

A SEMI-EMPIRICAL METHOD FOR ESTIMATION OF MAXIMUM SEISMIC
SHEAR STRAIN IN IMPERFORATED GROUND EXCITED BY VERTICALLY
PROPAGATING SHEAR WAVES

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

BY
KAVEH DEHGHANIAN

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

DECEMBER 2016

Approval of the thesis

A SEMI-EMPIRICAL METHOD FOR ESTIMATION OF MAXIMUM SEISMIC SHEAR STRAIN IN IMPERFORATED GROUND EXCITED BY VERTICALLY PROPAGATING SHEAR WAVES

submitted by **KAVEH DEHGHANIAN** in partial fulfillment of the requirements for the degree of **Doctor of Philosophy in Engineering Sciences Department, Middle East Technical University** by,

Prof. Dr. Gülbin Dural Ünver
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. Murat Dicleli
Head of Department, **Engineering Sciences**

Assoc. Prof. Dr. Mustafa Tolga Yılmaz
Supervisor, **Engineering Sciences Dept., METU**

Examining Committee Members:

Prof. Dr. Murat Dicleli
Engineering Sciences Dept., METU

Assoc. Prof. Dr. Mustafa Tolga Yılmaz
Engineering Sciences Dept., METU

Assoc. Prof. Dr. Ayşegül Askan Gündoğan
Civil Engineering Dept., METU

Assoc. Prof. Dr. Ahmet Hakan Argeşo
Manufacturing Engineering Dept., Atılım University

Assist. Prof. Dr. M. Abdullah Sandıkkaya
Civil Engineering Dept., Hacettepe University

Date: 05.12.2016



I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last name: KAVEH DEHGHANIAN

Signature:

ABSTRACT

A SEMI-EMPIRICAL METHOD FOR ESTIMATION OF MAXIMUM SEISMIC SHEAR STRAIN IN IMPERFORATED GROUND EXCITED BY VERTICALLY PROPAGATING SHEAR WAVES

Dehghanian, Kaveh

Ph.D., Department of Engineering Sciences

Supervisor: Associate Prof. Dr. Mustafa Tolga Yilmaz

December 2016, 116 pages

Seismic design of underground structures requires calculation of the deformation in surrounding geological formations. The seismic hazard on a site is usually expressed as a function of amplitude parameters of free-field motion. Therefore, simplified relations between depth and parameters of ground motion are necessary for preliminary designs. The objective of this study is to employ random vibration theory to develop a simple relationship between maximum shear strain ($\gamma_{\max}$), depth, and ground-motion intensity parameters related to seismic hazard. The ground motion on surface, which is assumed as a stationary random process in the wide sense, is represented by a power spectral density function. Considering one-dimensional shear-wave propagation, transfer functions between ground motion and $\gamma_{\max}$ amplitude at any arbitrary depth is formulated. The theoretical results are compared with the dynamic response of horizontally layered formations to seismic motions defined by accelerograms recorded on ground surface. The effect of material nonlinearity on peak strains is simply modeled by the method of equivalent linearization. It was concluded that the transition from PGA-sensitiveness of $\gamma_{\max}$ to its PGV sensitiveness in uniformly elastic half-space

occurs around the depth of $(PGV \cdot V_s)/(\sqrt{2} \cdot PGA)$. $\gamma_{\max}$ is proportional to d and PGA in very shallow ranges of d , whereas it reaches to figures around $(PGV/V_s)/\sqrt{2}$ by increasing d . $\gamma_{\max}$ is also related to the amplitudes of a pseudo-spectral acceleration. Both relationships can be used as a reasonable first-order estimator for $\gamma_{\max}$ in horizontally layered geological formations if the travel time of vertically incident shear waves to reach from free boundary to the depth of interest is substituted for the parameter d/V_s . The concept of equivalent travel time may yield overestimation of $\gamma_{\max}$ if V_s is increasing by depth. However it leads severe underestimation of $\gamma_{\max}$ in relatively soft layers embedded in stiffer geological formations. The method of equivalent linearization can be used for estimation of $\gamma_{\max}$ if the response of geological layers to shearing is nonlinear.

Keywords: Buried structures, equivalent linearization, random vibration theory, seismic design, shear strain

ÖZ

DÜŞEY YÖNDE İLERLEYEN KAYMA DALGALARIYLA HAREKETE GEÇEN ZEMİNLERDE MAKSİMUM SİSMİK GERİLME TAHMİNİ İÇİN BİR YARI AMPİRİK YÖNTEM

Dehghanian, Kaveh

Doktora, Mühendislik Bilimleri Bölümü

Tez Yöneticisi: Doç. Dr. Mustafa Tolga Yılmaz

Aralık 2016, 116 sayfa

Yeraltı yapıların depreme dayanıklı tasarımında kullanılan basit yöntemler için, çevredeki jeolojik katmanlar üzerinde deformasyonların da hesaplanması gereklidir. Derinlik ve zemin yüzeyindeki deprem hareketinin parametreleri arasında basitleştirilmiş ilişkiler ön tasarımlar için gereklidir. Bu çalışmanın amacı, rasgele titreşim teorisi kullanılarak maksimum birim makaslama uzaması ($\gamma_{\max}$), derinlik ve deprem tehlikesi parametreleri arasında bir ilişki geliştirmektir. Geniş anlamda bir durağan süreç olarak kabul edilen yer hareketinin istatistiksel özellikleri bir güç spektrum yoğunluk fonksiyonu tarafından temsil edilmektedir. Tek boyutlu kayma dalga yayılımı denklemleri dikkate alarak, zemin hareketi genlikleri ve $\gamma_{\max}$ arasında bir ilişki geliştirilmiştir. Zemin yüzeyinde kaydedilen ivmeler ile tanımlanan yatay deprem hareketleri kullanılarak katmanlı jeolojik oluşumların dinamik tepkisi ile basit teorik sonuçlar karşılaştırılmıştır. Jeolojik malzemelerin lineer olmayan davranışının $\gamma_{\max}$ üzerindeki etkisi eşdeğer lineer yöntem ile modellenmiştir. Uniform elastik yarı uzay içinde, PGA duyarlılıktan PGV duyarlılığına geçiş $(PGV \cdot V_s)/(\sqrt{2} \cdot PGA)$ derinliği etrafında ortaya çıkmaktadır. $\gamma_{\max}$ sığ derinliklerde derinlik ve PGA ile orantılıdır. Derinlik arttıkça $(PGV/V_s)/\sqrt{2}$ yakınlığında rakamlara ulaşır.

$\gamma_{\max}$ ile psödospektral ivme gerinlikleri arasında da bir ilişki geliştirilmiştir. Her iki ilişki de yatay tabakalı jeolojik oluşumlarda $\gamma_{\max}$ değerinin bir ön tahmini için kullanılabilir. Bu tahmin yöntemi üniform V_s değerinin derinlikle arttığı profillerde muhafazakar sonuçlar vermektedir. Ancak göreceli sert jeolojik oluşumlar arasında gömülü olan nispeten yumuşak katmanlar içinde tahminler düşük kalmaktadır. Eşdeğer lineer yöntem ile nonlinear tepki veren jeolojik katmanlar için de $\gamma_{\max}$ tahmin edilebilir.

Anahtar Kelimeler: Gömülü yapılar, eşdeğer lineer yöntem, rasgele titreşim teorisi, sismik tasarım, makaslama birim uzama



To my precious family and my dear and beloved Negar...

ACKNOWLEDGEMENTS

I would like to thank my advisor, Assoc. Prof. Dr. Mustafa Tolga Yılmaz for introducing this topic, and for his endless support and guidance during my research and Ph.D. period. I would like to thank my committee members, Prof. Dr. Murat Dicleli, Assoc. Prof. Dr. Ayşegül Askan Gündoğan, Assoc. Prof. Dr. Ahmet Hakan Argeşo, and Assist. Prof. Dr. Mustafa Abdullah Sandıkkaya for their valuable advices and endless supports throughout my thesis study. It has been a great honor for me to meet them.

I would like to express my special thanks to my dear friends for their support. I spent a wonderful time with them which I will keep in my mind and heart.

I express my sincere appreciation to my family for their endless support: my mother, my father, and my brother.

My final gratitude goes to my wife, Negar, for soothing the difficulties of Ph.D. study. Without her encouragement I could not finish my Ph.D.

TABLE OF CONTENTS

ABSTRACT.....	iii
ÖZ	v
ACKNOWLEDGEMENTS	viii
TABLE OF CONTENTS.....	ix
LIST OF TABLES	xii
LIST OF FIGURES	xiii
LIST OF ABBREVIATIONS.....	xvi
CHAPTERS	
1 INTRODUCTION	1
1.2. Literature Review	3
1.2.1. Simplified Estimation of Ground Deformations.....	3
1.2.3. Random Vibration Theory	9
1.2.4. Equivalent Linear Method (EQLM).....	10
1.3. Scope	10
2 FILTERING OF ACCELEROGRAMS.....	13
2.1. Introduction	13
2.2. Mathematics of filtering	13
2.3. Application	16
2.3.1. Low-Pass filtering.....	23
2.3.2. Band-Pass filtering.....	24
3 RELATIONSHIP BETWEEN PGA, PGV AND MAXIMUM SHEAR STRAIN .29	
3.1. INTRODUCTION.....	29

3.2. THEORY	29
3.2.1. The Basic Relationships among Amplitudes	29
3.2.2. An Estimator for $\gamma_{\max}$ in Uniformly Elastic Medium	32
3.3. THE EMPIRICAL DETERMINATION OF THE PARAMETERS	37
3.3.1. The Sample for Empirical Estimation of Parameters.....	39
3.3.2. The Effect of Material Damping	43
3.3.3. Applicability in Layered Geological Formations.....	47
4 AN EMPIRICAL RELATIONSHIP BETWEEN DESIGN SPECTRUM AND MAXIMUM SHEAR STRAIN.....	57
4.1. Introduction	57
4.1. Theory.....	58
4.2. An Approximation for p_{γ}/p_u	63
4.1.1. An assessment of the use of a design spectrum for estimation of $\gamma_{\max}$	67
4.2. Layered Geological Formations	73
5 SITE RESPONSE ANALYSIS.....	75
5.1. Introduction	75
5.2. The Concept of Equivalency in Travel Time	75
5.3. 1-D Site Response Analysis	76
5.4. Deconvolution Analysis	77
5.5. Convolution Analysis	82
5.6. Discussion.....	84
6 CONCLUSIONS.....	87
6.1. Conclusions	87
6.2. Future Works	89
REFERENCES.....	91

APPENDICES

A. Acceleration Time Histories.....	101
B. Velocity and Displacement Time Histories.....	111
C. Procedure of evaluating $\gamma_{\max}$ in layered medium.....	113
CURRICULUM VITAE	115



LIST OF TABLES

TABLES

Table 1. Strain and curvature due to body and surface waves (St. John and Zahrah,1987).	7
Table 2. Ratios of ground motion at tunnel depth to motion at ground surface (FHWA, 2004 and 2009).	8
Table 3. The list of ground-motion records.....	27
Table 4. The physical properties of seismic events in Table 3.	28
Table 5. Site coefficient at short-period range, F_a , according to ASCE 7-05.....	68
Table 6. Site coefficient at $T_n=1$ s, F_v , according to ASCE 7-05.....	68

LIST OF FIGURES

FIGURES

Figure 1-1. Types of deformations on tunnels under seismic actions (a) compression-extension, (b) longitudinal bending deformation, (c) compression of tunnel section (Owen and Scholl, 1981).....	2
Figure 1-2. Apparent propagation velocity of body waves (O'Rourke and El Hmadi, 1988).	4
Figure 1-3. Ovaling and racking deformation on buried structures (after Owen and Scholl, 1981).	5
Figure 2-1. Transfer functions of (a) an ideal low-pass, (b) an ideal high-pass and (c) an ideal band-pass filters.....	14
Figure 2-2. Distribution of PGA and PGV (a) before, and (b) after 15 Hz low-pass filter.	18
Figure 2-3. A comparison between PGV/PGA ratio before and after 15 Hz filtering.	19
Figure 2-4. Velocity time history of pulse-like records (a)#15, (b)#25, (c)#28, (d)#29, and (e)#33.....	19
Figure 2-5. Distribution of spectral acceleration after 15 Hz filtering (a) for site class C and (b) for site class D.	23
Figure 2-6. Fourier amplitude spectra of original and high cut filtered (a) EW, and (b) NS components of record #3 (Table 3) at 15 Hz.	24
Figure 2-7. FAS of records #31 in (a) E-W, and (b) N-S ; #32 in (c) E-W, and (d) N-S; #33 in (e) E-W, and (f) N-S; and #34 in (g) E-W, and (h) N-S directions.....	25
Figure 2-8. Acceleration time history of records #31 in (a) E-W and (b) N-S; #32 (c) E-W and (d) N-S, #33 in (e) E-W and (f) N-S, and #34 in (g) E-W and (h) N-S directions.	26
Figure 3-1. The variability of α due to the ground motion during Düzce earthquake.	38
Figure 3-2. (a) The sample for the relationship between α and d/V_s , and (b) the sample median.	40
Figure 3-3. The precision of Equation (3-29) in uniformly elastic half space.	42

Figure 3-4. The comparison of site B records with Equation (3-29) in uniformly elastic half space.....	42
Figure 3-5. The accuracy of Equation (3-28) in uniformly viscoelastic medium for the cases (a) $\xi = 5\%$, and (b) $\xi = 10\%$ before suppression of high frequencies.....	45
Figure 3-6. The effect of low-pass filter on the accuracy of Equation (3-28) in uniformly viscoelastic media with the damping ratios (a) $\xi = 5\%$, and (b) $\xi = 10\%$	46
Figure 3-7. Velocity profile used in site response analysis.	49
Figure 3-8. The relationship between ξ , modulus degradation ($G/G_{\max}$) and cyclic amplitude of γ	50
Figure 3-9. The precision of Equation (3-28) in estimation of $\gamma_{\max}$ in the layered medium due to (a) first set (rock outcrop), and (b) second set (ground surface) of ground-motion records.	52
Figure 3-10. The relationship between the maximum shear strain in the nonuniform V_s profile and that in uniformly elastic half space due to (a) first set (rock outcrop) , and (b) second set (ground surface) of ground-motion records.....	53
Figure 3-11. The distribution of ξ in the site-response analyses for the records numbered 10 and 31 (Table 3).	54
Figure 3-12. The underestimation of $\gamma_{\max}$ in a layer embedded in stiffer layers.	55
Figure 4-1. Basic elements of an idealized SDOF oscillator.	59
Figure 4-2. The accuracy of Equation (4-25) in uniformly elastic half space.	64
Figure 4-3. The PSA of records showing (a) highest bias (record #12), and (b) lowest bias (record #20) in estimation of $\gamma_{\max}$	65
Figure 4-4. The comparisons of bias in estimation of $\gamma_{\max}$ for (a) record #12, and (b) record #20.....	65
Figure 4-5. A comparison of median $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ for different selections of T_u	66
Figure 4-6. A comparison of median $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ for two different equations.....	67
Figure 4-7. The standard shape of design spectrum according to ASCE 7-05.	68
Figure 4-8. The design spectra for a generic class C site and a generic class D site in California.....	69
Figure 4-9. Consistency between median <i>PSA</i> of generic accelerograms and the design spectrum (a) for a class D site and (b) for a class C site.	71

Figure 4-10. The sample for the accuracy of estimation in case a smooth design spectrum for (a) class D site, and (b) class C site is used for estimation of $\gamma_{\max}$	72
Figure 4-11. The range of $\gamma_{\max}$ consistent with design spectrum as a function of (a) d/V_s , (b) and d	72
Figure 4-12. The precision of Equation (4-25) in estimation of $\gamma_{\max}$ in the layered medium, in case the seismic excitation at a site is defined by a rock outcrop motion.	74
Figure 4-13. The precision of Equation (4-25) in estimation of $\gamma_{\max}$ in the layered medium, in case the seismic excitation at a site is defined by a free field motion.	74
Figure 5-1. Schematic description of control motion on free field and rock outcrop.	76
Figure 5-2. Velocity profile used for the site response analyses for (a) <i>Bell - La Bulk Mail</i> site, for (b) <i>Gilroy #2 (USGS)</i> , and for (c) <i>Palo Alto Veterans Hospital</i>	78
Figure 5-3. The ranges of $\gamma_{\max-l}/\gamma_{\max-u}$ computed by supposing linear response of layers on the sites (a) <i>Bell - La Bulk Mail</i> , (b) <i>Gilroy #2 (USGS)</i> , and (c) <i>Palo Alto Veterans Hospital</i>	80
Figure 5-4. The ranges of $\gamma_{\max-l}/\gamma_{\max-u}$ computed by supposing deconvolution of free-field motion assigned for the sites (a) <i>Bell - La Bulk Mail</i> , (b) <i>Gilroy #2 (USGS)</i> , and (c) <i>Palo Alto Veterans Hospital</i>	81
Figure 5-5. The ranges of $\gamma_{\max-l}/\gamma_{\max-u}$ computed by considering convolution nonlinear response of geological layers on the sites: (a) <i>Bell - La Bulk Mail</i> , (b) <i>Gilroy #2 (USGS)</i> , and (c) <i>Palo Alto Veterans Hospital</i>	83

LIST OF ABBREVIATIONS

α	a depth-dependent normalization factor for $\gamma_{\max}$
ν_0	frequency of zero-crossing for a mean-zero Gaussian process
A	amplitudes of ground motion
a^F	Fourier transforms of acceleration (a)
$a_{\max}$	peak horizontal acceleration at free field
$a(t)$	acceleration time history of support motion
C	the apparent wave velocity
d	depth from ground surface
G	shear modulus
g	acceleration of gravity
G_{sec}	secant shear modulus
$H(\omega)$	transfer function
$H_{\text{sdoF}}(\omega)$	complex frequency response function of a SDOF oscillator
i	unit imaginary number of complex numbers
M_w	moment magnitude
n	order or the pole of the filter
p_a	peak value for a

p_v	peak value for v
p_r	peak values for γ
PGA	peak ground acceleration
PGV	peak ground velocity
PSA	pseudo spectral acceleration
PSD	power spectral density function
PSD_a	PSD of acceleration
r_d	depth-reduction factor for maximum shear stress
S_1	spectral acceleration at 1 sec
SD	spectral displacement
SH	polarized in the horizontal plane
SV	polarized in the vertical plane
S_s	Spectral response accelerations at short periods
T_n	natural period of undamped vibration
T_L	transition period
$u(t)$	absolute displacement of the mass
v^F	Fourier transforms of velocity
$v_{\max}$	the maximum particle velocity
V_s	shear-wave velocity

$x^F(\omega)$	Fourier transforms of $x(t)$
$y^F(\omega)$	Fourier transforms of $y(t)$
γ	shear strain
$\gamma_{\max}$	maximum shear strain in imperforated ground
$\gamma_{\max-c}$	maximum absolute shear strain computed by dynamic response analysis
ν_0	frequency of zero-crossing
ξ	damping ratio
ρ	mass density of the ground
σ_x	root mean square (RMS) of $x(t)$
τ	shear stress
$\tau_{\max}$	maximum shear stress
ϕ	angle of the incidence at the ground surface
ω	wave frequency

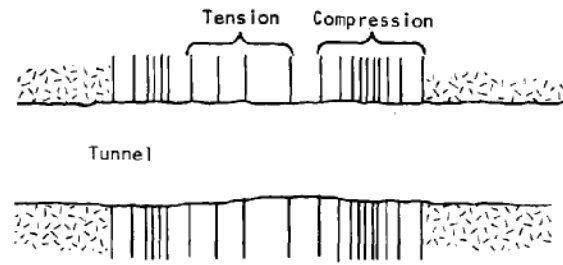
CHAPTER 1

INTRODUCTION

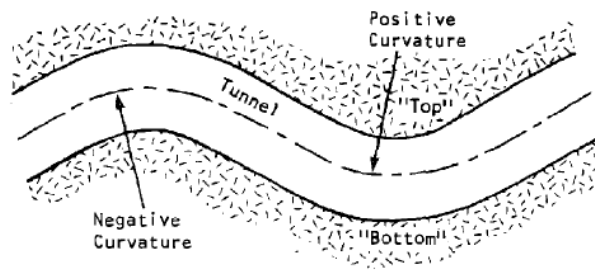
1.1 Introduction

According to their size, shape and location buried structures can be divided into three main categories, such as near surface structures (e.g., pipes and other facilities), large section structures (e.g., tunnels, subways, etc.), and vertical underground structures (e.g., shafts and ducts). Seismic assessments of these structures are important in areas close to severe seismic sources. Recent earthquakes such as 1995 Kobe, 1999 Kocaeli, 1999 Chi-Chi, 2004 Chuetsu, 2005 Kashmir and 2008 Wenchuan showed the seismic vulnerabilities of these structures (Hashash et al., 2001; Aydan et al., 2010).

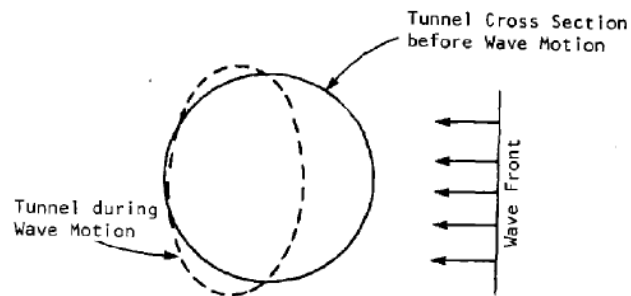
Owen and Scholl (1981) defined the deformation types of underground structures due to seismic excitation as axial compression/extension; longitudinal bending, ovaling and racking deformations (Figure 1-1). Axial deformations are induced by motions parallel to the tunnel axis, and cause compression and tension on structural tunnel elements. Bending deformations are caused by particle motions which are perpendicular to the longitudinal axis. Ovaling and racking deformations occur when shear waves propagate normal or nearly normal to the tunnel axes by causing distortion of tunnel cross section. Hence, simplified methods on seismic design of tunnels conventionally require estimation of maximum shear strain ($\gamma_{\max}$) at depth of tunnel in imperforated ground (Hoeg, 1968 quoted by Hashash, 2001). Therefore, the main objective of this study is to develop a practical theoretical procedure for estimation of $\gamma_{\max}$ at a given depth in ground as a function of seismic-hazard parameters (i.e, PGA, PGV, PSA). The parameters of this theoretical relationship are calculated semi-empirically. So, a set of semiempirical relationships are derived by considering a uniformly elastic half space. Then, a calculation procedure that relates the distribution of $\gamma_{\max}$ in layered geological formations to that in uniform half space is proposed.



(a)



(b)



(c)

Figure 1-1. Types of deformations on tunnels under seismic actions (a) compression-extension, (b) longitudinal bending deformation, (c) compression of tunnel section (Owen and Scholl, 1981).

1.2. Literature Review

The simple relationships widely used for calculation of $\gamma_{\max}$ in seismic design of tunnels are presented. A literature review on the applicability of random vibration theory in theoretical earthquake engineering is presented. Since these relationships require material linearity, the studies on the applicability and limitations of the method of equivalent linearization, which is widely used for nonlinear dynamic problems in the field of geotechnical earthquake engineering, are presented.

1.2.1. Simplified Estimation of Ground Deformations

The simplified method for seismic design of tunnels is generally based on the assumption that the strain field in the perforated ground is the same as that in the imperforated ground (this method is named as free field approach). The interaction between structure and ground is neglected, and it is assumed that structures conform to the imposed deformations from surrounding ground. The diametric strain, the maximum thrust, and the bending moment on tunnel section can be expressed as a function of the maximum shear strain ($\gamma_{\max}$) in imperforated ground (Wang, 1993).

Since a direct measure of ground strains is not generally available, a relationship between $\gamma_{\max}$ and peak ground velocity (PGV) in free-field is typically used for seismic design (FHWA, 2004). These relationships are principally based on the solution of Newmark (1968). Newmark considered one-directional propagation of harmonic wave in a homogeneous, isotropic and elastic unbounded medium. Consequently, the relationship between the maximum particle velocity ($v_{\max}$) and $\gamma_{\max}$ is

$$\gamma_{\max} = v_{\max} / C \quad (1-1)$$

where C is the apparent wave velocity (Newmark, 1968).

The appropriate selection of C is not straightforward and different suggestions have been presented in literature. The practical rules and findings for selection of C can be contradictory because $\gamma_{\max}$ is related to longitudinal deformation in horizontal plane instead of deformation in vertical plane in some of the documents (e.g., O'Rourke and Elhmadi, 1988; Paolucci and Smerzini, 2008). The parameter C is also dependent on the

type of waves, on the angle of incidence, on the spatial variability of material parameters, and on the plane pertinent to $\gamma_{\max}$ (St.John and Zahrah, 1987). Seismic energy originating at depth passes through increasingly softer materials, such that refraction of waves causes a concave travel path. Consequently, the body waves arrive at the ground surface with a small angle of incidence. If the angle of the incidence at the ground surface is ϕ and the shear wave velocity of the top layer is V_s , as shown in Figure 1-2, then the apparent propagation velocity of disturbance due to SV (polarized in the vertical plane) or SH (polarized in the horizontal plane) waves with respect to the ground surface will be (O'Rourke and Elhadi, 1988).

$$C = \frac{V_s}{\sin \phi} \quad (1-2)$$

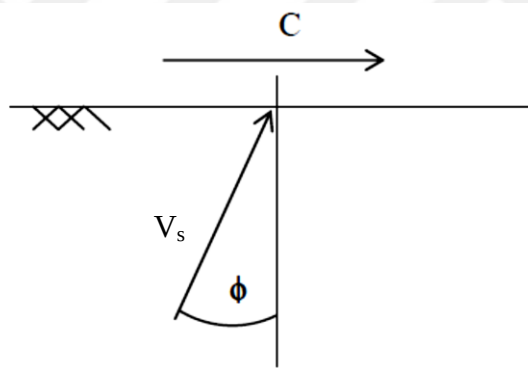


Figure 1-2.Apparent propagation velocity of body waves (O'Rourke and El Hmadi, 1988).

Equation (1-2) is useful in calculation of longitudinal deformation on buried pipes. O'Rourke and El Hmadi (1988) also proposed that apparent propagation velocity with respect to the ground surface is somewhere in the range of 2 to 5 km/sec for a site subject to body wave propagation only. Whereas figures around 1000 m/s or more can be taken according to the data from dense network of strong-motion stations according to Paolucci and Pitilakis (2007). According to Eurocode 8 Part 4 for buried pipelines (CEN, 2006), C is the "apparent wave speed" and the selection of the wave type shall be made "based on geophysical considerations". Therefore, the practical application of Equation (1-1) is subject to numerous, and partly arbitrary assumptions, mainly due to

the lack of sound methods for transient ground strain evaluation, and due to the lack of a comprehensive set of experimental validations.

Ovaling and racking deformations are correlated with $\gamma_{\max}$ on a vertical plane, so C is close to C_s , the incident horizontal shear-wave velocity in geological layers. The consequent structural deformations are basically related to $\gamma_{\max}$ in imperforated ground as shown in Figure 1-3 (Wang, 1993; Kontoe et.al, 2011).

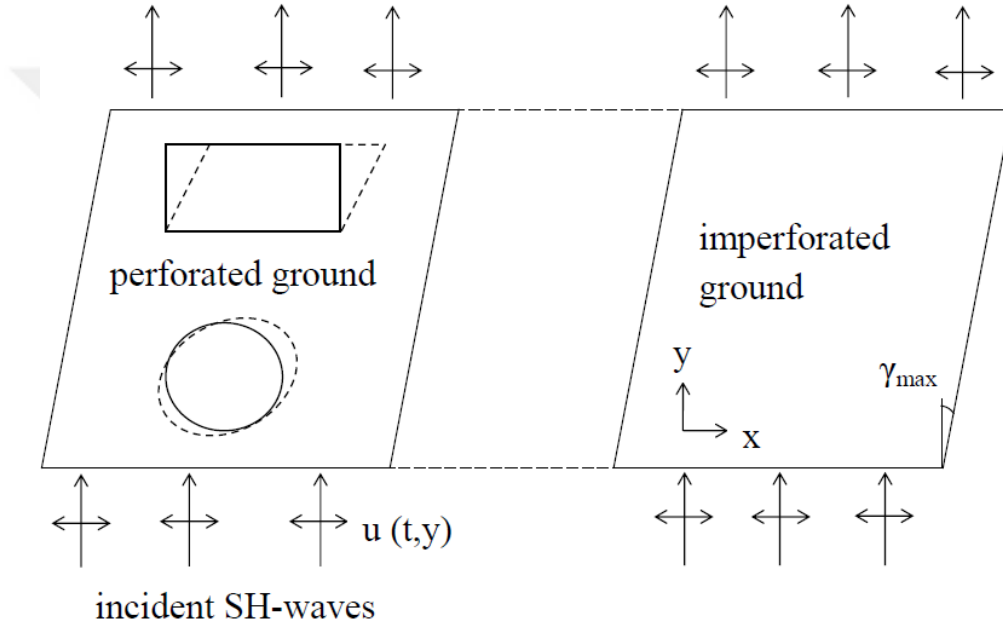


Figure 1-3. Ovaling and racking deformation on buried structures (after Owen and Scholl, 1981).

Wang (1993), considering the ovaling deformation, defined the parameter C as effective shear wave propagation velocity. Hence, C was related to effective shear modulus, G , and the mass density of the medium, ρ by

$$C = \sqrt{\frac{G}{\rho}} \quad (1-3)$$

The term C used in Equation (1-3) should reflect degradation of shear modulus due to softening of soil when a large range of strains is of concern. This was not indicated in Newmark's study. Hence, after substitution of Equation (1-3), Equation (1-1) can be

used as an estimator for $\gamma_{\max}$ around deep tunnels by assuming that body waves are propagating in one direction. However, the depth limit validating the assumptions for a “deep tunnel” is yet to be quantified for seismic design. Moreover, Equation (1-1) may not yield reasonable accurate estimates for $\gamma_{\max}$ in layered geological formations. Although these simplified formulas are adopted by most of the available technical guidelines (e.g. ALA, 2001; CEN, 2006; FWHA, 2009), their use may be questionable.

St. John and Zahrah (1987) utilized Newmark's equation to develop a set of simple theoretical formulas for evaluation of the longitudinal, normal and shear strains in free field, and for estimation the curvature on an axis by considering propagation of P-, S-, and Rayleigh waves. These formulas are shown in Table 1. The type of wave dominating ground strains is difficult to identify because of the complex nature of seismic excitation. Generally, strains produced by Rayleigh waves may govern only if the site is at a large distance from the earthquake source, and also if the buried structure is built in shallow ranges of depth (Wang, 1993).

If the shear waves are supposed to propagate vertically in a uniformly elastic halfspace, $\gamma_{\max}$ for a specific ground motion is a function of d/V_s , the ratio of depth below free boundary to shear-wave velocity in medium (Chen and Hou, 1992). In layered medium, the equivalent travel-time concept proposed by Imai et al. (1981) for estimation of maximum shear-stress ($\tau_{\max}$) may be used. This is based on the assumption that $\tau_{\max}$ is only a function of the travel time of shear waves propagating vertically from the free boundary to a depth of interest regardless of the distribution of V_s in layered medium. The travel time is equal to d/V_s in a linearly elastic half-space. Consequently, $\gamma_{\max}$ can be calculated by dividing $\tau_{\max}$ by the secant shear modulus of material G_{sec} , representing the average stiffness in a range of shear strain.

$$\gamma_{\max} = \frac{\tau_{\max}}{G_{\text{sec}}} \quad (1-4)$$

Table 1. Strain and curvature due to body and surface waves (St. John and Zahrah,1987).

Wave Type		Axial Strain	Shear Strain	Curvature
P-wave		$\epsilon = \frac{V_P}{C_P} \cos^2 \phi$	$\gamma = \frac{V_P}{C_P} \sin \phi \cos \phi$	$\frac{1}{\rho} = \frac{a_P}{C_P^2} \sin \phi \cos^2 \phi$
		$\epsilon_{\max} = \frac{V_P}{C_P} \text{ for } \phi = 0^\circ$	$\gamma_{\max} = \frac{V_P}{2C_P} \text{ for } \phi = 45^\circ$	$\frac{1}{\rho_{\max}} = 0.385 \frac{a_P}{C_P^2} \text{ for } \phi = 35.27^\circ$
S-wave		$\epsilon = \frac{V_S}{C_S} \sin \phi \cos \phi$	$\gamma = \frac{V_S}{C_S} \cos^2 \phi$	$\frac{1}{\rho} = \frac{a_S}{C_S^2} \cos^3 \phi$
		$\epsilon_{\max} = \frac{V_S}{2C_S} \text{ for } \phi = 45^\circ$	$\gamma_{\max} = \frac{V_S}{C_S} \text{ for } \phi = 0^\circ$	$\frac{1}{\rho_{\max}} = \frac{a_S}{C_S^2} \text{ for } \phi = 0^\circ$
R-wave	Compressional Component	$\epsilon = \frac{V_R}{C_R} \cos^2 \phi$	$\gamma = \frac{V_R}{C_R} \sin \phi \cos \phi$	$\frac{1}{\rho} = \frac{a_R}{C_R^2} \sin \phi \cos^2 \phi$
		$\epsilon_{\max} = \frac{V_R}{C_R} \text{ for } \phi = 0^\circ$	$\gamma_{\max} = \frac{V_R}{2C_R} \text{ for } \phi = 45^\circ$	$\frac{1}{\rho_{\max}} = 0.385 \frac{a_R}{C_R^2} \text{ for } \phi = 35.27^\circ$
	Shear Component		$\gamma = \frac{V_R}{2C_R} \cos \phi$	$\frac{1}{\rho} = \frac{a_R}{C_R^2} \cos^2 \phi$
			$\gamma_{\max} = \frac{V_R}{C_R} \text{ for } \phi = 0^\circ$	$\frac{1}{\rho_{\max}} = \frac{a_R}{C_R^2} \text{ for } \phi = 0^\circ$

where:

V_P = soil particle velocity caused by P-waves
 a_P = soil particle acceleration caused by P-waves
 C_P = apparent propagation velocity of P-waves
 V_S = soil particle velocity caused by S-waves
 a_S = soil particle acceleration caused by S-waves
 C_S = apparent propagation velocity of S-waves
 V_R = soil particle velocity caused by R-waves
 a_R = soil particle acceleration caused by R-waves
 C_R = propagation velocity of R-waves
 $1/\rho$ = curvature

An appropriate choice for $v_{\max}$ is still unclear in calculation of ovaling deformation. $v_{\max}$ is frequently assumed to be equal to the Peak Ground Velocity (PGV) in free field (O'Rourke and El Hmadi, 1988; Paolucci and Smerzini, 2008). In a document proposed by FHWA (2004), a reduction coefficient is defined which relates “ground motion” at ground surface to “ground motion” at depth based on an unknown citation. Moreover, the term “ground motion” is ambiguous and the applicability of these coefficients for different depth ranges and soil types are therefore questionable. PGV is also defined as the product of Peak Ground Acceleration (PGA) and an empirical factor, while PGV is

strongly correlated with spectral acceleration at 1 sec (S_1) (FHWA, 2009). This correlation is based on earthquake database gathered from accelerograms.

Table 2. Ratios of ground motion at tunnel depth to motion at ground surface (FHWA, 2004 and 2009).

Tunnel Depth (m)	Ratio of Ground Motion at Tunnel Depth to Motion at Ground Surface
≤ 6	1.0
6 to 15	0.9
15 to 30	0.8
> 30	0.7

For tunnels with shallow burial depths, maximum shear stress can be estimated by the product of PGA in ground surface and overburden pressure (FHWA, 2009). This product is corrected by an empirical depth-reduction factor (r_d) due to the deformability of medium (Seed and Idriss, 1971). In this method, maximum shear stress (on a horizontal plane) at depth d is

$$\tau_{\max} = PGA \cdot \rho \cdot d \cdot r_d \quad (1-4)$$

such that ρ is the density of the shallow geological formation, and d is the depth of interest (FHWA, 2009). Then, $\gamma_{\max}$ can be estimated by Equation (1-3).

Hence, in shallow ranges of depth, $\gamma_{\max}$ is sensitive to PGA instead of PGV in the case that the seismic disturbance is mostly due to vertically incident shear waves (SH-, or SV-). Thus, a unified simple relationship between $\gamma_{\max}$, PGA, PGV and depth is necessary for estimation of the ovaling and the racking deformations of structures located in shallow geological formations. The problem can be solved by using the random vibration theorem.

1.2.3. Random Vibration Theory

The Random Vibration Theory (RVT) relates the statistical properties of random response of a dynamic system to those of system properties, or to those of random excitation. Therefore, RVT can be used for statistical estimation of random response of a system by representing the ground motion by a power spectral density function (PSD). Simplified theoretical conclusions are possible by the assumption that the ground motion is a stationary (i.e. statistical properties of the motion are constant in time) Gaussian process. Although earthquake excitations are nonstationary, the strong phase of such motions can be supposed as stationary (Der Kiureghian, 1980). In this approach, the excitation is first defined by a PSD. The response PSD is either theoretically expressed or computed by using the transfer functions. Then, the statistical properties of response are estimated by using its PSD.

One known example on the use of RVT for developing theoretical solutions is the Complete Quadratic Combination (CQC) method, which is useful for estimating the peak values of displacements or forces within a structure (Wilson et.al, 1981; Der Kiureghian and Neuenhofer, 1991). CQC is also used for analyzing the non-stationary random responses of complex structures located in an inhomogeneous stochastic field (Lin et al., 1997). Seismic-response analysis of linear multi support-structural systems can be formulated by RVT which accounts for the multi-support input (Heredia-Zavoni and Vanmarcke, 1994).

The stationary filtered white noise model presented by Kanai and Tajimi (Kanai, 1957; Tajimi, 1960) yields a known PSD in the field of earthquake engineering. White noise is a stationary random process that has a zero mean and a constant spectral density for all frequencies. In the Kanai-Tajimi spectral model, the rock acceleration is assumed as a white noise process and the overlying soil deposits are simulated by a linear single-degree-of-freedom system. Modified Kanai-Tajimi models are also proposed in literature (Clough and Penzien, 1993). Hence, RVT can be used to build simple theoretical solutions. On the other hand, these simple solutions are restricted to linear systems.

1.2.4. Equivalent Linear Method (EQLM)

The simplified analyses on wave propagation and the use of RVT for development of basic relationships between amplitudes of motion require linearity in material and behavior. The effect of nonlinear material behavior on dynamic response can be approximated by using the method of equivalent linearization. In this method, the representative values for secant shear modulus and the damping ratio are modified until they are consistent with the level of the strain induced in the soil (Schnabel et al., 1972).

Equivalent linear method is among the most widely used methods for analyses of dynamic site response (Yoshida, et al., 2002). The main disadvantage of EQLM is that its predictions become more biased by increasing strains. There are always severe differences between the results of EQLM and those of transient nonlinear analysis for soft soils, or for soils prone to liquefaction (EPRI, 1988). A suggestion for the limit of $\gamma_{\max}$ in equivalent linear method is 1% (Gerolymos and Gazetas, 2005; Stewart and Kwok, 2008). So, the method is useful for a limited range of shear strain. The use of equivalent linear method in analytical solutions as an approximate way of resembling soil's nonlinearity can be considered for a preliminary seismic design. Moreover, the experimental studies have been presented in literature to show the consistency of the equivalent linear method with experimental studies on the tunnel behavior under seismic excitation (Amorosi et.al, 2014).

1.3. Scope

$\gamma_{\max}$ is related to ovaling deformation of circular tunnels, so several theoretical relationships were proposed in the literature to assess $\gamma_{\max}$. However, these equations are not united to build a single relationship between PGA, PGV, and $\gamma_{\max}$, which is applicable for both shallow buried structures and deep structures, so that the assumptions regarding the depth of tunnel can be avoided in seismic design. In this study, semi-empirical relationships for estimation of $\gamma_{\max}$ at any given depth in an elastic half space are developed. The relationships are based on the use of RVT, in order to relate $\gamma_{\max}$ to the parameters PGA, PGV and ordinates of a design spectrum. Because the relationships are limited to uniformly elastic media, simple methods which are based on EQLM and

on the concept of equivalent travel time are proposed to extend their applicability to the cases of layered geological formations.

This study includes six chapters. In Chapter one, introduction, objectives and organization of the study is presented. Simplified analysis methods suggested in literature are summerized.

In Chapter two, filtering of ground-motion records for dynamic- response analysis is explained. The filtered records are used as a sample for estimations of parameters in chapter three and four.

In Chapter three, a theoretical relationship based on RVT is derived to compute $\gamma_{\max}$ in homogeneous soil. An approximate relationship between the distribution of $\gamma_{\max}$ in uniformly elastic half space and that in layered medium which respond nonlinearly to seismic excitation is proposed. Then, this formula is compared with 1-D seismic site response analyses involving a generic layered V_s profile.

Chapter four represents another semi-empirical relationship between $\gamma_{\max}$ and spectral amplitudes.

In Chapter five, three actual soil profiles are introduced, and accuracy of the relationship between $\gamma_{\max}$ in uniform and layered medium is further investigated.

Chapter six includes summary of this study and most important findings.



CHAPTER 2

FILTERING OF ACCELEROGRAMS

2.1. Introduction

This chapter presents the filtering of accelerograms that were used for estimation of model parameters and for justification of theoretical functions in Chapter 3 and Chapter 4. There are different types of digital filters to remove noise from signals such as the Bessel, the Chebychev and the Butterworth filter.

The filtered velocity and displacement functions which are computed by integration of filtered accelerograms should attenuate down to zero by increasing time. The acceleration, velocity and displacement should start at zero, implying a complete at-rest condition before ground motion.

2.2. Mathematics of filtering

A filter response function has a value close to 1 in a frequency range that carries useful information, and close to 0 in the range carrying useless information. An ideal filter should be 1 in the desired range and zero in elsewhere. An ideal filter can be highpass, band pass and low pass, which are shown in Figure 2-1.

The filter can be applied in the frequency domain by multiplying the filter transfer function ($H(\omega)$) with the Fourier transform of time series ($X(\omega)$). This results in a filtered time series $Y(\omega)$ after transforming the data of time domain by the inverse Fourier transform, such that (Haykin and Moher, 2007)

$$Y(\omega) = H(\omega) \cdot X(\omega) \quad (2-1)$$

where $Y(\omega)$ is Fourier Transform of output $y(t)$.

An alternative procedure for filtering is the use of convolution integral in time domain (O’Neil, 2012). Convolution is a mathematical operation which integrates two functions and produces a third function.

$$(f * g)(t) = \int_0^t f(t - \tau)g(\tau)d\tau \quad (2-2)$$

An important signal processing operation frequently encountered in communication systems is that of linear filtering. This operation involves the convolution of the input signal with the impulse response of the filter; or, equivalently, the multiplication of the Fourier transform of the input signal by the transfer function (i.e., Fourier transform of the impulse response) of the filter (Haykin and Moher, 2007).

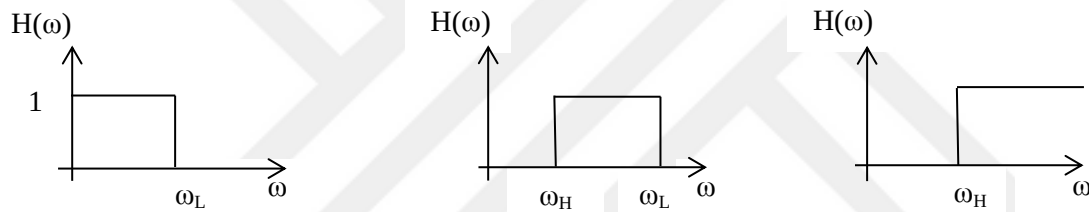


Figure 2-1. Transfer functions of (a) an ideal low-pass, (b) an ideal high-pass and (c) an ideal band-pass filters.

There are two main types of filtering, analog and digital. The analog filters are built using inductors, capacitors, resistors, and operational amplifiers. The advantage of analog filters is the simplicity of continuous implementation of filtering (Haykin and Moher, 2007). A digital filter uses a digital processor to perform numerical calculations on a discretely recorded (sampled) sample (Cunningham, 1995). Digital filters have many advantages because of their flexibility and economy, particularly when operating at relatively low frequencies (Kanasewich, 1981). All these filters approximately simulate the ideal filter.

Record filtering is an iterative method (Akkar and Bommer, 2006). First, Fourier Amplitude Spectrum (FAS) of accelerogram is plotted, and the spectrum is inspected visually. High-pass (low-cut) filtered data follows a trend which is proportional to frequency- squared (f^2) gradient. Low-pass (high-cut) filtered data follows a trend

proportional to $1/f^2$ gradient. The velocity and displacement time histories should be controlled after filtering.

A high-pass (low-cut) filter is a filter which allows the high-frequency amplitudes to pass through the filter. It is used to remove low-frequency amplitudes from a signal. A low-pass (high-cut) filter is a filter which allows the low-frequency amplitudes to pass through the filter. It is used to remove high-frequency amplitudes from a signal so it attenuates signals with frequencies higher than the cutoff frequency. The amount of attenuation for each frequency depends on the filter type. Low-pass filters provide a smoother form of a signal by removing the short-term fluctuations, and leaving the longer-term trend. In general, FAS of the accelerograms are inspected in the low and high frequency regions to determine the cut-off frequencies of filters (Kanasewich, 1975).

A band-pass filter may be constructed by using a high-pass filter and low-pass filter in series. In fact, band-pass filter is the combination of high-pass and low-pass filters. A band-pass filter attenuates frequencies outside of a certain frequency interval (Kanasewich, 1975). Although the idealized filters cannot be built, practical approximations can be found (Cunningham, 1995). One of these approximations is Butterworth filter.

The Butterworth filter is a type of signal processing filter which has a maximally flat magnitude range. The transfer function $H(\omega)$ of an n -order Butterworth low pass filter as (Cunningham, 1995).

$$|H(\omega)| = \frac{1}{1 + (\omega^{2n})^{1/2}} \quad (2-3)$$

where n is the order or the pole of the filter. By increasing n , the transfer function approaches to an ideal filter. The values of n that make the $H(\omega)$ be equal to zero are called zeros of the filter. The Butterworth filter approaches to zero only by increasing ω . The singularities of a filter transfer function are called poles of the filter (Cunningham, 1995). The flattest pass-band response belongs to Butterworth filter (Boore and Akkar, 2003; Akkar and Bommer, 2006). The order of a filter controls the roll off. Roll off is

the rate at which attenuation increases beyond the cut-off frequency. The roll-off becomes faster by increase in filter order. In this study, 4 pole / 4 pole Butterworth was used as a noise filter.

A system can be named as causal or realizable if it does not respond before the excitation is applied. For a linear time-invariant system to be causal, it is clear that the impulse response must vanish for negative time. Otherwise it is acausal (Haykin and Moher, 2007). There are no significant differences in the conditionally simulated acceleration time histories processed with either causal or acausal filtering (Zerva, 2009). In this study, filtering was done in MATLAB. Filter function performs causal filtering, so a moving average filter slides a window along the data, computing averages of the data (Mathworks, 2016).

2.3. Application

A set of filtered strong-motion was selected as input data and high-cut and band-pass filters were applied as it will be discussed in the following sections.

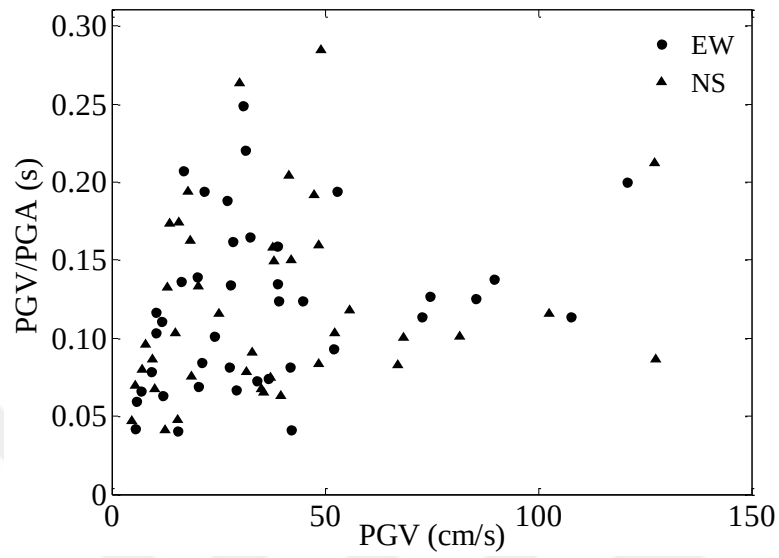
2.3.1. The Strong Ground Motion Database

A number of accelerograms were selected from the strong-motion databases PEER (2014) and DAPHNE (2014) to gather a sample for empirical estimation of parameters. The emphasis was placed on covering a reasonable range of the ratio PGV/PGA, and on using the records that involve significant ground-motion amplitudes for buried structures. Consequently, the moment magnitudes (M_w) of the earthquakes were between 5 and 9. The focal depth was limited by 20 km. The recording instruments are on free fields. The PEER database provides filtered records, and the pass-band frequencies are presented in Table 3. The DAPHNE records were filtered by using the Butterworth filters with pass-band frequencies shown in Table 3. The records are attributed to the seismic events presented in Table 4. The accelerograms of these records are shown in Appendix A. In this study, filtered acceleration time histories were gathered from PEER and Daphne databases (PEER, 2014; Daphne, 2014). Records adopted from Daphne

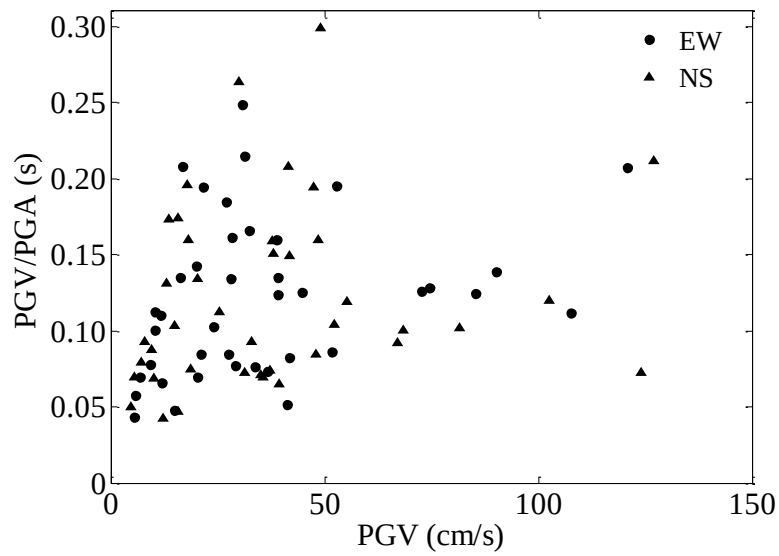
database were filtered again due to problems in velocity or displacement time histories so they would start and end at rest.

The filtering limits were selected as they were proposed by Daphne database. As it will explain in Chapter 3, the accelerograms to be used as ground motion in deconvolution analyses should be filtered by applying a 15 Hz low-cut filter to eliminate the tendency of developing unrealistic results at depth. Therefore, records were High-Cut filtered at 15 Hz.

Figure 2-2.a shows the sample of PGV/PGA, generally between 0.04 and 0.28 s with a median value of 0.10 s. PGA and PGV in each horizontal direction is not less than 0.05 g and 4 cm/s respectively for all records. Figure 2-2.b shows this ratio after filtering by 15 Hz low-pass filter. Figure 2-3 shows a comparison between PGV/PGA before and after 15 Hz filtering. Boore and Akkar (2003), and Boore and Bommer (2005) stated that spectral and peak ground values are less sensitive to the chosen filter cut-off when the data is filtered by acausal filters, but as it is shown in the figures, peak values do not change considerably after causal filtering. Near field ground motions are characterized by velocity or displacement pulse-like time histories occurring at the beginning of records (Somerville et.al, 1997). PEER database involves pulse-like records. These records were identified and particularly marked in this database. On the other hand, the records in Daphne were not distinguished according to near-field effects. Therefore, the Daphne records were plotted and velocity histories were inspected visually. Appendix B shows the velocity and displacement time histories of #32 and #33 which are identified as the records involving velocity pulses. The near-field records are marked by asterisk in Table 3. Due to different behavior of these records, special attention should be paid to them. In Figure 2-3, red points show the range of PGV/PGA for near-field ground motions before and after filtering. Figure 2-4 shows the velocity pulses of sample records in E-W and N-S directions. No significant distribution between the ranges of PGV/PGA in records showing pulse-like time histories could be observed. The effect of 15-Hz filtering on PGV/PGA is observed to be unrelated to the existence of near-field velocity pulses.



(a)



(b)

Figure 2-2. Distribution of PGA and PGV (a) before, and (b) after 15 Hz low-pass filter.

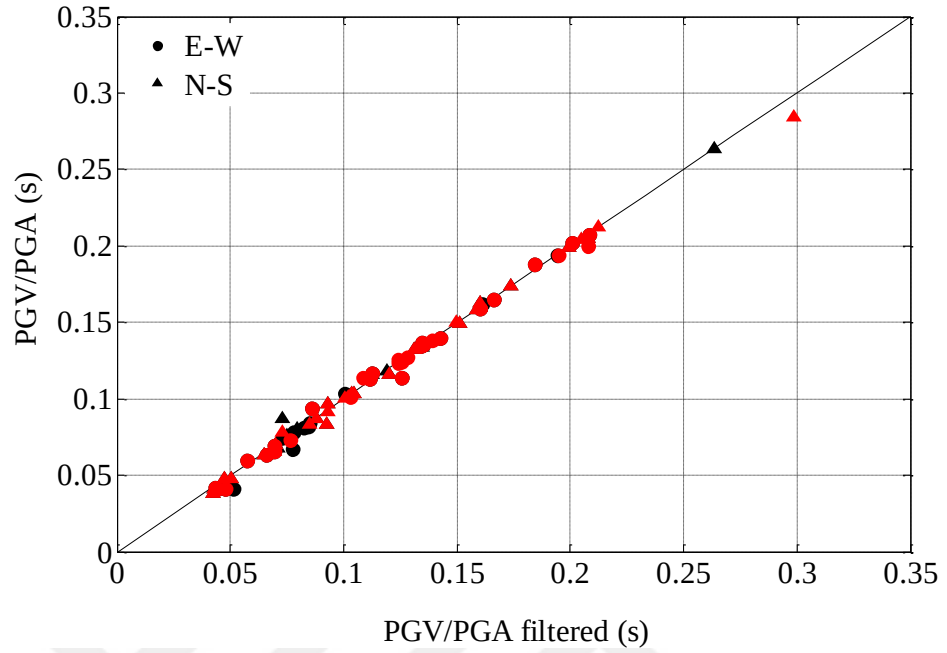
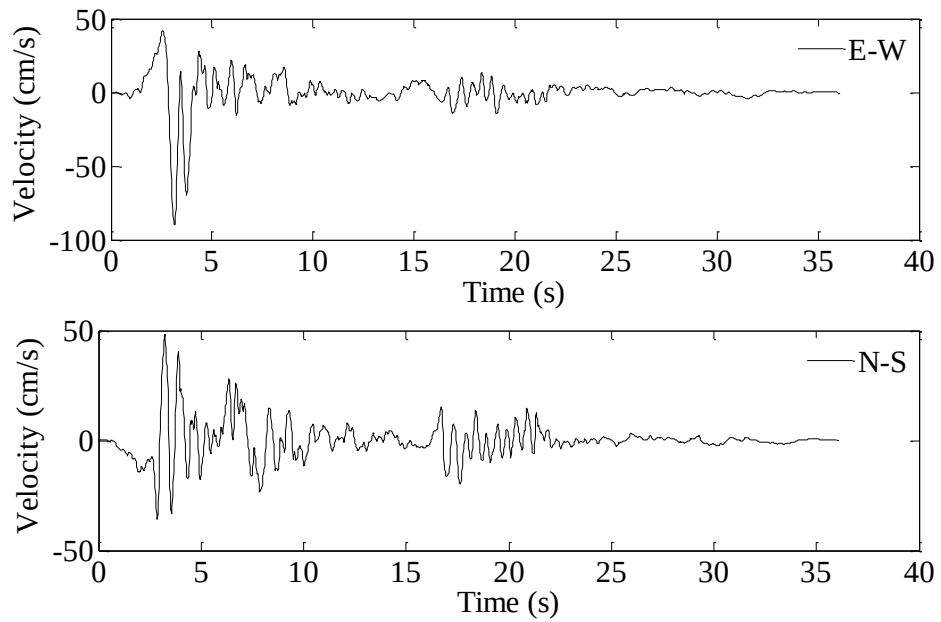
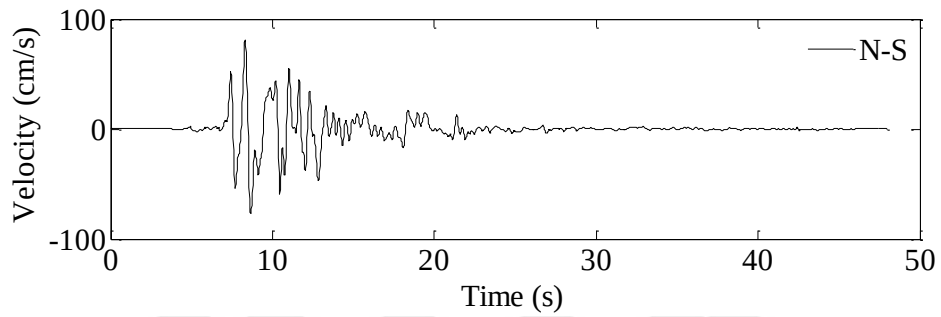
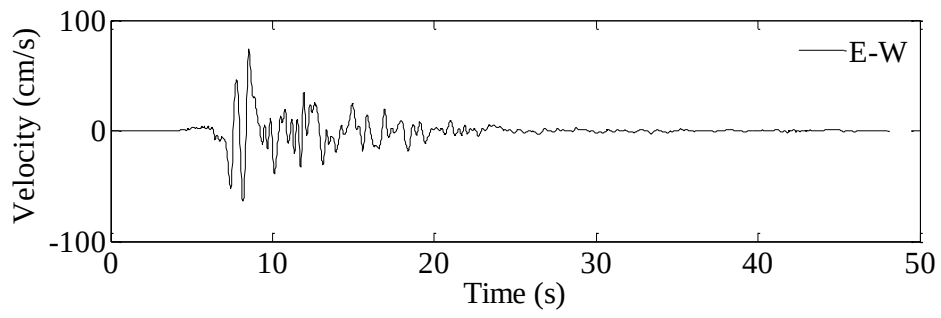


Figure 2-3. A comparison between PGV/PGA ratio before and after 15 Hz filtering.

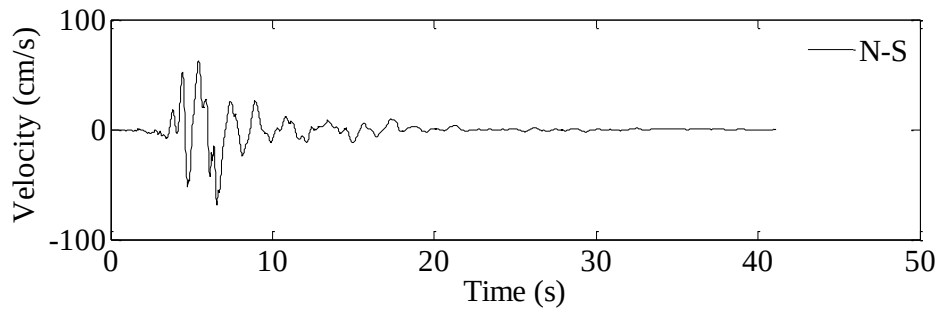
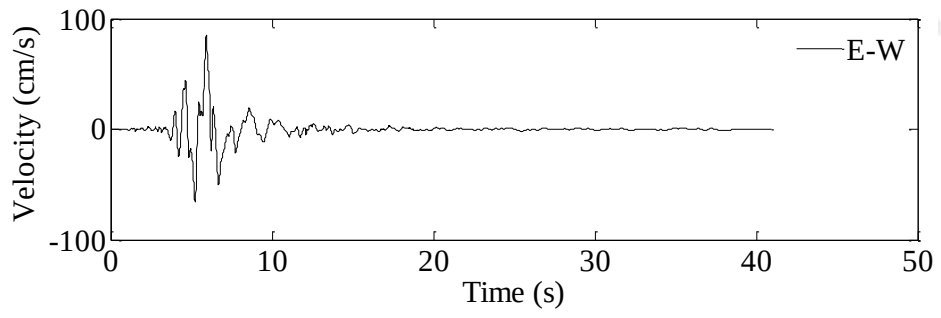


(a)

Figure 2-4. Velocity time history of pulse-like records (a)#15, (b)#25, (c)#28, (d)#29, and (e)#33.

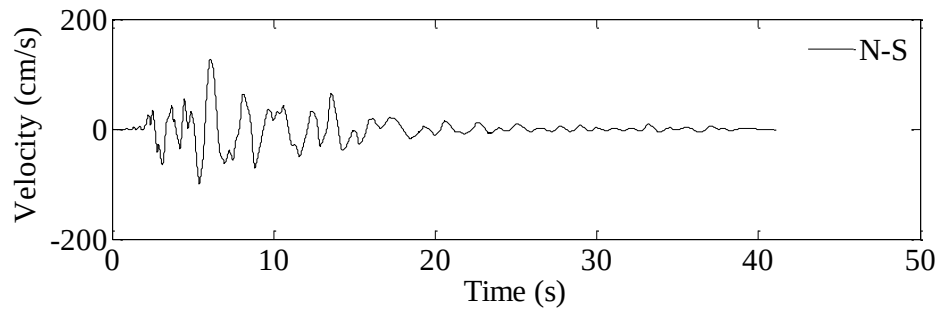
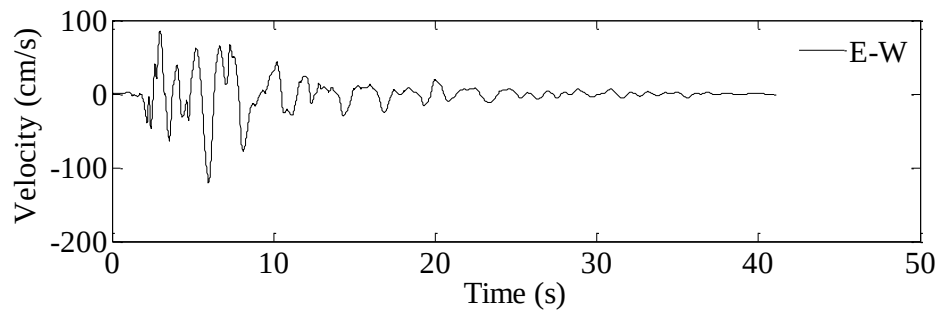


(b)

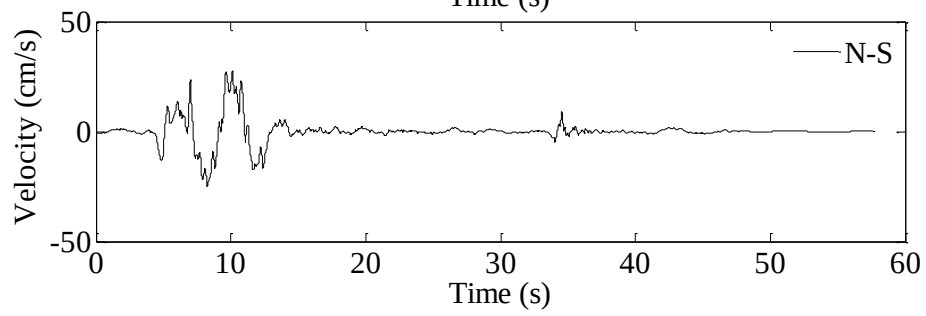
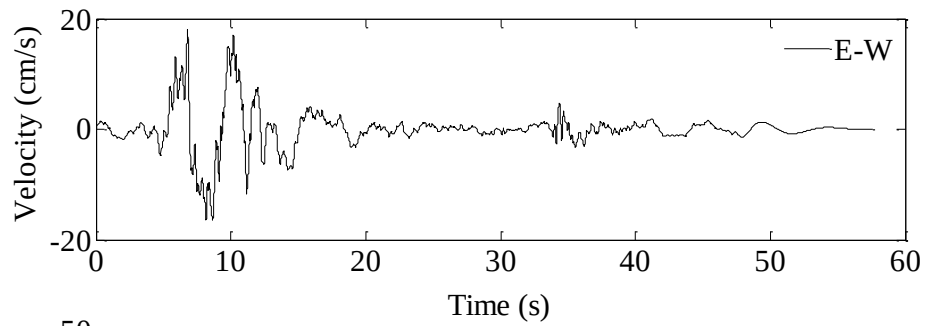


(c)

Figure 2-4. Continued.



(d)



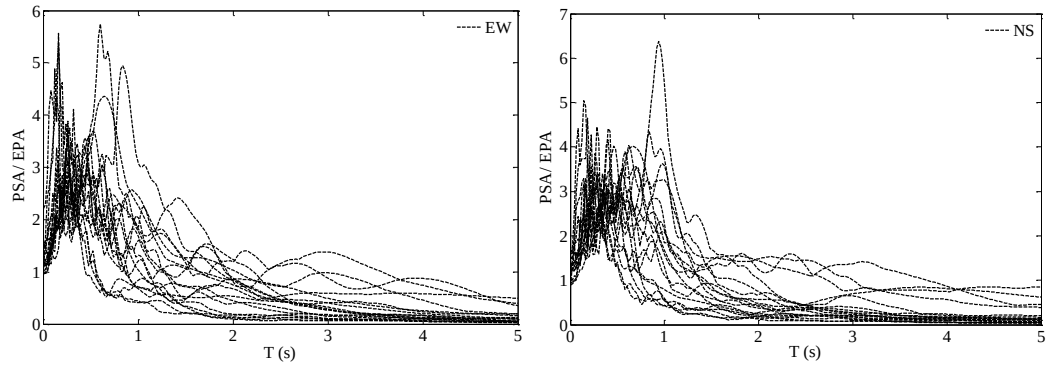
(e)

Figure 2-4. Continued.

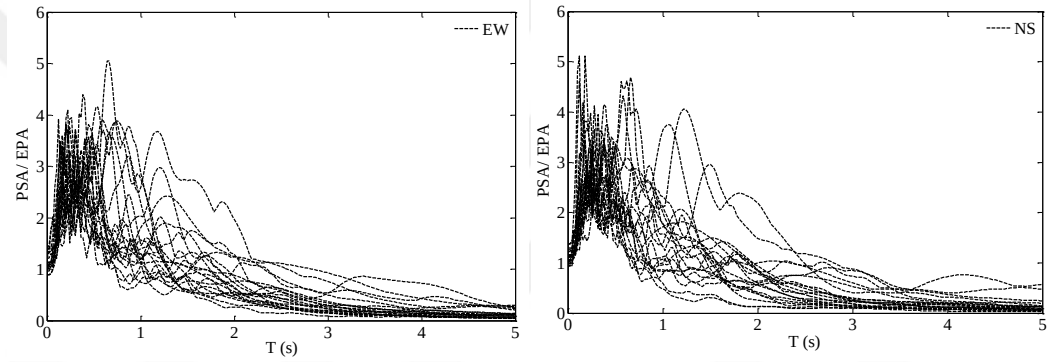
In Figures 2-5.a and 2-5.b, 5% damped acceleration response spectrum (PSA) was computed for each record. Each response spectrum was normalized by its effective ground acceleration, EPA, which is defined as the average of 5% damping spectral acceleration over the period range 0.1 to 0.5 s divided by 2.5 (Kramer, 1996).

$$EPA = \frac{1}{2.5} \int_{0.1}^{0.5} PSA(T, 5\%) dT \quad (2-4)$$

The shapes of normalized acceleration spectra show that a wide range of frequency amplitude contents of motion is used in analyses. The short-period range of spectra is consistent with the shape of design spectrum at short periods. The sites of records were classified according to the site classification system of International Building Code (IBC, 2009). This widely known system is related to the parameter V_{s30} , the weighted harmonic mean of V_s to the depth of 30 m (Borcherdt, 1994). The site class C is pertinent to very dense soils or soft rocks ($360\text{m/s} \leq V_{s30} \leq 760\text{m/s}$), and the class D depicts stiff-soil ($180\text{m/s} \leq V_{s30} \leq 360\text{m/s}$) conditions. The site class A was not selected due to the lack of recorded data. Site class E was not selected because it represents the site conditions and may affect the analysis results. Figures 2-5.a and 2-5.b show that the shapes of normalized PSA depend on the site class. Site class affects the frequency content of ground motions. The Records from class D sites show higher amplitudes in higher periods than those for class C sites. The records pertinent to site class B are also used for analyses for homogeneous medium in order to enhance the range of PGV/PGA, whereas only records belonging to site classes C and D were used in analyses for layered media because of the coherence between site conditions and record properties. The records from class B sites do not show a particular effect on the results explained in Chapter 3.



(a)



(b)

Figure 2-5. Distribution of spectral acceleration after 15 Hz filtering (a) for site class C and (b) for site class D.

2.3.1. Low-Pass filtering

In Chapter 3, the strong ground-motion records were filtered by a low-pass filter with a high-cut frequency of 15 Hz. Figures 2-6.a and 2-6.b show the Fourier Amplitude Spectrum (FAS) of a sample record before and after low-pass filtering.

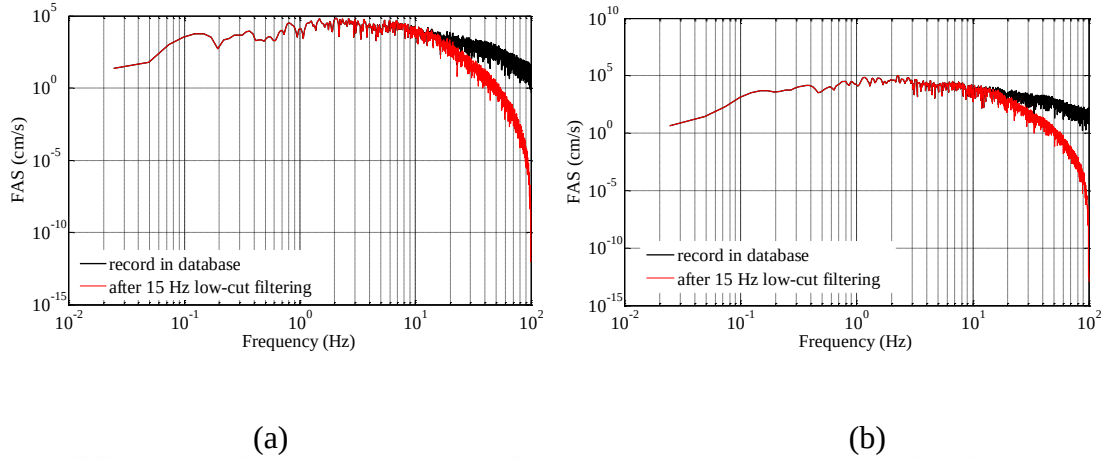
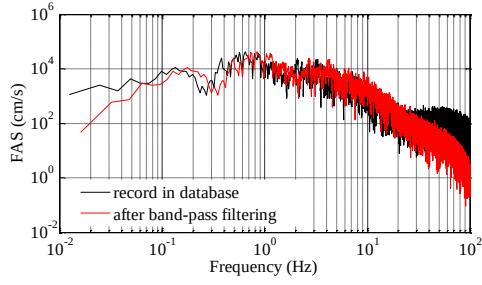


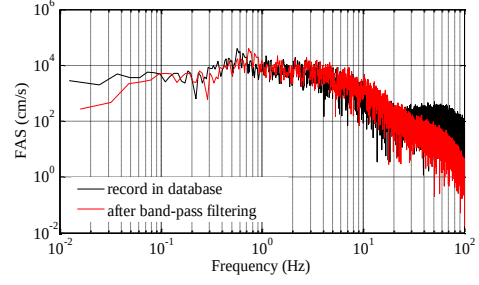
Figure 2-6. Fourier amplitude spectra of original and high cut filtered (a) EW, and (b) NS components of record #3 (Table 3) at 15 Hz.

2.3.2. Band-Pass filtering

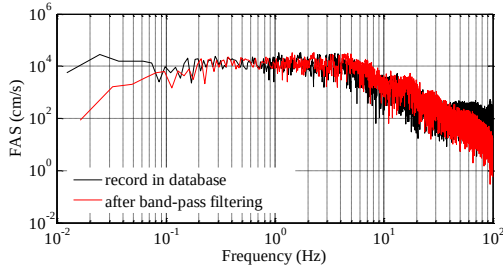
In this study, records #31 to #34 from the database Daphne were refiltered using the high-cut (low-pass) and low-cut (High-pass) filters specified at database. Figures 2-7 and 2-8 show the FAS and acceleration time histories of these records before and after filtering respectively. The filter cut-offs were chosen as indicated in Daphne database. The velocity and displacement histories of these figures are shown in Appendix B.



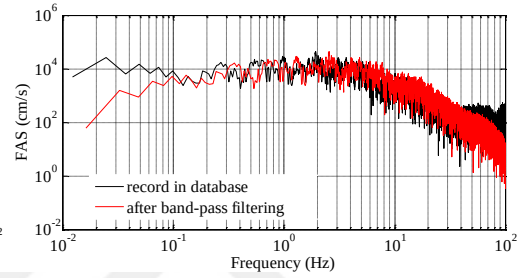
(a)



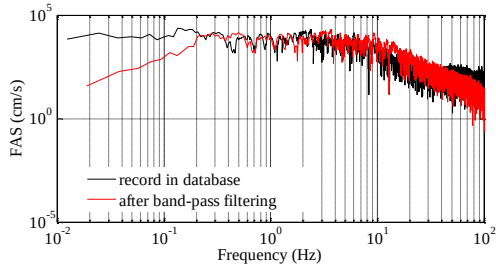
(b)



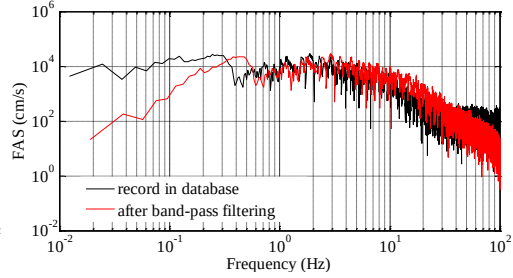
(c)



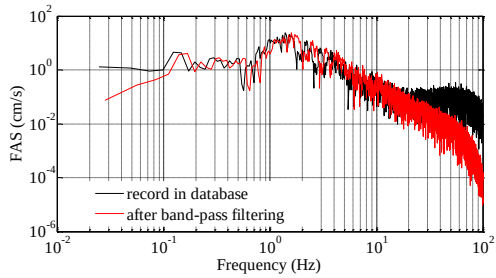
(d)



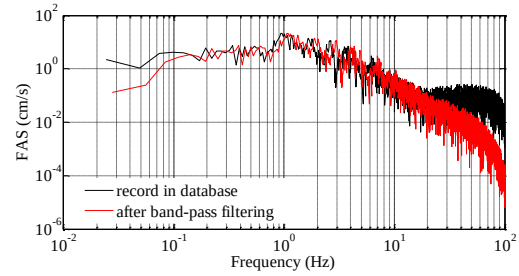
(e)



(f)



(g)



(h)

Figure 2-7. FAS of records #31 in (a) E-W, and (b) N-S ; #32 in (c) E-W, and (d) N-S; #33 in (e) E-W, and (f) N-S; and #34 in (g) E-W, and (h) N-S directions.

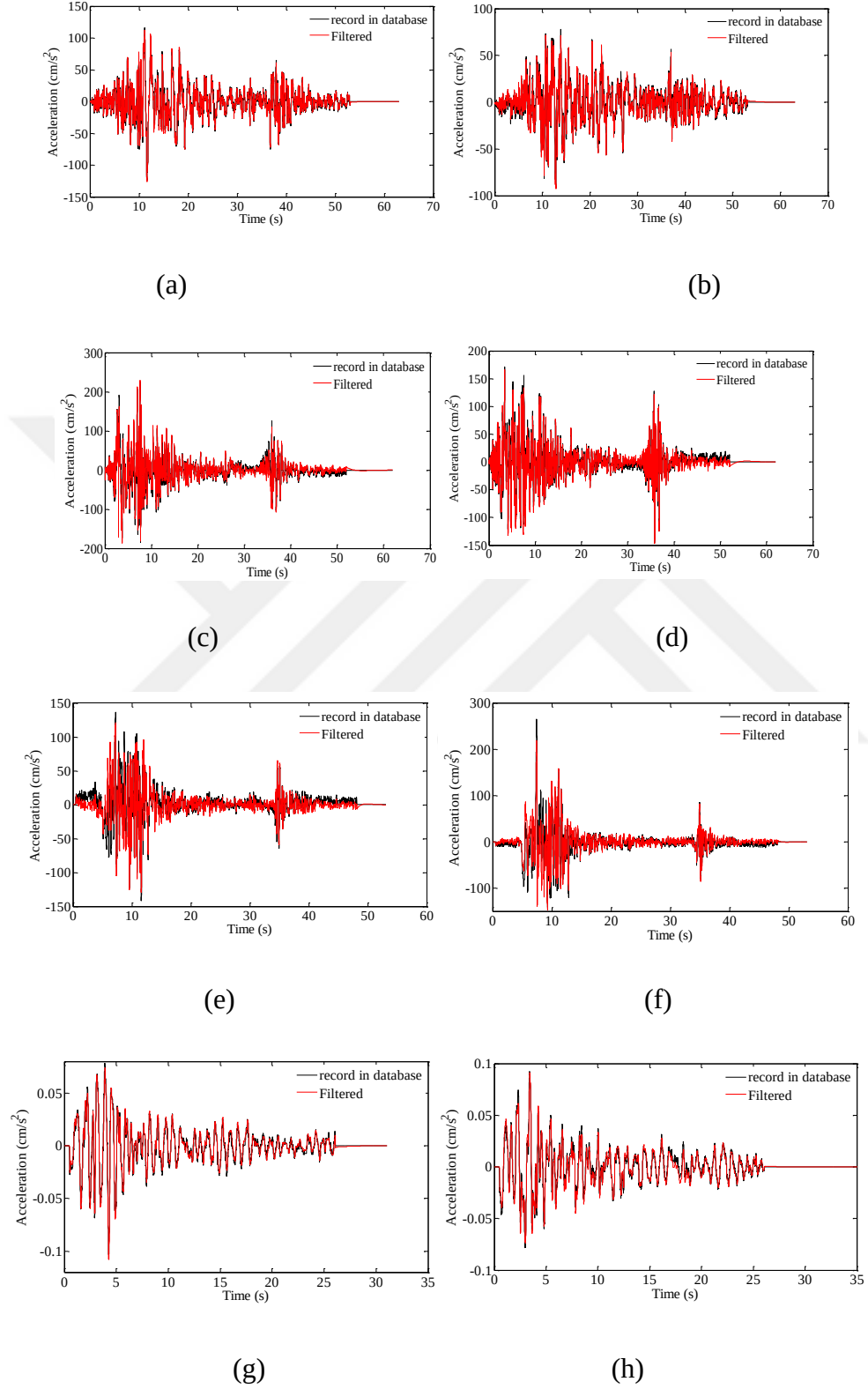


Figure 2-8. Acceleration time history of records #31 in (a) E-W and (b) N-S; #32 (c) E-W and (d) N-S, #33 in (e) E-W and (f) N-S, and #34 in (g) E-W and (h) N-S directions.

Table 3. The list of ground-motion records.

Rec. No.	Event No.	Station	Vs30	SC	LCN	HCN	LCE	HCE	Rjb (km)	PGV/PGA (NS)	PGV/PGA (EW)
1	1	Gormon - Oso Pump Plant	308	D	0.1	23	0.1	30	43.95	0.07	0.10
2	1	Pearblossom Pump	529	C	0.2	35	0.2	35	35.54	0.04	0.05
3	2	BRAN	376	C	0.08	90	0.1	73	3.85	0.09	0.12
4	2	Capitola	288	D	0.2	48	0.2	40	8.65	0.07	0.07
5	2	Gilroy Array #1*	1428	B	0.2	50	0.2	50	8.84	0.07	0.08
6	2	Gilroy Array #2*	270	D	0.2	40	0.2	31	10.38	0.12	0.09
7	2	Gilroy Array #3*	349	D	0.1	33	0.1	40	12.23	0.12	0.07
8	2	Gilroy - Historic Bldg.*	338	D	0.2	38	0.2	40	10.27	0.15	0.10
9	2	SF – Presidio	594	C	0.1	32	0.1	32	77.34	0.17	0.13
10	2	UCSC	714	C	0.1	60	0.1	60	12.15	0.03	0.03
11	2	Woodside	454	C	0.1	25	0.1	25	33.87	0.21	0.17
12	3	Cape Mendocino	513	C	0.07	23	0.07	23	0	0.04	0.09
13	3	Eureka - Myrtle & West	338	D	0.16	23	0.16	23	40.23	0.16	0.13
14	3	Fortuna - Fortuna Blvd	457	C	0.07	23	0.07	23	15.97	0.19	0.26
15	3	Petrolia*	712	C	0.07	23	0.07	23	0	0.14	0.08
16	4	Amboy	271	D	0.1	23	0.1	23	69.21	0.14	0.16
17	4	Fort Irwin	345	D	0.07	23	0.07	23	62.98	0.14	0.09
18	4	Hemet Fire Station	338	D	0.16	23	0.16	23	68.66	0.06	0.07
19	5	Inglewood - Union Oil	316	D	0.16	23	0.16	23	37.18	0.10	0.08
20	5	LA - Century City CC North	278	D	0.14	23	0.14	23	15.53	0.08	0.12
21	5	LA - E Vernon Ave	308	D	0.2	30	0.1	30	33.33	0.08	0.07
22	5	Leona Valley #5 – Ritter	446	C	0.2	23	0.2	23	37.47	0.12	0.10
23	5	5081 Topanga - Fire Sta	376	C	0.083	50	0.083	50	10.31	0.05	0.05
24	5	Castaic - Old Ridge Route	450	C	0.12	23	0.12	23	20.11	0.09	0.10
25	6	KJMA*	312	D	0.05	-	0.05	-	0.94	0.13	0.10
26	6	Kakogawa	312	D	0.1	-	0.1	-	22.5	0.08	0.08
27	6	Nishi-Akashi	609	C	0.1	23	0.1	23	7.08	0.07	0.07
28	6	Takarazuka*	312	D	-	40	0.13	33	0	0.13	0.10
29	6	Takatori*	256	D	-	-	-	-	1.46	0.20	0.21
30	6	Shin-Osaka	256	D	0.1	23	0.08	23	19.14	0.13	0.16
31	7	1612	197	D	0.05	20	0.05	25	33.16	0.22	0.22
32	7	4101*	826	B	0.1	35	0.1	30	0.63	0.17	0.14
33	7	4106*	701	C	0.2	30	0.2	30	4.87	0.11	0.10
34	7	5903	325	D	0.1	15	0.1	15	116.86	0.14	0.18
35	8	CHY024*	427	C	0.02	50	0.02	50	9.62	0.19	0.28
36	8	CHY028	524	C	0.1	50	0.12	50	3.12	0.11	0.08
37	8	CHY034	378	C	0.03	30	0.03	30	14.82	0.16	0.16
38	8	CHY036	233	D	0.03	50	0.05	50	16.04	0.13	0.20
39	8	CHY041	429	C	0.03	50	0.04	50	19.37	0.07	0.06
40	8	CHY080	553	C	0.05	50	0.1	50	0.11	0.11	0.12

SC: site class [30], LCN: “low- cut filter frequency” N-S (Hz), HCN: “high- cut filter frequency” N-S (Hz), LCE: low-cut filter E-W (Hz), HCE: high-cut filter E-W (Hz)

R_{jb}: Joyner-Boore distance

* pulse-like records

The records #31 to #34 are provided by Daphne database.

Table 4.The physical properties of seismic events in Table 3.

No	Event Name	Event Date	Mw	Fault type	Depth (km)
1	San Fernando	9.02.1971	6.6	Reverse	13.0
2	Loma Prieta	18.10.1989	7.1	Reverse oblique	17.5
3	Cape Mendocino	25.04.1992	7.1	Reverse	9.6
4	Landers	28.06.1992	7.4	Strike slip	7.0
5	Northridge	17.01.1994	6.7	Reverse	17.5
6	Kobe	16.01.1995	6.9	Strike slip	17.9
7	Kocaeli	17.08.1999	7.7	Strike slip	15.9
8	Chi-chi	20.09.1999	7.6	Reverse oblique	6.8

CHAPTER 3

RELATIONSHIP BETWEEN PGA, PGV AND MAXIMUM SHEAR STRAIN

3.1. INTRODUCTION

In this study, $\gamma_{\max}$ in a field of vertically incident shear waves propagating in an elastic half space was considered at first to propose a set of functional conclusions. The strain-spectrum concept (Chen and Hou, 1992) was used for a theoretical identification of critical depth separating PGA-sensitive range of depth from the PGV-sensitive range. Then, the random vibration theory was used for derivation of an approximate relationship between $\gamma_{\max}$, PGA, PGV and d/V_s . The parameters were empirically determined by using a sample of ground-motion records. The applicability of this relationship in layered geological formations was examined by considering two generic velocity profiles built in site-response analyses.

3.2. THEORY

3.2.1. The Basic Relationships among Amplitudes

The equation of particle motion due to incident shear waves that propagate in a linearly elastic medium is

$$\frac{\partial^2 u}{\partial t^2} = V_s^2 \frac{\partial^2 u}{\partial y^2} \quad (3-1)$$

where, $u(t,y)$ is the horizontal displacement of a particle located at coordinate y and at time t due to propagating disturbance in the direction y (Figure 1-3). Supposing that $u(t,y)$ and $du(t,y)/dt$ are both continuous in the interval $-\infty \leq t \leq \infty$, and that $u(t,y)$ converges to zero as $|t|$ goes to ∞ , the Fourier Transform of Equation (3-1) yields to an

homogeneous ordinary differential equation in frequency domain (O'Neil, 2012), such as

$$V_s^2 \frac{\partial^2 u^F}{\partial y^2} - \omega^2 u^F = 0 \quad (3-2)$$

ω denotes the wave frequency in Equation (3-2). The superscript F denotes the Fourier transform of a function in time domain. A general solution to Equation (3-2) is

$$u^F(\omega, y) = A_y \exp(-i \frac{\omega}{V_s} y) + B_y \exp(i \frac{\omega}{V_s} y) \quad (3-3)$$

where, A_y and B_y are the frequency-dependent amplitudes of waves that propagate in positive and negative y directions respectively, i is the unit imaginary number of complex numbers, and $\exp()$ is the complex natural exponential function. The shear strain (γ) is determined by the derivative $du(\omega, y)/dy$. The coordinates $y = 0$ for ground surface and $y = d$ for a dummy depth measured from the ground surface are assigned.

The free-boundary condition ($\gamma^F(\omega, 0) = 0$) imposed in Equation (3-3) yields to the relationship

$$\gamma^F(\omega, d) = -u^F(\omega, 0) \cdot \left(\frac{\omega}{V_s} \right) \cdot \sin\left(\frac{\omega}{V_s} d \right) \quad (3-4)$$

or,

$$V_s \cdot \gamma^F(\omega, d/V_s) = i \cdot v^F(\omega) \cdot \sin\left(\frac{\omega}{V_s} d \right) \quad (3-5a)$$

$$V_s \cdot \gamma^F(\omega, d/V_s) = \frac{a^F(\omega)}{\omega} \cdot \sin\left(\frac{\omega}{V_s} d \right) \quad (3-5b)$$

v^F and a^F in Equations (3-5) denote the Fourier transforms of velocity (v) and acceleration (a) history on free-field ($d=0$) respectively. The inverse Fourier Transform of Equation (3-5.b) yields the following solution in time domain.

$$V_s \cdot \gamma(t, d/V_s) = \frac{1}{2} [v(t + d/V_s) - v(t - d/V_s)] \quad (3-6)$$

Equation (3-6) was formerly derived by Chen and Hou (1992), and shows that the history of shear strain at any point in uniformly elastic halfspace is related to velocity history in free field, and to d/V_s , the travel time of vertically incident shear waves from free boundary to this point. Two further inferences about the magnitude of $\gamma_{\max}$ can be derived after Equation (3-6). The first is straight forward after expanding the terms $v(t \pm d/V_s)$ by Fourier series, yielding

$$v(t + d/V_s) - v(t - d/V_s) = a(t) \cdot (d/V_s) + O[(d/V_s)^3] \quad (3-7)$$

since $a(t) = dv(t)/dt$. The term $O[(d/V_s)^3]$ represents the remainder terms in the order of $(d/V_s)^3$. Substitution of Equation (3-7) in Equation (3-6) yields

$$\gamma(t, d/V_s) = a(t) \cdot (d/V_s^2) + \frac{1}{V_s} \cdot O[(d/V_s)^3] \quad (3-8)$$

or,

$$\gamma(t, d/V_s) - a(t) \cdot (d/V_s^2) = \frac{1}{V_s} \cdot O[(d/V_s)^3] \quad (3-9)$$

As d approaches to zero, the term $O[(d/V_s)^3]$ will vanish faster than any term proportional to (d/V_s) . Therefore, the limit of Equation (3-9) as d approaches to zero is

$$\lim_{d \rightarrow 0} \left(\gamma - \frac{a(t) \cdot d}{V_s^2} \right) = 0 \quad (3-10)$$

Hence, if d is close to zero,

$$\gamma_{\max} \cong \frac{\text{PGA} \cdot d}{V_s^2} \quad (3-11)$$

Equation (3-11) is consistent with the method of Seed and Idriss (1971). As d becomes large, the interval of width $2d/V_s$ in Equation (3-6) becomes comparable with the duration between the instant of peak velocity and the instant of zero-crossing closest to

this peak. Consequently, $\gamma_{\max}$ becomes strongly correlated to PGV by increasing d , and can be expressed as

$$\gamma_{\max} = \alpha \frac{\text{PGV}}{2 \cdot V_s} \quad (3-12)$$

where, α is a depth-dependent factor related to the frequency content of ground motion. The factor α approaches to 1.0 for a finite-duration motion as d/V_s approaches to the duration of ground-motion. If $v(t)$ is a sinusoidal waveform, α will be calculated as 2 for a sequence of d/V_s , such that the time $t - d/V_s$ in Equation (3-6) is coincident with the instant of a peak negative (or, positive) velocity, and the time $t + d/V_s$ will be the instant of a peak positive (or, negative) velocity. The actual figures for α in greater ranges of d/V_s are usually between 1.0 and 2.0 for finite-duration seismic excitations. Equation (3-12) was used for normalization of $\gamma_{\max}$ in this study, so that the variability in α among different ground-motion records reflects the dependency of $\gamma_{\max}$ on the frequency content of v . Equations (3-11) and (3-12) suggest two relationships for estimation of $\gamma_{\max}$ in shallow and in relatively deep range of d respectively. The factor α is related to PGA in shallow ranges of depth, such that

$$\alpha \cong \frac{2 \cdot d}{V_s} \cdot \frac{\text{PGA}}{\text{PGV}} \quad (3-13)$$

α becomes almost uniform by increasing depth, so that Equation (3-13) approximately marks the critical depth where $\gamma_{\max}$ becomes almost proportional to PGA after substitution of this ultimate value of α in Equation (3-13). The actual figures for α in the PGV-sensitive range are usually between 1.0 and 2.0 in case of a finite-duration ground motion.

3.2.2. An Estimator for $\gamma_{\max}$ in Uniformly Elastic Medium

The theorems of random vibration can be used for derivation of theoretical relationships between parameters of dynamic response and ground motion. The theoretical analysis of random response can be simplified by two assumptions. The first is that the excitation is statistically stationary in the wide sense. The second assumption is that the probabilistic

distribution of excitation is Gaussian, so that any linear operation on this random process produces another Gaussian process (Yang, 1986). Although the characteristics of transient seismic motions obviously contradict with these assumptions, the simplification may yield to reasonable theoretical functions reflecting the characteristic properties of dynamic systems. The applications regarding the combination of maximum modal displacements in structural dynamics (Wilson, et.al, 1981; Amini and Trifunac, 1985) and the transfer functions for kinematic soil-structure interaction (Luco and Wong, 1986; Veletsos et.al, 1997) are known examples. If the relationship between two random time series, namely excitation (x) and response (y), is linear, the power spectrum of response (S_y) is related to the power spectral density (PSD) of stationary excitation (S_x) by

$$S_y(\omega) = |H(\omega)|^2 S_x(\omega) \quad (3-14)$$

where, $H(\omega)$ is the transfer function between $x^F(\omega)$ and $y^F(\omega)$ which are respectively the Fourier transforms of $x(t)$ and $y(t)$. The variance of a stationary mean-zero process $x(t)$ is formulated by the integral

$$\sigma_x^2 = 2 \cdot \int_0^{\infty} S_x(\omega) d\omega \quad (3-15)$$

where σ_x is the root mean square (RMS) of $x(t)$. RMS of $y(t)$ is obtained by substitution of $S_y(\omega)$ in Equation (3-15). PSD of low-pass noise was considered for a simple representation of random particle velocity in free field. Hence,

$$S_v(\omega) = \begin{cases} A & 0 \leq \omega \leq \omega_c \\ 0 & \text{otherwise} \end{cases} \quad (3-16)$$

The constant A reflects the amplitudes of ground motion in the average sense, and the constant ω_c represents a hypothetical cut-off frequency in the Fourier spectrum of ground motion, so that RMS of v is finite. Substitution of $S_x = S_v$, and $S_y = S_a$ in Equation (3-14) yields to PSD of particle acceleration on free field,

$$S_a(\omega) = \begin{cases} A\omega^2 & 0 \leq \omega \leq \omega_c \\ 0 & \text{otherwise} \end{cases} \quad (3-17)$$

The substitution of the transfer function defined by Equation (3-5.b) in Equation (3-14) yields to the PSD of γ ,

$$S_\gamma(\omega) = \begin{cases} \frac{A}{V_s^2} \cdot \sin^2\left(\omega \frac{d}{V_s}\right) & 0 \leq \omega \leq \omega_c \\ 0 & \text{otherwise} \end{cases} \quad (3-18)$$

The RMS of v , a , and γ can be formulated by substituting Equations (3-16), (3-17) and (3-18) into Equation (3-15) respectively, such that

$$\sigma_v^2 = 2A\omega_c \quad (3-19a)$$

$$\sigma_a^2 = 2A\left(\frac{\omega_c^3}{3}\right) \quad (3-19b)$$

$$\sigma_\gamma^2 = \frac{2A}{V_s^2} \left[\frac{\omega_c}{2} - \frac{\sin\left(2\frac{d}{V_s}\omega_c\right)}{4\left(\frac{d}{V_s}\right)} \right] \quad (3-19c)$$

The peak value to be considered in engineering calculations is expressed as the product of a peak factor and RMS (Vanmarcke, 1977; Der Kiureghian, 1980). The peak values for v , a , and γ are related to their RMS by the respective peak factors p_v , p_a , and p_γ , such that

$$PGV = p_v \cdot \sigma_v \quad (3-20a)$$

$$PGA = p_a \cdot \sigma_a \quad (3-20b)$$

$$\gamma_{\max} = p_\gamma \cdot \sigma_\gamma \quad (3-20c)$$

Only the factor p_γ is dependent on d/V_s . Substitution of Equations (3-19.a-c) into Equations (3-20.a-c) respectively yields to the relationship

$$\gamma_{\max} = \frac{p_{\gamma}}{p_a} \frac{p_a}{p_v} \frac{\text{PGV}}{V_s} \sqrt{\frac{1}{2} \left(1 - \frac{\sin \left(2\sqrt{3} \frac{p_v}{p_a} \frac{\text{PGA}}{\text{PGV}} \frac{d}{V_s} \right)}{2\sqrt{3} \frac{p_v}{p_a} \frac{\text{PGA}}{\text{PGV}} \frac{d}{V_s}} \right)} \quad (3-21)$$

Hence, the peak factor ratios should be estimated for a useful relationship between $\gamma_{\max}$, PGA, and PGV. The peak factor is a variable that is dependent on v_0 , the frequency of zero-crossing for a mean-zero Gaussian process and the duration of a process (Davenport, 1964). Hence, if the duration is supposed to be a constant, the statistical properties of peak factor of two Gaussian processes will be identical provided that v_0 is the same for these processes. v_0 is related to the power spectrum by the relationship

$$v_0 = \frac{1}{\pi} \sqrt{\frac{\int_0^{\infty} \omega^2 S_x(\omega) d\omega}{\int_0^{\infty} S_x(\omega) d\omega}} \quad (3-22)$$

for a stationary Gaussian process $x(t)$. The variance of a peak factor becomes small by increasing duration of the process. Therefore, the variances of peak factors are ignored, and the peak factors are supposed to be a parameter in this study. Robust estimations of the ratios p_{γ} / p_a and p_a / p_v are possible by theoretical investigation of v_0 , but the assumptions inherent in derivation of Equation (3-21) limits the accuracy of rigorous theoretical solutions. Instead, these peak-factor ratios were considered as parameters to be estimated by using a sample of ground motion records, provided that the mathematical constraints were met. Substitutions of $S_x=S_a$, $S_x=S_v$, and $S_x=S_{\gamma}$ in Equation (3-22) respectively yield to v_0 for the random processes a , v , and γ , such as

$$(v_0)_a = \frac{\sqrt{3}}{\pi\sqrt{5}} \omega_c \quad (3-23a)$$

$$(v_0)_v = \frac{\sqrt{3}}{3\pi} \omega_c \quad (3-23.b)$$

$$(v_0)_\gamma = \frac{1}{\pi} \sqrt{\frac{\omega_c^3 - \frac{\left(\frac{d}{V_s}\right)^2 \cdot \omega_c^2 \cdot \sin\left(2\frac{d}{V_s}\omega_c\right)}{4} - \frac{\sin\left(2\frac{d}{V_s}\omega_c\right)}{8} + \frac{\left(\frac{d}{V_s}\omega_c\right) \cdot \cos\left(2\frac{d}{V_s}\omega_c\right)}{4}}{\frac{\left(\frac{d}{V_s}\right)^3}{\frac{\omega_c}{2} - \frac{\sin\left(2\frac{d}{V_s}\omega_c\right)}{4\left(\frac{d}{V_s}\right)}}}} \quad (3-23.c)$$

the limit of Equation (3-23.c) for large d is

$$\lim_{d \rightarrow \infty} (v_0)_\gamma = (v_0)_v$$

Hence,

$$\lim_{d \rightarrow \infty} \frac{p_\gamma}{p_v} = 1 \quad (3-24.a)$$

or,

$$\lim_{d \rightarrow \infty} \frac{p_\gamma}{p_a} = \frac{p_v}{p_a} \quad (3-24.b)$$

Substitution of Equation (3-24) in Equation (3-21) leads to the conclusion

$$\lim_{d \rightarrow \infty} \gamma_{\max} = (PGV/V_s)/\sqrt{2} \quad (3-25)$$

The limit of Equation (3-23.c) as d approaches to zero can be derived by using the limit of S_γ . Consequently,

$$\lim_{d \rightarrow 0} (v_0)_\gamma = (v_0)_a$$

So,

$$\lim_{d \rightarrow 0} \frac{p_\gamma}{p_a} = 1 \quad (3-26)$$

An arbitrary function providing a simple relationship between d/V_s and p_γ/p_a is used for empirical inferences. A function satisfying the limits (3-24.b) and (3-26) is

$$\frac{p_\gamma}{p_a} = \frac{1 + (p_v/p_a)(d/V_s)^m}{1 + (d/V_s)^m} \quad (3-27)$$

Substitution of Equation (3-27) in Equation (3-21) yields to the approximation

$$\gamma_{\max} \cong \frac{\text{PGV}}{2 \cdot V_s} \cdot \frac{(p_a/p_v) + (d/V_s)^m}{1 + (d/V_s)^m} \cdot \sqrt{2 - \frac{\sin\left(2\sqrt{3} \frac{p_v}{p_a} \frac{\text{PGA}}{\text{PGV}} \frac{d}{V_s}\right)}{\sqrt{3} \frac{p_v}{p_a} \frac{\text{PGA}}{\text{PGV}} \frac{d}{V_s}}} \quad (3-28)$$

The approximation of dimensionless parameter α is obtained by substitution of Equation (3-28) in Equation (3-13), such that

$$\alpha \cong \frac{(p_a/p_v) + (d/V_s)^m}{1 + (d/V_s)^m} \sqrt{2 - \frac{\sin\left(2\sqrt{3} \frac{p_v}{p_a} \frac{\text{PGA}}{\text{PGV}} \frac{d}{V_s}\right)}{\sqrt{3} \frac{p_v}{p_a} \frac{\text{PGA}}{\text{PGV}} \frac{d}{V_s}}} \quad \text{for } d/V_s \geq 0 \quad (3-29)$$

Consequently, the parameters necessary for estimation of α and $\gamma_{\max}$ are the ratio p_a/p_v , and the exponent m . The ground-motion parameters PGA and PGV are definite for any given ground-motion record. The most likely figures for the parameters p_a/p_v , and m were empirically determined by using a sample of ground-motion records.

3.3. THE EMPIRICAL DETERMINATION OF THE PARAMETERS

The dimensionless parameter α is determined by computing the product $V_s \cdot \gamma(t, d/V_s)$, by finding maximum absolute value of $V_s \cdot \gamma$ for any given d/V_s (i.e., $V_s \cdot \gamma_{\max}$), and then by normalization according to Equation (3-12). Two alternative methods are available for computation of $V_s \cdot \gamma$. The first is to compute $v(t)$ by numerical integration of $a(t)$. Then,

the product $V_s \cdot \gamma$ is computed by Equation (3-6). The second is the numerical convolution depicted by Equation (3-5.b). The second method was preferred in this study for a critical investigation of the effect of material damping on α . The Fast Fourier Transform (FFT) technique was used for numerical convolution (Newland, 1993). V_s is a dummy parameter in computation of α . An example containing the accelerogram recorded during 1999 Düzce Earthquake (Sucuoglu and Yilmaz, 2001) is presented by Figure 3-1. The record was filtered by a 4-pole/4-pole Butterworth filter (Akkar and Bommer, 2007). The high-pass and low-pass frequencies were respectively set as 0.6 Hz and 50 Hz after an examination of Fourier amplitude spectrum. PGA of filtered record is 0.53g and 0.35g in EW and NS directions respectively. The respective figures for PGV are 83 cm/s and 60 cm/s. The computed α is accurately estimated by Equation (3-13) in shallow range of depth, whereas it becomes more uniform by increasing d/V_s . The geometric mean of α reasonably represents the individual value of this parameter in two orthogonal horizontal directions. The consideration of geometric mean of α in calculation is also practically consistent with the ground-motion prediction equations that are based on the geometric mean of amplitudes in two orthogonal horizontal directions (Baker and Cornell, 2006).

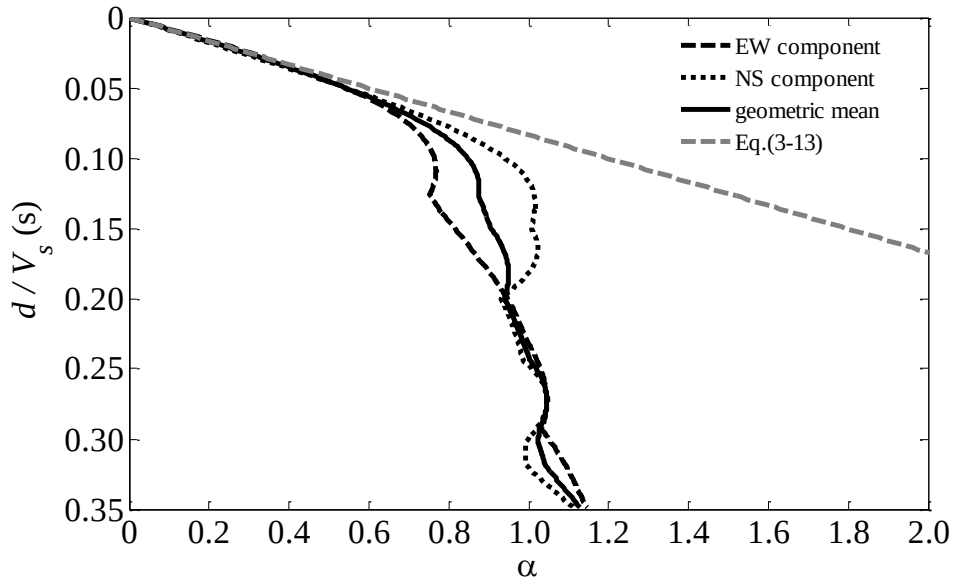
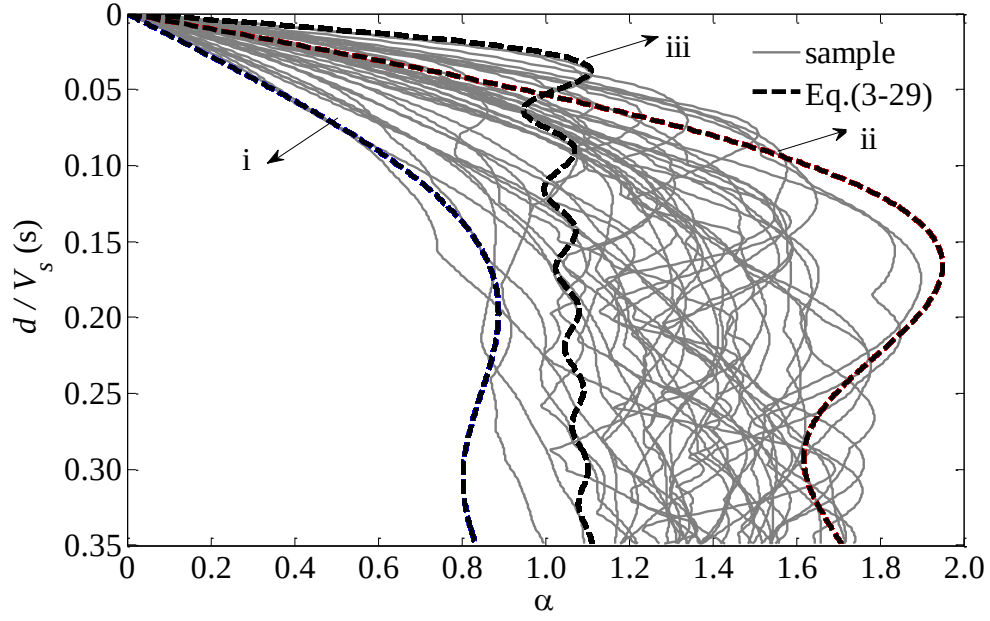


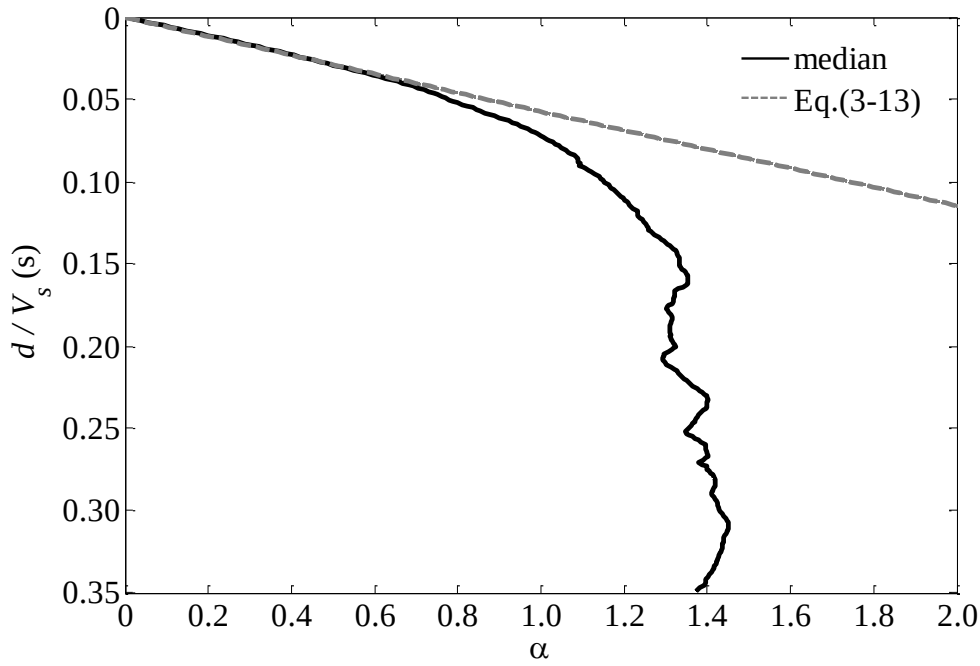
Figure 3-1. The variability of α due to the ground motion during Düzce earthquake.

3.3.1. The Sample for Empirical Estimation of Parameters

Figure 3-2.a shows the sample for α , representing the geometric mean of two horizontal components. The upper limit for d/V_s was chosen as 0.35 s in the following figures for clear presentations of the inferences related to shallow tunnels. Although the variability in α is considerable for any given d/V_s , this factor is reasonably correlated with the ratio PGV/PGA in lower ranges of d/V_s , and it is generally between 1.0 and 1.8 in greater ranges of d/V_s . The substitution of $v_{\max} = \text{PGV}$ in Newmark's formula, corresponding to the case $\alpha=2$ for all d/V_s , yields conservative estimations for $\gamma_{\max}$ particularly in the low range of d/V_s . The functional form of Equation (3-28) is reasonable for modeling the relationship between $\gamma_{\max}$ and d/V_s . This is shown by comparing Equation (3-29) with the sample for α in Figure 3-2.a for three arbitrary sets of the parameters PGV/PGA, p_a/p_v , and m respectively: (i) 0.28, 0.5, and 1.1; (ii) 0.10, 1.3, and 0.9; (iii) 0.04, 0.7, and 1.0. Figure 3-2.b shows the sample median for α , which depicts a depth-dependent empirical relationship for this factor. The median was chosen for empirical conclusions, because this statistic is less sensitive to outliers in a sample than the sample average (Devore, 2012). The median α in the range $d/V_s < 0.05$ s can be accurately estimated by Equation (3-13) by supposing that PGV/PGA is equal to 0.10 s, the median value for the sample. The median α converges to a value around $\sqrt{2}$ by increasing depth, which is supporting the limit in Equation (3-25). The transition between the PGA-sensitiveness range and PGV-sensitiveness of $\gamma_{\max}$ smoothly occurs in the interval $0.05 \text{ s} < d/V_s < 0.10 \text{ s}$. Hence, Equation (3-13) suggests a reasonable limit of d/V_s for the PGA-sensitive range of $\gamma_{\max}$ after the substitution of $\alpha=\sqrt{2}$.



(a)



(b)

Figure 3-2. (a) The sample for the relationship between α and d/V_s , and (b) the sample median.

The parameters m and p_a/p_v were empirically determined in the least-squares sense. The sum of squared error of Equation (3-29) is

$$SSE = \sum_{i=1}^n (\alpha - \alpha_{est})^2 \quad (3-30)$$

where n is the number of ground-motion records. The maximum of SSE in the range $0 \leq d/V_s \leq 0.35$ s was minimized by changing the parameters m and p_a/p_v . This error function reached to its smallest figures when m and p_a/p_v were respectively equal to 1.10 and 0.90.

This error in estimation of α was computed for an array of d/V_s with increments of 0.001 s. The maximum in the calculated range of sum of squared error was considered for minimization of bias. The relative differences between computed α and its estimation (α_{est}) by Equation (3-29) are shown in Figure 3-3. These differences were calculated by considering the geometric mean for each pair of records. Therefore, the geometric means of PGA and those of PGV were substituted in Equation (3-29). The ratio α/α_{est} is generally less than 1.2. The median of the ratio α/α_{est} reaches to the minimum value of 0.889 at $d/V_s = 0.084$ s, almost at the beginning of the PGV-sensitive range of depth. Therefore, Equation (3-28) can be considered as a reasonable estimator for γ_{max} , but the bias is reasonable for practical applications. Nonetheless, the applicability of this estimator in layered geological formations which show hysteretic response to severe seismic excitation should be examined. The relative differences between computed α and its estimation (α_{est}) by Equation (3-29) for the records class B sites are particularly shown in Figure 3-4. The results are compared with the median of all records. It is concluded that the records from class B sites do not produce data ranges that are particularly different from the sample for class C and class D sites.

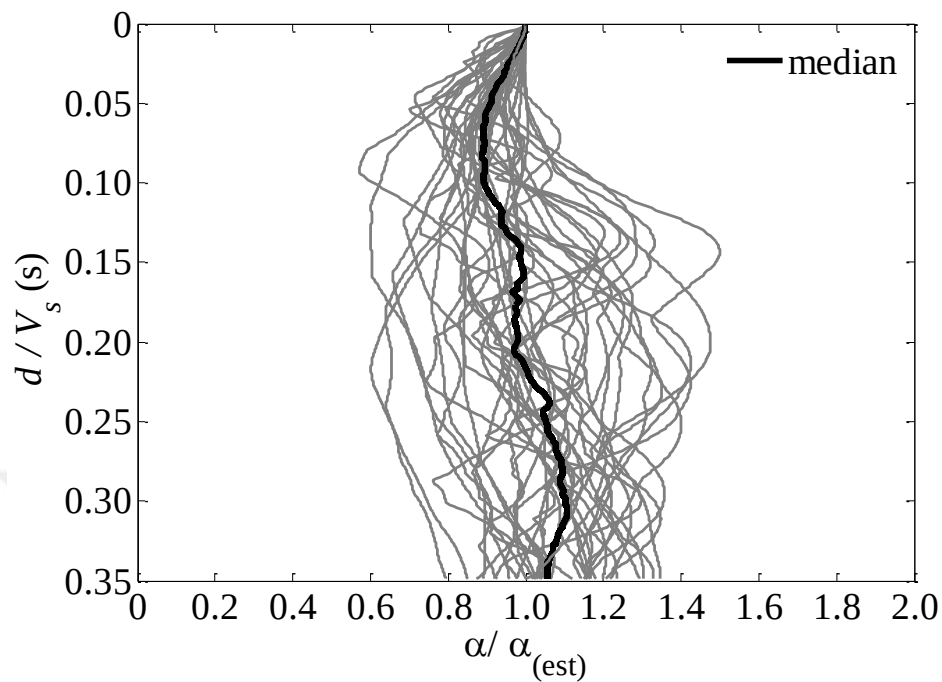


Figure 3-3. The precision of Equation (3-29) in uniformly elastic half space.

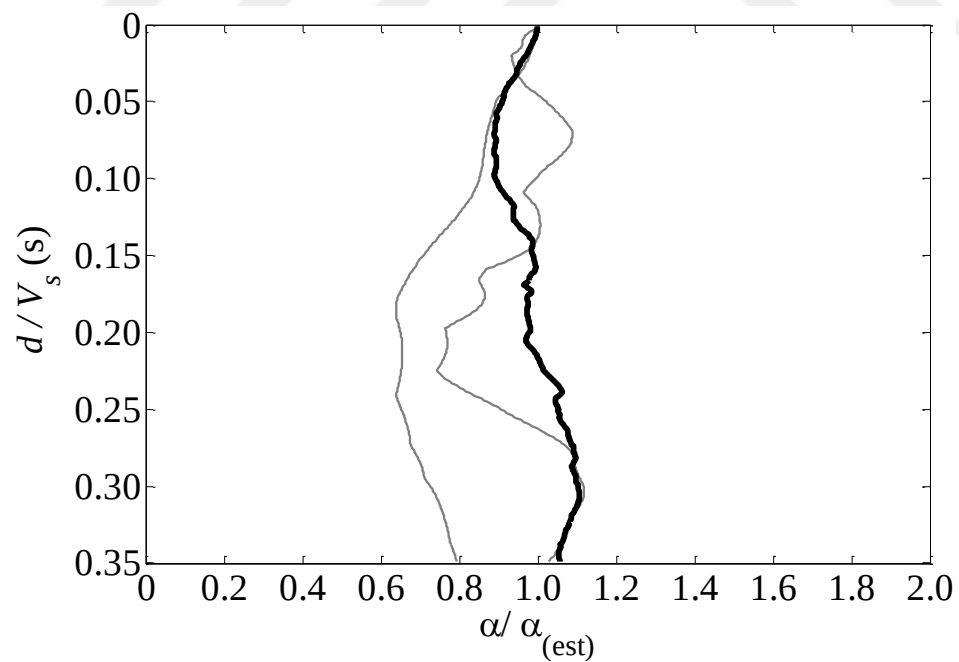


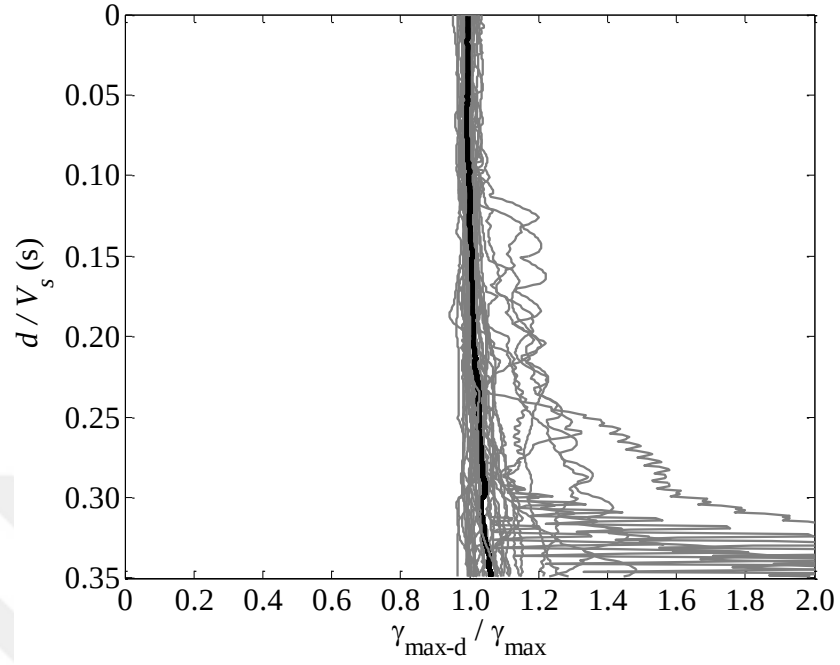
Figure 3-4. The comparison of site B records with Equation (3-29) in uniformly elastic half space.

3.3.2. The Effect of Material Damping

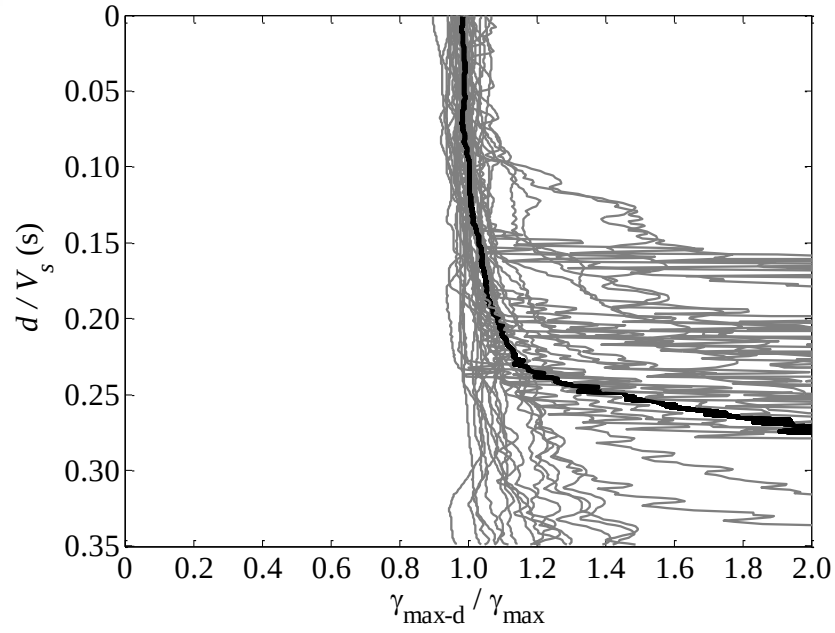
The theoretical analysis was based on the assumption that the waves propagate in a linearly elastic medium, so that the response to strain reversals is not hysteretic. A nonlinear relationship between stress and strain effectively causes a strain-dependent reduction in V_s , which shortens the PGA-sensitive depth interval for $\gamma_{\max}$. A strain-dependent reduction in V_s is possible by using the method of equivalent linearization (Schnabel et al., 1972; Kontoe et al., 2014). The details of the calculation steps are presented in the next section. However, the effect of material damping, which is a consequence of hysteretic stress-strain loops, deserves a preliminary investigation. The frequency-independent material damping is introduced to the linear analysis by defining a complex damping ratio (ξ) in frequency domain. Hence, the term V_s in Equation (3-5.b) is approximately scaled by $\sqrt{1+2i\xi}$ for the viscoelastic medium. This scaling results in a complex sine function. The modulus of complex sine increases considerably by increasing ω in the case $\xi > 0$. If the amplitudes of $a^F(\omega)$ are significant in higher frequencies, the material damping will necessitate unrealistically high amplitudes of ground motion (or, γ) in greater ranges of d/V_s . The frequencies above 15 Hz can be heavily contaminated for accelerograms by high-frequency surface waves and by waves reflected from stratified formations. The high-frequency contamination yields to amplitudes that are not consistent with the presumed properties of viscoelastic medium in analyses, and should be suppressed by a low-pass filter to improve the coherence between generic velocity profiles and the motion presumed on free-field (Silva, 1988).

In order to investigate the effect of damping on the precision of Equation (3-28), $V_s \cdot \gamma$ was computed by considering two different damping ratios, $\xi = 5\%$ and $\xi = 10\%$. The ratio of maximum absolute shear strain in damped response of viscoelastic medium, denoted as $\gamma_{\max-d}$ for the sake of distinction, to that in undamped response of elastic medium, denoted as $\gamma_{\max}$ for consistency with Equation (3-12), was computed for each ground-motion record. Figure 3-5 shows the dispersion of geometric mean of $\gamma_{\max-d}/\gamma_{\max}$ computed for two horizontal components of a ground motion. ξ causes an unrealistic increase in maximum absolute shear strain in greater ranges of d/V_s for a number of accelerograms. The unrealistic increase implies a lack of coherence between the

assumption of uniform ξ in the medium and $a(t)$. The lack of coherence becomes more prominent by increasing ξ . The suppression of spectral amplitudes for frequencies above 15 Hz by the Butterworth filter can solve this problem for most of the records as shown in Figure 3-6. The filter causes a limited change in the sample for PGV/PGA (Figure 2-3). The ratio $\gamma_{\max-d} / \gamma_{\max}$ is observed to be reasonably close to 1.0 in the range $0 < d/V_s < 0.35$ for the records that are coherent with the uniform damping in the medium, and for all records in lower ranges of d/V_s . The inefficiency of low-pass filtering in improvement of coherence can be explained by the assumption of uniform ξ . The more realistic case of nonuniform ξ in medium can be simulated by computing the response of layered geological formations to incident shear waves.

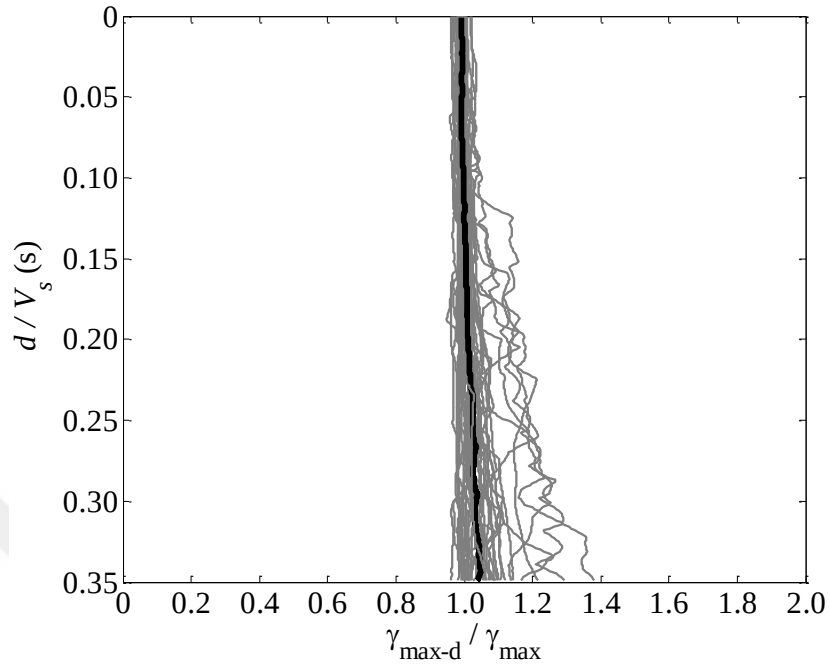


(a)

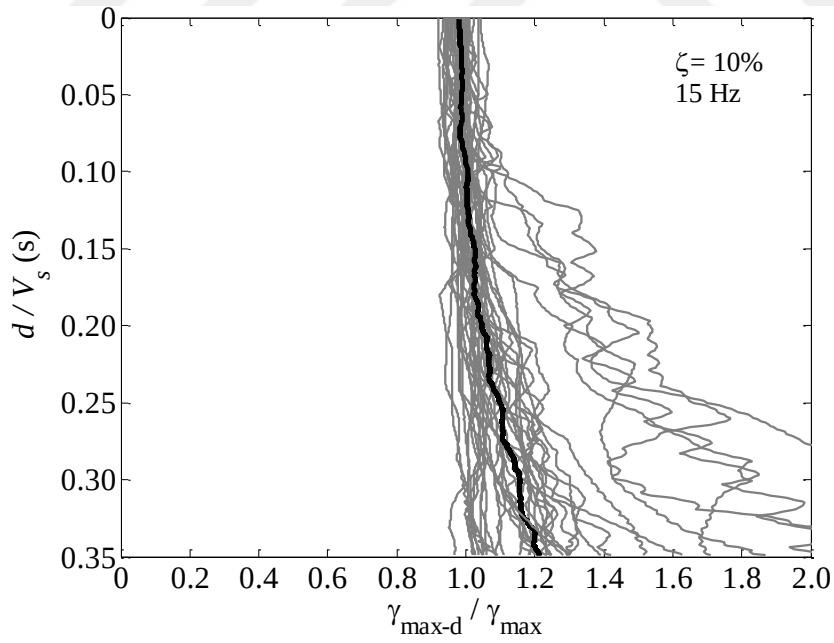


(b)

Figure 3-5. The accuracy of Equation (3-28) in uniformly viscoelastic medium for the cases (a) $\xi = 5\%$, and (b) $\xi = 10\%$ before suppression of high frequencies.



(a)



(b)

Figure 3-6. The effect of low-pass filter on the accuracy of Equation (3-28) in uniformly viscoelastic media with the damping ratios (a) $\xi = 5\%$, and (b) $\xi = 10\%$.

3.3.3. Applicability in Layered Geological Formations

The use of equivalent linear method (EQLM) in analytical solutions as an approximate way of resembling soil's nonlinearity can be successfully used for the preliminary seismic design of circular tunnels. For the detailed design, time-domain finite element analysis should be used (Gomes et al, 2015). Moreover, experimental studies have been conducted to show the capability of the EQLM to predict the soil and tunnel behavior (Amorosi et al, 2014). Therefore, EQLM can be implemented in estimation of $\gamma_{\max}$.

The implementation of EQLM in Equation (3-28) is straightforward by a spread-sheet computer program. The iterative procedure involves the following calculation steps i) Estimate equivalent V_s for each layer due to the previous estimation of $\gamma_{\max}$, initially supposed to zero.

ii) Calculate travel times of vertically propagating shear waves from the free boundary to the middle of each layer by

$$\left(\frac{d}{V_s}\right)_j = \sum_{i=1}^{j-1} \frac{h_i}{(V_s)_i} + \frac{h_j}{2(V_s)_j} \quad (3-31)$$

where, h_i and $(V_s)_i$ are respectively the thickness of and shear-wave velocity in i^{th} layer, $(d/V_s)_j$ is the travel time to the middle of j^{th} layer. This travel time corresponds to the parameter d/V_s in uniformly elastic medium.

iii) Estimate $\gamma_{\max}$ for all layers by substituting $d/V_s = (d/V_s)_j$ in Equation (3-28). The remaining denominator term V_s in Equation (3-28) is equal to $(V_s)_j$.

iv) Return back to the first step in the case that the change in $\gamma_{\max}$ in any layer is greater than the tolerable relative error. The assumption of equivalence between d/V_s in uniform medium and $(d/V_s)_j$ in layered medium was previously suggested for estimation of maximum shear stress at any depth in layered shallow soil deposits (Imai et al, 1981). These steps are also summarized by a flowchart presented in Appendix C.

Equation (3-28) suggests that the $\gamma_{\max}$ is a function of d , so that a unique strain-dependent reduction factor for V_s is not possible. Therefore, regardless of the depth-

dependent variability of V_s , it is necessary to divide V_s profile into layers for appropriate calculation of strain-dependent reduction in V_s for any given set of PGA and PGV.

Imai et al. (1981) empirically showed that the maximum shear stress in layered elastic media that are excited by vertically incident shear waves can be estimated by using the maximum shear stress in uniformly elastic halfspace, if equivalence between travel times of shear waves to reach from free boundary to the depth of interest is assumed. Therefore, the precision of Equation (3-28) in horizontally layered geological formations was examined by using the method of Imai et al. The profile was consisting of 16 uniformly viscoelastic layers, and an elastic half-space representing the bedrock beneath. A generic velocity profile consisting of 16 uniformly viscoelastic layers, and an underlying elastic half space representing the soft rock was considered. The material density of all generic formations was set to 2000 kg/m^3 , so that the results of analyses can be solely attributed to the distribution of V_s . V_s in each layer was calculated by the exponential relationship $V_s = 120 \cdot (1+d)^{0.32}$ to model a reasonable depth-dependent increase in the velocity, such that the free field is pertinent to the site class D, and the hypothetical outcrop of elastic half space is pertinent to the class C. The units of d and V_s are m and m/s respectively. d was taken as the average depth of each uniformly viscoelastic layer. d was taken as the average depth of each uniformly viscoelastic layer. The generic velocity profile is shown in Figure 3-7 by a continuous line. The velocity profile is pertinent to class D, and the hypothetical outcrop of elastic half-space is pertinent to site class C. The effect of nonlinear material behavior on $\gamma_{\max}$ was incorporated by using the equivalent linear method (Schnabel et.al, 1972). The records specified in the following paragraphs were scaled to limit $\gamma_{\max}$, because the accuracy of equivalent linearization in site-response analysis can be considerably low if $\gamma_{\max}$ is greater than 1% (Gerolymos and Gazetas, 2005). The program PROSHAKE, computing the dynamic site response due to equivalent-linear method (Proshake ,1998; Facciorusso et.al, 2012; BSSC, 2003), was used for computation of $\gamma_{\max}$. The strain-dependent degradation in shear modulus and increase in material damping were reasonably modeled by the empirical relationship of Vucetic and Dobry (1991) applicable to lightly overconsolidated fine soils with a Plasticity Index of 15, so that the strain-dependent reduction in material rigidity was significant in the dynamic response analyses.

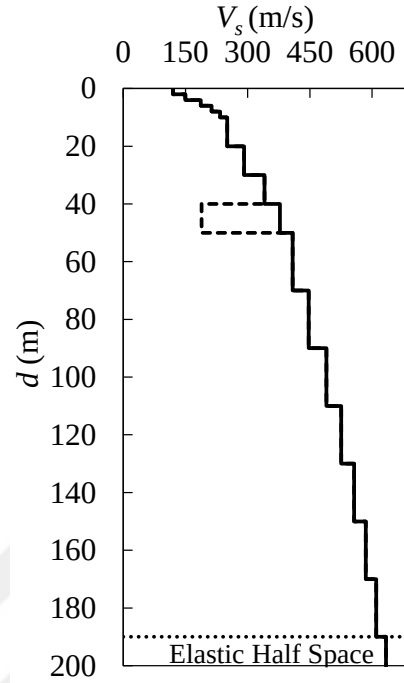
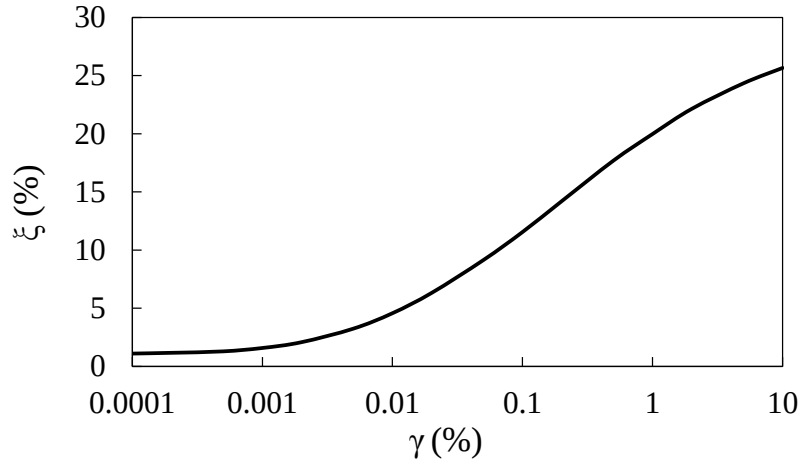
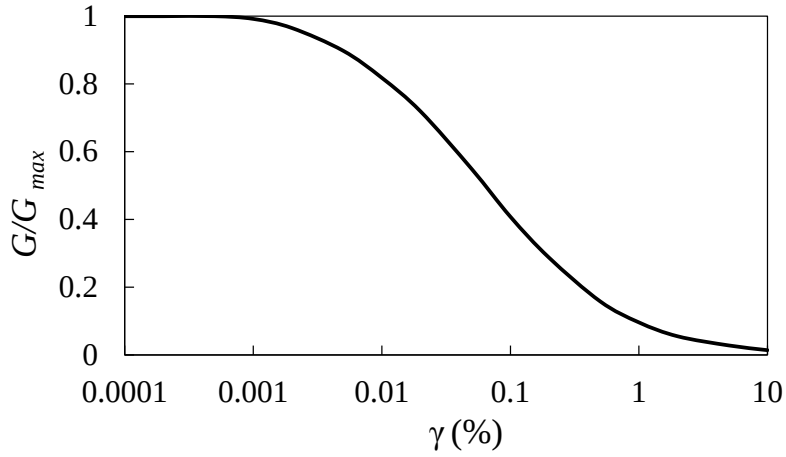


Figure 3-7. Velocity profile used in site response analysis.

Figure 3-8.a shows the relationship between ξ and cyclic amplitude of γ . Figure 3-8.b shows the relationship between the degradation of secant shear modulus (G) in terms of maximum shear modulus (G_{max} , attained at very small amplitudes of γ), and cyclic amplitude of γ . V_s is proportional to the square-root of G , so it is dependent on γ_{max} in equivalent-linear analyses. The representative value for ξ and the reduction in V_s were estimated by supposing that the cyclic amplitude of γ is equal to 65% of γ_{max} . The ground motion records in Table 3 were used for two sets of dynamic response analyses.



(a)

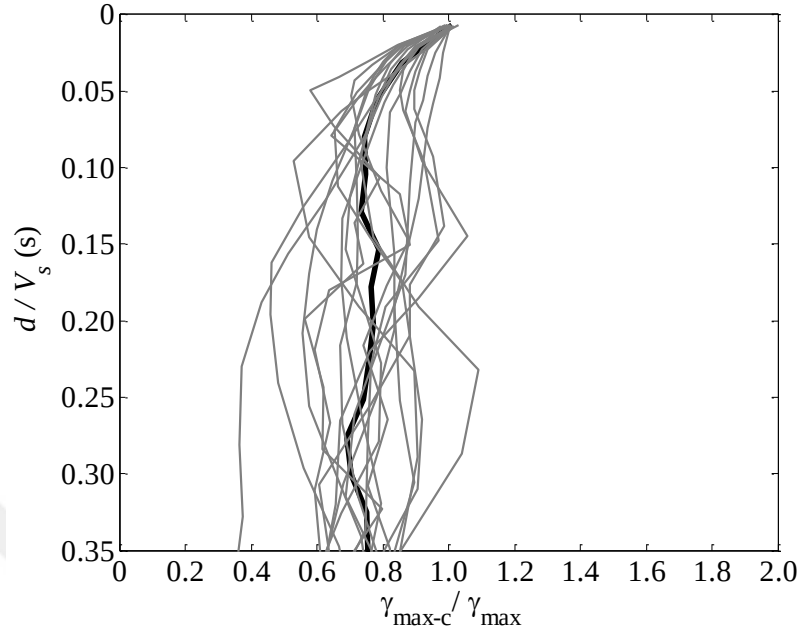


(b)

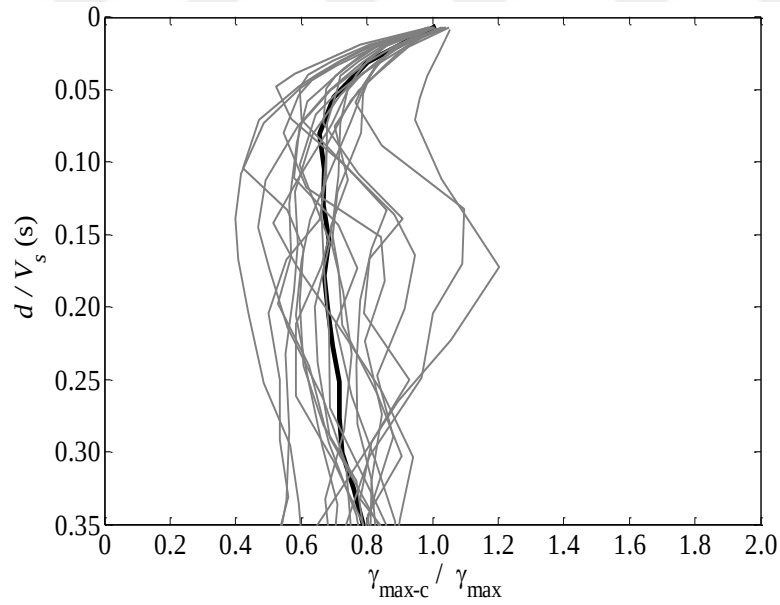
Figure 3-8. The relationship between ξ , modulus degradation (G/G_{max}) and cyclic amplitude of γ .

In the first set, the records from class C sites (Table 3) were supposed to represent the ground-motion on the hypothetical outcrop of the elastic halfspace. In the second set, the records from class D sites were supposed as the records of ground motion on top layer after suppressing frequencies above 15 Hz. Because of the size limitations in the computer program, the Chi-Chi earthquake records numbered from 35 to 40 were cropped to shorten record lengths by 70 seconds that involve most significant amplitudes of ground motion. The cropped records were also filtered at high-pass frequency of 0.1

Hz. The record 29 (Table 3) was scaled by the factor 0.60 to keep $\gamma_{\max}$ below 1%. Each computed value of $\gamma_{\max}$ was normalized by the estimated value due to Equation (3-28) for the pertinent set of ground-motion parameters. Then, the geometric mean of each pair of normalized $\gamma_{\max}$ computed for two horizontal components of ground motion. The tolerable relative error, the limit for relative difference between the results of successive iterations, was accepted as 1%. PGA and PGV of free-field motion that was computed by dynamic response analyses were used for the sake of consistency among parameters in the second set of analyses. Figure 3-9 shows the ratio of maximum absolute shear strain computed by dynamic response analysis ($\gamma_{\max-c}$) to that ($\gamma_{\max}$) estimated by Equation (3-28). The site classes, and the description of ground motion in response analyses (i.e., whether considered as free-field motion or the rock-outcrop motion) does not have any significant effect on the ratio $\gamma_{\max-c}/\gamma_{\max}$. Equation (3-28) conservatively biased estimations for $\gamma_{\max}$ in greater ranges of depth due to increasing V_s . The variability in $\gamma_{\max-c}/\gamma_{\max}$ is related to the sensitivity of α to the frequency content of ground motion as shown in Figure 3-3. This was justified by using $\gamma_{\max}$ computed via Equation (3-5.b) instead of Equation (3-28). The consequent ratios of $\gamma_{\max-c}/\gamma_{\max}$, shown in Figure 3-10, depict much less variability around the sample median for any given d/V_s . Similar bias has been observed in analyses that presume linear material response to shearing.

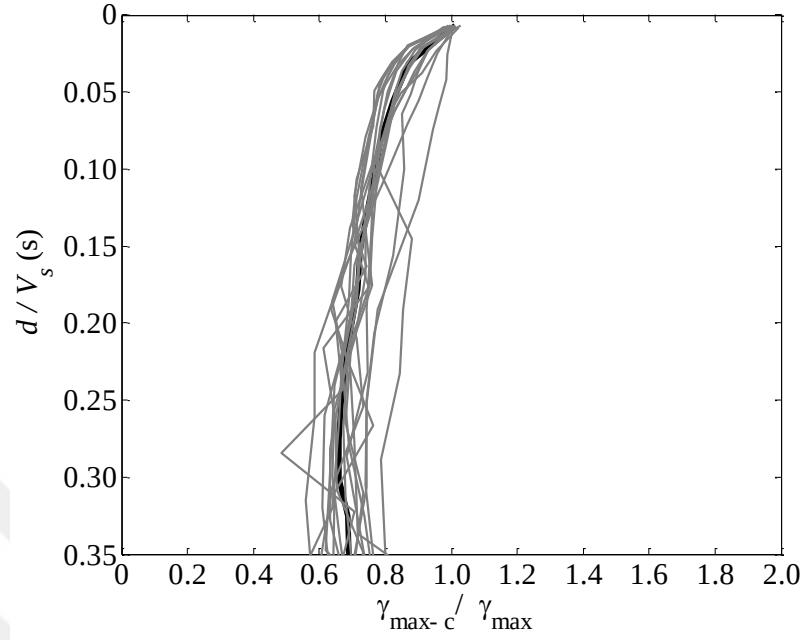


(a)

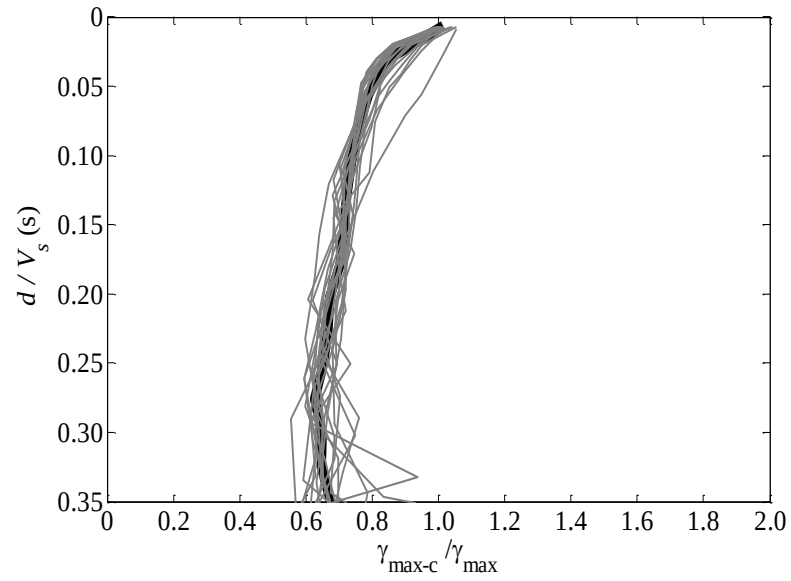


(b)

Figure 3-9. The precision of Equation (3-28) in estimation of $\gamma_{\max}$ in the layered medium due to (a) first set (rock outcrop), and (b) second set (ground surface) of ground-motion records.



(a)



(b)

Figure 3-10. The relationship between the maximum shear strain in the nonuniform V_s profile and that in uniformly elastic half space due to (a) first set (rock outcrop) , and (b) second set (ground surface) of ground-motion records.

The consistency between ξ and $\gamma_{\max}$, and the nonuniform distribution of ξ inhibited unrealistic increase in $\gamma_{\max}$. Figure 3-11 shows the relationship between $\gamma_{\max}$ and d/V_s for two records produced unrealistically high ranges of $\gamma_{\max}$ in uniformly viscoelastic medium. Typically, ξ is smaller in the PGA-sensitive range of $\gamma_{\max}$, and almost uniform in its PGV-sensitive range. Hence, the bias due to the depth-dependent increase in V_s can be reduced by developing relationships between the factor α in the uniformly elastic half space and that in a non-uniformly elastic medium, both related to the same free-field motion.

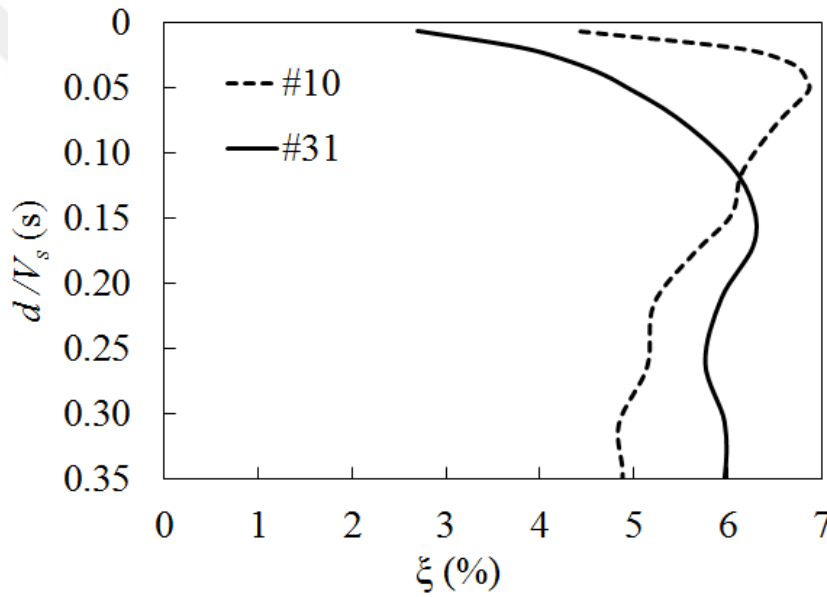


Figure 3-11. The distribution of ξ in the site-response analyses for the records numbered 10 and 31 (Table 3).

In the case study of Düzce tunnel, which partially collapsed during the 1999 Kocaeli Earthquake in Turkey, Kontoe et al. (2011) showed that $\gamma_{\max}$ in a relatively soft stratum embedded in rock formations, such as fault breccia or fault gouge, was much larger than that computed for stiffer rock formations enclosing the softer stratum. The computed range of $\gamma_{\max}$ in the soft stratum was also sensitive to the constitutive material model used in dynamic-response analyses. In order to realize the limitations of the use of Equation (3-28), shear-wave velocity in 10-m-thick layer located between the depths of 40 and 50 m in the velocity model (Figure 3-7) was reduced by 50%, which was

reasonably consistent with the velocity differences reported for Düzce tunnel. The dynamic response analyses were repeated by the same ground-motion sets with the exception that the records numbered as 4, 37, and 40 in Table 3 were scaled by the factors 0.45, 0.45 and 0.40 respectively, so that $\gamma_{\max}$ in the soft layer was limited by 1%. The range of $\gamma_{\max-c}/\gamma_{\max}$ is shown in Figure 3-12. The parameters PGA and PGV in Equation (3-28) were set according to the computed free-field motion. $\gamma_{\max}$ in the soft layer was considerably underestimated. The error is sensitive to the level of nonlinearity in material's response to shearing, such that the maximum $\gamma_{\max-c}/\gamma_{\max}$ in Figure 3-12 reduces from 7.2 to 1.9 when linearly elastic behavior is supposed for all layers. Therefore, Equation (3-28) may considerably underestimate $\gamma_{\max}$ in layers embedded in stiffer stratum. Nonlinear site-response analyses with appropriate constitutive relationships for material behavior are deemed as necessary for the relatively soft layers embedded in stiffer geological formations.

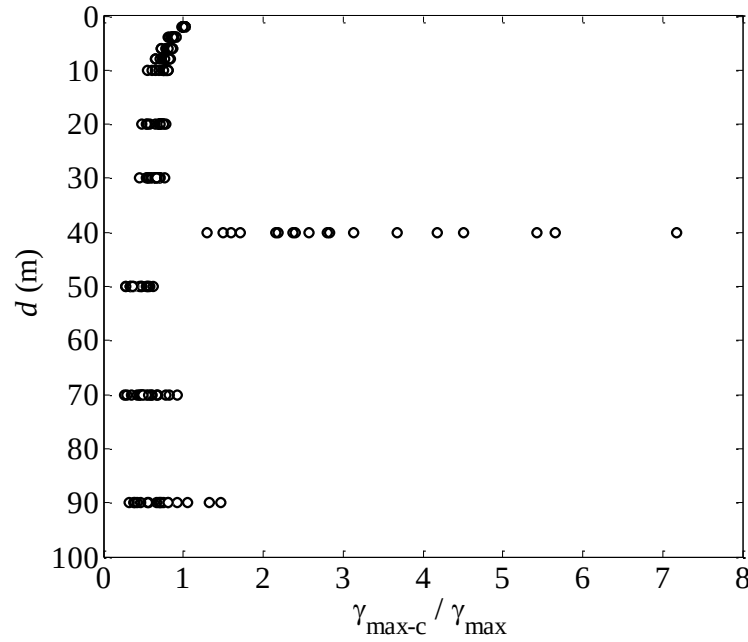


Figure 3-12. The underestimation of $\gamma_{\max}$ in a layer embedded in stiffer layers.



CHAPTER 4

AN EMPIRICAL RELATIONSHIP BETWEEN DESIGN SPECTRUM AND MAXIMUM SHEAR STRAIN

4.1. Introduction

The function named as response spectrum is widely used for estimation of maximum response amplitudes of structures to seismic ground motion after the theorems of modal analysis. The response spectrum defines the maximum response of a linear Single-Degree-of-Freedom (SDOF) oscillator to a specific excitation as a function of its natural period. Considering the uncertainty regarding the properties of ground motion in a possible future event, the amplitudes to be considered in seismic design of structures are generally expressed by a smooth response spectrum, namely a design spectrum. A design spectrum is eventually based on statistical analyses of a sample of the spectra of ground motions recorded in different past earthquakes. The documents on seismic design, such as Eurocode 8 (2012) or ASCE 7-05 (2006) usually employ standard forms of design spectrum that are fundamentally based on the normalized spectrum developed by Newmark (Chopra, 2007). The parameters of a standard design spectrum related to seismic sources are shown on maps, and the parameters related to site conditions are generally expressed as functions of a number of site classes.

Because a response spectrum is related to the dynamic response of an oscillator, there is no simple theoretical relationship between maximum amplitudes of strains in ground and spectral ordinates of design spectrum. In the following, the random vibration theory is used for derivation of a simple relationship between $\gamma_{\max}$ and the spectral ordinates of a design spectrum. The theoretical parameters were empirically estimated by using a sample of ground-motion records. The applicability of this relationship in layered geological formations was examined by considering a generic velocity profile built in site-response analyses.

4.1. Theory

A simple linear oscillator that has a stiffness coefficient of k , viscous damping coefficient of c , and mass m is shown in Figure 4-1. The differential Equation of motion for this single-degree-of-freedom (SDOF) oscillator is

$$m \cdot \frac{d^2 u}{dt^2} + c \cdot \frac{du}{dt} + k \cdot u(t) = -m \cdot a(t) \quad (4-1)$$

such that $a(t) = \frac{d^2 y}{dt^2}$ is the acceleration time history of support motion $y(t)$, and $u(t)$ is displacement of mass relative to the support (Chopra, 2007). Normalization of equation (4-1) by m yields

$$\frac{d^2 u}{dt^2} + 2\xi\omega_n \frac{du}{dt} + \omega_n^2 u = -a(t) \quad (4-2)$$

such that,

$$\omega_n = \sqrt{\frac{k}{m}} \quad (4-3)$$

is the natural frequency of undamped vibration of oscillator and,

$$\xi = \frac{c}{2\sqrt{km}} \quad (4-4)$$

is the damping ratio of the linear oscillator. The natural period of undamped vibration is

$$T_n = \frac{2\pi}{\omega_n} \quad (4-5)$$

T_n and ω_n are shortly referred as natural period and natural frequency of SDOF oscillator. The displacement u can be computed numerically by using a number of methods, such as the numerical integration of Duhamel's integral, the finite differences, the Newmark's method, and numerical convolution in frequency domain (Chopra, 2007).

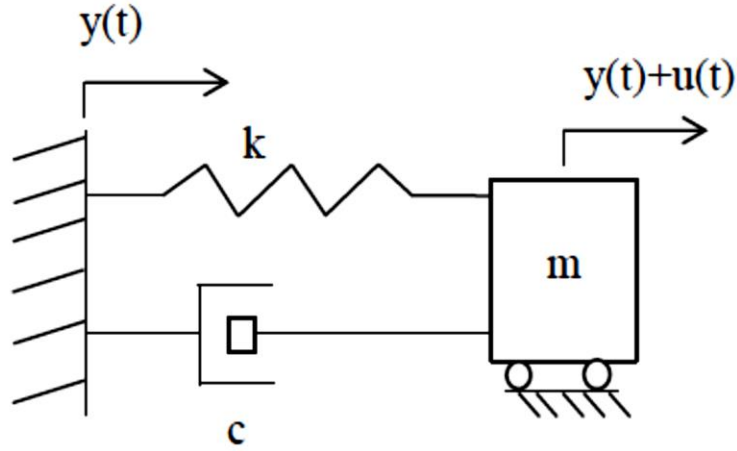


Figure 4-1. Basic elements of an idealized SDOF oscillator.

The transfer function (H_{u-a}) between $u(\omega)$ and $a(\omega)$ in frequency domain is derived by the complex Fourier transform of Equation (4-2), such that

$$u(\omega) = H_{u-a}(\omega) \cdot a(\omega) \quad (4-6.a)$$

where,

$$H_{u-a}(\omega) = \frac{1}{(\omega_n^2 - \omega^2) + 2i\xi\omega_n\omega} \quad (4-6.b)$$

ω is a frequency of excitation and i is the imaginary unit. The maximum absolute relative displacement, which is proportional to the maximum absolute load on spring, is usually of engineering concern. The maximum absolute relative displacement of an oscillator due to a history of support acceleration is conventionally known as the spectral displacement, $SD(\omega_n, \xi)$. Hence,

$$SD(\omega_n, \xi) = |u|_{\max} \quad (4-7)$$

since $SD(\omega_n, \xi)$ is dependent on ω_n and ξ . The pseudo-spectral acceleration is, defined as

$$PSA(\omega_n, \xi) = \omega_n^2 \cdot SD(\omega_n, \xi) \quad (4-8.a)$$

or,

$$PSA(T_n, \xi) = \left(\frac{2\pi}{T_n} \right)^2 \cdot SD(T_n, \xi) \quad (4-8.b)$$

A response spectrum is a function showing the relationship between PSA and T_n for a given ξ . To develop a simple relationship between PSA and $\gamma_{\max}$ the random vibration theory was used. $a(t)$ was supposed to be a random process that is stationary in the wide sense. According to Equation (3-14), the power spectrum of the random response u is,

$$PSD_u(\omega) = |H_{u-a}(\omega)|^2 \cdot PSD_a(\omega) \quad (4-9)$$

According to Equation (3-15), the variance of u is

$$\sigma_u^2 = 2 \cdot \int_0^{\infty} PSD_u(\omega) d\omega \quad (4-10)$$

or, by substitution of Equation (4-6.b) and Equation (4-9) in Equation (4-10),

$$\sigma_u^2 = \int_{-\infty}^{\infty} \left| \frac{1}{(\omega_n^2 - \omega^2) + 2i\xi\omega_n\omega} \right|^2 PSD_a(\omega) d\omega \quad (4-11)$$

where σ_u is the RMS of u . A reasonable approximation for this integral is

$$\sigma_u^2 \cong PSD_a(\omega_n) \frac{\pi}{2\xi\omega_n^3} \quad (4-12)$$

provided that $\xi < 20\%$ (Newland, 1993). This criterion is not a limitation for the scope of this study, because $\xi = 5\%$ is conventionally supposed for most design spectra. According to Equation (3-5.b), the transfer function between γ and a is

$$H_{\gamma-a}(\omega) = \frac{1}{V_s \cdot \omega} \cdot \sin\left(\omega \frac{d}{V_s}\right) \quad (4-13)$$

Hence, substitution of $H(\omega) = H_{\gamma-a}(\omega)$ in Equation (3-14) yields

$$PSD_{\gamma}(\omega) = \frac{\sin^2(\omega \frac{d}{V_s})}{V_s^2 \cdot \omega^2} \cdot PSD_a(\omega) \quad (4-14)$$

The variance of γ is

$$\sigma_{\gamma}^2 = 2 \cdot \int_0^{\infty} PSD_{\gamma}(\omega) d\omega \quad (4-15)$$

or, by substitution of Equation (4-14) in (4-15),

$$\sigma_{\gamma}^2 = 2 \cdot \int_0^{\infty} \frac{\sin^2(\omega \frac{d}{V_s})}{V_s^2 \omega^2} \cdot PSD_a(\omega) d\omega \quad (4-16)$$

where σ_{γ} is namely the RMS of γ . Equations (4-11) and (4-16) show that σ_{γ} and σ_u are both related to PSD_a . A relationship between maximum and RMS of random variables is necessary for a simple relationship between $\gamma_{\max}$ and ordinates of a design spectrum. A simple approach to the solution of this problem is to consider a peak value of a random process as a product of a peak factor and RMS (Vanmarcke, E.H, 1977; Der Kiureghian, 1980; Chai and Faijfar, 2000). Hence, the peak values of u and γ are related to their RMS, σ_u and σ_{γ} , by the peak factors p_u and p_{γ} respectively.

$$SD = p_u \cdot \sigma_u \quad (4-17)$$

$$\gamma_{\max} = p_{\gamma} \cdot \sigma_{\gamma} \quad (4-18)$$

Substitution of Equation (4-12) in (4-17) yields an approximation of spectral displacement, such as

$$SD(\omega_n, \xi) = p_u \cdot \sqrt{\frac{\pi \cdot PSD_a(\omega_n)}{2\omega_n^3 \xi}} \quad (4-19)$$

Substitution of Equation (4-8.a) in (4-19) yields a relationship for PSA,

$$PSA(\omega_n, \xi) \cong p_u \cdot \sqrt{\frac{\pi \cdot \omega_n \cdot PSD_a(\omega_n)}{2\xi}} \quad (4-20)$$

The only frequency term in Equation (4-20) is ω_n , therefore the subscript n can be eliminated for the range $\omega > 0$. Consequently, substitution of Equation (4-20) in Equation (4-14) results in the approximate relationship

$$PSD_\gamma(\omega) \cong \frac{2\xi}{\pi V_s^2 \omega^3 p_u^2} \sin^2\left(\omega \frac{d}{V_s}\right) PSA^2(\omega) \quad (4-21)$$

for $\omega > 0$. Substitution of Equation (4-21) in (4-15) yields RMS of γ .

$$\sigma_\gamma = \frac{2\sqrt{\xi}}{\sqrt{\pi} V_s p_u} \sqrt{\int_0^\infty \frac{1}{\omega^3} \sin^2\left(\omega \frac{d}{V_s}\right) PSA^2(\omega) d\omega} \quad (4-22)$$

Equation (4-22) can be calculated for any given PSA spectrum after substitution $\omega = 2\pi/T_n$.

$$\sigma_\gamma = \frac{\sqrt{\xi}}{\pi^{1.5} V_s p_u} \sqrt{\int_0^\infty T_n \sin^2\left(\frac{2\pi d}{T_n V_s}\right) PSA^2(T_n) dT_n} \quad (4-23)$$

Finally, substitution of Equation (4-23) in Equation (4-18), yields a relationship between PSA and $\gamma_{\max}$.

$$V_s \cdot \gamma_{\max} \cong \frac{p_\gamma}{p_u} \cdot \frac{\sqrt{\xi}}{\pi^{1.5}} \sqrt{\int_0^\infty T_n \sin^2\left(\frac{2\pi d}{T_n V_s}\right) PSA^2(T_n) dT_n} \quad (4-24)$$

The only parameter to be estimated is the ratio p_γ/p_u . These peak ratios are dependent on T_n , ξ and d/V_s . Nevertheless, it is supposed that the ratio p_γ/p_u is only dependent on d/V_s for the sake of simplicity in final relationships. This ratio is substituted by a function of d/V_s to build an approximate relationship between $\gamma_{\max}$ and PSA spectrum, such that

$$V_s \cdot (\gamma_{\max})_{\text{est}} = f\left(\frac{d}{V_s}\right) \cdot \frac{\sqrt{\xi}}{\pi^{1.5}} \sqrt{\int_{T_0}^{T_l} T_n \sin^2\left(\frac{2\pi d}{T_n V_s}\right) PSA^2(T_n) dT_n} \quad (4-25)$$

$(\gamma_{max})_{est}$ is an estimator for γ_{max} . The integral in Equation (4-24) is truncated by the interval $T_0 < T < T_l$ to apply a simple numerical integration scheme such as the trapezoidal rule (Chapra and Canale, 2010).

4.2. An Approximation for p_γ/p_u

The ratio p_γ/p_u that minimize the differences between $(\gamma_{max})_{est}$ and γ_{max} is estimated by using a sample of ground-motion records. The product $V_s \cdot \gamma_{max}$ is determined for each record by computing the history of $V_s \cdot \gamma(t, d/V_s)$, then by finding maximum absolute value of $V_s \cdot \gamma$ for any given d/V_s as explained in Chapter 3. The computed $V_s \cdot \gamma_{max}$ is normalized by $V_s \cdot (\gamma_{max})_{est}$ to calculate the ratio $\gamma_{max}/(\gamma_{max})_{est}$ for a practical range of d/V_s . The geometric mean of two horizontal components of $\gamma_{max}/(\gamma_{max})_{est}$ computed for each ground-motion record constituted the sample for p_γ/p_u . The upper limit for d/V_s was chosen as 0.35 s for a practical conclusion that can be particularly useful for design of shallow tunnels and buried structures.

An empirical relationship between p_γ/p_u and d/V_s is developed by trying a number of functions. These functions are selected according to their ability to reduce the difference between γ_{max} and $(\gamma_{max})_{est}$. It is concluded that Equation (4-25) can reasonably estimate $V_s \cdot \gamma(t, d/V_s)$, if

$$f\left(\frac{d}{V_s}\right) = \frac{1 + \left(7.01 \frac{d}{V_s}\right)}{0.91 + \left(4.67 \frac{d}{V_s}\right)} \quad (4-26)$$

The numerical coefficients in Equation (4-26) were determined in the least squares sense as explained in Chapter 3. All of the ground-motion records in Table 3 were used to build the sample for $\gamma_{max}/(\gamma_{max})_{est}$. The integral limits were chosen as $T_0 = 0.02$ s, and $T_l = 5.0$ s by considering the practical period range of a PSA spectrum. Figure 4-2 shows the scattering of $\gamma_{max}/(\gamma_{max})_{est}$ around 1.0. This ratio is generally less than 1.25 for the sample of records. Nonetheless, the median of this ratio, shown in Figure 4-2, is reasonably close to 1.0 in the range $0 < d/V_s < 0.35$ s. Hence, Equation (4-25) can

provide reasonably unbiased estimations for $\gamma_{\max}$ after substitution of Equation (4-26). On the other hand, deviations of $(\gamma_{\max})_{\text{est}}$ from $\gamma_{\max}$ is generally greater for Equation (4-25) than the estimations by Equation (3-28). An exceptional level of estimation error is reached in case record #12 (Table 3) is used. For this particular ground-motion record, the ratio $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ reaches to figures around 2.0, implying that the estimation can be considerably nonconservative for specific ground-motion records. The frequency content of ground motion, which is related to the shape of response spectrum, is expected to have significant effect on the bias of estimation.

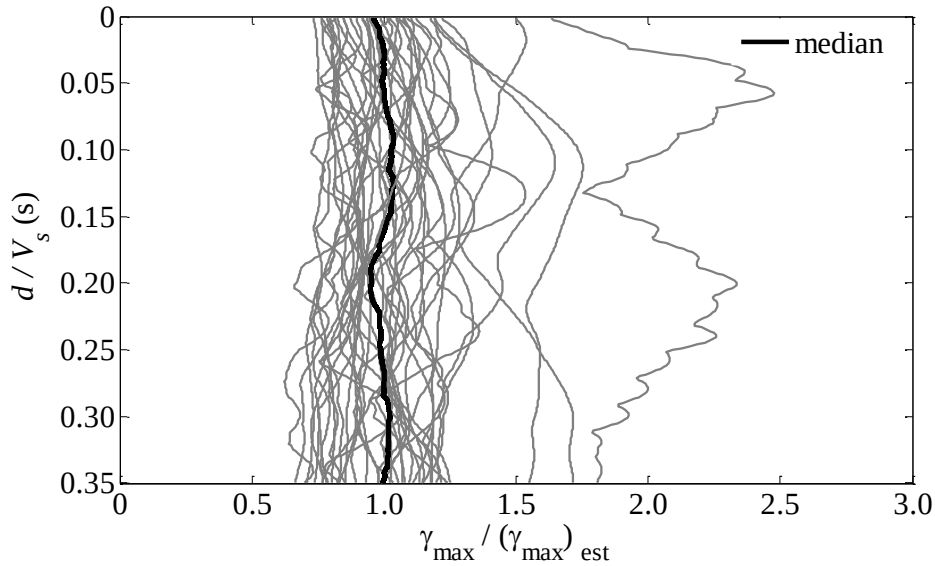


Figure 4-2. The accuracy of Equation (4-25) in uniformly elastic half space.

In order to study the biased behavior of records in Figure 4-2, a comparison of *PSA* spectra is necessary. Therefore, *PSA* spectrum of record #12 (Table 3) is compared with that of record #20 in Figure 4-3. The geometric mean spectra of these records are providing the most and the least biased estimations for $\gamma_{\max}$ respectively. Figure 4-4 shows the isolated ranges of $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ for these records. The record #12 in N-S direction produces the most significant deviation of $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ from 1.0. A preliminary observation on this component of ground motion is that the short-period spectral ordinates are relatively high. The ratio of *PSA* at $T_n=2.0$ s to that at $T_n=0.2$ s (i.e., $PSA(2)/PSA(0.2)$) is respectively 0.162 and 0.215 for E-W and N-S components of record #20. The figures are 0.267 and 0.081 for E-W and N-S components of record #12

respectively. Hence, Equation (4-25) may produce nonconservative estimates for $\gamma_{\max}$, if the short-period spectral amplitudes reach to relatively high figures with respect to those of long-period spectral amplitudes. This is an important limitation in case a response spectrum of a specific ground-motion record is used for estimation for $\gamma_{\max}$. On the other hand, the bias of estimation by Equation (4-25) will be limited if an average of several spectra or a smooth design spectrum is of concern. The accuracy of approximation in this case is examined by using a sample of records matching with the design spectrum in the average sense. This is shown in the following section.

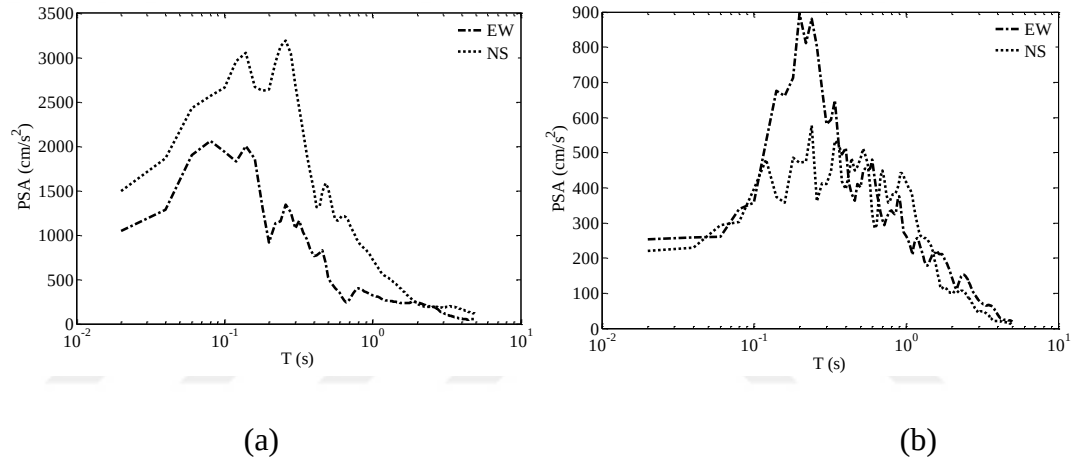


Figure 4-3. The PSA of records showing (a) highest bias (record #12), and (b) lowest bias (record #20) in estimation of $\gamma_{\max}$.

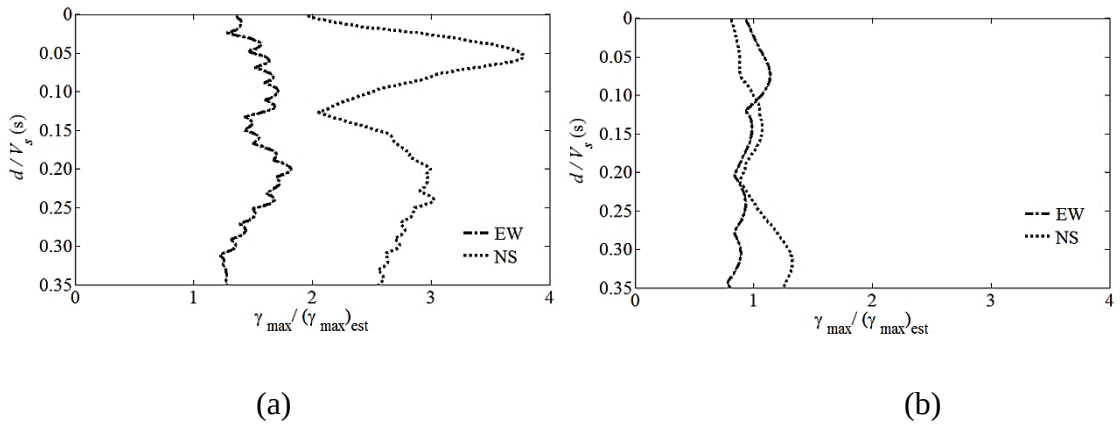


Figure 4-4. The comparisons of bias in estimation of $\gamma_{\max}$ for (a) record #12, and (b) record #20.

The integral limits were altered to estimate the optimum bounds for T_n in integration. The lower limit has a minor effect limited to a very shallow range of depth. 0.02 s, which is equal to the largest sampling interval of records used in the analysis, is a reasonable lower limit for T_0 . T_l , which was previously considered as 5 s in derivation of Equation (4-25), was set to 8 s, and also to 12 s arbitrarily before recalculation of coefficients of Equation (4-25). As it is shown in Figure 4-5, there is not any significant difference among medians of $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$. Therefore, $T_l = 5$ s is a reasonable limit for truncation of integral (4-25). Equation (4-25) yields reasonable bias in estimation of $\gamma_{\max}$. Nonetheless, the applicability of Equation (4-25) in geological formations which show hysteretic response to severe seismic excitation should be examined.

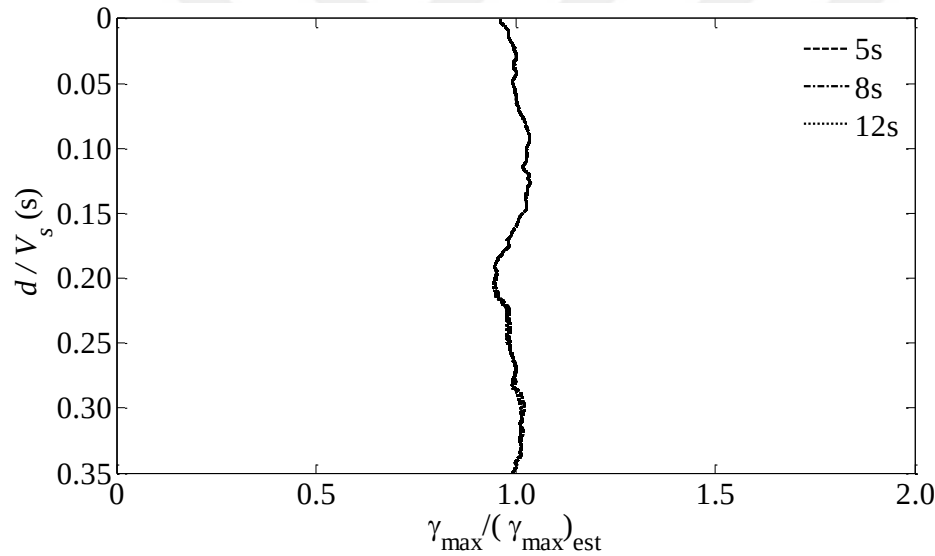


Figure 4-5. A comparison of median $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ for different selections of T_l .

Figure 4-6 compares medians of estimations by Equations (3-28) and (4-25). Equation (4-25) produces more accurate estimations of $\gamma_{\max}$ than Equation (3-28) especially in large ranges of d/V_s . On the other hand, Equation (3-28) is safer in shallow range of depths. These observations may be dependent on the properties of the sample of ground motion records. Nonetheless, the sample size, and the variability in the frequency contents of motions reasonably support generalization of semi-empirical results.

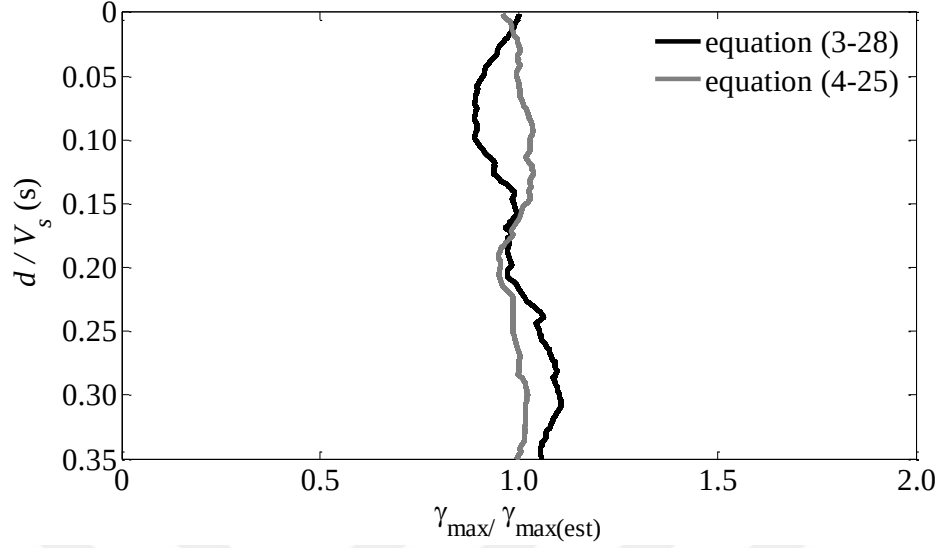


Figure 4-6. A comparison of median $\gamma_{max}/(\gamma_{max})_{est}$ for two different equations.

4.1.1. An assessment of the use of a design spectrum for estimation of γ_{max}

A design spectrum consistent with ASCE 7-05 (2013) was used for an examination of the accuracy of Equations (4-25) and (4-26). The shape of this design spectrum is shown in Figure 4-7. The construction of design spectrum requires three parameters: The design earthquake spectral response acceleration parameter at short period, S_{DS} , that at 1 s period, S_{D1} , and the transition period, T_L . S_{DS} and S_{D1} are related to the spectral response acceleration at short periods, S_s , and that at a period of 1 s (S_1) for maximum considered earthquake, MCE. The spatial distribution of these MCE amplitudes for rock sites (i.e., site class B), and T_L on the United States are shown on maps presented on the document ASCE 7-05. T_L is always greater than 5 s, so this parameter has very limited practical significance in case Equation (4-25) for estimation of γ_{max} .

The spatial coordinate 35° E-W and 120° N-S in California is considered to define a design spectrum reflecting a severe level of seismic hazard. Spectral response accelerations at short periods (S_s) and PSA at a period of 1 s (S_1) for a damping ratio (ξ) of 5% are specified as 1g and 0.5 g respectively. Long-term transition period (T_L) is 12 s for this spatial coordinate according to ASCE 7-05.

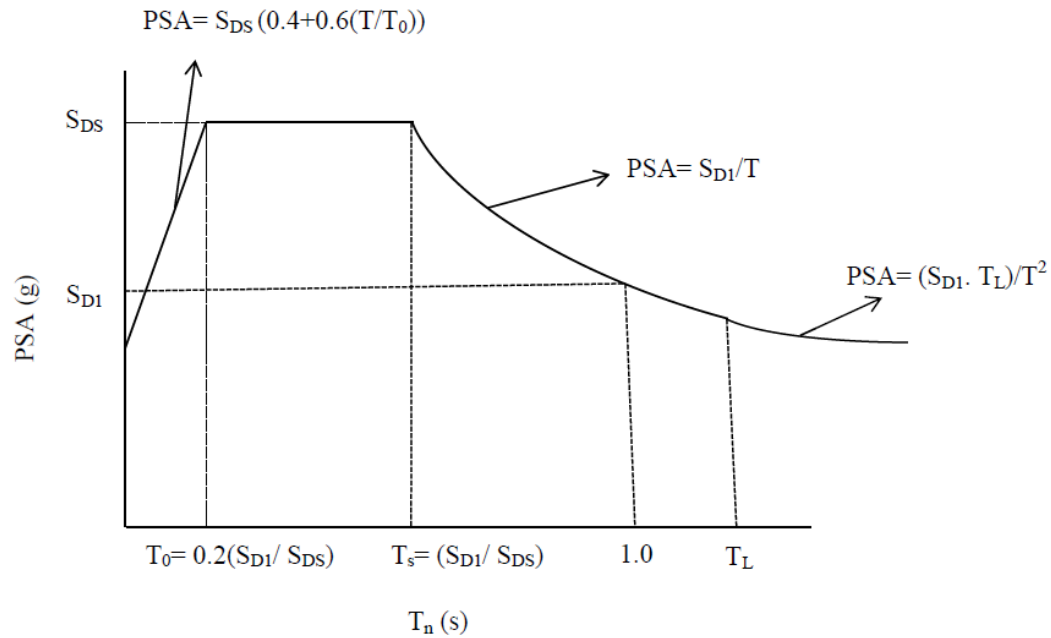


Figure 4-7. The standard shape of design spectrum according to ASCE 7-05.

Table 5. Site coefficient at short-period range, F_a , according to ASCE 7-05.

Site Class	$S_s \leq 0.25g$	$S_s = 0.5g$	$S_s = 0.75g$	$S_s = 1.0g$	$S_s \geq 1.25g$
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

Table 6. Site coefficient at $T_n=1$ s, F_v , according to ASCE 7-05.

Site Class	$S_1 \leq 0.1g$	$S_1 = 0.2g$	$S_1 = 0.3g$	$S_1 = 0.4g$	$S_1 \geq 0.5g$
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

The MCE spectral accelerations are adjusted for site effects by

$$S_{DS} = \frac{2}{3} F_a \times S_s \quad (4-27)$$

and,

$$S_{D1} = \frac{2}{3} F_v \times S_1 \quad (4-28)$$

where F_a and F_v are respectively site factors presented in Tables 5 and 6, S_{DS} and S_{D1} are the design spectral acceleration parameters used for construction of design spectrum shown in Figure 4-7. The site classes C and D are assigned in order to be consistent with the sample of ground motion records and site conditions that are frequently encountered in seismic design of buried structures. Consequently, the design parameters are calculated as $S_{DS}=0.66g$ and $S_{D1}=0.43g$ for a site classified as C, and $S_{DS}=0.73g$ and $S_{D1}=0.50g$ for a site classified as D. The consequent design spectra are shown in Figure 4-7.

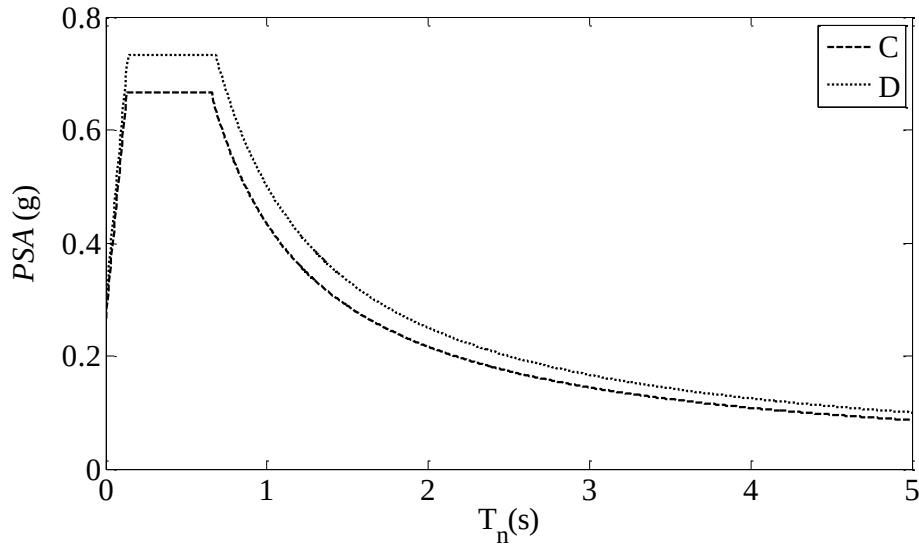


Figure 4-8. The design spectra for a generic class C site and a generic class D site in California.

A set of accelograms that are consistent with the design spectra in Figure 4-8 are used for computation of $\gamma_{max}/(\gamma_{max})_{est}$. Acceleration series are generated by modifying the

sample of records gathered from PEER database shown in Table 3. There are two approaches used for modifying acceleration time series to make them consistent with a design spectrum: scaling and spectral matching (Causse et al., 2014). In scaling method, each accelerogram is multiplied by a factor so that the ordinates median spectrum of the scaled response spectra are equal to or exceed the ordinates of design spectrum over a specified period range. In spectral matching, the frequency content of each accelerogram is modified so that its response spectrum matches the design spectrum at all spectral periods (Al Atik, 2010). Spectral matching is commonly used in engineering practice (Katsanos et al., 2010). There are three main approaches for spectral matching: the frequency-domain method, the frequency-domain method with RVT, and the time-domain method. Although the time-domain spectral-matching method is generally more complex than the frequency-domain method, its convergence is satisfactory, and in most cases this method preserves the nonstationary character of the reference accelerograms (Al Atik, 2010). Design-spectrum-compatible records can be generated by the programs RspMatch (Abrahamson, 1992; Fouad and Rathje, 2012), and SeismoMatch (Seismosoft CO.). RspMatch implements time-domain spectral matching by adding adjustment wavelets to an initial accelerogram to generate a modified time series whose response spectrum is compatible with a specified design spectrum. These wavelets are added at times where there is already significant amplitude in that period range in the time history (Fouad and Rathje, 2012). RspMatch was originally developed by N.A. Abrahamson (1992) as an implementation of the algorithm of Lilhanand and Tseng (1988). RspMatch was further developed by Hancock et al. (2006). This version was named as RspMatch 2005. Grant and Diaferia (2012) mentioned that the spectral-matching method by RSPmatch 2005 does not cause significant bias in structural analysis, and leads to accurate predictions of structural response. Hence RspMatch was used in fragility analyses (Cimellaro et al., 2011), for generation of accelerograms for dynamic analyses of structures (Iervolino et al., 2010, Bommer and Acevedo, 2008), and for analyses of a base-isolated bridge (Ozbulut and Hurlbaas, 2011).

The program Rspmatch 2005 was used in this study to generate a number of accelerograms that match with the design spectra. The scaling was performed for periods between 0.02 and 5 s. The mismatch tolerance was set to 10% which shows maximum

deviation of each record from the target spectrum. The maximum number of iterations was set to 30. All records in Table 3 were used as initial accelerograms to be modified by the program. Both components of 14 records from class D sites (#4, #13, #16, #17, #19, #20, #21, #25, #26, #28, #29, #30, #31, and #38 in Table 3) converged within the specified tolerance. Figure 4-9.a shows the comparison between the median of matched spectra, the geometric mean spectral acceleration of matched spectra, and the design spectra for site classes D. It can be seen that the median is consistent with the design spectrum. In Figure 4-10.a, the geometric mean of $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ is plotted for each of horizontal components of 14 records. The median of the sample for $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ is between 0.954 and 1.03 in the range $d/V_s \leq 0.35$ s. It can be concluded that the records whose spectrum is similar to that of design spectrum yield reasonably precise estimations of $\gamma_{\max}$. Both components of 11 records from class C sites (#2, #9, #10, #11, #14, #23, #27, #33, #35, #39, and #40) converged with the specified tolerance. In figure 4-10.b, the geometric mean of $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ is plotted for each of horizontal components of records. The median of the sample for $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ is between 0.964 and 1.084 in the range $d/V_s \leq 0.35$ s.

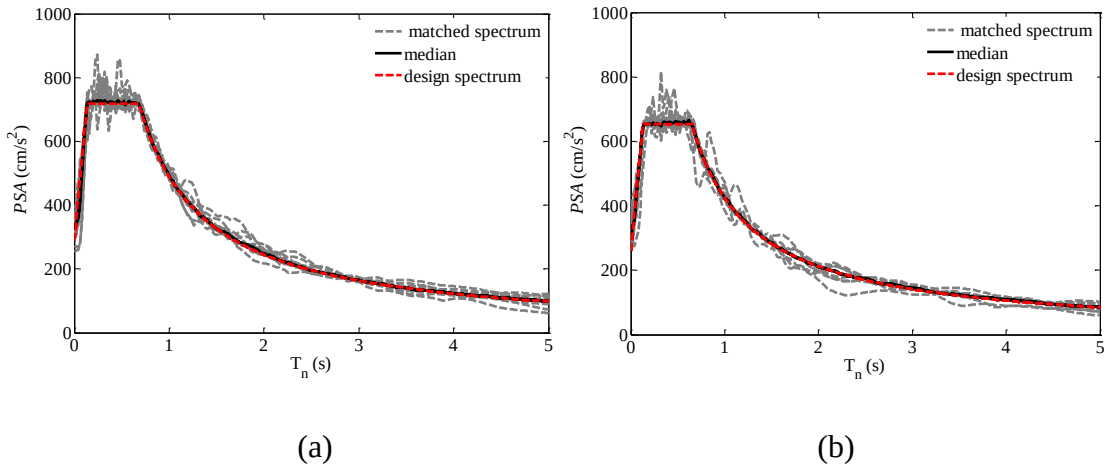


Figure 4-9. Consistency between median *PSA* of generic accelerograms and the design spectrum (a) for a class D site and (b) for a class C site.

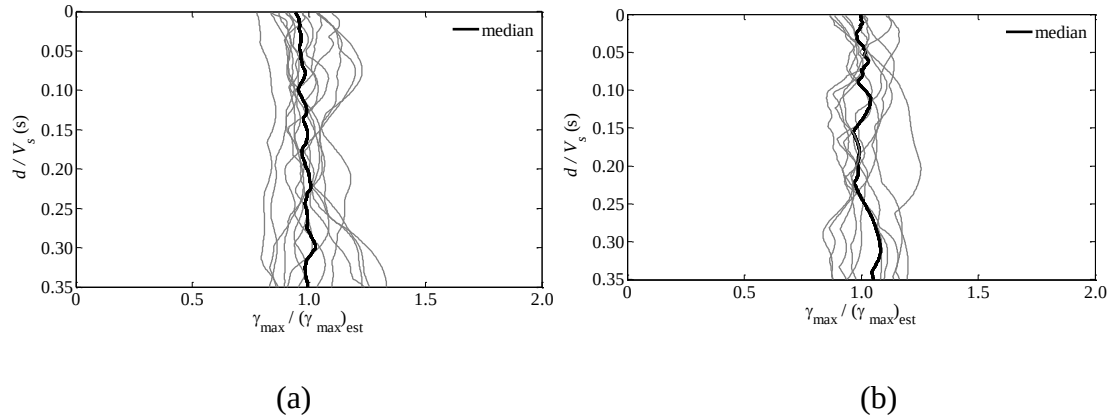


Figure 4-10. The sample for the accuracy of estimation in case a smooth design spectrum for (a) class D site, and (b) class C site is used for estimation of $\gamma_{\max}$.

The range of $\gamma_{\max}$ estimated by the design spectra is shown for both site classes C and D in Figure 4-11. V_s was set to 300 m/s for NEHRP D soil class range and to 550 m/s for NEHRP C soil class range. $V_s \cdot \gamma_{\max}$ for site class C and D relatively differs by 13% at most as shown in Figure 4-11.a, whereas, the relative difference in $\gamma_{\max}$ between two sites is about 1%. Hence, the effect of V_s , related to the deformability of medium on $\gamma_{\max}$ is more pronounced than its effect on seismic hazard.

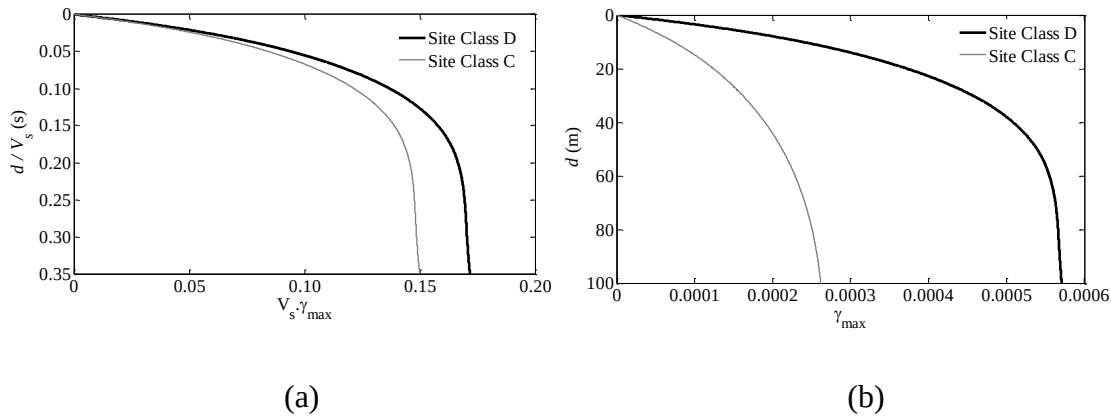


Figure 4-11. The range of $\gamma_{\max}$ consistent with design spectrum as a function of (a) d/V_s , (b) and d .

4.2. Layered Geological Formations

A generic velocity profile consisting of 16 uniformly viscoelastic layers, and an underlying elastic half space representing the soft rock was considered for an observation on the variability of $\gamma_{\max}/(\gamma_{\max})_{\text{est}}$ in layered sites. The generic velocity profile is shown in Figure 3-6. The effect of nonlinear material behavior on $\gamma_{\max}$ was incorporated by using the method of equivalent-linearization (Schnabel et.al,1972; Kontoe et.al, 2014). Some of the records specified in the following paragraphs were scaled to limit $\gamma_{\max}$ as explained in Chapter 3, because the accuracy of results produced by equivalent linearization in site-response analyses can be considerably low if $\gamma_{\max}$ is greater than 1% (Gerolymos and Gazetas, 2005).

The assumption of equivalence between travel time in uniform medium and travel time of vertically incident shear waves from the free boundary to the depth of interest in layered medium was suggested for estimation of maximum shear stress at any depth in layered shallow soil deposits. Therefore, the accuracy of Equation (4-25) in horizontally layered geological formations was examined by comparing the estimated range of $\gamma_{\max}$ with the results of dynamic site-response analyses. In the first set of site-response analyses, the records from the class C sites were supposed to represent the ground-motion on the hypothetical outcrop of the elastic half space. In the second set, the records from the class D sites were supposed as the records of ground motion in free field after suppressing the frequencies above 15 Hz by a 4-pole Butterworth filter. PSA spectra of free-field motion that were computed by SeismoSignal program (Seismosoft Co.) were used in the second set of analyses. Figure 4-12 shows the geometric mean of $\gamma_{\max-c}/\gamma_{\max}$, the ratio of maximum absolute shear strain computed by defining the rock outcrop motion in the first set to that estimated by Equation (4-25). Two horizontal components of each ground-motion record were separately employed in the analyses. Figure 4-13 shows the geometric mean of $\gamma_{\max-d}/\gamma_{\max}$, the ratio of maximum absolute shear strain computed in the second set of analyses to $\gamma_{\max}$. The site classes, and the description of ground motion in site-response analyses (i.e., whether it is considered as free-field motion or the rock-outcrop motion) does not have any pronounced effect on the accuracy of the estimated $\gamma_{\max}$. The median of $\gamma_{\max-c}/\gamma_{\max}$ and that of $\gamma_{\max-d}/\gamma_{\max}$ reach

to similar ratios, 0.635 and 0.640 respectively at $d/V_s=0.35$. The bias in estimation of $\gamma_{\max}$ is because of the depth-dependent increase in V_s . Hence, Equation (4-25) yields to conservative estimates for $\gamma_{\max}$ in case of layered formations with increasing V_s by depth.

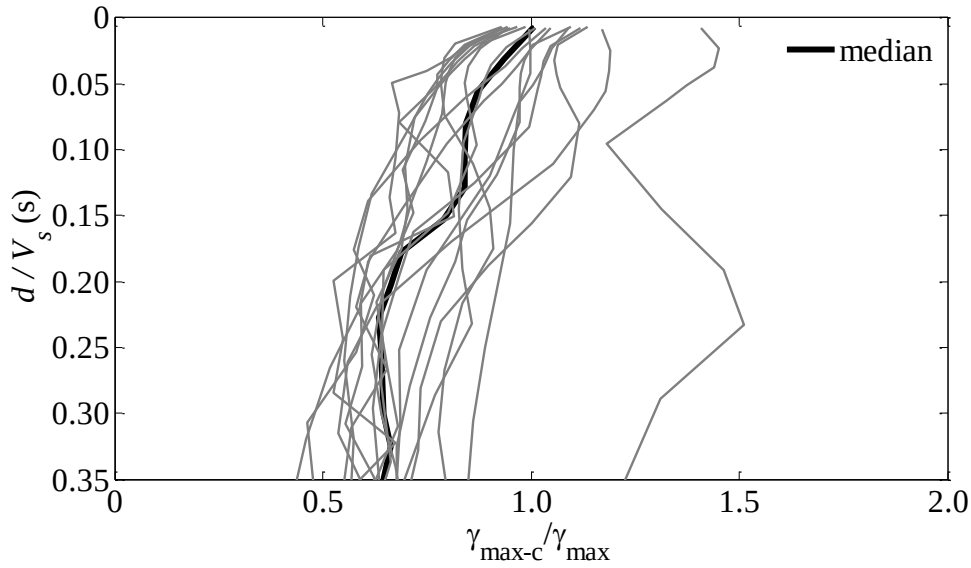


Figure 4-12. The precision of Equation (4-25) in estimation of $\gamma_{\max}$ in the layered medium, in case the seismic excitation at a site is defined by a rock outcrop motion.

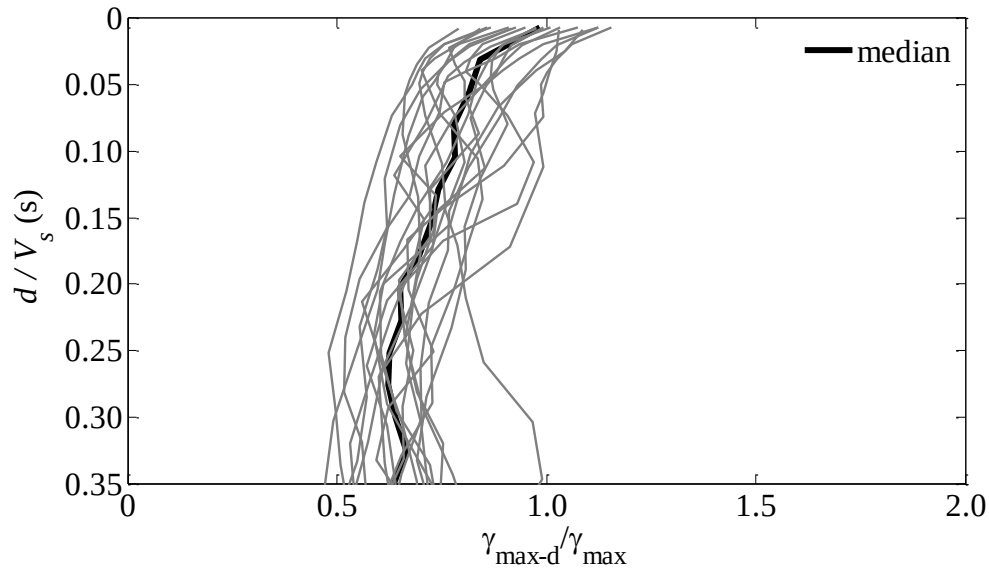


Figure 4-13. The precision of Equation (4-25) in estimation of $\gamma_{\max}$ in the layered medium, in case the seismic excitation at a site is defined by a free field motion.

CHAPTER 5

SITE RESPONSE ANALYSIS

5.1. Introduction

This chapter presents the coherence between the distribution of $\gamma_{\max}$ in elastic half space and that in layered medium by supposing that the travel time of shear waves from the free boundary to a depth of interest in a layered medium is equal to d/V_s . For this purpose, a set of strong ground motion time histories and three V_s profiles are used for analyses. The effect of nonlinear response of material to shearing was investigated by using the method of equivalent linearization.

5.2. The Concept of Equivalency in Travel Time

The shear strain (γ) at any point in an elastic halfspace is a function of time, acceleration history in free field, depth (d) and the shear-wave velocity (V_s) in the medium. Therefore, the emphasis can be put on shear-strain spectra (Chen and Hou, 1992) in selection of ground-motion records for analyses of buried structures. Hence, Equations (3-5a) and (3-5b) are applicable for uniformly elastic half space. According to Imai et al. (1981), the maximum shear stress ($\tau_{\max}$) at any given depth in layered soils can be estimated by supposing that the travel time of vertically propagating shear waves from the free boundary to the depth of interest is equivalent to the travel time for the same distance in a uniform elastic medium. This travel time is calculated by Equation (3-30) in Chapter 3 for estimation of $\tau_{\max}$ in the l^{th} layer below the ground surface. $\gamma_{\max}$ is related to $\tau_{\max}$ by

$$\gamma_{\max} = \frac{\tau_{\max}}{\rho_l \times (V_s)_l^2} \quad (5-1)$$

where ρ_l and $(V_s)_l$ are respectively the density and the shear-wave velocity of the layer of interest. Hence, d/V_s calculated by Equation (3-30) is first substituted in Equations

(3-5b) or (3-6) for estimation of $V_s \cdot \gamma(t)$. Then, by supposing that $V_s = (V_s)_1$, the maximum absolute value of $V_s \cdot \gamma(t)$ yields an estimation for $\gamma_{\max}$ in the l^{th} layer. The procedure can be simplified if the relationship between d/V_s and $V_s \cdot \gamma_{\max}$ is first computed for any given $a(t)$, and then d/V_s calculated by Equation (3-31) is used for interpolation in the calculated range of $V_s \cdot \gamma_{\max}$. This estimator for $\gamma_{\max}$ is shown by $\gamma_{\max-u}$. The implementation of equivalent-linear method in Equation (3-5b) is similar to the procedure described in section 3.3.3. The strain-dependent degradation in shear modulus and increase in material damping are modeled by the empirical relationship of Vucetic and Dobry (1991) applicable to lightly overconsolidated fine soils with a Plasticity Index of 15.

5.3. 1-D Site Response Analysis

One dimensional (1-D) site response analysis is widely used for computation of dynamic response of horizontally layered geological formations to seismic motion. It is supposed that the vertically propagating body waves are causing the seismic disturbance in layered media. The seismic motion is defined on a particular point of the 1-D analysis domain and proposes the propagation of control motion from the control point over the vertical extent of the site profile. The control motion is usually defined as the free-field motion on top of layered media (a_{ff}), or as the motion on hypothetical bedrock outcrop (a_r), as it is shown in Figure 5-1. Acceleration history in soil layer is denoted by $a_l(t)$.

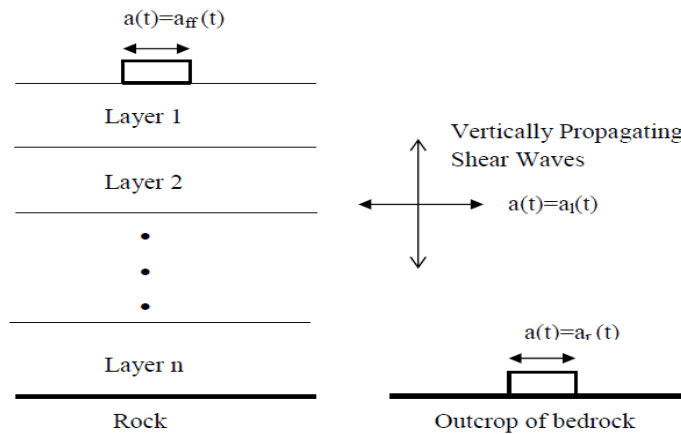


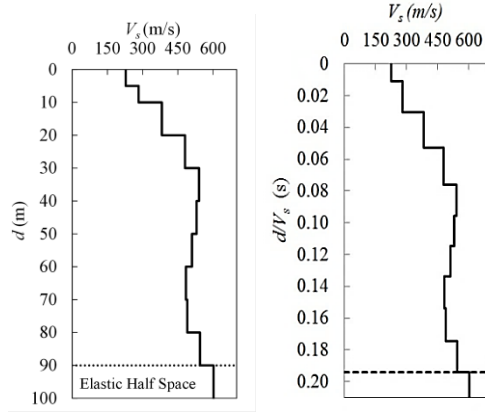
Figure 5-1. Schematic description of control motion on free field and rock outcrop.

Deconvolution is the procedure of acquiring the bedrock motion from a known free field motion. With the lack of representative control motions for rock, as well as the difficulty in accurately characterizing the stratigraphy down to a depth of engineering bedrock, deconvolving free-field motions becomes a feasible alternative, if the layers are supposed to show linearly elastic or viscoelastic response to stresses (EPRI, 1988). However, Silva (1988) states that the frequencies above 15 Hz can be heavily contaminated for accelerograms by high-frequency surface waves and by waves reflected from stratified formations. Therefore, it is recommended that the control motion at free field should be filtered by a 15 Hz lowpass filter to eliminate the tendency of the deconvolution analysis to yield unrealistically large ranges of peak acceleration in deeper layers. In the following section, deconvolution analysis will be described.

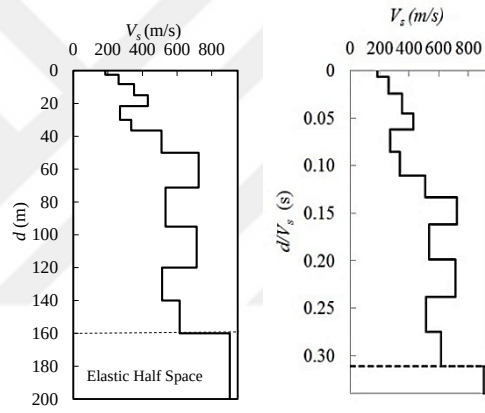
5.4. Deconvolution Analysis

The control motion is defined as the free field motion in deconvolution analyses. A number of accelerograms (38 accelerograms) were selected from the strong-motion databases PEER and DAPHNE to collect a sample for empirical determination of parameters. The records belonging to the site class D were selected for deconvolution analysis. The records are attributed to the seismic events presented in Tables 3 and 4.

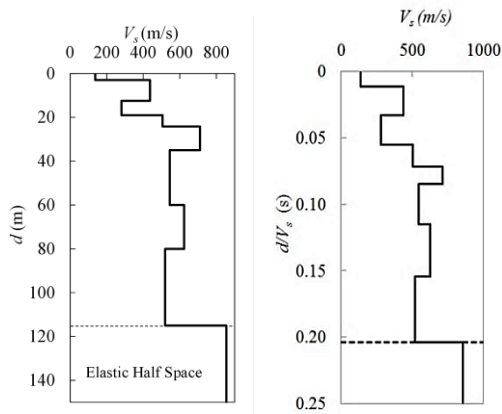
Three real velocity profiles, shown in Figure 5-2, were considered in site-response analyses. The first profile (Figure 5-2.a) is adopted from the site *Bell-La Bula Mail* presented by Nigbor and Swift (2001). The second and third profiles (5-2.b, and 5-2.c) are adopted from the sites *Gilroy #2 (USGS)* and *Palo Alto Veterans Hospital* presented by Boore et al. (2003). An elastic half space representing the soft rock was considered to be lying beneath the visco-elastic layers. The distribution of V_s with respect to d/V_s , total travel time of shear waves from free boundary (top) to the depth d in three sites is shown in this Figure too. d was taken as the average depth of each uniformly viscoelastic layer. The material density of all actual formations was set to 2000 kg/m^3 , so that the results of analyses can be solely attributed to the distribution of V_s .



(a)



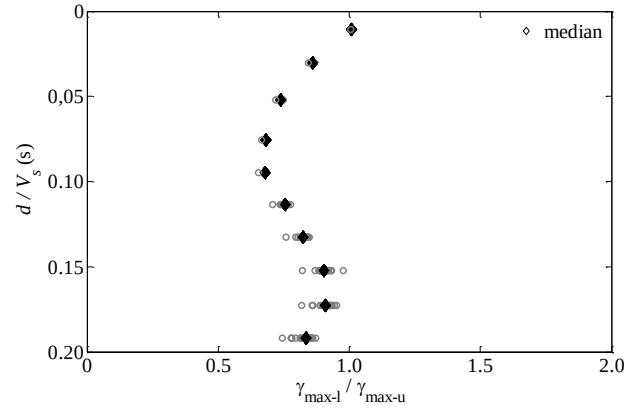
(b)



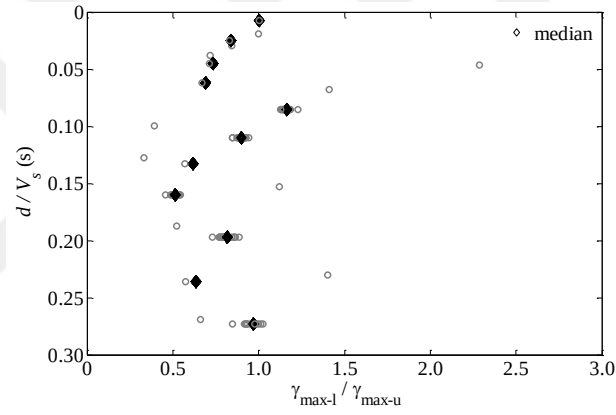
(c)

Figure 5-2. Velocity profile used for the site response analyses for (a) *Bell - La Bulk Mail site*, for (b) *Gilroy #2 (USGS)*, and for (c) *Palo Alto Veterans Hospital*.

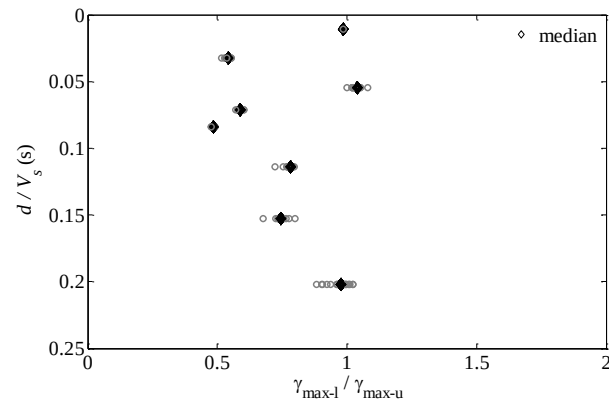
The program PROSHAKE, explained in section 3.3.3, is used for the dynamic site response analysis. Figure 5-3 shows the range of geometric means of maximum strain ratio ($\gamma_{\max-l}/\gamma_{\max-u}$) for two horizontal components of ground motion used in PROSHAKE analyses, such that a linear relationship between stress and strain is considered. $\gamma_{\max-l}$ denotes maximum absolute shear strain computed by PROSHAKE and is supposed to provide exact solution for $\gamma_{\max}$. $V_s \cdot \gamma_{\max}$ is estimated by Equation (3-5b) or (3-6). d/V_s can be calculated for layered medium using Equation (3-31) and the procedure described in section 3.3.3. Then, the estimation for $\gamma_{\max}$ is calculated by supposing that $V_s = (V_s)_l$. Here $\gamma_{\max}$ represents the maximum shear strain in a uniform deposit, and is estimation for $\gamma_{\max-l}$, and therefore can be replaced by the symbol $\gamma_{\max-u}$. The comparisons of Figures 5-2 and 5-3 reveal that accuracy of this estimation method is sensitive to the variability in V_s . An investigation of the ratio between V_s in soft layers and that in stiffer layers shows that the method may slightly underestimate $\gamma_{\max}$ in soft layers located beneath much stiffer layers. However the predictions are conservative for most of the layers. For instance, the lowest V_s ratio of the layer with respect to the upper layer (0.62) occurs at the travel time $d/V_s = 0.07$ s at the site *Gilroy #2 (USGS)*, such that $\gamma_{\max-l}/\gamma_{\max-u}$ is 1.16. In contrast, at the site *Bell - La Bulk Mail site*, the V_s ratio is the highest (0.94) for $d/V_s = 0.14$ s, but $\gamma_{\max-l}/\gamma_{\max-u}$ is almost 1.0. Whereas, at the site *Palo Alto Veterans Hospital* the V_s ratio is considerably low (0.64) for $d/V_s = 0.05$ s, which is close to the minimum ratio at the site *Gilroy #2 (USGS)*, but the underestimation of $\gamma_{\max}$ is negligible. One important observation is that, the scattering of $\gamma_{\max-l}/\gamma_{\max-u}$ for any given d/V_s is very limited.



(a)



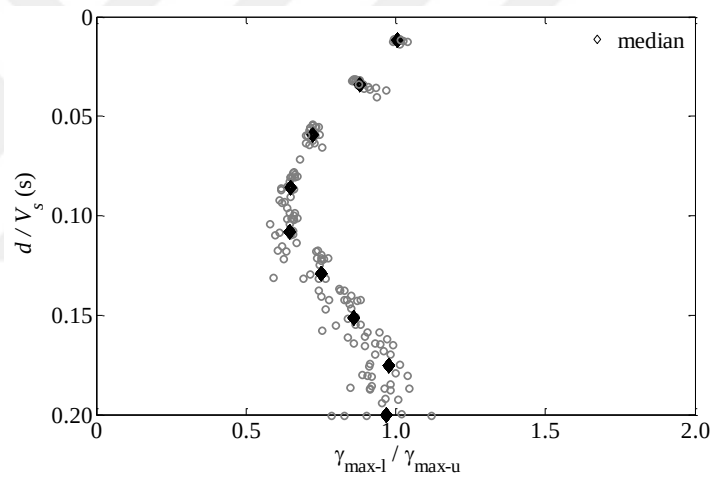
(b)



(c)

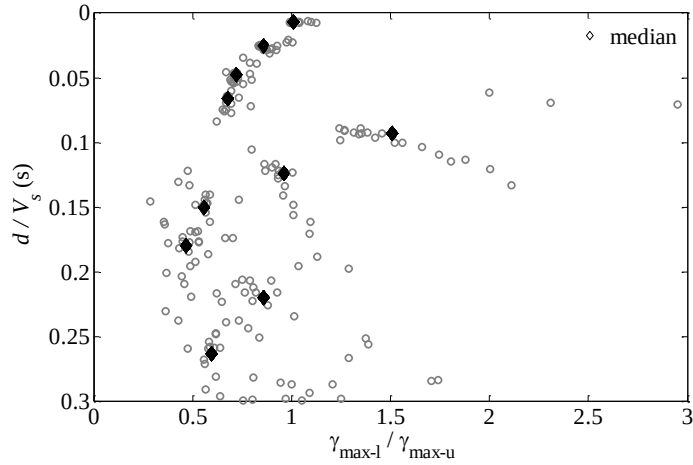
Figure 5-3. The ranges of $\gamma_{\max-l} / \gamma_{\max-u}$ computed by supposing linear response of layers on the sites (a) *Bell - La Bulk Mail*, (b) *Gilroy #2 (USGS)*, and (c) *Palo Alto Veterans Hospital*.

In order to investigate the effect of nonlinearity of soils' response to shearing, the same profiles were analyzed by using the relationships between ξ , $(G/G_{\max})$ and cyclic amplitude of γ that were suggested by Vucetic and Dobry (1991). Figure 5-4 shows the scattering of $\gamma_{\max-l}/\gamma_{\max-u}$ by d/V_s . The comparisons of Figures 5-3 and 5-4 reveal that the scattering of $\gamma_{\max-l}/\gamma_{\max-u}$ and underestimation of $\gamma_{\max}$ by the concept of equivalent travel time in embedded soft layers can be more prominent if the soils behave nonlinearly. This can be explained by the reduction in G , consequently in equivalent V_s . Except for these soft layers embedded in much stiffer layers, Equation (3-5.b) usually yields reasonable or conservative estimates for $\gamma_{\max}$.

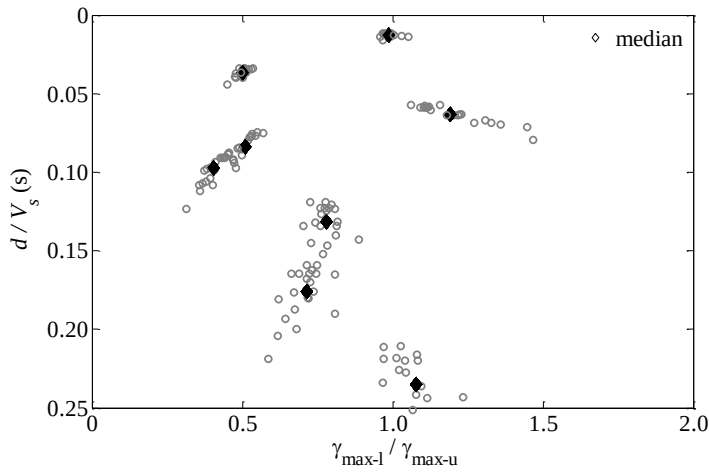


(a)

Figure 5-4. The ranges of $\gamma_{\max-l}/\gamma_{\max-u}$ computed by supposing deconvolution of free-field motion assigned for the sites (a) *Bell - La Bulk Mail*, (b) *Gilroy #2 (USGS)*, and (c) *Palo Alto Veterans Hospital*.



(b)



(c)

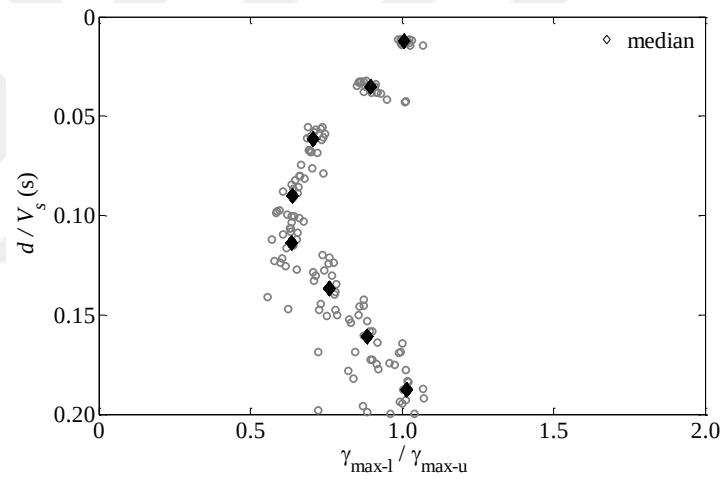
Figure 5-4. Continued.

5.5. Convolution Analysis

The control motion is defined as the rock outcrop motion in convolution analyses. In these analyses series, records adopted from PEER and Daphne were used as input motions. PEER database provided filtered records. The records are attributed to the seismic events presented in Table 4. The records from sites classified as *C* in Table 3 are supposed as the rock outcrop motion. Three real velocity profiles, shown in Figure 5-2 were considered in site-response analyses. The material density of all actual formations

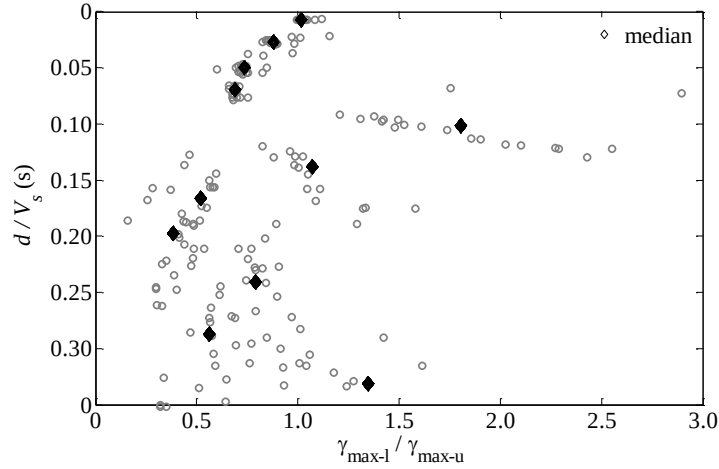
was set to 2000 kg/m^3 . In order to prevent 1% exceeding of $\gamma_{\max}$, records #15 and #40 were scaled by the factor 0.80 and 0.45.

Figure (5-5) shows the range of geometric means of maximum strain ratio ($\gamma_{\max-l}/\gamma_{\max-u}$) for two horizontal components of ground motion used in convolution analyses for the site *Gilroy #2 (USGS)*. The results reveal that accuracy of $\gamma_{\max-l}/\gamma_{\max-u}$ is sensitive to the variability in V_s . Furthermore, scattering of $\gamma_{\max-l}/\gamma_{\max-u}$ and underestimation of $\gamma_{\max}$ for embedded soft layers can be more prominent if the soils behave nonlinearly. Also the control motion type does not change the results significantly.

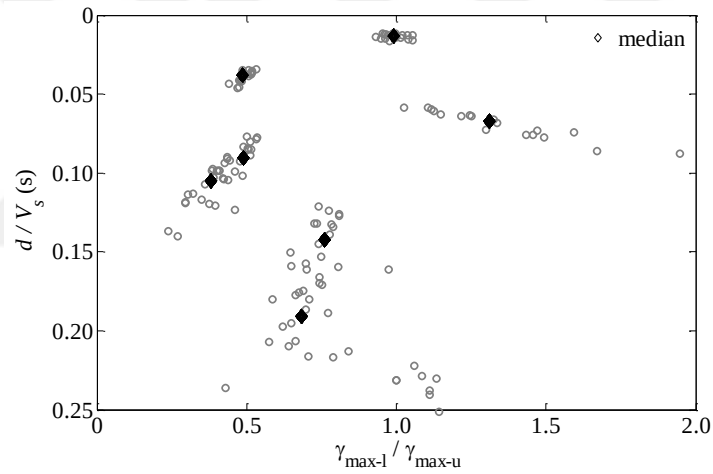


(a)

Figure 5-5. The ranges of $\gamma_{\max-l}/\gamma_{\max-u}$ computed by considering convolution nonlinear response of geological layers on the sites: (a) *Bell - La Bulk Mail*, (b) *Gilroy #2 (USGS)*, and (c) *Palo Alto Veterans Hospital*.



(b)



(c)

Figure 5-5. Continued.

5.6. Discussion

$\gamma_{\max-l}/\gamma_{\max-u}$ gives reasonable and conservative results for more uniform profiles. This ratio is underestimated in soft layers embedded between relatively stiffer layers. This issue is not discussed in codes such as FHWA (2009) that have proposed simplified relationships to estimate $\gamma_{\max}$. As the ratio of shear wave velocity in lower soft layer with respect to upper stiffer layer decreases, the underestimation of $\gamma_{\max}$ becomes more severe. For instance, in linear analysis, as it can be observed in Figure 5-3.b, the lowest

V_s ratio occurs at $d/V_s = 0.07$. $\gamma_{\max-l}/\gamma_{\max-u}$ of median at that travel time is the highest (1.16) in the profile. As V_s ratio increases, $\gamma_{\max-l}/\gamma_{\max-u}$ of median approaches to 1.0. For layers with high V_s ratio, the $\gamma_{\max-l}/\gamma_{\max-u}$ is conservative. As an example in Figure 5-3.b, the V_s ratio at $d/V_s = 0.16$ with respect to upper layer is 1.42 and consequently the $\gamma_{\max-l}/\gamma_{\max-u}$ is the lowest. In nonlinear analyses, the scattering of $\gamma_{\max-l}/\gamma_{\max-u}$ increases. Whether the control motion is defined as a free field motion or on a bedrock outcrop does not have a significant effect on the conclusions. A nonlinear dynamic site-response analysis is necessary particularly for relatively soft layers.





CHAPTER 6

CONCLUSIONS

6.1. Conclusions

Semi-empirical relationships between the maximum absolute shear strain ($\gamma_{\max}$) in imperforated ground, depth (d), and a set of ground-motion amplitude parameters are developed. These amplitude parameters are frequently used for quantifying the seismic hazard on a site. First a semi-empirical relationship between $\gamma_{\max}$, Peak Ground Acceleration (PGA), the Peak Ground Velocity (PGV), and d is proposed. In the second one, PGA and PGV are substituted by ordinates of a pseudo-spectral acceleration (PSA) spectrum. A number of accelerograms were selected from the strong-motion databases to gather a sample for empirical estimation of parameters. Random vibration theory is used to develop a theoretical solution for $\gamma_{\max}$ in a uniformly elastic half-space. The statistical properties of ground motion are represented by power spectral density function assuming a stationary process. Regarding one-dimensional shear-wave propagation, transfer functions between amplitudes of ground motion and shear strain at the depth of interest is formulated in frequency domain. $\gamma_{\max}$ can be expressed as the product of a peak factor and root mean square of shear strain. The final coefficients that are related to the ratio of peaks are empirically determined by using the sample of ground-motion records. In the next step, the effect of material damping is studied. It is shown that damping causes an unrealistic increase in $\gamma_{\max}$ in greater ranges of d/V_s for a number of accelerograms; however low-pass filtering at 15 Hz can diminish this effect.

By using equivalent linear method and the concept of equivalent travel time, the applicability of the relationship proposed for estimation of $\gamma_{\max}$ was examined for a layered medium. It is shown that the parameter $\gamma_{\max}$ is strongly related to the travel time of shear waves from the free field boundary to the depth of interest. The travel time of vertically incident shear waves in layered medium is supposed to be equivalent to the

ratio of depth to the shear-wave velocity (d/V_s) in uniformly elastic domain. It is concluded that the concept of equivalent travel time can be used as a reasonable preliminary estimator for $\gamma_{\max}$ in horizontally layered geological formations. Both relationships yield either reasonably accurate or conservative estimates for $\gamma_{\max}$ in layered geological formations. Other significant findings of this study are explained below:

- The transition from PGA-sensitiveness of $\gamma_{\max}$ to its PGV-sensitiveness occurs around the depth of $(\text{PGV} \cdot V_s)/(\sqrt{2} \cdot \text{PGA})$ in uniformly viscoelastic medium.
- $\gamma_{\max}$ is proportional to d and PGA in shallow geological formations, and reaches to figures around $(\text{PGV}/V_s)/\sqrt{2}$ by increasing depth in uniformly viscoelastic medium.
- The method of equivalent linearization can be easily introduced in calculation of $\gamma_{\max}$, so that nonlinear response of geological materials to severe seismic deformations can be taken into account approximately, provided that the level of nonlinearity in any layer does not hinder the applicability of the equivalent linearization method.
- The accuracy of estimated $\gamma_{\max}$ in case a PSA spectrum is used is not better than that estimated by PGA and PGV. The proposed semi-empirical relationships between $\gamma_{\max}$ and these amplitude parameters can be useful for first-order estimations of $\gamma_{\max}$, particularly in the absence of the data necessary for more accurate analysis methods.
- The estimation for $\gamma_{\max}$ in soft layers embedded in much stiffer layers is sensitive to the nonlinearity of material's response to shearing. As the reduction in G and consequently V_s increases, the underestimation of $\gamma_{\max}$ by using the concept of equivalent travel time increases too. Thus the procedure explained in this study may result in underestimation of $\gamma_{\max}$ in such soft layers.
- The PSA-dependent relationship is sensitive to the shape of spectrum. Matching the spectral accelerations with desired design spectrum, which usually has a

smoothly varying shape, reveal that records with the error in prediction of $\gamma_{\max}$ is more limited in case a design spectrum is used for estimation of $\gamma_{\max}$.

- The PSA-dependent relationship may produce unconservative estimations of $\gamma_{\max}$, if the short-period spectral amplitudes reach to relatively high figures with respect to those of long-period spectral amplitudes.

6.2. Future Works

This study can be developed in the following ways:

- The semi-empirical relationships can be compared with experimental studies.
- A more comprehensive study in layered media implementing the matrix of transfer function between free field motion and $\gamma_{\max}$ at any depth will improve the estimation methods.
- The propagation of surface waves can be theoretically analyzed to develop a relationship between ground-motion parameters and $\gamma_{\max}$ for cases in which surface waves other than shear waves are of concern.
- 2-D and 3-D analyses can be performed and the results can be compared with those of 1-D analyses in order to understand the limitations originating from the assumptions of this study.



REFERENCES

- Abrahamson, N., Non-stationary Spectral Matching. Seismological Research Letters, Vol.63, No.1, 1992.
- Akkar, S. and Bommer, J., Empirical Prediction Equations for Peak Ground Velocity Derived from Strong-Motion Records from Europe and the Middle East, Bulletin of the Seismological Society of America , Vol.97, No.2, pp. 511–530, 2007.
- Akkar S, Bommer J., Influence of Long-Period Filter Cut-Off on Elastic Spectral Displacements. Earthquake Engineering and Structural dynamics, Vol.35, No.9, pp.1145–1165, 2006.
- Al Atik, L., and Abrahamson N., an Improved Method for Nonstationary Spectral Matching. Earthquake Spectra, August 2010, Vol.26, No.3, pp. 601-617, 2010.
- American Lifelines Alliance, Seismic Fragility Formulations for Water Systems. Part 1-Guideline. Technical report prepared by a public–private partnership between FEMA and ASCE, 104 pages, 2001a.
- American Lifelines Alliance, Seismic Fragility Formulations for Water Systems. Part 2-Appendices. Technical report prepared by a public–private partnership between FEMA and ASCE, 239 pages, 2001b.
- Amini A., Trifunac M.D., Statistical Extension of Response Spectrum Superposition. Soil Dynamics and Earthquake Engineering, Vol.4, No.2, pp. 54-63, 1985.
- American Society of Civil Engineers. ASCE/SEI Standard 7-05: Minimum Design Loads for Buildings and Other Structures, 2006.
- Amorosi A., Boldini D, Falcone G., Numerical Prediction of Tunnel Performance during Centrifuge Dynamic Tests. Acta Geotechnica, Vol.9, No.4, pp.581-596, 2014.

- Aydan O., Ohta Y., Genis M., Tokashiki N., Ohkubo K., Response and Stability of Underground Structures in Rock Mass During Earthquakes. *Rock Mechanics and Rock Engineering*, Vol.43, No.6, pp.857–875, 2010.
- Baker J.W., Cornell C.A., Which Spectral Acceleration Are You Using? *Earthquake Spectra*, Vol.22, No.2, pp. 293–312, 2006.
- Bommer J.J., Acevedo A.B., the Use of Real Earthquake Accelerograms as Input to Dynamic Analysis. *Journal of Earthquake Engineering*, Vol.8, (Special Issue I), pp.43-91, 2004.
- Boore, D. M. and Akkar, S., Effect of Causal and Acausal Filters on Elastic and Inelastic Response Spectra, *Earthquake Engineering and Structural Dynamics*, Vol.32, No.11, pp. 1729-1748, 2003.
- Boore, D.M., Bommer J.J., Processing of Strong-Motion Accelerograms: Needs, Options and Consequences, *Soil Dynamics and Earthquake Engineering*, Vol.25, No.10, pp. 93-115, 2005.
- Boore, D.M., Gibbs, F. and Rodriguez, M., A Compendium of P- And S-Wave Velocities from Surface-To-Borehole Logging; Summary and Reanalysis Of Previously Published Data And Analysis Of Unpublished Data. Open-File Report 2003-191, USGS, California, United States of America, 2003.
- Borcherdt, RD., Estimates of Site-Dependent Response Spectra for Design (Methodology and Justification). *Earthquake Spectra*, Vol.10, No.4, pp.617-653, 1994.
- Building Seismic Safety Council, BSSC. NEHRP Recommended Seismic Provisions for New Buildings And Other Structures, FEMA P-750, part 1 – provisions and part 2 – commentary. Federal Emergency Management Agency, Washington DC, 2003.

- Causse, M., Laurendeau, A., Perrault, M., Douglas, J., Bonilla, L. F. and Guéguen, P., Eurocode 8-Compatible Synthetic Time-Series as Input to Dynamic Analysis, Bulletin of Earthquake Engineering, Vol. 12, No.2, pp.755–768, 2014.
- Chapra, S.C., Canale, R.P., Numerical Methods for Engineers, McGraw-Hill, New York, 2010.
- CEN, Eurocode 8-Design of Structures for Earthquake Resistance. Part 4: Silos, Tanks and Pipelines. EN 1998-4, Final draft, January 2006, Brussels.
- Chai, Y.H., and Faijfar, P., A Procedure for Estimating Input Energy Spectra for Seismic Design, Journal of Earthquake Engineering, Vol.4, No.4, pp.539-561, 2000.
- Chen, CH. and Hou, PC., Response Spectrum of Ground Shear Strain. 10th World Conference on Earthquake Engineering, Rotterdam, 1992.
- Chopra, A.K., Dynamics of Structures: Theory and Applications to Earthquake Engineering, Prentice-Hall, New Jersey, 2007.
- Cimellaro, G., Reinhorn, A., D'Ambrisi, A., and De Stefano, M., Fragility Analysis and Seismic Record Selection, Journal of Structural Engineering, Vol. 137, No.3, pp. 379-390, 2011.
- Clough, R.W., Penzien, J., Dynamics of structures, McGraw-Hill, New York, 2003.
- Cunningham, E.P., Digital Filtering: An Introduction, Wiley and Sons, New York, 1995.
- Davenport A.G., Note on the Distribution of the Largest Value of a Random Function with Application to Gust Loading, ICE Proceedings, Vol. 28, No.2, pp. 187 –196, 1964.
- Daphne, Strong Ground Motion Database of Turkey. (15 May 2014). http://daphne.deprem.gov.tr/2K/daphne_v4.php.

Der Kiureghian A., A Response Spectrum Method for Random Vibration, Report No. UCB/ EERC-80/15, Earthquake Engineering Research Center, Berkeley, California, pp.1-31, 1980.

Devore J.L., Probability and Statistics for Engineering and the Sciences, 8th Edition, Cengage Learning, 2012.

EduPro Civil Systems., PROSHAKE – Ground Response Analysis Program, EduPro Civil Systems Inc Redmond, Washington., 1998.

Electronic Power Research Institute (EPRI). Site Response to Earthquake Ground Motion, NP-5747, project 2556-7, California, pp.1-306, 1988.

Facciorusso, J., Madiati, C. and Vannucchi, G., A Grade-3 Method of Zonation for Seismic Slope Stability: An Italian Case Study. Soil Dynamics and Earthquake Engineering, Vol. 36, No.5, pp. 96-110. 2012.

Federal Highway Administration (FHWA)., Seismic Retrofitting Manual for Highway Structures: part 2 – Retaining Structures, Slopes, Tunnels, Culverts and Roadways. Final Report No. FHWA-HRT-05-067. Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo: New York, pp.162-163, 2004.

Federal Highway Administration (FHWA)., Technical Manual for Design and Construction of Road Tunnels- Civil Elements. Publication No. FHWA-NHI-10-034, the United States, pp.1-702, 2009.

Fouad, L., Rathje, E.M., RSPMatch09, <https://nees.org/resources/rpsmatch09>, 2012.

Gerolymos N., Gazetas G., Constitutive Model for 1-D Cyclic Soil Behavior Applied to Seismic Analysis. Soils and Foundation, Vol.45, No.3, pp. 147-159, 2005.

Gomes R.C., Gouveia F., Torcato D., Santos J., Seismic Response of Shallow Circular Tunnels in Two-Layered Ground. Soil Dynamics and Earthquake Engineering, Vol.75, No.8, pp. 37-43, 2015.

- Grant, D. N., and Diaferia, R., Assessing Adequacy of Spectrum-Matched Ground Motions For Response History Analysis. *Earthquake Engineering & Structural Dynamics*, Vol. 42, No.9, pp.1265–1280, 2012.
- Hancock, J., Watson-Lamprey, J., Abrahamson, N. A., Bommer, J. J., Markatis, A., McCoy, E., and Mendis, R., an Improved Method of Matching Response Spectra of Recorded Earthquake Ground Motion Using Wavelets, *Journal of Earthquake Engineering*, Vol.10,pp. 67-89, 2006.
- Hashash, YMA, Hook, JJ, Schmidt, B and Yao JIC., Seismic Design and Analysis of Underground Structures. *Tunneling and Underground Space Technology*, Vol.16, No.4, pp. 247–293, 2001.
- Haykin, S., Moher, M., *Introduction to Analog and Digital Communications*, Wiley and Sons, 2nd edition, USA, 2007.
- Heredia-Zavoni, E., Vanmarcke, E.H., Seismic Random Vibration Analysis of Multi support Structural Systems. *Journal of Engineering Mechanics*, Vol. 120, No.5, pp.1107-1128, 1994.
- Iervolino,I., De Luca, F., Cosenza, E., Spectral Shape-Based Assessment of SDOF Nonlinear Response to Real, Adjusted And Artificial Accelerograms, *Engineering Structures*, Vol. 32 , No.9, pp.2776–2792, 2010.
- Imai T, Tonouchi K. and Kanemori T., the Simple Evaluation Method of Shear Stress Generated by Earthquakes in Soil Ground. *Bureau of Practical Geological Investigation: Japan*, pp. 39-58, 1981.
- International Code Council, *International Building Code*, the United States, 2009.
- Kanasewich, E.R., *Time Sequence Analysis in Geophysics*, The University of Alberta Press, Alberta, 1981.

- Kanai, K., Semi-Empirical Formula for the Seismic Characteristics of the Ground Motion. Bulletin of the Earthquake Research Institute, University of Tokyo Vol.35, No.2, pp.308-325, 1957.
- Katsanos, E.I., Sextos, A.G. and Manolis, G.D., Selection Of Earthquake Ground Motion Records: A State-Of-The-Art Review from a Structural Engineering Perspective, Soil Dynamics and Earthquake Engineering, Vol.30, No.4, pp.157-169, 2010.
- Kramer, S.L., Geotechnical Earthquake Engineering, Prentice Hall, 1996.
- Kontoe, S, Zdravkovic, L, Potts, D.M. and Menkiti, C.O., On the Relative Merits of Simple and Advanced Constitutive Models in Dynamic Analysis of Tunnels. Geotechnique, Vol.61, No.10, pp.815-829, 2011.
- Kontoe, S, Avgerinos, V. and Potts, D.M., Numerical Validation of Analytical Solutions and Their Use for Equivalent Linear Seismic Analysis of Circular Tunnels. Soil Dynamics and Earthquake Engineering, Vol.66, No.11, pp.206-219, 2014.
- Lilhanand, K. and Tseng, W.S., Development and Application of Realistic Earthquake Time Histories Compatible with Multiple-Damping Design Spectra, Ninth World Conference on Earthquake Engineering, Vol. 2, August 2-9, pp.819-824,1988.
- Lin, J., Li, J., Zhang, W., and Williams, F.W., Random Seismic Responses of Multi-Support Structures in Evolutionary in Homogenous Random Fields. Earthquake Engineering and Structural Dynamics, Vol.26, No.1, pp.135-145, 1997.
- Luco JE, Wong HL. Response of a rigid foundation to a spatially random ground motion. Earthquake Engineering and Structural Dynamics, Vol.14, No.6, 1986.
- Mathworks. (2016). <https://www.mathworks.com/help>.
- Newland, D.E., An Introduction to Random Vibrations, Spectral and Wavelet Analysis. Dover Publications, Third edition, pp.150-165, 1993.

- Newmark N.M. Problems in Wave Propagation in Soil and Rock: Symposium on Wave Propagation and Dynamic Properties of Earth Materials, August 23-25, Univ. of New Mexico Press, pp.7-26, 1968.
- Nigbor, R.L. and Swift J.N., Resolution of Site Response Issues from the Northridge Earthquake, ROSRINE/ USC-01, Revision 0.9, 26 November, 2001.
- O'Rourke, M. and El Hmadi, K., Analysis of Continuous Buried Pipelines for Seismic Wave Effects. Earthquake Engineering and Structural Dynamics, Vol.16, No.6, pp. 917-929, 1988.
- Owen G.N., Scholl R.E., Earthquake Engineering of Large Underground Structures. Report No. FHWA/RD 80/195, Federal Highway Administration, Washington, DC, 1981; 38-73.
- Ozbulut, O.E., and Hurlbaas, S., Seismic Assessment of Bridge Structures Isolated by a Shape Memory Alloy-Based Isolation Devices. Structures Congress, pp.2066-2077, 2011.
- Paolucci, R. and Pitilakis, K. Seismic Risk Assessment of Underground Structures under Transient Ground Deformations: Earthquake Geotechnical Engineering, Volume 6 of the series Geotechnical, Geological and Earthquake Engineering. pp.433-459, 2007.
- Paolucci, R. and Smerzini, C. Earthquake-induced Transient Ground Strains from Dense Seismic Networks. Earthquake Spectra, Vol. 24, No.2, pp.453–470, 2008.
- PEER Strong Ground Motion Database. (15 May 2014). <http://peer.berkeley.edu/smcat/>.
- Penzien P. Seismically Induced Racking of Tunnel Linings. Earthquake Engineering and Structural dynamics, Vol.29, No. 5, pp.683-691, 2000.

Sandikkaya M.A., Yılmaz M.T., Bakır S, Yılmaz Ö. Site Classification Of Turkish National Strong Motion Stations. Journal of Seismology, Vol.14, No.3, pp.543-563, 2010.

Schnabel, P.B., Lysmer, J. and Seed, H.B., SHAKE: a Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. Report No. UCB/EERC-72/12, Earthquake Engineering Research Center, University of California, Berkeley, pp.1-92, 1972.

Seed H.B., Idriss I.M., Simplified Procedure for Evaluating Soil Liquefaction Potential. Journal of Soil Mechanics and Foundations Division, ASCE, Vol.107, No.9, pp.1249-1274, 1971.

Seismosoft. Seismomatch-A Computer Program for Spectrum Matching Of Earthquake Records, 2015.

Silva, W. Soil Response to Earthquake Ground Motion, EPRI NP-5747 Final Report, Woodward-Clyde Consultants, California, 1988.

St. John C.M. and Zahrah T.F., Aseismic Design of Underground Structures. Tunneling and Underground Space Technology, Vol. 2, No. 2, pp.165 -197, 1987.

Stewart J.P. and Kwok A.O.L., Nonlinear Seismic Ground Response Analysis: Code Usage Protocols and Verification against Vertical Array Data. Geotechnical Earthquake Engineering and Soil Dynamics IV, May 18-22, pp.1-24, 2008.

Sucuoğlu H, and Yılmaz M.T., Düzce, Turkey: A City Hit by Two Major Earthquakes in 1999 within Three Months, Seismological Research Letters, Vol.72, No.6, pp.679-689, 2001.

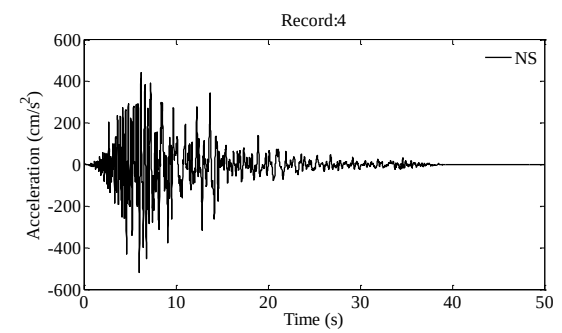
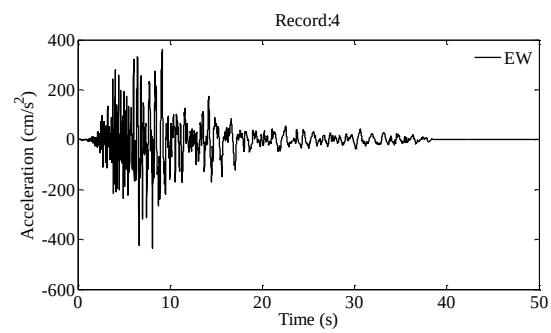
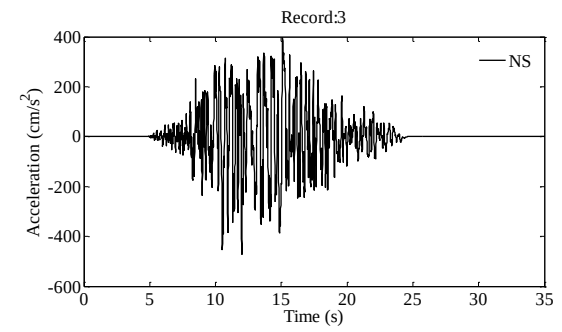
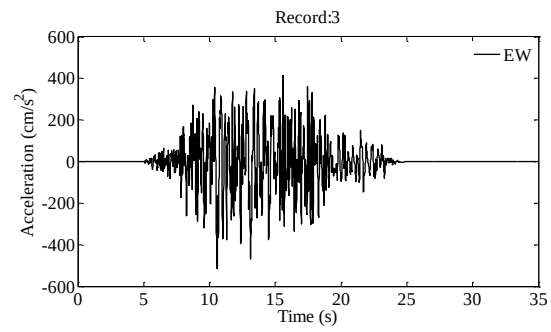
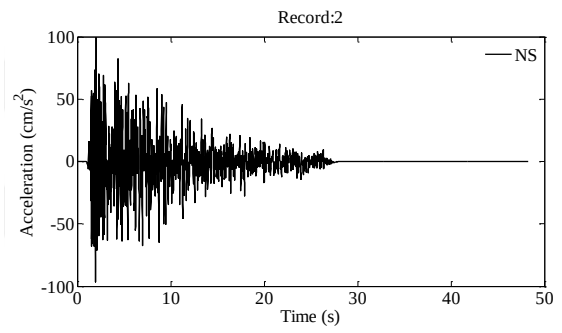
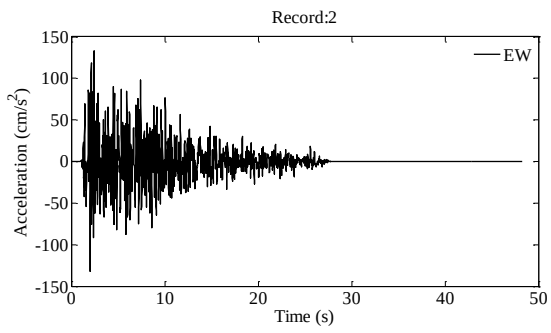
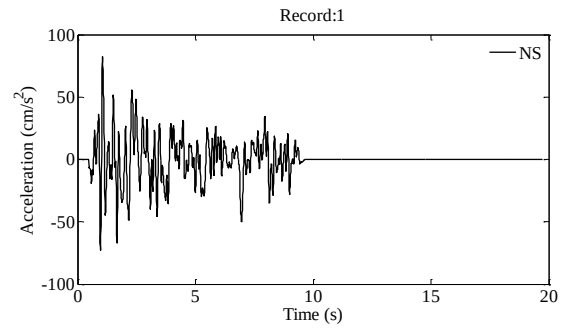
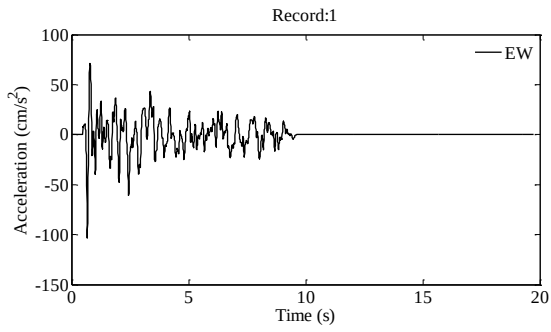
Somerville P.G., Smith N.F., Graves R.W., Abrahamson N.A., Modification of Empirical Strong Ground Motion Attenuation Relations to Include the Amplitude and Duration Effects of Rupture Directivity. Seismological Research Letters, Vol.68, No.1, pp.199-222, 1997.

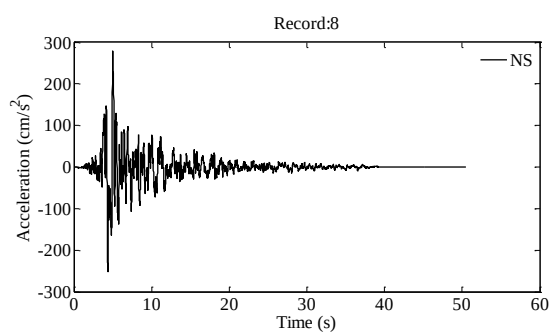
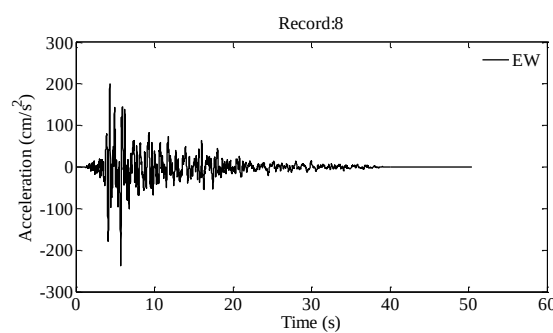
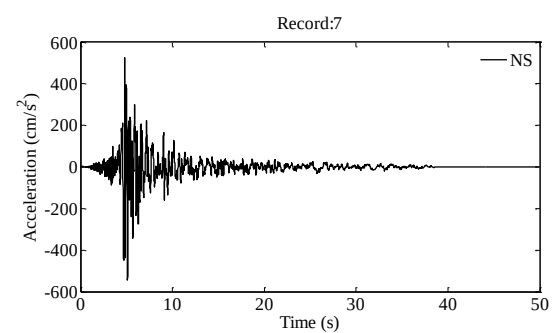
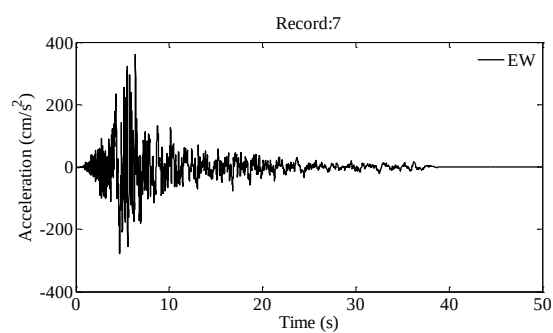
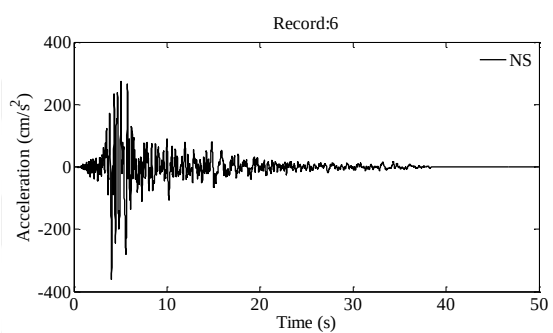
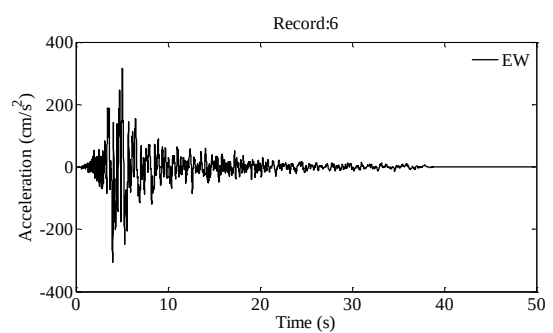
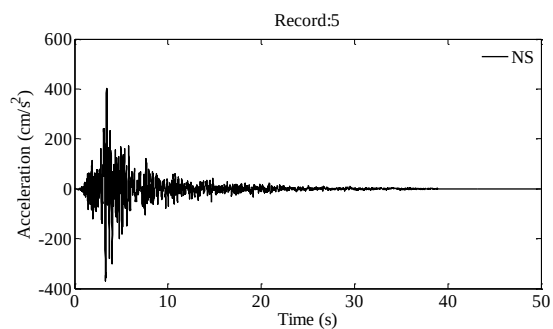
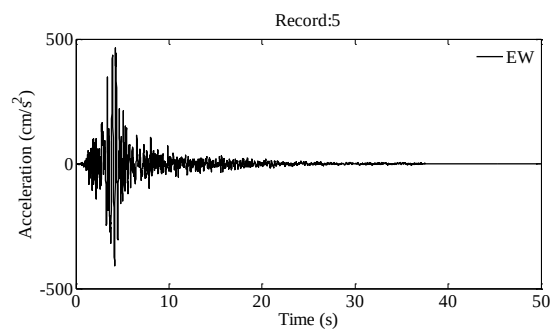
- Tajimi, H., A statistical method of determining the maximum response of a building structure during an earthquake. Second World Conference on Earthquake Engineering, Vol.2, session 2, pp.781-798, 1960.
- Vanmarcke E.H., Seismic Safety Assessment. Book chapter of Random Excitation of Structures by Earthquakes and Atmospheric Turbulence, Volume 225 of the series International Centre for Mechanical Sciences, pp.1-76 (Edt. H. Parkus), Springer, 1977.
- Veletsos A.S., Prasad A.M., Wu W.H., Transfer Functions for Rigid Rectangular Foundations, Earthquake Engineering and Structural Dynamics, Vol.26, No.1, pp.5-17, 1997.
- Vucetic, M. and Dobry, R., Effect of Soil Plasticity on Cyclic Response. Journal of Geotechnical Engineering, Vol.117, No.1, pp.89-107, 1991.
- Wang, J.N., Seismic Design of Tunnels, a Simple State-of-the-Art Design Approach. Parsons Brinckerhoff Inc. Publication, New York, pp.53-133, 1993.
- Wilson E.L., Der Kiureghian A., Bayo E.P., A Replacement for the SRSS Method in Seismic Analysis, Earthquake Engineering and Structural Dynamics, Vol.9, No.2, pp.187-192, 1981.
- Yang C.Y., Random Vibration of Structures. John Wiley & Sons: Canada, pp.201-205, 1986.
- Yoshida, N., Kobayashi, S., Suetomi, I., Miura, K., Equivalent Linear Method Considering Frequency Dependent Characteristic of Stiffness and Damping. Soil Dynamics and Earthquake Engineering, Vol.22, No.3, pp.205-222, 2002.
- Zerva, A., Spatial Variation of Seismic Ground Motions: Modeling and Engineering, Taylor and Francis Group, pp.333-334, 2009.

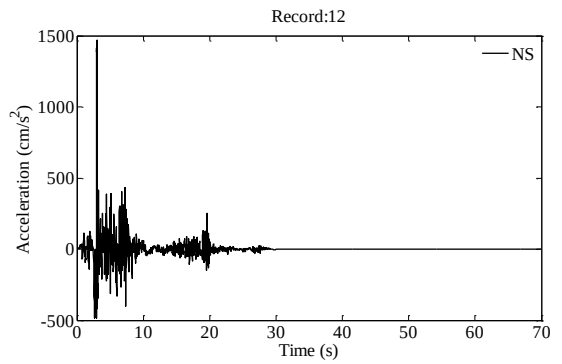
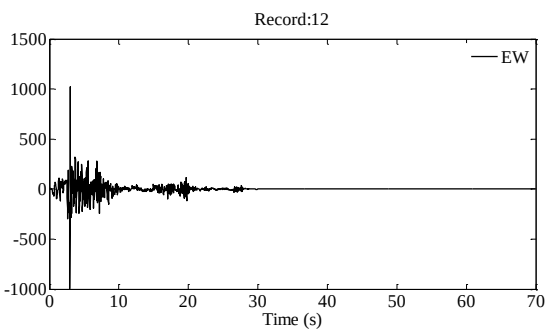
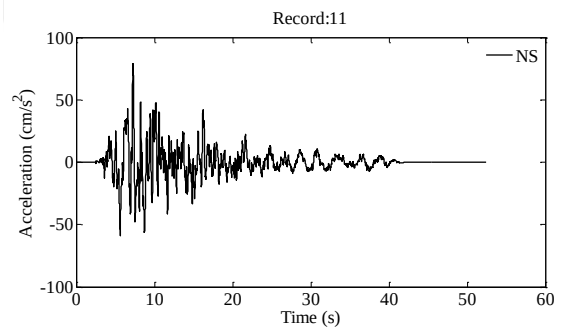
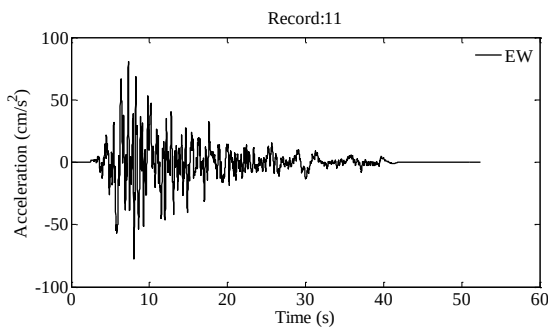
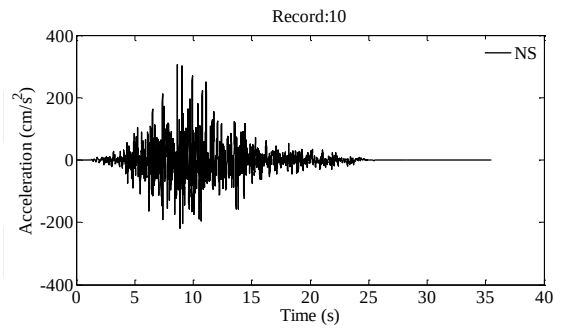
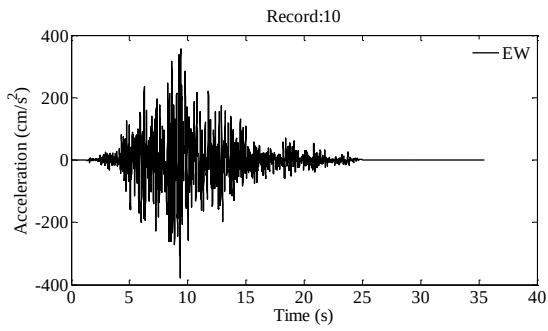
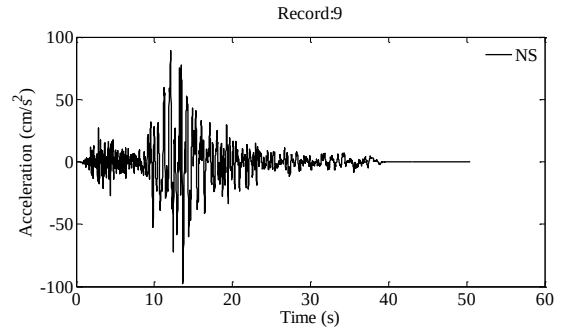
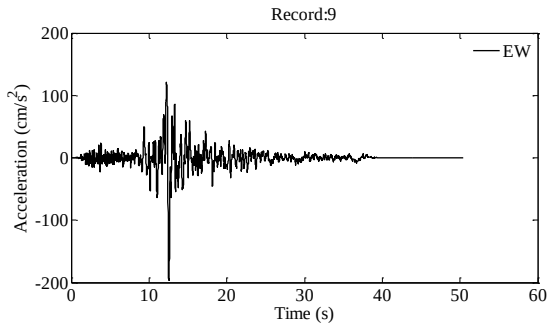


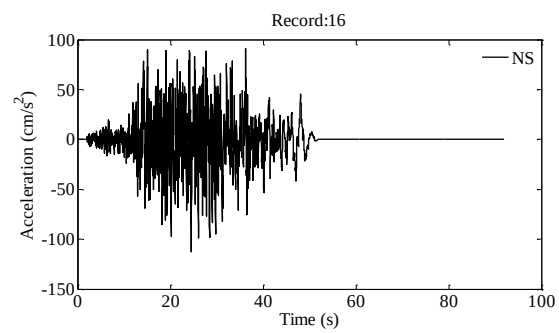
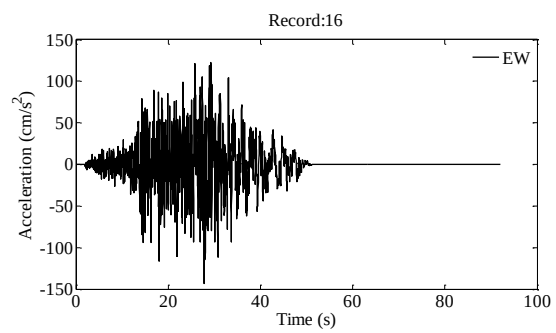
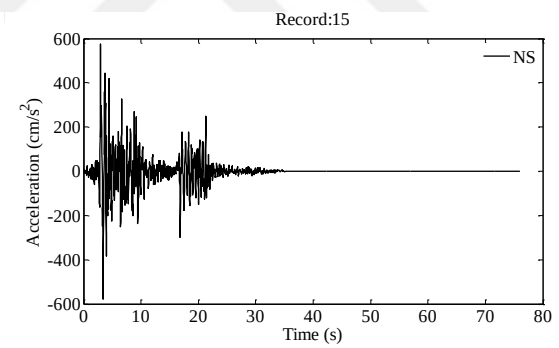
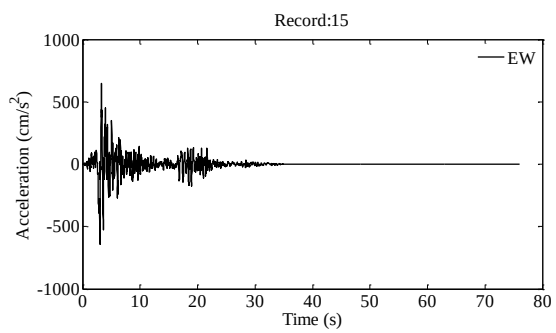
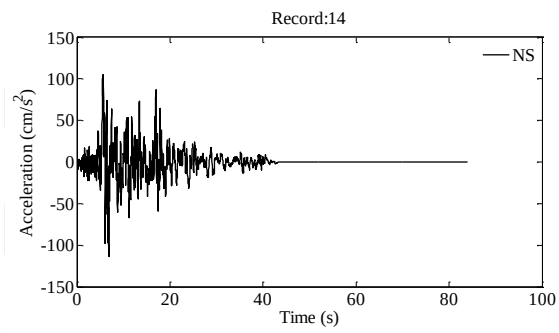
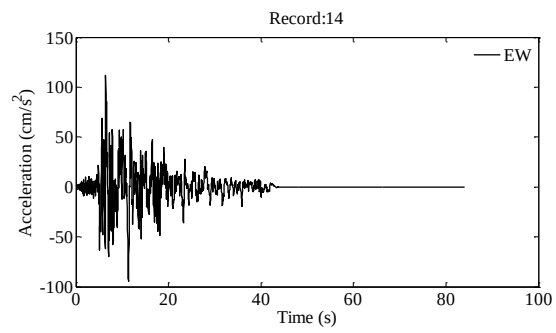
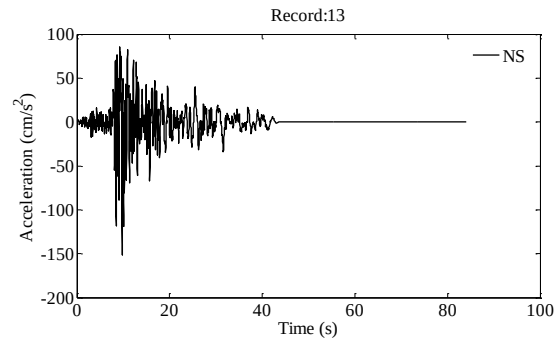
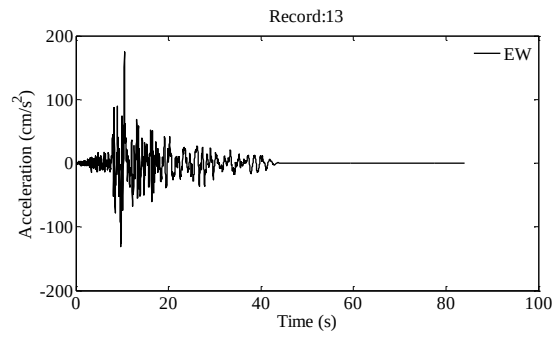
Appendix A

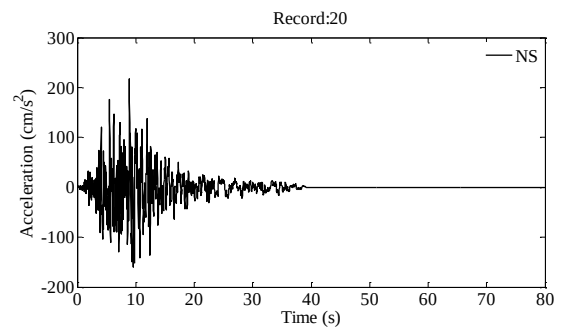
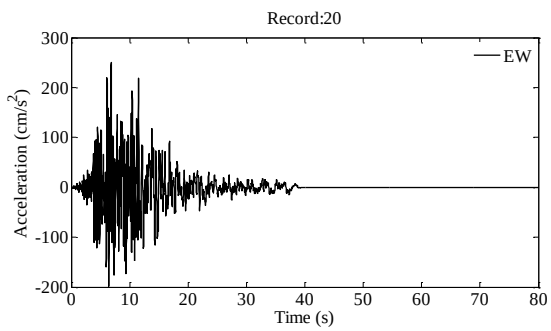
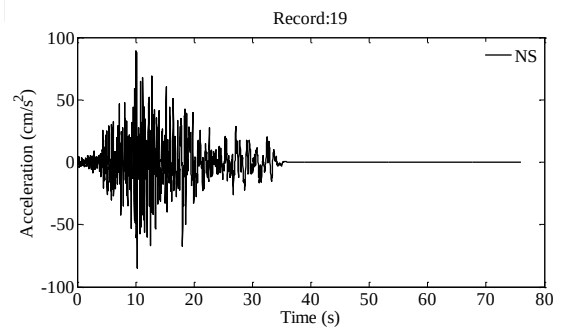
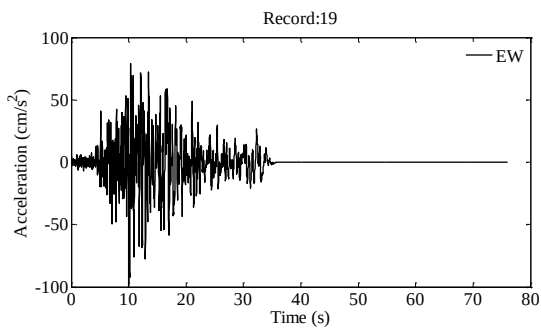
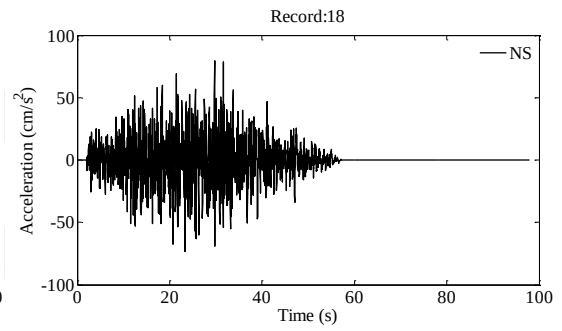
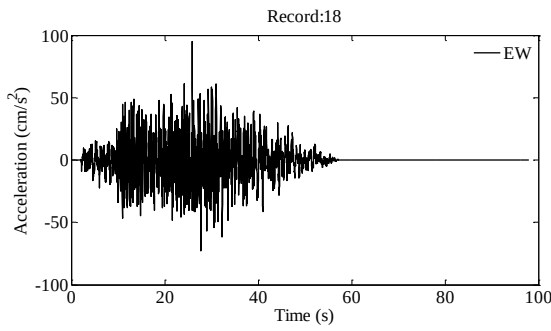
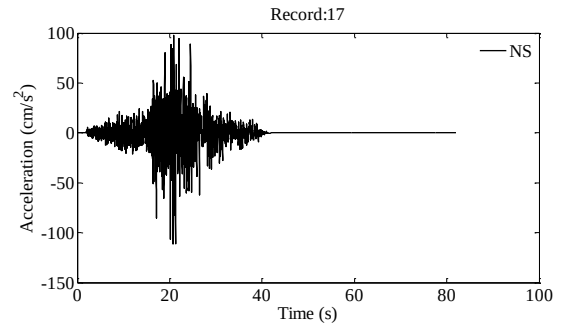
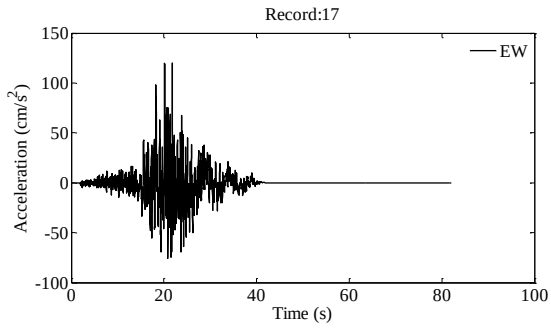
Acceleration Time Histories

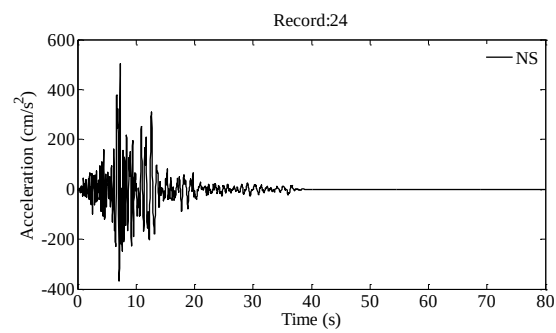
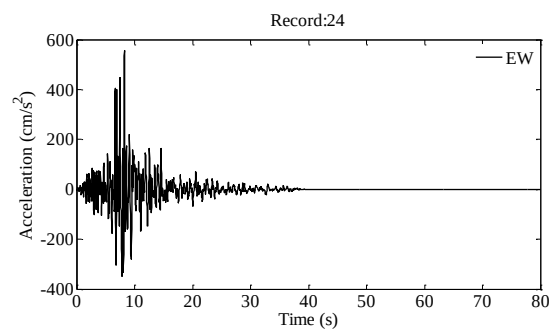
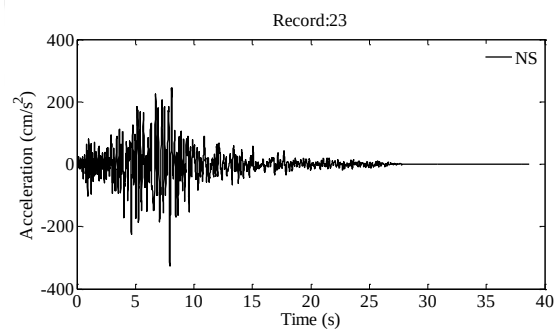
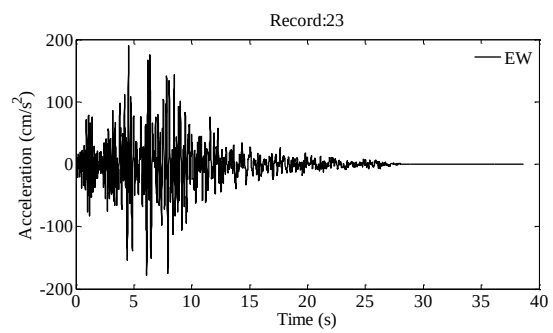
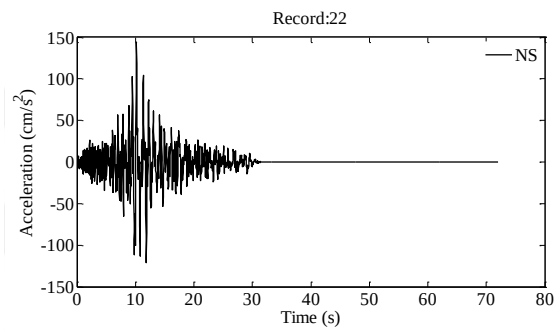
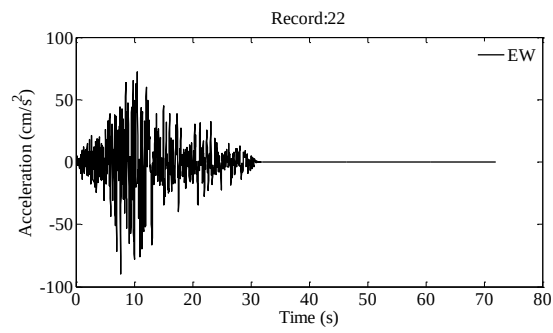
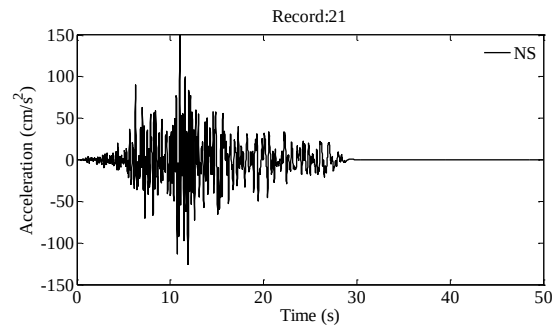
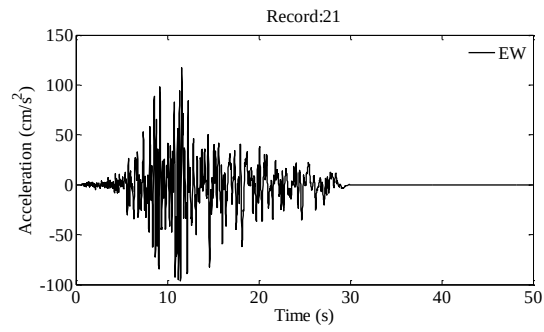


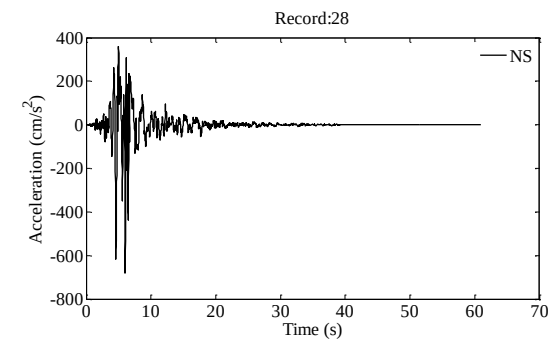
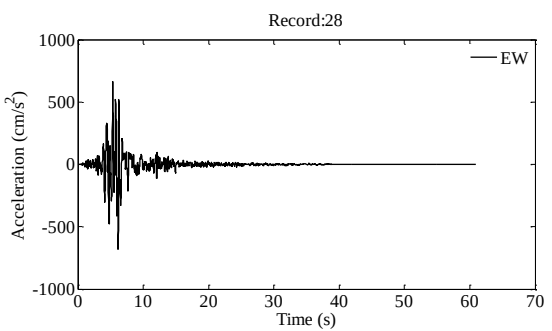
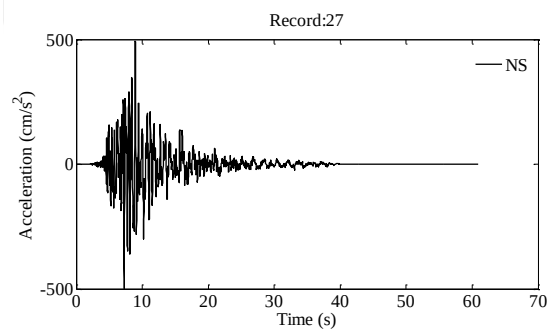
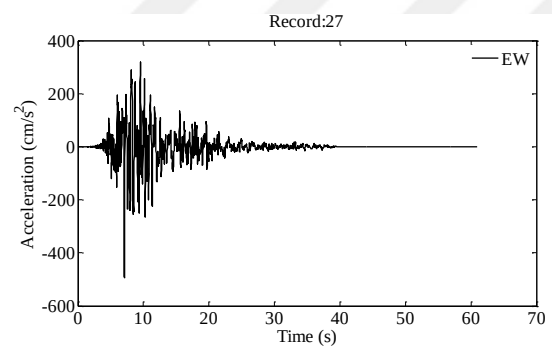
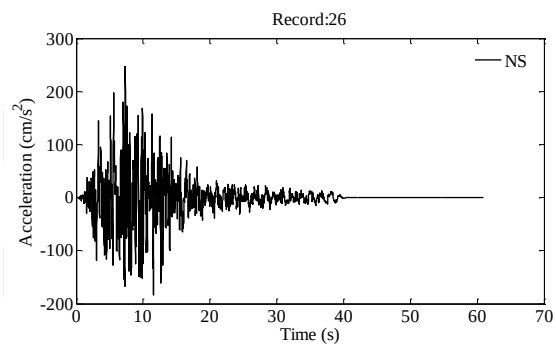
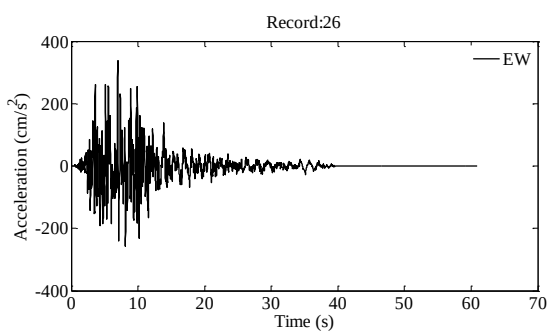
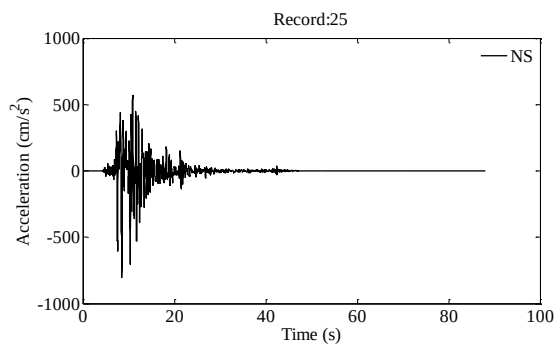
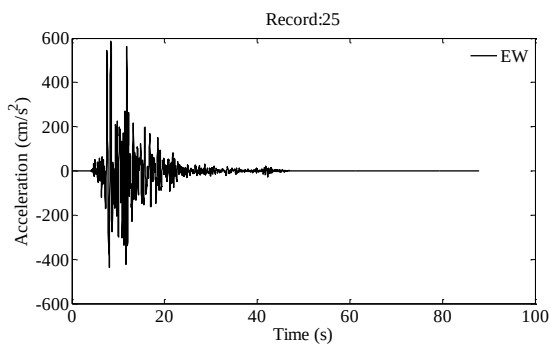


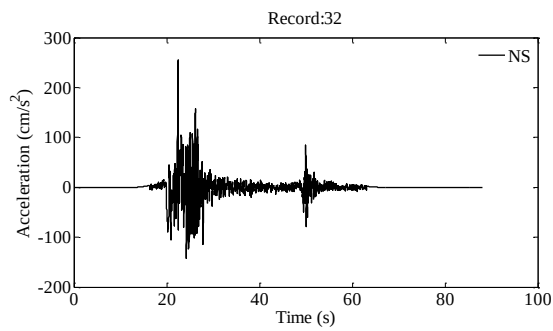
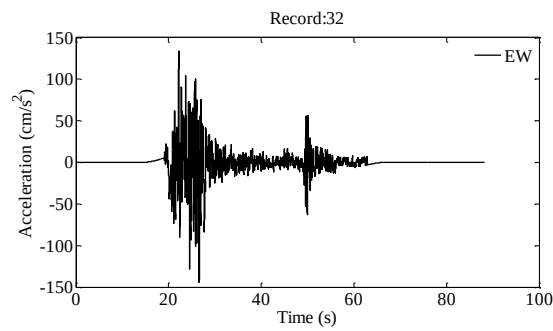
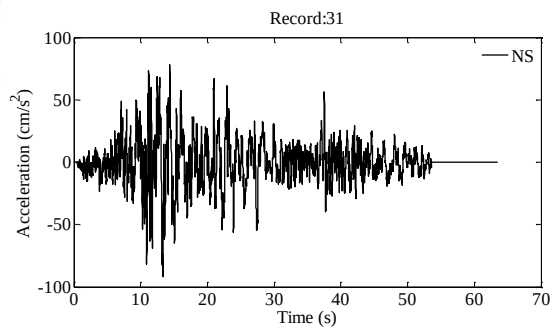
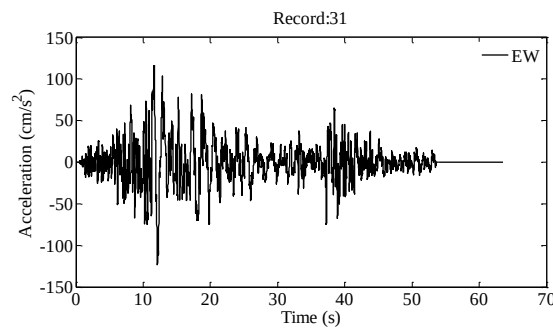
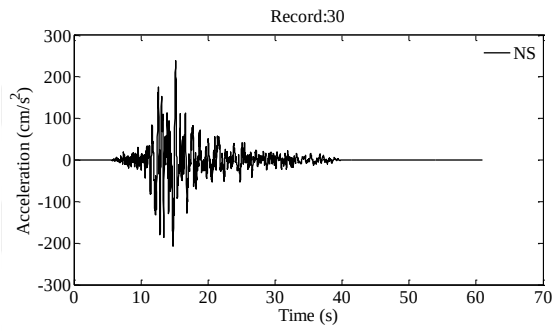
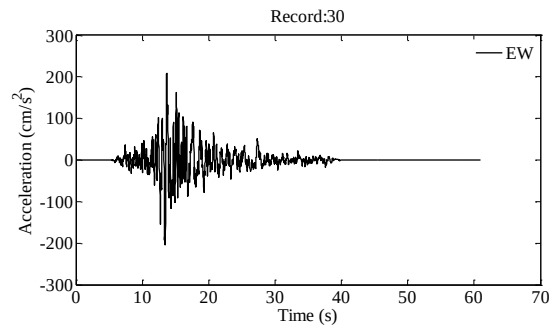
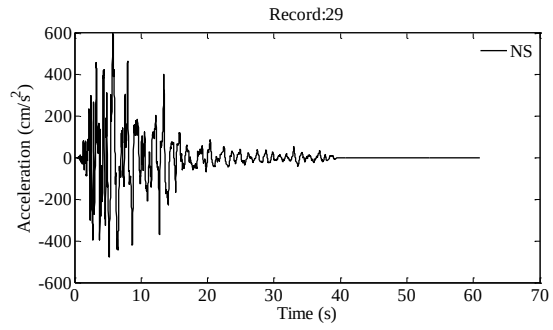
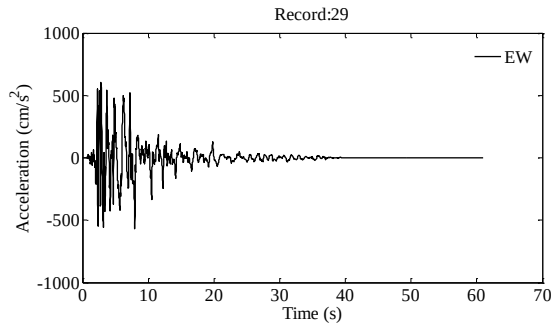


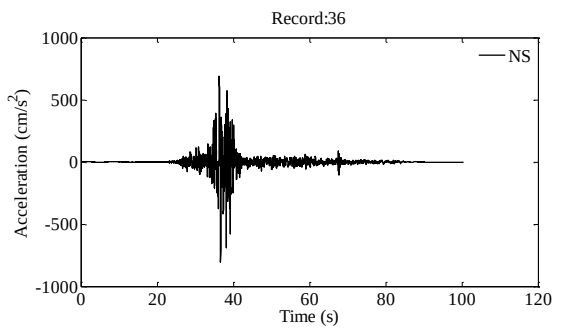
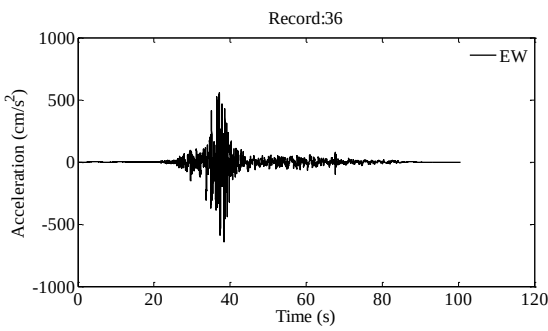
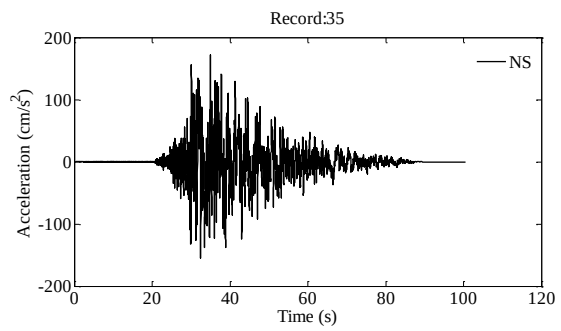
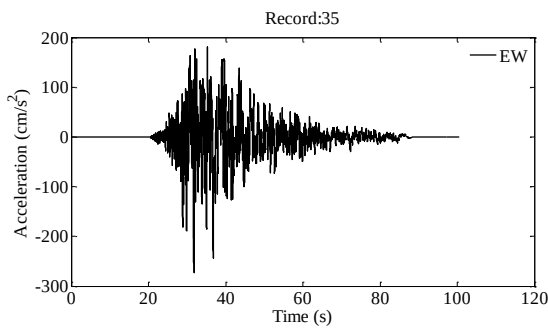
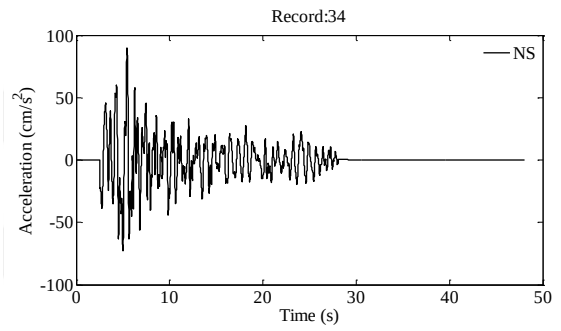
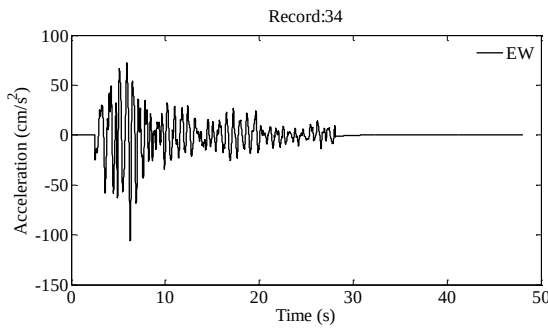
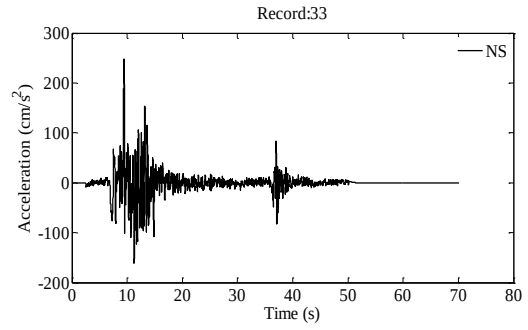
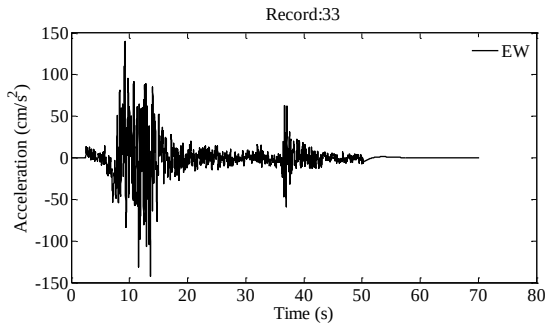


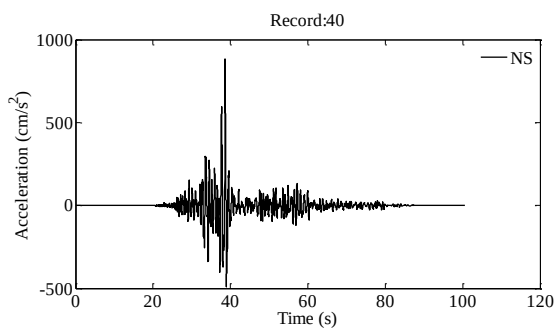
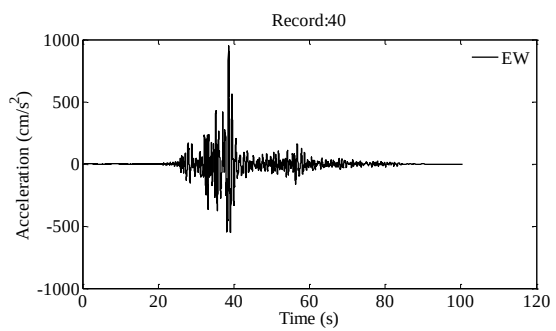
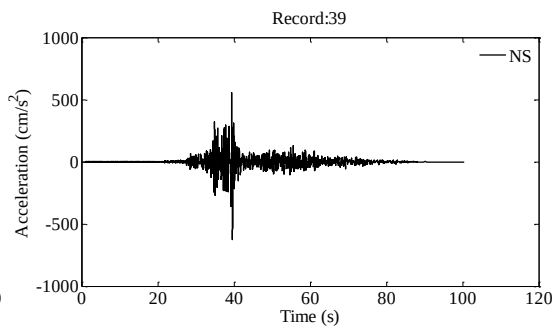
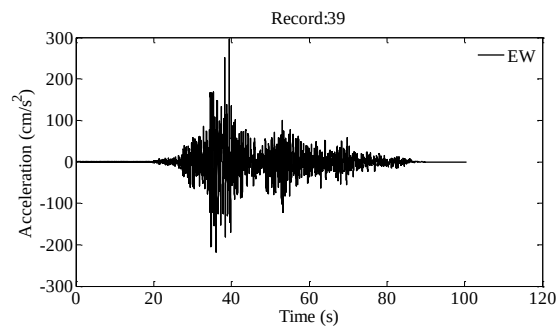
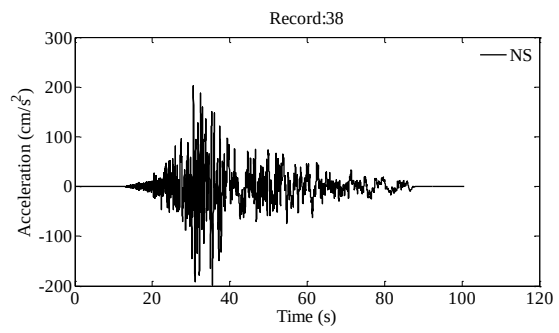
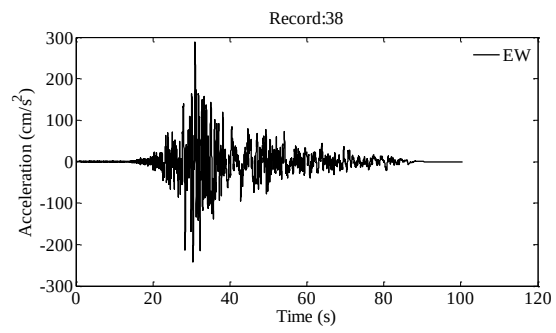
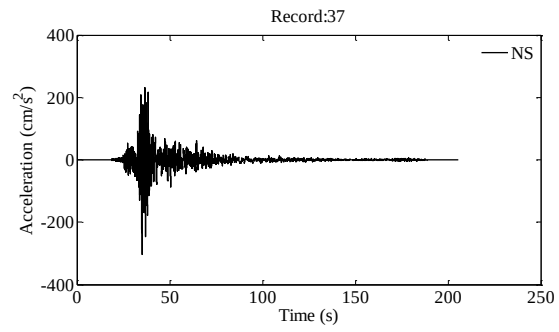
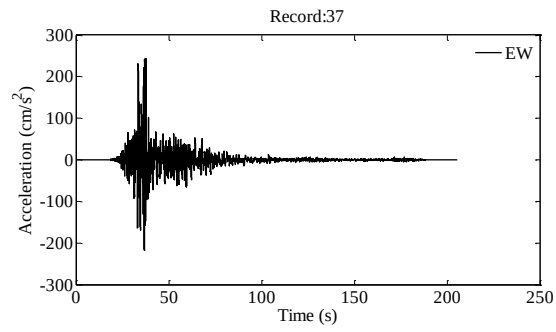








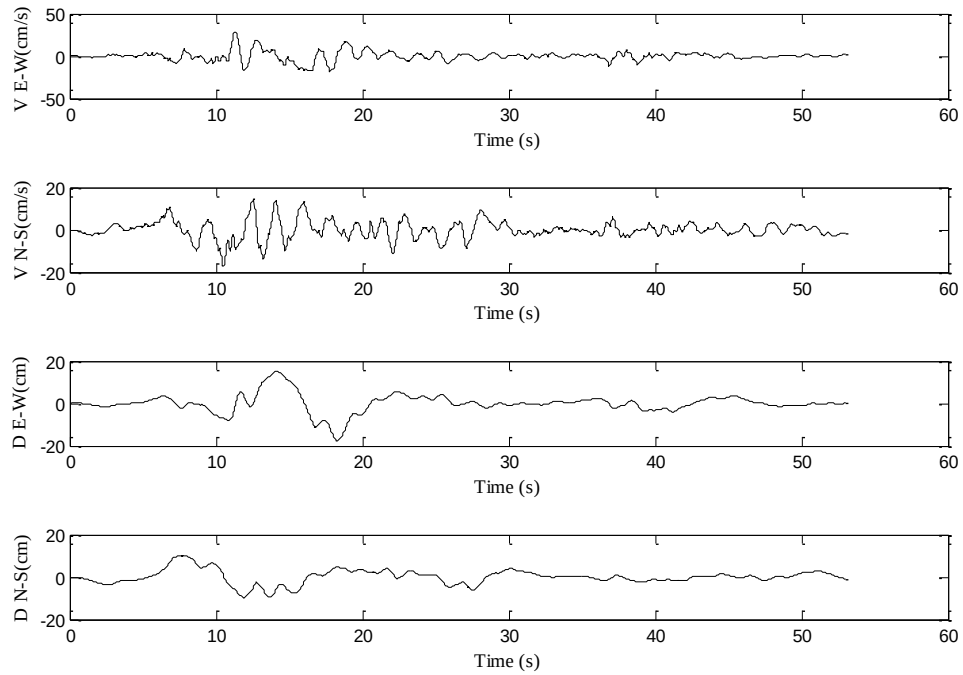




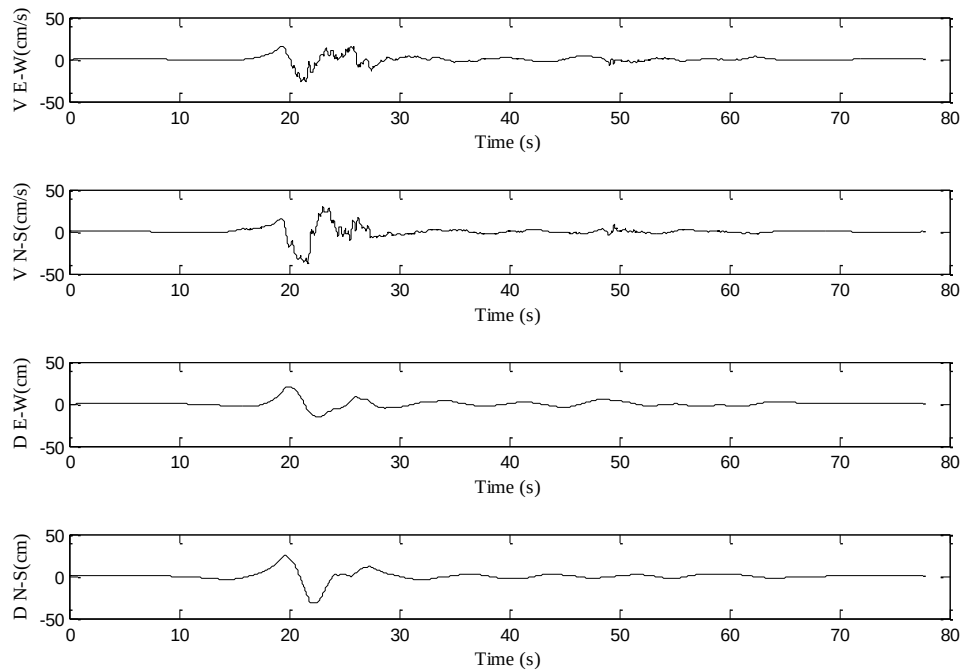
Appendix B

Velocity and Displacement Time Histories

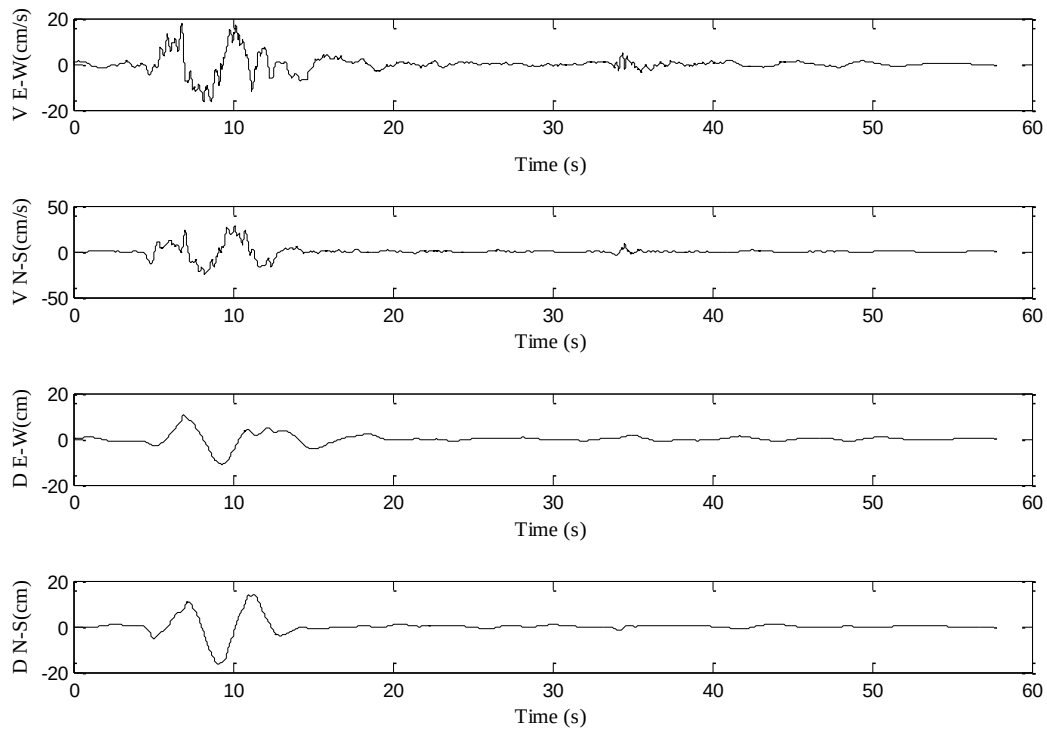
Record #31



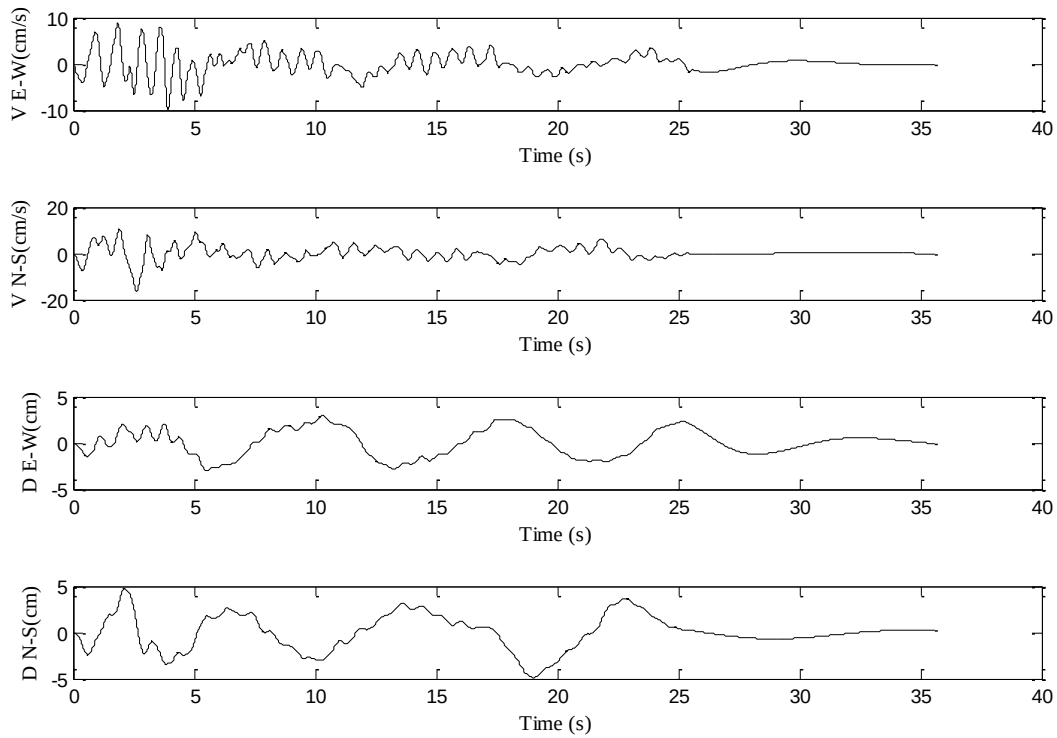
Record #32



Record #33

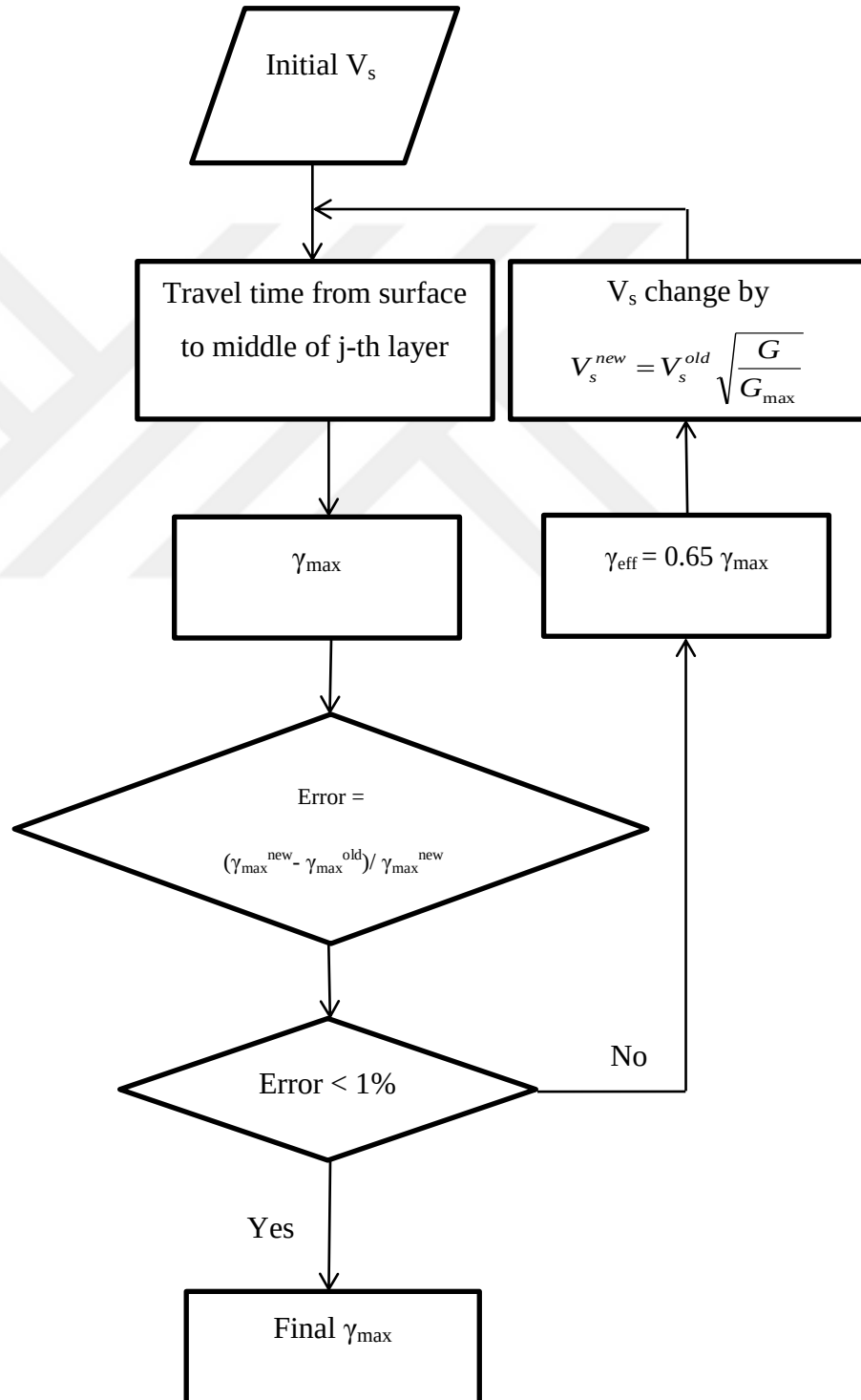


Record #34



Appendix C

Procedure of evaluating $\gamma_{\max}$ in layered medium





CURRICULUM VITAE

PERSONAL INFORMATION

Surname, Name: Dehghanian, Kaveh

Nationality: Iranian

Date and Place of Birth: 27 May 1982, Tabriz

Marital Status: Married

Phone: +90 5368991331

email: e175691@metu.edu.tr, kaveh_dehghanian@yahoo.com

EDUCATION

Degree	Institution	Year of Graduation
Ph.D.	Middle East Technical University	2016
MS	Urmia University (Iran), Geotechnical and Geoenvironmental Engineering	2008
BS	Tabriz Azad University (Iran), Civil Engineering	2004
High School	Saadi High School, Tabriz	2000

WORK EXPERIENCE

Year	Place	Enrollment
2008-2009	Koohsar- Schusse joint venture company	Engineer

FOREIGN LANGUAGES

English (Fluent)

Azeri (Mother tongue)

Farsi (Fluent)

Turkish (Fluent)

German (Basic)

PUBLICATIONS

1. Dehghanian K, Yilmaz M.T., “A Comparison Between Maximum Shear Strain Distribution in a Uniformly Elastic Half Layer and that in a Layered Viscoelastic Medium Due to the Same Free Field Motion”, 12th International Congress on Advances in Civil Engineering, Istanbul, Turkey (Chapter 5).
2. Yilmaz M.T. Dehghanian K, Zehtab K., “Developing 1-D field response using seismic and geotechnical data for Adapazari”. 1st Earthquake and Seismology Conference in Turkey 11-14 October 2011, Ankara, Turkey.
3. Badv K, Dehghanian K., “Comparing and Equivalency Assessment of Volatile Conduction in Composite Liners of Municipal Modern Landfills”. 8th International Congress on Civil Engineering, Shiraz University, Shiraz, Iran, May 11-13, 2009.

HOBBIES

Travelling, Learning Languages, Reading Books