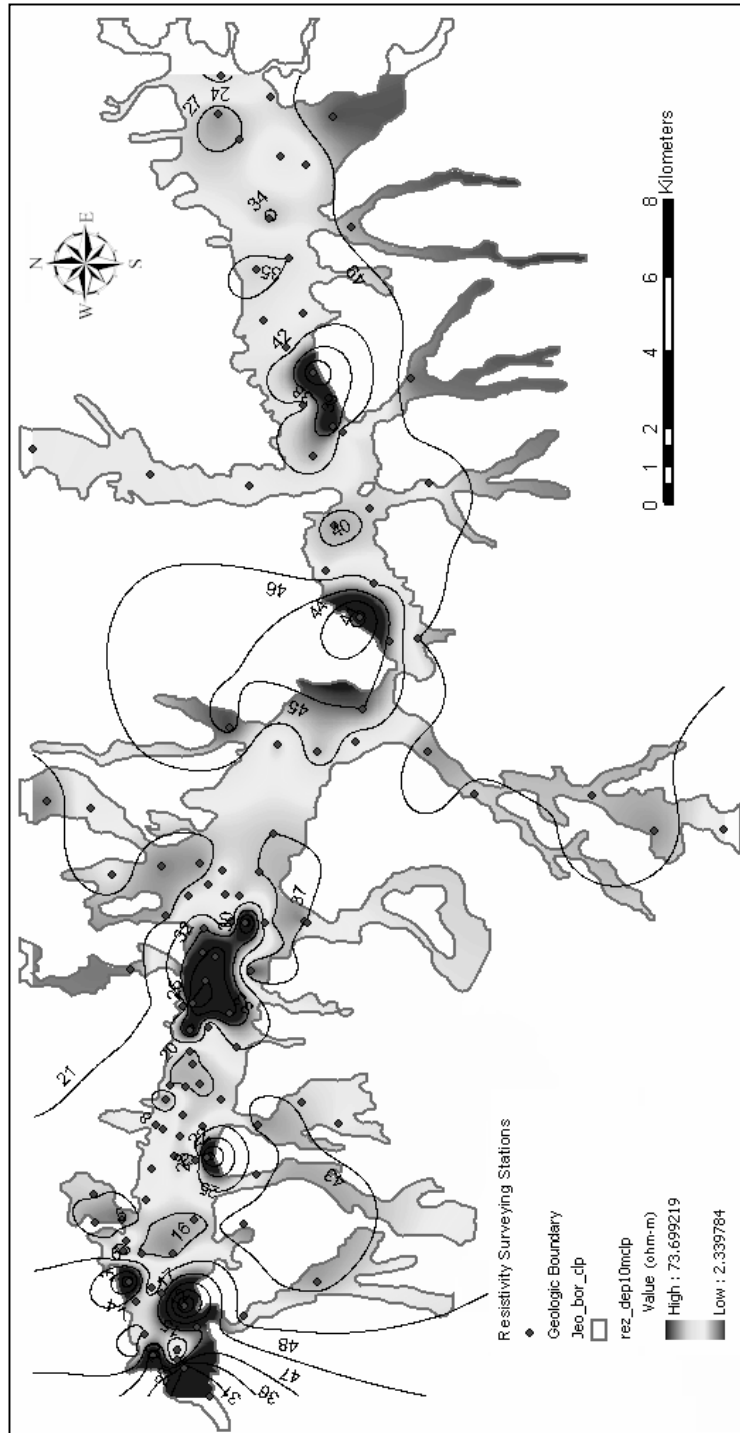


Figure 5.37. The isoresistivity contour map at a depth of 10 m in Holocene alluvium of the Ankara basin.



Figure 5.38. The isoresistivity contour map at a depth of 15 m in Holocene alluvium of the Ankara basin.

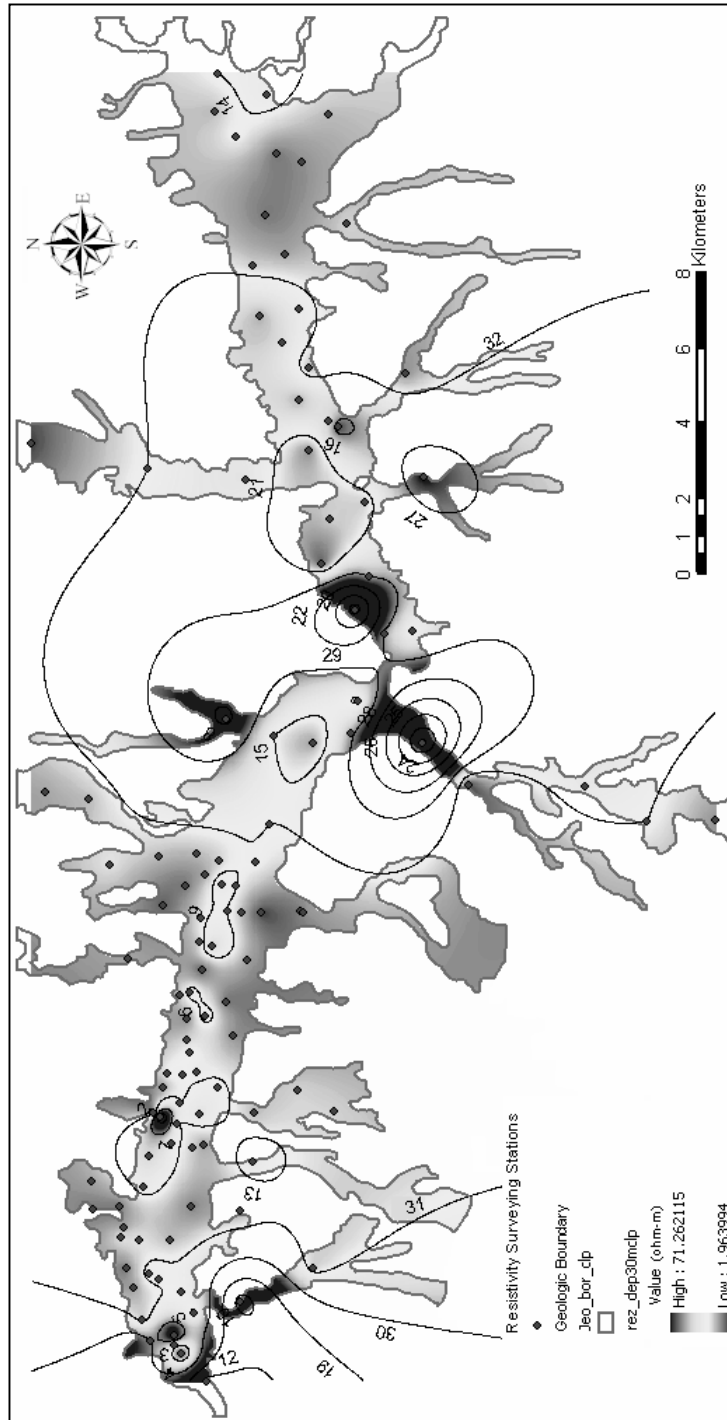


Figure 5.39. The isoresistivity contour map at a depth of 30 m in Holocene alluvium of the Ankara basin.

Briefly, it might be observed from these maps that the construction of iso-resistivity maps can well contribute to the successful performance and interpretation of electrical soundings. The variability of the iso-resistivity contour maps at all depths was relatively consistent between each depth and concordant with the general sediment characteristics of the Holocene alluvial units. Hence, these results might be correlated with other site characterization studies that can be applied to a particular area for better characterizing the physical properties of the geologic materials. These results will be correlated with the P wave velocity measurements and boring profiles later on to identify the variability of the soil textures, the ground water conditions and the thicknesses of the younger lithological units.

5.3.1.4.2. Site Conditions Based on P-Wave Velocity Measurements of Sedimentary Units in the Ankara Basin

The P-wave measurements (as well as S-wave) have been conducted simultaneously with electrical resistivity surveys (at or nearby locations) by using seismic refraction surveying at 204 locations in a spectrum of different sedimentary materials, especially on Quaternary alluvial and terrace sediments. Although, the conducted seismic refraction survey study data locations coincided with the 55 surface wave measurements that were compiled from different studies as mentioned in Section 5.3.1.1, these compiled measurements were not taken into account here during the site characterization studies because P-waves were assessed simultaneously with electrical resistivity survey to better characterize the physical properties of the geologic materials and to determine the reliability of the characterization studies of this research. In this particular research, the seismic travel times were measured at each measurement location using seismic refraction technique to provide profiles of near-surface compressional and shear wave

velocities. A histogram of depth distributions for conducted velocity profiles (P and S-waves) used in this study were previously given by Figure 5.1.

Many studies, both theoretical and experimental, have been conducted to investigate the parameters which affect compressional wave velocities in porous media such as soils and rocks. Of particular interest are those laboratory measurements of P-wave velocities which have been made in conjunction with studies of shear wave velocities in dry and water-saturated sands (Hardin and Richart, 1963), sand-clay mixtures (Rao, 1966) and sandstones (King, 1968). These studies demonstrated that compressional wave velocities varied with changes in effective stress and void ratio in a manner similar to that of shear waves. However, compressional wave velocities measured in the sedimentary deposits and bedrock materials did not show the strong correlation with other physical properties found for shear wave velocities. Most of the physical property classifications showed wide (300-1700 m/s) and overlapping velocity ranges. The most significant difference between the behavior of P and S waves lied with the effect of interstitial fluids. The shear wave generally showed only slight changes with degree of saturation primarily due to the effect on bulk density. In cohesive soils, however, degree of saturation strongly affects soil structure and pore pressure and thus the shear wave velocity (Hardin and Dreneovich, 1972).

On the other hand, interstitial fluids strongly influenced P-wave velocities in soils and rock through effects on the bulk modulus. It has been pointed out that the bulk modulus is a function of the aggregate bulk modulus of the mineral grains, the bulk modulus of the pore fluid, the bulk modulus of sediment or rock structure and the porosity (Hamilton, 1969). The bulk modulus of a gas-water mixture at low pressure is nearly equal to the bulk modulus of the gas alone. Thus, the introduction of any amount of gas into a water-saturated sediment results in a large initial reduction in P-wave velocity, followed by a leveling off at this decreased velocity level (Brandt, 1960; Rao, 1966; and Wyllie et. al, 1956). Natural solid-water-gas mixtures occur above the water table in soil and rock materials. Thus, sharp

increases in P-wave velocity occur at the water table. For sedimentary deposits, the bulk modulus of the sediment structure is much lower than that of water (Hamilton, 1969); the water table is marked by an increase in P-wave velocity to approximately 1500 m/sec. For materials above the water table, the wide variation can be attributed to differences in degree of saturation which strongly affects compressibility. The variation for saturated materials, however, is not as readily explained. As a results of many studies that have been conducted to investigate the relative degree of saturation for sediments as indicated by P-wave velocities (Hardin and Richart, 1963; Murphy, 1972; and Fumal, 1978). The sediments have been divided into three categories: $V_p < 1000$ m/sec (dry), $1150 \text{ m/sec} \leq V_p \leq 1300$ m/sec (nearly saturated), and $V_p \geq 1500$ m/sec (saturated).

In this research study, using P-wave velocities, seismically distinct units cannot be delineated with the degree of refinement possible for shear waves. P-wave velocities are useful, however, in predicting some aspects of the seismic response of geologic materials. For this reason, the P-wave velocities of Quaternary alluvial and terrace sediments that were found to have distinct velocity characteristics are presented in Table 5.15 for the sedimentary units, velocities for both saturated and unsaturated materials are listed. Even when this division is made, the sedimentary units do not show distinct P-wave velocity ranges.

Table 5.15. Distinct velocity characteristics of P-wave velocities for Quaternary alluvial and terrace sediments

Geological Unit and their Textural Character	Relative Water Content	No. of Data	Mean (m/s)	±Std. Dev. (m/s)	Range of P-Wave Velocity (m/s)
Holocene very fine grained	Unsaturated	36	627	169	430-940
	Saturated	29	1648	162	1200-1800
Holocene fine grained	Unsaturated	13	631	159	450-920
	Saturated	14	1700	135	1500-1900
Holocene silty sand to sand	Unsaturated	38	828	160	630-1100
	Saturated	38	1713	104	1600-2000
Holocene to Pleistocene sand to gravelly sand	Unsaturated	6	970	185	760-1100
	Saturated	6	1883	183	1600-2100

As a result, it might be inferred from these studies that are reported and proved with Table 5.15 that the P-wave velocity measurements are particularly useful for the determination of the relative degree of saturation for sediments along the unsaturated and saturated zone leading to locating the water table. Although there is slight disagreement concerning the velocity values that correspond to saturated media, it appears that a single relationship of 1500 m/sec equals saturation (100 %) is a common occurrence for the majority of field studies concerned with groundwater. Nevertheless, it is always advisable to use complementary methods in an integrated exploration program rather than relying on a single exploration method to avoid misinterpretation of the physical properties of the geologic materials and to control those results for increasing the reliability, particularly extensive site characterization for large scale seismic zonation studies such as this research study.

By using this concept, the interpretations of these results were assembled to be synthesized with the electrical resistivity measurements mentioned in Section 5.3.1.4.1 for identifying the variability of the soil textures and ground water conditions in younger alluvial sediments. These methods are particularly best suited for getting a better understanding regarding the depth to the water table and the thickness of the saturated zone.

Consequently, interpretation of these results that have been generalized from resistivity of VES and P- wave velocity measurements were used for construction of groundwater depth contour maps in order to plot the thickness of the saturated zone for younger alluvial sediments in the Ankara basin that can be satisfactorily used in site evaluation studies that will be discussed in the later chapters. A contour map that shows the general characteristics of the depth to groundwater for alluvial sediments prepared through GIS query is by Figure 5.40. According to this figure, the mean depth to groundwater $\pm$ one standard deviation is 5.56 ± 1.58 m with a depth ranging from 1m to 9 m. It needs to be mentioned that these results are highly

consistent with the groundwater depths reported in the hydrogeological maps of DSI (DSİ, 1975).

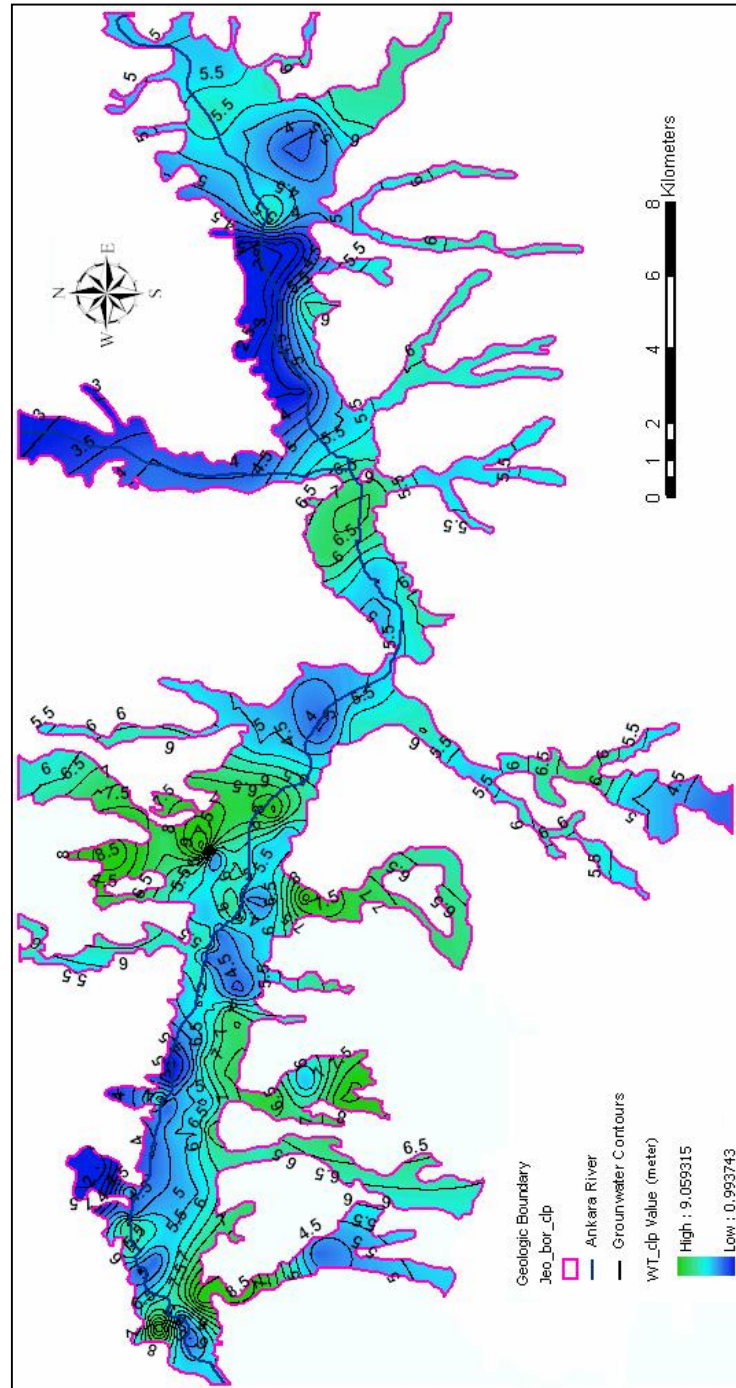


Figure 5.40. The groundwater contour map that shows the depth of groundwater table levels in alluvial sediments of the Ankara basin based on resistivity of VES and P- wave velocity measurements.

CHAPTER 6

LIQUEFACTION SUSCEPTIBILITY AND LIQUEFACTION POTENTIAL STUDIES FOR EVALUATION OF THE LIQUEFACTION HAZARD IN THE WESTERN PART OF THE ANKARA BASIN

6.1. Introduction

Liquefaction is a process by which water saturated sediment loses strength due to elevated pore water pressure. Earthquake shaking induces shear stresses in the soil that cause the saturated cohesionless granular soil particles to rearrange and excess pore water pressures to build up (Chang, et al., 1991). Historic large earthquakes throughout the world demonstrate that liquefaction-related ground failure commonly causes extensive structural and lifeline damage in urbanized areas. Delineating areas that are susceptible to liquefaction hazards is critical for evaluating and reducing the risk from liquefaction through appropriate mitigation and emergency response.

The types of failures associated with liquefaction include: sinking or overturning of structures; excessive differential settlement of structures; sand boils; and surface lateral spreading. There are several factors that influence liquefaction such as the geologic history of the deposits, the depth of the ground water table, the grain size distribution, the density of the soil, and the ground slope, often requiring professional and expert evaluation (Juang and Elton, 1991). Thus, a detailed investigation is necessary for a site-specific soil liquefaction assessment, but more simplified models are required for quantitative analysis on a regional basis.

Because liquefaction generally occurs in areas underlain by low-density, saturated granular sediments, liquefaction susceptibility can be mapped using specific, well established geologic and geotechnical criteria. In general, liquefaction-related hazards are likely in areas containing three items: (1) unconsolidated granular sediments, (2) shallow groundwater (<15 m), and (3) exposure to long period, long duration strong ground motions. Recent earthquakes (e.g., 1989 Loma Prieta, 1994 Kobe and 1999 Kocaeli) have shown that late Holocene floodplain deposits are particularly susceptible to liquefaction, and subsequent lateral spreading and loss of bearing strength (EERI). The Ankara basin surrounded by the seismically active fault systems (i.e., NAFS, SLFZ), is underlain by saturated late Holocene floodplain deposits that are particularly vulnerable to liquefaction hazards.

The primary goal of this study is to utilize the compiled and conducted data resources to evaluate seismic hazards, which, in this chapter being liquefaction susceptibility. Hence, near surface soil parameters and shallow groundwater conditions were investigated in detail followed by delineating potential liquefaction susceptible comprising Quaternary sediments in the Ankara basin. In this respect, this chapter briefly discusses general concepts of liquefaction of soils during earthquakes, and can be divided into several parts. The first part focuses on preliminary geological site evaluation and identifies the liquefaction susceptible zones based on Quaternary geological maps. In the second part of this chapter, more quantitative evaluation methods of the standard penetration and shear wave velocity simplified procedures for evaluating the liquefaction resistance of soil, and the concept of liquefaction potential index and its evaluation methods are explained. Consequently, the primary products of this liquefaction hazard evaluation studies showing the liquefaction susceptibility and liquefaction potential of the Ankara basin might be applied for engineering, emergency response, and planning purposes. The studies presented herein are useful for insurers, engineers, planners, and emergency-response personnel in order to properly assess and mitigate the potential risk from future liquefaction hazards in the study area.

6.2. Liquefaction Hazard Evaluation

As mentioned above, the investigation of a possible liquefaction hazard at a site can proceed in two general steps or stages: (1) a preliminary geologic site evaluation, and/or (2) a more quantitative geotechnical evaluation of liquefaction potential and its potential consequences. It should be noted that these are intended to provide a fairly comprehensive overview of the steps which should be taken both to (a) investigate possible liquefaction hazards, and (b) provide a basis for development of recommendations for mitigation of such hazard, if necessary.

The scope of the investigation required is dependent not only on the nature and complexity of geological site conditions, but also on the economics of the project and on the level of risk acceptable for the proposed structure or development. Naturally, a more detailed liquefaction field study is necessary for more critical structures and facilities (e.g., hospitals, schools, high-rise buildings, earth dams, power plants, air fields, critical harbor facilities, major bridge abutments, etc.) than for less critical structures and facilities (Rathje, 2003).

Evaluation of potential liquefaction hazard for a specific project or site should, at a minimum, include appropriate consideration of the entire potential hazard modes. Not all will be applicable to each project or site, but for those hazard modes of potential concern it is generally necessary to evaluate the associated potential liquefaction hazard. This is generally done in three steps or stages, as follows (Rathje, 2003):

1. *Preliminary Geological/Geotechnical Evaluation:* These preliminary studies are performed to determine: (a) whether deposits of potentially liquefiable soil types are present, (b) whether they are in a potentially liquefiable condition, and (c) whether site stratigraphy and project/site geometry would render the potential liquefaction of these soils a “hazard”. These preliminary studies are briefly mentioned here and discussed in detail in Section 6.3.1.

2. *Quantitative Evaluation of Liquefaction Potential:* These geotechnical studies are performed to provide a basis for quantitative evaluation of: (a) the resistance of the soil deposits to cyclic pore pressure generation (or “triggering” of

liquefaction), and (b) the undrained residual strength characteristics (or “post-liquefaction” strengths) of the potentially liquefiable deposits. These quantitative analyses of liquefaction potential are explained here and discussed in detail in Section 6.3.2.

3. *Hazard Evaluation and Liquefaction Hazard Map Preparation to Seismic Zonation for development of Mitigation Alternatives if Necessary:* Based on the results of Steps 1 and 2 above, an evaluation of the potential hazard is performed, liquefaction hazard maps pertaining to the project site are prepared, and suitable methods to mitigate any unacceptable hazard is developed. These analyses are presented Section 6.3.3.

Liquefaction hazard mapping is typically done using either a geologic or a geotechnical technique. The mapping of liquefaction susceptibility began in the 1970’s using the qualitative procedure (Youd 1991). The early maps were compiled from existing geologic maps and the depth to the water table was compiled from existing data or was measured. Youd and Perkins (1978) clarified geologic criteria and added to the mapping technique by introducing a constituent map showing “liquefaction susceptibility,” which were superimposed to produce a “liquefaction potential” map. Liquefaction susceptibility is a function of the capacity of the soil to resist liquefaction. The primary factors controlling susceptibility are sediment type, relative density and water table depth. From a study of liquefaction occurrences during past earthquakes and the sedimentary units affected (Youd and Hoose, 1977), Youd and Perkins (1978) developed the criteria listed in Table 6.1. Those qualitative relations form the basis for most geologic criteria used for mapping liquefaction susceptibility. To construct the susceptibility map, the matrix of mapping guides listed in Table 6.2 was derived from geologic criteria listed in Table 6.1, along with ground water depth guidance given by Youd and Perkins (1978).

Iwasaki et al. (1982) developed two sets of simplified criteria for compiling liquefaction susceptibility maps which were based on topology. This study incorporated topographic criteria (Table 6.3) that are generally compatible with the geologic criteria given in Table 6.1. Additionally Lajoie and Halley

(1975), Dupre (1990) and Ishihara and Yasuda (1991) have used these criteria to identify and map zones of liquefaction susceptibility. Recent studies (Sowers et al. 1998 and Knudsen et al., 2000) also used this model.

The criteria that were developed from Youd and Hoose (1977), Youd and Perkins (1978) and Iwasaki et al. (1982) indicated that sediments generally gain resistance to liquefaction with age, and that fluvial processes that sort and deposit granular sediments in a relatively loose state endanger high liquefaction susceptibility. Youd and Perkins (1978) also noted that depth-to-free ground water is an important factor, with susceptibility generally decreasing with increasing ground-water depth. Saturation reduces the normal effective stress acting on loose sandy-silty sediments. This condition, particularly in the upper 10 m of the ground surface, increases the likelihood of liquefaction and resulting ground failure (Youd, 1973). Those geologic factors directly or indirectly influence geotechnical properties that control the liquefaction susceptibility of natural sediment; such properties include relative density, grain-size distribution, degree of saturation and degree of cementation (Youd, 1991). Because of this general association between geologic factors and geotechnical properties, geologic criteria are generally reliable, but not unique, predictions of liquefaction susceptibility.

Table 6.1 Estimated susceptibility of sedimentary deposits to liquefaction during strong seismic shaking (After Youd and Perkins, 1978).

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by age of deposit)			
		<500 yr	Holocene	Pleistocene	Pre-Pleistocene
(1)	(2)	(3)	(4)	(5)	(6)
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave energy	Widespread	Moderate	Low	Very low	Very low
Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

Table 6.2. Probable liquefaction susceptibility of unconsolidated, granular, non-gravelly layers as criteria used to compile a liquefaction map (modified from Dupre, 1990 and Tinsley and Fumal, 1985).

Sedimentary Unit	Depth to ground water, in meters			
	0-3	3-10	10-15	>15
Holocene				
Latest -----	Very high to high ¹	Moderated ²	Low	Very low
Earlier -----	High	Moderate	Low	Very low
Pleistocene				
Late -----	Low	Low	Very low	Very low
Middle and early - - - -	Very low	Very low	Very low	Very low

1-Areas are mapped as having very high susceptibility if fluvial channel and levee deposits are known to be present; sediment deposited in other sedimentary environments is considered to have high susceptibility.

2-Fluvial deposits having high susceptibility occur rarely and are not widely distributed; other sediments are moderately susceptible to liquefaction

Table 6.3. A microzonation procedure based upon topographical information (Modified from Iwasaki et al, 1982).

Rank	Topography	Liquefaction potential
A	Present river bed , old river bed, swamp, reclaimed land, interdune lowland	Liquefaction likely
B	Fan, natural levee, sand dune, Hood plain, beach, other plains	Liquefaction possible
C	Terrace, hill, mountain	Liquefaction not likely

Because of the lack of uniqueness in the relationship of geologic criteria to geotechnical properties, susceptibility maps based on geologic criteria generally do not provide definitive information for site specific evaluations. Specific subsurface investigations are required for such assessments. Application of these criteria, however, allow relatively inexpensive compilation of susceptibility maps which are useful for preliminary evaluations, general land-use planning, and delineation of special study zones where site-specific studies may be required before major development is approved. Geologic criteria are also commonly used

to delineate bounds of susceptibility zones evaluated from other criteria, such as geotechnical analysis. For example, most susceptibility zones derived from geotechnical analysis have used geologic criteria to delineate the zone boundaries. Conversely, most susceptibility maps based on geologic criteria have used geotechnical criteria to verify or quantify the classification system.

Regarding the quantitative geotechnical evaluation of liquefaction potential, several techniques for analyzing soil liquefaction have been proposed during the past 30 years, resulting from the need for a more quantitative estimate of regional liquefaction hazard. Evaluation of the liquefaction resistance of soils is an important step in many geotechnical investigations in earthquake prone regions. The procedure widely used for evaluating soil liquefaction resistance is termed the “simplified procedure.” This simplified procedure was originally developed by Seed and Idriss (1971) using blow counts from the standard penetration test (SPT) correlated with a parameter called the cyclic stress ratio that represents the cyclic loading on the soil. Since 1971, this procedure has been revised and updated (Seed 1979; Seed and Idriss 1982; Seed et al. 1983, 1985; Youd et al. 1997). In the mid-1980s, a parallel procedure based on the cone penetration test (CPT) was introduced by Robertson and Campanella (1985), which also has been revised and updated (Seed and de Alba 1986; Stark and Olson 1995; Olsen 1997; Robertson and Wride 1998).

A promising alternative, or supplement, to the penetration based approaches is provided by in-situ measurements of small-strain shear-wave velocity. The use of V_s as an index of liquefaction resistance is soundly based because both V_s and liquefaction resistance are similarly influenced by many of the same factors (e.g., void ratio, state of stress, stress history, and geologic age). Some advantages of using V_s are that (1) the measurements are possible in soils that are hard to sample, such as gravelly soils where penetration tests may be unreliable; (2) measurements can also be performed on small laboratory specimens, allowing direct comparisons between laboratory and field behavior; (3) V_s is a basic mechanical property of soil materials, directly related to small-strain shear modulus $G_{\max}$ [$G_{\max} = \rho(V_s)^2$, where: ρ = mass density of the soil]; (4)

$G_{\max}$ or V_s , is normally a required property in earthquake site response and soil-structure interaction analyses (Dobry et al. 1981; Seed et al. 1983; Stokoe et al. 1988a; Tokimatsu and Uchida 1990). Over the past 20 years, numerous studies have been conducted to investigate the relationship between V_s and liquefaction resistance. These studies involved field performance observations [e.g., Stokoe and Nazarian (1985); Robertson et al. (1992); Kayen et al. (1992); Andrus and Stokoe (1997); Andrus and Stokoe (2000); Andrus et al. (2001)], penetration- V_s correlations [e.g., Seed et al. (1983)], analytical investigations [e.g., Bierschwale and Stokoe (1984); Stokoe et al. (1988b)], and laboratory tests [e.g., Dobry et al. (1981); de Alba et al. (1984); Tokimatsu and Uchida (1990)].

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore water pressure and reduced effective stress (Martin et al., 1975; Casttro and Poulos, 1977). Increased pore water pressure is induced by the tendency of granular materials to compact when subjected to cyclic shear deformations. The change of state occurs most readily in loose to moderately dense granular soils, such as silty sands and sands and gravels capped by or containing seams of impermeable sediment. As liquefaction occurs soil stratum softens, allowing large cyclic deformations to occur. In loose materials, softening may also be accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or a sloping ground. In moderately dense to dense materials, liquefaction leads to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations. A condition of cyclic mobility or cyclic liquefaction may develop following the liquefaction of moderately dense materials. Beneath gently sloping to flat ground, liquefaction may lead to ground oscillation or lateral spread as a consequence of either flow deformation or cyclic mobility. Loose soils also compact during liquefaction and reconsolidation, leading to ground settlement. Sand boils may also erupt as excess pore water pressure dissipates (Youd et al., 2001).

The initial step in engineering assessment of the potential for “triggering” or initiation of soil liquefaction is the determination of whether or not soils of “potentially liquefiable nature” are present at a site that was stated previously. This, in turn, raises the important question regarding which types of soils are potentially vulnerable to soil liquefaction (Seed et al., 2001).

It has been recognized that relatively “clean” sandy soils, with few fines, are potentially vulnerable to seismically induced liquefaction. There has, however, been significant controversy and confusion regarding the liquefaction potential of silty soils (and silty/clayey soils), and also of coarser, gravelly soils. The NCEER Working Group has published many of their consensus findings in NCEER (1997) and Youd et al. (2001), and it was agreed that there was a need to reexamine the “Modified Chinese Criteria” (Finn et al., 1994) for defining the types of fine “cohesive” soils potentially vulnerable to liquefaction (Seed et al., 2001).

According to these criteria as shown in Figure 6.1 soils may be considered susceptible to significant strength loss:

- (1) Fraction finer than 0.005 mm $\leq$ 15 %,
- (2) Liquid Limit, LL $\leq$ 35%,
- (3) Natural water content $\geq$ 0.9 LL
- (4) Liquidity index $\leq$ 0.75

Both experimental research and review of liquefaction field case histories show that for soils with sufficient “fines” (particles finer than 0.074 mm) to separate the coarser (larger than 0.074 mm) particles, the characteristics of the fines control the potential for cyclically induced liquefaction. This separation of the coarser particles typically occurs as the fines content exceeds about 12% to 30%, with the precise fines content required being dependent principally on the overall soil gradation and the character of the fines. In soils wherein the fines content is sufficient as to separate the coarser particles and control behavior, cyclically-induced soil liquefaction appears to occur primarily in soils where these fines are either non-plastic or are low plasticity silts and/or silty clays ($PI \leq 10$ to

12%). In fact, low plasticity or non-plastic silts and silty sands can be among the most dangerous of liquefiable soils, as they not only can cyclically liquefy; they also “hold their water” well and dissipate excess pore pressures slowly due to their low permeabilities (Seed et al., 2001).

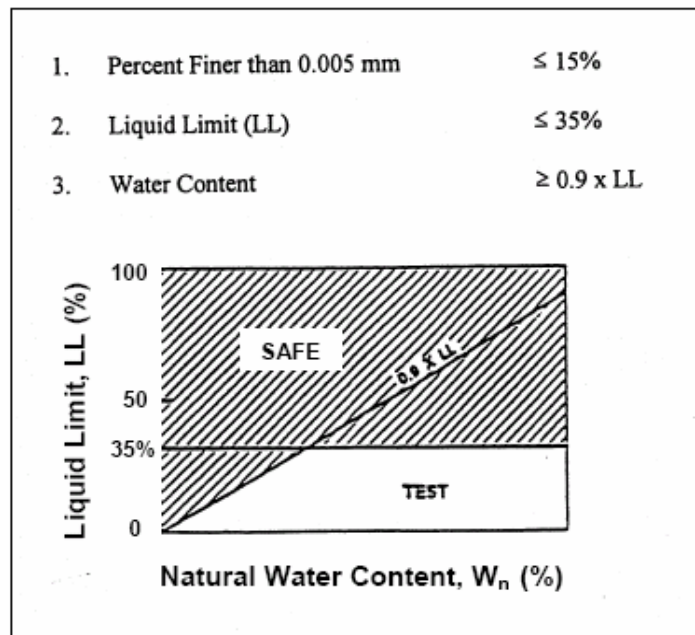


Figure 6.1. Modified Chinese Criteria (after Finn et al., 1994)

The use of in-situ index testing is the dominant approach in common engineering practice for quantitative assessment of liquefaction potential. Calculation or estimation of two variables is required for evaluation of liquefaction resistance of soils (Youd et al., 2001).

The original simplified procedure was developed and published by Seed and Idriss (1971). The evaluation procedure requires the calculation of two parameters: (1) The level of cyclic loading on the soil caused by the earthquake, expressed as a cyclic stress ratio (CSR); (2) resistance of the soil to liquefaction, expressed as a cyclic resistance ratio (CRR). These two values can then be compared as a way to predict if the soil is likely to liquefy in a given earthquake.

Seed and Idriss hypothesized that the earthquake-induced stress could be predicted using the cyclic stress ratio (CSR_{eq})

$$CSR_{eq} = (\tau_{av} / \sigma'_{vo}) = 0.65 (a_{max} / g) (\sigma_{vo} / \sigma'_{vo}) r_d \quad (6.1)$$

where τ_{av} = average shear stress in the soil profile; a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake; g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively; and r_d = stress reduction coefficient which accounts for the flexibility of the soil profile.

For routine practice and non-critical projects, the following equations may be used to estimate average values of r_d (Liao and Whitman, 1986 as quoted by Youd et al., 2001):

$$r_d = 1.0 - 0.00765z \text{ for } z \leq 9.15 \text{ m} \quad (6.2a)$$

$$r_d = 1.1742 - 0.0267z \text{ for } 9.15 \text{ m} < z \leq 23 \text{ m} \quad (6.2b)$$

where z = depth below ground surface in meters. Some investigators have suggested additional equations for estimating r_d at greater depths (Robertson and Wride, 1998), but evaluation of liquefaction at these greater depths is beyond the depths where the simplified procedure is verified and where routine applications should be applied. Mean values of r_d calculated from Eq. (6.2) are plotted in Figure 6.2, along with the mean and range of values proposed by Seed and Idriss (1971). Note that other r_d values have been proposed but are not recommended by the NCEER workshop participants. Youd and Idriss (2001) do not recommend using simplified procedures to calculate a CSR below a depth of 23 meters due to a lack of case histories to verify the procedure at this depth.

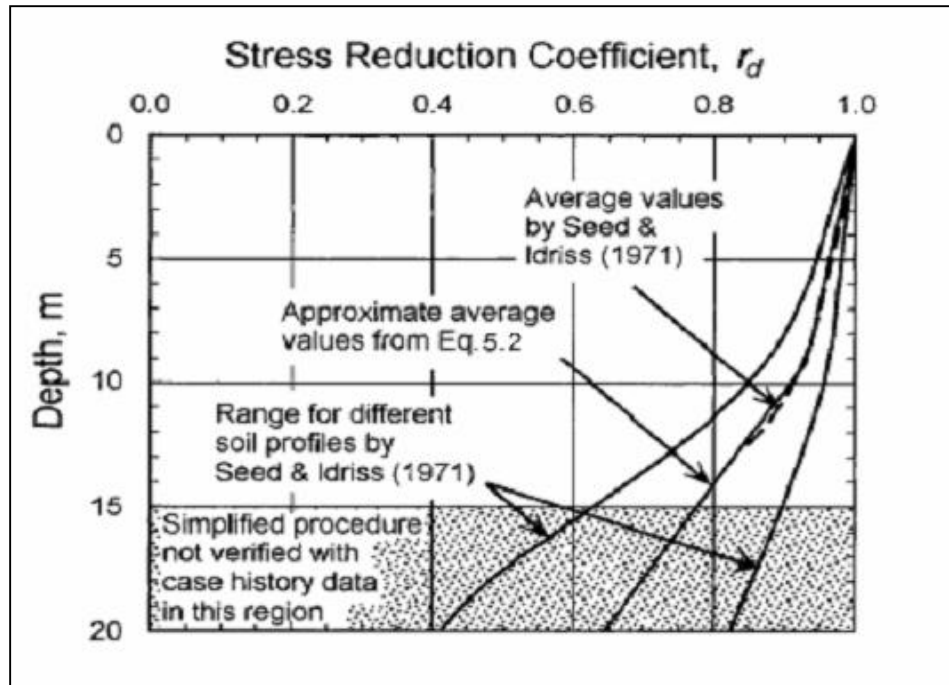


Figure 6.2. r_d versus depth curves developed by Seed and Idriss (1971) with added mean-value lines plotted from Eq. (6.2)

In the evaluation procedures, the other important concern was on procedures for evaluating liquefaction resistance (CRR) as well as cyclic stress ratio (CSR). Several field tests have gained common usage for evaluation of liquefaction resistance. The simplified procedure provides for the evaluation of soil liquefaction resistance by using an empirical graph. This graph was developed primarily by compiling blow count data $(N_1)_{60}$ obtained from the standard penetration test at sites that did and did not liquefy in previous earthquakes with magnitudes of approximately 7.5. Those criteria are largely embodied in the CSR versus $(N_1)_{60}$ plot reproduced in Fig. 6.3. $(N_1)_{60}$ is the SPT blow count normalized to an overburden pressure that was extensively discussed in Chapter 4 in Section 4.3.1.2. CRR curves were then drawn on this graph that visually bounded and separated the liquefaction case histories from the nonliquefaction case histories. Curves were developed for granular soils with fine contents of 5% or less, 15%, and 35% as shown on the plot. The CRR curve for fines contents $\leq 5\%$ is the basic

penetration criterion for the simplified procedure and is referred to hereafter as the “SPT clean sand base curve.” The CRR curves in Figure 6.3 are valid only for magnitude 7.5 earthquakes. From this graph it is possible to predict the CSR that will induce liquefaction in a soil deposit by entering the graph with a given SPT blow count (Youd and Idriss, 1997). Knowing the earthquake induced CSR and the natural CRR of the soil, one can easily predict the factor of safety against soil liquefaction.

Over the years the equation developed by Seed and Idriss (1971) for estimating earthquake-induced CSR's (Eq. 6.1) has remained essentially unchanged. However, the empirical chart used to predict soils CRR through SPT blow counts has been updated and modified as new data have become available (Youd and Idriss, 2001). In addition, other test methods have been developed with similar empirical charts for estimating CRR's. The three most common liquefaction index tests used presently, in addition to the SPT, are the cone penetration test (CPT) and shear wave velocity measurements (V_s). Note that considering the collected and conducted data that were used in this study, only the simplified standard penetration and shear wave velocity procedure, and their scaling factors to adjust CRR curves will be discussed further in this chapter.

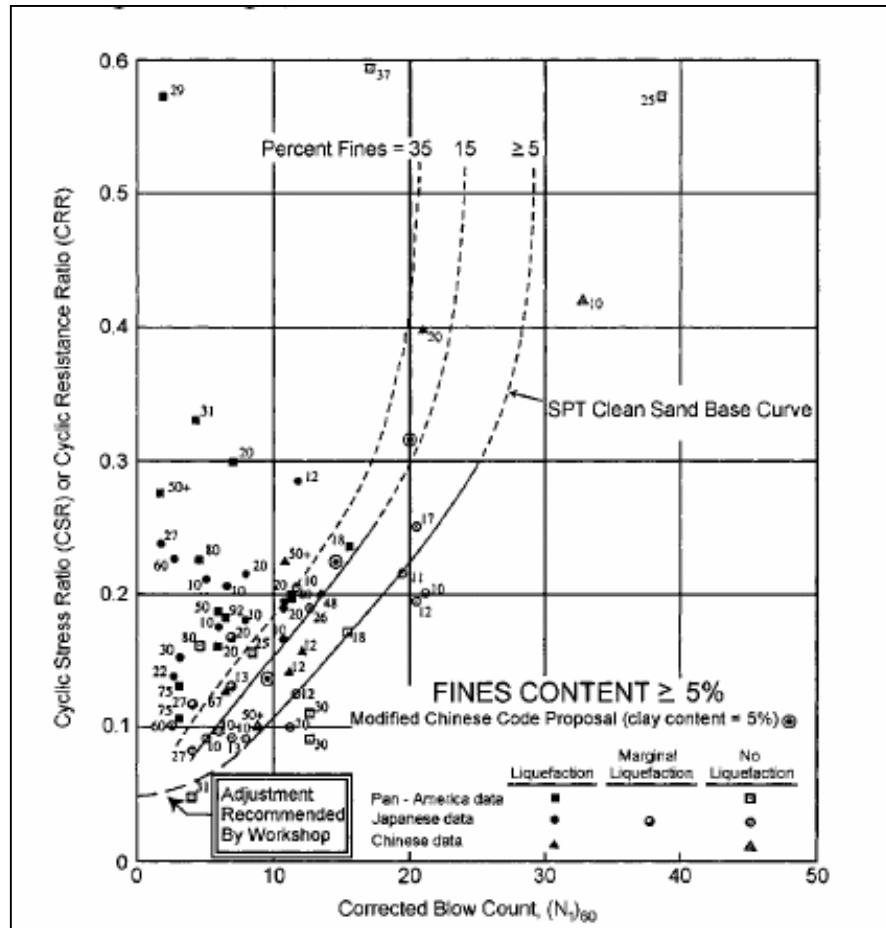


Figure 6.3. SPT clean-sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories (modified from Seed et al., 1985 as quoted by Youd and Idriss, 2001).

Regarding the consistency between the procedures, several modifications to the SPT criteria were recommended (Youd et al., 2001). The first change was to curve the trajectory of the clean-sand base curve at low $(N_1)_{60}$ to a projected intercept of about 0.05 (Figure 6.3). This adjustment reshapes the clean-sand base curve to achieve greater consistency with CRR curves developed for the CPT and shear-wave velocity procedures.

Based on the empirical data available, Seed et al. (1985) developed CRR curves for various fines contents reproduced in Figure 6.3. By the aid of the

procedure stated above, a revised correction for the fines content was developed to better fit the empirical database and to better support computations with spreadsheets and other electronic computational aids (Youd et al., 2001). The following equations were developed for correction of $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$:

$$(N)_{60cs} = \alpha + \beta(N_1)_{60} \quad (6.3)$$

where, α and β are coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for } FC \leq 5\% \quad (6.4a)$$

$$\alpha = \exp[1.76 - (190/FC^2)] \quad \text{for } 5\% < FC < 35\% \quad (6.4b)$$

$$\alpha = 5.0 \quad \text{for } FC \geq 35\% \quad (6.4c)$$

$$\beta = 1.0 \quad \text{for } FC \leq 5\% \quad (6.5a)$$

$$\beta = [0.99 + (FC^{1.5}/1.0)] \quad \text{for } 5\% < FC < 35\% \quad (6.5b)$$

$$\beta = 1.2 \quad \text{for } FC \geq 35\% \quad (6.5c)$$

Regarding the V_s -based simplified procedure of evaluating liquefaction resistance, Andrus and Stokoe (1997) suggested the simplified procedure for evaluating liquefaction resistance using shear wave velocity measurements, and this procedure was updated by Andrus et al. (1999). This procedure requires the calculation of three parameters in order to evaluate liquefaction potential: (1) the level of cyclic loading caused by the earthquake expressed as a CSR, (2) the stiffness of the soil expressed as an overburden corrected shear wave velocity, and (3) the resistance of the soil to liquefaction expressed as a CRR. This section addresses the calculation of each of these parameters (Andrus and Stokoe, 2000).

Andrus and Stokoe (2000) employed the original CSR developed by Seed and Idriss (1971) which is expressed as Eq. (6.1). This CSR is mostly used for the simplified method for liquefaction resistance evaluation using SPT, CPT, and V_s measurements.

Regarding the overburden stress-corrected shear wave velocity, V_s is influenced by the void ratio and the effective confining stress. So the shear wave velocity will increase with depth if the soil has a constant void ratio. Because of this, V_s should be normalized by effective overburden stress to make V_s to be a standard index independent on effective confining stress. This is the same idea of correcting SPT blow counts and CPT tip resistance. The corrected V_s can be obtained by the following relationship (Robertson et al. 1992):

$$V_{s1} = V_s \times C_v = V_s \left(\frac{P_a}{s_{vo}'} \right)^{0.25} \quad (6.6)$$

where: V_{s1} = overburden stress-corrected shear wave velocity,

V_s = measured shear wave velocity,

C_v = factor to correct measured shear wave velocity for overburden pressure,

P_a = reference stress of 100 kPa or about atmospheric pressure, and

σ_{vo}' = initial effective overburden stress with the same units of that of P_a .

Note that Andrus et al. (2001) suggest limiting C_v to a maximum value of 1.4 at shallow depths.

Several researchers have proposed liquefaction criteria in forms of CRR- V_{s1} curves shown in Figure 6.4. Among these curves, only the curve developed by Andrus et al. (1999) was recommended for engineering practice by NCEER workshops (Youd et al., 2001). This is because the curve by Andrus et al. (1999) was developed using the largest case history data, while other curves were determined based on no or limited case histories. The proposed relationship between CRR and V_{s1} is the following:

$$CRR_{eq} = \left\{ 0.022 \left(\frac{V_{s1}}{100} \right) + 2.8 \left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*} \right) \right\} MSF \quad (6.7)$$

where: CRR_{eq} = cyclic resistance ratio, MSF = magnitude scaling factor to account for the effect of earthquake magnitude, V_{s1} = normalized shear wave velocity, and V_{s1}^* = limiting upper value of V_{s1} for cyclic liquefaction occurrence.

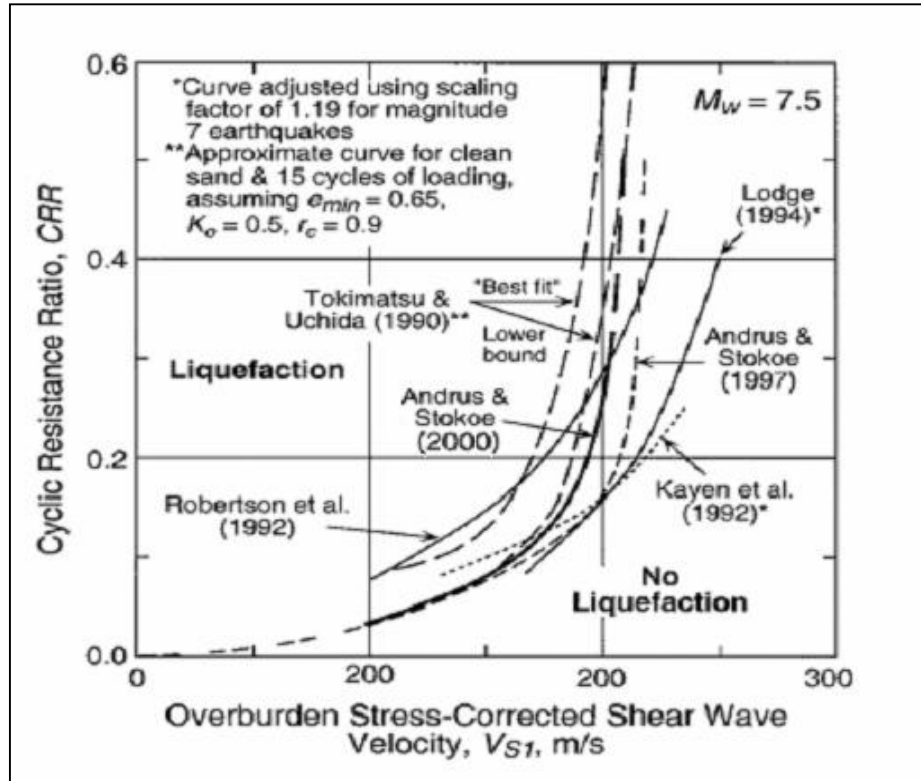


Figure 6.4. Comparison of seven relationships CRR - V_{s1} curves for clean granular soils proposed by various researchers (from Andrus and Stokoe, 2000).

The curves in Figure 6.4 are only appropriate for uncemented Holocene-aged soils with less than 5% fines, shaken by magnitude 7.5 earthquakes. However, Andrus et al. (1999) have developed an equation that can be used to apply their curve to soils with various fines contents shaken by various magnitude earthquakes with various soil ages and cementation conditions. Finally, the three

recommended CRR - V_{s1} curves by Andrus and Stokoe (2000) for various fines contents using Eq. (6.7) are shown in Figure 6.5, and this curve was produced for a 7.5 magnitude earthquake and uncemented Holocene-aged soils. The dashed line beyond 0.35 of CRR indicates that case history data were limited in this range.

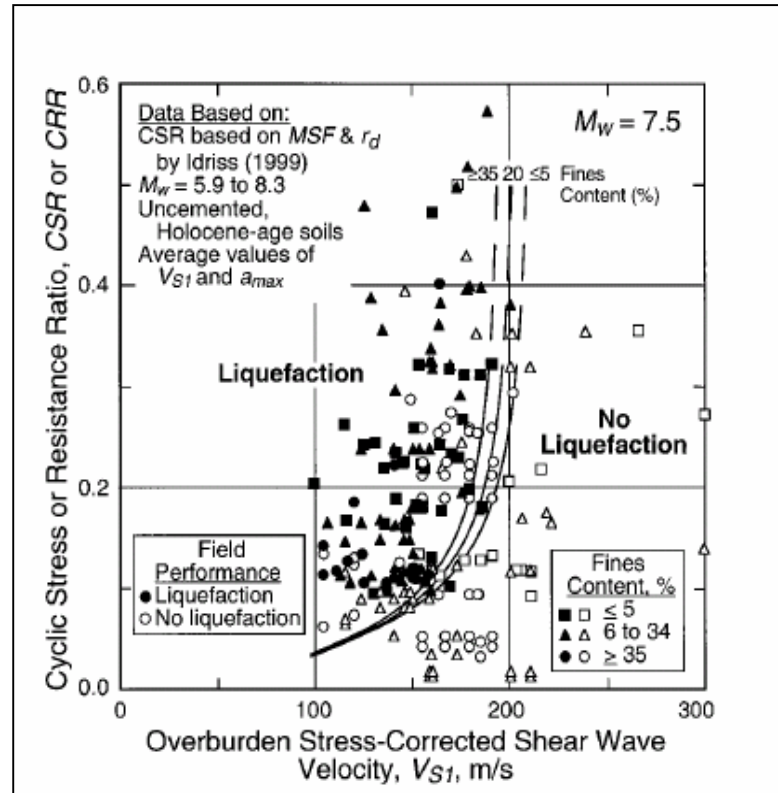


Figure 6.5. CRR - V_{s1} curves recommended in sands and gravels along with case history data based on revised values of MSF and r_d proposed by Idriss (1999) as quoted by Andrus and Stokoe (2000).

The assumption of a limiting upper value of V_{s1} is similar to the assumption of a limiting upper value of blow count or tip resistance, for the SPT and CPT tests, respectively. Upper limits for V_{s1} and penetration resistance can be explained partially by the tendency of dense soils to dilate at large strains. This dilative behavior will cause negative pore pressures, which in turn may help to

increase the effective stress. So it is expected that at some point the corrected shear wave velocity of a soil will be too great to experience liquefaction regardless of the nature of the earthquake-induced cyclic stress ratio. The maximum velocity at which a soil will liquefy seems to decrease with increasing fines content. Andrus et al. (2001) define the maximum liquefiable velocity (V_{sl}^*) as:

$$V_{sl}^* = 215 \text{ m/s} \quad \text{for sands with } FC < 5\%, \quad (6.8a)$$

$$V_{sl}^* = 215 - 0.5(FC - 5) \text{ m/s} \quad \text{for sands with } 5\% < FC < 35\% \quad (6.8b)$$

$$V_{sl}^* = 200 \text{ m/s} \quad \text{for sands and silts with } FC > 35\%, \quad (6.8c)$$

where: V_{sl}^* = limiting upper value of V_{sl} for cyclic liquefaction occurrence in meters per second, and FC = average fines content in percent by mass.

Additionally, there are some additional correction factors to determine the cyclic resistance ratio of a soil deposit through use of standard penetration and shear wave velocity measurements. These are primarily the magnitude scaling factor (MSF), high overburden stresses (high normal stresses, K_σ) and static shear stresses (non-level ground, K_α).

To adjust the clean-sand curves to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors termed “magnitude scaling factors (MSF).” MSF was needed to extend the proposed relationship between CRR and V_{sl} or $(N_1)_{60}$ for use in other ranges of earthquake magnitude besides magnitude of 7.5. Hence, a MSF of 1.0 will be assigned to an earthquake of magnitude 7.5. The 1996 NCEER workshop recommended MSF values with a range among the proposed MSFs from various researchers (Youd et al. 2001).

For < 7.5 earthquakes, the following equation proposed by Andrus and Stokoe (1997) can be used:

$$\text{MSF} = \left(\frac{M_w}{7.5} \right)^{-2.56} \quad (6.9)$$

where: MSF = magnitude scaling factor, and M_w = moment magnitude of earthquake.

For > 7.5 earthquakes, the values defined by the following equation, suggested by Idriss, should be used for engineering practice.

$$\text{MSF} = \left(\frac{10^{2.24}}{M_w^{2.56}} \right) \quad (6.10)$$

Correction factors K_σ and K_α were developed by Seed (1983) to extrapolate the simplified procedure to larger overburden pressure and static shear stress conditions than those embodied in the case history data set from which the simplified procedure was derived. As noted previously, the simplified procedure was developed and validated only for level to gently sloping sites (low static shear stress) and depths less than about 15 m (low overburden pressures). Thus applications using K_σ and particularly K_α are beyond routine practice and require specialized expertise (Youd et al, 2001). The NCEER workshop participants considered the work of previous investigators (Seed, 1983, Seed and Harder, 1990) and recommended the following values for f along the work of Hynes and Olsen (1999) who compiled and analyzed an enlarged data set to provide guidance and formulate equations for selecting K_σ values (Figure 6.6). For relative densities between 40 and 60%, $f = 0.7$ – 0.8 ; for relative densities between 60 and 80%, $f = 0.6$ – 0.7 . Consequently, these factors are applied to include MSF, K_σ and K_α as follows:

$$\text{CRR}_{7.5} = (\text{CRR}_{M=7.5}) (\text{MSF}) (K_\sigma) (K_\alpha) \quad (6.11)$$

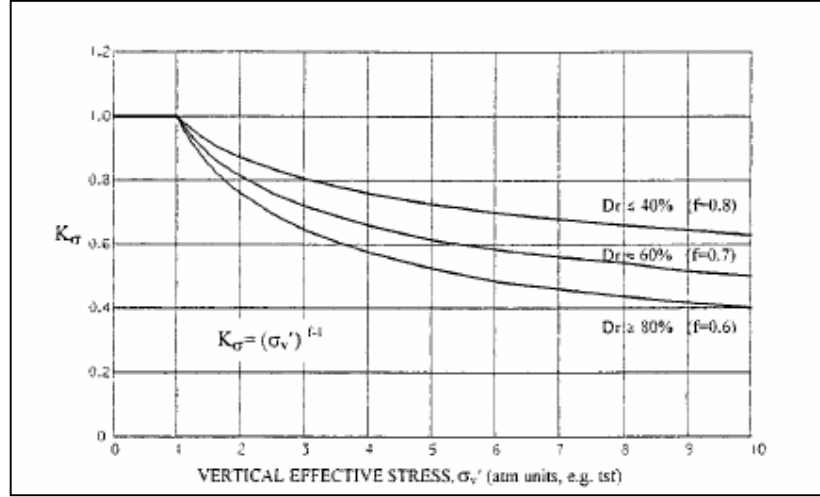


Figure 6.6. Recommended curves for estimating K_σ in engineering practice

As a summary, by using the simplified standard penetration and shear wave velocity procedures discussed above, one can readily evaluate the liquefaction potential of a soil deposit. The clean-sand base or CRR curves in $(N_1)_{60}$ and V_{s1} apply only to magnitude 7.5 earthquakes. Magnitude weighting factors may be applied to correct CSR_{eq} for other magnitudes. Either correcting CRR via magnitude scaling factors, or correcting CSR via magnitude weighting factors, leads to the same final result. Because the original papers by Seed and Idriss (1971) were written in terms of magnitude scaling factors, the use of magnitude scaling factors is continued (Youd, 2001). To illustrate the influence of magnitude scaling factors on calculated hazard, the equation for factor of safety (FS) against liquefaction is written in terms of CRR and CSR, as follows:

$$FS_{liq} = (CRR_{7.5} / CSR_{eq}) \quad (6.11)$$

where CSR_{eq} = calculated cyclic stress ratio generated by the earthquake shaking and $CRR_{7.5}$ = cyclic resistance ratio for magnitude 7.5 earthquakes. Note that $CRR_{7.5}$ is determined from Figure 6.3 for SPT data or Figure 6.5 for V_{s1} data.

6.3. Evaluation of the Liquefaction Susceptibility and Liquefaction Potential of the Research Site

The concept of a liquefaction hazard map merits a brief discussion, because the liquefaction susceptibility maps presented in this section have to be considered together with liquefaction potential maps since in the research area where the geotechnical and seismic data are insufficient in volume and are not equally distributed to provide reliable regional liquefaction potential maps for seismic hazard mapping in Quaternary deposits. As originally proposed by Youd and Perkins (1978), a liquefaction potential map is derived by superimposing a liquefaction susceptibility map. A liquefaction hazard map thus would express the concept that the probability of liquefaction-induced ground failure in certain areas may be greater than that in other areas, either owing to differences in the physical properties of near-surface earth materials (liquefaction susceptibility) or to quantify the liquefaction hazards by measuring the severity of liquefaction at the ground surface (liquefaction potential). The liquefaction susceptibility mapping is based on the qualitative rankings. These rankings typically attempt to characterize the likelihood of liquefaction rather than the potential for damage from the liquefaction.

Liquefaction potential maps delineate areas where a relative potential for liquefaction and associated ground failure may exist. Maps drawn at regional scales, however, are not sufficient for evaluating the actual liquefaction potential at a specified site. Although, site-specific geotechnical studies are required as well to evaluate the liquefaction potential, considering the available data in late Quaternary basin sediments, the liquefaction susceptibility maps in this research, therefore, might also be used to serve evaluating liquefaction potential maps.

6.3.1. Mapping Liquefaction Susceptibility

The geological and hydrological factors that affect liquefaction susceptibility are (1) the age and type of sedimentary deposit, (2) the looseness of cohesionless sediment, and (3) the depth to perched or other ground water (Tinsley et al., 1985). Physical properties of soil such as sediment grain size distribution, compaction, cementation, saturation, and depth govern the degree of resistance to liquefaction. Some of these properties can be correlated to the geological age and environment of deposition of a particular deposit. With increasing age, relative density may increase through cementation of the particles or compaction caused by the weight of the overlying sediment. Grain-size characteristics of a soil also influence susceptibility to liquefaction. Sand is more susceptible than silt or gravel, although silt of low plasticity is treated as liquefiable in this investigation. Cohesive soils generally are not considered susceptible to liquefaction. Such soils may be vulnerable to strength loss with remolding and represent a hazard that is not addressed in this investigation. Moreover, saturation is required for liquefaction, and the liquefaction susceptibility of a soil varies with the depth to ground water. Very shallow ground water increases the susceptibility to liquefaction.

Liquefaction susceptibility map of areas containing soils susceptible to liquefaction begins with evaluation of geologic maps and historical occurrences, cross-sections, geomorphology, and ground-water hydrology. Soil properties and soil conditions such as type, age, texture, and consistency, along with depths to ground water are used to identify, characterize, and correlate susceptible soils. Because Quaternary geologic mapping is based on similar soil observations, liquefaction susceptibility maps typically are similar to Quaternary geologic maps.

Preliminary characterization of liquefaction hazard within the western part of the Ankara basin required preparation of maps that delineate areas underlain by liquefiable sediments. The liquefaction hazard maps presented in this study incorporated evaluation of geological data for late Quaternary deposits that exhibit the appropriate age, textural, and groundwater conditions suitable for liquefaction-

related failure. In general, our approach to mapping liquefaction susceptibility involved four steps that are summarized on the decision flow chart in Figure 6.6. These steps are based on: (1) modification and then digitization of the existing geological and geomorphological maps of Ankara as stated previously in order to construct a 1:25,000-scale late Quaternary geologic map for the study area; (2) construction of the geotechnical characterization map showing the depth to groundwater; (3) assessment of liquefaction susceptibility of near surface deposits using conducted and compiled site characterization data; and (4) synthesis of these data to create a liquefaction susceptibility map (Figure 6.7). The subsurface deposit properties such as soil type, fines content and groundwater condition is characterized through particularly conducted (i.e., resistivity measurements, P-wave velocity results) site characterization data. Finally, the resultant multi-layer approach to mapping the susceptibility categories is included in the GIS map database to facilitate the use and distribution of the susceptibility maps. These maps might be used to improve the assessment of liquefaction hazards in the western part of the Ankara basin and allow communities to mitigate the effects of liquefaction on the urbanized environment.

6.3.1.1. Integration Methodology

The preliminary evaluation regarding the regional seismic hazard integration is intended to be general enough so as to allow the application of more or less sophisticated models depending on the availability of information in the desired analysis region. Every analysis region is different; therefore the quantification of the site effects and the weighting scheme for combining the various seismic hazards is heuristic, based on judgment and expert opinion about the influence of local site conditions in the region and the accuracy of the available geologic and information (King, 1994). Therefore, the resulting grade for each alternative is a function of the weighting scheme adopted for example, from Ertas and Jones (1993) and the basic selection criteria is assigned weighting

categories, noting that the sum of the weighting factors are equal to 1.0 for a probabilistic approach in regards to an assessment of “decision making under risk” (Dieter, 2000).

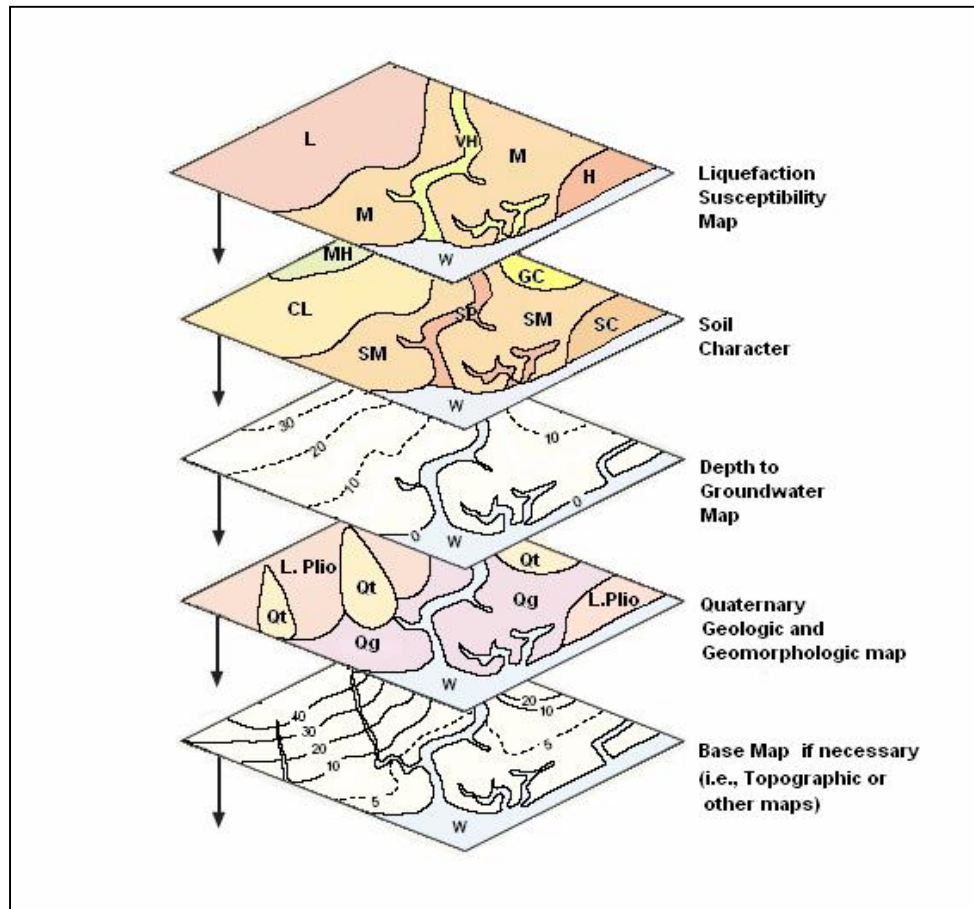


Figure 6.7. Data sources and integration scheme to produce a liquefaction susceptibility map (the figure was modified from Hitchcock et al., 1999).

According to the basic steps stated previously, regional seismic hazard integration methodology might be summarized as follows:

(4) Considering the first three steps presented above (i.e., steps (1) to (3)), maps showing the distribution of hazards due to site effects in the region are developed.

(5) Rules are defined to quantify the regional hazard distributions developed in Step (4). The rules should quantify the hazards in a consistent manner that allows them to be combined as described in Step (6).

(6) The hazard combination rules are in the form of weighted averages with the weights determined by local expert opinion. The weights depend on knowledge about the behavior of the local geology and the relative accuracy associated with hazards. The increased hazard due to two or more effects occurring simultaneously is also considered in these rules.

(7) A final map showing the distribution of combined hazard in the region is developed by overlaying the multi-layer maps developed in Steps (1) to (4) according to the heuristic rules defined in Steps (5) and (6).

6.3.1.2. Evaluation Results of the Liquefaction Susceptibility Mapping in Ankara Basin

Liquefaction susceptibility is the relative likelihood that a geologic unit, particularly loose, saturated cohesionless soils, would undergo liquefaction during seismic shaking. The liquefaction susceptibility can be determined on the basis of geological or geotechnical data. According to Youd and Perkins (1978) and Youd (1991), based on geological information, the alluvial deposits are likely to undergo liquefaction. Regarding to this concern, the preliminary evaluation of liquefaction susceptibility of surficial deposits in the Ankara basin were analyzed on the basis of the geological and geomorphological surface, age of the deposits, textural composition and sediment characteristics, and the degree of the deposits' water saturation. The potential for liquefaction susceptibility are compiled and modified using the criteria from Tables 6.1 and 6.2. Hence, data from Quaternary geologic mapping, our compilation of shallow groundwater depths, and analysis of subsurface data were integrated to create a map of liquefiable sediments. The liquefaction susceptibility map shows the susceptibility of surficial deposits

within the Ankara basin to failure from liquefaction during seismically induced ground shaking.

As stated previously, the late Quaternary geology map was used as a basis to determine the liquefaction susceptibility of the deposits that were susceptible to liquefaction because these deposits are most susceptible to liquefaction (Youd and Perkins, 1978). The areas have flat or gentle sloping geomorphological features such as river terraces, flood plains, alluvial fans. Liquefaction can cause different kinds of ground failure depending on the ground slope, particularly the most critical slopes considered were 0-2% (University of Washington, 2000). The geomorphic surfaces in the surrounding highland of this low land basin are Late Pliocene to Pleistocene age and older and have a very low susceptibility to liquefaction and were not analyzed in this study. These units that have slopes above 5% were not considered in this liquefaction analysis because the depth to groundwater of these units is in general greater than 10-15 m and has very low liquefaction susceptibility. Additionally, since these units are also consolidated and indurated cohesive soils (generally CH and CL according to USCS), they are generally not considered susceptible to liquefaction.

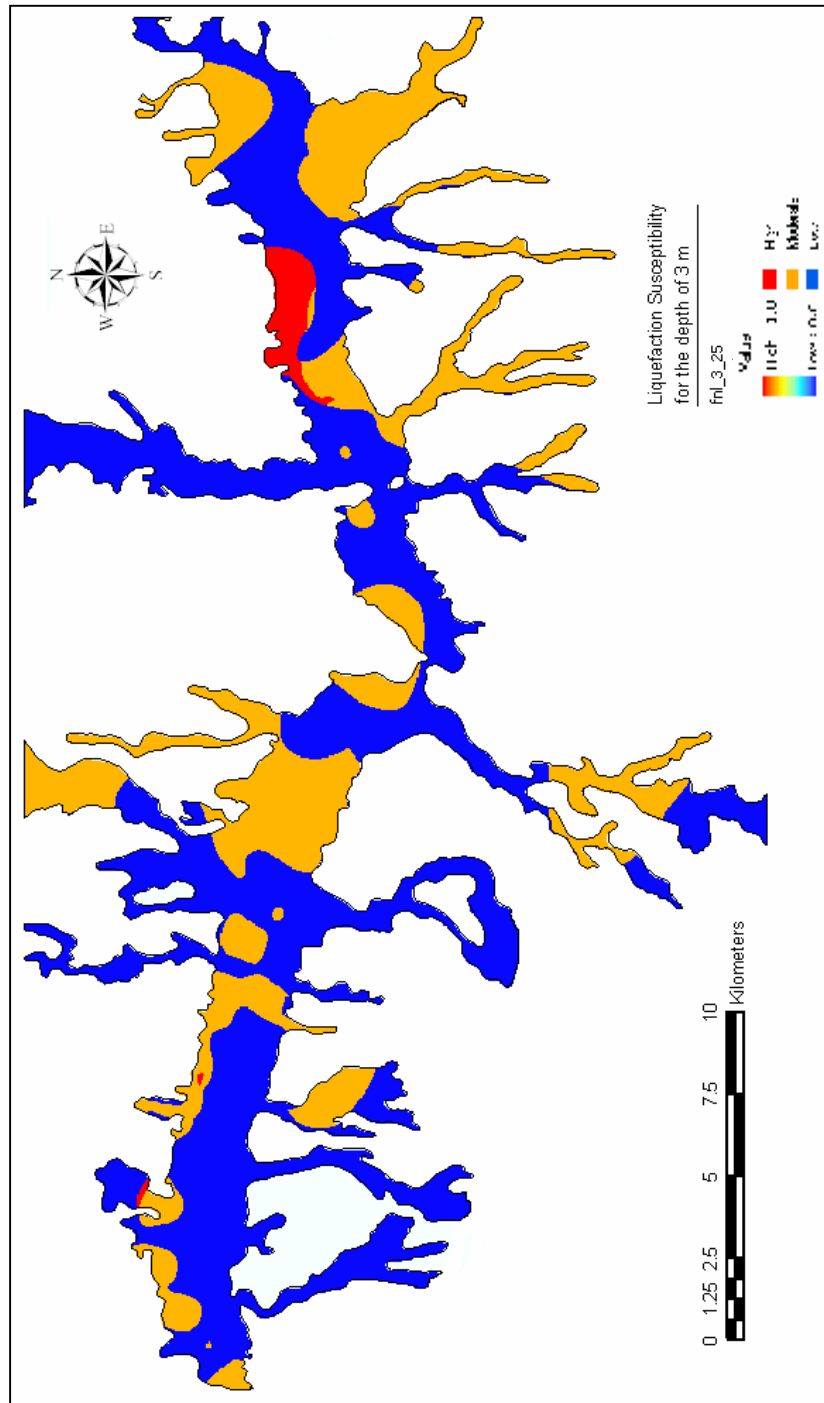
The relative susceptibility of the late Quaternary geological sedimentary units is determined by depth to ground water and the deposit's potential for becoming saturated. The typical depth to ground-water have been generalized from resistivity of VES and P-wave velocity measurements stated in Section 5.3.1.4.2 in order to idealize the thickness of saturated zone for these sediments in the Ankara basin. The liquefaction susceptibility varies from moderate (3-10 m) to very high (0-3 m) depending on the age of the deposits and depth to groundwater (see Table 6.2) because the depth to groundwater is about less than 10 m (5.5 m as an mean) for the late Quaternary geologic units in Ankara basin. Considering the seismic hazard potential of the Ankara basin explained above, the older geological units that are situated at higher elevations and further from the river basin where the depth to groundwater is greater than 10-15 m were not taken into account during the evaluation of the liquefaction susceptibility mapping.

Another primary importance to relative liquefaction susceptibility of the late Quaternary geologic sedimentary units is the sediment characteristics (i.e., soil type and estimated fines content) of the near surface deposits. As it was stated in Section 6.2, the sandy soils with few fines (less than 50 % particles finer than 0.074 mm); and in soils wherein the fines content are either non-plastic or are low plasticity silts and/or silty clays ($PI \leq 12$ to 15%) are potentially vulnerable to seismically induced liquefaction. Considering the available data to identify the soil profile in late Quaternary basin, the soil profiles from the boring results are not sufficient for evaluating the liquefaction susceptibility; therefore, other site characterization methods such as resistivity measurements might be used to develop consistent results. Hence, shallow surface isoresistivity results that have been prepared to classify for various depths ranging from 3, 5, 10, 15 m were used to identify the sediment characteristics. These data were used to quantitatively evaluate deposit susceptibility based on availability of the predicted sandy deposits within 15 m of the ground surface and the estimated threshold value (i.e., ≥ 25 ohm-m) required to indicate liquefaction susceptibility or not based on Table 5.14 given in Section 5.3.1.4.1.

The qualitative degree of susceptibility of the Late Quaternary geological unit was evaluated by Figures 6.8 through 6.11 which display the distribution of high, moderate and low liquefaction susceptibility hazards due to site effects in the research site for various depths ranging from 3, 5, 10 and 15 m, respectively. Finally, Figure 6.12 displays the overall distribution of combined hazard of liquefaction susceptibility within 15 m depth for the late Quaternary sedimentary unit that was developed by overlaying the maps of all different depths. The legend shows each of the geological mapping unit and its color with respect to its degree of susceptibility.

Regarding the methodology for the preliminary evaluation, all this has been integrated with the use of mapping tool to facilitate the use and distribution of the susceptibility maps. These liquefaction susceptibility maps should be useful for land use planning in order to properly assess the risk from liquefaction hazards. Note that the geological criteria maps are for determining potential

liquefaction hazards on a regional planning basis and not for individual sites that were stated before. Therefore, regarding the available data, the liquefaction susceptibility maps in this research area can also be used as complementary for evaluating liquefaction potential in late Quaternary sediments in the western part of the Ankara basin.



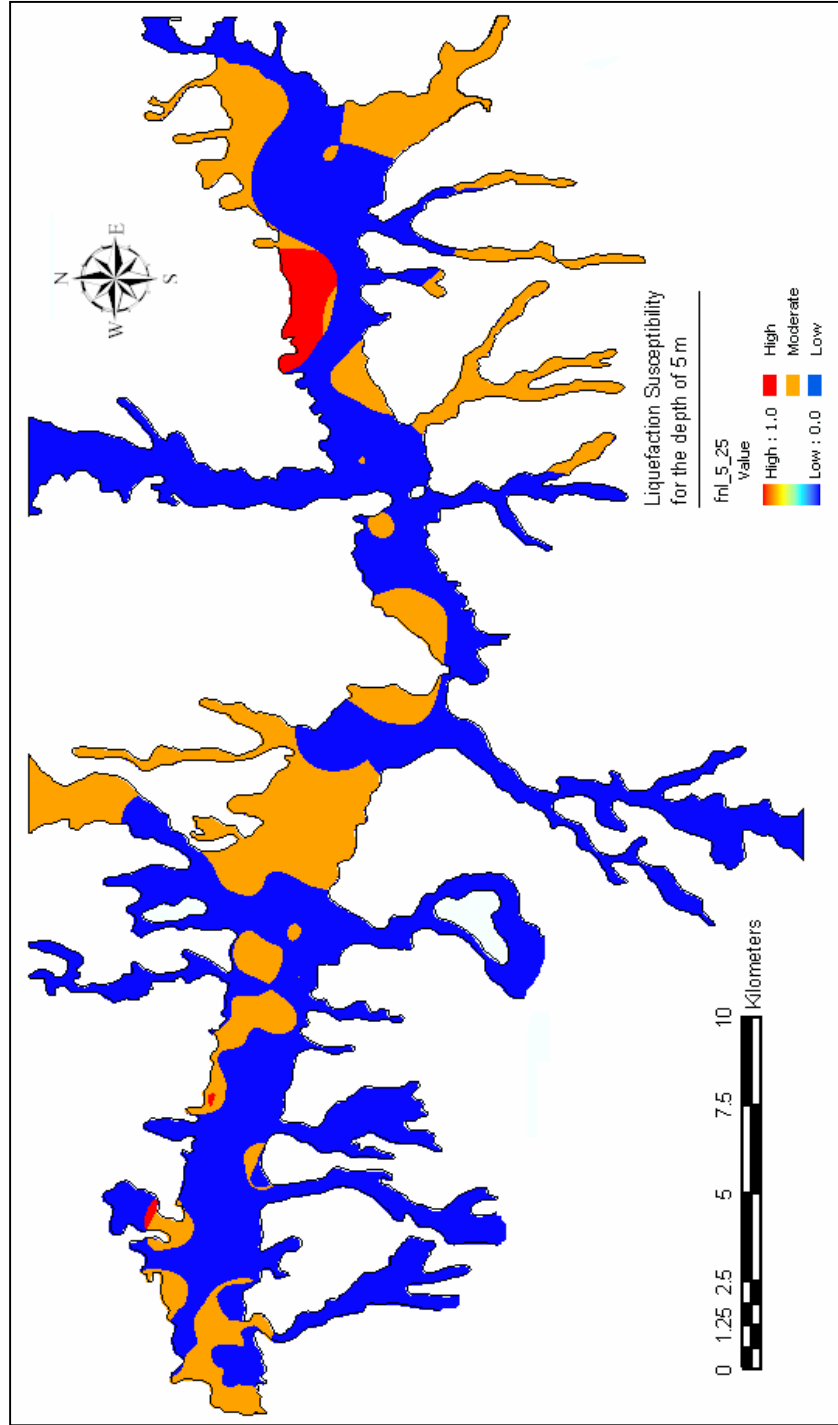


Figure 6.9. Liquefaction susceptibility hazard map due to site effects in Upper Quaternary deposits for a depth of 5 m.

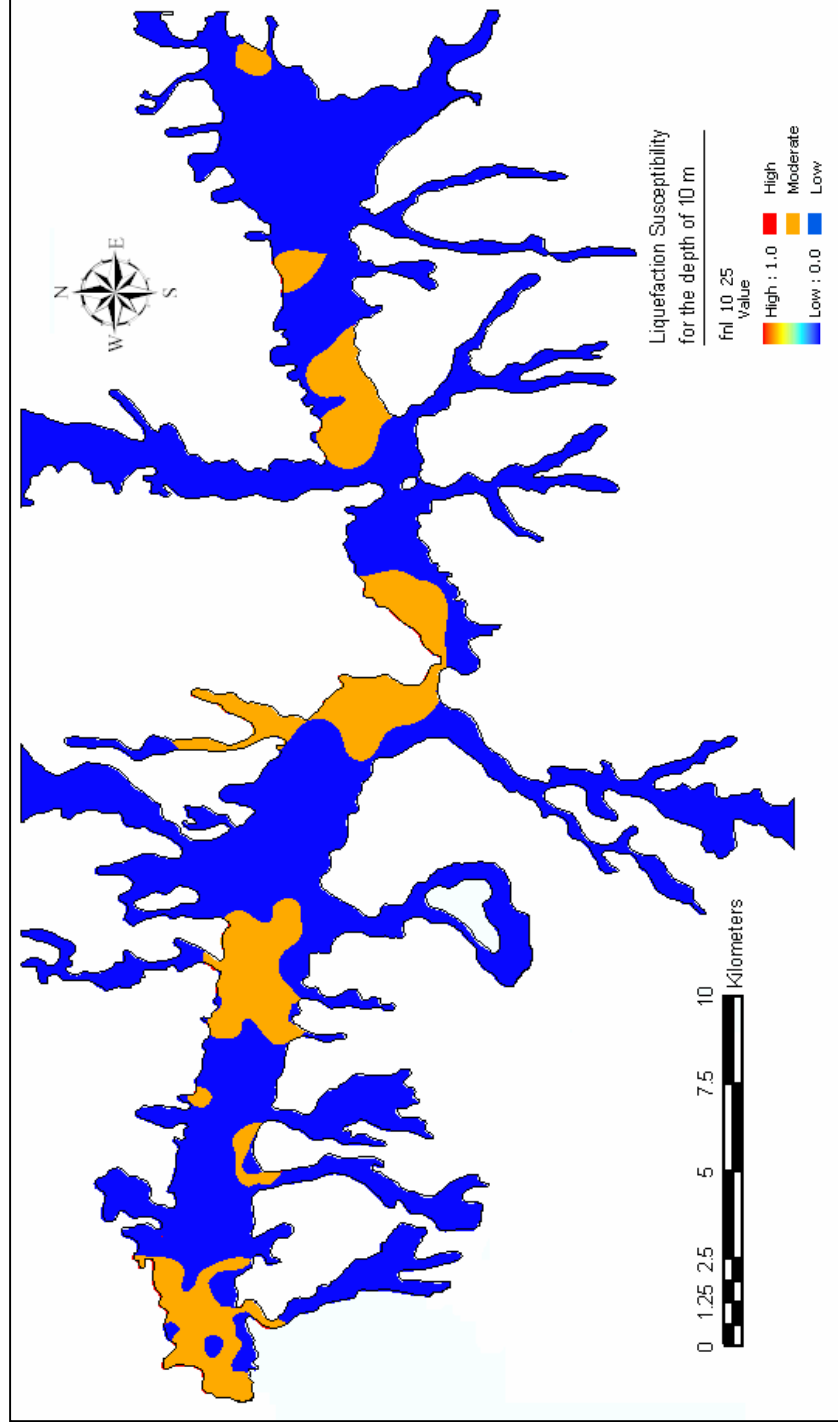


Figure 6.10. Liquefaction susceptibility hazard map due to site effects in Upper Quaternary deposits for a depth of 10 m.

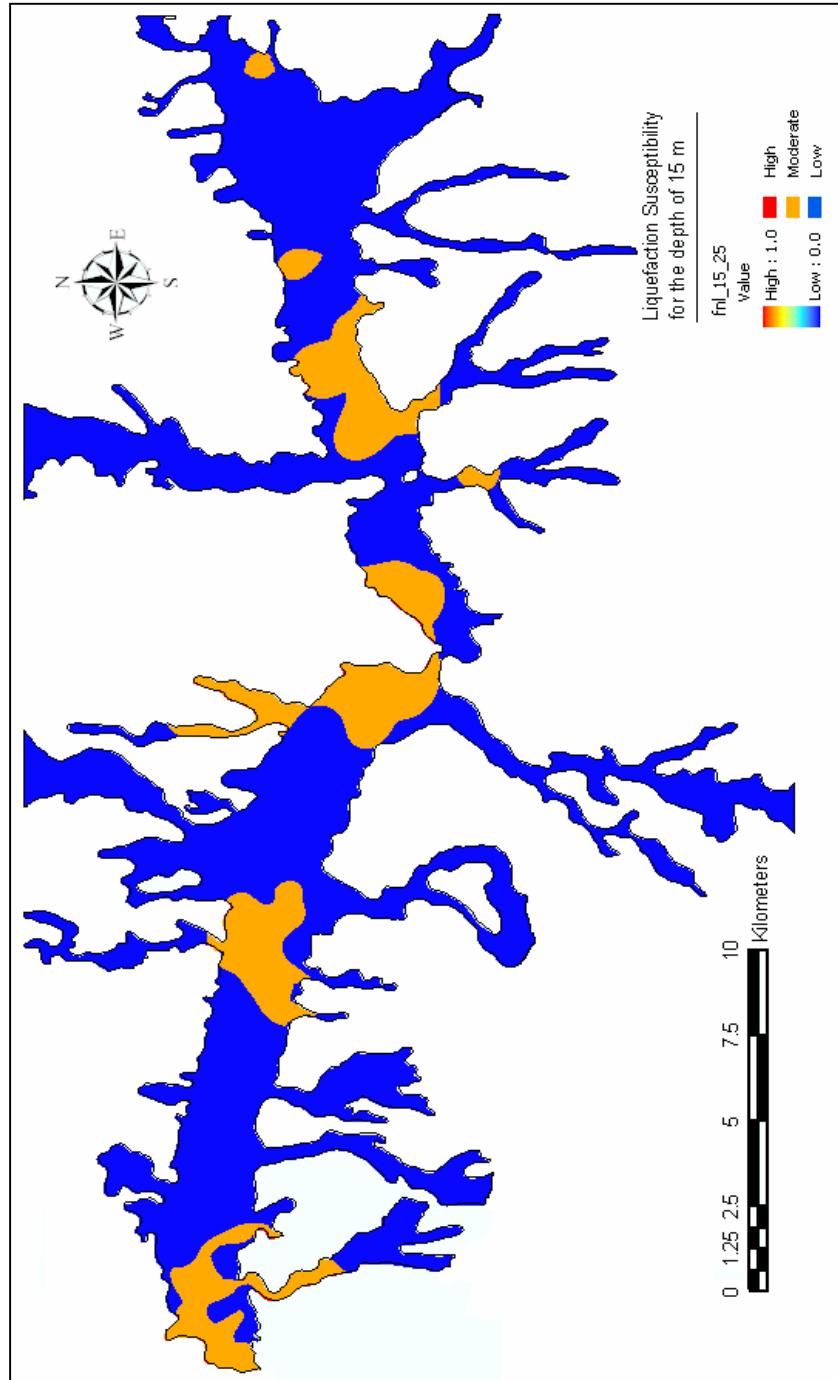


Figure 6.11. Liquefaction susceptibility hazard map due to site effects in Upper Quaternary deposits for a depth of 15 m.

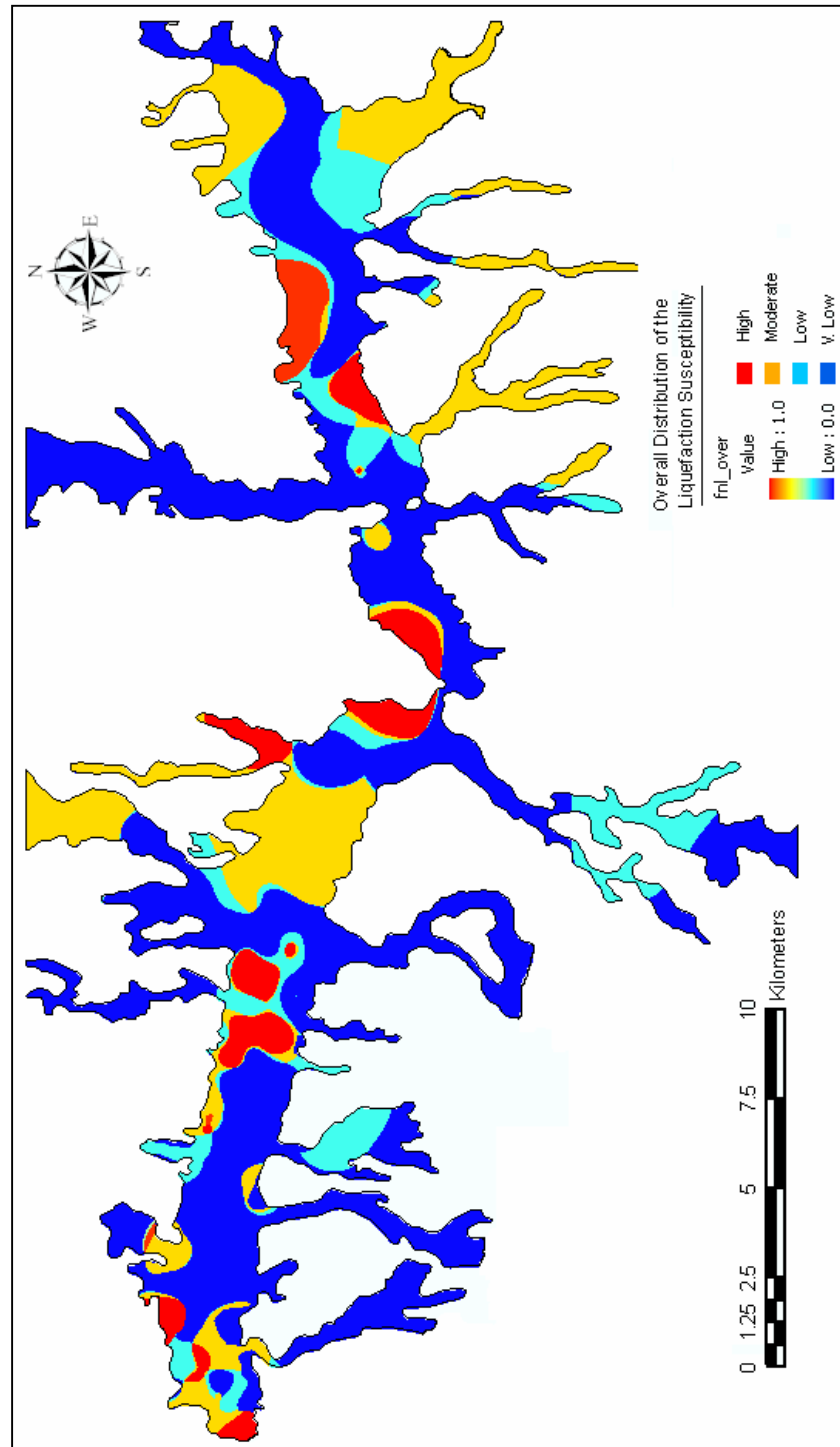


Figure 6.12. The overall distribution of combined hazard of liquefaction susceptibility within all depths down to 15 m depth in the Upper Quaternary deposits

6.3.2. Evaluation of Liquefaction Potential

Because of the lack of uniqueness in the relationship between geological criteria and geotechnical properties, susceptibility maps based on geologic criteria generally do not provide definitive information for site specific evaluations. Specific subsurface investigations are required for evaluating the actual liquefaction potential at a specified site. Hence, site-specific studies may be required before major development is approved. Nevertheless, most susceptibility maps based on geologic criteria serve to perform a preliminary evaluation to complement geotechnical criteria in regards to verify or quantify the classification system.

Liquefaction potential maps may yield quantitative measures that can be contoured to define zones with different probabilities of liquefaction occurring within a given interval. Whether or not these maps provide reliable and useful estimates of liquefaction potential depends on the scale and accuracy with which they are compiled. The quantity of geotechnical test data directly affects the statistical accuracy where potential units can be characterized.

Application of geotechnical criteria allows more quantitative rating of liquefaction potential and more precise definition of categories. The various categories that have been used, however, are no more uniform between maps than qualitative measures. It should be noted that for either of the qualitative or quantitative classification systems, definition of categories for mapping should be carefully selected to interpret the systems.

Although such databases are invaluable and should be used wherever they are available, two common deficiencies are associated with such data. (1) The areal distribution of data is usually non-uniform. Considerable data are typically available from a few sites where major development has occurred, some data may be available from other developed areas, and few or no data are usually available from undeveloped areas. Qualitative classification schemes should reflect uncertainties caused by disparities in the data (Youd, 1991). (2) The standard test methods and their procedures have long been questioned and criticized (deMello,

1971; Ireland et al., 1980; Kovacs, 1981). To reduce errors associated with these difficulties, NCEER (1997) and Youd et al. (2001) recommended procedures which, if carefully applied, improve test quality and reduce test variability. For instance, many standard penetration tests have been performed with insufficient quality control because test data usually come from a variety of sources; and test results have been assessed with empirical correlations and approximations. Use of those imprecise factors leads to results with some degree of uncertainty in the calculated results that is caused by sediment variability and possible errors in the measurement of soil properties (Liao et al, 1988). Where quality control is questionable and test variability may be significant, averaging of many test values yields more statistically significant results than reliance on individual tests. Thus, for mapping purposes, usage of statistical or probabilistic analysis of geotechnical data might be recommended, if feasible. It should be noted that the degree of uncertainty should be estimated or at least acknowledged in the explanation of the susceptibility ratings.

6.3.2.1. Liquefaction Potential Analysis

The methods to evaluate liquefaction potential of level grounds have been studied from several different points of view. Among them, the basics are the quantitative methods based on observations of the performance of saturated cohesionless granular soil deposits in previous earthquakes (empirical methods) and those based on evaluation of stress conditions in the field determinations of these conditions causing cyclic liquefaction of soils (analytical methods) (Seed, 1976).

In this study, liquefaction potential focuses on the western part of the Ankara basin particularly in the Quaternary alluvial plain. The liquefaction potential is a key element for the regional seismic hazard assessment in the study area because of the presence of alluvial granular deposits that are susceptible to liquefaction and proximity of the study area to the major earthquake shear zones

and systems. In practice, liquefaction potential is affected by a number of factors, including in-situ density N-values and/or Vs results, soil properties, groundwater table, overburden pressure, intensity and duration of earthquake shaking and the local geology that are the most commonly used indicators of the strength and liquefaction resistance.

Considering the application of general index criteria for evaluating the actual liquefaction potential at a specified site, the quality and accuracy of an overall regional liquefaction evaluation depends upon three major factors: (1) knowledge of local site conditions, (2) regional seismicity, and (3) use of proper evaluation criteria.

(1) Local Site Conditions: Subsurface conditions in the Ankara basin have been studied in detail by using standard penetration data from a total of 949 boreholes throughout the study area as discussed in Section 5.3.1.2. Among these data, standard penetration results were obtained at 429 testing points within the Quaternary deposits. The laboratory testing information associated with each boring was also included in the geotechnical database of the Ankara basin. These data represent the most comprehensive compilation of soil data available regarding general subsurface conditions in the study area, including soil type and stratification, in situ strength and ground water level. However, the volume of the compiled boring data available for the research area has mainly concentrated on Late Pliocene to Pleistocene fluvial deposits rather than Quaternary alluvial deposits which might result in a poorly distributed database for the purpose of this research. In addition to the SPT borings, shear-wave velocity measurements have been compiled as well as conducted at 259 locations at the project site that were incorporated into the geotechnical database as stated in Section 5.3.1.1. Among these 259 shear wave velocity measurements, 215 were performed in Quaternary deposits described either as younger, Holocene or older Quaternary deposits. Since boring data and geotechnical laboratory information at the surface wave measurement locations was relatively scarce, it was not possible to perform liquefaction analysis at those particular locations where boring and geotechnical data was missing.

The actual liquefaction potential at a specified site, the standard penetration results and shear-wave velocity measurements that were particularly performed in the Quaternary deposits were taken into consideration in the initial screening process. However, during this process, the liquefaction potential of the sites lacking geotechnical information (i.e., laboratory test results, index soil properties, soil classification, etc.) along with the in-situ test results that were categorized as “undefined” and particularly cohesive materials (i.e., fines content, $FC > 50\%$ and $PI \geq 15-20\%$ as threshold values) encountered within the soil profile of the in-situ test measurements (commonly encountered within the top 5 m) were considered as non-liquefiable. Therefore, a large volume of the SPT boring and shear wave velocity results were excluded from the liquefaction analysis. As a final screening, 71 SPT borings (note that two of the borings that were located in the fluvial deposits, namely BY49 and BY50 that were included since they were very close to the alluvial sediment boundary) and 21 surface wave measurements were incorporated into the database having appropriate geotechnical information to perform the site-specific quantitative analysis and being susceptible to liquefaction potential. A summary of the database of the in-situ testing results that were used in the liquefaction potential study is given in Table 6.4. The in-situ testing results of the SPT boring and shear wave velocity measurements along with the required geotechnical information are given in Appendix A.

(2) *Regional seismicity*: In order to assess the liquefaction potential of the sedimentary deposits in the Ankara basin, the peak ground acceleration of the soil deposits within the project site had to be determined. Accordingly, the liquefaction index criteria were used to establish the liquefaction potential maps for the study area in the event of $M = 7.5$ and 6.0 earthquakes in the Ankara region that is surrounded by the seismically active fault systems (i.e., NAFS, SLFZ, SFZ).

Table 6.4. Summary of the database of the in-situ testing results that were used in the liquefaction potential study for Quaternary deposits in the Ankara basin.

Method of In-situ Testing	Geologic Unit	# of Data and their Percentage (%)			
		Total # of data (pre-initial screening)	Appropriate Geotech. Lab. info. along with the in-situ test results (initial screening)	FC > 50% and PI ≥ 15 % (Cohesive material; post-initial screening)	Data Susceptible to Liquefaction (final screening process)
Standard Penetration Test of N_{30}	Quaternary alluvial and terrace deposits	429 (100 %)	358 (83,44 %)	287 (66,89 %)	71 (16,55 %)
Surface wave testing of V_s		215 (100 %)	78 (36 %)	56 (26,04 %)	21 (9,76 %)

Large earthquakes that may occur along the Gerede segment of the NAFS, SLFZ or SFZ might have a significant impact and pose a severe threat to the study area in Ankara. On the basis of results from a probabilistic seismic hazard analysis map of Turkey by Erdik et al. (1996) and Gülkan et al. (1993), PGA (g) values corresponding to a 475 year return period (10 % Probability of exceedance in 50 years) of the study area were estimated to be about 0.2g for bedrock acceleration which is consistent with the attenuation relationship studies that were developed for Ankara. Then, considering the estimated intensity for bedrock acceleration in the Ankara Region (PGA_{ROCK}), the site amplification of the soil deposits (PGA_{SOIL}) have been determined from the site characterization results (i.e., Site Class-D and Class-E) of the Ankara basin in regards to site coefficients (F_a) specified from IBC 2003 as discussed in Section 5.2.

The liquefaction evaluation in this study considers two major earthquakes assumed to occur in the frame of the “background seismicity” of the Ankara region (bedrock acceleration is about 0.2g) given in Table 6.5 according to Frankel et al. (1996) as described below.

1. $M = 7.5$ earthquake: an extreme case with a low probability of occurrence that might occur along the relatively distant (closest distance to the fault rupture is about 80 km) and seismically active fault systems (i.e., NAFS,

SLFZ, SFZ) being capable of producing large destructive seismic events in Ankara.

2. $M = 6.0$ earthquake: an event likely to occur in the foreseeable future with a moderate probability of occurrence that might occur along the close distance fault systems (closest distance to the fault rupture is about 10-15 km that were explained in Section 3.2.1) being capable of producing moderate destructive seismic events in Ankara.

Table 6.5. Log of peak ground acceleration in g as a function of moment magnitude M (columns) and log hypocentral distance R in km (rows), B-C boundary sites, Central and Eastern United States (CEUS). Magnitude ranges from 4.4 to 8.2. Distance ranges from 10 to 1000 km.

logR\M: 4.40 4.60 4.80 5.00 5.20 5.40 5.60 5.80 6.00 6.20 6.40 6.60 6.80 7.00 7.20 7.40 7.60 7.80 8.00 8.20																			
1.00	-0.71	-0.62	-0.53	-0.44	-0.36	-0.29	-0.21	-0.14	-0.07	0.00	0.06	0.12	0.19	0.25	0.31	0.37	0.43	0.49	0.54
0.60																			
1.10	-0.84	-0.75	-0.66	-0.57	-0.49	-0.41	-0.33	-0.26	-0.19	-0.12	-0.05	0.01	0.07	0.14	0.20	0.26	0.32	0.38	0.43
0.49																			
1.20	-0.98	-0.88	-0.79	-0.70	-0.62	-0.53	-0.46	-0.38	-0.31	-0.24	-0.17	-0.10	-0.04	0.02	0.08	0.14	0.20	0.26	0.32
0.38																			
1.30	-1.12	-1.02	-0.93	-0.84	-0.75	-0.66	-0.58	-0.51	-0.43	-0.36	-0.29	-0.22	-0.16	-0.10	-0.03	0.03	0.09	0.15	0.21
0.27																			
1.40	-1.27	-1.17	-1.07	-0.98	-0.89	-0.80	-0.72	-0.64	-0.56	-0.49	-0.42	-0.35	-0.28	-0.22	-0.15	-0.09	-0.03	0.03	0.09
0.15																			
1.50	-1.43	-1.32	-1.22	-1.13	-1.03	-0.94	-0.86	-0.78	-0.70	-0.62	-0.55	-0.48	-0.41	-0.34	-0.28	-0.22	-0.15	-0.09	-0.03
0.03																			
1.60	-1.59	-1.48	-1.38	-1.28	-1.19	-1.09	-1.01	-0.92	-0.84	-0.76	-0.69	-0.61	-0.54	-0.48	-0.41	-0.35	-0.28	-0.22	-0.16
0.10																			
1.70	-1.76	-1.65	-1.54	-1.44	-1.35	-1.25	-1.16	-1.07	-0.99	-0.91	-0.83	-0.76	-0.69	-0.62	-0.55	-0.48	-0.42	-0.35	-0.29
0.23																			
1.80	-1.93	-1.82	-1.72	-1.61	-1.51	-1.42	-1.33	-1.24	-1.15	-1.07	-0.99	-0.91	-0.83	-0.76	-0.69	-0.63	-0.56	-0.50	-0.43
0.37																			
1.90	-2.07	-1.95	-1.84	-1.74	-1.64	-1.54	-1.44	-1.35	-1.26	-1.18	-1.10	-1.02	-0.94	-0.86	-0.79	-0.72	-0.65	-0.59	-0.53
0.46																			
2.00	-2.17	-2.05	-1.94	-1.83	-1.73	-1.63	-1.53	-1.43	-1.34	-1.25	-1.17	-1.09	-1.01	-0.93	-0.86	-0.78	-0.71	-0.65	-0.58
0.52																			
2.10	-2.28	-2.16	-2.04	-1.93	-1.83	-1.72	-1.62	-1.53	-1.43	-1.34	-1.25	-1.17	-1.09	-1.01	-0.93	-0.86	-0.78	-0.71	-0.65
0.58																			
2.20	-2.44	-2.32	-2.20	-2.09	-1.98	-1.87	-1.77	-1.67	-1.58	-1.48	-1.39	-1.30	-1.22	-1.14	-1.06	-0.98	-0.91	-0.83	-0.77
0.70																			
2.30	-2.63	-2.50	-2.38	-2.26	-2.15	-2.04	-1.94	-1.84	-1.74	-1.64	-1.55	-1.46	-1.37	-1.29	-1.20	-1.12	-1.05	-0.97	-0.90
0.83																			
2.40	-2.83	-2.70	-2.57	-2.45	-2.34	-2.23	-2.12	-2.01	-1.91	-1.81	-1.72	-1.62	-1.53	-1.45	-1.36	-1.28	-1.20	-1.12	-1.05
0.98																			
2.50	-3.05	-2.92	-2.78	-2.66	-2.54	-2.42	-2.31	-2.20	-2.10	-2.00	-1.90	-1.80	-1.71	-1.62	-1.53	-1.45	-1.36	-1.28	-1.21
1.13																			
2.60	-3.29	-3.15	-3.01	-2.88	-2.75	-2.63	-2.52	-2.41	-2.30	-2.19	-2.09	-1.99	-1.90	-1.80	-1.71	-1.62	-1.54	-1.46	-1.38
1.30																			
2.70	-3.55	-3.40	-3.25	-3.11	-2.98	-2.86	-2.74	-2.62	-2.51	-2.40	-2.30	-2.19	-2.09	-2.00	-1.90	-1.81	-1.72	-1.64	-1.56
1.48																			
2.80	-3.84	-3.67	-3.52	-3.37	-3.23	-3.10	-2.97	-2.85	-2.73	-2.62	-2.51	-2.41	-2.30	-2.20	-2.10	-2.01	-1.92	-1.83	-1.74
1.66																			
2.90	-4.15	-3.97	-3.80	-3.65	-3.50	-3.36	-3.22	-3.09	-2.97	-2.85	-2.74	-2.63	-2.52	-2.42	-2.31	-2.22	-2.12	-2.03	-1.94
1.85																			
3.00	-4.48	-4.29	-4.11	-3.94	-3.78	-3.63	-3.49	-3.35	-3.22	-3.10	-2.98	-2.86	-2.75	-2.64	-2.53	-2.43	-2.33	-2.23	-2.14
2.05																			

(3) *Evaluation Criteria:* Quantitative analysis of geotechnical data were performed to evaluate liquefaction potential using the simplified standard penetration and shear wave velocity procedures discussed previously. Briefly, these procedures calculate soil resistance to liquefaction, expressed in terms of cyclic resistance ratio ($CRR_{7.5}$). $CRR_{7.5}$ values are then compared to calculated earthquake-generated shear stresses expressed in terms of cyclic stress ratio (CSR_{eq}). The factor of safety (FS_{liq}) relative to liquefaction is: $FS_{liq} = (CRR_{7.5} / CSR_{eq})$. FS_{liq} , therefore, is a quantitative measure of liquefaction potential. In this research, a factor of safety was taken as 1.0 or less, where CSR_{eq} equals or exceeds $CRR_{7.5}$, to indicate the presence of potentially liquefiable soil. While an FS_{liq} of 1.0 is considered the “trigger” for liquefaction, for a site specific analysis and FS_{liq} of as much as 1.25 may be marginally appropriate depending on the vulnerability of the site related structures for a regional assessment (Rathje, 2003). Note that these FS_{liq} vary in reliability according to the quality of the geotechnical data. These FS_{liq} as well as other considerations such as thickness and depth of potentially liquefiable soil were evaluated in order to construct liquefaction potential maps.

Considering the application of the general index criteria for evaluating the actual liquefaction potential at a specified site, some of the examples that represent the quantitative geotechnical evaluations and factor of safety calculations for different depths and site classes regarding-site specific analysis of SPT borings and shear wave V_s measurements in the event of both $M = 7.5$ and 6.0 earthquakes for Quaternary sediments in the Ankara basin are given in Tables 6.6 and 6.7, respectively.

Table 6.7. Examples that represent the quantitative geotechnical evaluations and the factor of safety calculations for different depths and site classes regarding-site specific analysis of shear wave V_s measurements in the event of both $M = 7.5$ and 6.0 earthquakes for Quaternary sediments in the Ankara basin.

SIS-13 (E-Site) Vs _{avg} =138																								SIS-17 (D-Site) Vs _{avg} =195																							
Q (Alluvial Deposit)																								Q (Alluvial Deposit)																							
BAP																								BAP																							
GWT=3.7 m																								GWT=4.8 m																							
Seismic Refraction																								Seismic Refraction																							
Description	Depth (m)	PI	γ (kN/m ³)	σ' _v (kPa)	σ _u (kPa)	σ' _v (atm)	depth (m)	rd (m)	PGA soil (E)	CSR _{eq}	V _s	C _v	V _{s1} [*]	CRR _{T-5}	MSF (7.5)	MSF (6)	Kσ	Kα	CRR (7.5)	CRR (6.0)	F _{sliq}	F _{sliq}	M _{low}																								
																						7.5	6.0																								
SM/SMW(5<FC(10)<35)	1.50-1.95	0	18,50	36,08	36,08	0,36	1,95	1,00	0,34	0,220	100	1,29	129	0,06	1,0	1,8	1,00	1	0,060	0,106	0,273	0,483																									
SM/SMW(5<FC(10)<35)	3.00-3.46	0	18,50	63,83	63,83	0,63	3,46	0,98	0,34	0,217	100	1,12	112	0,05	1,0	1,8	1,00	1	0,050	0,088	0,231	0,409																									
SM/SMW(5<FC(10)<35)	4.50-4.96	0	18,50	79,31	91,58	0,78	4,96	0,97	0,34	0,186	100	1,06	106	0,05	1,0	1,8	1,00	1	0,050	0,088	0,269	0,477																									
SM/SMW(5<FC(10)<35)	6.00-6.46	0	18,50	92,35	119,33	0,91	6,46	0,96	0,34	0,164	170	1,02	174	0,14	1,0	1,8	1,00	1	0,135	0,239	0,822	1,465																									
SM/SMW(5<FC(10)<35)	7.50-7.96	0	18,50	105,38	147,08	1,04	7,96	0,95	0,34	0,151	170	0,99	168	0,12	1,0	1,8	0,99	1	0,119	0,210	0,786	1,392																									
Q (Alluvial Deposit)																								Q (Alluvial Deposit)																							
BAP																								BAP																							
GWT=4.8 m																								GWT=4.8 m																							
Seismic Refraction																								Seismic Refraction																							
Description	Depth (m)	PI	γ (kN/m ³)	σ' _v (kPa)	σ _u (kPa)	σ' _v (atm)	depth (m)	rd (m)	PGA soil (D)	CSR _{eq}	V _s	C _v	V _{s1}	CRR _{T-5}	MSF (7.5)	MSF (6)	Kσ	Kα	CRR (7.5)	CRR (6.0)	F _{sliq}	F _{sliq}	M _{low}																								
																						7.5	6.0																								
SC(5<FC(30)<35)	3.00-3.46	13	18,00	62,10	62,10	0,61	3,46	0,98	0,3	0,191	130	1,13	147	0,08	1,0	1,8	1,00	1	0,080	0,142	0,418	0,741																									
SC(5<FC(30)<35)	4.50-4.96	13	18,00	87,63	89,10	0,87	4,96	0,97	0,3	0,186	130	1,04	136	0,07	1,0	1,8	1,00	1	0,065	0,115	0,349	0,618																									
SC(5<FC(30)<35)	6.00-6.46	13	18,00	99,91	116,10	0,99	6,46	0,96	0,3	0,161	220	1,00	221	0,35	1,0	1,8	1,00	1	0,350	0,619	2,172	3,846																									
SC(5<FC(30)<35)	7.50-7.96	13	18,00	112,20	143,10	1,11	7,96	0,95	0,3	0,146	220	0,97	214	0,35	1,0	1,8	0,98	1	0,341	0,604	2,339	4,141																									

The calculations of the liquefaction potential considering the previous explanations were used to generate profiles of the factor of safety against liquefaction potential versus depth for each engineering geological and geotechnical seismic testing (SPT and V_s) as included in Appendix A. These profiles can be used to visualize the data effectively in 2-dimensions as plots of factor of safety versus depth. Some of the representative examples of the factor of safety results regarding site-specific analysis of SPT borings and shear wave V_s measurements in the event of both $M = 7.5$ and 6.0 earthquakes for Quaternary sediments in the Ankara basin are given in Figures 6.13 and 6.14, respectively. However, these profiles are essentially 4-dimensional as they have geographical coordinates (latitude and longitude) as well as the factor of safety and depth. Visualizing data with more than 3-dimensions is impossible with graphical software. Such data can be visualized as a 2-D cross section with the graphs of factor of safety versus depth overlaid at their respective locations. It should be noted that the Quaternary deposits are heterogeneous, showing variations both in vertical and horizontal directions in the Ankara basin. Interpolating the data sets with sudden changes may provide less meaningful results. Thus, developing a cross section with factor of safety versus depth profiles and interpolating the values between the soundings might be useless because the prediction by the liquefaction potential index is different. The factor of safety procedure predicts what will happen to a soil element whereas the potential index predicts the performance of the whole soil column and the consequences of liquefaction at the ground surface (Toprak and Holzer, 2003). Therefore, the thickness and depth of potentially liquefiable soil might be evaluated in order to obtain reliable liquefaction potential results.

Similarly, Ishihara (1985) also stated that the potential for liquefaction-induced ground failure is related to the thickness of liquefiable soil layers and non-liquefiable soil layers. If the thickness of the overburden non-liquefiable layer is smaller than the thickness of underlying liquefiable layer, ground failure will occur. If the thickness of non-liquefiable layer is greater than a threshold value, which depends on the magnitude of the peak horizontal ground acceleration, there

will be no ground failure at this site. Note that if the water table is below the ground surface, the definition of non-liquefiable layer depends on the nature of the superficial deposits.

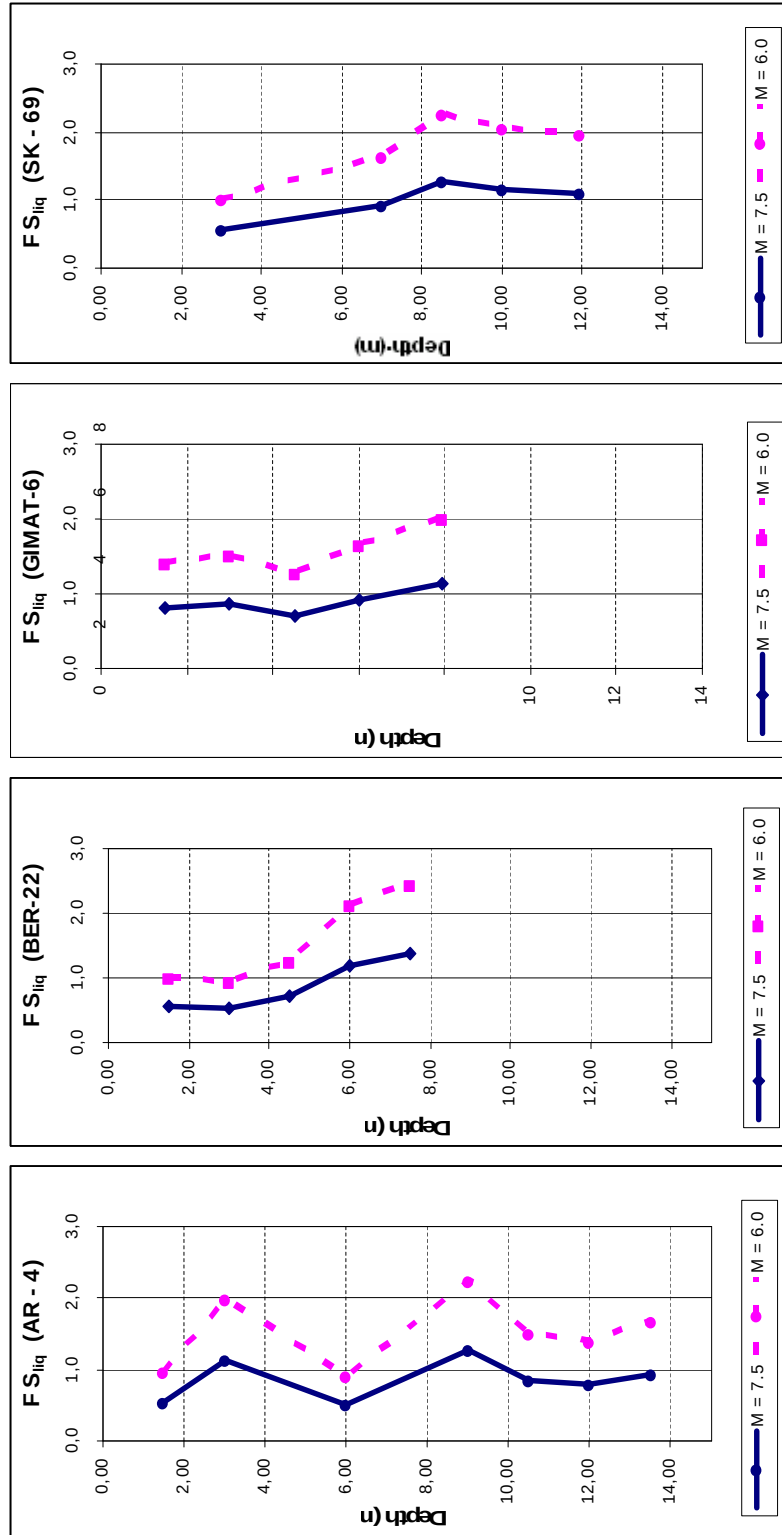


Figure 6.13. Some of the representative examples of the factor of safety results against liquefaction potential versus depth for site-specific analysis of SPT borings in the event of M = 7.5 and 6.0 earthquakes, respectively, regarding the Quaternary deposits in the Ankara Basin

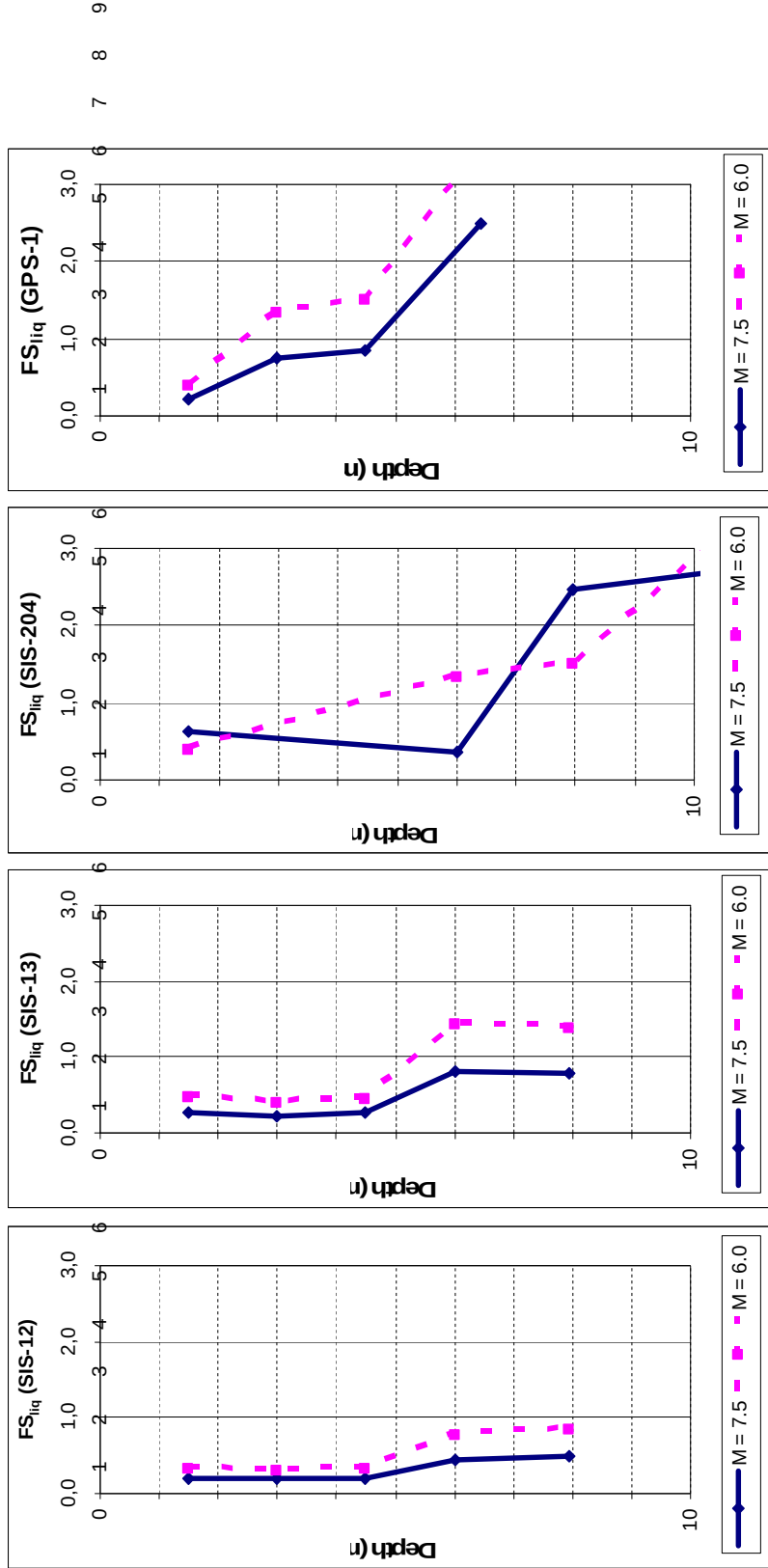


Figure 6.14. Some of the representative examples of the factor of safety results against liquefaction potential versus depth for site-specific analysis of surface wave V_s measurements in the event of $M = 7.5$ and 6.0 earthquakes, respectively, regarding the Quaternary deposits in the Ankara Basin

6.3.2.2. Evaluation of the Liquefaction Potential through the Liquefaction Potential Index (LPI)

The Liquefaction Potential Index (LPI) has been presented in the literature by Iwasaki et al. (1978 and 1982) and more recently by Shinozuka (1990), Chameau et al. (1990), Chameau et al (1991) and Luna and Frost (1998). While the liquefaction potential is defined by Seed and Idriss (1971) and Youd et al. (2001) to be evaluated for a soil at a particular depth in the subsurface, the LPI is an integrated effect of the likely liquefaction over the entire depth of the profile. Whereas this index is not without pitfalls, it is easy to use and particularly useful for mapping liquefaction damage or failure potential of an area. They assumed that the severity of liquefaction should be proportional to the

1. Thickness of the liquefied layer;
2. Proximity of the liquefied layer to the surface; and
3. Amount by which the factor safety (FS) is less than 1.0, where FS is the ratio of the liquefaction resistance to the load imposed by the earthquake.

The Liquefaction Potential Index (LPI), as defined by Iwasaki et al. (1982) as follows:

$$LPI = \int_0^{\infty} w(z) \cdot [1 - FS_L(z)] \cdot dz \quad (6.12)$$

where $w(z)$ = weight function (defined subsequently); $FS_L(z)$ = factor of safety against liquefaction, and dz = differential increment of depth. Iwasaki et al. (1982) suggested a linear function to represent the impact of the layers with depth below the surface as follows:

$$w_i(z) = 10 - 0.5z \quad (6.13)$$

where z = depth from ground surface in meters. This function gives more weight to the layers near the ground surface (maximum value of 10) and decreases linearly to zero at a depth of 20 m. It is well documented that most damage due to liquefaction occurs in the upper 20 m of the soil deposit. Other weighing functions could be used, such as a constant value or nonlinear distribution with depth, as might be appropriate to reflect the strain or stress distribution beneath an isolated footing. When using other weighting functions, care must be taken to keep the LPI as an index in percentage. This is done by having the area under the weighting function equal to 100% as is the case of the linear function in Eq. (6.13). Regardless of which weighting function is used, this procedure provides an efficient way to represent a 3D phenomenon (liquefaction with depth) on a 2D surface (Luna and Frost, 1998). This is not possible with most other liquefaction evaluation procedures as stated previously.

The factor of safety against liquefaction, $FS_L(z)$, as defined previously by Eq. (6.12), gives a deterministic indication of whether or not liquefaction will occur; however, a more graduated indication can be obtained by using the severity function expressed as follows:

$$S=0 \quad \text{for } FS_L(z) > 1 \quad (6.14a)$$

$$S = 1 - FS_L(z) \quad \text{for } 0 \leq FS_L(z) \leq 1 \quad (6.14b)$$

where S = degree of severity.

The liquefaction potential index equation shown as a continuous function in Eq. (6.12) can be expressed in a discretized form as follows:

$$LPI = \sum_{i=1}^{NL} w_i S_i H_i \quad (6.15)$$

where w_i = weight function for layer i ; S_i = severity for layer i ; H_i = thickness of discretized layer; and NL = number of discretized layer. The severity function is assumed to be directly proportional to the difference between the factor of safety

and the critical value of 1.0. Therefore, LPI is proportional to the liquefaction severity for the layer, the thickness of the liquefied layer, and the weight function.

The LPI is calculated for each boring or sounding. The term sounding is used since the data available can be in the form of a standard penetration test (SPT), a cone penetrometer test (CPT) or might be applicable for shear wave velocity test methods (V_s). The procedures to calculate the factor of safety for liquefaction potential stated previously can be performed for any depth in the soil deposits that contains a measurement of the resistance to penetration, and therefore, the LPI can be calculated as shown in the following Figure 6.15. Once the LPI has been calculated for the soil column, this index can be compared with an interval criteria that corresponds to different levels of liquefaction. The higher the index, the higher the potential is for liquefaction. These intervals are based on a comparison of computed values of LPI with observed performance.

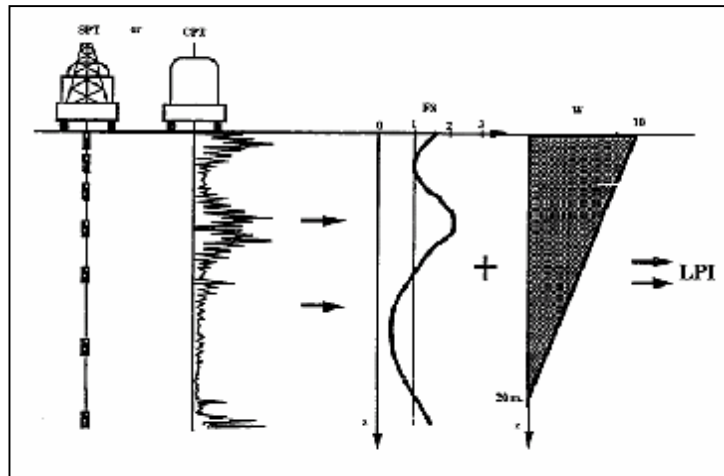


Figure 6.15. Conceptual graphic representation of LPI calculation (After Luna, 1994)

Iwasaki et al. (1982) used the liquefaction evaluation method recommended in the Japanese Highway Bridge Design Code (JSHE, 1990) to calculate the factor of safety. Based on their analysis of a database containing 64

liquefied sites and 23 non-liquefied sites resulting from six earthquakes, Iwasaki et al. (1982) provided the following liquefaction potential criteria, referred herein as the Iwasaki criteria:

LPI= 0, the liquefaction failure potential is little to none;

$0 < \text{LPI} < 5$, the liquefaction failure potential is low;

$5 < \text{LPI} < 15$, the liquefaction failure potential is moderate;

$\text{LPI} > 15$, the liquefaction failure potential is high.

The LPI calculated with this methodology facilitates converting the factor of safety versus depth profile into a single value at the ground surface, which is ideal for visualization regarding the liquefaction potential map. Hence, the method gains increasing usage because it permits 2D representation of a 3D phenomena with contours or grid values as the third dimension through the vertical weighting function, and then it can be incorporated into earthquake hazard mitigation practice useful for mapping liquefaction damage or failure potential of an area. Due to its ease of use, the Iwasaki et al. (1982) method has widely been adopted for the evaluation of the liquefaction potential sites of the Ankara basin. Once the LPI values have been determined for each in-situ test, namely, the standard penetration test (SPT) or the shear wave velocity test (V_s) or both, ranges of severity were applied to facilitate interpretation and visualization of the results.

A total of 71 sites with SPT borings and 21 sites with surface wave measurements that were previously used in safety factor calculations were analyzed. Then, the liquefaction potential for each borehole was calculated using the variation of the safety factors obtained along the entire depth of the borehole for all liquefiable soil layers. Additionally, the SPT and V_s data were combined later to generate the overall liquefaction hazard of the area. Each in-situ measurements were independently evaluated for the liquefaction potential such that the resulting LPI value was associated with a specific coordinate location. The LPI values were then applied to facilitate interpretation and visualization the results across the entire study area and to generate coverage of low, moderate and high liquefaction potential values. The results of the LPI analyses of SPT borings

and surface wave V_s measurements along with the required site characterization information in the event of both $M = 7.5$ and 6.0 earthquakes for Quaternary sediments in the Ankara basin are tabulated in Tables 6.8 and 6.9, respectively. These databases result in the construction of the histograms and their accumulative percentages presented by Figures 6.16 through 6.23 which summarize the relation between the distribution of the calculated LPI values according to Iwasaki et al. (1982) and the frequency of the potential liquefied sites (number of cases) based on their SPT boring and surface wave velocity V_s results in the event of both $M = 7.5$ and 6.0 earthquakes, respectively. Note that the liquefaction hazard for the study area was determined for the two assumed events of $M = 7.5$ and 6.0 earthquakes in the Ankara region which was stated previously. Finally, a summary of the overall results for the two assumed earthquake events of $M = 7.5$ and 6.0 earthquakes through each testing method are presented individually by Tables 6.10 and 6.11. Note that the combined SPT boring and surface wave V_s measurement results are also tabulated in these tables.

Maps depicting regions of liquefaction potential based on SPT boring results and surface wave V_s measurement for the two assumed events of $M = 7.5$ and 6.0 earthquakes are presented in Figures 6.24 through 6.27, respectively. Once interpreting the available database that can be seen from Figures 6.24 to 6.26, where SPT boring and surface wave V_s data were not sufficient, and also the areal distribution of data was usually non-uniform in interpolating the results as a zonation and in preparing a contour map for evaluating the actual liquefaction potential (i.e., in overcoming the degree of uncertainty when determining the hazard ratings that do not represent the actual LPI values during the geostatistical analysis), liquefaction potential maps and their hazard zones were depicted as point based hazard regions by using the site-specific evaluation of the Liquefaction Potential Index to generate appropriate results and to reduce the degree of uncertainty during the development of the hazard maps of the research study in Ankara.

Table 6.8. LPI analysis of SPT borings along with the required site characterization informations in the event of M = 7.5 and 6.0 earthquakes for Quaternary sediments in the Ankara basin

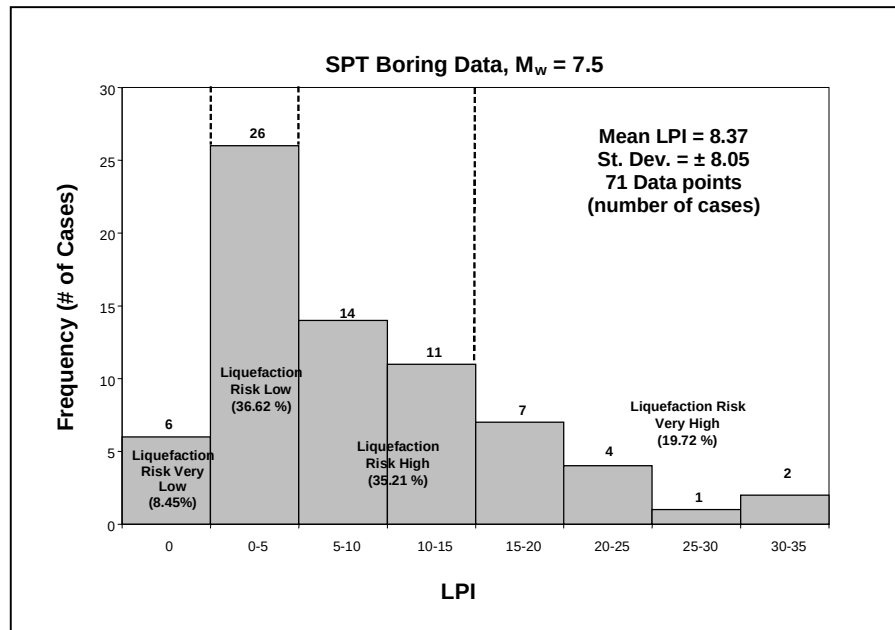
No of study	Site Information for SPT Testing	SITE CLASS (IBC 2003)	N(30) (blow/m)	Liquefaction Potential Index (LPI)	
				(M=7.5)	(M=6.0)
1	BM-20	D	19	0,57	0,00
2	SB SK-2	D	30	0,14	0,00
3	S-60	E	14	2,77	0,00
4	SB SK-A5	D	16	11,65	4,62
5	SB SK-A3	E	10	2,74	0,00
6	SK-A9	D	20	2,93	0,00
7	BH-14	D	16	3,09	0,00
8	BH-15	D	28	2,76	0,00
9	SK-22	E	14	0,08	0,00
10	SK-37	D	25	2,13	0,00
11	SK-47	D	23	3,04	0,00
12	SK-69	E	14	6,98	0,00
13	SK-70	D	25	0,00	0,00
14	SK-79	E	12	7,62	0,00
15	SK-75	E	14	7,69	0,00
16	SK-73	E	13	5,57	0,00
17	AR-1	E	10	33,61	6,06
18	AR-2	E	13	31,43	13,92
19	AR-3	E	13	8,48	0,00
20	AR-4	D	17	17,31	1,32
21	AR-5	E	14	24,91	14,32
22	AR-6	E	14	6,85	3,79
23	AR-7	E	9	20,39	4,43
24	AR-8	E	9	27,45	8,02
25	BY-26	D	27	0,16	0,00
26	BY-27	D	29	6,05	0,00
27	BY-36	D	18	6,65	0,00
28	BY-36/A	D	23	10,93	0,00
29	BY-36/B	E	13	10,66	0,00
30	BY-37	D	19	5,76	0,00
31	BY-38	E	14	18,35	3,29
32	BY-38/A	D	18	0,42	0,00
33	BY-39	D	21	3,62	0,00
34	BY-49	D	48	2,36	0,00
35	BY-35	D	26	5,61	0,00
36	BY-40	E	12	9,75	0,00
37	BY-34	D	33	3,21	0,00
38	BY- 50	D	30	3,71	0,00
39	BY- 52	E	14	13,76	8,38
40	BY- 51	D	27	10,05	4,81
41	S-24	E	14	12,80	1,84
43	S-65	D	21	0,26	0,00

Table 6.8. Continued.

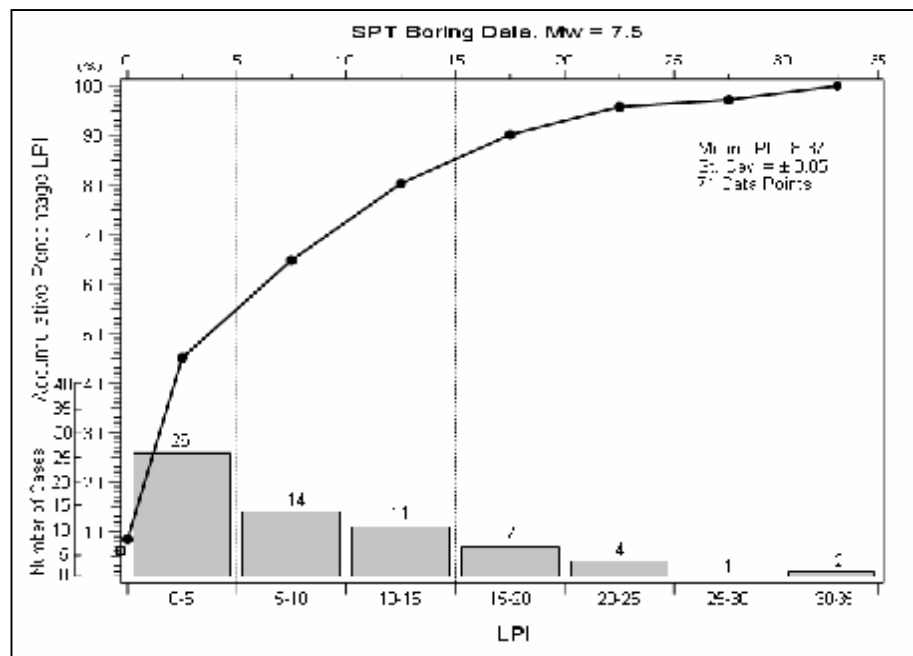
44	S-515	D	22	0,92	0,00
45	S-524	E	12	3,21	0,00
46	S- 525	D	18	15,87	2,07
47	S- 526	E	13	12,69	2,25
48	BER-THK-1	E	13	20,58	2,73
49	BER-THK-2	E	12	13,22	2,14
50	BER-THK-3	E	12	21,27	8,11
51	BER-22	E	14	18,44	1,01
52	EKA-20	D	16	11,23	0,00
53	BER-28	E	14	15,46	0,89
54	EKA-33	D	20	0,00	0,00
55	HIPOD-1	D	19	2,00	0,00
56	HIPOD-3	E	10	4,67	0,00
57	HIPOD-5	D	17	8,11	0,45
58	STAD-1	E	7	19,10	8,74
59	STAD-2	E	14	0,00	0,00
60	STAD-4	E	10	2,81	0,00
61	OPER-1	D	17	4,37	0,00
62	A-ETI-3	D	29	2,33	0,00
63	A-AYA-2	D	14	14,17	4,00
64	U Sub BH-7	E	14	7,63	1,43
65	KHO-2	D	26	0,00	0,00
66	GES-1	E	9	11,80	0,49
67	GIMAT-2	D	20	0,59	0,00
68	GIMAT-4	E	14	18,04	0,86
69	GIMAT-5	D	24	0,00	0,00
70	GIMAT-6	D	25	9,69	0,00
71	YEG-5	E	14	2,06	0,00

Table 6.9. LPI analysis of surface wave V_s measurements along with the required site characterization information in the events of both $M = 7.5$ and 6.0 earthquakes for Quaternary sediments in the Ankara basin

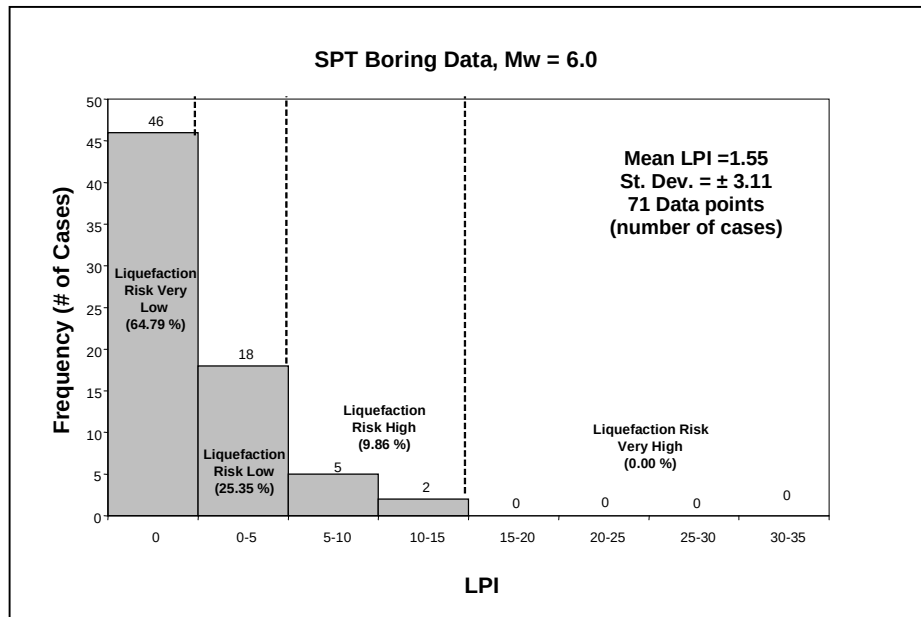
No of study	Site Information for V_s Testing	SITE CLASS (IBC 2003)	V_s (30) (m/s)	Liquefaction Potential Index (LPI)	
				($M_w=7.5$)	($M_w=6.0$)
1	BM-20	D	227	4,71	0,00
2	SB SK-2	D	289	0,00	0,00
3	SB SK-A5	D	211	10,71	0,00
4	SB SK-A3	E	180	3,59	0,00
5	SIS-45	D	200	19,01	0,00
6	SIS-169	D	250	0,00	0,00
7	SIS-165	E	180	12,02	7,42
8	SIS-168	D	240	0,00	0,00
9	SIS-102	D	232	0,00	0,00
10	SIS-130	E	173	8,37	3,25
11	SIS-131	E	175	21,01	15,61
12	SIS-108	E	175	17,32	5,14
13	SIS-12	E	125	35,07	23,91
14	SIS-9	E	172	20,55	10,18
15	SIS-13	E	138	27,86	18,06
16	SIS-17	E	195	9,85	5,10
17	SIS-182	D	205	8,79	0,00
18	SIS-204	D	193	12,09	2,42
19	SIS-183	E	154	19,64	7,37
20	GPS-1	E	180	18,78	10,60
21	GPS-2	D	246	0,00	0,00



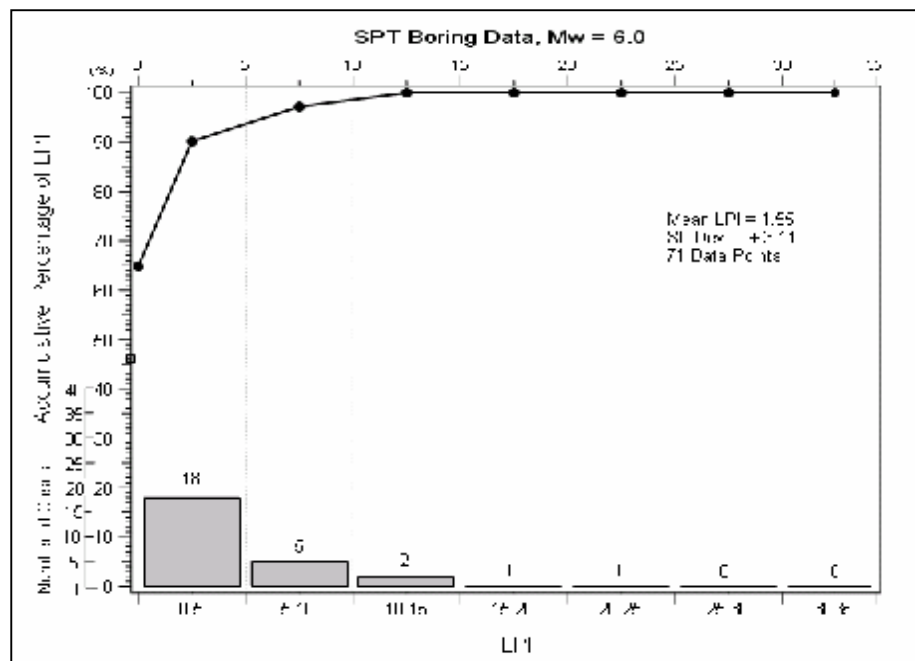
Figures 6.16. The relation between the histogram distribution of the calculated LPI values and frequency of the potential liquefied sites based on their SPT boring database in the event of $M = 7.5$ earthquake



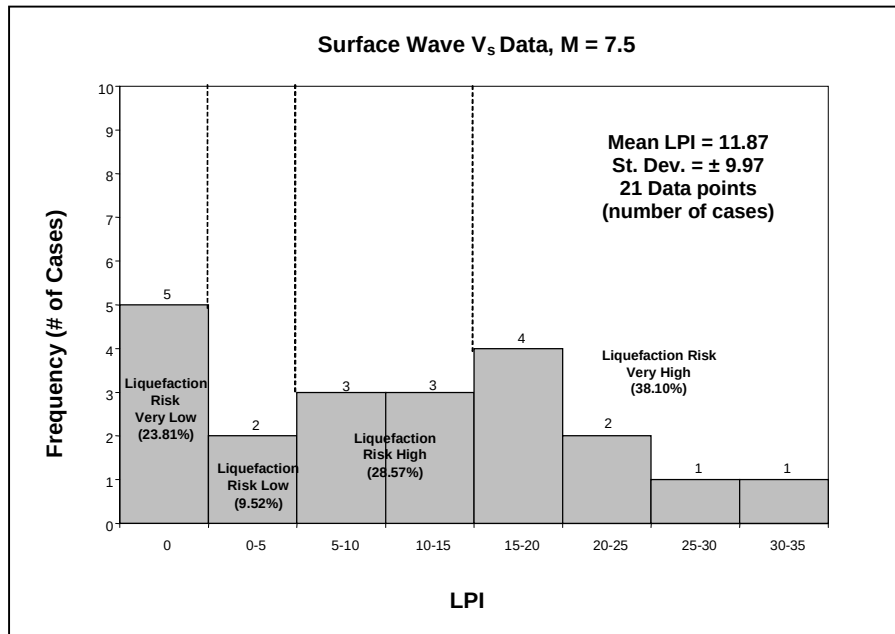
Figures 6.17. The relation between the percent accumulative distribution (%) of the calculated LPI values and frequency of the potential liquefied sites based on their SPT boring database in the event of $M = 7.5$ earthquake



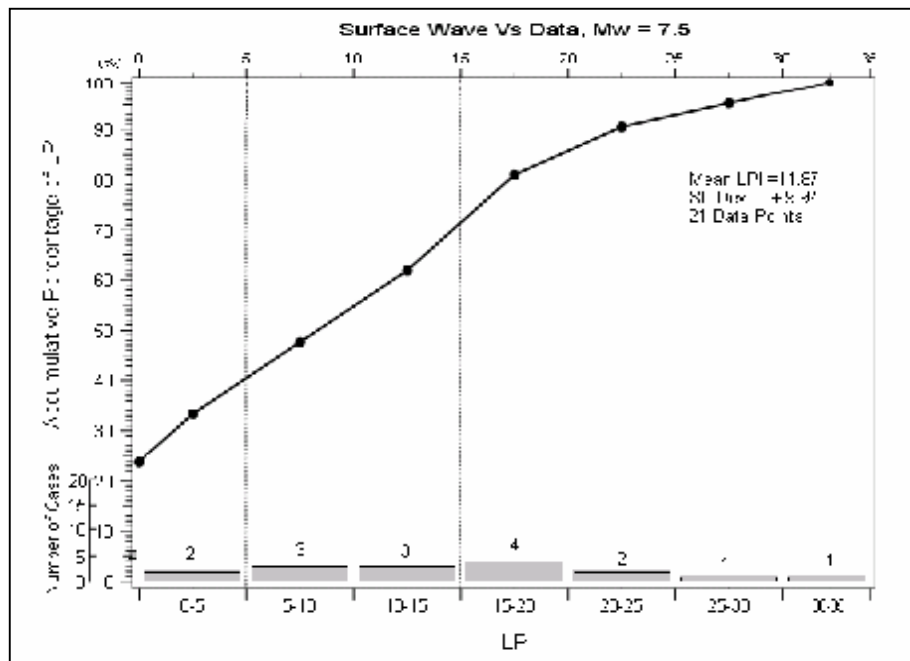
Figures 6.18. The relation between the distribution of the calculated LPI values and frequency of the potential liquefied sites based on their SPT boring database in the event of M = 6.0 earthquake



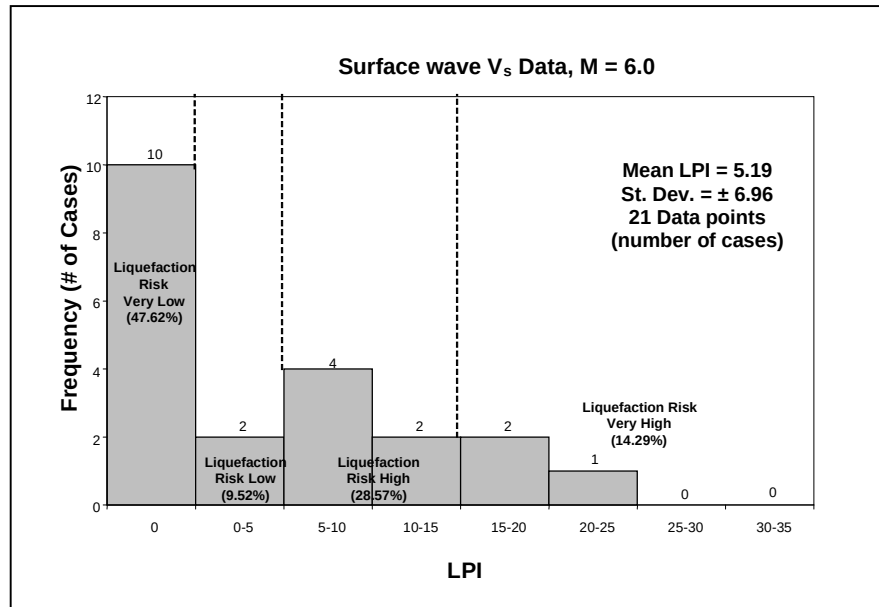
Figures 6.19. The relation between the percent accumulative distribution (%) of the calculated LPI values and frequency of the potential liquefied sites based on their SPT boring database in the event of M = 6.0 earthquake



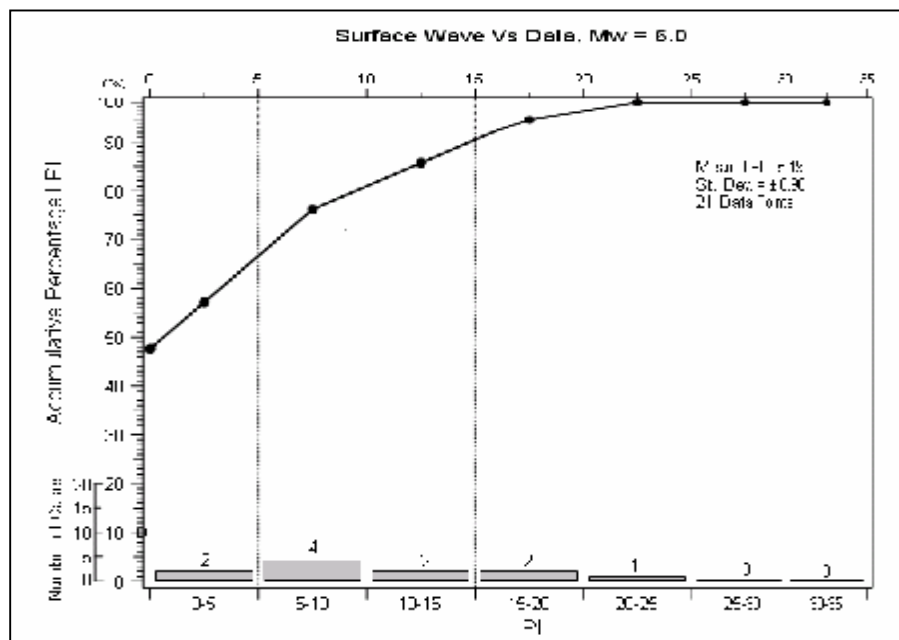
Figures 6.20. The relation between the distribution of the calculated LPI values and frequency of the potential liquefied sites based on their surface wave V_s database in the event of $M = 7.5$ earthquake



Figures 6.21. The relation between the percent accumulative distribution (%) of the calculated LPI values and frequency of the potential liquefied sites based on their surface wave V_s database in the event of $M = 7.5$ earthquake



Figures 6.22. The relation between the distribution of the calculated LPI values and frequency of the potential liquefied sites based on their surface wave V_s database in the event of $M = 6.0$ earthquake



Figures 6.23. The relation between the percent accumulative distribution (%) of the calculated LPI values and frequency of the potential liquefied sites based on their surface wave V_s database in the event of $M = 6.0$ earthquake

Table 6.10. Summary of the overall results for each testing method individually and results that combine the SPT borings and surface wave V_s measurements for an assumed event of $M = 7.5$ earthquake

Liquefaction Risk Criteria (Iwasaki et al., 1982)	Earthquake event of $M_w = 7.5$					
	SPT boring results (71 data points)		Surface wave V_s results (21 data points)		Combined SPT boring and V_s results (92 data points)	
Severity Range	Frequency	%	frequency	%	frequency	%
LPI= 0 (little to none)	6	8.45	5	23.81	11	11.96
$0 < \text{LPI} < 5$ (low)	26	36.62	2	9.52	28	30.43
$5 < \text{LPI} < 15$ (moderate)	25	35.21	6	28.57	31	33.70
LPI >15 (high)	14	19.72	8	38.10	22	23.91

Table 6.11. Summary of the overall results for each testing method individually and results that combine the SPT borings and surface wave V_s measurements for an assumed event of $M = 6.0$ earthquake

Liquefaction Risk Criteria (Iwasaki et al., 1982)	Earthquake event of $M_w = 6.0$					
	SPT boring results (71 data points)		Surface wave V_s results (21 data points)		Combined SPT boring and V_s results (92 data points)	
Severity Range	frequency	%	frequency	%	frequency	%
LPI= 0 (little to none)	46	64.79	10	47.62	56	60.87
$0 < \text{LPI} < 5$ (low)	18	25.35	2	9.52	20	21.74
$5 < \text{LPI} < 15$ (moderate)	7	9.86	6	28.57	13	14.13
LPI >15 (high)	0	0.00	3	14.29	3	3.26

For a $M=7.5$ earthquake, the majority of the study area is rated as possessing a low to moderate liquefaction hazard along with high liquefaction hazard (Figures 6.16 through 6.19, Table 6.10). More than 50% and 65% of them had a LPI value of greater than 5 (moderate to high) in the SPT boring and V_s results, respectively. A few occurrences of little to no hazard, particularly on SPT boring results (8.45%) are observed in the research area. The areas of estimated high liquefaction potential include the centrum and northern side of the Sincan

area along the Ankara River; Ergazi and Bahçeleriçi section of the Etimesgut area along the Ankara River; and some sections of the city center of the Ankara (i.e., GİMAT, Atatürk Conventional Center-Hipodrom, Dışkapı and Kızılay). This study shows that these sites have liquefiable soil layers ranging from a thickness of 6 to 15 m that extend from below the groundwater level (particularly high, ranging from a depth of 2 to 8 m (4.5 m as an average)) from the ground surface. Thus, loss in bearing capacity will likely occur at most of these sites. This shows that the mode of foundation failure due to liquefaction potential in the study area might be caused through the partial failure of supporting soils, causing differential settlement of structures, cracking, and buckling of pavements on ground. Note that these results are based on the availability of the database that has been utilized, therefore, a more refined database will lead to a more site specific liquefaction potential analysis to increase the accuracy and general distribution of the analysis. The general occurrences of these potential liquefaction sites are depicted in Figures 6.24 and 6.25. It needs to be stressed that throughout the Sincan-Eryaman-Batıkent Third Phase Metro Route and Kızılay-Çayyolu Second Phase Metro Route that possess low liquefaction potential at some sections lead to considerable liquefaction potential due to presence of heavy urbanization and high rise buildings present along the alluvial plains particularly along the tributaries of the Ankara River in Eryaman, Batıkent and Çayyolu sections. Note that when compared with the high volume of the SPT boring database, the surface wave V_s measurements, due to their low volumes, are not sufficient to be used individually for interpreting the liquefaction potential indices. This is the reason why the V_s results are generally coupled with the SPT boring results during the statistical interpretations and explanations of the results.

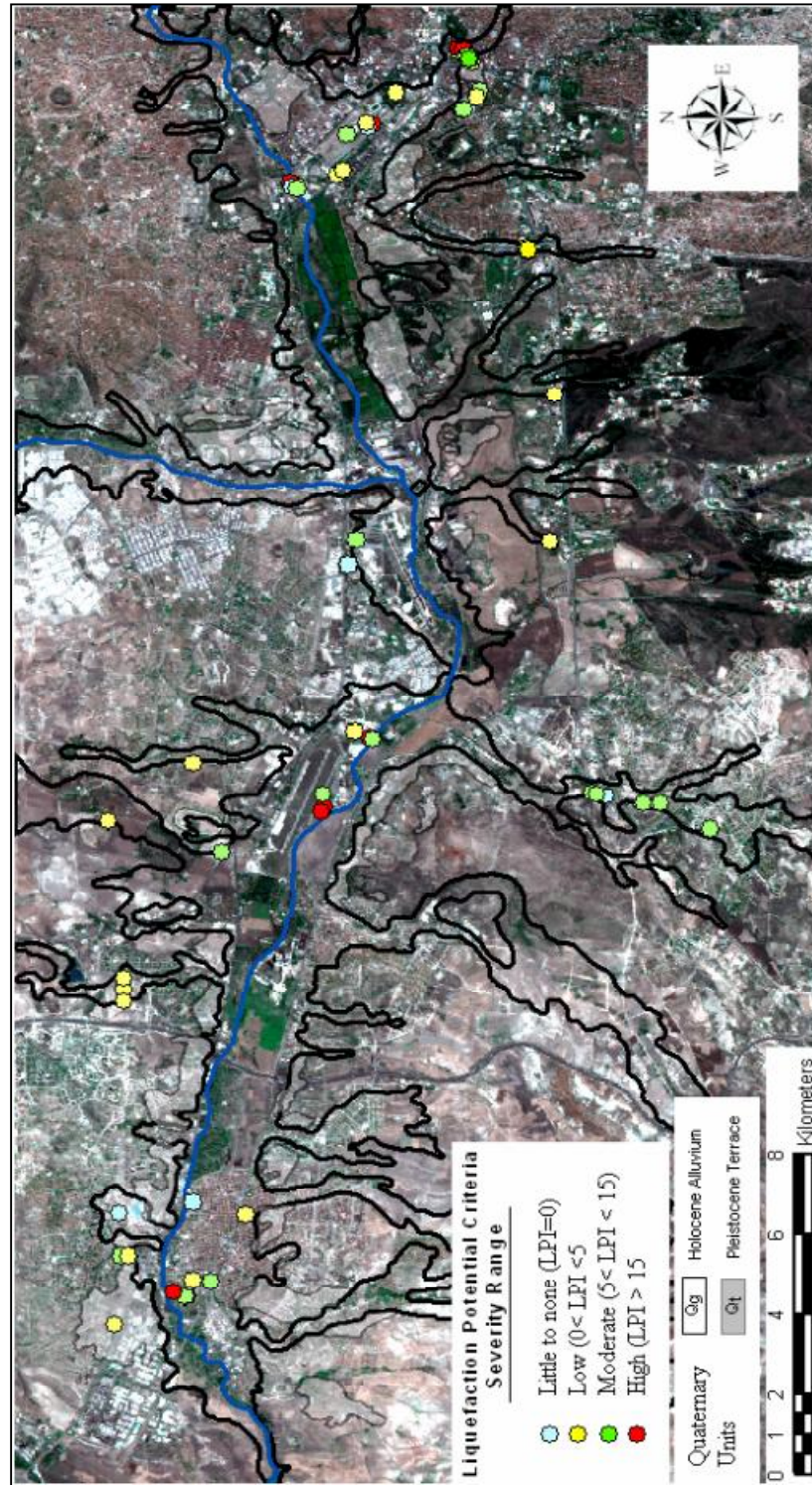


Figure 6.24. Map depicting regions of liquefaction potential based on the SPT boring database for an assumed earthquake event of $M = 7.5$ in the Quaternary deposits of the Ankara basin

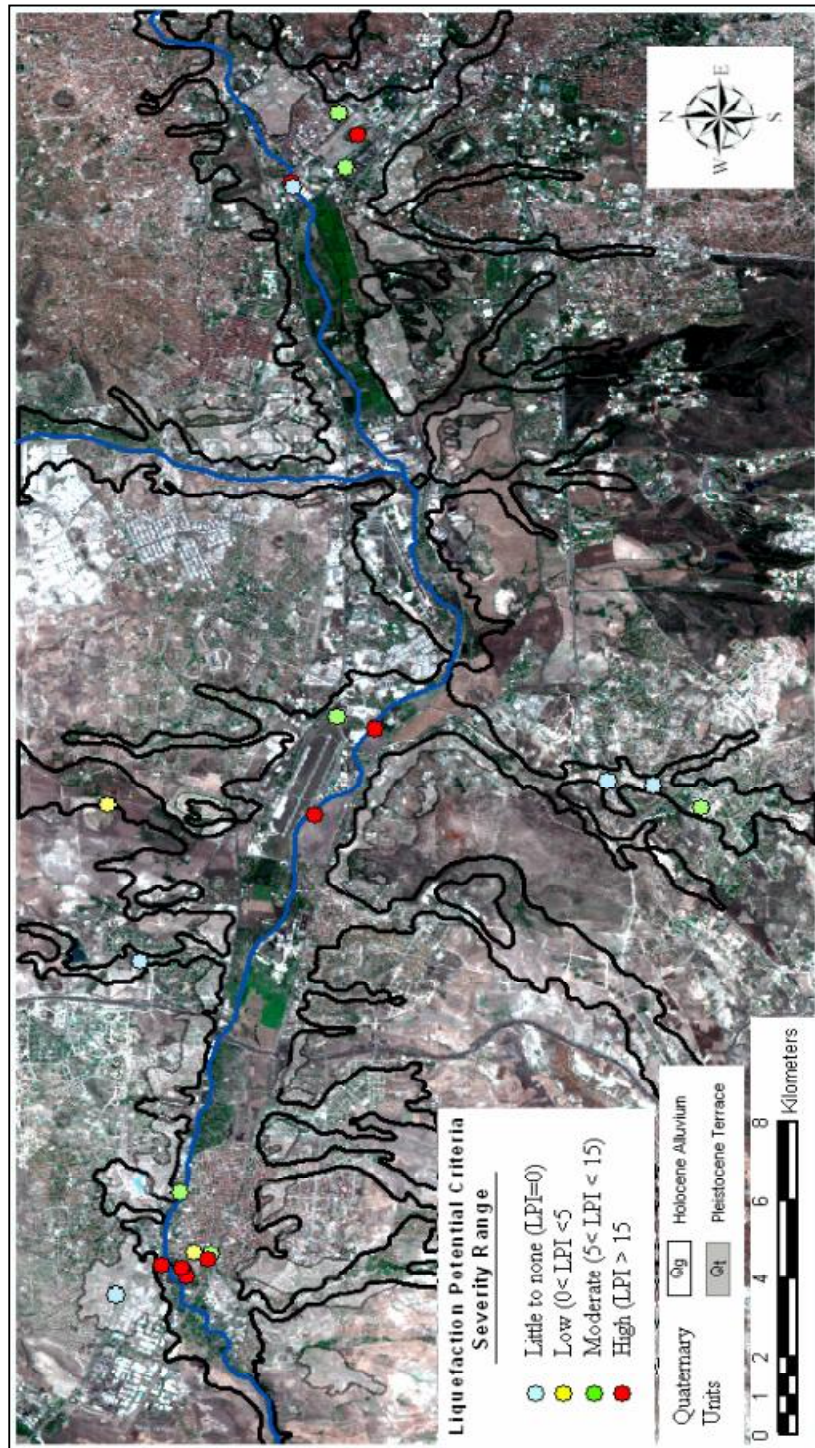


Figure 6.25. Map depicting regions of liquefaction potential based on the surface wave V_s database for an assumed earthquake event of $M = 7.5$ in the Quaternary deposits of the Ankara basin

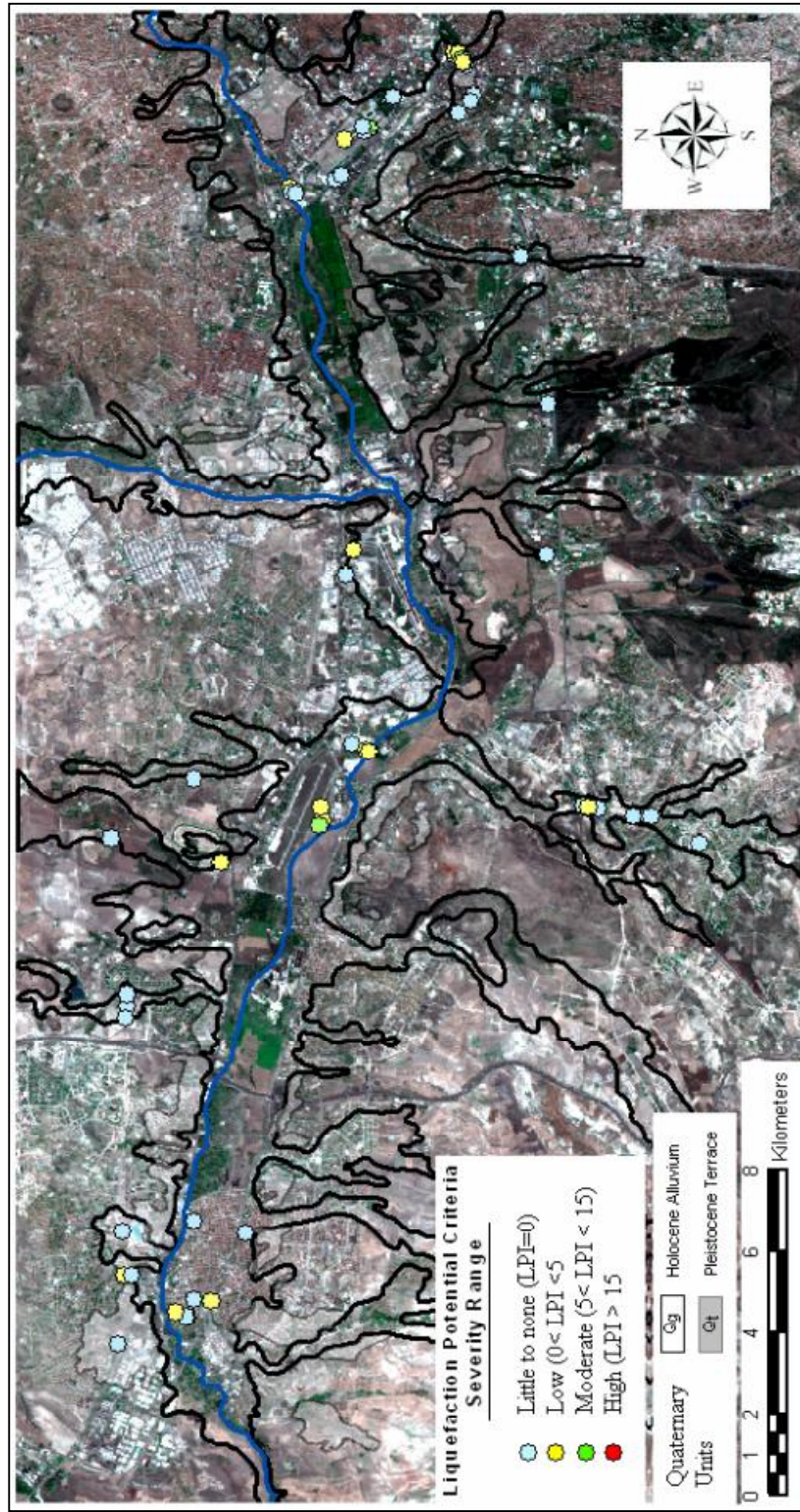


Figure 6.26. Map depicting regions of liquefaction potential based on the SPT boring database for an assumed earthquake event of $M = 6.0$ in the Quaternary deposits of the Ankara basin

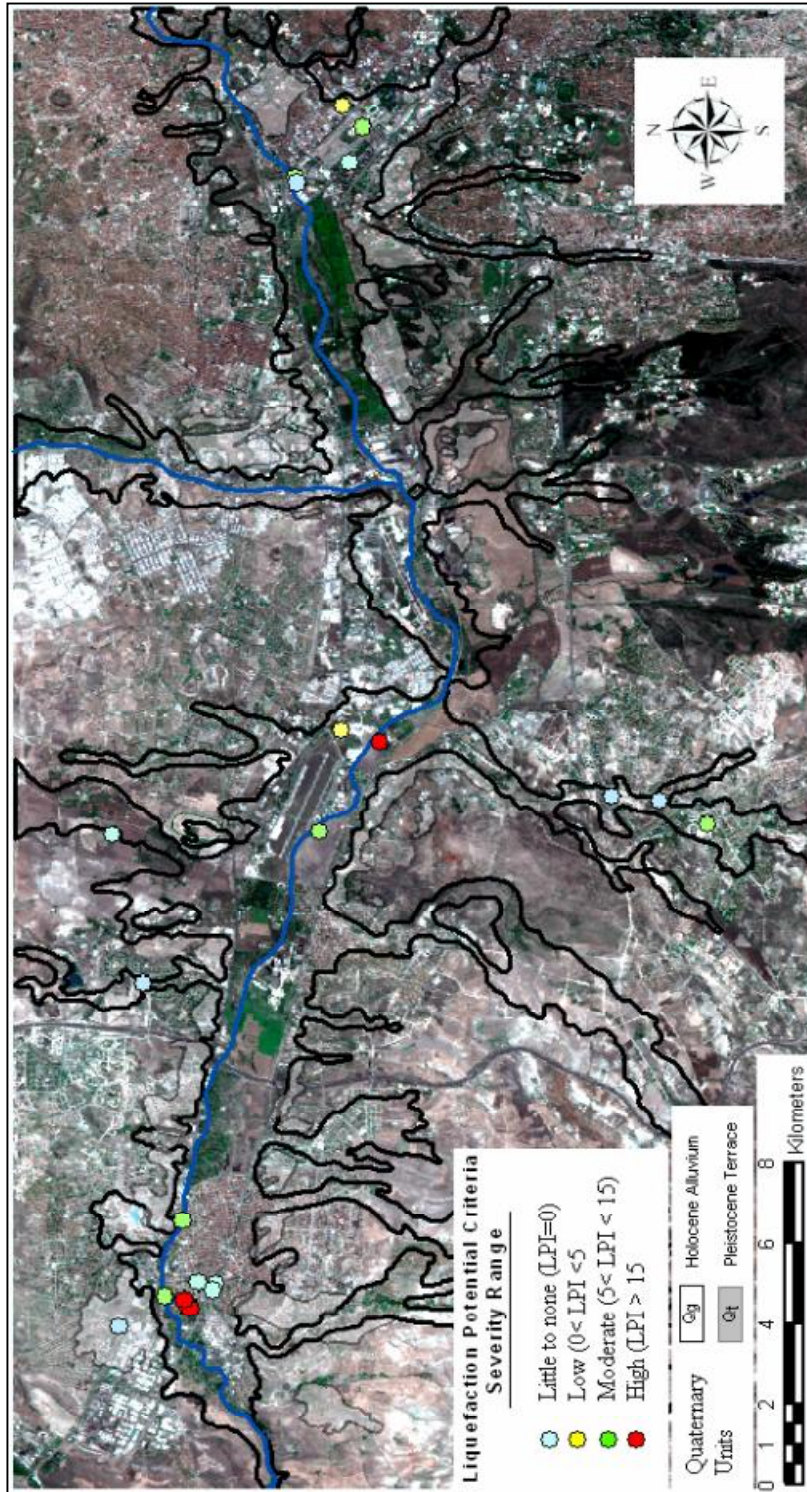


Figure 6.27. Map depicting regions of liquefaction potential based on the surface wave V_s database for an assumed earthquake event of $M = 6.0$ in the Quaternary deposits of the Ankara basin

As shown previously in Figures 6.20 through 6.23, the overall liquefaction potential of the study area is significantly reduced when subjected to an $M = 6.0$ event earthquake that appears to promote low liquefaction hazard in the Ankara basin. More than 90% and 55% of them had a LPI value of less than 5 (low to little) in the SPT boring and V_s results, respectively. The entire study area might be classified as little to none and low hazard except for a small section of moderate to high hazard when the surface wave V_s measurements are to be considered (Figure 6.27 and Table 6.11). Generally, areas of high liquefaction potential as related to an $M = 7.5$ event earthquake were estimated as low liquefaction hazards when related to an $M = 6.0$ event earthquake (Figures 6.26 and 6.27). The reasons of reduction in the magnitude of the liquefaction potential are the presence of liquefied granular soil layers having thicknesses less than 3 to 5 m. Besides, in some places of the study area, the thickness of the overlying non-liquefiable surface layers is larger than 3 m when compared to the underlying relatively thick liquefiable layers as proposed by Ishihara (1985), and thus liquefaction potential is considerably reduced in the study area in the event of an $M=6.0$ earthquake.

Consequently, a detailed liquefaction potential evaluation for each individual project site may be necessary to be performed for the design of structures in the areas of high and moderate liquefaction potential since $M= 7.5$ and particularly $M = 6.0$ (if high liquefaction potential still exists in the moderate magnitude) earthquakes may occur in the foreseeable future.

6.3.3. Hazard Evaluation and Liquefaction Hazard Map Preparation for Seismic Zonation

Considering the results of the preliminary geological evaluation and quantitative evaluation of liquefaction, an evaluation of the potential hazard is performed and combined liquefaction hazard maps pertaining to the project site were prepared.

The liquefaction hazard maps have to be considered together with the liquefaction potential maps since in the research area where the geotechnical and seismic data are not sufficient in volume and are not equally distributed to provide reliable regional liquefaction potential maps for seismic hazard mapping in Quaternary deposits of the Ankara basin. In addition to this, hazard study on a regional scale by using the liquefaction susceptibility, however, is not sufficient for evaluating the actual liquefaction potential at a specified site, and thus geologic criteria were used to delineate bounds of susceptibility zones have to be evaluated complementary with the geotechnical analysis to verify the classification system. As a concluding step, the liquefaction hazard maps were used to combine the liquefaction susceptibility and liquefaction potential results to generate single coverage hazard regions for the two assumed event of earthquakes in the Quaternary deposits of the Ankara basin are given in Figures 6.28 and 6.29, respectively.

Comparing the available site-specific data for the evaluation of the liquefaction potential results with the preliminary geological map in the evaluation of the liquefaction susceptibility, liquefaction hazard zones show either consistency or slight difference in some sections. However, a general consistency is observed in the combined liquefaction hazard maps considering the overall distribution of the severity ranges. Particularly, the liquefaction susceptibility map of “moderate” to “low susceptibility zones” were more reliably estimated with the site-specific liquefaction potential zones. However, it should be noted that the liquefaction potential maps may yield quantitative measures that can be used to define zones with different probabilities of liquefaction occurring within a given interval. Hence, these types of maps provide more reliable and useful estimates of liquefaction potential depending on the scale and accuracy with which they are compiled from site specific evaluations.

Consequently, the primary products of these liquefaction hazard evaluation studies performed herein that show the liquefaction susceptibility and liquefaction potential of the Ankara basin might be applied in order to properly assess and mitigate the potential risk from future liquefaction hazards in the study

area. The application of these evaluation results by using a combined methodology is useful for preliminary evaluations, general land-use planning, and delineation of special study zones where additional site-specific studies may be required before major development is approved. It is recommended that additional geotechnical field work regarding simultaneous seismic V_s and SPT boring studies be performed in the future to enlarge the database and to increase the accuracy and validity of the hazard zoning map presented herein.

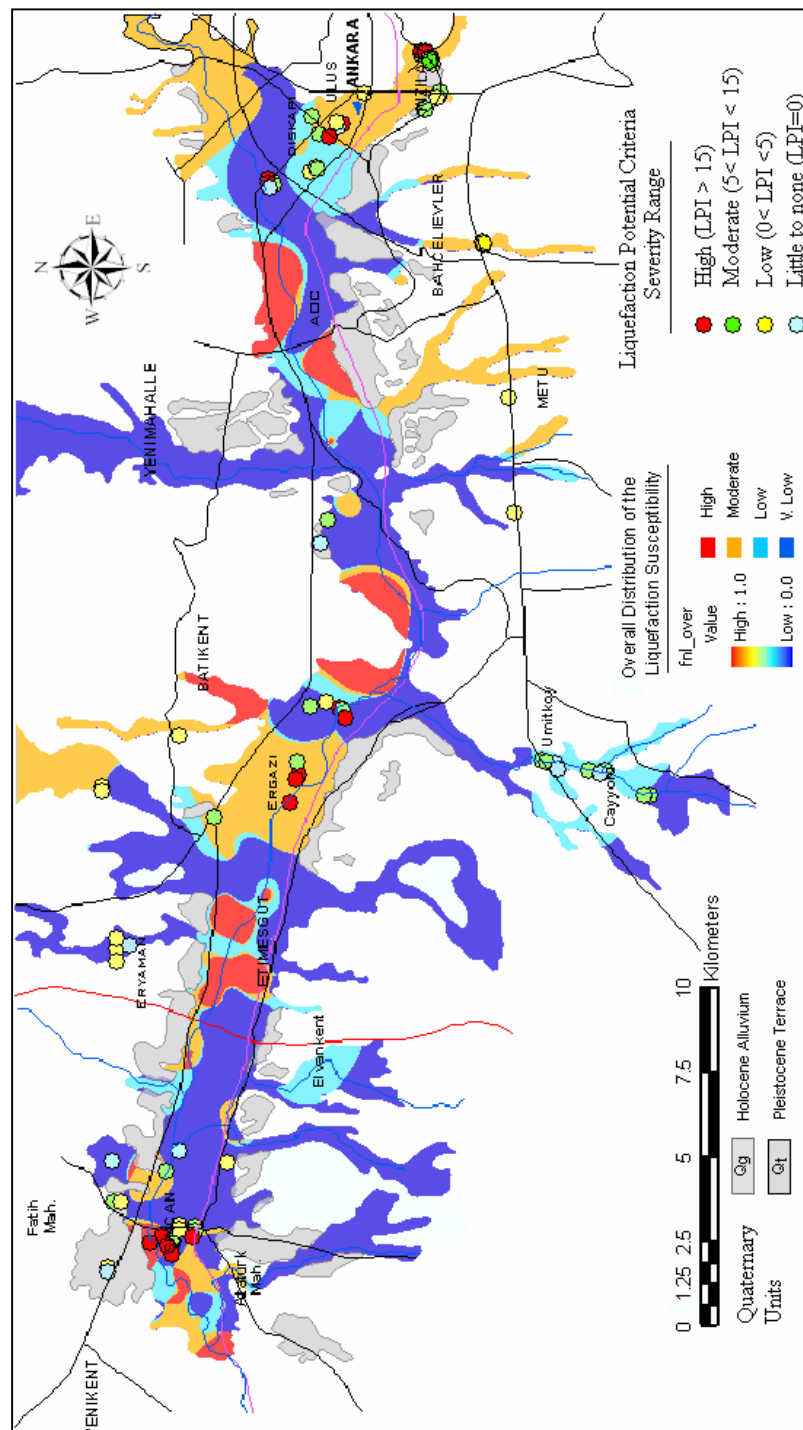
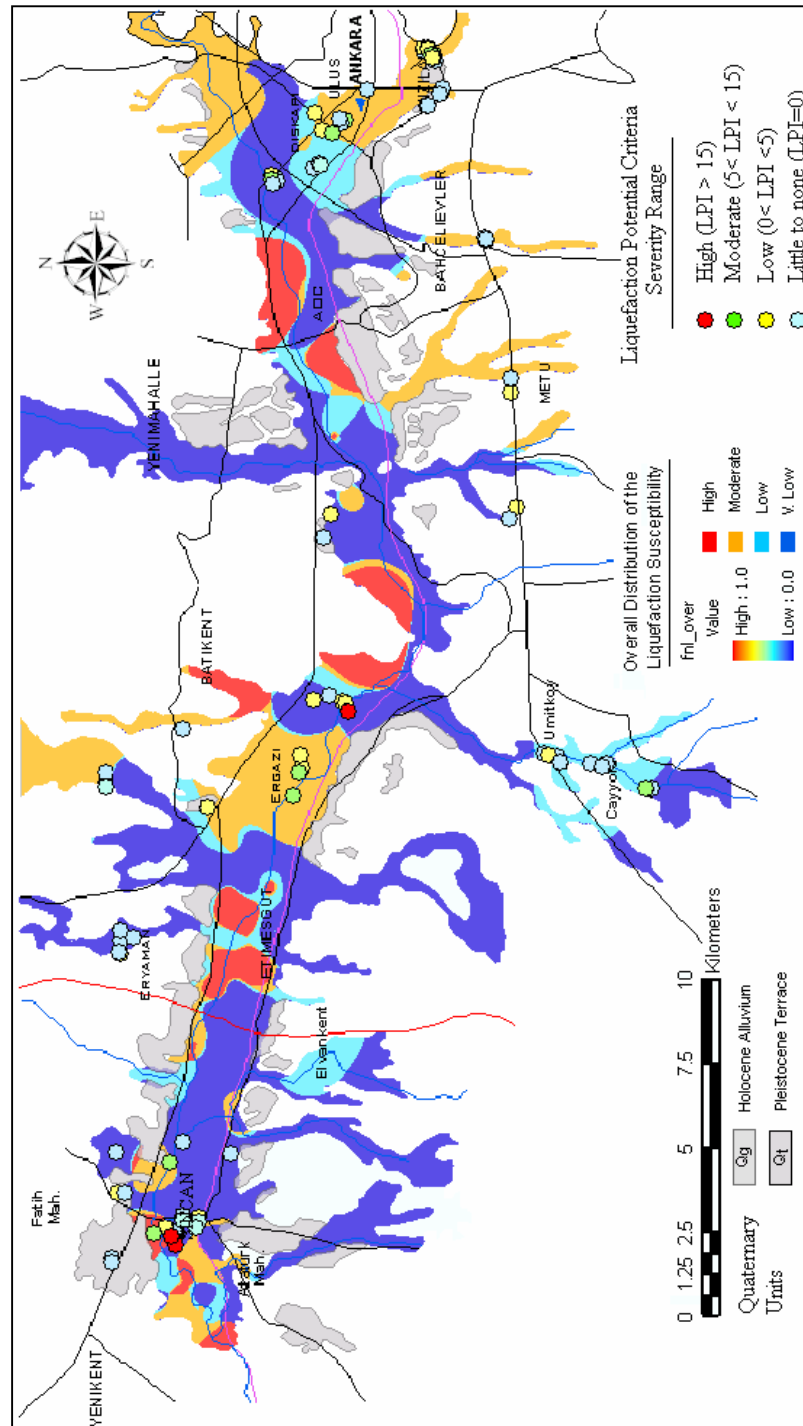


Figure 6.28. Liquefaction hazard map that combines the liquefaction susceptibility and liquefaction potential results to generate a single coverage map for a $M = 7.5$ earthquake in Quaternary deposits of the Ankara basin



CHAPTER 7

SITE EFFECTS IN SEISMIC HAZARD ASSESSMENT

7.1. Introduction

The effect of local site conditions has been directly related to significant damage and loss of life. During the past decades, the effect of local soil conditions is known to have caused serious damage during several earthquakes. Some well known examples include the earthquake in Michoacan, Mexico in 1985 (Celebi et al., 1987), Loma Prieta, CA in 1989 (Hough et al., 1990), Northridge, CA in 1994 (EERI, 1994), Kobe, Japan in 1995 (EERI, 1995), Kocaeli and Düzce, Turkey in 1999 (Rathje et al., 2000 and 2003) and Chi-Chi, Taiwan in 1999 (Rathje, 2004 and Rathje et al., 2005). While there are other potential factors contributing to damage (such as topography, basin effects, liquefaction, structural deficiencies, etc.), the amplification of ground motion due to local site conditions (such as ground motion resonance and amplification, etc.) plays an important role in increasing seismic damage. These observations, as well as numerous others, indicate that quantification of site effects is a necessary component of a comprehensive assessment of seismic hazard (Rodriguez-Marek, 2001).

Even though Ankara may be considered safe in terms of earthquake hazards owing to its considerable distance to major fault systems (i.e., approximately 75-100 km), the damage that occurred for example through the Michoacan Earthquake (1985) that was located more than 350 km from Mexico City but caused extensive damage due to a large impedance contrast between the very soft material and underlying bedrock amplified shaking at the ground surface should not be disregarded. Further, the Loma Prieta Earthquake (1989) which

caused widespread secondary failures such as liquefaction, lateral spreading, and slope and bearing capacity failures throughout San Francisco of which, the epicenter was located more than 80 km south of the city; and even the Kocaeli Earthquake (1999) which resulted in significant damage to the Avcılar area that is located about 90 km away from the earthquake source in Gölçük-İzmit should not be disregarded. Hence, the impact of local site conditions subsequent to the significant influence of strong ground motions on site amplification carries an important weight in earthquake geotechnics. In terms of understanding the mechanism of site amplification, a so-called “long-distance effect” problem that is related to the local site conditions of the project site creates a site amplification concept that is not yet well understood. Due to these reasons, even though Ankara may be considered to be situated distant to major fault systems, the influence of the Upper Quaternary sediments under earthquake triggered motions need to be investigated.

According to the concerns listed above, an understanding of site amplification effects due to sedimentary deposits near the earth surface has been the main goal of this dissertation. There has been a pursuit for increasing knowledge regarding the parameters and mechanisms responsible for the widespread destruction caused by this phenomenon. Some of this destruction might have been avoided if more information on the resonant frequencies of the ground had been available. A better understanding of these resonant frequencies (or periods) and their relationship to buildings and the infrastructure will improve earthquake engineering related structural design in Ankara.

In addition, areas of high seismicity, such as the Marmara region, present opportunities for determining amplification and resonant frequencies through analysis of strong motion data. The frequency of high magnitude events and the vast array of seismic instruments in operation provide abundant data for obtaining these site factors. However, in Central Anatolia such as the Ankara region, seismic activity is less frequent and stations are widely spaced or almost absent mentioned in Chapter 3. Therefore, other methods must be employed to estimate the local site conditions of amplitudes and resonance. The use of ambient noise

measurements to obtain ground motion response has proven successful in several studies in constraining resonance using the Sediment-to-Bedrock Spectral Ratio (Reference Site) technique (Field et al., 1990; Hough et al., 1992; Kagami et al., 1982 and 1986). However, in some cases the Horizontal-to-Vertical Component (H/V) spectral ratio of the Nakamura technique has proven to be a more reliable method that produces more correlative results (Nakamura, 1989; Chavez-Garcia, 1994; Field and Jacob, 1995; Field et al., 1995). Hence, it appears that H/V techniques are useful in seismic zonation or microzonation, but a thorough understanding of subsurface heterogeneities and non-linear effects at the study site is essential to a reliable interpretation of the data (Finn, 1991 and Bour et al., 1998).

Since the majority of Ankara constitutes Upper Pliocene to Pleistocene fluvial and particularly Quaternary alluvial and terrace deposits that are formed in and near fault-bounded depressions, microtremor measurement studies in order to predict earthquake response present a high priority for the Ankara area. The city has been growing towards the potential settlement areas consisting of these basin fill type of sediments where buildings and infrastructure have been built upon. Since the Ankara region has experienced destructive earthquakes in the past from the surrounding large-scale Fault Systems and Fault Zones that might have affected the soil conditions of these younger sediments, it is seismically active and serious damages might occur during future earthquakes. Due to the basin fill type of Quaternary sediments which cover a large area in the Ankara basin, it is necessary to assess potential hazards associated with ground motion amplification and resonance. The purpose of this study is to determine the viability of using ambient noise measurements to estimate the earthquake response of Ankara.

7.2. Site Effects

Local site conditions such as deep, soft sediments strongly affect the amplitude of ground motions. Ground motion amplification can cause subsequent

ground failures due to excessive ground shaking. The identification of soil deposits susceptible to ground motion amplification is required for accurate risk assessment and loss estimation. Ground shaking is affected by subsurface geology. Previous studies have identified Quaternary deposits, particularly those from the Holocene sediments, as susceptible to ground motion amplification due to the loose, unconsolidated nature of deposition (Tinsley and Fumal, 1985).

Soil resonance is the term used to describe periodic amplification of energy detected at a receiver on the ground surface. There are several factors involved which cause this periodic amplification. Seismic energy can become trapped in a soil layer above a high impedance boundary and reflect within the layer at a period of twice the travel time of the wavetrain. At the same time, some of the energy which is transmitted across the high impedance boundary comes back up and constructively or destructively interferes with the energy in the soil layer (Finn, 1991). Heterogeneities also scatter the waves, resulting in more interference. All of these factors contribute to a particular soil resonance. Many different frequencies may be amplified depending on the heterogeneity of the soil layer (Kanai and Tanaka, 1960). However, layer thickness controls which frequencies will be amplified.

The mechanism that contributes to amplification considers the effect of resonances within the soil column that occur when the frequency of seismic wave energy is equal to the natural frequency of the deposit. A simple estimate of the natural frequency or period of a geologic deposit (f_n or $1/T_n$) is given by

$$1/T_n \text{ or } f_n = \frac{V_s}{4H} \quad (7.1)$$

where V_s is the shear wave velocity and H is the thickness of the deposit (depth to strong, competent material with $V_s > 750$ m/s). If the frequency of the seismic wave is approximately equal to the natural frequency of the deposit, amplification will occur increasing the amplitude of ground motion significantly at the natural frequency. It is related to the quarter wavelength rule in that $4H$ is an estimate of the longest wavelength that will resolve the upper layer stated in Chapter 5. The

first takes the velocity over the depth range corresponding to one-quarter wavelength of the period of interest (Joyner et al., 1981), which produces frequency-dependent values. The intensity of amplification is based on the impedance contrast between the two geologic deposits. The impedance ratio (I) is defined as:

$$I = \frac{r_r V_r}{r_s V_s} \quad (7.2)$$

where ρ is the mass density of the deposit, and the subscripts r and s refer to the rock and soil, respectively. Amplification increases as the impedance ratio between two layers increases.

Amplification is affected by changes in velocity and frequency of the wave. Neglecting attenuative effects, energy conservation states that, for a harmonic wave, $E = 1/2\rho\omega^2 A^2$ (Aki and Richards, 1980) where E is energy density, ρ is soil density, ω is angular frequency, and A is the amplitude of the wave. If $\omega=2\pi f=2\pi V/\lambda$, where f is frequency and λ is wavelength, then the total energy density may be stated as:

$$E = 2\pi\rho f^2 A^2 = 2\pi\rho (V^2/\lambda^2) A^2 \quad (7.3)$$

Therefore, if the energy is kept constant, a low velocity results in high amplification which is the case in a low velocity soil layer. Both the energy reverberating in the soil and the energy from the bedrock increase in amplitude as they travel up towards the surface through low velocity material. The energy density equation also shows period (frequency) dependence. Relatively higher periods (lower frequencies) result in high amplification for constant energy. Drake (1980) investigated amplitude dependence on the frequency content of Rayleigh and Love waves using a finite element model of horizontally propagating surface waves through an alluvial valley. His results showed that at periods between 1.5

and 2 sec and between 0.375 and 0.75 sec, the horizontal component of ground motion was amplified more than the vertical component. However, at 1 sec the vertical component had a higher relative amplification. Table 7.1 gives undamped surface amplification results from his model. It can be seen that there is not a definite linear relationship between frequency and amplitude as is suggested by the energy density equation. Drake also claims that the shape of the ellipse and the direction of motion of the surface particles change gradually with the period of the fundamental Rayleigh mode. This study suggests that the frequency-amplitude relationship is not straightforward. Inhomogeneities in the ground can cause a mode of transfer of energy (Drake, 1980), resulting in amplification at some frequencies and very little or no amplification at others.

Table 7.1. Computer-modelled undamped surface amplification due to horizontally propagating Love and Rayleigh waves through an alluvial valley (after Drake, 1980)

Undamped Surface Amplification						
Period (sec)	2	1.5	1	0.75	0.5	0.375
Love, horizontal	23.53	10.07	5.68	8.75	6.88	18.26
Rayleigh, horizontal	40.04	14.02	7.75	22.97	11.11	17.24
Rayleigh, vertical	3.43	4.21	8.32	10.57	6.14	5.17

Regardless of inhomogeneities in the earth, some assumptions that can be made in order to estimate the resonant frequency of the soil through microtremor measurements were discussed in Section 4.3.2.4.2 in Chapter 4. In most cases resonant frequencies cannot be read directly from noise spectra that might create problem. Rather, the spectra are superimposed by the influences of the source effect, namely, the frequency content and the amplitudes of the microtremor sources. There are particularly two assumptions that use spectral ratios of microtremors to overcome this problem that were stated before (Figure 7.1). In the Spectral Ratios Relative to a Reference Site (Sediment-to-Bedrock) technique, the

same source and path affects at the sediment and the bedrock sites. It is not required that the spectrum of the excitation be flat, in contrast, a necessary assumption is that the motion recorded at the reference station is representative of the excitation arriving at the interface substratum/sediments, under the soft soil site (Drake, 1980; Kagami et al. 1982 and 1986; Ibs-von Seth and Wohlenberg, 1999). However, if there is no bedrock site available, or if one of these assumptions is not valid for a particular study, then the Nakamura's method should be used.

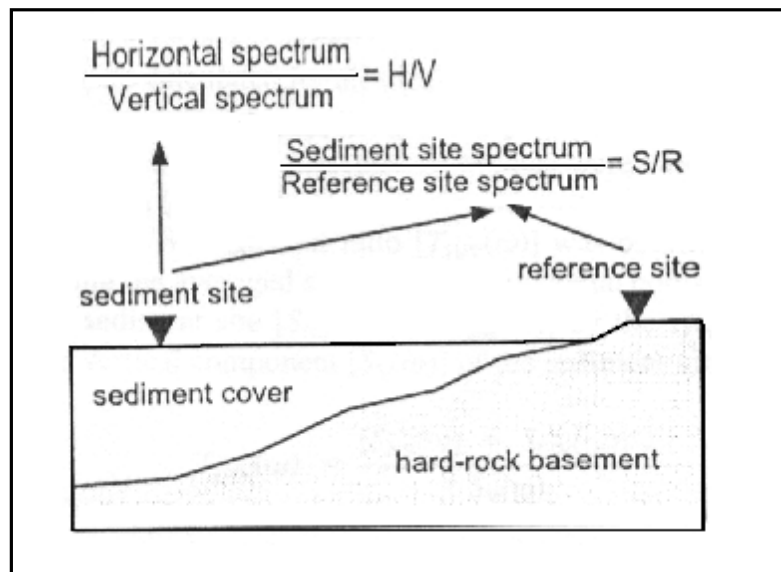


Figure 7.1. Two particular assumptions for site response evaluations using noise spectra

Comprehensive explanations regarding the theoretical background and experimental studies of the Nakamura method (1989) were also discussed in Section 4.3.2.4.2. The horizontal-to-vertical component ratio technique (H/V technique) assumes that microtremors are Rayleigh waves propagating through a single soil layer overlying a half-space of relatively higher velocity than the soil. Thus, there are two components of motion along the free surface and two components of motion along the low-to-high velocity interface (Figure 4.22). This

method is based on the premise that the horizontal component of Rayleigh waves is amplified by the soil at the surface, while amplification of the vertical component is negligible. It has already been recognized that this happens only at certain frequencies, but the major consensus seems to be that in general, it is the horizontal component of motion which is amplified the most at the surface (Akamatsu, 1984; Nakamura, 1989; Lermo and Chavez-Garcia, 1994; Lachet and Bard, 1994). Another assumption is that the noise source is due to local cultural disturbances; therefore, deep sources can be neglected. This implies that local sources will not affect the microtremor motion at the base of the soil layer. Many theoretical and experimental studies have shown that the spectral ratio obtained in this manner enables an adequate determination of amplitudes and the site fundamental frequency. However, the Nakamura method does not seem to be able to provide all the information required for a reliable estimation of the amplification of surface ground motion. One should not forget that this technique is based on various assumptions, as yet not totally verified, concerning the nature of the incident background noise (Bour et al. 1998).

Consequently, the estimate of the fundamental resonance frequency of soil is easier on the H/V ratio than on absolute spectra or site to reference spectral ratios, especially with the improvements allowed by the phase processing for the determination of frequency where the horizontal and vertical noise spectra is the minimum in sites with low impedance contrast used extensively for urban mapping, especially in the short period range and in some cases as an engineering geological and geotechnical tool. Therefore, it is well adapted in the urban environments to be of great interest for site effects and to determine the fundamental frequency and establish the seismic zonation. In other words, microtremor measurements are very useful in seismic hazard assessment, as they can be used to obtain both the dominant period of motion at a soft soil site with good reliability and a rough estimate of amplification level if geological conditions are relatively simple.

7.3. Experimental Study and Field Works

Short-period noise recordings of mobile microtremor measurements were carried out at 352 project site locations on the Upper Pliocene to Pleistocene Fluvial and Quaternary Alluvial sediments to study the seismic response in the western part of the Ankara basin (Figure 4.3). Of the 352 measured data points, about 207 (58.81%) and 132 (37.50%) of the data fell within the units of the Quaternary alluvial and terrace sediments and Upper Pliocene to Pleistocene fluvial sediments, respectively. Additionally, 13 (3.69%) of the data was taken from rock sites in surrounding environment of the Ankara basin and used as reference stations for forming of Reference Site (Sediment-to-Bedrock, S/R) Spectra. Briefly, these reference locations of the rock units are namely Upper Jurassic-Lower Cretaceous limestone, and also Upper Miocene-Lower Pliocene volcanics and fluvial-lacustrine sedimentary rocks, respectively. The field measurements were conducted by adopting a grid system and the microtremor recordings were attempted to be spaced approximately 500 m depending on the suitability of the particular location in the study area. There were basically two reasons behind this approach: (1) to utilize these measurements so that the compiled and performed site investigation (i.e., boring, shear wave velocity, etc.) measurement locations were as close as possible for sound correlation of data (i.e., in order to obtain a more comprehensive and reliable information on the soil profile and hence to check on the validity of the experimental results); and, (2) to eliminate the effects of different distances among measurement points during the preparation of seismic zonation map for the hazard assessment. The information regarding the mobile measurement points that were taken during the field study on the “microtremor record cards” were given in Appendix B.

In the experimental field measurements, the digital equipment utilized comprised a DATAMARK LS-8000 WD type measuring instrument as a data recorder and an Akashi JEP-6A3, three component, built-in acceleration seismometer. The seismometer incorporates three overdamping type acceleration sensors (two horizontal sensors and one vertical sensor). The frequency response

of the acceleration seismometer is given in Figure 7.2 (characteristic frequency is 3 ± 0.5 Hz.). During the recording of the measurements, the sampling interval rate was 100 Hz and the duration of each sample recording was 180 s. For further processing, only quiet sections of the noise recording were used. Therefore, all recordings were examined to identify and trim-off those parts of the traces disturbed by very near-local sources, such as traffic, permanent vibrations or footsteps. Then, 20 s quiet time interval (particularly noiseless) was chosen to process microtremor data in order to filter the noise. Amplitude spectra were calculated for all sites and components. During processing of the data for each of the measurement points that were recorded in the field study, a Fast Fourier Transform (FTT) procedure was applied on each window, and then the obtained Fourier spectrum were smoothened with a Parzen window by applying appropriate low and high pass filters and band width. Finally, the Fast Fourier spectrum for three components of the ambient noise, namely two horizontal components of E-W and N-S and vertical component of U-D were developed to calculate the final H/V and S/R spectrum.

During the evaluation of the ambient noise records, the recording files have been processed and analyzed by using SPECTRATIO (Version 5.3, Rosset, 2002) that runs under the Matlab programming software (Rosset, 2005) and MicPlot (Version 1.1, Motoki, 2002) which runs under the UNIX (Mirzaoglu, 2005). SPECTRATIO (Rosset, 2002) is a quite useful Matlab programming tool that has been particularly used in this research study to process and analyze the ambient noise records. Because its modular environment allows modifying and adapting the analysis to various types of instrument and field environments that might lead to adapt the processing methods to his the particular study. Hence, this programming tool was modified in accordance to the aim of this research methodology. Briefly, it is composed of a set of codes written with Matlab language with a main file named “spcratio.m” and the related sub-routines presented in Appendix C. The organigram is also presented in this Appendix which explains the functionalities of each module.

To validate the experimental results, H/V and S/R spectral ratios were analyzed with both programming softwares. The processed results of the horizontal and vertical acceleration spectrum (Fourier spectra), and H/V and S/R spectral ratios were developed.

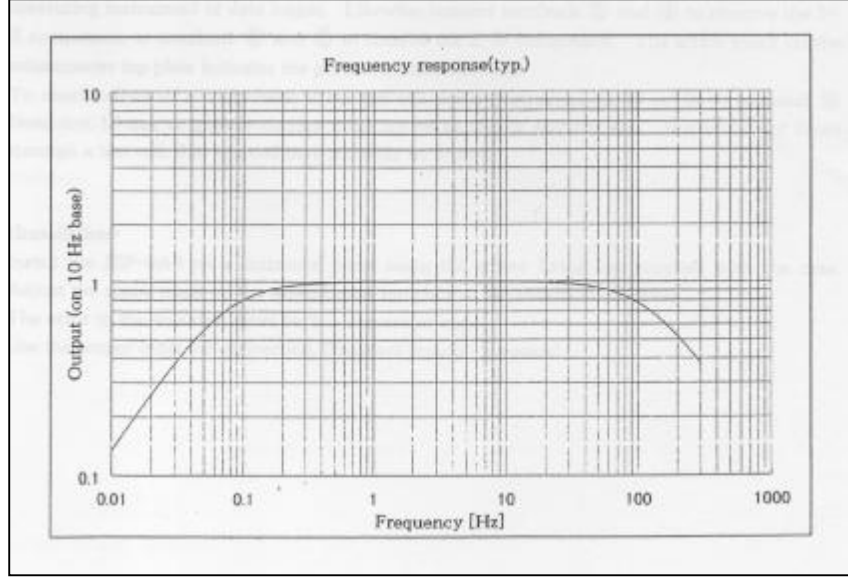


Figure 7.2. The frequency response of the Akashi JEP-6A3 type acceleration seismometer (characteristic frequency is 3 ± 0.5 Hz.).

Considering Eq. (4.23), the H/V spectral ratios $[S_{H/V}(w)]$ were obtained by dividing the resultant spectra of the horizontal components of the sediment site $[S_{NS}(w)$ and $S_{EW}(w)]$ by the spectrum of the vertical component $[S_V(w)]$ of the sediment site:

$$S_{H/V}(w) = \frac{\sqrt{[S_{NS}(w)]^2 + [S_{EW}(w)]^2}}{S_V(w)} \quad (7.4)$$

The S/R spectral ratio $[S_{S/R}(w)]$ was formed by dividing the spectra of the resultant horizontal components of the sediment site by an averaged spectrum of

the averaged horizontal component spectra (R_{NS} and R_{EW}) of the 10 rock (reference) sites [$R_H(w)$] as stated by Ibs-von Seth and Wohlenberg (1999):

$$S_{S/R}(w) = \frac{\sqrt{(S_{NS}(w)^2 + S_{EW}(w)^2)}}{R_H(w)} \quad (7.5)$$

where

$$R_H(w) = \left[\sum_{i=1}^{10} R_{NS}(w)(i) + \sum_{i=1}^{10} R_{EW}(w)(i) \right] / 20 \quad (7.6)$$

7.4. Results and Discussion of the Results on Site Effects

The processed FFT results of both the horizontal and vertical acceleration spectrum (Fourier spectra), and H/V spectral ratio spectrum according to the Nakamura technique and spectral ratios relative to a firm site reference station (S/R ratio) were developed to check the accuracy and hence the reliability of the results and to validate the experimental techniques.

In general, relatively more competent Upper Pliocene to Pleistocene Fluvial sediments and particularly competent rock sites possess a relatively flat response curve, while alluvial soft soil sites generally exhibit a peak of maximum amplitude defining their fundamental frequency or period. In particular, the boundary between component material units and soft sediments can be clearly inferred from the change in the spectral shape of records for various sites. Hence, a conclusion might be obtained from the processed Fourier spectrum results, where the conducted microtremor measurements indicate that a impedance contrast exist between the Quaternary Alluvial sediments (particularly Holocene age deposits), and more competent Upper Pliocene to Pleistocene Fluvial sediments and rock units.

The variation of the H/V peak period may not reflect the geologic profile in such way that the larger the H/V peak period, the thicker the alluvial deposits.

These outcomes were also proved with the shear wave velocity measurements that were conducted on these particular sites. Although the relation may not be necessarily 1 to 1, there is a trend in which the higher the H/V value the higher the amplification factor becomes. Based on the analyses of microtremor data, Lermo and Chavez-Garcia (1993 and 1994) mentioned similar results. If the microtremor H/V spectrum of a site has neither distinct maximum nor minimum, the impedance ratio of the site is low (i.e., soil site with V_s gradually increasing with depth as the Fluvial sediments or rock site). The natural site period of the site may not be determined from microtremor measurements made with only one measurement point (mobile stations) that were also mentioned by other researchers (i.e., Lachet and Bard, 1995).

There is still some discussion to determine the fundamental frequency of soft soil sites as stated extensively in Chapter 4, where it could be concluded that the natural period of the site could be equal to or slightly larger than (10-20 % percent points) the period of the H/V peak; or approximately equal to twice the period of the H/V minimum (Tokimatsu, 1997). The natural period estimated from above may not always correspond to the fundamental site period, but rather reflect the second one or the depth of the interface between layers with the high impedance ratio. Hence, during the determination of the natural periods, this methodology has been used in conjunction with available geological information or with other geotechnical and geophysical data that were obtained from the research site.

The figures given below present examples of calculated S/R spectra as compared the H/V spectra for various sites. The plots show clear similarities between the different types of spectra. The spectral ratios always have dominant maxima (similar peak frequencies) at higher periods for both spectral ratios of acceleration spectrums (Figure 7.3). Although, relatively small differences along the spectral shapes and amplitudes exist, these figures reveal good overall patterns and agreements between the transfer functions obtained by the two methodologies. Furthermore, a general decay of spectral ratios towards low periods (high frequencies) is observed. In the research area, however, some of the

mobile stations, for instance MOB 55 and 206; and MOB 273 and 342 are exceptions. In case of these mobile stations, neither the period nor the amplitude of the main peak of the S/R spectral ratios showed similarities with the H/V spectral ratios (Figure 7.4). The stations MOB 55 and 206 were situated in the vicinity of a high-density traffic highway, where high-noise amplitudes have spoiled the shape of the S/R spectrum (see “microtremor record cards” given in the Appendix B; i.e., high frequencies in the microtremors in the city center were detected as compared to the relatively quieter reference sites outside the city). The lack of hard-rock sites in or around the research area presented obstacles in using the S/R method around the Ankara basin. MOB 273 and 342 are sites where the alluvial sedimentary unit thickness is relatively high, which appears to be the reason why the information reflected through the S/R spectrum is low. In contrast, these factors seem to have little or no effect on the H/V spectra. The H/V peak period increases with increased amplitude accelerations due to non-linearities in the behavior of the loose alluvial soils. When the thicknesses of the surficial soil deposits decrease, the amplitudes obtained by the two methods become similar. These results were also proved by Field and Jacob (1993) and Bour et al. (1998). However, it is interesting to note that at few locations (i.e., MOB-345), there might have been some ground motion resonance possibly in earthquake shaking even though the surficial soils comprise relatively small layer of sediment over the competent material of bedrock probably due to the sharp impedance contrast between these two layers related with the non-uniform configurations of surface topography (Figure 7.5). The mobile microtremor measurement point MOB-345 was conducted on the side of a very narrow river valley that comprises a thin layer of loose alluvial sediment over the fractured limestone bedrock unit. It should be emphasized that patterns of surface motion may be influenced in a major way by surface topography and by non-uniform subsurface configurations (Boore, 1972). Particularly, seismic waves are amplified at ridge crests or more generally on convex features such as cliffs (Aki, 1988, Finn, 1991).

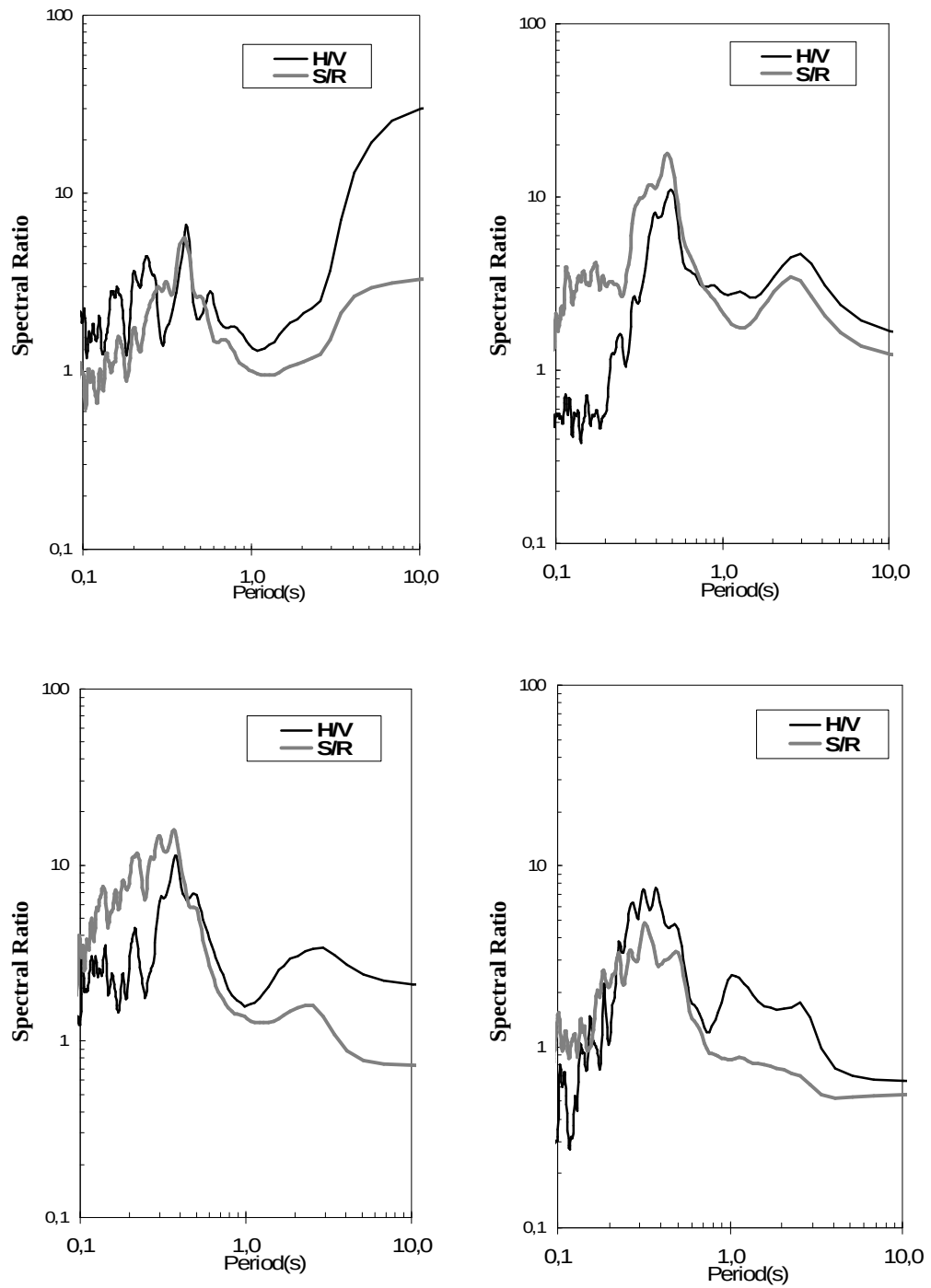


Figure 7.3. Some examples of calculated S/R spectra as compared with H/V spectra at four different sites that show clear similarities between the different types of spectra at mobile stations MOB 36, 79, 226 and 353, respectively.

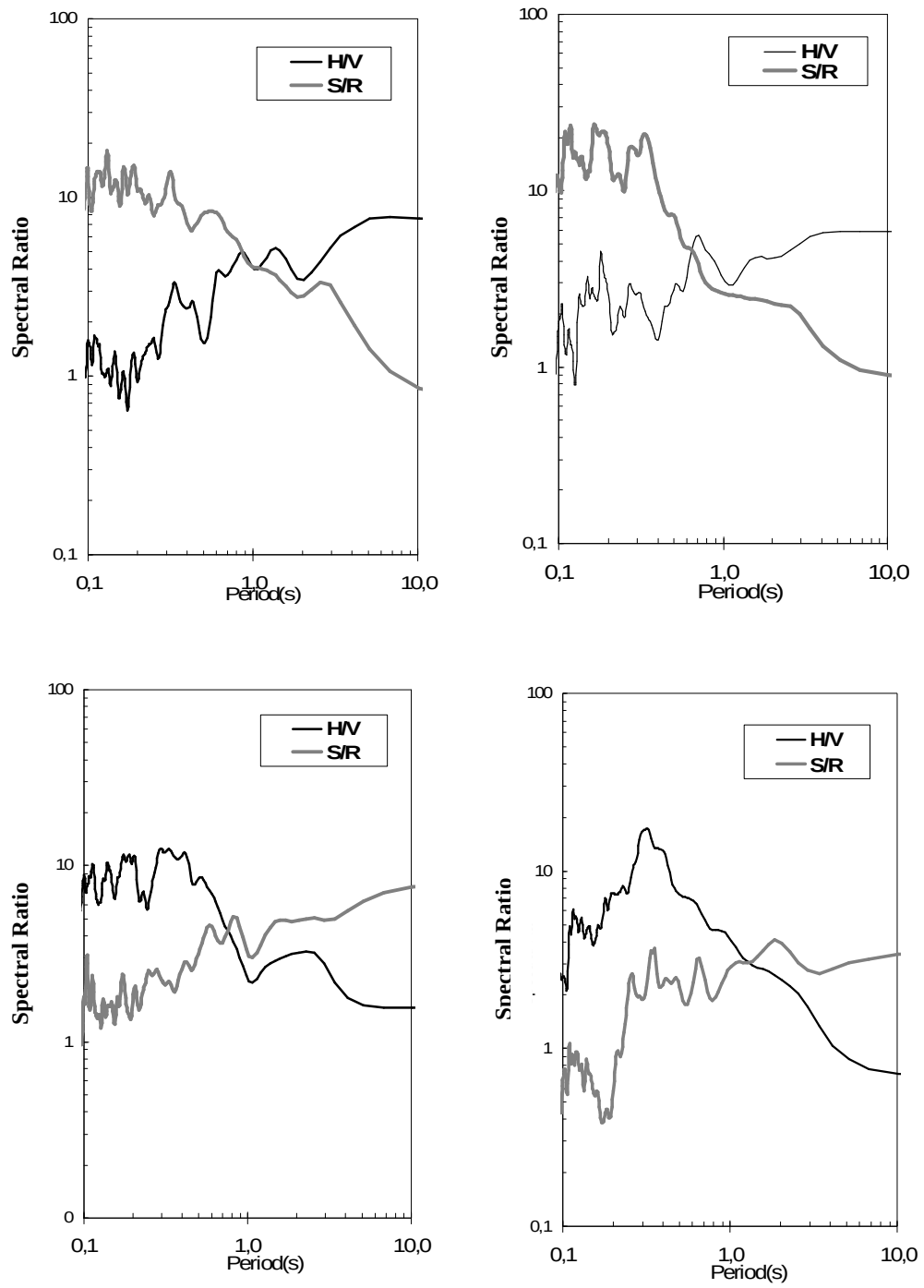


Figure 7.4. Some examples of calculated S/R spectra as compared with H/V spectra at four different sites that show dissimilarities in the period and in the amplitude of the main peak of the S/R spectral ratios at mobile stations MOB 55, 206, 273 and 342, respectively.

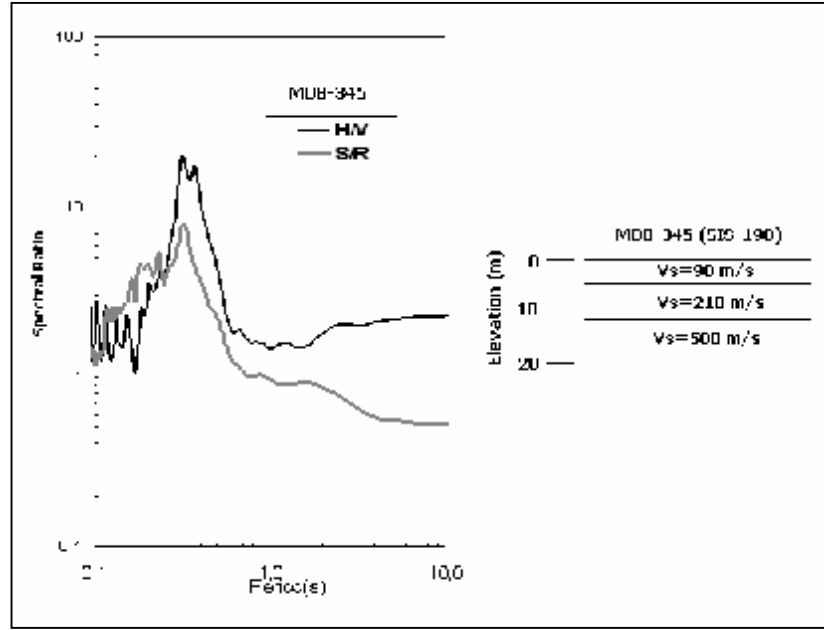


Figure 7.5. The sharp impedance contrast between the thin layer of loose alluvial sediments and the fractured limestone bedrock unit in relation to the non-uniform configuration of the surface topography (MOB-345).

As a consequence of these results, it needs to be noted that the frequencies of the main peaks in the S/R spectra are being influenced by local noise sources in the urban regions of the Ankara basin, where they are not controlled by the thickness of the cover layer. The observed sensitivity of the S/R spectra to local noise sources can be explained as follows. As the thicknesses measured on the base are relatively small, the frequencies obtained are too high. This corresponds to the fact that smaller distances to noise sources cause increasing noise amplitudes, while the portion of high frequencies in the signal spectrum grows. As a consequence, high-frequency peaks occur. The spectrum provided by the reference site is not normally influenced by the same local noise source. Therefore, the high-frequency part of the spectrum is not eliminated during the formation of the S/R spectral ratio giving rise to the high-frequency main peaks in the S/R spectra. Unfortunately, reference sites situated at relatively quiet locations without local noise sources in order to compare the noise amplitude of the

sediments particularly in urban studies that might adversely affect some of the S/R results that have to be considered during the determination of the fundamental frequencies.

In the case of the H/V spectra, both numerator and denominator spectra are taken at the same site. It is thus at least possible that influences of local noise sources are eliminated when forming the spectral ratio. The result for MOB 55 and 206 in Figure 7.4 indicate that the noise amplitudes do not significantly affect the reliability of cover thickness values calculated from H/V main frequencies. These observations strongly suggest that the H/V ratio actually is not affected by the presence of local noise sources.

On the basis of these results, it would appear that it is possible to use the H/V spectral ratio to determine the fundamental resonant frequency and establish the seismic microzonation in terms of a predominant period map of the Ankara basin. Even though it is generally accepted that H/V ratios obtained from microtremor measurements would at times lead to very reliable spectral amplification values as discussed in Section 3.3.2.4.2, they can be well adapted to the urban environments to be of great value for site effects and thus can be taken into consideration when finalizing the seismic zonation assessment with respect to site amplification.

7.5. Site Effect Assessment in the Ankara Basin for Seismic Zonation

In the case study of the Ankara basin, different lithologies associated with typical amplification factors have been identified and surveyed using the field approach. Preliminary results show a good correlation between thickness and/or the type of soft soil and the fundamental frequency obtained with the H/V method.

The results obtained from the microtremor study were utilized to map the variation of spectral amplifications for the Ankara basin as presented below. Therefore, each microtremor measurement point provides a spectral ratio and enables an estimation of the fundamental period and the maximum value of the

amplification at the site studied. Through performing spatial interpolation between these points, one can deduce a map of resonance periods over the Ankara basin (Figure 7.6), and a map of the maximum amplifications observed at these fundamental periods (Figure 7.7).

It is important to note the qualitative character of the maximum amplification values. The Nakamura method does not presently enable the level reached by the peak of the H/V spectral ratio to be related to the amplification of a signal at the surface relative to that in the bedrock during a strong tremor. Only the relative amplifications between two measurement points are assumed to be significant (Lachet and Bard, 1994; Bard, 1999).

Regarding the map presented in Figure 7.6, it can be clearly observed that the fundamental periods are consistent with the geological setting of the research site. Regarding the general distribution of the fundamental periods, they are ranging from about 0.1 to 0.9 s that appear to be relatively variable due to the presence of different units ranging from sedimentary units to competent rock in the Ankara basin. The fundamental periods relatively increase to a range from about 0.4 to 0.9 s along the Upper Quaternary deposits of Ankara basin that comprise the flood water plains trending in the east-west direction of the Ankara River and its main tributaries. These ranges of the fundamental periods also appear to be variable within this range for this unconsolidated deposit. The possible reasons for this variability of fundamental periods, which is also highly connected with the variability of the shear wave velocities of the sediments as discussed in Chapter 5, is particularly due to the variability of the material properties including thickness of the alluvial sediment and density. Hence, the fundamental frequencies relatively increase along the Upper Quaternary deposit, which is interpreted as thickening of these low velocity younger sediments at these particular locations (Figure 5.12). Apart from Upper Quaternary deposits, fundamental periods gradually decrease with a range from about 0.4 to 0.1 s along the remaining older sedimentary and competent rock units in relation to the depositional setting and surface topography.

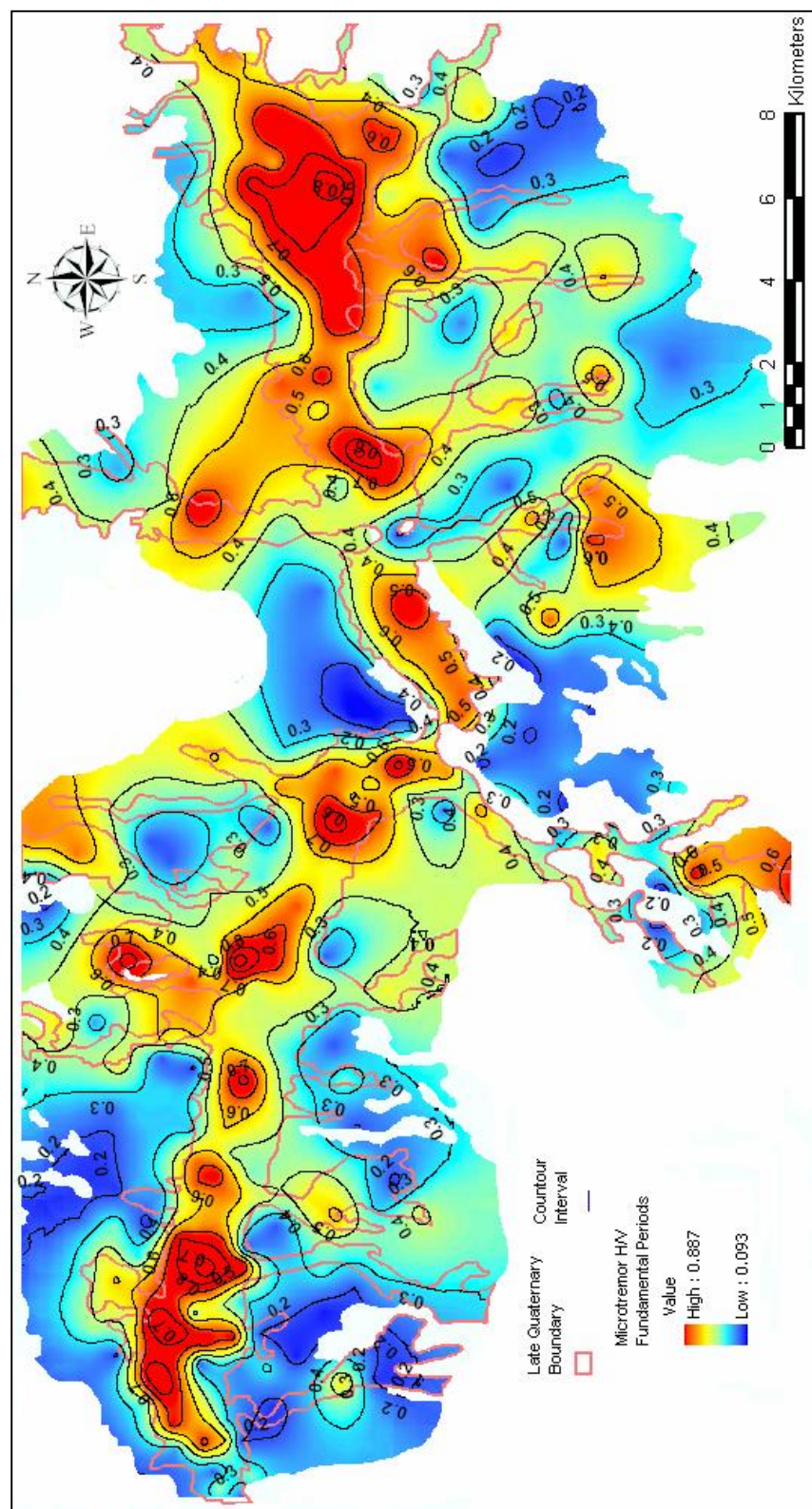


Figure 7.6. Map of fundamental frequencies (resonance periods) obtained with the H/V method over the Quaternary alluvial and Upper Pliocene to Pleistocene Fluvial sediments of the Ankara Basin

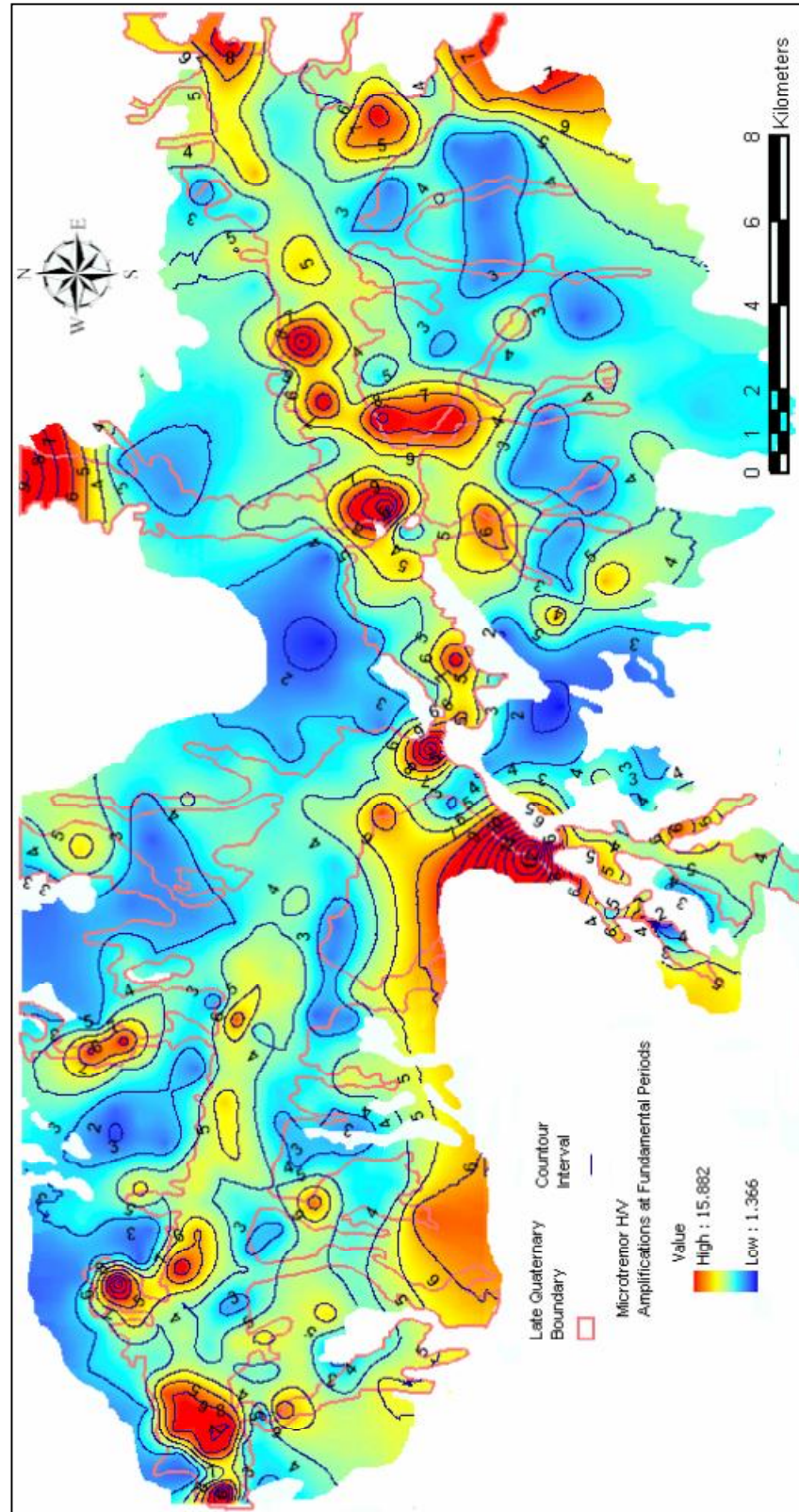


Figure 7.7. Map of amplifications at resonance periods obtained with the H/V method over the Quaternary alluvial and Upper Pliocene to Pleistocene Fluvial sediments of the Ankara Basin

Regarding the map in Figure 7.7, it can be interpreted that the maximum amplifications observed at these fundamental periods are also relatively consistent with the geological setting of the research site in the Ankara basin. Considering the general distribution, relative site amplifications (H/V) are generally ranging from about 2 to 11 [H/V is 16 at only one mobile station (MOB-345) in Upper Quaternary deposit] that appear to be relatively variable due to depositional character and topographical conditions of the geologic environment of the various lithologies situated in the Ankara basin. The presence of the Upper Quaternary sediments amplifies the ground motion larger than that of the surrounding older geologic units generally by two to four folds. This confirms the thickening of the unconsolidated alluvial sediments covering the seismic substratum which appears to increase at some locations in the Upper Quaternary sediments of the Ankara basin. Particularly, the maximum amplifications of these locations are from west to east: western and central part of the settlement area of the Sincan Municipality; some section of the Eryaman housing estate of the Etimesgut Municipality; northern section of the Macun River (Karşıyaka County), main pumping network of EGO and some AOÇ sections of Yenimahalle Municipality; and settlement area of some section in the city center of Ankara. These Upper Quaternary alluvial sites that show significant H/V amplification of ground motion at fundamental periods ($H/V > 6$) are also depicted in Figure 7.8. However, some of the peak amplification sites do not present a clear correlation with thickness of alluvial soils [i.e., location of the amplification sites at the Ergazi small industrial estate (MOB-155) and TAFIX location in the Armour Unity of Land Force (MOB-345) in Etimesgut Municipality Region] due to the sharp impedance contrast between the two geologic deposits regarding the inhomogeneities of surface topography mentioned previously (Figure 7.8).

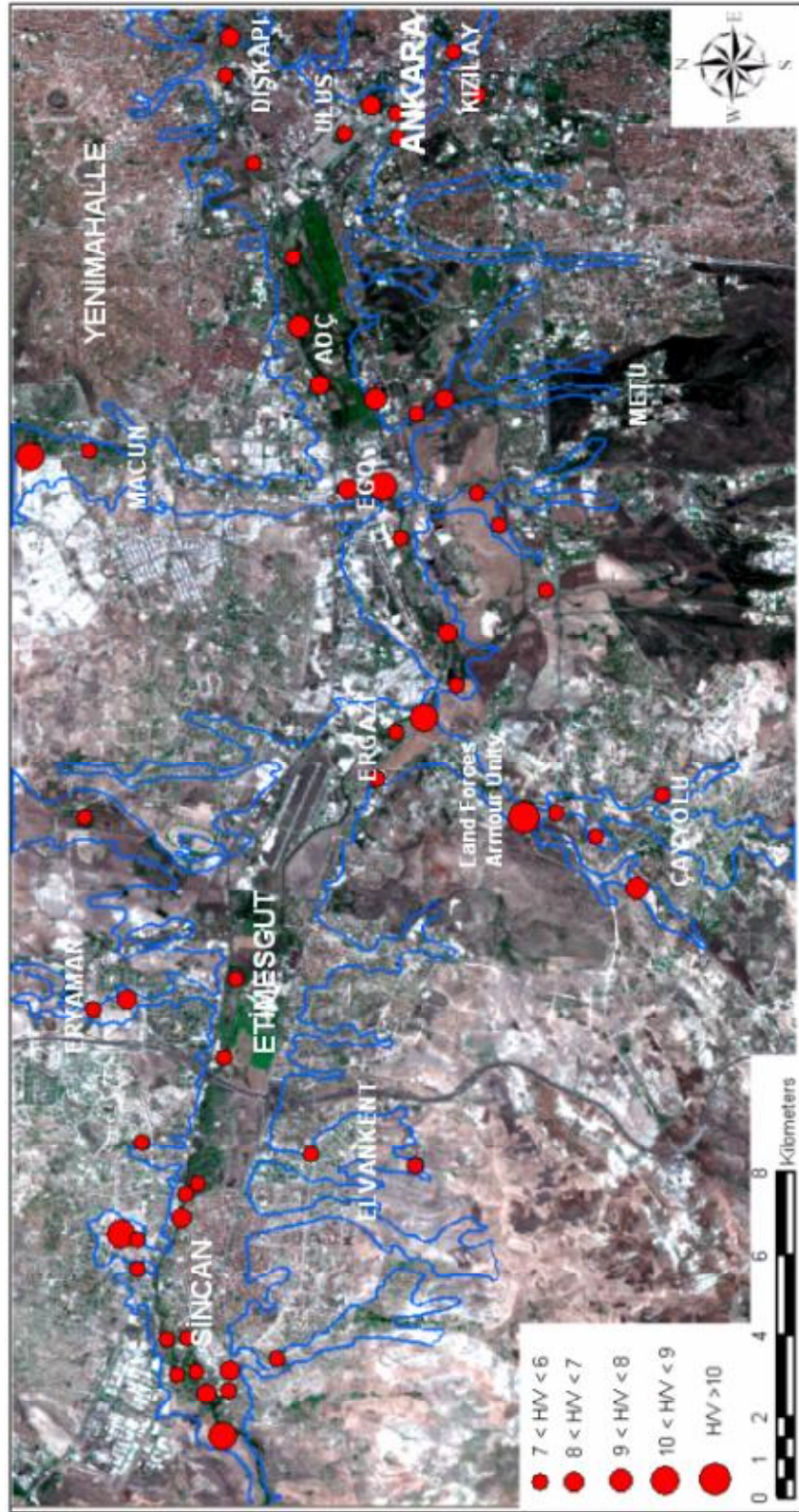


Figure 7.8. Map depicting regions of maximum amplifications ($H/V > 6$) at resonance periods obtained with the H/V method along the Quaternary alluvial sediments of the Ankara Basin

Regarding the comparison of geotechnical seismic data along with the H/V spectral ratios in the Ankara basin, it was observed that the fundamental period map agreed well with the seismic characterization results conducted in the research area. The higher amplification results at fundamental periods were observed along the Upper Quaternary sediments in the studied region which generally correspond to the thicker unconsolidated materials that have low shear wave velocity characteristics as discussed in Section 5.3.1.1.1 (Figure 5.12). Hence these results appear to complement the seismic site characterization studies used in microzonation studies for reliably correlating and determining the local site character.

The comparison of these results indicates that the H/V peak period increases with increased amplitude accelerations due to non-linearities in the behavior of the soils. The amount by which the period shifts at each site depends on the resonant period of the site. A lower peak period, corresponding to shallower bedrock surface (thinning of the sediments), generally results in a greater shift in period than a higher peak period associated with a deeper bedrock surface (thickening of the sediments).

This study has shown that the method of H/V spectral ratios, based on the recording of background noise, might provide reliable data in relation to the seismic behavior of gently dipping alluvial soft soil layers that generally show maximum amplitude defining their resonance period. It is clear that the various experimental approaches should be combined so as to better constrain the microzonation of a given region, in particular those of weak seismic activity and/or high levels of ambient noise.

CHAPTER 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1. Introduction

Earthquake hazard zonation has become an important consideration for city planners and engineers since many governmental agencies attempt to take a pro-active role in emergency response planning. By knowing the type and extent of anticipated damage, the agencies hope to control additional development in damage-prone areas and to promote the retrofiting of structures.

This dissertation presents the development and implementation of site characterization data to assess the engineering geological and geotechnical characteristics in regards to perform seismic hazard and zonation studies of the Upper Pliocene and Quaternary deposits located towards the west of Ankara. Hence, concepts, methods and procedures from various engineering fields that were used in the course of this research were discussed in the preceding chapters of this dissertation. These fields included various seismic hazard evaluations combined with site specific engineering geological, geotechnical and geophysical information to enable the development of seismic zonation studies for summarizing the potential for respective hazards within the specified area.

This chapter summarizes the conclusions of the research undertaken to implement seismic zonation studies. Recommendations for further research are also provided.

8.2. Summary, Conclusions and Recommendations

This dissertation mainly focuses on the development of a methodology to integrate the various components and to identify the soil deposits which is necessary for a regional multi-hazard seismic risk assessment of the Ankara basin that is located towards the west of the city center of Ankara. The study area is an approximately E-W-trending, 25-30 km long and 10-15 km wide structural depression that is drained in the east-west direction through the present-day course of the Ankara River and its major tributaries.

Regarding the scope of this research, the engineering geological properties of the neotectonic units, particularly Upper Pliocene to Pleistocene fluvial and Quaternary alluvial and terrace deposits in the Ankara basin have been investigated to account for seismic hazard assessments. Since these sedimentary units are formed in and near the fault-bounded depression as a result of fault-controlled continental sedimentation in the Ankara basin, the basin fill types of sedimentary units were especially defined and mapped on the basis of geological and genetic criteria such as geomorphic expression, inferred depositional environment and sediment characteristics.

Considering the paleogeographical characteristics, there are different views regarding the sedimentary formations of the Ankara basin during the Neotectonic period whether they are depositional or erosional because the existence of this source of overburden pressure has not been well supported by sound geological evidence or reasoning. However, the geotechnical and seismic site characterization studies that were conducted through this research, particularly on these sediments, conclusively support the view that the Pleistocene terraces are the result of a cut and fill sequence of a relatively softer sequence as compared with the sediments of erosional environments despite sometimes being confused with the older fluvial deposits, especially at higher terrace elevations. According to these studies, it might be concluded that the alluvium deposited by the flood waters has not been in place long enough to show any appreciable effect of soil forming factors (i.e., overburden pressure) as such the recently developed

fluvial sediments that are formed in the Ankara basin due to their depositional characteristics. These comments are very useful in interpreting the sediment characteristics and soil behavior of these fluvial deposits of the Ankara basin in an account for seismic hazard assessments.

In the Ankara Region, the seismic events that are associated with the faults in the Ankara basin are seismically active but rather have short extent ($M < 5$). These faults are particularly the southeastern segment of the İnönü-Eskişehir Fault Zone, Çeltikçi Fault Zone, Çubuk Fault Zone, Balaban Fault Zone, Kesikköprü Fault Zone, Kızılırmak Fault Zone and Kalecik Fault, etc. On a regional scale, the Ankara Region might be affected destructively from the surrounding large-scale fault systems and fault zones, especially the North Anatolian Fault Systems, the Salt Lake Fault Zone and the Seyfe Fault Zone that have capabilities to produce large destructive earthquakes ($M > 6.0$). The sources of historical earthquakes and past recent examples also prove that the significant seismic events have taken place along these fault systems that have seriously affected Ankara and its surroundings.

Considering the areas of seismicity of the Ankara region, seismic activity is less frequent and stations are widely spaced or almost absent. The lack of seismic activity and the scarcity of data for large magnitude events make earthquake hazard estimation in this region uncertain and thus it is difficult to identify the seismic source regionalization. Therefore, is it not only difficult to predict the magnitude of a potential earthquake, but also the lack of earthquake information makes the attenuation of the earthquake motions difficult to characterize. Hence, a deterministic approach was utilized at first for individual earthquake scenarios that were developed for each relevant seismic source and a specified ground motion probability level. By using the estimated magnitude along with the distance relationships, the ground motion was then computed for each earthquake scenario, using ground motion attenuation relationships in respect to the suitability of the attenuation models depending on the characteristics of the faults in the study area. Examining the developed median PGA results with one plus standard error predictions ($PGA + \sigma$), it might be concluded that the

attenuation relationship results are successful in predicting the peak horizontal acceleration values in a given interval where the bedrock peak ground motion for the Ankara region was determined to be 0.2 g based on the maximum seismic source event surrounding the Ankara region. Then, the results obtained from the deterministic studies were compared with a probabilistic seismic hazard analysis map of Turkey that was prepared by Erdik et al. (1996) and Gülkan et al. (1993) for the evaluation of seismic hazards. A comparison of these results with those of this study leads to the conclusion that the attenuation results ($PGA + \sigma$) that were developed for the Ankara region are in close agreement in predicting the iso-horizontal PGA contours. Consequently, the bedrock peak ground motion that will be used to evaluate the local site effects in the regional seismic hazard study for the Ankara region was determined to be 0.2 g resulting from both approaches.

Considering the regional site characterization results of the measurements of $V_s(30)$ and $N(30)$ according to the design code of IBC2003, Quaternary alluvium deposits in the Ankara basin were differentiated into three units, namely younger alluvium of E-Site (Mean $V_s(30)$: 169 ± 13 m/s and Mean $N(30)$: 13 ± 2 blows/0.3m), younger alluvium of D-Site (Mean $V_s(30)$: 217 ± 26 m/s and Mean $N(30)$: 23 ± 4 blows/0.3m) and older terrace deposits (Mean $V_s(30)$: 251 ± 24 m/s and Mean $N(30)$: 31 ± 6 blows/0.3m), respectively, based on the environment of deposition that has a great deal of control over the sediment characteristics and physical properties. Similarly, measurements on the Upper Pliocene to Pleistocene Fluvial Deposits in the Ankara basin led to a decision to classify them as relatively less dense or stiff Fluvial Deposits of D-Site (Mean $V_s(30)$: 319 ± 23 m/s and Mean $N(30)$: 39 ± 6 blows/0.3m) and stiffer or denser Fluvial Deposits of C-Site (Mean $V_s(30)$: 392 ± 27 m/s and Mean $N(30)$: 56 ± 6 blows/0.3m) based on the depositional environment.

The regional seismic zonation maps of $V_s(30)$ including some characteristic indices (i.e., standard penetration test results, geology and soil type) with respect to the site classes specified in the IBC 2003 according to this study based on the actual data, according to Ohta and Goto (1978) and according to Andrus et al. (2001) were developed through improved regression equations to be

used in predicting the site characteristics of the Ankara basin. Hence, the $V_s(30)$ and $N(30)$ data pairs with other characteristic indices were analyzed to derive the regression equations for younger Holocene alluvial and older Upper Pliocene to Pleistocene fluvial sediments which may be presented by the following relationships:

$$V_s = 59.94(N_{60})^{0.428} \text{ for younger alluvial deposits} \quad (8.1)$$

$$V_s = 37.05(N_{60})^{0.572} \text{ for older fluvial deposits.} \quad (8.2)$$

An interpretation of the processed data of resistivity measurements obtained from the research site as well as the previous studies and available complementary studies (i.e., geology, seismic field testing and boring logs) along with an interpretation of the electrical soundings in terms of soil types or lithology suggests the general true resistivity estimated threshold ranges in alluvial sediments in the project site, namely clay to silty clay (~2-15 ohm-m), silty or clayey sand (~15-25 ohm-m), sand (~25-50 ohm-m), sandy gravel to gravel (~31-75 ohm-m) and soft to hard bedrock (~44-130 ohm-m). It should be noted that the ranges of the values shown are those that are commonly encountered but that do not represent extreme values. By using supplementary characterization methods of using P-wave velocity measurements that have been conducted to investigate the relative degree of saturation for unconsolidated sedimentary deposits, the P-wave velocities have been divided into three distinct velocity categories: $V_p < 1000$ m/sec (dry), $1150 \text{ m/sec} \leq V_p \leq 1300 \text{ m/sec}$ (nearly saturated), and $V_p \geq 1500$ m/sec (saturated) for the determination of the relative degree of saturation for sediments along the unsaturated and saturated zones leading to locating the water table.

The interpretation of supplementary characterization results that have been generalized from resistivity of VES and P- wave velocity measurements were used for construction of groundwater depth contour maps to determine the thickness of the saturated zone for younger alluvial sediments that could be satisfactorily used in site evaluation and liquefaction susceptibility studies.

According to these results, shallow groundwater level is encountered throughout the alluvial site. The mean depth to groundwater $\pm$ one standard deviation is 5.6 ± 1.6 m with a depth ranging from 1m to 9 m.

In this research study, a novel methodology for a preliminary evaluation of liquefaction susceptibility of surficial deposits in the Ankara basin was developed and then integrated to create a map of liquefiable sediments on the basis of data from Quaternary geologic mapping, data on shallow groundwater depths, and interpretation of textural composition and sediment characteristics of the subsurface data, which were obtained from the interpretation of supplementary site characterization results. The qualitative degree of susceptibility of the Upper Quaternary geological unit was evaluated which displays the distribution of liquefaction susceptibility hazards due to site effects in the research site for various depths and the overall distribution of combined hazard of liquefaction susceptibility within a 15 m depth. The areas of estimated high liquefaction susceptibility include the western and central sections of the Sincan Municipality; central and eastern sections of the Etimesgut Municipality; the AOC and Varlık County of the Yenimahalle Municipality. It should be recommended that this weighting scheme in regards to hazard integration can also be improved by adding various other physical properties of the near-surface sedimentary data sources to increase the accuracy of the susceptibility hazard map. These liquefaction susceptibility maps should be useful for land use planning in order to properly assess the risk from liquefaction hazards. It should be emphasized that the susceptibility maps based on geologic criteria generally do not provide definitive information for site specific evaluations. Therefore, regarding the available data, the liquefaction susceptibility maps in this research area can also be used as complementary tool for preliminary evaluation of the liquefaction potential in the Upper Quaternary sediments in the western part of the Ankara basin.

Regarding the application of general index criteria for evaluating the actual liquefaction potential at a specified site, each in-situ measurements were independently evaluated for the liquefaction potential where the resulting liquefaction potential index (LPI) value was associated with a specific coordinate

location. Hence, LPI results were depicted as zonation maps of liquefaction potential. For a $M=7.5$ earthquake, the majority of the study area is rated as possessing a low to moderate liquefaction hazard along with high liquefaction hazard. More than 50% and 65% of the area possesses a LPI value greater than 5 (moderate to high) in the SPT boring and V_s results, respectively. This research shows that these sites have liquefiable soil layers ranging from a thickness of 6 to 15 m that extend from below the groundwater level to the ground surface. Note that these results are based on the availability of the database that has been utilized, therefore, it might be recommended that a more refined database will lead to the determination of a more site specific liquefaction potential analysis to increase the accuracy and general distribution of the analysis. It needs to be stressed that throughout the Sincan-Eryaman-Batıkent Third Phase Metro Route and Kızılay-Çayyolu Second Phase Metro Route that both possess low liquefaction potential at some sections lead to considerable liquefaction potential due to presence of heavy urbanization and high rise buildings present along the alluvial plains particularly along the tributaries of the Ankara River in Eryaman, Batıkent and Çayyolu sections. Additionally, the overall liquefaction potential of the study area is significantly reduced when subjected to an $M = 6.0$ event earthquake that appears to promote low liquefaction hazard in the Ankara basin. More than 90% and 55% of the area possesses a LPI value of less than 5 (low to little) in accordance with the SPT boring and V_s results, respectively. The entire study area might be classified as little to none and low hazard except for a small section of moderate to high hazard where the surface wave V_s measurements are to be considered. Generally, areas of high liquefaction potential as related to an $M = 7.5$ event earthquake were estimated as a low liquefaction hazard prone area as related to an $M = 6.0$ event earthquake. The reasons of reduction in the magnitude of the liquefaction potential are the presence of liquefied granular soil layers having thicknesses less than 3 to 5 m. Consequently, a detailed liquefaction potential evaluation for each individual project site may be necessary to be performed for the design of structures in the areas of high and moderate

liquefaction potential since $M = 7.5$ and particularly $M = 6.0$ earthquakes may occur in the foreseeable future.

By comparing the available site-specific data for the evaluation of the liquefaction potential results with the preliminary geological map in the evaluation of the liquefaction susceptibility, a general consistency was observed in the combined liquefaction hazard maps considering the overall distribution of the severity ranges. Particularly, the liquefaction susceptibility map of “moderate” to “low susceptibility zones” was more reliably estimated with the site-specific liquefaction potential zones. The primary products of the liquefaction hazard evaluation studies performed in this research that show liquefaction susceptibility and liquefaction potential of the Ankara basin might be applied in order to properly assess and mitigate the potential risk from future liquefaction hazards in the study area. The application of these evaluation results by using a combined procedure is effective for preliminary evaluations, general land-use planning, and delineation of special study zones where additional site-specific studies may be required before major development is approved. Hence, it is recommended that additional geotechnical field work regarding simultaneous seismic V_s and SPT boring studies be performed in the future particularly in the Municipalities of the research site to enlarge the database and to increase the accuracy and validity of the hazard zoning map presented herein.

Regarding the site effects in seismic hazard assessment, according to the processed Fourier spectrum results, the relatively more competent Upper Pliocene to Pleistocene Fluvial sediments and particularly rock sites have a relatively flat response curve, while alluvial soft soil sites generally exhibit a peak of maximum amplitude defining their fundamental frequency or period. In particular, the boundary between component material units and soft sediments can be clearly inferred from the change in the spectral shape of records for various sites. Hence, the conducted microtremor measurements indicate that an impedance contrast exists between the Quaternary Alluvial sediments (particularly Holocene age deposits), and more competent Upper Pliocene to Pleistocene Fluvial sediments and rock units.

From the seismic zonation studies, the H/V fundamental periods are consistent with the geological setting of the research site. The fundamental periods that range from about 0.4 to 0.9 s relatively increase along the Upper Quaternary deposits of Ankara basin throughout the flood water plains of the Ankara River and its main tributaries. These ranges of the fundamental periods also appear to be variable for this unconsolidated deposit. The possible reasons for the variability of fundamental periods are attributed particularly to the variability of material properties including thickness of the alluvial sediment and density that led to the variability of the shear wave velocities of the sediments.

It can also be concluded that the maximum amplifications observed at these fundamental periods are also relatively consistent with the geological setting of the research site in the Ankara basin. Considering the general distribution, relative site amplifications (H/V) are generally ranging from about 2 to 11 in the entire geologic environment of the Ankara basin. The presence of the Upper Quaternary sediments relatively amplifies the ground motion larger than that in the surrounding older geologic units by a factor relatively ranging from 2 to 4. This confirms the thickening of the unconsolidated alluvial sediments covering the seismic substratum which appears to increase maximum amplification at some locations in the Upper Quaternary sediments throughout the flood water plains of Ankara River and its main tributaries.

The results of this study identified three main factors that influences site response, namely, the age of the near-surface deposits, depth of the sediment thickness in the Upper Quaternary deposits and non-linear soil behavior. Upper Quaternary sediments amplify ground motions at longer periods more than older Upper Pliocene to Pleistocene Fluvial sediments due to the low-velocity deposits in the near-surface. Regarding the comparison of geotechnical seismic data along with the H/V spectral ratios in Ankara basin, variation of the fundamental period map agrees with the maximum value of the amplification as well as with the seismic characterization results. The higher amplification results at fundamental periods are observed along the Upper Quaternary sediments of the studied region which generally corresponds to the thicker unconsolidated materials that have low

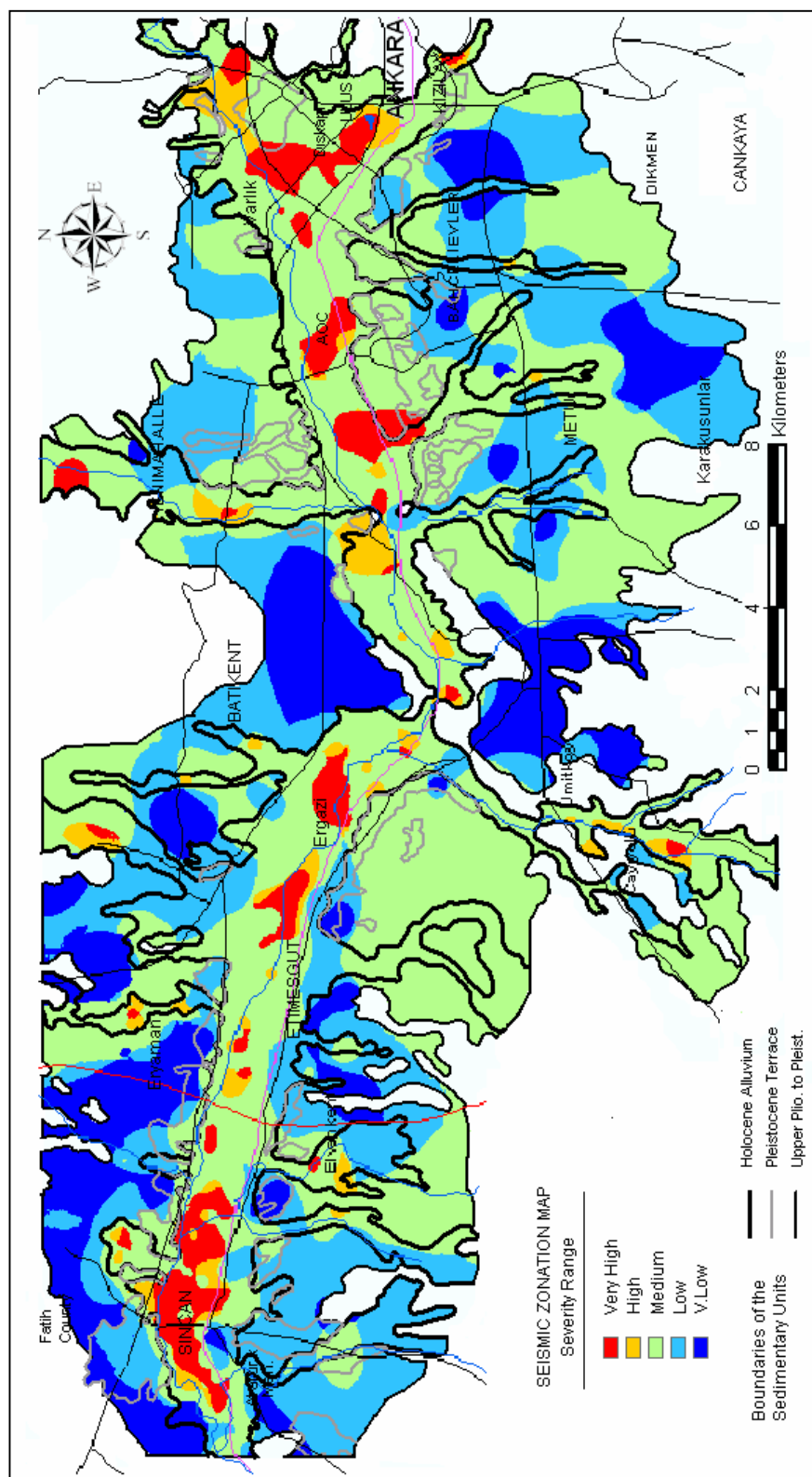
shear wave velocity characteristics on this unit. Hence these results appear to complement and correlate seismic site characterization studies used in microzonation studies for reliably determining the local site character. These results also indicate that the H/V peak period increases with increased amplitude accelerations due to the non-linearities in the behavior of the soils. The amount by which the period shifts at each site depends on the resonant period of the site. A lower peak period, corresponding to shallower bedrock surface, generally results in a greater shift in the period than a higher peak period associated with a deeper bedrock surface.

To validate the experimental technique at few locations in the research site, *H/V* spectral ratios were also tried to be compared with the transfer functions obtained from a one-dimensional numerical simulation model (SHAKE91; Idriss and Sun, 1992) which consists of the response analysis of horizontally layered soils under seismic solicitation, with linear equivalent soil behavior. However, spectral ratios that were calculated by the numerical simulation software require the characterization of the soil profile from the ground surface down to bedrock. Most geotechnical investigations that were obtained from the research area did not penetrate to those depths, particularly on Quaternary alluvial sediments. Hence these studies along with the other studies related to the development of the *V_s* profiles from ground surface to bedrock were studied in a limited extent. It is recommended that future work concentrate on obtaining quality shear wave velocity data at large depths to produce a shear wave velocity profile that adequately models the deep soil column of the Ankara basin.

This study has shown that the method of *H/V* spectral ratios, based on the recording of background noise, might provide reliable data on the seismic behavior of gently dipping alluvial soft soil layers that generally show maximum amplitude defining their resonance period. It is recommended that the various experimental approaches should be combined so as to better constrain the microzonation of a given region, in particular those of weak seismic activity and/or high levels of ambient noise.

Finally, the regional site classification map of the Ankara Basin in regards to the site classes as specified by the IBC 2003 was prepared based on measured and estimated average $V_s(30)$ measurements resulting from this study; the preliminary geological and site-specific data for the evaluation of the liquefaction potential results; and the maps of H/V spectral ratios for the resonance periods and maximum amplifications observed at these fundamental periods over the Ankara basin. All these data have been synthesized and assembled (or aggregated) in each of the hazard assessment studies defined for sedimentary deposits to construct a final seismic zonation map of the estimated seismic hazards for assessing local site characteristics to be used in predicting the site response in the western part of the Ankara basin in order to properly assess and mitigate the potential risk from future seismic hazards in the study area (Figure 8.1).

It is important to stress that these results were based on the extensive research study that were estimated by using the available conducted and collected data based on the site characterizations, liquefaction assessments and evaluation of the site effects studies and this particular seismic zonation procedure could be considered to be one of the best methodologies for assessing the local site conditions that is to be solely used for regional seismic design purposes and not for individual structural design purposes. For example, especially in case of heavy urbanization, high rise and high priority buildings, and industrial facilities, detailed investigation of local soil conditions and determination of site specific design parameters must be obligatory in evaluating the seismic forces controlling the structural response.



The application of these evaluation results by using the integrated seismic zonation methodology is very crucial for Ankara for preliminary evaluations, general land-use planning, emergency response and delineation of special study zones where additional site-specific studies may be required before major development is approved. Hence, the methodology reported through this dissertation will form a starting point as well as a systematic study for engineering geological, geotechnical and seismic characterization studies of similar nature, and then applies to more elaborative and detail studies, particularly microzonation studies in the particular Municipalities that are seismically more critical than the others (i.e., Sincan Municipality, Etimesgut Municipality) and also have growing potential. It can be easily inferred from this integrated seismic zonation map of Ankara along the younger Quaternary sediments, Municipality of the Sincan and Etimesgut, and some portions of the city center of Ankara (i.e., AOC, Varlık, Dıřkapı, Kızılay, Kolej) and some portions along the Ankara River and its tributaries (i.e., Macun, GİMAT, Çayyolu, Eryaman), have to be considered in detail with the additional site-specific studies or microzonation studies. The studies presented herein are very useful for insurers, engineers, planners, and emergency-response personnel in order to properly assess and mitigate the potential risk from future liquefaction hazards in the study area.

Even though the methodology reported in this dissertation is not only important for Ankara but is important for Turkey in its entirety, especially for regions situated in or close to the vicinity of earthquake prone areas (i.e., North Anatolian and East Anatolian Fault Systems), there are a very limited number of studies of such character and detail that have been performed in Turkey. If the detailed geological and geotechnical characterization work presented through this dissertation were to be considered as a “whole”, it seems to represent a “prototype” research work that needs to be applied to a wide range of areas in Turkey, especially those regions situated in or close to the vicinity of earthquake prone areas (i.e., Adapazarı, Düzce, Gerede, etc.) that were mentioned previously. Hence, the methodology reported through this dissertation will form a starting point as well as a systematic study for geological and geotechnical

characterization studies of similar nature. As a sum, it is strongly believed that this dissertation that involves the engineering geological, geotechnical and seismic characterization of the foundation soils situated towards the west of Ankara comprises a novel and scientific study in regards to developing a methodology to protect the safety and welfare of the public along with providing information and technology for future work of similar characteristics.

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APPENDIX A

THE IN-SITU TESTING RESULTS OF THE SPT BORING AND SHEAR WAVE VELOCITY MEASUREMENTS ALONG WITH THE REQUIRED GEOTECHNICAL INFORMATION

Table A.1 The in-situ testing results of the SPT boring measurements (N30 studies) along with the required geotechnical information

Boring No	Depth	USCS	Fines Content %	PI %	N ₆₀	Lithology	WT (m)	g (kN/m ³)
SB SK-2	6.0-6.45	SC	31,00	18	11	Q	4,50	18,00
	7.5-7.95	SC	31,00	18	39	Q		18,00
	10.5-10.95	SM	18,80	0	39	Q		18,00
SB SK-A3	5.5-5.95	SM	8,10	0	10	Q	2,00	18,00
	7.5-7.95	SM	8,10	0	12	Q		18,00
SB SK-A5	1.5-1.95	GM	5,90	0	7	Q	2,00	19,10
	3.0-3.45	GM	5,90	0	11	Q		19,10
	5.0-5.45	SM	6,00	0	24	Q		19,10
	6.5-6.95	SM	32,70	0	25	Q		19,10
SB SK-A9	4.5-4.95	SM	20,20	0	18	Q	4,50	19,10
	6.5-6.95	SM	12,00	0	18	Q		19,10
	7.5-7.95	SM	12,00	0	16	Q		19,10
BH-14	3.30-3.75	GC-SC	15-20	11	23	Q	5,35	20,00
	4.50-4.95	GC-SC	15-20	11	21	Q		20,00
	6.00-6.45	GC-SC	15-20	13	14	Q		20,00
BH-15	4.50-4.95	GC	36,60	17	10	Q	4,75	19,10
	6.00-6.45	SC	28,00	13	20	Q		19,10
SK-22	4.00-4.45	SC	28,00	16	19	Q	4,75	19,00
	9.00-9.45	GC	35,00	21	12	Q		19,00
SK-37	6.00-6.45	SC	30,00	36	12	Q	6,00	19,00
	7.00-7.45	SC	30,00	34	16	Q		19,00
	9.00-9.45	SC	30,00	32	18	Q		19,00
SK-47	2.00-2.45	SC	44,00	22	10	Q	2,35	21,10
	3.00-3.45	SC	31,80	24	8	Q		21,10
SK-69	3.00-3.45	SC	48,00	16	5	Q	2,50	18,40
	7.00-7.45	SC	6,00	0	16	Q		18,80
	8.50-8.95	GM-GP	9,00	0	22	Q		18,80
	10.00-10.45	SC	27,00	21	13	Q		18,80
	11.5-11.95	SC	11,00	24	19	Q		18,80
SK-70	1.00-1.45	SC	21,00	16	27	Q	4,00	20,80
	3.00-3.45	SC	43,60	25	31	Q		20,80
	4.00-4.45	GC	26,00	15	18	Q		20,80
	6.00-6.45	SC	44,00	19	14	Q		20,80
	7.00-7.45	SC	42,00	23	21	Q		20,80
SK-73	4.00-4.45	SC	49,20	19	7	Q	2,00	20,10
	6.00-6.45	SC	43,00	24	7	Q		20,10
	8.50-8.95	GM	13,20	0	10	Q		20,10
	10.00-10.45	GM-GW	3,40	0	25	Q		20,10
	11.50-11.95	GM	21,40	0	27	Q		20,10
SK-75	5.50-5.95	GM	17,40	0	7	Q	5,50	20,10
	6.00-6.45	GM-GP	9,40	0	7	Q		20,10
	8.50-8.95	SC	45,60	24	10	Q		20,10
SK-79	5.50-5.95	GM	10,00	0	9	Q	5,30	19,60
	7.00-7.45	SM	11,00	0	15	Q		19,60
BM-20	3.35-3.80	SC	30,00	12	10	Q	2,80	19,57
	6.30-6.75	GC	9,60	9	14	Q		19,57
	7.50-7.95	SC	35,00	23	7	Q		19,57
S-60	1.50-1.95	SC	30-35	19	15	Q	5,50	19,57
	3.35-3.80	SC	30-35	14	11	Q		19,57
	4.50-4.95	SC	30-35	14	22	Q		19,57
AR-1	1.50-1.95	GC	15,00	8	10	Q	5,00	19,00
	2.20-2.65	GC	15,00	8	11	Q		19,00
	8.50-8.95	SC	10-15	11	23	Q		19,00
	9.50-9.95	SC	10-15	12	9	Q		19,00
	10.50-10.95	SC	10-15	12	4	Q		19,00
	11.50-11.95	SC	10-15	11	10	Q		19,00
	12.50-12.95	SP-SC	15-20	12	9	Q		19,00
	13.50-13.95	SP-SC	15-20	12	12	Q		19,00
	14.50-14.95	GC	10-15	9	9	Q		19,00

Table A.1 Continued.

AR-2	2.1-2.55	GC	10-15	0	6	Q	7,50	19,50
	7.5-7.95	SP-SC	15-20	13	9	Q		19,50
	10.5-10.95	GW-SC	5,00	0	20	Q		19,50
	12.0-12.45	GW-SC	5,00	0	26	Q		19,50
	13.5-13.95	SP-SC	10-20	10	8	Q		19,50
	15-15.45	SP-SC	10-20	10	12	Q		19,50
AR-3	6.00-6.45	SC	15,00	9	19	Q	7,20	19,00
	7.50-7.95	GW-GC	10,00	0	25	Q		19,00
	10.5-10.95	GW-GC	5.0-10	0	27	Q		19,00
	12.00-12.45	GW-GC	5.0-10	0	43	Q		19,00
AR-4	1.5-1.95	SC	30,00	14	5	Q	7,60	19-20
	3.00-3.45	SC	30,00	14	18	Q		19-20
	6.00-6.45	SP-SC	5,00	8	15	Q		19-20
	9.00-9.45	GW-GC	5.0-10	0	36	Q		19-20
	10.5-10.95	GW-GC	5.0-10	0	24	Q		19-20
	12.0-12.45	GW-GC	5.0-10	0	23	Q		19-20
	13.5-13.95	GW-GC	5.0-10	0	24	Q		19-20
AR-5	1.5-1.95	GW-GC	5.0-15	0	10	Q	4,00	19-20
	3.00-3.45	GW-GC	5.0-15	0	8	Q		19-20
	7.50-7.95	GW-GC	15,00	0	17	Q		19-20
	9.00-9.45	SP-SC	15,00	9	13	Q		19-20
	10.5-10.95	GW-GC	5.0-10	0	20	Q		19-20
	12.0-12.45	GW-GC	5.0-10	0	25	Q		19-20
	13.5-13.95	GW-GC	5.0-10	0	25	Q		19-20
AR-6	6.00-6.45	GW-GC	5-10	0	10	Q	6,45	19,00
	7.5-7.95	GW-GC	5-10	0	23	Q		19,00
	10-50-10-95	GC	5-10	9	28	Q		19,00
AR-7	7.5-7.95	GC	15-25	11	5	Q	4,50	19,00
	9.0-9.45	GC	15-25	8	14	Q		19,00
	10.0-10.45	GC	15-25	10	5	Q		19,00
	12.0-12.45	GC	15-25	10	7	Q		19,00
	13.50-13.95	GC	15-25	9	9	Q		19,00
	15.0-15.45	GC	15-25	8	13	Q		19,00
AR-8	4.50-4.95	SC-SM	30-35	13	7	Q	4,20	19-20
	6.00-6.45	SC-SM	30-35	9	11	Q		19-20
	7.50-7.95	SC-SM	30-35	9	3	Q		19-20
	8.00-8.45	SC-SM	30-35	0	9	Q		19-20
	9.00-9.45	GW-GC	12,00	0	20	Q		19-20
	10.5-10.95	GW-GC	15,00	0	4	Q		19-20
	12.0-12.45	GW-GC	15,00	0	28	Q		19-20
	13.5-13.95	GW-GC	15,00	0	9	Q		19-20
	15.50-15.95	GW-GC	15,00	0	3	Q		19-20
BY-26	1.5-1.95	SM	40,00	0	14	Q	8,40	19-20
	3.0-3.45	SM	40,00	0	14	Q		19-20
	6.0-6.45	SM	21,00	0	24	Q		19-20
BY-27	1.5-1.95	SM	33,00	0	11	Q	9,75	19-20
	4.50-4.95	SM	32,00	0	17	Q		19-20
	6.00-6.45	SM	47,00	0	14	Q		19-20
	7.50-7.95	SC	49,00	15	24	Q		19-20
	9.00-9.45	SM	27,00	0	22	Q		19-20
BY-36	3.0-3.45	SM	14,00	0	33	Q	4,65	19,50
	4.5-4.95	SM	12,00	0	16	Q		19,50
	6.0-6.45	SM	12,00	0	16	Q		19,50
	7.5-7.95	SM	15,00	0	37	Q		19,50
BY-36/A	1.5-1.95	SM	18,00	0	12	Q	5,10	19-20
	3.0-3.45	SM	21,00	0	20	Q		19-20
	4.5-4.95	SM	13,00	0	27	Q		19-20
	6.0-6.45	SM	38,00	0	6	Q		19-20
	7.5-7.95	SM	20,00	0	19	Q		19-20
	9.0-9.45	SW-SM	7,00	0	18	Q		19-20
	10-50-10.95	SW-SM	11,00	0	19	Q		19-20
BY-36/B	1.5-1.95	GM	30,00	0	7	Q	4,00	19-20
	3.00-3.45	SC	36,00	12	10	Q		19-20
	4.50-4.95	SM	25,00	0	13	Q		19-20
	6.00-6.45	SC	36,00	12	7	Q		19-20

Table A.1 Continued.

BY37	3.0-3.45	SM	27,00	0	10	Q	4,10	19-20
	4.5-4.95	SC	37,00	9	11	Q		19-20
	7.5-7.95	SM-SC	26,00	6	22	Q		19-20
	9.0-9.45	SM	13,00	0	23	Q		19-20
	12.0-12.45	SM	24,00	0	20	Q		19-20
	18-18-45	SC	27,00	21	26	Q		19-20
BY-38	1.50-1.95	SC	41,00	11	6	Q	4,10	19-20
	3.00-3.45	SM	30,00	0	9	Q		19-20
	6.00-6.45	SC	38,00	15	14	Q		19-20
	7.50-7.95	GM	15,00	0	35	Q		19-20
	9.00-9.45	SC	49,00	15	8	Q		19-20
	13.5-13.95	SC	36,00	3	4	Q		19-20
BY-38/A	1.50-1.95	SC	30,00	9	12	Q	4,50	20,00
	3.00-3.45	SM	20,00	0	20	Q		20,00
	6.00-6.45	GM	15,00	0	27	Q		20,00
	7.50-7.95	GM	15,00	0	26	Q		20,00
BY-39	3.00-3.45	SM	33,00	0	14	Q	3,90	20,00
	6.00-6.45	SM	18,00	0	8	Q		20,00
	7.50-7.95	SM	19,00	0	31	Q		20,00
	9.0-9.45	SM	15,00	0	19	Q		20,00
	12.0-12.45	SM	42,00	0	10	Q		20,00
	13.5-13.95	SC	23,00	12	26	Q		20,00
	15.0-15.45	SC	43,00	16	14	Q		20,00
	16.50-16.95	SM	28,00	0	10	Q		20,00
BY-49	3.0-3.45	SM	31,00	0	26	L. Plio-Pleist.	9,60	19,00
	4.5-4.95	SM	48,00	0	14	L. Plio-Pleist.		19,00
	7.5-7.95	SC	23,00	18	31	L. Plio-Pleist.		19,00
BY-35	1.50-1.95	SM	27,00	0	12	Q	6,65	19,50
	3.00-3.45	GM	15,00	0	10	Q		19,50
	4.5-4.95	SC	41,00	24	10	Q		19,50
	10.5-10.95	SC	47,00	37	22	Q		19,50
BY-40	1.50-1.95	SM	43,00	0	7	Q	4,80	20,00
	15.0-15.45	SC	37,00	21	14	Q		20,00
	16.50-16.95	SC	24,00	16	8	Q		20,00
BY-34	1.50-1.95	SC	33,00	23	10	Q	6,36	19,50
	3.00-3.45	SC	44,00	22	16	Q		19,50
BY-50	3.0-3.45	SM	38,00	0	11	L. Plio-Pleist.	11,50	19,50
	4.5-4.95	SC	39,00	23	20	L. Plio-Pleist.		19,50
	6.5-6.95	GM	18,00	0	31	L. Plio-Pleist.		19,50
	7.5-7.95	SM	30,00	0	18	L. Plio-Pleist.		19,50
BY-52	1.50-1.95	SM	14,00	0	3	Q	6,50	19,50
	6.00-6.45	SC	45,00	28	11	Q		19,50
BY-51	1.50-1.90	SP-SM	8,00	0	4	Q	7,00	19,50
	3.00-3.45	SM	47,00	0	17	Q		19,50
	4.50-4.95	SC	36,00	17	12	Q		19,50
S-24	3.0-3.45	SC/SM	23,00	0	7,0	Q	2,60	19,00
	4.5-4.95	SC	29,00	13	15,0	Q		19,00
	7.50-7.95	GM	30,00	17	23,0	Q		19,00
	9.0-9.45	GC/GM	18,00	0	27,0	Q		19,00
S-33	4.55-5.0	SM	30,00	16	10,0	Q	0,85	19,00
	6.0-6.45	SP/SM	45,00	16	18,0	Q		19,00
S-65	1.50-1.95	SC	34,00	14	9	Q	0,30	18,40
	3.00-3.45	SM/SC	23,00	0	15	Q		18,40
	7.50-7.95	GM/GC	18,00	0	10	Q		18,40
S-515	3.0-3.45	SC	41,00	10	11,0	Q	3,00	18,50
	4.5-4.95	SC	37,00	11	17,0	Q		18,50
S-524	4.5-4.95	SC	46,00	8	8,0	Q	4,75	19,79
	7.50-7.95	SM	44,00	10	13,0	Q		19,79
	13.5-13.95	GC/GM	32,00	0	11,0	Q		19,79
S- 525	4.5-4.95	SW/SM	5,00	0	17	Q	5,25	19,00
	6.5-6.95	SW/SM	5,00	0	15	Q		19,00
	14.0-14.45	SM	35,00	13	18	Q		19,00
S- 526	7.50-7.95	SP/SM	11,00	0	9	Q	5,50	19,00
	10.0-10.45	SP/SM	11,00	0	12	Q		19,00
	14.0-14.45	SW/SM	5,00	0	15	Q		19,00

Table A.1 Continued.

BER-THY-1	1.50-1.90	SW/SM	10,00	11	10	Q	5,00	19,50
	3.00-3.45	SW/SM	10,00	11	12	Q		19,50
	4.50-4.95	SW/SM	10,00	11	14	Q		19,50
BER-THY-2	1.50-1.90	SW/SM	15,00	15	13	Q	4,50	19,20
	3.00-3.45	SW/SM	15,00	15	10	Q		19,20
BER-THY-3	1.50-1.90	SW/SM	10,00	10	8	Q	4,50	19,70
	3.00-3.45	SW/SM	10,00	10	11	Q		19,70
BER-22	1.50-1.95	SM	15,00	0	9	Q	4,50	18,00
	3.00-3.45	SM	15,00	0	10	Q		18,00
	4.50-4.95	SM	15,00	0	15	Q		18,00
	6.00-6.45	SM	15,00	0	22	Q		18,00
	7.50-7.95	SM	15,00	0	25	Q		18,00
BER-28	1.50-1.95	SM	10	0	12	Q	4,50	18,50
	3.00-3.45	SM	10	0	13	Q		18,50
	4.50-4.95	SM	20	10	14	Q		18,50
EKA-20	1.50-1.95	SM/SW	10,00	0	12	Q	3,70	18,50
	3.00-3.45	SM/SW	10,00	0	14	Q		18,50
	4.50-4.95	SM/SW	10,00	0	18	Q		18,50
	6.00-6.45	SM/SW	10,00	0	21	Q		18,50
	7.50-7.95	SM/SW	10,00	0	23	Q		18,50
EKA-33	3.00-3.45	SC	30,00	13	15	Q	4,80	18,00
	4.50-4.95	SC	30,00	13	19	Q		18,00
	6.00-6.45	SC	30,00	13	21	Q		18,00
	7.50-7.95	SC	30,00	13	24	Q		18,00
HIPOD-1	4.50-4.95	SM	20,00	0	17	Q	4,30	19,50
	6.00-6.45	SM	12,00	0	20	Q		19,50
	7.50-7.95	SM	33,00	0	9	Q		18,50
HIPOD-3	3.00-3.45	SM	20,00	13	19	Q	4,35	20,00
	4.50-4.95	SM	18,00	13	19	Q		20,00
	6.00-6.45	SM	12,00	13	13	Q		20,00
	7.50-7.95	SM	12,00	13	27	Q		20,00
HIPOD-5	1.50-1.95	SM	18	0	20	Q	4,10	18,80
	6.00-6.45	SM	31,00	0	4	Q		18,80
	7.50-7.95	SP/SM	6,00	0	14	Q		18,80
	10.5-10.95	SP/SM	8,00	0	17	Q		18,80
STAD-1	1.50-1.95	SC	41,00	25	2	Q	1,50	19,00
	5.00-5.45	SM	27,00	0	2	Q		19,00
	8.50-8.95	SW/SP	7,00	0	22	Q		19,00
STAD-2	4.50-4.95	SM	25,00	0	12	Q	1,50	19,50
	7.50-7.95	SM	18,00	0	15	Q		19,50
	9.0-9.45	SW/SM	4,00	0	25	Q		19,50
STAD-4	4.50-4.95	SC	39,00	0	6	Q	1,60	19,50
	9.0-9.45	SM	24,00	0	11	Q		19,50
OPER-1	4.50-4.95	SM	18,00	0	16	Q	6,70	17,00
	7.50-7.95	SM	18,00	0	16	Q		17,00
	9.0-9.45	SM	18,00	0	20	Q		17,00
A-ETI-3	3.00-3.45	SM	12,00	0	16	Q	8,27	19,00
	4.50-4.95	SC	38,00	0	23	Q		19,00
A-AYA-2	7.50-7.95	SC	29,00	11	31	Q	8,00	20,30
	9.00-9.45	SC	13,00	0	13	Q		19,50
	10.50-10.95	SC	13,00	0	15	Q		19,50
	12.00-12.45	SM	19,00	0	7	Q		19,50
U Sub BH-7	4.50-4.95	SC	20,00	10	9	Q	4,90	19,50
	6.40-6.95	SC	20,00	10	13	Q		19,50
KHO-2	1.00-1.45	SC	20,00	12	14	Q	4,80	18,44
	3.00-3.45	SC	20,00	12	22	Q		18,44
	4.00-4.45	SC	20,00	12	22	Q		18,44
GES-1	4.50-4.95	SM	43,00	0	8	Q	3,80	20,00
	6.00-6.45	SM	43,00	0	8	Q		20,00
	8.00-8.45	SC	20,00	0	6	Q		20,00

Table A1 Continued.

GIMAT-2	3.00-3.45	SW/SM	8,00	0	25	Q	3,30	20,00
	4.50-4.95	SW/SM	8,00	0	20	Q		20,00
	6.00-6.45	SW/SM	8,00	0	22	Q		20,00
	7.50-7.95	SW/SM	8,00	0	21	Q		20,00
GIMAT-4	1.50-1.95	SC	39,00	14	9	Q	3,30	19,70
	3.00-3.45	SW/SM	5,00	0	14	Q		19,70
	4.50-4.95	SW/SM	5,00	0	16	Q		19,70
	6.00-6.45	SW/SM	5,00	0	17	Q		19,70
GIMAT-5	3.00-3.45	SW/SM	6,00	0	22	Q	3,10	19,70
	4.50-4.95	SW/SM	6,00	0	25	Q		19,70
GIMAT-6	1.50-1.95	SC/SM	22,00	6	17	Q	4,20	19,70
	3.00-3.45	SC/SM	22,00	6	21	Q		19,70
	4.50-4.95	SW/SM	7,00	0	18	Q		19,70
	6.00-6.45	SW/SM	7,00	0	29	Q		19,70
	7.50-7.95	SW/SM	7,00	0	23	Q		19,70
YEG-5	1.50-1.95	SC	35,00	15	9	Q	1,30	19,10
	3.00-3.45	SC	35,00	15	8	Q		19,10

Table A.2 The in-situ testing results of the shear wave velocity measurements (V_s studies) along with the required geotechnical information

TESTING NO	Depth	USCS	Fines Content %	PI %	V_s	Lithology	WT (m)	γ (kN/m ³)
SB SK-2	6.0-6.45	SC	31,00	18	210	Q	4,50	18,00
	7.5-7.95	SC	31,00	18	320	Q		18,00
	10.5-10.95	SM	18,80	0	320	Q		18,00
SB SK-A3	5.5-5.95	SM	8,10	0	140	Q	2,00	18,00
	7.5-7.95	SM	8,10	0	140	Q		18,00
SB SK-A5	1.5-1.95	GM	5,90	0	130	Q	2,00	19,10
	3.0-3.45	GM	5,90	0	130	Q		19,10
	5.0-5.45	SM	6,00	0	250	Q		19,10
	6.5-6.95	SM	32,70	0	250	Q		19,10
SIS-45	1.5-1.95	GM	5,90	0	200	Q	2,00	19,00
	3.0-3.45	GM	5,90	0	200	Q		19,00
	5.0-5.45	SM	6,00	0	200	Q		19,00
	6.5-6.95	SM	32,70	0	200	Q		19,00
SIS-169	1.00-1.45	SC	21,00	16	240	Q	4,00	20,80
	3.00-3.45	SC	43,60	25	240	Q		20,80
	4.00-4.45	GC	26,00	15	240	Q		20,80
	6.00-6.45	SC	44,00	19	240	Q		20,80
	7.00-7.45	SC	42,00	23	240	Q		20,80
SIS-168	5.50-5.95	GM	17,40	0	220	Q	5,50	20,10
	6.00-6.45	GM-GP	9,40	0	220	Q		20,10
	8.50-8.95	SC	45,60	24	220	Q		20,10
SIS-165	5.50-5.95	GM	10,00	0	140	Q	5,30	19,60
	7.00-7.45	SM	11,00	0	140	Q		19,60
BM-20	3.35-3.80	SC	30,00	12	143	Q	2,80	19,57
	6.30-6.75	GC	9,60	9	143	Q		19,57
	7.50-7.95	SC	35,00	23	143	Q		19,57
SIS-102	1.50-1.95	SC	34,00	14	120	Q	0,30	18,40
	3.00-3.45	SM/SC	23,00	0	180	Q		18,40
	7.50-7.95	GM/GC	18,00	0	180	Q		18,40
SIS-130	4.5-4.95	SC	46,00	8	110	Q	4,75	19,79
	7.50-7.95	SM	44,00	10	170	Q		19,79
	13.5-13.95	GC/GM	32,00	0	170	Q		19,79
SIS-131	4.5-4.95	SW/SM	5,00	0	130	Q	5,25	19,00
	6.5-6.95	SW/SM	5,00	0	130	Q		19,00
	14.0-14.45	SM	35,00	13	240	Q		19,00
SIS-108	1.50-1.90	SW/SM	10,00	10	140	Q	4,50	19,70
	3.00-3.45	SW/SM	10,00	10	140	Q		19,70
SIS-12	1.50-1.95	SM	15,00	0	90	Q	4,50	18,00
	3.00-3.45	SM	15,00	0	90	Q		18,00
	4.50-4.95	SM	15,00	0	90	Q		18,00
	6.00-6.45	SM	15,00	0	150	Q		18,00
	7.50-7.95	SM	15,00	0	150	Q		18,00
SIS-9	1.50-1.95	SM	10	0	130	Q	4,50	18,50
	3.00-3.45	SM	10	0	130	Q		18,50
	4.50-4.95	SM	20	10	130	Q		18,50
SIS-13	1.50-1.95	SM/SW	10,00	0	100	Q	3,70	18,50
	3.00-3.45	SM/SW	10,00	0	100	Q		18,50
	4.50-4.95	SM/SW	10,00	0	100	Q		18,50
	6.00-6.45	SM/SW	10,00	0	170	Q		18,50
	7.50-7.95	SM/SW	10,00	0	170	Q		18,50
SIS-17	3.00-3.45	SC	30,00	13	130	Q	4,80	18,00
	4.50-4.95	SC	30,00	13	130	Q		18,00
	6.00-6.45	SC	30,00	13	220	Q		18,00
	7.50-7.95	SC	30,00	13	220	Q		18,00
SIS-182	4.50-4.95	SM	20,00	0	160	Q	4,30	19,50
	6.00-6.45	SM	12,00	0	160	Q		19,50
	7.50-7.95	SM	33,00	0	160	Q		18,50

Table A.2 Continued.

SIS-204	1.50-1.95	SM	18	0	130	Q	4,10	18,80
	6.00-6.45	SM	31,00	0	130	Q		18,80
	7.50-7.95	SP/SM	6,00	0	220	Q		18,80
	10.5-10.95	SP/SM	8,00	0	220	Q		18,80
SIS-183	4.50-4.95	SM	25,00	0	120	Q	1,50	19,50
	7.50-7.95	SM	18,00	0	120	Q		19,50
	9.0-9.45	SW/SM	4,00	0	200	Q		19,50
GPS-1	1.50-1.95	SC	39,00	14	90	Q	3,30	19,70
	3.00-3.45	SW/SM	5,00	0	170	Q		19,70
	4.50-4.95	SW/SM	5,00	0	170	Q		19,70
	6.00-6.45	SW/SM	5,00	0	250	Q		19,70
GPS-2	1.50-1.95	SC/SM	22,00	6	190	Q	4,20	19,70
	3.00-3.45	SC/SM	22,00	6	190	Q		19,70
	4.50-4.95	SW/SM	7,00	0	190	Q		19,70
	6.00-6.45	SW/SM	7,00	0	190	Q		19,70
	7.50-7.95	SW/SM	7,00	0	300	Q		19,70

APPENDIX B

THE INFORMATION REGARDING THE MOBILE MEASUREMENT POINTS DURING THE FIELD STUDY ON THE “MICROTREMOR RECORD CARDS”

Table B.1 Microtremor Record Card-1

LOCATION: Sincan 1						
No	Date	Time	Altitude	Record	Point	Comments
1	22.09.2004	10:30	783	1	1	Quaternary
2	22.09.2004	10:55	780	2	2	quite
3	22.09.2004	11:03	783	3	3	Quaternary, quite, tributary
4	22.09.2004	11:20	784	4	4	Quaternary
5	22.09.2004	11:38	784	5	5	Quaternary, noisy
6	22.09.2004	11:51	786	6	6	L. Pliocene to Pleistocene, noisy
7	22.09.2004	12:04	781	7	7	Quaternary, quite
8	22.09.2004	12:20	788	8	8	Quaternary, noisy
9	22.09.2004	12:33	785	9	9	Quaternary, noisy
10	22.09.2004	12:42	791	10	10	L. Pliocene to Pleistocene
11	22.09.2004	12:56	786	11	11	noisy
12	22.09.2004	13:09	787	12	12	
13	22.09.2004	13:20	784	13	13	Quaternary, noisy
14	22.09.2004	14:42	790	14	14	Quaternary and L. Pliocene to Pleistocene, noisy
15	22.09.2004	14:53	800	15	15	L. Pliocene to Pleistocene, noisy
16	22.09.2004	15:16	808	16	16	L. Pliocene to Pleistocene, noisy
17	22.09.2004	15:43	800	17	17	Quaternary, quite
18	22.09.2004	15:52	801	18	18	Quaternary
19	22.09.2004	16:05	816	19	19	Quaternary
20	22.09.2004	16:18	820	20-21	20	Two measurements were taken because of noise
21	22.09.2004	16:35	829	22	21	Quaternary
22	22.09.2004	17:00	842	23	22	Quaternary, windy
23	22.09.2004	17:11	851	24	23	Miocene and L. Pliocene to Pleistocene
24	22.09.2004	17:32	817	25	24	L. Pliocene to Pleistocene, noisy
25	22.09.2004	17:43	796	26	25	L. Pliocene to Pleistocene, noisy
26	22.09.2004	17:53	790	27	26	Quaternary and L. Pliocene to Pleistocene, noisy
27	22.09.2004	18:02	796	28	27	Quaternary
28	22.09.2004	18:16	795	29	28	Quaternary and L. Pliocene to Pleistocene, noisy
29	22.09.2004	18:38	796	30	29	Quaternary and L. Pliocene to Pleistocene
30	22.09.2004	18:50	794	31	30	Quaternary and L. Pliocene to Pleistocene
31	22.09.2004	19:07	800	32	31	Quaternary and L. Pliocene to Pleistocene

Table B.2 Microtremor Record Card-2

LOCATION: Sincan 1						
No	Date	Time	Altitude	Record	Point	Comments
1	23.09.2004	09:54	801	33	32	Quaternary
2	23.09.2004	10:21	810	34	33	Quaternary, noisy
3	23.09.2004	10:35	831	35	34	Quaternary and L. Pliocene to Pleistocene
4	23.09.2004	10:45	830	36	35	Quaternary
5	23.09.2004	11:05	834	37	36	Quaternary
6	23.09.2004	11:21	817	38	37	Quaternary and L. Pliocene to Pleistocene
7	23.09.2004	11:29	821	39	38	Quaternary, tributary
8	23.09.2004	11:44	824	40	39	L. Pliocene to Pleistocene, noisy
9	23.09.2004	12:01	820	41	40	Quaternary
10	23.09.2004	12:12	823	42	41	
11	23.09.2004	12:30	818	43	42	Quaternary
12	23.09.2004	12:46	831	44	43	Quaternary, windy
13	23.09.2004	13:12	807	45	44	Quaternary, noisy
14	23.09.2004	13:24	837	46	45	Quaternary and L. Pliocene to Pleistocene, noisy
15	23.09.2004	13:38	825	47	46	L. Pliocene to Pleistocene
16	23.09.2004	15:08	794	48	47	noisy
17	23.09.2004	15:21	780	49	48	Quaternary
18	23.09.2004	15:32	783	50	49	Quaternary, noisy
19	23.09.2004	15:50	788	51	50	Quaternary, noisy
20	23.09.2004	16:04	787	52	51	Quaternary
21	23.09.2004	16:18	783	53	52	tributary
22	23.09.2004	16:30	794	54	53	Quaternary, noisy
23	23.09.2004	16:45	793	55	54	Quaternary, noisy
24	23.09.2004	17:01	784	56	55	Quaternary, noisy
25	23.09.2004	17:16	783	57	56	Quaternary, embankment
26	23.09.2004	17:33	783	58	57	Quaternary
27	23.09.2004	17:45	792	59	58	noisy
28	23.09.2004	17:59	789	60	59	Quaternary
29	23.09.2004	18:12	784	61	60	Quaternary
30	23.09.2004	18:25	789	62	61	Quaternary, noisy, tributary

Table B.3 Microtremor Record Card-3

LOCATION: Sincan 2						
No	Date	Time	Altitude	Record	Point	Comments
1	24.09.2004	09:55	792	1	62	Quaternary and L. Pliocene to Pleistocene
2	24.09.2004	10:16	820	2	63	embenkant
3	24.09.2004	10:30	824	3	64	L. Pliocene to Pleistocene
4	24.09.2004	10:42	833	4	65	L. Pliocene to Pleistocene
5	24.09.2004	11:15	792	5	66	Quaternary
6	24.09.2004	11:36	795	6	67	Quaternary
7	24.09.2004	11:46	790	7	68	Quaternary
8	24.09.2004	11:58	794	8	69	noisy, tributary
9	24.09.2004	12:15	798	9	70	Quaternary
10	24.09.2004	12:27	798	10	71	Quaternary
11	24.09.2004	12:41	802	11	72	L. Pliocene to Pleistocene
12	24.09.2004	13:09	811	12	73	L. Pliocene to Pleistocene, noisy
13	24.09.2004	14:36	805	13	74	Quaternary and L. Pliocene to Pleistocene, noisy
14	24.09.2004	14:49	795	14	75	noisy, high voltage line
15	24.09.2004	15:10	818	15	76	L. Pliocene to Pleistocene
16	24.09.2004	15:21	810	16	77	Quaternary and L. Pliocene to Pleistocene
17	24.09.2004	15:38	793	17	78	L. Pliocene to Pleistocene
18	24.09.2004	15:58	794	18	79	
19	24.09.2004	16:12	797	19	80	Quaternary
20	24.09.2004	16:28	786	20	81	Quaternary
21	24.09.2004	16:41	786	21	82	Quaternary
22	24.09.2004	16:54	798	22	83	L. Pliocene to Pleistocene
23	24.09.2004	17:06	804	23	84	L. Pliocene to Pleistocene, electricity line
24	24.09.2004	17:18	790	24	85	L. Pliocene to Pleistocene
25	24.09.2004	17:31	787	25	86	Quaternary
26	24.09.2004	17:50	781	26	87	Quaternary and L. Pliocene to Pleistocene

Table B.4 Microtremor Record Card-4

LOCATION: Etimesgut 1						
No	Date	Time	Altitude	Record	Point	Comments
1	29.09.2004	10:40	831	1	88	L. Pliocene to Pleistocene
2	29.09.2004	11:03	803	2	89	Quaternary
3	29.09.2004	11:15	808	3	90	L. Pliocene to Pleistocene
4	29.09.2004	11:29	850	4	91	L. Pliocene to Pleistocene
5	29.09.2004	11:54	827	5	92	L. Pliocene to Pleistocene
6	29.09.2004	12:16	839	6	93	L. Pliocene to Pleistocene
7	29.09.2004	12:33	815	7	94	L. Pliocene to Pleistocene
8	29.09.2004	12:54	817	8	95	L. Pliocene to Pleistocene
9	29.09.2004	13:04	800	9	96	L. Pliocene to Pleistocene
10	29.09.2004	13:18	797	10	97	Quaternary and L. Pliocene to Pleistocene
11	29.09.2004	13:32	816	11	98	L. Pliocene to Pleistocene
12	29.09.2004	13:57	804	13	99	12 th measurement is missing
13	29.09.2004	15:15	816	14	100	Quaternary
14	29.09.2004	15:37	833	15	101	Quaternary, noisy, pond
15	29.09.2004	15:50	823	16	102	L. Pliocene to Pleistocene
16	29.09.2004	16:08	804	17	103	Quaternary
17	29.09.2004	16:20	813	18	104	Quaternary and L. Pliocene to Pleistocene
18	29.09.2004	16:35	822	19	105	L. Pliocene to Pleistocene
19	29.09.2004	17:02	806	20	106	Quaternary
20	29.09.2004	17:19	807	21	107	Quaternary and L. Pliocene to Pleistocene
21	29.09.2004	17:42	800	22	108	Quaternary
22	29.09.2004	17:58	794	23	109	Quaternary
23	29.09.2004	18:13	795	24	110	Quaternary
24	29.09.2004	18:32	799	25	111	Quaternary

Table B.5 Microtremor Record Card-5

LOCATION: Etimesgut 1						
No	Date	Time	Altitude	Record	Point	Comments
1	30.09.2004	10:30	798	26	112	Quaternary
2	30.09.2004	10:42	803	27	113	Quaternary, embankment
3	30.09.2004	11:01	797	28	114	Quaternary
4	30.09.2004	11:18	797	29	115	Quaternary
5	30.09.2004	11:35	797	30	116	Quaternary
6	30.09.2004	11:57	798	31	117	Quaternary
7	30.09.2004	12:11	787	32	118	Quaternary, tributary
8	30.09.2004	12:31	795	33	119	Quaternary
9	30.09.2004	12:44	790	34	120	Quaternary
10	30.09.2004	13:03	791	35	121	Quaternary
11	30.09.2004	13:20	793	36	122	Quaternary, noisy
12	30.09.2004	13:50	805	37	123	
13	30.09.2004	15:10	786	38	124	Quaternary and L. Pliocene to Pleistocene
14	30.09.2004	15:24	803	39	125	Quaternary
15	30.09.2004	15:34	813	40	126	Quaternary
16	30.09.2004	15:46	842	41	127	Miocene and L. Pliocene to Pleistocene, noisy
17	30.09.2004	15:58	808	42	128	Quaternary
18	30.09.2004	16:10	804	43	129	Quaternary
19	30.09.2004	16:31	815	44	130	Quaternary
20	30.09.2004	16:46	842	45	131	L. Pliocene to Pleistocene
21	30.09.2004	17:06	856	46	132	L. Pliocene to Pleistocene
22	30.09.2004	17:18	806	47	133	Quaternary and L. Pliocene to Pleistocene, noisy
23	30.09.2004	17:29	804	48	134	Quaternary
24	30.09.2004	17:40	813	49	135	Quaternary
25	30.09.2004	17:54	831	50	136	Quaternary, noisy
26	30.09.2004	18:15	810	51	137	L. Pliocene to Pleistocene

Table B.6 Microtremor Record Card-6

LOCATION: Etimesgut 2							Comments	
No	Date	Time	Altitude	Record	Point			
1	01.10.2004	09:40	822	1	138	Quaternary		
2	01.10.2004	09:52	813	2	139	Quaternary		
3	01.10.2004	10:15	817	3	140	Quaternary, noisy		
4	01.10.2004	10:30	822	4	141	Quaternary, noisy		
5	01.10.2004	10:50	842	5	142	Quaternary, noisy		
6	01.10.2004	11:10	862	6	143	noisy		
7	01.10.2004	11:45	817	7	144	Quaternary, noisy		
8	01.10.2004	12:00	820	8	145	Quaternary and L. Pliocene to Pleistocene		
9	01.10.2004	13:49	834	9	146	L. Pliocene to Pleistocene		
10	01.10.2004	13:59	820	10	147	L. Pliocene to Pleistocene		
11	01.10.2004	14:19	807	11	148	Quaternary		
12	01.10.2004	14:32	805	12	149	Quaternary		
13	01.10.2004	14:59	799	13	150	Quaternary, tributary		
14	01.10.2004	15:20	802	14	151	Quaternary, noisy		
15	01.10.2004	15:33	801	15	152	Quaternary		
16	01.10.2004	15:44	802	16	153	Quaternary, noisy		
17	01.10.2004	16:09	800	17	154	Quaternary		
18	01.10.2004	16:20	825	18	155	Quaternary, noisy, windy, tributary		
19	01.10.2004	16:45	855	19	156	Limestone		
20	01.10.2004	17:06	812	20	157	Quaternary		
21	01.10.2004	17:54	828	21	158	L. Pliocene to Pleistocene, noisy		
22	01.10.2004	18:07	820	22	159	L. Pliocene to Pleistocene, noisy		
23	01.10.2004	18:24	809	23	160	Quaternary		
24	01.10.2004	18:42	808	24	161	Quaternary		

Table B.7 Microtremor Record Card-7

LOCATION: Etimesgut 2						
No	Date	Time	Altitude	Record	Point	Comments
1	04.10.2004	10:22	800	25	162	Quaternary, noisy
2	04.10.2004	10:33	801	26	163	Quaternary
3	04.10.2004	10:50	802	27	164	Quaternary
4	04.10.2004	11:07	810	28	165	Quaternary
5	04.10.2004	11:36	828	29	166	Miocene and L. Pliocene to Pleistocene, Quaternary
6	04.10.2004	11:53	813	30	167	Quaternary, swamp
7	04.10.2004	12:21	814	31	168	Quaternary, swamp
8	04.10.2004	12:56	808	32	169	Quaternary and L. Pliocene to Pleistocene
9	04.10.2004	13:11	813	33	170	Quaternary, swamp
10	04.10.2004	13:25	825	34	171	L. Pliocene to Pleistocene
11	04.10.2004	14:31	817	35	172	Quaternary
12	04.10.2004	14:41	834	36	173	Quaternary
13	04.10.2004	14:55	833	37	174	L. Pliocene to Pleistocene
14	04.10.2004	15:07	840	38	175	Quaternary
15	04.10.2004	15:23	904	39	176	Basalt
16	04.10.2004	15:46	875	40	177	Quaternary
17	04.10.2004	16:06	849	41	178	Quaternary
18	04.10.2004	16:17	858	42	179	Quaternary
19	04.10.2004	16:35	850	43	180	L. Pliocene to Pleistocene
20	04.10.2004	16:47	838	44	181	Quaternary
21	04.10.2004	17:03	845	45	182	Quaternary
22	04.10.2004	17:24	832	46	183	Quaternary
23	04.10.2004	17:38	830	47	184	Quaternary and L. Pliocene to Pleistocene
24	04.10.2004	17:54	810	48	185	Quaternary and L. Pliocene to Pleistocene
25	04.10.2004	18:11	808	49	186	Quaternary

Table B.8 Microtremor Record Card-8

LOCATION: Cayyolu 1						
No	Date	Time	Altitude	Record	Point	Comments
1	05.10.2004	09:54	893	1	187	Quaternary
2	05.10.2004	10:15	879	2	188	Quaternary
3	05.10.2004	10:30	913	3	189	Limestone
4	05.10.2004	10:40	878	4	190	Quaternary, noisy
5	05.10.2004	11:00	888	5	191	Limestone, L. Pliocene to Pleistocene
6	05.10.2004	11:21	863	6	192	Quaternary
7	05.10.2004	11:40	874	7	193	Quaternary and L. Pliocene to Pleistocene, noisy
8	05.10.2004	11:53	896	8	194	Quaternary
9	05.10.2004	12:06	885	9	195	Limestone
10	05.10.2004	12:20	859	10	196	Quaternary, noisy
11	05.10.2004	12:32	838	11	197	Quaternary, noisy
12	05.10.2004	12:41	864	12	198	Quaternary, noisy
13	05.10.2004	12:53	875	13	199	Quaternary
14	05.10.2004	14:15	930	14	200	Limestone, L. Pliocene to Pleistocene
15	05.10.2004	14:31	920	15	201	L. Pliocene to Pleistocene
16	05.10.2004	14:45	887	16	202	L. Pliocene to Pleistocene
17	05.10.2004	15:01	949	17	203	Limestone, L. Pliocene to Pleistocene
18	05.10.2004	15:15	895	18	204	L. Pliocene to Pleistocene
19	05.10.2004	15:34	888	19	205	Limestone
20	05.10.2004	15:46	846	20	206	L. Pliocene to Pleistocene, noisy
21	05.10.2004	15:59	858	21	207	L. Pliocene to Pleistocene
22	05.10.2004	16:15	886	22	208	Limestone, L. Pliocene to Pleistocene
23	05.10.2004	16:37	874	23	209	L. Pliocene to Pleistocene
24	05.10.2004	16:51	932	24	210	L. Pliocene to Pleistocene
25	05.10.2004	17:09	916	25	211	Limestone
26	05.10.2004	17:25	880	26	212	L. Pliocene to Pleistocene
27	05.10.2004	17:41	873	27	213	L. Pliocene to Pleistocene
28	05.10.2004	18:00	884	28	214	L. Pliocene to Pleistocene
29	05.10.2004	18:18	876	29	215	L. Pliocene to Pleistocene
30	05.10.2004	18:39	872	30	216	L. Pliocene to Pleistocene

Table B.9 Microtremor Record Card-9

LOCATION: Batikent 1							Comments	
No	Date	Time	Altitude	Record	Point			
1	06.10.2004	10:27	817	1	217	L. Pliocene to Pleistocene, noisy		
2	06.10.2004	10:48	817	2	218	L. Pliocene to Pleistocene		
3	06.10.2004	11:24	820	3	219	Quaternary, windy		
4	06.10.2004	11:53	819	4	220	Quaternary, windy		
5	06.10.2004	12:09	831	5	221	L. Pliocene to Pleistocene, windy		
6	06.10.2004	12:23	832	6	222	L. Pliocene to Pleistocene, windy		
7	06.10.2004	12:50	841	7	223	L. Pliocene to Pleistocene, windy		
8	06.10.2004	13:01	835	8	224	L. Pliocene to Pleistocene, windy		
9	06.10.2004	13:16	847	9	225	L. Pliocene to Pleistocene, windy		
10	06.10.2004	15:16	881	10	226	Quaternary, windy		
11	06.10.2004	15:48	878	11	227	Quaternary, windy, noisy		
12	06.10.2004	16:20	884	12	228	L. Pliocene to Pleistocene		
13	06.10.2004	16:38	863	13	229	Quaternary		
14	06.10.2004	16:50	848	14	230	Quaternary		
15	06.10.2004	17:02	858	15	231	Quaternary		
16	06.10.2004	17:17	840	16	232	Quaternary		
17	06.10.2004	17:30	830	17	233	Quaternary		
18	06.10.2004	17:56	829	18	234	Quaternary		
19	06.10.2004	18:08	848	19	235	L. Pliocene to Pleistocene		
20	06.10.2004	18:20	850	20	236	L. Pliocene to Pleistocene		

Table B.10 Microtremor Record Card-10

LOCATION: Batikent 1						
No	Date	Time	Altitude	Record	Point	Comments
1	07.10.2004	09:52	833	21	237	Quaternary, wetty
2	07.10.2004	10:08	830	22	238	Quaternary
3	07.10.2004	10:26	826	23	239	Quaternary, noisy
4	07.10.2004	10:40	833	24	240	Quaternary
5	07.10.2004	10:54	824	25	241	Quaternary
6	07.10.2004	11:16	837	26	242	Quaternary
7	07.10.2004	12:04	830	27	243	Quaternary
8	07.10.2004	12:22	828	28	244	Quaternary
9	07.10.2004	13:27	832	29	245	Quaternary, noisy
10	07.10.2004	13:50	818	30	246	Quaternary
11	07.10.2004	13:58	830	31	247	Quaternary
12	07.10.2004	14:09	835	32	248	Quaternary, windy
13	07.10.2004	14:23	821	33	249	Quaternary
14	07.10.2004	14:42	845	34	250	Quaternary and L. Pliocene to Pleistocene, noisy

Table B.11 Microtremor Record Card-11

LOCATION: Batikent 1						
No	Date	Time	Altitude	Record	Point	Comments
1	12.10.2004	10:41	833	35	251	Quaternary and L. Pliocene to Pleistocene
2	12.10.2004	11:05	845	36	252	Quaternary and L. Pliocene to Pleistocene, close to Ankara River
3	12.10.2004	11:22	836	37	253	Quaternary
4	12.10.2004	11:49	842	38	254	L. Pliocene to Pleistocene
5	12.10.2004	12:10	830	39	255	Quaternary
6	12.10.2004	12:32	832	40	256	Quaternary
7	12.10.2004	12:58	860	41	257	L. Pliocene to Pleistocene
8	12.10.2004	13:13	845	42	258	L. Pliocene to Pleistocene
9	12.10.2004	13:26	843	43	259	Quaternary, windy
10	12.10.2004	14:51	843	44	260	Quaternary and L. Pliocene to Pleistocene
11	12.10.2004	15:17	863	45	261	L. Pliocene to Pleistocene
12	12.10.2004	15:36	875	46	262	Quaternary
13	12.10.2004	15:54	855	47	263	Quaternary
14	12.10.2004	16:09	861	48	264	L. Pliocene to Pleistocene
15	12.10.2004	16:32	852	49	265	Quaternary
16	12.10.2004	16:49	848	50	266	Quaternary
17	12.10.2004	17:03	872	51	267	L. Pliocene to Pleistocene
18	12.10.2004	17:21	891	52	268	L. Pliocene to Pleistocene
19	12.10.2004	17:48	850	53	269	Quaternary and L. Pliocene to Pleistocene

Table B.12 Microtremor Record Card-12

LOCATION: Çiftlik 1						
No	Date	Time	Altitude	Record	Point	Comments
1	13.10.2004	13:00	836	1	270	Quaternary, quite, swamp
2	13.10.2004	13:20	844	2	271	Quaternary, quite, swamp
3	13.10.2004	13:30	840	3	272	Quaternary
4	13.10.2004	13:45	842	4	273	Quaternary, windy
5	13.10.2004	14:10	839	5	274	Quaternary, windy
6	13.10.2004	14:30	844	6	275	L. Pliocene to Pleistocene, noisy
7	13.10.2004	14:50	832	7	276	Quaternary, noisy
8	13.10.2004	16:05	894	8	277	L. Pliocene to Pleistocene, quite
9	13.10.2004	16:25	873	9	278	L. Pliocene to Pleistocene, windy
10	13.10.2004	16:50	891	10	279	L. Pliocene to Pleistocene
11	13.10.2004	17:25	847	11	280	Quaternary and L. Pliocene to Pleistocene, noisy
12	13.10.2004	18:00	855	12	281	L. Pliocene to Pleistocene, noisy
13	13.10.2004	19:00	829	13	282	Quaternary and L. Pliocene to Pleistocene, noisy

Table B.13 Microtremor Record Card-13

LOCATION: Çiftlik 1						
No	Date	Time	Altitude	Record	Point	Comments
1	14.10.2004	11:50	852	14	283	L. Pliocene to Pleistocene, noisy
2	14.10.2004	12:15	849	15	284	Quaternary and L. Pliocene to Pleistocene, noisy
3	14.10.2004	12:30	859	16	285	L. Pliocene to Pleistocene
4	14.10.2004	12:50	863	17	286	Quaternary and L. Pliocene to Pleistocene, noisy
5	14.10.2004	13:20	896	18	287	L. Pliocene to Pleistocene
6	14.10.2004	13:40	897	19	288	L. Pliocene to Pleistocene
7	14.10.2004	14:00	938	20	289	L. Pliocene to Pleistocene, tibutary
8	14.10.2004	14:10	904	21	290	L. Pliocene to Pleistocene, tibutary
9	14.10.2004	14:30	900	22	291	L. Pliocene to Pleistocene, tibutary
10	14.10.2004	15:35	929	23	292	L. Pliocene to Pleistocene
11	14.10.2004	16:10	902	24	293	L. Pliocene to Pleistocene, tibutary
12	14.10.2004	16:25	897	25	294	L. Pliocene to Pleistocene, tibutary, noisy
13	14.10.2004	16:49	876	26	295	L. Pliocene to Pleistocene, tibutary
14	14.10.2004	17:00	874	27	296	L. Pliocene to Pleistocene
15	14.10.2004	17:20	885	28	297	L. Pliocene to Pleistocene, noisy
16	14.10.2004	17:40	899	29	298	Quaternary and L. Pliocene to Pleistocene, noisy
17	14.10.2004	18:25	866	30	299	Quaternary and L. Pliocene to Pleistocene, noisy
18	14.10.2004	18:45	857	31	300	L. Pliocene to Pleistocene

Table B.14 Microtremor Record Card-14

LOCATION: Çankaya 1						
No	Date	Time	Altitude	Record	Point	Comments
1	16.10.2004	10:20	862	1	301	Quaternary and L. Pliocene to Pleistocene
2	16.10.2004	10:50	874	2	302	Quaternary
3	16.10.2004	11:05	840	3	303	Quaternary
4	16.10.2004	11:20	833	4	304	Quaternary
5	16.10.2004	11:35	837	5	305	Quaternary
6	16.10.2004	11:55	854	6	306	Quaternary
7	16.10.2004	12:15	837	7	307	Quaternary
8	16.10.2004	12:40	856	8	308	Quaternary and L. Pliocene to Pleistocene, noisy
9	16.10.2004	13:00	841	9	309	Quaternary
10	16.10.2004	13:15	839	10	310	Quaternary
11	16.10.2004	14:10	866	11	311	Quaternary, noisy, andezite
12	16.10.2004	15:05	830	12	312	Quaternary, noisy
13	16.10.2004	15:25	842	13	313	Quaternary
14	16.10.2004	15:50	857	14	314	Quaternary
15	16.10.2004	16:20	858	15	315	Quaternary and L. Pliocene to Pleistocene, noisy
16	16.10.2004	16:40	828	16	316	Quaternary and L. Pliocene to Pleistocene
17	16.10.2004	17:10	838	17	317	Quaternary, noisy
18	16.10.2004	17:20	835	18	318	Quaternary, noisy
19	16.10.2004	17:35	835	19	319	Quaternary
20	16.10.2004	17:45	837	20	320	Quaternary, noisy
21	16.10.2004	18:00	838	21	321	Quaternary, noisy

Table B.15 Microtremor Record Card-15

LOCATION: Kızılay 1						
No	Date	Time	Altitude	Record	Point	Comments
1	18.10.2004	10:23	848	1	322	Quaternary
2	18.10.2004	10:43	843	2	323	L. Pliocene to Pleistocene
3	18.10.2004	10:59	866	3	324	Quaternary
4	18.10.2004	11:18	850	4	325	Quaternary and L. Pliocene to Pleistocene, noisy
5	18.10.2004	11:35	863	5	326	L. Pliocene to Pleistocene, noisy
6	18.10.2004	11:49	861	6	327	L. Pliocene to Pleistocene
7	18.10.2004	12:05	852	7	328	Quaternary and L. Pliocene to Pleistocene
8	18.10.2004	12:28	855	8	329	Quaternary
9	18.10.2004	12:45	875	9	330	L. Pliocene to Pleistocene
10	18.10.2004	13:03	866	10	331	Quaternary and L. Pliocene to Pleistocene
11	18.10.2004	13:15	860	11	332	L. Pliocene to Pleistocene
12	18.10.2004	13:41	840	12	333	Quaternary and L. Pliocene to Pleistocene
13	18.10.2004	14:03	883	13	334	L. Pliocene to Pleistocene
14	18.10.2004	15:09	870	14	335	L. Pliocene to Pleistocene
15	18.10.2004	15:56	880	15	336	Quaternary and L. Pliocene to Pleistocene
16	18.10.2004	16:11	854	16	337	Quaternary and L. Pliocene to Pleistocene
17	18.10.2004	16:32	882	17	338	L. Pliocene to Pleistocene, noisy
18	18.10.2004	16:47	907	18	339	Quaternary, noisy
19	18.10.2004	17:02	925	19	340	Quaternary and L. Pliocene to Pleistocene

Table B.16 Microtremor Record Card-16

LOCATION: Military Zones						
No	Date	Time	Altitude	Record	Point	Comments
1	26.10.2004	10:05	836	1	341	Quaternary and L. Pliocene to Pleistocene
2	26.10.2004	10:33	844	2	342	Quaternary
3	26.10.2004	10:59	840	3	343	Quaternary
4	26.10.2004	11:18	842	4	344	Quaternary
5	26.10.2004	11:25	839	5	345	Quaternary
6	26.10.2004	11:39	844	6	346	Quaternary
7	26.10.2004	12:05	832	7	347	L. Pliocene to Pleistocene
8	26.10.2004	12:18	894	8	348	Quaternary
9	26.10.2004	12:25	873	9	349	Quaternary
10	26.10.2004	14:04	891	10	350	Quaternary
11	26.10.2004	14:55	847	11	351	L. Pliocene to Pleistocene
12	26.10.2004	15:41	855	12	352	L. Pliocene to Pleistocene and Quaternary
13	26.10.2004	16:03	829	13	353	L. Pliocene to Pleistocene

APPENDIX C

**MATLAB CODE WITH A MAIN FILE NAMED “SPCRATIO.M” AND THE
RELATED SUB-ROUTINES**

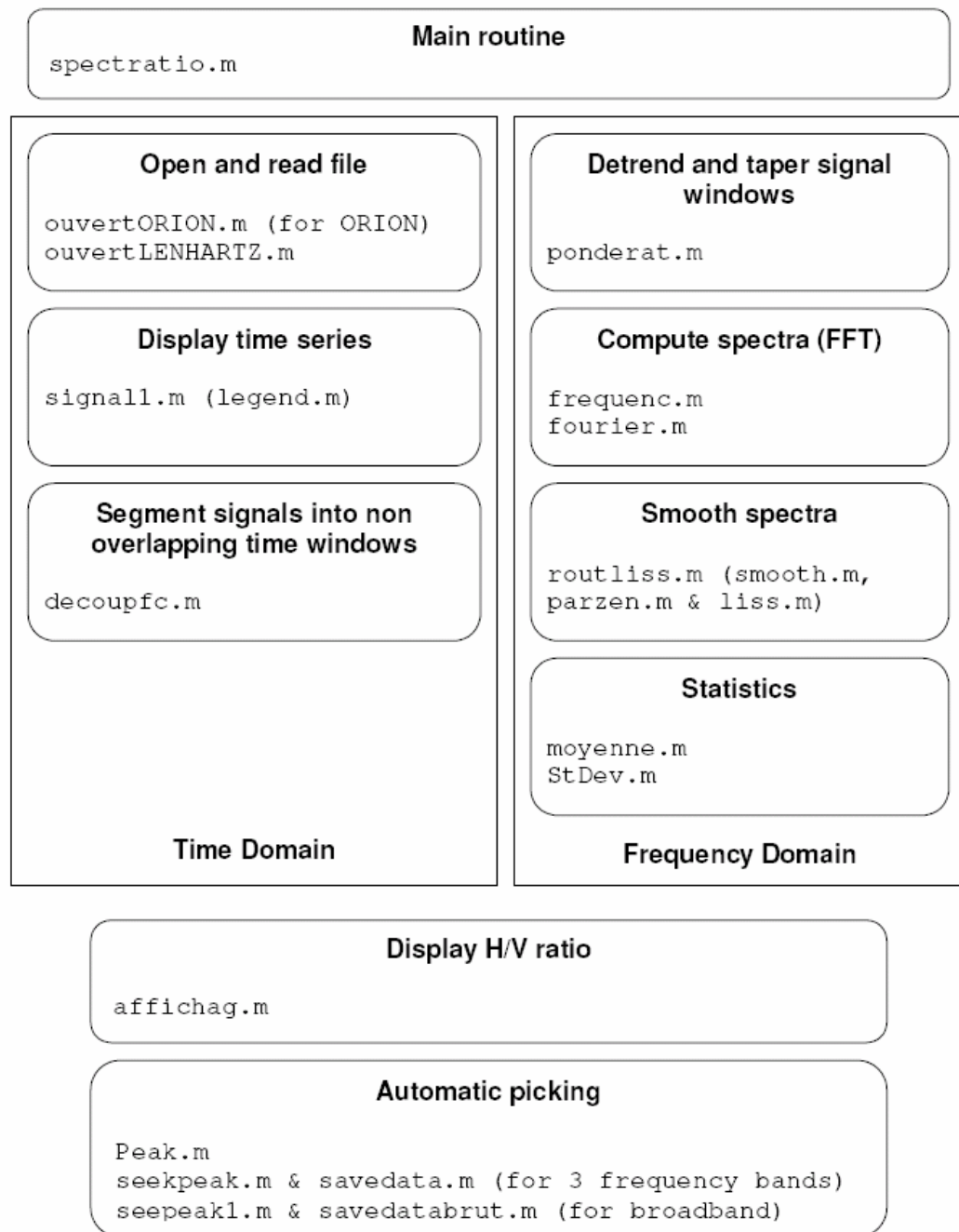


Figure C.1 The organigram of the Matlab Programming Code of SPECTRATIO

“SPCRATIO.M” Main Matlab Code

```
%---//// SPCRATIO \\\--
%---\\\ ***** ////---
%
% Programme d'analyse spectrale du bruit de fond sismique
% calcul du rapport spectral H/V
% recherche automatique des pics dans plusieurs gammes de fréquences
% affichage interactif etc...
%
% Interactive program for spectral analysis of ambient noise records
% Calculation of H/V ratio, automatic picking and graphics display
%
%% P. Rosset - McGill Université de Montréal (2001-2002)%%
%%
%
welcomefig
pause

% LECTURE DES DONNEES / READING OF DATA

ouvertORIION

if success %% condition principale, existence des donnees

% SELECTION D'UNE PORTION DU SIGNAL / WINDOWING OF THE SIGNAL

signal1 %affichage du signal brut pour permettre la selection

%tronc = input('Voulez-vous: (t)out le signal, en choisir une
(p)ortion, (q)uitter ? ','s');
tronc = input('Do you want: (t)whole signal, choose a (p)ortion,
(q)uit
? ','s');

if strcmp(tronc,'q')
    close
    break
%error('programme terminé par l'utilisateur');
end % if strcmp

if strcmp(tronc,'p') % si l'utilisateur a dit "p", il ne veut qu'une
partie
    zoom on;
% a=input('Entrez le début de la portion choisie (secondes) (ou
selectionnez avec la souris): ');

    a=input('Enter the begin of the window (secondes) (or select with
the mouse): ');

if isempty(a)
```

```

        SelectedRange=axis;
        a=SelectedRange(1);
        b=SelectedRange(2);
    if a<1, a=0;, end
    else
        % d=input('Entrez la durée de la portion choisie (secondes): ');
        d=input('Enter the duration of the selected signal (secondes):
    ');

        b=a+d; %b=borne supérieure de l'intervalle de temps choisi
    end % if a==[]

        if b>To, b=To;, end
        na=floor(a/dt+1);
        nb=floor(b/dt);
        xv=xv(na:nb);
        xh1=xh1(na:nb);
        xh2=xh2(na:nb);
    end %if tronc=p

close %referme la fenêtre signal

% DECOUPAGE EN FENETRES / WINDOW CUTTING

%fprintf('\nQuelle longueur de fenetre voulez-vous');
%fprintf('\npour le calcul des spectres? (default=20 sec.) ')
%L=input('\n(tapez seulement enter pour utiliser toutes les valeurs
par
default) ');
%if isempty(L) %l'utilisateur veut toutes les valeurs par default
% default=1;
% disp('le programme va utiliser les valeurs par default...');
% L=20.48; % valeur donnant un nombre de points 2^n (n entier)
% end %if
default=1;
L=40.96 %%%% LARGEUR DE FENETRE / WINDOW WIDTH %%%%
h1=decoupf(xh1,L,dt);
h2=decoupf(xh2,L,dt);
v=decoupf(xv,L,dt);

% PONDERATION %%%
h1pond=ponderat(h1);
h2pond=ponderat(h2);
vpond=ponderat(v);

freq=frequenc(h1pond,dt);

H1=fourier(h1pond);
H2=fourier(h2pond);
V=fourier(vpond);
[Npts, Nfen]=size(H1);

```



```

clear h1 h2 v h1pond h2pond vpond

% LISSAGE / SMOOTHING

if default
    pro=0;
else
    fprintf('\nVoulez-vous un lissage progressif? ');
    prog=input('"o" pour oui (par défaut: lissage uniforme) ','s');
    pro=strcmp(prog,'o');
end %if default

routliss %tous les lissages se font là

%clear H1 H2 V

% RAPPORTS SPECTRAUX et DEVIATION STANDARD / SPECTRAL RATIO and
STANDARD DEVIATION

%Hliss=(H1liss.^2+H2liss.^2).^0.5; %somme vectorielle
Hliss=H1liss;
HVliss=Hliss./Vliss;
HVlissm=moyenne(HVliss);
HVlissd=HVlissm + StDev(HVliss);

%HV1lissm=moyenne(H1liss./Vliss);
%HV2lissm=moyenne(H2liss./Vliss);

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

affichag % effectue l'affichage principal / main window
seekpeak1 % effectue la recherche DU maximum / seek broadband maximum
values
seekpeak % effectue la recherche DES maxima / seek maximum for each
frequencies band

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

%% -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --
--
-
else % cas ou l'ouverture des fichiers de donnees a echoue
fprintf('\n le programme ne peut pas s'exécuter: manque de
données\n');
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

end % if principal

```

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MS	METU Geological Engineering	2001
BS	METU Geological Engineering	1998
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WORK EXPERIENCE

Year	Place	Enrollment
2000-2006	METU Department of Geological Engineering	Research Assistant

FOREIGN LANGUAGES

English

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HOBBIES

Music, Basketball, Skiing, Computer Technologies, Movies