

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**DYNAMIC SOIL AMPLIFICATION IN
SATURATED SOIL PROFILES USING
EFFECTIVE STRESS BASED NONLINEAR
METHODS**

by
Pınar AKDOĞAN

August, 2019
İZMİR

DYNAMIC SOIL AMPLIFICATION IN SATURATED SOIL PROFILES USING EFFECTIVE STRESS BASED NONLINEAR METHODS

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**by
Pınar AKDOĞAN**

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M.Sc THESIS EXAMINATION RESULT FORM

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ABSTRACT

Seismic waves from bedrock during their propagation are affected by the properties of the layer through which they passed; there may be changes in time, frequency and amplitude. Seismic waves is affected by local condition such as bedrock depth, thickness of soil layer, dynamic properties, lateral discontinuity of the soil and topographical properties. In this study, how to the effects of local condition on soil response analysis were investigated. One-dimensional (1D) nonlinear site response analyses were performed using DEEPSOIL software in an area located on the coast of the Izmir Bay where of deep alluvial soils govern the local geology. The soil profile is predominantly composed of thick clay layers and occasional sand-gravel layers in the study area. Nonlinear analyses were performed with the acceleration-time records that are taken from the PEER Ground Motion Database. In this study, two different bedrock levels are studied during dynamic site response analyses. The free field design spectra obtained by bedrock level depth of 240m with nonlinear methods were compared with those obtained by bedrock level depth of 1000m using effective and total stress based nonlinear methods. Also specific site response analyses results were compared code-based design. Results of the study showed that model depth and effective stress analysis affect site response analysis significantly and also obtained response spectra according to code-based was more conservative than according to the specific site response analysis. The other important result of the specific site response analysis is extension of the period.

Keywords: Nonlinear site response analysis, response spectra, non-linear analysis in time domain, strong ground motion, soil amplification, massing rules, dynamic shear modulus, dynamic soil properties, bedrock level

SUYA DOYGUN ZEMİN PROFİLLERİNDE DOĞRUSAL OLMAYAN EFEKTİF GERİLME ANALİZLERİ YOLUYLA DİNAMİK ZEMİN BÜYÜTMESİ

ÖZ

Ana kayadan gelen sismik dalgalar, içinden geçtikleri tabakaların özelliklerinden etkilenirler ve bu durum deprem dalgasının zaman, frekans ve genliğinde değişikliklere sebep olur. Deprem dalgaları; ana kaya derinliği, zemin tipi, tabakaların kalınlığı, dinamik özellikleri, zeminin yanal süreksizliği ve topografik özellikleri gibi yerel koşullardan etkilenir. Bu çalışma özelinde, yerel koşulların zemin tepki analizi sonuçlarını ne derecede etkilediği araştırılmıştır. İzmir Körfezi'nin kıyısında yer alan suya doygun derin alüvyonlu zeminlerin DEEPSOIL yazılımı kullanılarak tek boyutlu (1D) doğrusal olmayan saha tepki analizleri gerçekleştirilmiştir. Zemin profili ağırlıklı olarak kalın kil tabakalarından ve yer yer kum-çakıl karışımından meydana gelmektedir. Doğrusal olmayan zemin tepki analizi, PEER Yer Hareketi Veri Tabanından alınan ivme-zaman kayıtları ile yapılmıştır. Bu çalışma içeriğinde ana kaya derinliği 240 m ve 1000 m olan iki farklı zemin modeli için doğrusal olmayan yöntemlere göre efektif ve toplam gerilme analizleri kullanılarak sahaya özel tasarım tepki spektrumları elde edilmiştir. Elde edilen sahaya özel tasarım tepki spektrumları, yönetmelik bazlı elde edilen tepki spektrumları karşılaştırılmıştır. Ayrıca efektif ile toplam gerilme analizlerinin sonuçları karşılaştırılmış, analiz tipinin tasarım tepki spektrumlarına etkisi araştırılmıştır. Çalışma sonuçları değerlendirildiğinde, model derinliğinin ve efektif gerilme analizinin saha tepki analizlerini önemli derecede etkilediği ve yönetmelik bazlı elde edilen tasarım tepki spektrumlarının, sahaya özel tasarım tepki spektrumlarına göre peak değerlerin konservatif olduğu görülmüştür. Diğer önemli sonuç ise sahaya özel tasarım spektrumlarında periyot uzamasının görülmesidir.

Anahtar Kelimeler: Doğrusal olmayan saha tepki analizi, tepki spektrumu, zaman tanım alanında doğrusal olmayan analiz, kuvvetli yer hareketi, zemin büyütmeye, massing kuralları, dinamik kayama modülü, dinamik zemin özellikleri, ana kaya derinliği

CONTENTS

	Page
M.SC. THESIS EXAMINATION RESULT FORM.....	ii
ACKNOWLEDGEMENTS	iii
ABSTRACT.....	iv
ÖZ	vi
LIST OF FIGURES	ix
LIST OF TABLES	xv
CHAPTER ONE - INTRODUCTION	1
CHAPTER TWO - NONLINEAR SITE RESPONSE ANALYSIS	3
2.1 Introduction	3
2.2 1D Nonlinear Site Response Analysis.....	3
2.3 Time Domain Solution	6
2.4 Cyclic Nonlinear Soil Model.....	8
2.5 Viscous and Hysteretic Damping	13
2.6 Viscous (Small-Strain) Damping	14
2.7 Hysteretic Damping.....	15
2.8 Total Stress and Effective Stress Analysis	17
2.9 Dynamic Soil Models and Parameters	19
2.10 Ishibashi and Zhang Formulation for Non-Plastic Soils	21
2.11 Dynamic Soil Parameters for Plastic Soils.....	23
2.12 Dynamic Soil Parameters for Rock	24
2.13 Porewater Pressure Generation Models and Parameters	25
2.14 Definition and Selecting of Strong Ground Motion.....	27
2.14.1 Amplitude Parameters	28
2.14.2 Frequency Content Parameters	30

2.14.3 Duration of Motion	32
2.15 Selection of Strong Ground Motion from PEER	33
CHAPTER THREE - STUDIES OF SITE RESPONSE ANALYSES.....	36
3.1 Introduction	36
3.2 Investigation of Site Seismicity.....	37
3.3 Design Response Spectra Based on TBDY	41
3.4 Selection and Scaling of Earthquake Record	46
3.5 Location of the Study Area	57
3.6 Soil Profile of the Project Site	58
3.7 Standard Penetration Test (SPT) Results	58
3.8 Laboratory Test Results.....	62
3.9 Geophysical Test Results	66
3.10 Seismic Site Classification	74
3.11 Definition of Soil Properties.....	75
3.12 Idealized Soil Profile	78
3.13 Soil-Bedrock Model	80
3.14 Modulus Reduction and Damping Ratio for Soil.....	81
3.14.1 Modulus Reduction and Damping Ratio for Granular Layers.....	82
3.14.2 Modulus Reduction and Damping Ratio for Clay	83
3.14.3 Modulus Reduction and Damping Ratio for Rock	84
3.15 Maximum Layer Thickness.....	86
3.16 Porewater Pressure Parameters of Soil.....	87
CHAPTER FOUR - RESULTS AND DISCUSSIONS	91
4.1 Introduction	91
4.2 SiteResponse Analysis Results.....	91
4.3 Total Stress Analysis Results	92
4.3.1 Total Stress Analysis Results for DD-1 Earthquake Level.....	92
4.3.2 Discussion of Total Stress Analysis Results for DD-1 Earthquake Level.	97
4.3.3 Total Stress Analysis Results for DD-2 Earthquake Level.....	98
4.3.4 Discussion of Total Stress Analysis Results for DD-2 Earthquake Level	102

4.4 Effective Stress Analysis Results	103
4.4.1 Effective Stress Analysis Results for DD-1 Earthquake Level	118
4.4.2 Pore Water Pressure Generation Near Ground Surface for DD-1	108
4.4.3 Discussion of Effective Stress Analysis Results for DD-1 Earthquake Level	110
4.4.4 Effective Stress Analysis Results for DD-2 Earthquake Level	111
4.4.5 Pore Water Pressure Generation Near Ground Surface for DD-2	116
4.4.6 Discussion of Effective Stress Analysis Results for DD-2 Earthquake Level	118
4.5 Discussions	119
4.6 Comparison of Code-Based Design and Specific Site Response Analyses Results	125
CHAPTER FIVE - CONCLUSIONS AND RECOMMENDATIONS	129
REFERENCES	130
APPENDICES	136

LIST OF FIGURES

	Page
Figure 2.1 Methods for site response analysis	4
Figure 2.2 Normalized and mean response spectrum using equivalent linear and nonlinear analysis	5
Figure 2.3 Comparison of linear, equivalent linear and nonlinear analysis.....	6
Figure 2.4 Idealized soil stratigraphy (a) layered soil column and (b) multi-degree-of-freedom lumped parameter idealization, used for time domain solution...	8
Figure 2.5 Skeleton curve defined by stress-strain curve	9
Figure 2.6 Ramberg-Osgood model	9
Figure 2.7 Hyperbolic model skeleton curve	11
Figure 2.8 Normalized modulus reduction curve from MKZ, KZ and measured valu.	11
Figure 2.9 Hyperbolic soil model and extended masing rule.....	13
Figure 2.10 Comparison of different damping models for a) 100 m soil column b) 1000 m soil column.....	15
Figure 2.11 Damping curve comparison based on masing rule and proposed method	16
Figure 2.12 Evaluation of proposed damping reduction factor: (a) modulus reduction; (b) using damping curve using Darendeli's sand curves as target	17
Figure 2.13 Example shear modulus and damping ratio curve.....	18
Figure 2.14 Comparison of spectral velocity according to total stress and effective stress analysis	19
Figure 2.15 Relations between G/G_{max} versus γ_c and Soil Plasticity	21
Figure 2.16 Relations between λ versus γ_c and Soil Plasticity	21
Figure 2.17 Influence of effective confining pressure on modulus reduction versus cyclic shear strain curves: (a) Non-plastic soil ($PI = 0$) and (b) Plastic soil ($PI=50$)	24
Figure 2.18 EPRI modulus reduction and damping ratio curves for rock	25
Figure 2.19 Kocaeli Earthquake, time acceleration record taken from DEEPSOIL software earthquake motion database.....	28

Figure 2.20 Kocaeli Earthquake, time velocity record taken from DEEPSOIL software earthquake motion database	29
Figure 2.21 Kocaeli Earthquake, time displacement record taken from DEEPSOIL software earthquake motion database	29
Figure 2.22 Fourier amplitude spectrum of Kocaeli Earthquake which is taken from earthquake motion database of DEEPSOIL software	31
Figure 2.23 Narrow and wide fourier spectrum	31
Figure 2.24 Response spectra (Spectral Acceleration) of Kocaeli Earthquake which is taken from earthquake motion database of DEEPSOIL software.....	32
Figure 2.25 The bracketed duration of Kocaeli Earthquake, time acceleration record taken from DEEPSOIL software earthquake motion database	33
Figure 3.1 Site response analysis flow chart.....	37
Figure 3.2 Major tectonic plates structures of Türkiye	38
Figure 3.3 Active fault map of Eastern Izmir Bay of Turkey	39
Figure 3.4 Active fault map of Urla of Turkey	39
Figure 3.5 Earthquakes with 5 to 10 Mw occurred in Izmir and vicinity	40
Figure 3.6 Turkey earthquake hazard maps interactive web application.....	42
Figure 3.7 Horizontal elastic design response spectrum for DD-1 earthquake level and ZD soil class.....	45
Figure 3.8 Horizontal elastic design response spectrum for DD-2 earthquake level and ZD soil class.....	45
Figure 3.9 Unscaled acceleration time graph of Coyote Lake Earthquake.....	47
Figure 3.10 Unscaled acceleration time graph of Norcia Italy Earthquake	47
Figure 3.11 Unscaled acceleration time graph of Irpinia Italy-01(Cascia station) Earthquake.....	48
Figure 3.12 Unscaled acceleration time graph of Irpinia Italy-02 (Brienza Station) Earthquake.....	48
Figure 3.13 Unscaled acceleration time graph of Morgan Hill Earthquake	49
Figure 3.14 Unscaled acceleration time graph of Kobe Japan Earthquake.....	49
Figure 3.15 Unscaled acceleration time graph of Duzce Earthquake	50
Figure 3.16 Unscaled acceleration time graph of Hector Mine Earthquake	50
Figure 3.17 Unscaled acceleration time graph of Tottori Japan Earthquake	51

Figure 3.18 Unscaled acceleration time graph of Parkfield Earthquake.....	51
Figure 3.19 Unscaled acceleration time graph of L'Aquila Earthquake.....	52
Figure 3.20 Scaled arithmetic mean PSA based on Target PSA of DD-1 earthquake level (Log Scale)	55
Figure 3.21 Scaled arithmetic mean PSA based on Target PSA of DD-2 earthquake level (Log Scale)	56
Figure 3.22 Area of investigation location.....	58
Figure 3.23 (Uncorrected) N-Depth relationship	61
Figure 3.24 $N_{60} / N_{1,60}$ -Depth relationship	61
Figure 3.25 MASW (multichannel analysis of surface waves) methods	67
Figure 3.26 average shear wave velocity with depth from MASW methods	68
Figure 3.27 Measurement point of downhole methods.....	69
Figure 3.28 Original shear wave velocity with depth from downhole methods	69
Figure 3.29 Corrected shear wave velocity with depth from Downhole Methods and velocity versus depth correlation graphs.....	70
Figure 3.30 Measurement points of Spatial Auto-Correlation (SPAC) Method.....	71
Figure 3.31 Shear wave velocity with depth from Spatial Auto-Correlation (SPAC) Method	71
Figure 3.32 Design shear wave velocity and results of geophysical methods vs. depth	73
Figure 3.33 Comparison of the shear wave velocity values obtained by field tests and empirical relations.	74
Figure 3.34 Idealized soil profile	79
Figure 3.35 Soil-Engineering bedrock model (Model-1).....	80
Figure 3.36 Soil-Seismic bedrock model (Model-2).....	81
Figure 3.37 Modulus reduction curves of granular layers	82
Figure 3.38 Damping curve of granular layers	83
Figure 3.39 Modulus reduction curve of clay layers.....	83
Figure 3.40 Damping curve of clay layers	84
Figure 3.41 Modulus reduction curves of rock layers based on EPRI.....	85
Figure 3.42 Damping ratio curves of rock layers based on EPRI.....	86
Figure 3.43 Hydraulic conductivity of gravelly clays for different gravel percents..	88

Figure 4.1 Max. strain and max. displacement for Model-1 with DD-1 earthquake level	93
Figure 4.2 Max. strain and max. displacement for Model-2 with DD-1 earthquake level	93
Figure 4.3 PGA value versus depth for a) Model-1 and b) Model-2 with DD-1	94
Figure 4.4 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S1-1, b) EQ-S1-2, c) EQ-S1-3, d) EQ-S1-4, e) EQ-S1-5, f) EQ-S1-6, g) EQ-S1-7, h) EQ-S1-8	95
Figure 4.5 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for j) EQ-S1-9, k) EQ-S1-10, l) EQ-S1-11.	96
Figure 4.6 Average PSA curve for Model-1 (depth of 240m) with DD-1 earthquake level	97
Figure 4.7 Average PSA curve for Model-2 (depth of 1000m) with DD-1 earthquake level	97
Figure 4.8 Comparison of Model-1 and Model-2 for DD-1 earthquake level	98
Figure 4.9 Relative difference of between Model-1 and Model-2 for DD-1 earthquake level	98
Figure 4.10 Max. strain and max. displacement for Model-1 with DD-2 earthquake level	99
Figure 4.11 Max. strain and max. displacement for Model-2 with DD-2 earthquake level	99
Figure 4.12 PGA value versus depth for a) Model-1 and b) Model-2 with DD-1 ...	100
Figure 4.13 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S2-1, b) EQ-S2-2, c) EQ-S2-3, d) EQ-S2-4	100
Figure 4.14 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for e) EQ-S2-5, f) EQ-S2-6, g) EQ-S2-7, h) EQ-S2-8, j) EQ-S2-9, k) EQ-S2-10, l) EQ-S2-11	101

Figure 4.15 Average PSA curve for Model-1 (depth of 240m) with DD-2 earthquake level	102
Figure 4.16 Average PSA curve for Model-2 (depth of 1000m) with DD-2 earthquake level	102
Figure 4.17 Comparison of Model-1 and Model-2 for DD-2 earthquake level	103
Figure 4.18 Relative difference of between Model-1 and Model-2 for DD-2 earthquake level	103
Figure 4.19 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-1 with DD-1 earthquake level.....	104
Figure 4.20 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-2 with DD-1 earthquake level.....	105
Figure 4.21 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S1-1, b) EQ-S1-2, c) EQ-S1-3, d) EQ-S1-4, e) EQ-S1-5, f) EQ-S1-6	106
Figure 4.22 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for g) EQ-S1-7, h) EQ-S1-8, j) EQ-S1-9, k) EQ-S1-10, l) EQ-S1-11 .	107
Figure 4.23 Average PSA curve for Model-1 (depth of 240m) with DD-1 earthquake level	108
Figure 4.24 Average PSA curve for Model-2 (depth of 1000m) with DD-1 earthquake level	108
Figure 4.25 Pore water pressure generation near the ground surface for Model-1 (240m)	109
Figure 4.26 Pore water pressure generation near the ground surface for Model-2 (1000m)	110
Figure 4.27 Comparison of Model-1 and Model-2 for DD-1 earthquake level	111
Figure 4.28 Relative difference of between Model-1 and Model-2 for DD-1 earthquake level	111
Figure 4.29 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-1 with DD-2 earthquake level.....	112

Figure 4.30 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-2 with DD-2 earthquake level.....	113
Figure 4.31 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S2-1, b) EQ-S2-2, c) EQ-S2-3, d) EQ-S2-4, e) EQ-S2-5, f) EQ-S2-6	114
Figure 4.32 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for g) EQ-S2-7, h) EQ-S2-8, j) EQ-S2-9, k) EQ-S2-10, l) EQ-S2-11 .	115
Figure 4.33 Average PSA curve for Model-1 (depth of 240m) with DD-2 earthquake level	116
Figure 4.34 Average PSA curve for Model-2 (depth of 1000m) with DD-2 earthquake level	116
Figure 4.35 Pore water pressure generation near the ground surface for Model-1 (240m), DD-2	117
Figure 4.36 Pore water pressure generation near the ground surface for Model-2 (1000m), DD-2	118
Figure 4.37 Comparison of Model-1 and Model-2 for DD-2 earthquake level	119
Figure 4.38 Relative difference of between Model-1 and Model-2 for DD-2 earthquake level	119
Figure 4.39 Comparison of effective and total stress analysis for Model-1 (240m) with a) DD-1 and b) DD-2 earthquake level	120
Figure 4.40 Relative difference effective and total stress analysis for Model-1 (240m) with a) DD-1 and b) DD-2 earthquake level	120
Figure 4.41 Comparison of effective and total stress analysis for Model-2 (1000m) with a) DD-1 and b) DD-2 earthquake level	121
Figure 4.42 Relative difference effective and total stress analysis for Model-2 (1000m) with a) DD-1 and b) DD-2 earthquake level	121
Figure 4.43 Effective stress analysis results DD-1 and DD-2 earthquake level	122
Figure 4.44 The variation of PGA value with depth using effective stress analysis for Model-2 and DD-1 earthquake level	122

Figure 4.45 Comparison of Model-1 and Model-2 for DD-1 earthquake level using effective stress analysis	124
Figure 4.46 Relative difference of between Model-1 and Model-2 for DD-1 earthquake level using effective stress analysis (+ difference means Model-1 results in higher acceleration and vice versa.)	124
Figure 4.47 Comparison of Model-1 and Model - 2 for DD - 2 earthquake level using effective stress analysis	125
Figure 4.48 Relative difference of between Model-1 and Model-2 for DD-2 earthquake level using effective stress analysis (+ difference means Model-1 results in higher acceleration and vice versa.)	125
Figure 4.49 The comparison of the obtained design response spectra based on DBYBHY and specific site response analyses.....	127
Figure 4.50 The comparison of design response spectra results for DD-1 earthquake level at the surface	127
Figure 4.51 The comparison of design response spectra results for DD-2 earthquake level at the surface	128

LIST OF TABLES

	Page
Table 2.1 Effects of different soil and loading properties on maximum shear modulus, modulus ratio and damping ratio	20
Table 2.2 Amplitude parameters of Kocaeli Earthquake which taken from DEEPSOIL software earthquake motion database	29
Table 3.1 Active fault properties nearby Izmir Region.....	41
Table 3.2 Local soil condition factors for short period accelerations	43
Table 3.3 Local soil condition factors for 1 sec period spectral accelerations	43
Table 3.4 Selected earthquake records and scale factors for soil site class, ZB and earthquake ground motion level, DD-1.....	53
Table 3.5 Selected earthquake records and scale factors for soil site class, ZB and earthquake ground motion level, DD-2.....	54
Table 3.6 The amplitude parameter, frequency content parameters and duration of selected earthquake motions	57
Table 3.7 Borehole properties	59
Table 3.8 Standard Penetration Test Result (N) and Corrected SPT-N60 and N1,60 values.....	59
Table 3.9 Sieve analysis test results, Atterberg consistency limit test results and soil classification	62
Table 3.10 Unconfined compression test result	64
Table 3.11 Triaxial compression test result	65
Table 3.12 Consolidation test result.....	65
Table 3.13 Coefficient of consolidation and description of over consolidation ratio	66
Table 3.14 Idealized shear wave velocity from field test.....	72
Table 3.15 Empirical shear wave velocity	73
Table 3.16 Soil class from TBDY	74
Table 3.17 Physical Properties of Soil Layers	77
Table 3.18 Estimated values of friction angle, ϕ from SPT-N value	77
Table 3.19 Pressures acting on soil layers	78

Table 3.20 Input parameters for Ishibashi-Zhang method	82
Table 3.21 Shear modulus reduction and damping ratio for rock depth of 150-300 m	84
Table 3.22 Shear modulus reduction and damping ratio for rock depth of 300-600 m	85
Table 3.23 Shear modulus reduction and damping ratio for rock depth of 600-1500m	85
Table 3.24 Maximum layer thicknesses of soil layers for 30 Hz cut-off frequency..	87
Table 3.25 Determining hydraulic conductivity and coefficient of consolidation.....	89
Table 3.26 The selected coefficient of consolidation and soil parameters	89
Table 3.27 The excess pore pressure parameters for sand layers	90
Table 3.28 The excess pore pressure parameters for clay layers	90
Table 4.1 General characteristics of Model-1 and Model-2	91
Table 4.2 Abbreviations assigned to the earthquakes	92
Table 4.3 The obtained local soil effect coefficients for the ZF soil class with deep alluvial deposits in the Izmir Bay.....	128

CHAPTER ONE

INTRODUCTION

During an earthquake, seismic waves propagate from the earthquake source to surface. As they travel through the soil profile, both the frequency content and the amplitude is altered and the ground motion at the surface is no longer the same engineering bedrock. The intensity and duration of these earthquake motions depend on the earthquake magnitude, location, and characteristics of the site.

It is well known that earthquake waves have three major characteristics that deserve attention of the engineers which are amplitude, frequency content and duration. Soil layers, while amplifying the amplitude of earthquake motions in some frequencies, attenuate the others. Since soil conditions may change considerably in short distances, the ground motion may also change accordingly. Variables of the site soil characteristics that affect the strong ground motion characteristics are depth of bedrock, thickness of soil layers and their dynamic soil properties. Site characteristics also influence the frequency of ground motion, therefore response spectrum produced by that motion.

Mostly, soil conditions or site effects amplify or deamplify the earthquake motions amplitude, which is called soil amplification. As widely known, softer soils tend to amplify low-frequency bedrock motions more than stiffer soils. In case of higher frequency bedrock motions, opposite occurs. Many researchers have studied about the site effects on ground motion, one of them is Aki (1993) researched the local soil effects on weak and strong ground motion and concluded that “the strong motion amplification factor is greater at soil sites than rock sites only for frequencies lower than 5 Hz and, on average, the relation is reversed for higher frequencies.”

In the scope of this study, one of the properties that affects the strong ground motion, depth of bedrock's influence on the response spectrum will be computed and compared using effective stress analysis based on nonlinear method. Besides that, the total stress analysis is used and their results are compared with effective stress analysis results. The other main purpose is that the obtained specific site response spectra is

compared with the obtained design spectra according to Turkish Building Code (TBDY-Türkiye Bina Deprem Yönetmeliği, 2018).

This study has been planned to be carried out at alluvial soil in nearby İzmir Bay. Before the starting to perform the analyses, geological and geophysical field test of the study area was investigated and idealized soil profile was created. Also, the nonlinear soil model was obtained using soil parameters with the obtained correlations according to the laboratory test results by some researchers (Konder & Zelasko, 1963; Masing, 1926; Matasovic, 1993).

The seismicity of the site was researched and the 11 acceleration-time history recordings were selected to be appropriate for the site from PEER Ground Motion Database. The analyses were performed for each of the 11 acceleration-time history recordings. This study investigates the behavior of ground motions with different frequency content on soft alluvial soils. Therefore, nonlinear method was chosen for analyses. Also, nonlinear site response analysis performed using DEEPSOIL software which is able to perform one-dimensional nonlinear site response analysis model with Hyperbolic soil model in deep soil deposits.

The used analyses methods, soil models, selecting acceleration-time recordings and researcher comments related to the subject were involved in the upcoming chapters. Respectively, in Chapter Two, dynamic site response analysis methods, theories of soil model and strong ground motion properties were described. In Chapter Three, the seismicity of studied site has been researched, the design response spectra were obtained according to TBDY (2018), selecting acceleration-time recordings of earthquake motion has been described and geological and geophysical soil investigations results, obtained idealize soil profile and parameters were given. In Chapter Four, results and comparison of analyses were given. Finally, in Chapter Five, conclusion and recommendations were given.

CHAPTER TWO

NONLINEAR SITE RESPONSE ANALYSIS

2.1 Introduction

Determination of ground response is one of the essential tasks in geotechnical engineering profession. Site response analyses are used in developing design spectra, liquefaction risk assessment and determining the earthquake forces acting on the engineering structures.

One, two- and three-dimensional ground response analyses methods were developed over years. Although some problems require 2D or 3D dynamic analyses, such approaches can only be performed by means of numerical methods that are, time consuming and therefore usually quite expensive. 1D ground response analyses were widely used in geotechnical engineering calculations. The analyses in this study are also performed in using 1D ground response analyses.

In this chapter, 1D nonlinear site response analysis, features of earthquake motions used in the analyses, soil and material models are described. Nonlinear site response analysis is to be carried out for soil class of ZF based on TBDY (Earthquake Code for Buildings, 2018). In addition, according to the code, such analyzes can also be carried out on other soil class depending on the project engineer preference based on TBDY (Earthquake Code for Buildings, 2018). In this study, nonlinear site response analysis was carried out on a soil profile with soil class ZF with earthquake level of DD-1 and DD-2.

2.2 1D Nonlinear Site Response Analysis

Flow chart demonstrating the available methods for site response analysis is given in Figure 2.1. In this study, 1D nonlinear ground response analyses were performed, other methods were shortly described for reference.

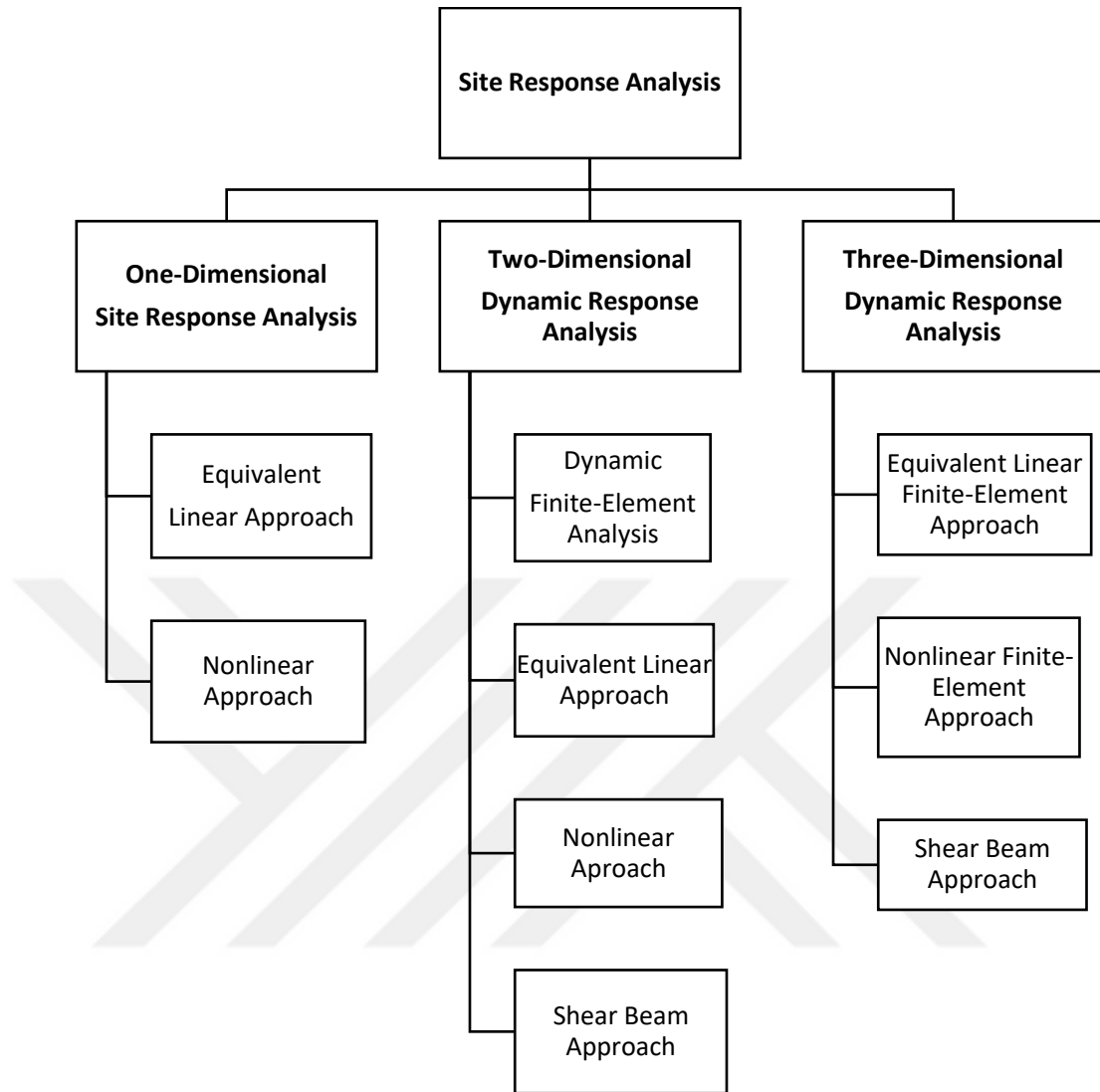


Figure 2.1 Methods for site response analysis

1D ground response analyses assume that all boundaries are infinitely long in horizontal direction and soil response is caused by vertical SH waves propagating from engineering bedrock.

Although linear and equivalent linear methods perform reasonably well with respect to ground response analyses, they are still approximate to predict nonlinear behavior of ground response. Equivalent linear and nonlinear ground response analyses were compared by many researchers. Some of the important and current studies in comparison of equivalent linear and nonlinear site response analyses can be listed as Pruiksma (2016), Carlton and Tokimatsu (2016) and Selcuk et al. (2007). According to the mentioned studies, equivalent linear or nonlinear methods should be

chosen based on the site and input ground motion properties. In cases where strains are low (stiff soil and/or weak bedrock motion) both ground response analysis methods perform reasonably well, however, in case of high strain problems nonlinear analyses perform better than equivalent linear analysis methods. This is mainly due to elastic behavior of soils at lower strains however, at high strain levels, the soils exhibit plastic behavior.

Carlton and Tokimatsu (2016) compared the equivalent linear and nonlinear ground response analyses. Their results indicate that as shear strains increase, equivalent linear analyses predict smaller maximum strains and damping ratios, bigger accelerations and velocities. Also, spectral accelerations predicted by equivalent linear methods are higher than nonlinear methods for periods less than 0.1s and for periods near natural site periods which are between 0.2s and 2.0s. For other ranges, equivalent methods result in similar spectral accelerations with nonlinear methods.

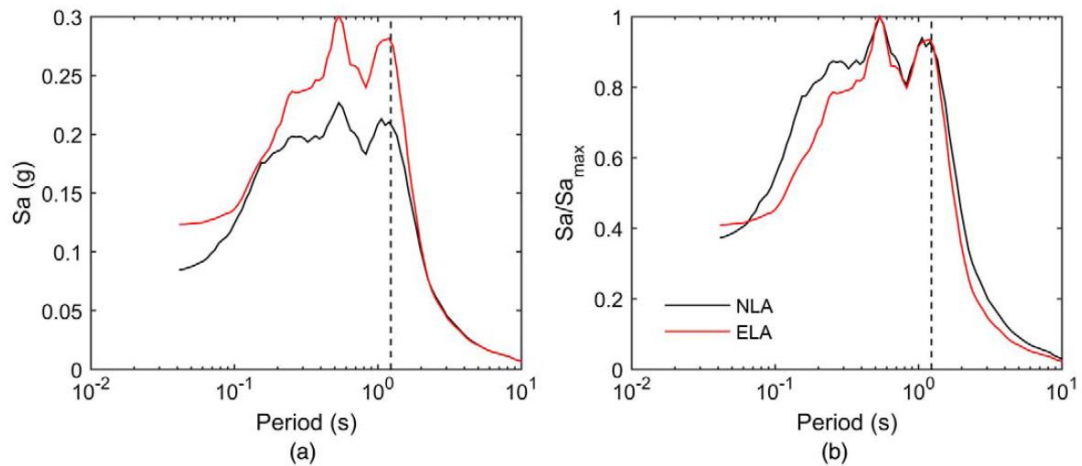


Figure 2.2 Normalized and mean response spectrum using equivalent linear and nonlinear analysis (Carlton & Tokimatsu, 2016)

Similar results were found by Pruiksma (2016) when comparing linear and nonlinear methods. It was specially emphasized on spectral period 0.01s range where difference between two methods increase significantly. Also, Pruiksma (2016) states that difference between two methods increase with increasing input acceleration. The difference reaches up to factor 2 for peak ground acceleration (PGA).

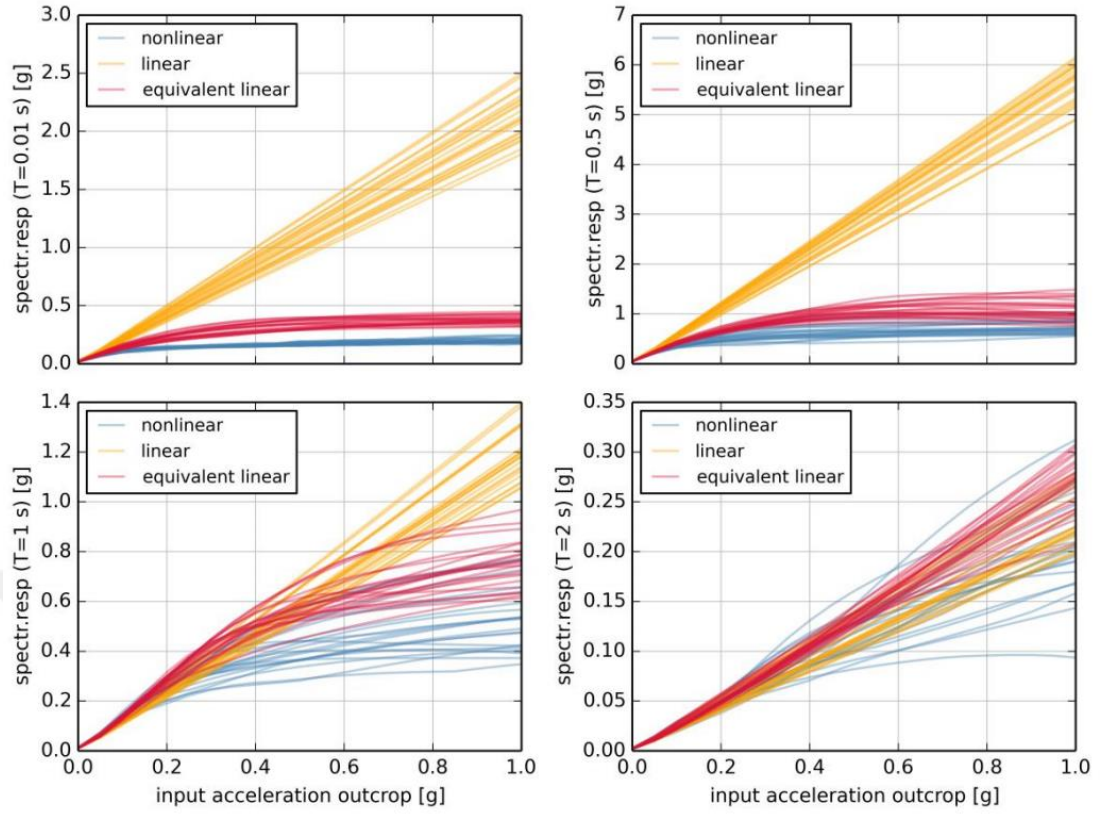


Figure 2.3 Comparison of linear, equivalent linear and nonlinear analysis (Pruiksma, 2016)

2.3 Time Domain Solution

In order to investigate the nonlinear behavior of the soil, direct numerical integration should be used in time domain. Small step integration of motion equation allows using any linear or nonlinear stress-strain models or constitutive models.

If the soil is subjected to a ground motion at the bedrock level, equation of motion of the soil can be expressed as in Equation 2.1. In this formula, ρ is the density, τ is shear stress, u is displacement and z is depth below the ground surface.

$$\frac{\partial \tau}{\partial z} = \rho \frac{\partial^2 u}{\partial t^2} = \rho \frac{\partial \dot{u}}{\partial t} \quad (2.1)$$

Soil behavior is approached as a Kelvin-Voight solid and the stress-shear strain relationship is expressed based on Kelvin-Voight or soil model as:

$$\tau = G\gamma + \eta \frac{\partial \gamma}{\partial t} \quad (2.2)$$

where $G = \text{shear modulus}$, $\gamma = \text{shear strain}$ and $\eta = \text{viscosity}$.

Then Eq. (2.2) into Eq. (2.1) results in:

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial z^2} + \eta \frac{\partial^3 u}{\partial z^2 \partial t} \quad (2.3)$$

The soil column is decoupled into individual layer using a multi-degree-of-freedom lumped parameter model or finite elements model shown in Figure 2.4 (Park & Hashash, 2004). Each individual layer i is represented by a spring, a corresponding mass, and a dashpot for viscous damping. In time domain analysis, Figure 2.4 presents lumped mass system for one-dimensional wave propagation. The wave propagation equation, Equation (2.3) is written as:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\{I\}\ddot{u}_g \quad (2.4)$$

Where $[M]$ = mass matrix; $[C]$ = viscous damping matrix; $[K]$ = stiffness matrix; $\{\ddot{u}\}$ = vector of nodal relative acceleration; $\{\dot{u}\}$ = vector of nodal relative velocities; and $\{u\}$ = vector of nodal relative displacement; $\ddot{u}_g$ is the acceleration at the base of the soil column and $\{I\}$ is the unit vector (Park & Hashash, 2004). Equation (2.4) is solved numerically at each time step using the Newmark β method (Newmark, 1959).

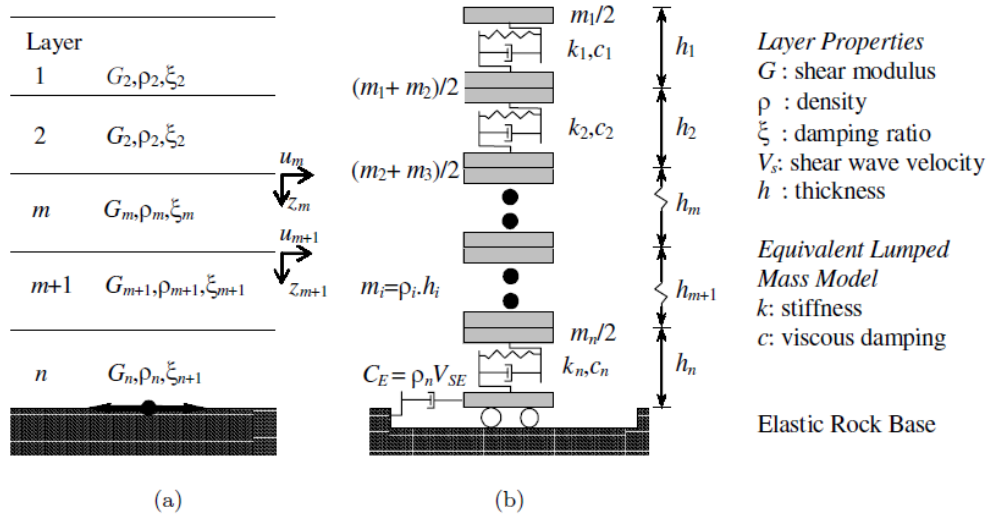


Figure 2.4 Idealized soil stratigraphy (a) layered soil column and (b) multi-degree-of-freedom lumped parameter idealization, used for time domain solution (Park & Hashash, 2004)

Energy input during earthquake is expressed in terms of shear waves, which results in terms shear strains. The stress-strain response of the soils in nonlinear range can be taken into consideration by means of hysteresis curves established centering around a nonlinear backbone curve.

2.4 Cyclic Nonlinear Soil Model

Nonlinear behavior of the soils can be modeled using cyclic nonlinear soil models that follow a path like real behavior. All cyclic nonlinear models have similar properties such as a backbone curve and numerical rules that implement loading-unloading behavior.

Nonlinear stress- strain model can be expressed using the following equation that define unload-reload behavior of the soil in cyclic loading (Figure 2.5).

$$\frac{\tau \pm \tau_a}{2} = f\left(\frac{\gamma - \gamma_a}{2}\right) \quad (2.5)$$

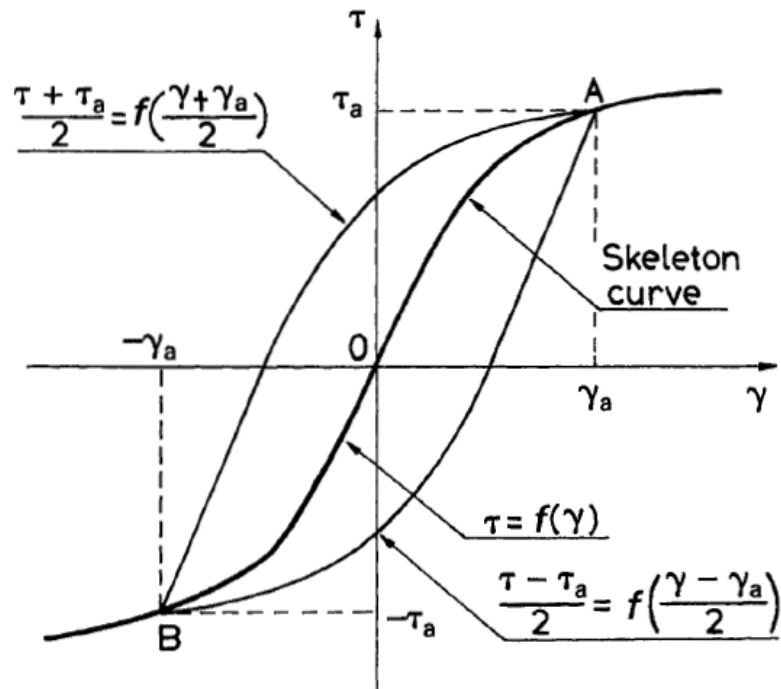


Figure 2.5 Skeleton curve defined by stress-strain curve (Ishihara, 1996)

Backbone curves can be set either as a function of shear strain and the maximum shear modulus to express variation of shear stress with strain or vice versa (Ishihara, 1996). Backbone curve of cyclic nonlinear soil models can be expressed using two approaches. These are named as “Ramberg-Osgood Model” and “Hyperbolic Model”. Shear modulus ratio curve of “Ramberg-Osgood Model” is given in Figure 2.6.

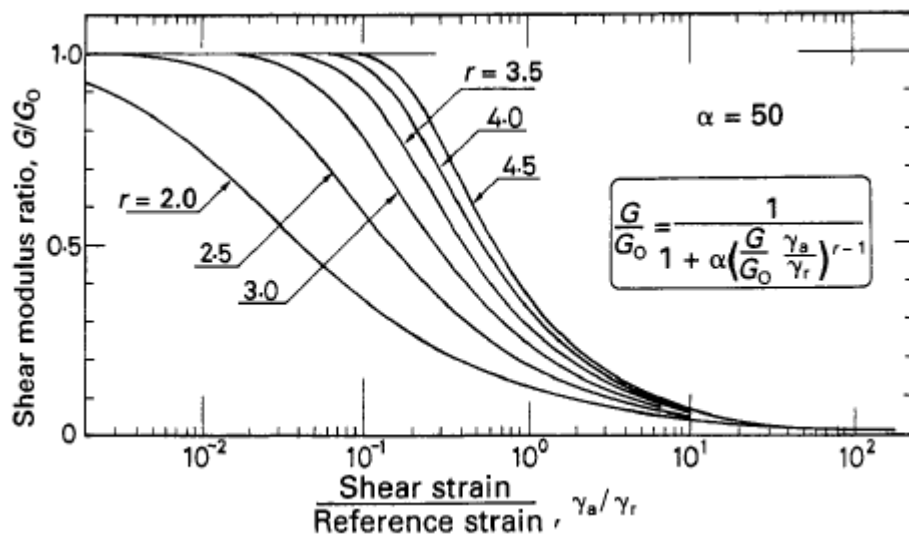


Figure 2.6 Ramberg-Osgood model (Ishihara, 1996)

Ramberg-Osgood model uses four parameters to define the backbone curve and equation is given below:

$$\frac{\gamma}{\gamma_y} = \frac{\tau}{\tau_y} \left[1 + a \left| \frac{\tau}{\tau_y} \right|^{r-1} \right] \quad (2.6)$$

Hyperbolic model is developed using the approach presented by Kondner and Zelasko (1963). The shape of the backbone curve is given Figure 2.7, it depends on the soil's maximum shear modulus at small strains and reference strain. Functional form of hyperbolic backbone curve can be expressed as below (Konder & Zelasko, 1963).

$$\tau = \frac{G_{max}\gamma}{1 + \gamma_a/\gamma_{ref}} \quad (2.7)$$

In this equation, reference strain equals to strain at $G/G_{max}=0.5$.

Matasovic (1993) has developed the hyperbolic backbone (skeleton) curve based on the Konder and Zelasko (1963) curve. Matasovic (1993) has developed two parameters which are β and s in order to represent the wide range of soil behavior and fit the backbone curve shape. (Eq. 2.8.) Matasovic (1993) presented comparison between MKZ (Modified Kodner and Zelasko model with β and s curve fitting parameters) and KZ (Konder and Zelasko) model and showed that MKZ model is constant with the measured value from laboratory test, which is given in Figure 2.8.

$$\tau = \frac{G_0\gamma}{1 + \beta \left(\frac{\gamma}{\gamma_r} \right)^s} \quad (2.8)$$

Where τ : Shear strenght, G_0 : Initial shear modulus, γ : Shear strain. β , s are model fitting parameters and γ_r is reference shear strain.

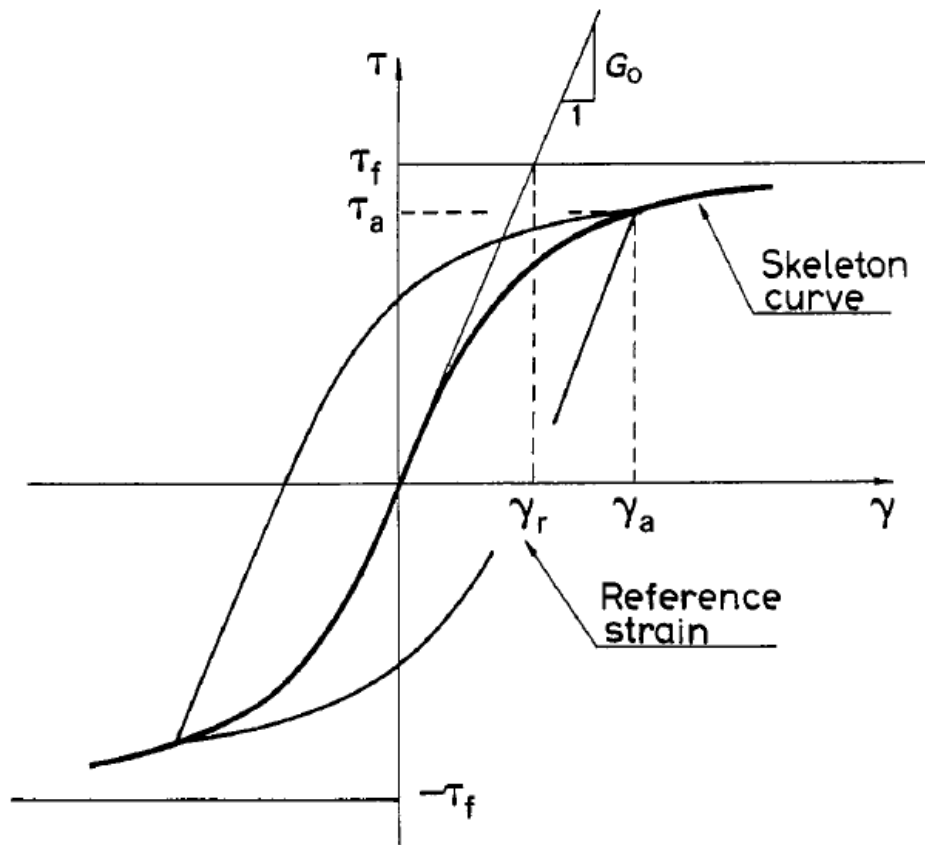


Figure 2.7 Hyperbolic model skeleton curve (Ishihara, 1996)

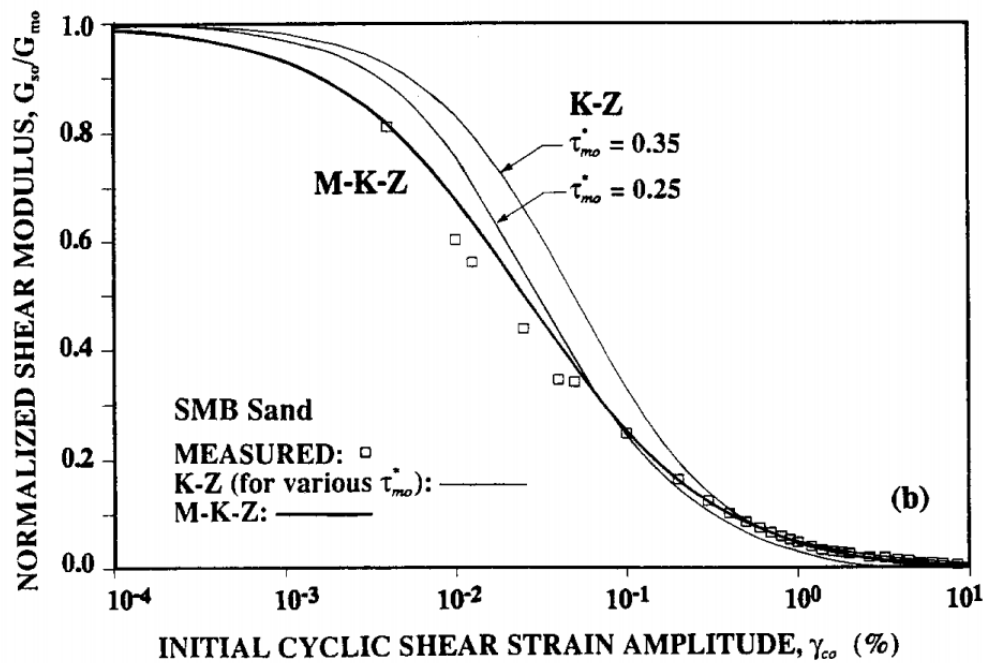


Figure 2.8 Normalized modulus reduction curve from MKZ, KZ and measured value (Matasovic & Vucetic, 1993)

Based on the hyperbolic model which is developed by Matasovic, Hashash and Park (2001) have investigated the effect of effective pressure on the hyperbolic soil model and developed the Eq. 2.9. which shows the effect of lateral soil pressure on shear strains. Hashash and Park (2001) named this new soil model as “Pressure dependent Hyperbolic Model.”

$$\gamma_r = a \left(\frac{\sigma'}{\sigma_{ref}} \right)^b \quad (2.9)$$

where a and b are curve fitting parameters and σ_{ref} is a reference confining pressure (Hashash & Park, 2001). Using this equation, reference strain is attributed to stress level. Reference stress should be selected as the stress level where parameter equals to reference strain.

Figure 2.9 shows the extended Masing rule that represents loading and unloading behavior of soils. Masing rules are summarized by Hashash, Philips and Groholski (2010). These rules can be listed as following: Virgin loading can be represented by backbone curve. Stress reversal can be represented using the following formula:

$$\frac{\tau - \tau_{rev}}{2} = f_{bb} \left(\frac{\gamma - \gamma_{rev}}{2} \right) \quad (2.10)$$

If Eq. 2.10 becomes equal to backbone curve (in other words, if reversal curve intersects with backbone curve), stress-strain behavior follows the backbone curve. Similarly, if Eq. 2.10 becomes equal to previous reversal cycles, previous cycles' curve governs and stress-strain behavior follows the behavior of the previous cycle.

Unloading reloading curves can be represented by the following equation (Phillips & Hashash, 2009).

$$\tau = \frac{2G_0 \left(\frac{\gamma - \gamma_{rev}}{2} \right)}{1 + \beta \left(\frac{\gamma - \gamma_{rev}}{2\gamma_r} \right)^s} + \tau_{rev} \quad (2.11)$$

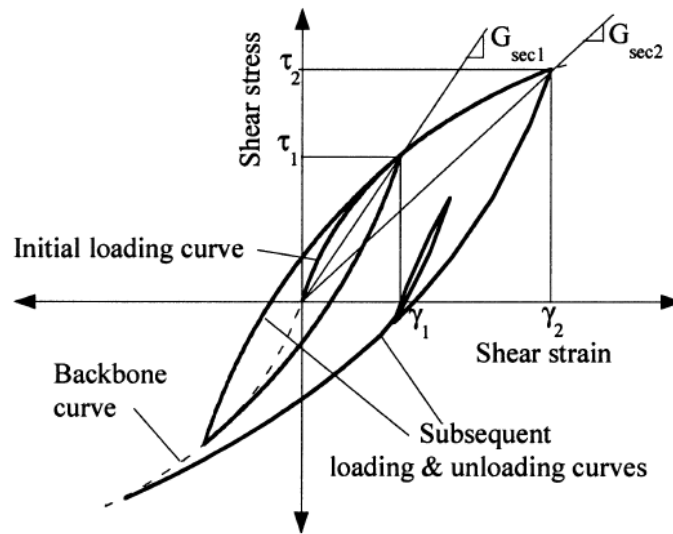


Figure 2.9 Hyperbolic soil model and extended masing rule (Hashash & Park, 2001)

Deepsoil (Hashash et al., 2017) software can perform one-dimensional non-linear time domain site response analysis model in deep soil deposits. Soil response is modeled using a modified hyperbolic model, developed by Matasovic and Vucetic (1993) with extended Masing criteria to represent hysteretic loading and unloading of soil (Hashash & Park, 2001). Hashash and Park (2001) explain the new soil model's difference as "The new soil model accounts for the influence of large confining pressures on strain dependent modulus degradation and damping of soil". The new soil model is called as "Pressure-dependent hyperbolic model" by Hashash and Park (2001) in Deepsoil (Hashash et al., 2017). This approach is put into engineering use by means of the well-known Deep Soil Site Response Analysis software.

2.5 Viscous and Hysteretic Damping

For site response analyses in nonlinear time domain, two types of damping models exist. These are known as viscous and hysteretic damping. Hashash, Philips and Groholski (2010) states that hysteretic damping results in zero damping at small strains. However, laboratory results and field cases prove the opposite. Therefore, hysteretic damping is supplemented by viscous damping in order to capture real behavior (Park & Hashash, 2004; Kwok, et al., 2007).

2.6 Viscous (Small-Strain) Damping

Rayleigh and Lindsay (1945) have proposed a damping matrix which is a combination of mass matrix and stiffness matrix. This expression includes small-strain damping based on the frequency.

$$[C] = a_0[M] + a_1[K] \quad (2.12)$$

where $[M]$ is the mass matrix, $[K]$ is the stiffness matrix and a_0 and a_1 are model parameters calculated based on the two distinct control frequencies for a given damping. (Rayleigh & Lindsay, 1945). However, this frequency based damping model of Rayleigh and Lindsay depends on the frequency and does not fit well enough to experimental results such as Lai and Rix (1998) which shows that damping is independent of the frequency at small strains in the range of 0.001 to 10 Hz.

Using the matrix formulation of viscous damping, Park and Hashash (2004) have defined four modes -extended Rayleigh scheme- in addition to two modes defined by Rayleigh formulation which is an extended version of Eq.2.13. Using the formula below and suggestions by Liu and Gorman (1995), authors have reached a frequency-independent damping formulation

$$[C] = [M] \sum_{b=0}^{N-1} a_b ([M]^{-1}[K])^b \quad (2.13)$$

Hashash, Philips and Groholski (2010) have compared the results of exact solution (frequency domain solution) with 2 mode Rayleigh damping and frequency independent analyses. Frequency independent analyses have match better with exact results as which is seen in Figure 2.10. With the increase of soil depth, the error in 2 mode Rayleigh damping increases.

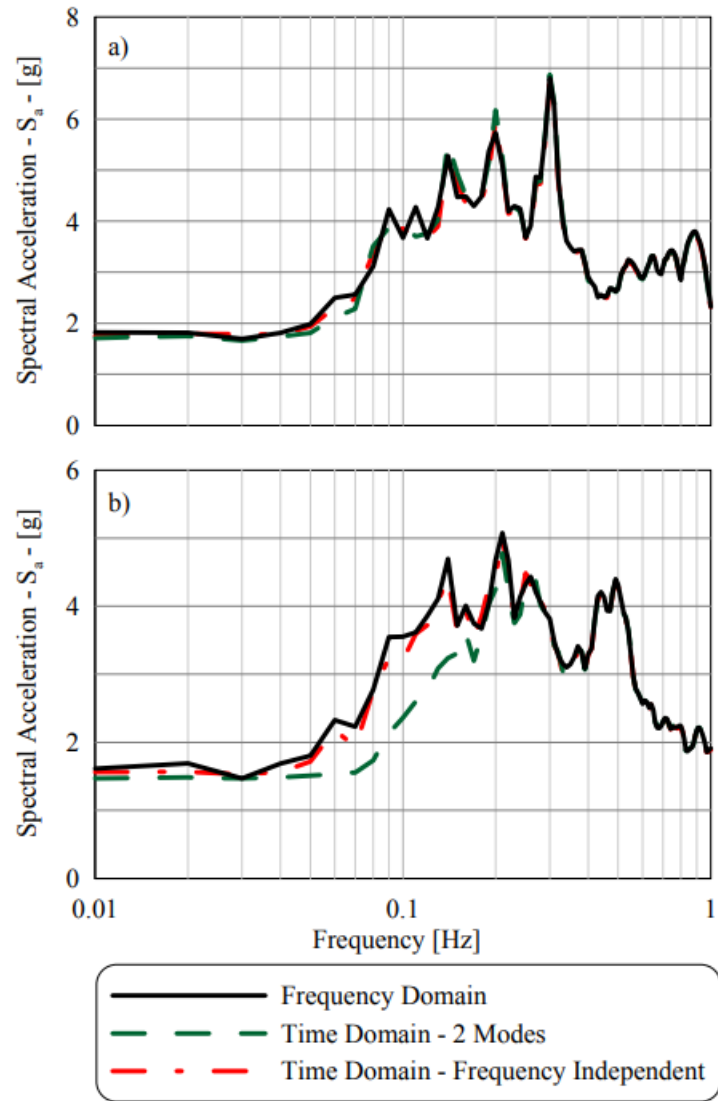


Figure 2.10 Comparison of different damping models for a) 100 m soil column b) 1000 m soil column (Hashash, Philips, & Groholski, 2010)

2.7 Hysteretic Damping

Hysteretic damping is calculated based on the Masing rules. However, hysteretic damping may result in very high dampings in high displacements (Kwok et al., 2007). Alternative models were proposed by many researchers.

Darandeli (2001) has proposed a reduction factor to reduce the over-estimated damping factor in high displacements. Definition of reduction factor is given by Eq. 2.14. Comparison of damping curve using Masing rule and adjusted curve is given by Darandeli (2001) as in Figure 2.11.

$$F = b * (G/G_{max})^p \quad (2.14)$$

where b and p are curve fitting parameters.

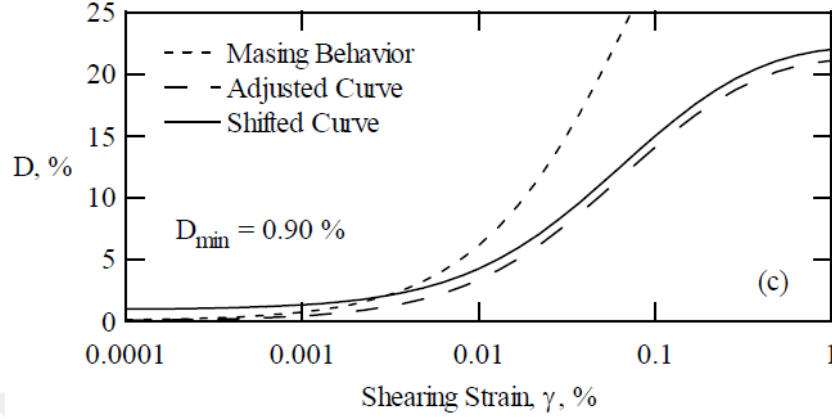


Figure 2.11 Damping curve comparison based on masing rule and proposed method (Darendeli, 2001)

Philips and Hashash (2009) proposed a new formulation for modification of Masing rule based on the studies of Darendeli (2001). The proposed formulation fits better than Darandeli's equation. Proposed damping reduction factor is given in Eq. 2.15.

$$F(\gamma_m) = p_1 - p_2 \left(1 - \frac{G_{\gamma m}}{G_0}\right)^{p_3} \quad (2.15)$$

Where p_1 , p_2 and p_3 are curve fitting parameters.

Procedure to determine the curve fitting parameters are described by Hashash, Philips and Groholski (2010). After determining the best backbone curve, resulting damping curve is calculated with the best backbone curve and Masing rule. Then, curve fitting parameters are calculated to reach best fit possible.

The reduction factor by Phillips and Hashash (2009) is used to modify the unloading-reloading curves. Eq. 2.16 was modified using the reduction factor (Eq. 2.15) as below:

$$\tau = F(\gamma_m) \left[\frac{2G_0 \left(\frac{\gamma - \gamma_{rev}}{2} \right)}{1 + \beta \left(\frac{\gamma - \gamma_{rev}}{2\gamma_r} \right)^s} - \frac{G_0(\gamma - \gamma_{rev})}{1 + \beta \left(\frac{\gamma_m}{\gamma_r} \right)^s} \right] + \frac{G_0(\gamma - \gamma_{rev})}{1 + \beta \left(\frac{\gamma_m}{\gamma_r} \right)^s} + \tau_{rev} \quad (2.16)$$

where, γ is the given shear strain, γ_r is the reference shear strain, β is the dimensionless factor, s is the dimensionless exponent, γ_{rev} is the reversal shear strain, τ_{rev} is the reversal shear stress, γ_m is the maximum shear strain, $F(\gamma_m)$ is the reduction factor and G_0 is the initial shear modulus.

Phillips and Hashash (2009) compared the newly developed MRDF procedure with MRD (Modulus Reduction + Damping) and MR (Modulus reduction only) procedures using the Darandeli's curves on sand. These results are presented in Figure 2.12. As can be seen, MRDM approach fits the Darandeli method very well and does not reduce the damping at high strains contrary to Darandeli method.

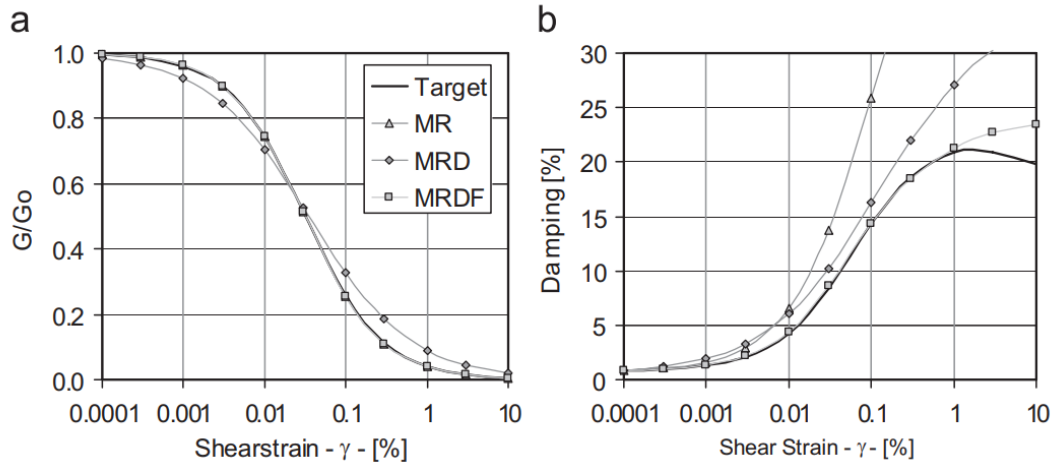


Figure 2.12 Evaluation of proposed damping reduction factor: (a) modulus reduction; (b) using damping curve using Darendeli's sand curves as target

2.8 Total Stress and Effective Stress Analysis

In this study, results of total and effective stress analysis are compared, the most significant difference of between these methods is the effective stress analysis considers excess pore water pressure generated during earthquake while total stress analysis does not consider. In general, effective and total stress is described as below:

$$\sigma = \sigma_{ef} + u \quad (2.17)$$

Where σ is total stress, σ_{ef} is effective stress and u is excess pore water pressure. Effective stress decreases with increasing excess pore water pressure based on Equation 2.17 but total stress does not change with increasing excess pore water

pressure. Many researchers showed that dynamic soil parameters (shear moduli and damping ratio) are also influenced by effective confining pressure. One of them, Ishibashi (1992) expressed that modulus reduction ratio ($G/G_{\max}$) increase with increasing effective stress and damping ratio (β) decrease with increasing effective stress. The detailed equations related with dynamic soil parameters will be given in the next chapters.

To explain more clearly the difference between effective and total stress analysis, an example is given in Figure 2.13. Total stress analysis uses high modulus ratio corresponding to low damping, but effective stress is vice versa. In the other words totals stress analysis causes low damping in the soil, effective stress causes high damping ratio in the soil. Low damping causes high spectral acceleration, velocity etc.

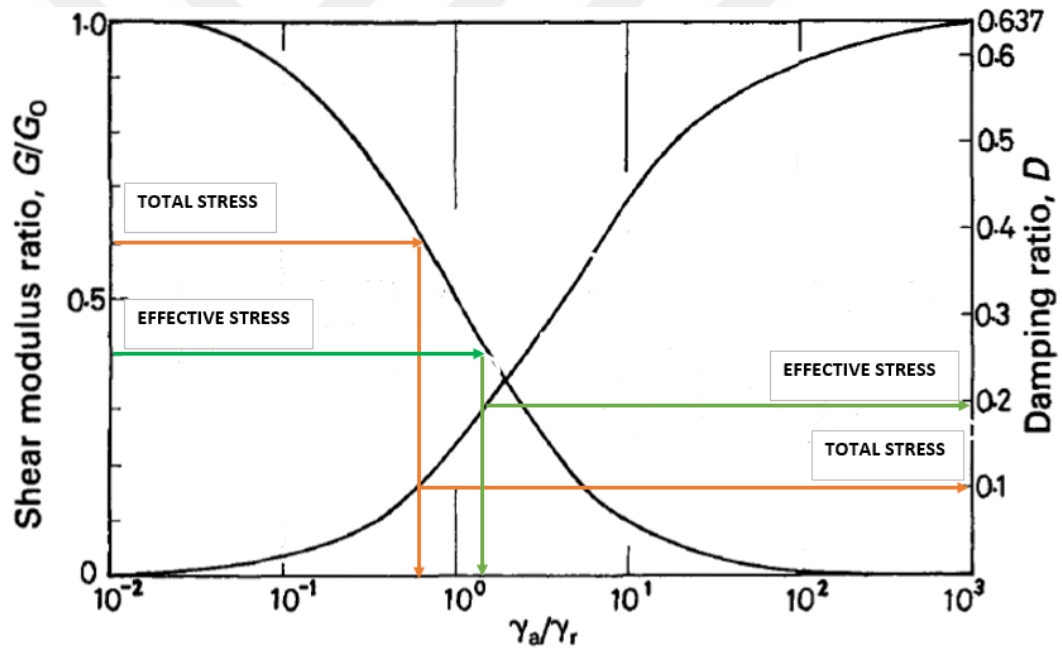


Figure 2.13 Example shear modulus and damping ratio curve

Ghaboussi and Dikmen (1984) studied on effective stress analysis of seismic response and liquefaction. Effective and total stress analysis were compared by Ghaboussi and Dikmen (1984) and results showed that total stress analysis gives higher spectral accelerations and velocities than effective stress analysis. Analysis results shows that the total stress analysis significantly overestimates the response of the system, analysis result is given Figure 2.14.

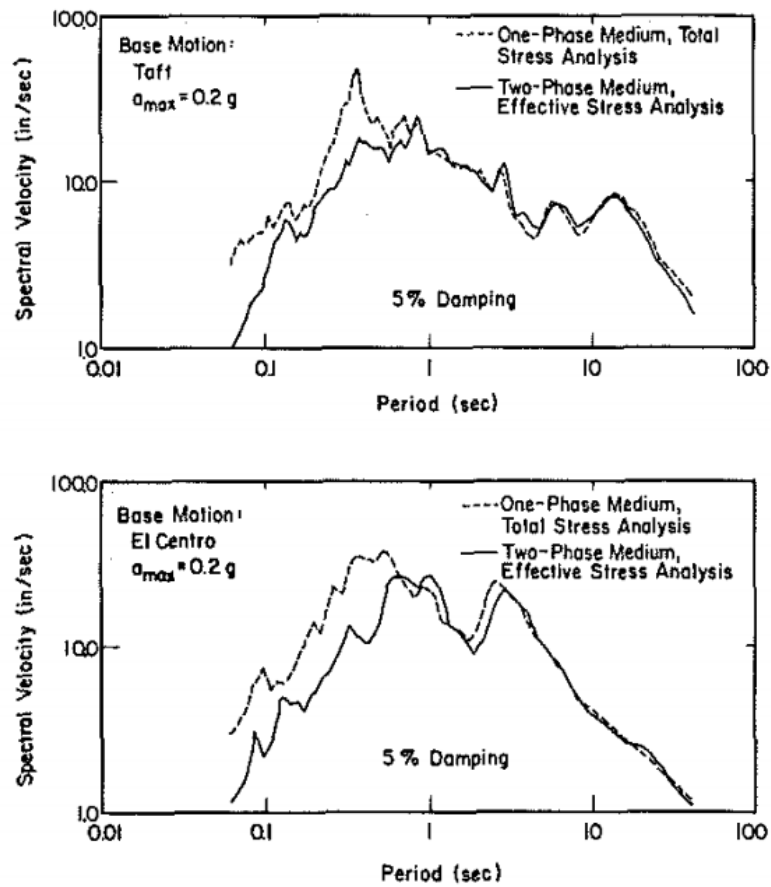


Figure 2.14 Comparison of spectral velocity according to total stress and effective stress analysis (Ghaboussi & Dikmen, 1984)

2.9 Dynamic Soil Models and Parameters

Dynamic response of soils is governed by dynamic soil parameters. These parameters are defined by: (1) the value of shear modulus at small strains G_{max} , which can be also expressed in terms of the shear wave velocity $V_s = \sqrt{G_{max}/\rho}$, where ρ = the mass density of the soil; (2) the relation between the secant shear modulus and the cyclic shear strain amplitude γ_c , expressed as a curve of G/G_{max} versus γ_c ; (3) the relation between the material damping ratio, λ and the cyclic shear strain amplitude, γ_c ; (4) the decrease of the stiffness G after N cycles of γ_c (Vucetic & Dobry, 1991).

Maximum shear modulus, modulus reduction with strain and damping ratio with strain are the three main parameters that affect the behavior of soil during dynamic excitation. Different soil properties effect these parameters and these effects are summarized in Table 1.1.

Table 1.1 Effects of different soil and loading properties on maximum shear modulus, modulus ratio and damping ratio (Kramer, 1996)

Increasing Factor	Max. Shear Modulus, G_{max}	Modulus Reduction, G/G_{max}	Damping Ratio, ξ
Effective confining pressure, σ'_m	Increases with σ'_m	Increases with σ'_m ; effect decreases with increasing PI	Decreases with effect decreases with increasing PI
Void ratio, e	Decrease with e	Increases with e	Decreases with e
Geologic age, t_g	Increases with t_g	May increases with t_g	Decreases with t_g
Cementation, c	Increases with c	May increases with c	May decreases with c
Over consolidation ratio, OCR	Increases with OCR	Not affected	Not affected
Plasticity index, PI	Increases with PI if OCR > 1; stays about constant if OCR = 1	Increases with PI	Decreases with PI
Cyclic strain, γ_c		Decrease with γ_c	Increases with γ_c
Strain rate, γ	No effect for non-plastic soils; increase with γ for plastic soils (up to -10% increase per log cycle increase in γ)	G increases with γ , but G/G_{max} probably not affected if G and G_{max} are measured at same γ	Stays constant or may increase with γ
Number of loading cycles, N	Decrease after N cycles of large γ_c , but recovers later with time in clays; increases with N for sand	Decreases after N cycles of large γ_c (G_{max} measured before N cycles) for clays; for sands, can increase (under drained conditions) or decrease (under indrain conditions)	Not significant for moderate γ_c and N

Dynamic soil parameters of sandy soils were investigated by various researchers. Seed et al., (1986) and Vucetic and Dobry (1991) and Ishibashi and Zhang (1993) have performed experiments both on sandy soils and clayey soils. Vucetic and Dobry (1991) have analyzed the effect of plasticity on dynamic soil parameters such as dynamic shear modulus and damping ratio which were presented in Figure 2.15 and Figure 2.16.

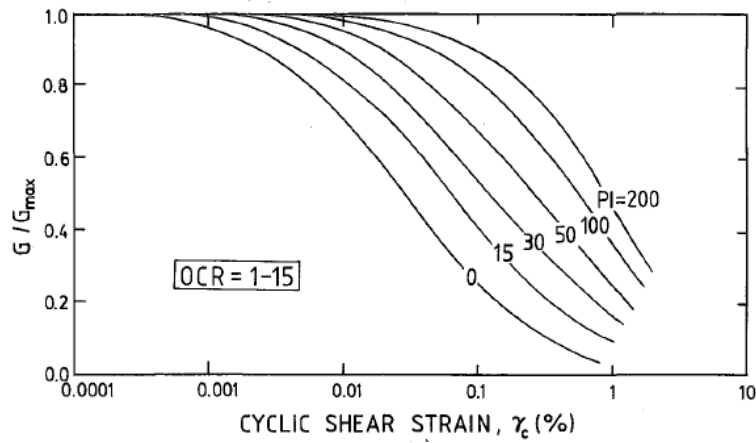


Figure 2.15 Relations between $G/G_{\max}$ versus γ_c and Soil Plasticity (Vucetic & Dobry, 1991)

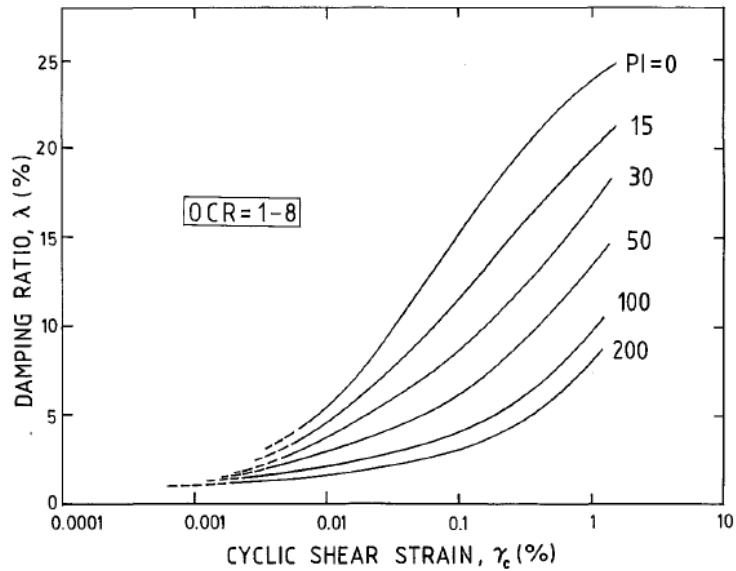


Figure 2.16 Relations between λ versus γ_c and Soil Plasticity (Vucetic & Dobry, 1991)

Ishibashi and Zhang (1993), using the hyperbolic equation proposed by Khouri (1984) based on number of tests on sandy soils, developed an expression which also includes the effect of plasticity index. In this study, the formulation proposed by Ishibashi and Zhang (1993) was adopted for shear reduction and damping calculations since this formulation considers both plasticity index and effect of confining pressure.

2.10 Ishibashi and Zhang Formulation for Non-Plastic Soils

The equivalent shear modulus, G is expressed by many researchers (Iwasaki et al., (1981); Kokusho, (1980); etc.) in the form of:

$$G = K(\gamma)f(e)\sigma'_0{}^{m(\gamma)} \quad (2.18)$$

where $K(\gamma)$ is a decreasing function of the cyclic shear strain amplitude γ , $f(e)$ is a function of void ratio e , σ'_0 is the mean effective confining pressure, and power $m(\gamma)$ is increasing function of γ . $G_{\max}$, the maximum value of G and is usually obtained at $\gamma=10^{-6}$ or less. Therefore, $G_{\max}$ is:

$$G_{\max} = K_0 f(e) \sigma'_0{}^{m_0} \quad (2.19)$$

where;

$$K_0 = K(\gamma \leq 10^{-6}) = 1 \quad (2.20)$$

$$m_0 = m(\gamma \leq 10^{-6}) \quad (2.21)$$

Where Eqs. (2.17) and (2.18), Eq. (2.19) were combined:

$$\frac{G}{G_{\max}} = K(\gamma) \sigma'_0{}^{m(\gamma)-m_0} \quad (2.22)$$

The result of $K(\gamma)$ and $m(\gamma)-m_0$ versus cyclic shear strain amplitude, γ are plotted, the following equations were obtained by Ishibashi and Zhang (1993),

$$K(\gamma) = 0.5 \left[1 + \tanh \left\{ \ln \left(\frac{0.000102}{\gamma} \right)^{0.492} \right\} \right] \quad (2.23)$$

$$m(\gamma) - m_0 = 0.272 \left[1 - \tanh \left\{ \ln \left(\frac{0.000556}{\gamma} \right)^{0.4} \right\} \right] \quad (2.24)$$

The damping ratio, D is expressed as a function of $G/G_{\max}$:

$$D_{sand} = 0.333 \left\{ 0.586 \left(\frac{G}{G_{max}} \right)^2 - 1.547 \left(\frac{G}{G_{max}} \right) + 1 \right\} \quad (2.25)$$

2.11 Dynamic Soil Parameters for Plastic Soils

Based on Ishibashi and Zhang (1993), $k(\gamma)$ and $m(\gamma)-m_0$ and D equations are modified to include plasticity index (I_p):

$$\frac{G}{G_{max}} = K(\gamma, I_p) \sigma_0'^{m(\gamma, I_p) - m_0} \quad (2.26)$$

$$D = D_{sand} \cdot A(I_p) \quad (2.27)$$

$$m(\gamma) - m_0 = 0.272 \left[1 - \tanh \left\{ \ln \left(\frac{0.000556}{\gamma} \right)^{0.4} \right\} \right] e^{-0.0145 I_p^{1.3}} \quad (2.28)$$

$$K(\gamma) = 0.5 \left[1 + \tanh \left\{ \ln \left(\frac{0.000102 + n(I_p)}{\gamma} \right)^{0.492} \right\} \right] \quad (2.29)$$

Following functions were found to satisfy the data reasonably well:

$$n(I_p) = \begin{cases} 0.0 & \text{for } I_p = 0 \quad (\text{sandy soils}) \\ 3.37 \times 10^{-6} I_p^{1.404} & \text{for } 0 < I_p \leq 15 \quad (\text{low plastic soils}) \\ 7.0 \times 10^{-7} I_p^{1.976} & \text{for } 0 < I_p \leq 70 \quad (\text{medium plastic soils}) \\ 2.7 \times 10^{-5} I_p^{1.115} & \text{for } I_p > 70 \quad (\text{highly plastic soils}) \end{cases} \quad (2.30)$$

$$D = \frac{0.333(1 + e^{-0.0145 I_p^{1.3}})}{2} \left\{ 0.586 \left(\frac{G}{G_{max}} \right)^2 - 1.547 \left(\frac{G}{G_{max}} \right) + 1 \right\} \quad (2.31)$$

Influence of effective confining pressure on modulus reduction curve with different plasticity indices are shown in Figure 2.17.

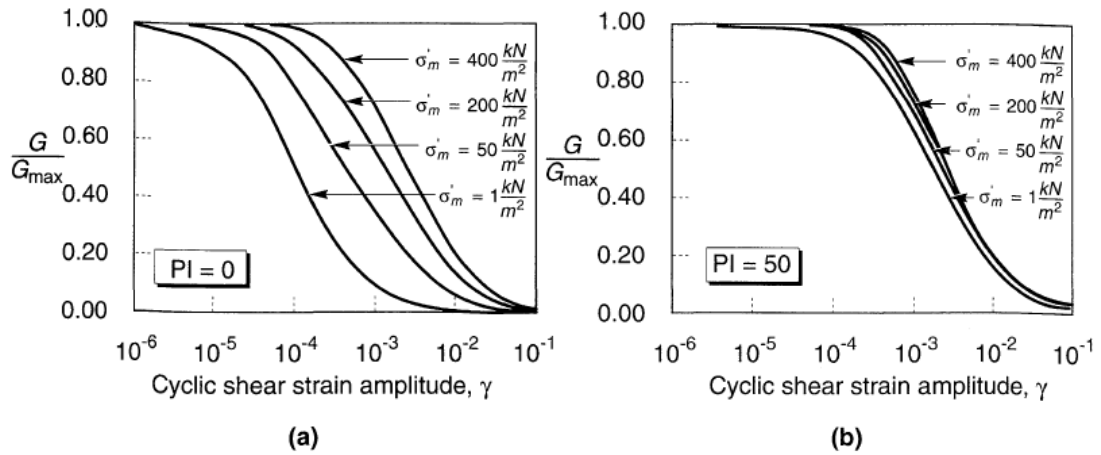


Figure 2.17 Influence of effective confining pressure on modulus reduction versus cyclic shear strain curves: (a) Non-plastic soil (PI = 0) and (b) Plastic soil (PI=50), (Ishibashi, 1992)

2.12 Dynamic Soil Parameters for Rock

EPRI (Electric Power Research Institute, 1993) has produced shear modulus reduction and damping curves for rocks. According to Hartzell et al. (2004), EPRI curves are better than previous curves for rock since they include the effect of depth. EPRI curves as quoted by Hartzell et al. (2004) are given in Figure 2.18.

These curves show that nonlinear properties of rock which are shear modulus and damping ratio change with depth. As it is expected, as depth increases, shear strain and damping ratio of rocks decreases besides that shear modulus of rocks increase.

The shear modulus reduction and damping ratio curve of rocks proposed by EPRI (1993) should be evaluated according to rock depth of site soil model. For example, specific to the current project, given curves for rocks located in depth of 501 to 1000, 1001 to 2000 and 2001 to 5000 feet were used.

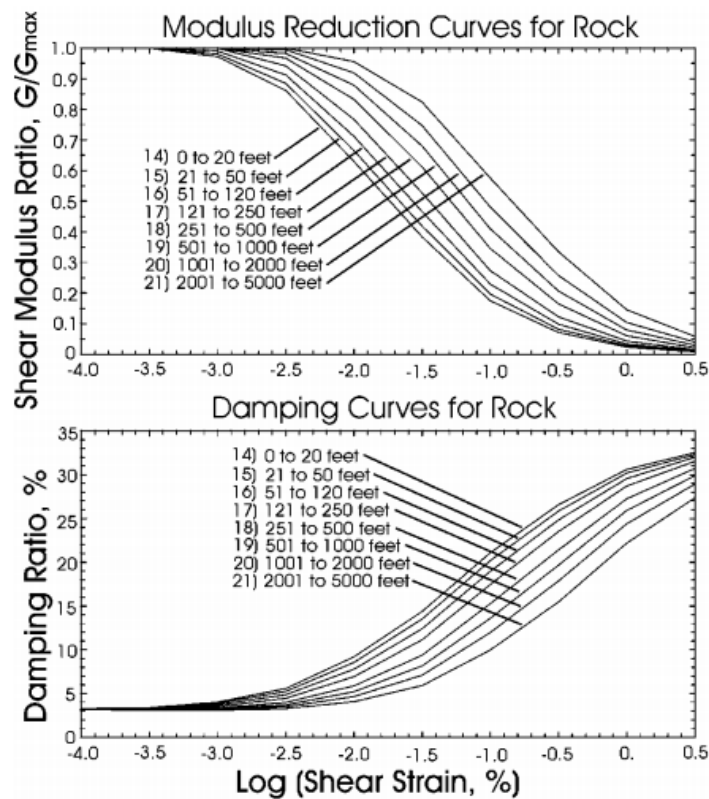


Figure 2.18 EPRI modulus reduction and damping ratio curves for rock (Quoted from Hartzell et al., 2004)

2.13 Porewater Pressure Generation Models and Parameters

In order to take into account the effect of pore pressure generation during nonlinear effective stress analyses, many studies exist correlation results with soil parameters to calculate pore pressure easily. These methods are included in DEEPSOIL and selected methods to calculate excess pore pressures are described below.

These methods are divided into two for cohesive soils and granular soils. Excess pore pressure calculations in nonlinear effective stress analyses are calculated based on Matasovic and Vucetic (1993) for granular layers and Matasovic and Vucetic (1995) for cohesive layers.

Correlation to calculate excess pore pressure for granular layers is presented below:

$$u_N = \frac{p * f * N_c * F * (\gamma_c - \gamma_{tvp})^s}{1 + f * N_c * F * (\gamma_c - \gamma_{tvp})^s} \quad (2.32)$$

where u_N is normalized excess pore pressure, N_c is equivalent number of cycles, γ_c is the current reversal shear strain, γ_{tvp} is threshold shear strain value, p, s and F are curve fitting parameters, f is dimensionality factor (Matasovic & Vucetic, 1993). This model also uses v parameter, which is pore pressure degradation coefficient.

Carlton (2014) has developed following correlations for Matasovic and Vucetic's (1993) curve fitting parameters. Using the shear wave velocity and fines content (FC), curve fitting parameters F and s can be determined.

$$F = 3810 \times V_s^{-1.55} \quad (2.33)$$

$$s = (FC + 1)^{0.1252} \quad (2.34)$$

Last curve fitting parameter p can be assumed as 1 in case of absence of laboratory experiments. Dimensionality factor can be taken as 1 for 1D analyses.

Excess pore pressure for cohesive soils are calculated as below:

$$u_N = AN_c^{-3s(\gamma_c - \gamma_{tvp})^r} + BN_c^{-2s(\gamma_c - \gamma_{tvp})^r} + CN_c^{-s(\gamma_c - \gamma_{tvp})^r} + D \quad (2.35)$$

Where u_N is normalized excess pore pressure, N_c is equivalent number of cycles, γ_c is the current reversal shear strain, γ_{tvp} is threshold shear strain value, r, s, A, B, C and D are curve fitting parameters.

Carlton (2014) has developed following correlations for Matasovic and Vucetic's (1995) curve fitting parameters. Using PI and OCR, curve fitting parameters can be calculated.

$$s = 1.6374xPI^{-0.802}xOCR^{-0.417} \quad (2.36)$$

$$r = 0.7911xPI^{-0.113}xOCR^{-0.147} \quad (2.37)$$

$$A = \begin{cases} 7.6451 & \text{for } OCR < 1.1 \\ 15.641xOCR^{-0.242} & \text{for } OCR \geq 1.1 \end{cases} \quad (2.38)$$

$$B = \begin{cases} -14.714 & \text{for } OCR < 1.1 \\ -33.691xOCR^{-0.33} & \text{for } OCR \geq 1.1 \end{cases} \quad (2.39)$$

$$C = \begin{cases} 6.38 & \text{for } OCR < 1.1 \\ 21.45xOCR^{-0.468} & \text{for } OCR \geq 1.1 \end{cases} \quad (2.40)$$

$$D = \begin{cases} 0.6922 & \text{for } OCR < 1.1 \\ -3.4708xOCR^{-0.857} & \text{for } OCR \geq 1.1 \end{cases} \quad (2.41)$$

DEEPSOIL (Hashash et al., 2017) performs nonlinear excess pore pressure analyses using the models described above.

2.14 Definition and Selection of Strong Ground Motion

Ground motion due to the earthquakes is quite complicated. It is essential to determine ground motion parameters, in order to accurately characterize ground motion features. Evaluation of ground motion parameters requires the examination of strong ground motion measured during actual earthquakes. Strong ground motion is measured by the accelerograph and the measured motion is called as an accelerogram.

Earthquake motion has three important features which are amplitude (max. acceleration), frequency content (predominant period) and duration (effective duration) of motion, respectively. These parameters are adapted to describe the primary characteristics of the ground motion.

2.14.1 Amplitude Parameters

One of the features of ground motion is amplitude parameters which are acceleration, velocity and displacement. Time records are used for the evaluation of a ground motion features. In order to determine amplitude of particular a ground motion, the most common used parameter is the peak horizontal acceleration (PHA) value. The peak horizontal acceleration can be defined as the highest values obtained from an accelerogram. Figure 2.19 shows the acceleration vs. time history of Kocaeli 1999 earthquake. Although peak acceleration value is very useful, it does not give information about frequency content and duration.

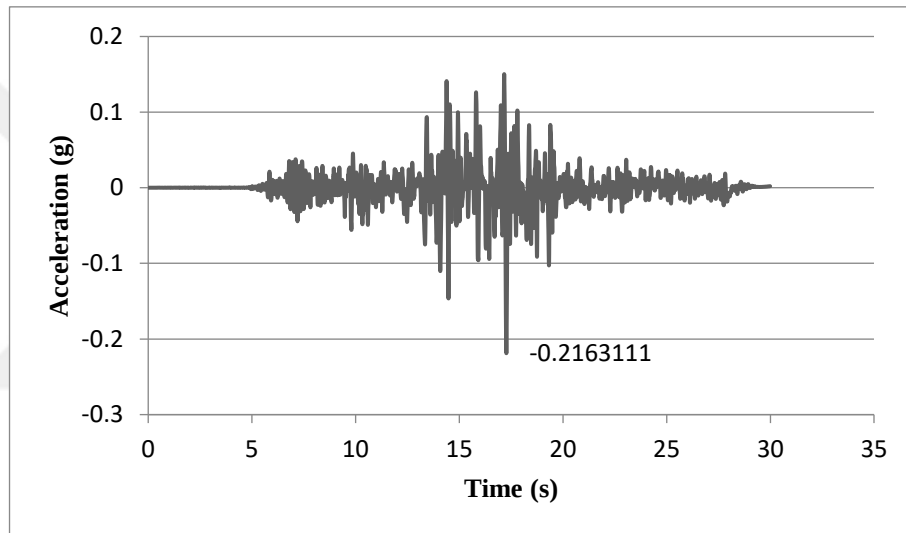


Figure 2.19 Kocaeli Earthquake time acceleration record taken from DEEPSOIL software earthquake motion database

Among amplitude parameters, peak horizontal velocity is useful as (Figure 2.20). Ground motion can be described by peak velocity value rather than peak acceleration value at medium frequency ground motions since velocities are less sensitive to higher frequencies. The other amplitude parameter is peak displacement value which is related to low frequency content of earthquake motion and typical displacement time history of Kocaeli 1999 earthquake is given in Figure 2.21.

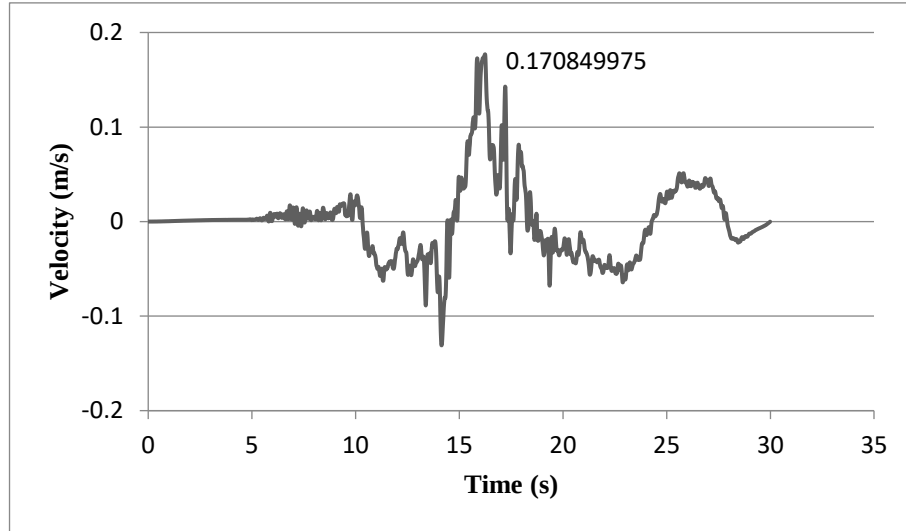


Figure 2.20 Kocaeli Earthquake time velocity record taken from DEEPSOIL software earthquake motion database

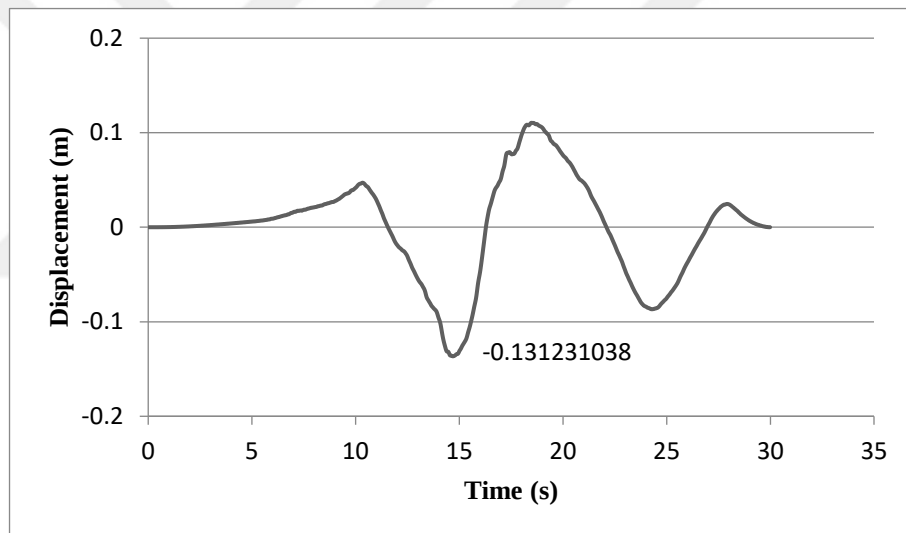


Figure 2.21 Kocaeli Earthquake time displacement record taken from DEEPSOIL software earthquake motion database

The amplitude parameters obtained from the acceleration time, velocity time and displacement time graphs of Kocaeli earthquake are given in Table 2.2 for example.

Table 2.2 Amplitude parameters of Kocaeli Earthquake which taken from DEEPSOIL software earthquake motion database

Parameter	Kocaeli Earthquake, 1999
Peak Acceleration (g)	0.22
Peak Velocity (m/s)	0.17
Peak Displacement (m)	0.13

2.14.2 Frequency Content Parameters

The frequency content describes how the ground motion amplitude is distributed among the various frequencies or period. The frequency content parameters of a strong ground motion can be obtained from ground motion spectra which are Fourier Spectrum, Power Spectrum and Response Spectrum. The ground motion spectra are described to characterize the strong ground motion. The complex ground motion spectra can be expressed as sum of simple harmonic motion terms at different frequency, amplitude and phase of the periodic function.

The simple harmonic function can be expressed in terms of displacement using a trigonometric notation which is expressed as:

$$x(t) = A \sin(\omega t + \phi) \quad (2.34)$$

The sum of simple harmonic motion terms is implemented using Fourier series equation which is expressed as:

$$x(t) = c_0 + \sum_{n=1}^{\infty} c_n \sin(\omega_n t + \phi_n) \quad (2.35)$$

where c_n , ω_n and ϕ_n are the amplitude, angular frequency and phase angle, respectively of the given harmonic series.

The Fourier amplitude spectrum of strong ground motion shows how the ground motion amplitude is distributed according to the frequency or period as shown in Figure 2.22. The complex behavior of the ground motion in the time domain may be expressed by help of Fourier amplitude spectra. Furthermore, when the Fourier amplitude spectrum of the earthquake motion is smoothed and expressed with logarithmic scales therefore, the characteristic shape of the earthquake motion can be observed better.

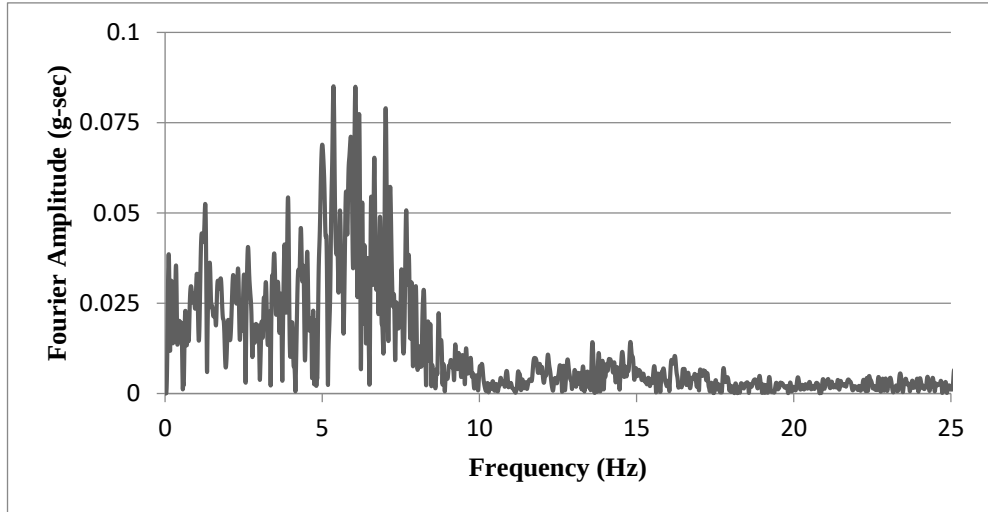


Figure 2.22 Fourier amplitude spectrum of Kocaeli Earthquake which is taken from earthquake motion database of DEEPSOIL software

The predominant period (T_p) which is described as corresponding to maximum value at the Fourier amplitude spectra, is a useful parameter to represent the frequency content of the ground motion roughly. To avoid undesirable peak values, the predominant period value is obtained from smoothed Fourier amplitude spectrum. Figure 2.23 shows two different Fourier amplitude spectra with narrow and wide spectra but with same predominant period.

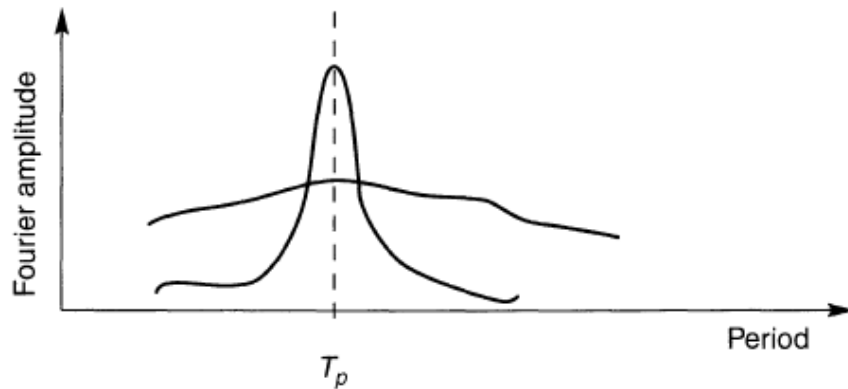


Figure 2.23 Narrow and wide fourier spectrum (Kramer, 1996)

Response spectra (shown in Figure 2.24) expresses a maximum response which is given by single degree of freedom system with the natural frequency and the damping ratio (damping is generally accepted as 5%) versus to the ground motion. Typical response spectra shape shows that peak spectral acceleration, velocity, and

displacement values occur at different frequencies or periods. Studies have shown that acceleration is constant at higher frequencies as shown in Figure 2.24.

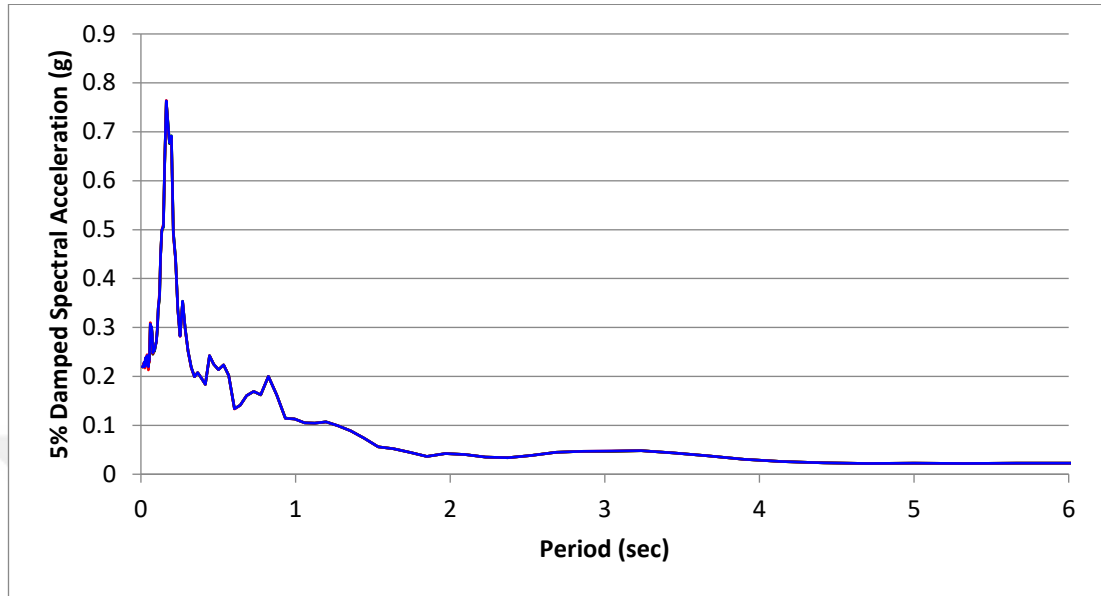


Figure 2.24 Response spectra (Spectral Acceleration) of Kocaeli Earthquake which is taken from earthquake motion database of DEEPSOIL software.

2.14.3 Duration of Motion

The duration of strong ground motion can play an important role in the response of structures. The strong ground motion duration can have a very strong impact on structural damage. So, earthquake motion duration is important parameter to determine the earthquake motion. Strong ground motion was defined by number of investigators to be used for assessment of seismic response and the potential for structural damage due to earthquake events.

Strong ground motion duration is defined as the time between the first and last time of the threshold acceleration ($0.05g$) is exceeded. This time is called as bracketed duration (Bolt, 1969). Bracketed duration is associated with magnitude and source to site distance (Bolt, 1969). The strong ground motion duration usually increases with increasing magnitude of earthquake.

The bracketed duration of Kocaeli Earthquake 1999 is determined as 9.5 sec (first exceedance is 10 sec and last exceedances is 19.5 sec) shown in Figure 2.25.

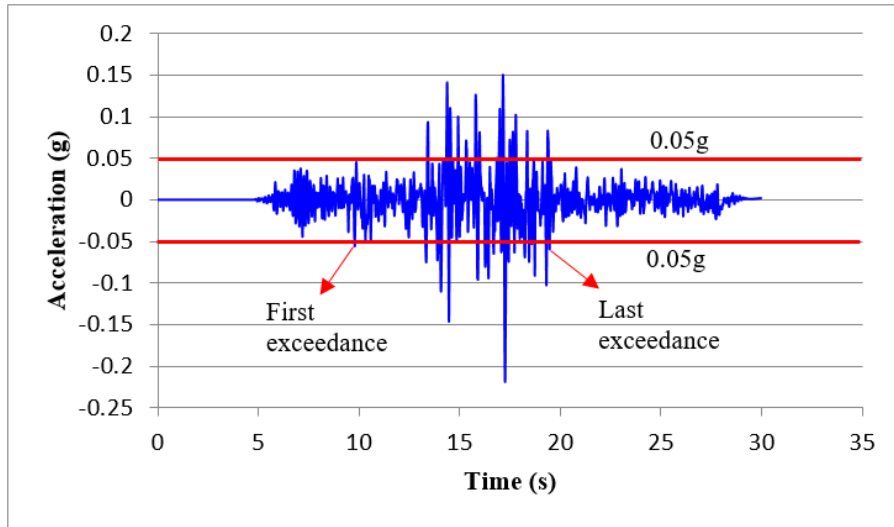


Figure 2.25 The bracketed duration of Kocaeli Earthquake, time acceleration record taken from DEEPSOIL software earthquake motion database

One of the most common definitions of strong ground motion is given by Trifunac and Brady (1975), which expresses definition of strong ground motion as a percentage of cumulative energy. It is defined as; $T = T_{0.95} - T_{0.05}$, where $T_{0.95}$ and $T_{0.05}$ represent the times at %95 and %5 of the cumulative energy. The other definition of strong ground duration is given by Kennedy et al. (1984) that is defined as; $T = T_{0.75} - T_{0.05}$, where $T_{0.75}$ and $T_{0.05}$ represent the times at %75 and %5 of the cumulative energy.

After definition of strong ground motion parameters, selecting strong ground motion according to study site is important. And then selected ground motion records should be scaled according seismicity of study site. Next chapter describes how the selecting and scaling strong ground motion from PEER (Pacific Earthquake Engineering Research Center).

2.15 Selection of Strong Ground Motion from PEER

Pacific Earthquake Engineering Research Center (PEER, 2006) ground motion database provides tools for searching, selecting and downloading ground motion data. All download records are unscaled, however, the scaling tool for the determination of scaling factor exists. In the following paragraphs, selection and scaling of strong ground motion from PEER is described.

In this study, real earthquake records were used in ground response analyses. These records are selected from PEER Ground Motion Database (PEER, 2006) based on the seismicity, soil condition, fault type, earthquake magnitude, distance between site and rupture zone, shear wave velocity, duration of earthquake. The procedure for the selection and scaling of earthquake motion is specified below:

- 1) First of all, the coefficients of map spectral acceleration should be determined according to earthquake ground motion level (DD-1, DD-2, etc.) which are short period, S_s and 1 second period, S_1 . Also period of transition to constant displacement zone, T_L should be determined which is taken 6 seconds based on TBDY (2018). Target spectrum is created using spectral acceleration coefficients (S_s , S_1) and period of transition to constant displacement zone (T_L).
- 2) In second step, earthquake motion records should be researched in accordance with created target spectrum. Before selection of earthquake motion records, parameters of earthquake motion should be searched which are fault type, magnitude, distance from fault rupture plane and shear wave velocity of bedrock according to study site.
- 3) In third step, earthquake motions should be scaled. PEER online data base performs scaling of earthquake motions using three approaches. One of the approaches is Minimize MSE (mean squared error) method which produces scaled recordings that provide the best match to the spectral shape of the target spectrum over the user-specified period range of interest. The scaling is just applied on amplitude, the method does not change period content. MSE factor is calculated as below:

$$MSE = \frac{\sum_i w(T_i) \{ \ln(SA^{target}(T_i)) - \ln(SA^{record}(T_i)) \}^2}{\sum_i w(T_i)} \quad (2.36)$$

Scaling factor is calculated as below:

$$\ln f = \frac{\sum_i w(T_i) \ln \left(\frac{SA^{target}(T_i)}{SA^{record}(T_i)} \right)}{\sum_i w(T_i)} \quad (2.37)$$

Where SA^{record} is real acceleration spectrum, SA^{target} is target acceleration spectrum, T_i (the period range from 0.01 second to 10 seconds) is the period of the scaling, parameter $w(T_i)$ is a weight function, f is the scale factor.

- 4) In fourth step, the properties of selected earthquake motion records are given. But selected earthquake motion records are unscaled, selected earthquakes' amplitudes are multiplied by scaling factors to obtain scaled earthquakes.

CHAPTER THREE

STUDIES OF SITE RESPONSE ANALYSES

3.1 Introduction

Ground response analyses will be performed for both real earthquakes taken from of Pacific Earthquake Engineering Research Center (PEER, 2006) and Turkish Earthquake Code (TBDY, Earthquake Code for Buildings, 2018) design response spectrum. Selected real earthquakes will be scaled based on the TBDY design spectrum. Specific site response analyses results depend on many properties which can be listed as below:

- Depth of engineering bedrock
- Soil stratification
- Soil types
- Water table elevation
- Epicenter distance
- Magnitude and frequency content
- Geologic condition / Site seismicity
- Types of dominating seismic waves

Flow-chart of specific site response analysis is given in Figure 3.1.

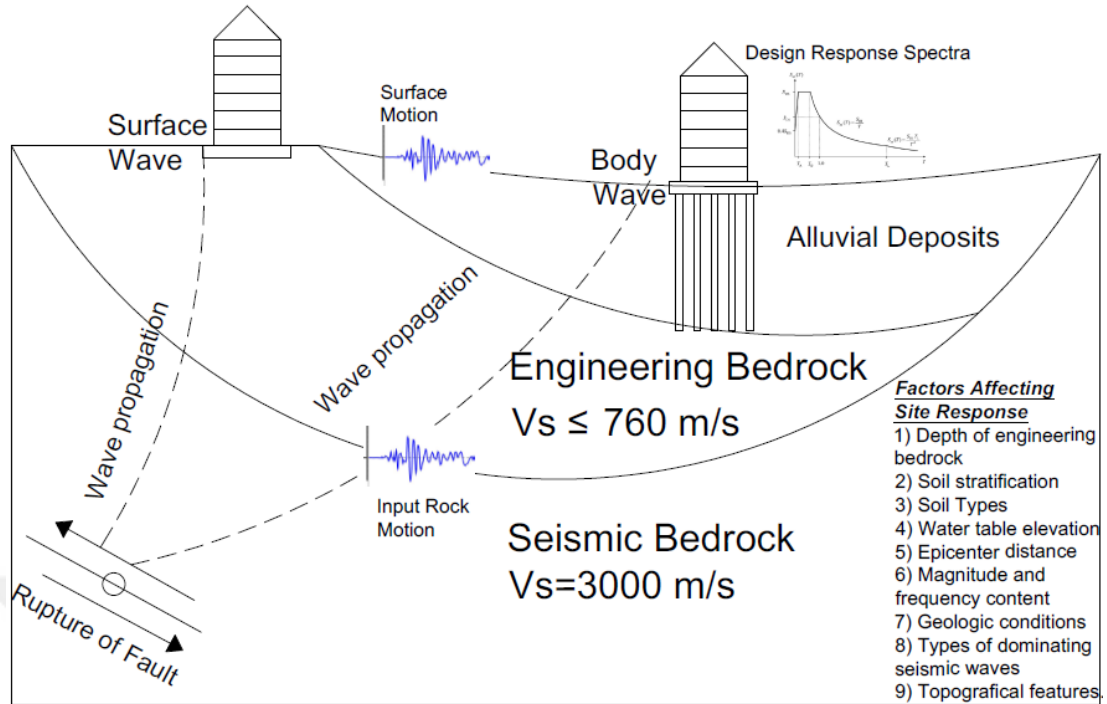


Figure 3.1 Site response analysis flow chart

This chapter describes parameters for the site under consideration such as soil conditions required in the analyses, bedrock depth, soil models, earthquake motions, mathematical model and others.

3.2 Investigation of Site Seismicity

İzmir is located at the west of Turkey, near Aegean sea. Since the seismicity of the region depends on the active faults in the vicinity of project area, active faults are investigated. Seismic activity is controlled by 4 main plate which are Anatolian, Arabian, African and Eurasian plates. Since plate moves constantly, plate contacts are squeezed and faults emerge. Tectonic structure of Turkey is shown in Figure 3.2.

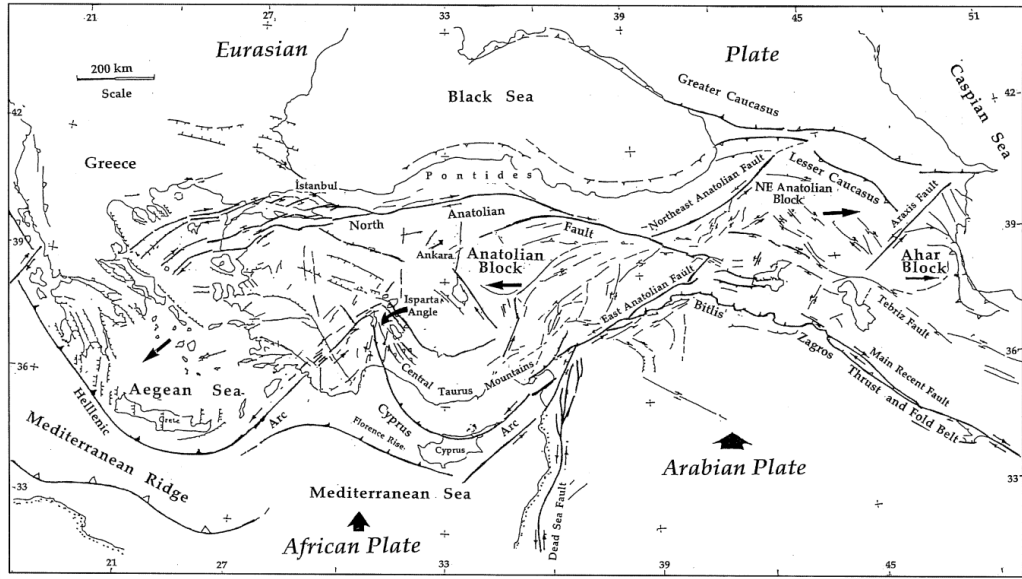


Figure 3.2 Major tectonic plates structures of Türkiye (Barka & Reilinger, 1997)

In İzmir region (Western Anatolia), Major graben systems are Gediz, Büyük Menderes and Küçük Menderes. Normal faults and strike slip faults developed parallel to these major graben systems. The active faults are searched at 50km vicinity of the İzmir city center. In total, 10 active faults are identified. In the region, active tectonic structures are strike-slip and normal faults. Normal faults which are İzmir Fault, Manisa Fault and Kemalpaşa Fault are in the north and east of İzmir Bay. The NNW-SSE directed Karaburun Fault on the west end of İzmir Bay has normal faults characteristics. Strike-slip faults which are Tuzla Fault, Foça-Bergama Fault, Seferihisar Fault, Gülbahçe Fault, Yağcılar Fault and Dağkızılca Fault take place in the north and south of İzmir. Active faults in the vicinity of project area are shown in Figure 3.3 and Figure 3.4.

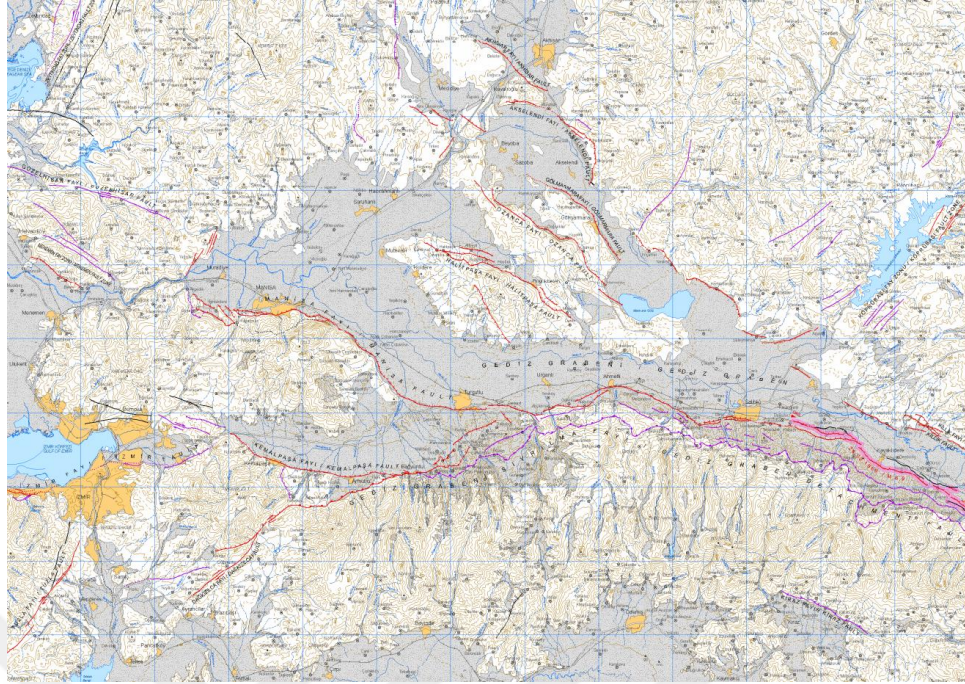


Figure 3.3 Active fault map of Eastern Izmir Bay of Turkey (MTA, 2016)



Figure 3.4 Active fault map of Urla of Turkey (MTA, 2016)

Records of earthquake that occurred near İzmir since 1900 are collected in AFAD (Disaster and Emergency Management Authority - www.afad.gov.tr) website. View of the earthquakes within the range of 5.00-10.00 Mw is given in the figure below.

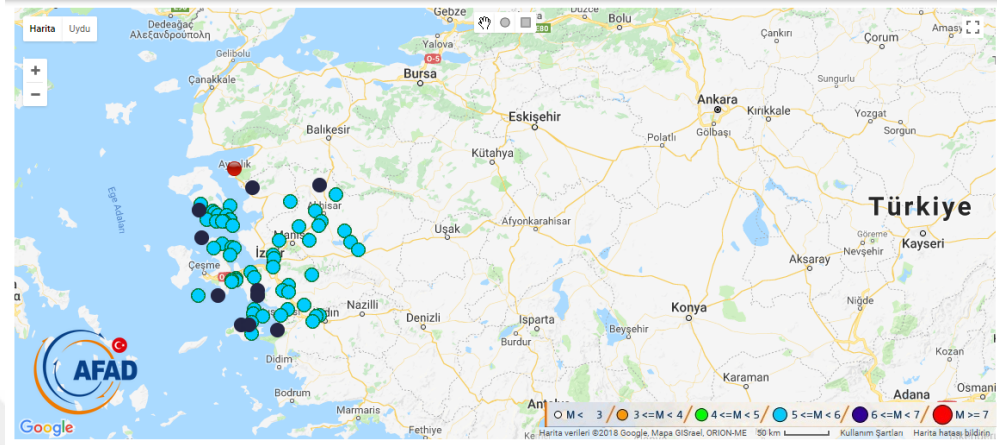


Figure 3.5 Earthquakes with 5 to 10 Mw occurred in İzmir and vicinity (AFAD, 2018)

Based on the formulation of Wells and Coppersmith (1994) that relates active fault length and distance of active fault to the project location with earthquake magnitude, expected earthquake magnitudes for each active fault are calculated. Table 3.1 shows the major active faults, their fault types, length, general direction and expected magnitude based on the Wells and Coppersmith (1994).

Based on the table below, an earthquake between 6.0-8.0 M_w is expected to occur in İzmir. Mostly, normal and strike slip type faults are observed. During the earthquake case selection to use in the analyses, seismicity of the region and information given in Table 3.1 are considered.

Table 3.1 Active fault properties nearby Izmir Region

Fault Name	Fault Type	Length (Km)	General Direction	Magnitude	Distance of Investigation Location (km)
Izmir Fault	NORMAL	35	D-B	6.5-7.0	1
Tuzla Fault	RIGHT DIRECTION STRIKE SLIP	50	K30D	7.0-8.0	12
Seferihisar Fault	RIGHT DIRECTION STRIKE SLIP	30	K20D	6.5-7.0	25
Gulbahçe Fault	STRIKE SLIP	70	K-G	7.0-8.0	50
Gediz Graben Main Detachment Fault	NORMAL	27	K70D	6.0-6.5	15
Kemalpasa Fault	NORMAL	24/27	K75D	6.0-6.5	15
Manisa Fault	NORMAL	20	K65B	6.0-6.5	20
Dagkizilca Fault	RIGHT DIRECTION STRIKE SLIP	27	K70D	6.5-7.0	15
Gumuldur Fault	NORMAL	15	K55B	6.0-6.5	45
Yagcilar Fault	RIGHT DIRECTION STRIKE SLIP	12	K-G	6.0-6.5	45

3.3 Design Response Spectra Based on TBDY (2018)

To design earthquake-resistant buildings, reaction of buildings to the ground motion should be known. Reaction of buildings to ground motion can be explained with response spectrum. In more general way, response spectrum represents the seismic loads for dynamic analyses of the buildings. Design response spectra include the reaction of a site to earthquakes that may occur in the field based on the soil conditions and seismicity.

Response spectra can be developed in two different ways: Site specific response spectra and code-based response spectra. In this section, code-based response spectra derivation is explained.

Turkish Earthquake Code (TBDY, Earthquake Code for Buildings, 2018) explains earthquake hazard levels as follows:

- Earthquake Level DD-1: Probability of exceedance of spectral amplitudes in 50 years is 2%, hence 2475 years return period. Named as very rare, highest earthquake.

- Earthquake Level DD-2: Probability of exceedance of spectral amplitudes in 50 years is 10%, hence 475 years return period. Standard design earthquake.
- Earthquake Level DD-3: Probability of exceedance of spectral amplitudes in 50 years is 50%, hence 72 years return period. Frequent earthquakes.
- Earthquake Level DD-4: Probability of exceedance of spectral amplitudes in 50 years is 68%, hence 43 years return period. Very frequent.

In this study, highest earthquake DD-1 and standard design earthquake DD-2 will be used for elastic design spectrum analyses.

Using the earthquake maps prepared by Disaster and Emergency Management Authority of Turkey (AFAD, 2018) earthquake amplitudes – design parameters, i.e. spectral accelerations at 0.2s and 1s periods, expected for a location can be obtained with design response spectra.

Figure 3.6 shows the project location in earthquake maps. To determine the design parameters, earthquake level and soil parameters should be given to the website. Earthquake levels are taken as DD-1 and DD-2 along with ZF soil type based on geotechnical and geophysical parameters such as shear wave velocity, undrained shear strength, SPT value. Detailed description of soil conditions will be given in the following sections.

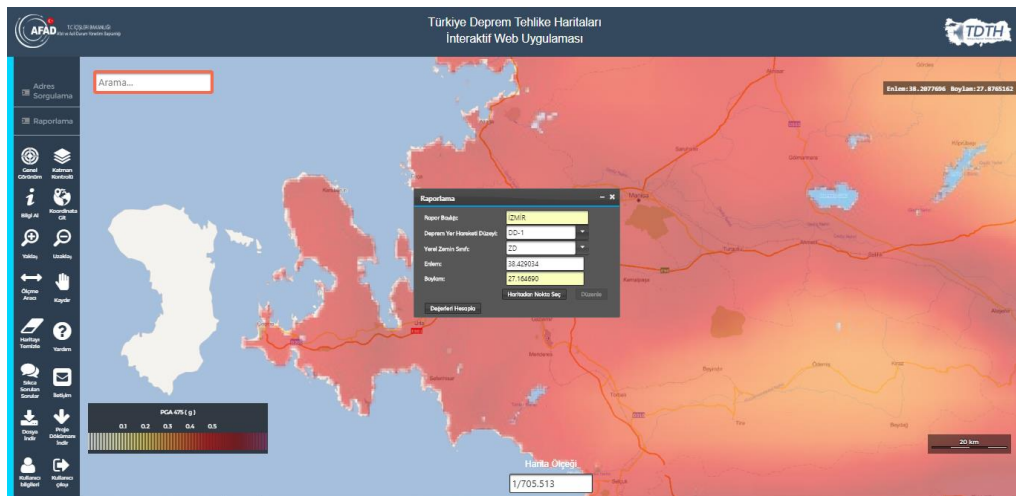


Figure 3.6 Turkey earthquake hazard maps interactive web application (AFAD, 2018)

First step to create the design response spectra is determining map based spectral accelerations. These values can be determined based on the coordinates of the site using the earthquake hazard maps. Map based spectral accelerations include short period spectral acceleration (S_s) and spectral acceleration for 1 sec period (S_1). Equations that are used to develop response design spectra are given below.

Calculations of design spectral accelerations (S_{DS} and S_{D1}) are given below:

$$S_{DS} = F_s * S_s \quad (3.1)$$

$$S_{D1} = F_1 * S_{S1} \quad (3.1)$$

Local soil condition factors to calculate design accelerations, F_s and F_1 can be determined using the Table 3.2 and Table 3.3 below.

Table 3.2 Local soil condition factors for short period accelerations

Soil Class	Local Soil Condition Factors for Short Period Accelerations, F_s					
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s = 1.25$	$S_s \geq 1.50$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1	1
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF	Specific site response analysis will be performed.					

Table 3.3 Local soil condition factors for 1 sec period spectral accelerations

Soil Class	Local Soil Condition Factors for 1 s Period Spec. Acc., F_1					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	Specific site response analysis will be performed.					

Ordinates of horizontal elastic response spectra can be calculated using the formulas below with building's natural period (TBDY, 2018).

$$S_{ae}(T) = \left(0.4 + 0.6 \frac{T}{T_A}\right) S_{DS} \quad (0 \leq T \leq T_A) \quad (3.2)$$

$$S_{ae}(T) = S_{DS} \quad (T_A \leq T \leq T_B) \quad (3.3)$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \quad (T_B \leq T \leq T_L) \quad (3.4)$$

$$S_{ae}(T) = \frac{S_{D1} T_L}{T^2} \quad (T_L \leq T) \quad (3.5)$$

$$T_A = 0.2 \frac{S_{D1}}{S_{DS}}; \quad T_B = \frac{S_{D1}}{S_{DS}} \quad (3.6)$$

Limit period for plastic deformation will be assumed as $T_L = 6$ sec. (TBDY, Earthquake Code for Buildings, 2018).

Using the formulations above, elastic design response spectra without local soil condition factors for DD-1 and DD-2 earthquake levels are calculated for Halkapınar district of Konak sub-province of İzmir city.

For the project site and DD-1 earthquake level, spectral accelerations for short period is $SS = 2.150$; spectral acceleration for 1 sec period is $S1 = 0.550$. Local site condition factors for DD-1 earthquake level. Using the calculated parameters, horizontal elastic design spectrum is given in Figure 3.7. Also, peak ground acceleration (PGA) of the site is given as $0.853g$ and peak ground velocity (PGV) of the site is 53.389 cm/sec.

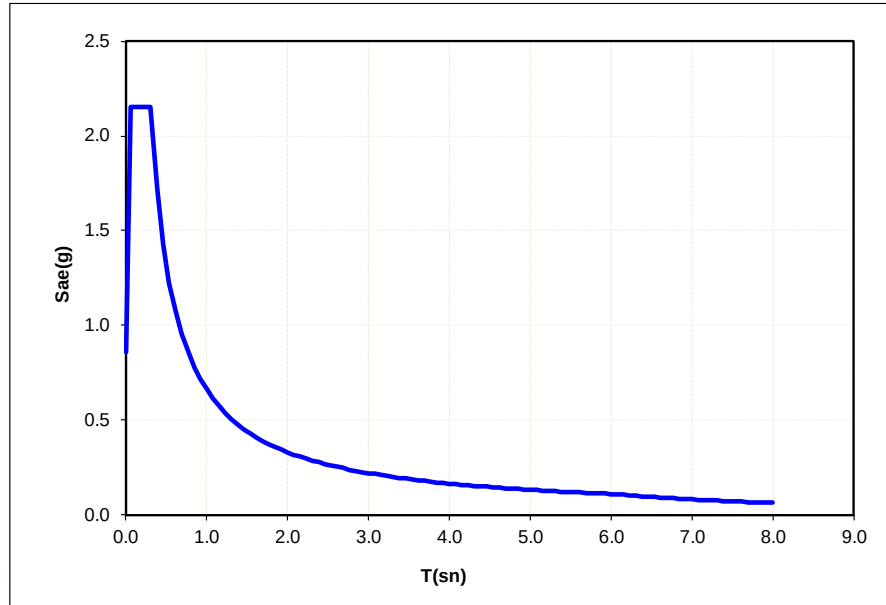


Figure 3.7 Horizontal elastic design response spectrum for DD-1 earthquake level

For the project site and DD-2 earthquake level, spectral accelerations for short period is $S_s = 1.124$; spectral acceleration for 1 sec period is $S_1 = 0.276$. Local site condition factors for DD-2 earthquake level. Using the calculated parameters, horizontal elastic design spectrum is given in Figure 3.8. Also, peak ground acceleration (PGA) of the site is given as 0.458g and peak ground velocity (PGV) of the site is 27.911 cm/sec.

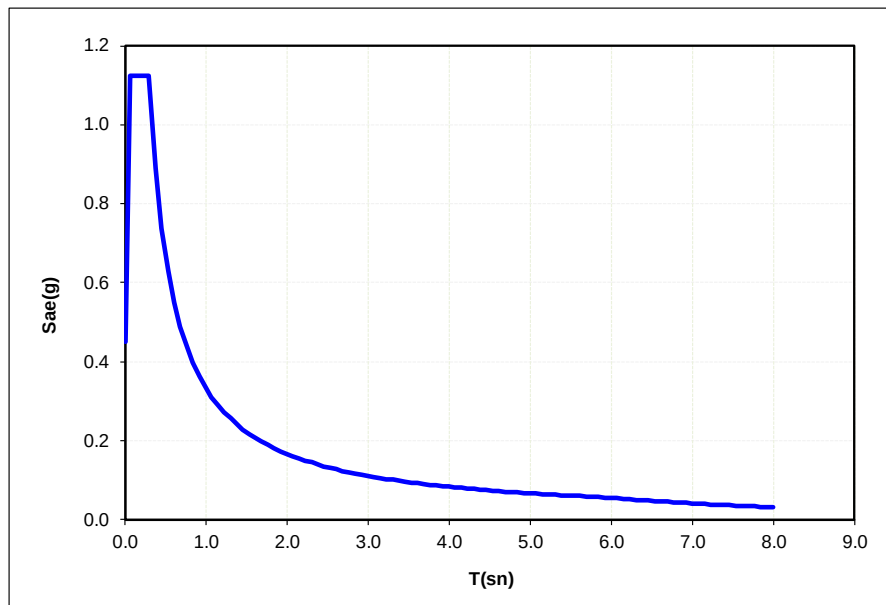


Figure 3.8 Horizontal elastic design response spectrum for DD-2 earthquake level

3.4 Selection and Scaling of Earthquake Record

In this study, eleven earthquake records were used in ground response analyses. These records are selected from PEER Ground Motion Database (PEER, 2006) based on the İzmir's seismicity, soil condition, fault type, earthquake magnitude, distance between site and rupture zone, shear wave velocity, duration of earthquake.

It is important to select real earthquakes that are appropriate to the project location. Appropriateness of the selected earthquakes depends on the fault type, depth, length and corresponding earthquake magnitude. Using the approach developed by Wells and Coppersmith (1994), earthquake magnitude that may occur in İzmir is calculated as between 6.0 and 8.0. Corresponding earthquake levels are DD-1 and DD-2. Active faults in İzmir are usually strike slip and normal faults; therefore, earthquakes that are generated by these types of faults are selected. Bedrock level of the project location is selected as ZB based on TBDY classification. According to Turkish Earthquake Code (2018), shear wave velocity of the ZB type soils is between 760 and 1500 m/s. During the selection of earthquakes, bedrock levels of the earthquakes are kept in this range, with few exceptions.

Unscaled raw data of the selected earthquakes are presented below with short summaries. PEER presents earthquake records' horizontal and vertical components separately. Since horizontal components are studied in this thesis, only horizontal components are used.

1979 Coyote Lake earthquake occurred in Santa Clara, California, near Coyote river. Earthquake emerged from strike slip San Andreas fault and magnitude of earthquake is 5.7 Mw. Earthquake's acceleration-time record is given in Figure 3.9.

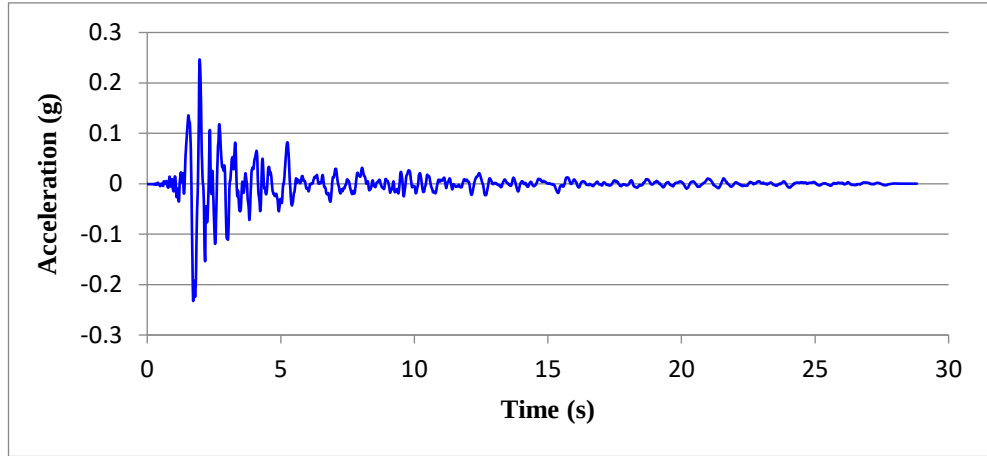


Figure 3.9 Unscaled acceleration time graph of Coyote Lake Earthquake

1979 Norcia earthquake occurred in Norcia, Italy. Earthquake emerged from a normal fault and magnitude of earthquake is 5.9 Mw. Earthquake's acceleration-time record is given in Figure 3.10.

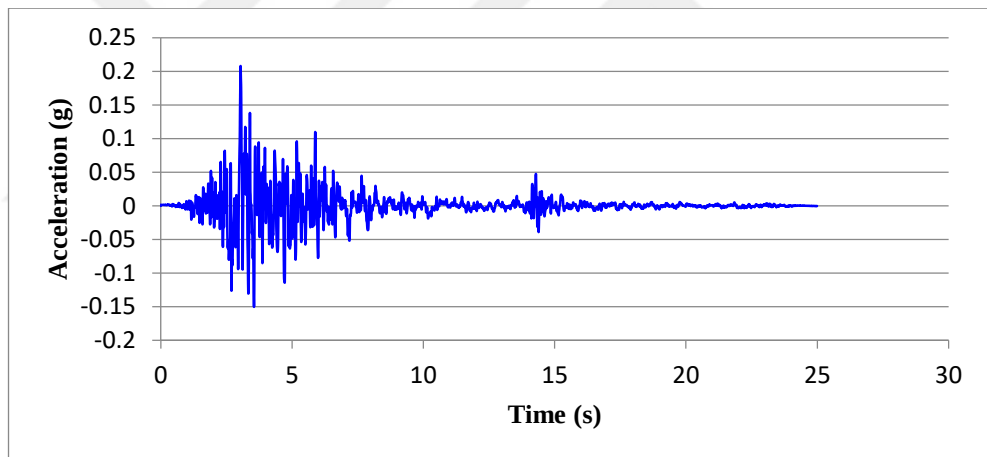


Figure 3.10 Unscaled acceleration time graph of Norcia Italy Earthquake

1979 Irpinia earthquake occurred in Conza village in the south of Italy. Earthquake emerged from normal fault. Records from two different stations are used in this study. Record from Cascia station is named as Irpinia Italy-01, while record from Brienza station is named as Irpinia Italy-02. Acceleration-time records of both record are presented in Figure 3.11 and Figure 3.12.

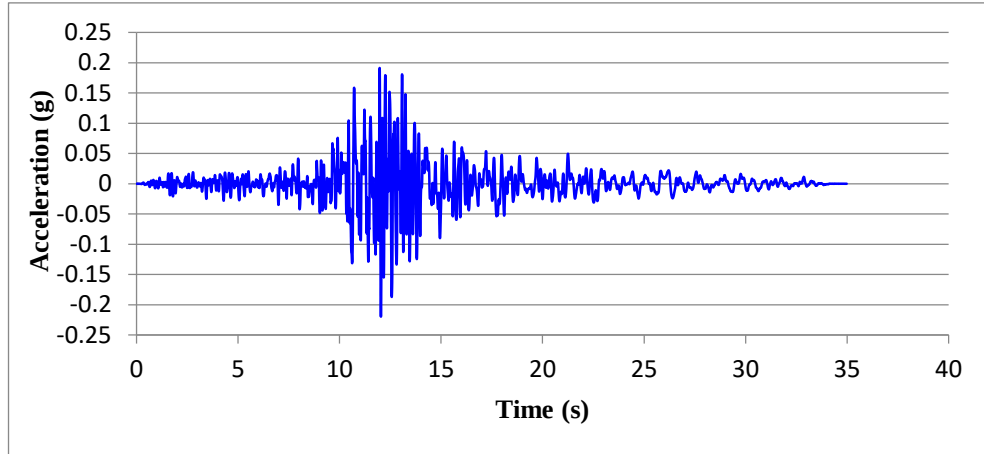


Figure 3.11 Unscaled acceleration time graph of Irpinia Italy-01 (Cascia station) Earthquake

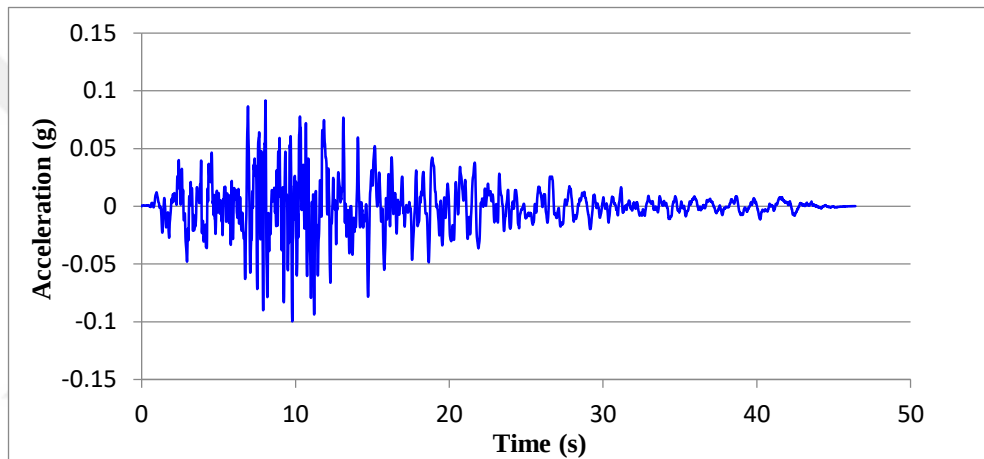


Figure 3.12 Unscaled acceleration time graph of Irpinia Italy-02 (Brienza Station) Earthquake

1979 Morgan Hill earthquake occurred in Santa Clara Valley located in the north of California. Earthquake emerged from strike slip San Andreas fault and magnitude of earthquake is 6.2 Mw. Earthquake's acceleration-time record is given in Figure 3.13.

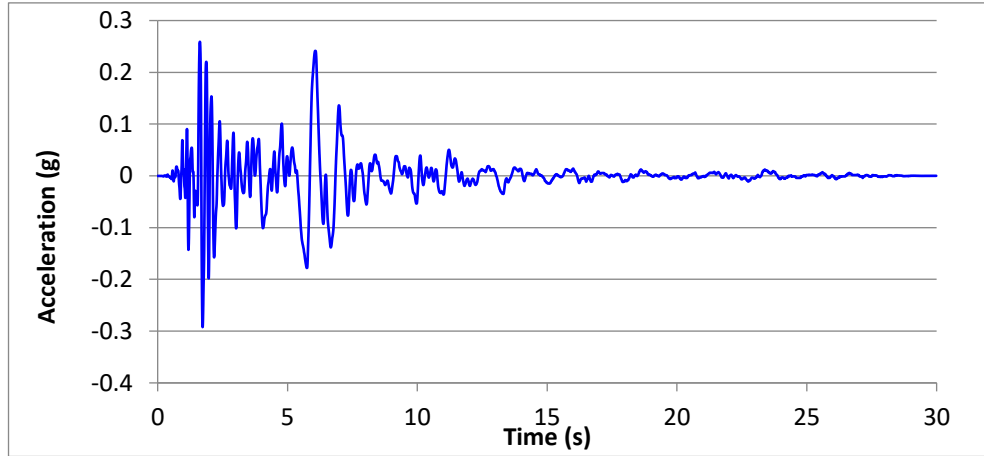


Figure 3.13 Unscaled acceleration time graph of Morgan Hill Earthquake

1995 Kobe Japan earthquake occurred in Kobe, Japan. Earthquake emerged from a strike slip fault and magnitude of earthquake is 6.9 Mw. Earthquake's acceleration-time record is given in Figure 3.14.

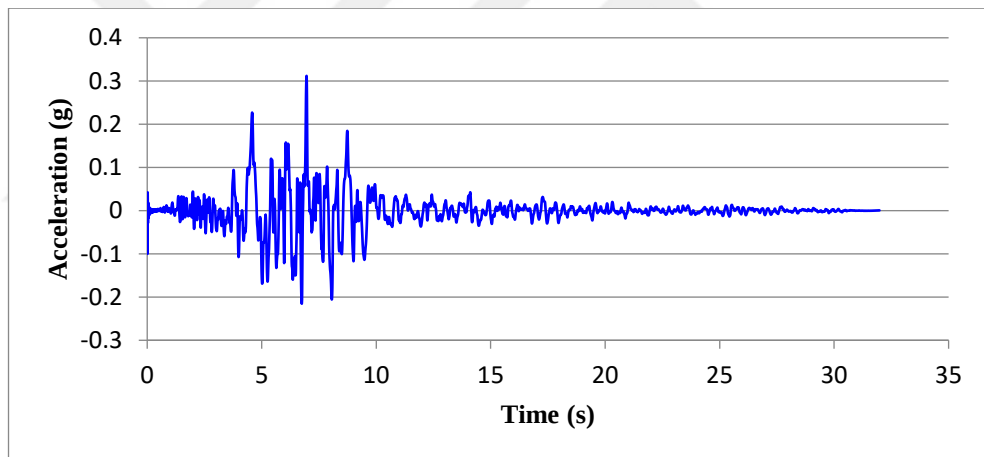


Figure 3.14 Unscaled acceleration time graph of Kobe Japan Earthquake

1999 Düzce earthquake occurred in Düzce, Turkey. Earthquake emerged from a strike slip fault and magnitude of earthquake is 7.2 Mw. Earthquake's acceleration time record is given in Figure 3.15.

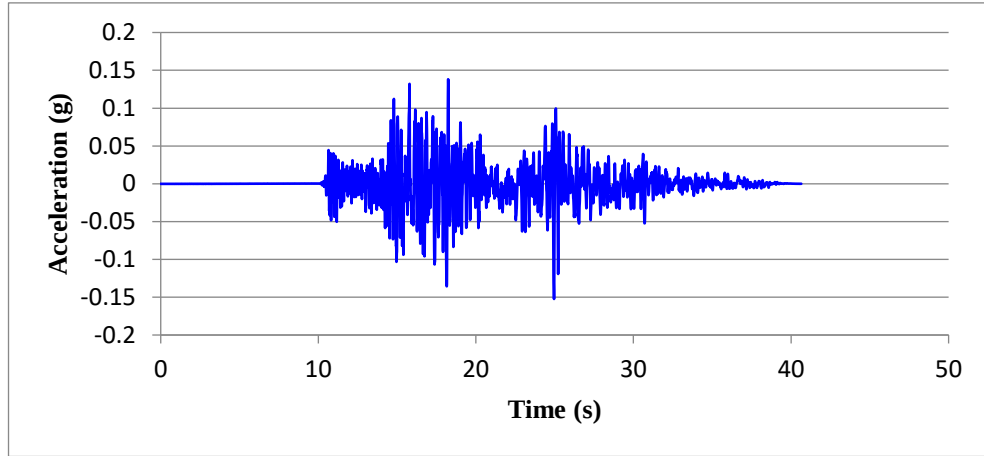


Figure 3.15 Unscaled acceleration time graph of Duzce Earthquake

1999 Hector Mine earthquake occurred in Barstow, California. Earthquake emerged from a strike slip fault and magnitude of earthquake is 7.1 Mw. Earthquake's acceleration time record is given in Figure 3.16.

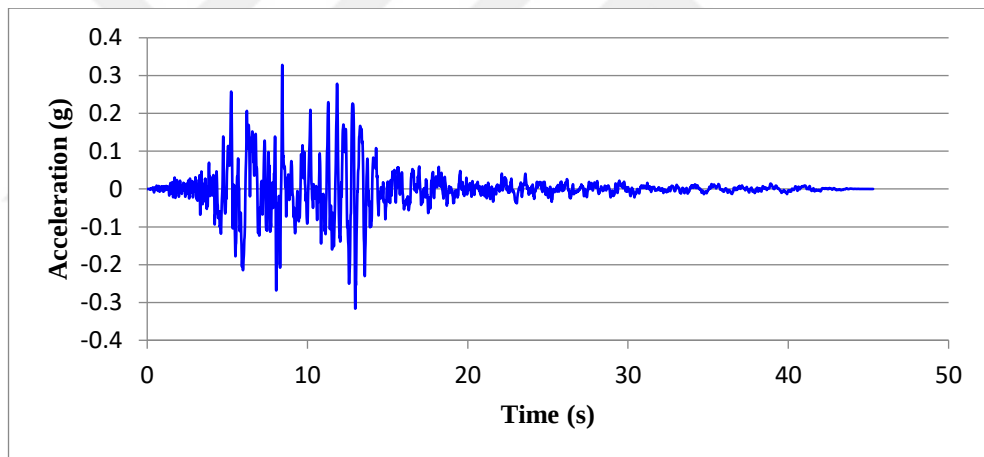


Figure 3.16 Unscaled acceleration time graph of Hector Mine Earthquake

2000 Tottori Japan earthquake occurred near Yanago and Matsue, Japan. Earthquake emerged from a strike slip fault and magnitude of earthquake is 6.6 Mw. Earthquake's acceleration time record is given in Figure 3.17.

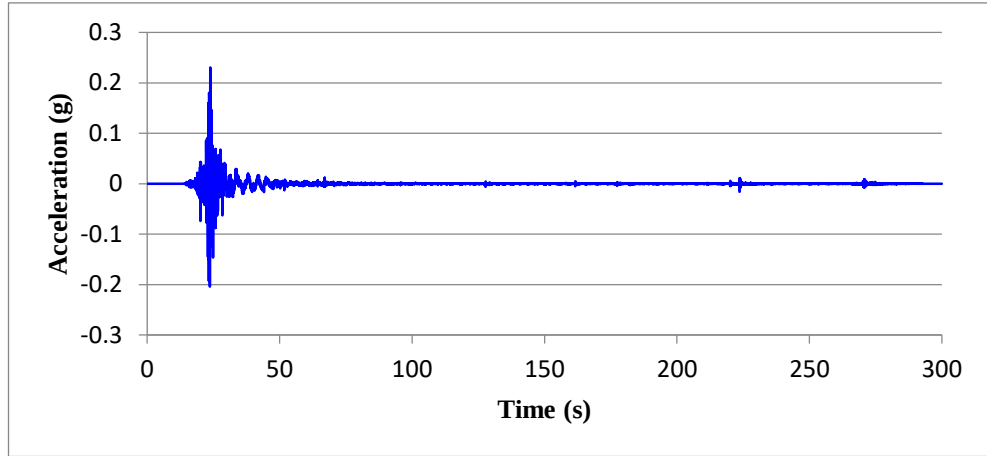


Figure 3.17 Unscaled acceleration time graph of Tottori Japan Earthquake

1999 Parkfield earthquake occurred in Parkfield, California. Earthquake emerged from a strike slip fault and magnitude of earthquake is 6.0 Mw. Earthquake's acceleration time record is given in Figure 3.18.

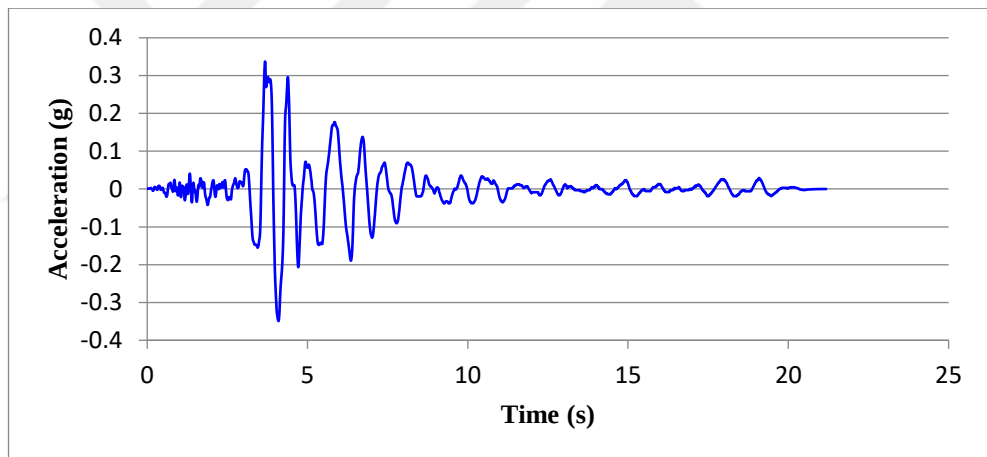


Figure 3.18 Unscaled acceleration time graph of Parkfield Earthquake

2009 L'Aquila earthquake occurred in L'Aquila in the south of Italy. Earthquake emerged from a normal fault and magnitude of earthquake is 6.3 Mw. Earthquake's acceleration time record is given in Figure 3.19.

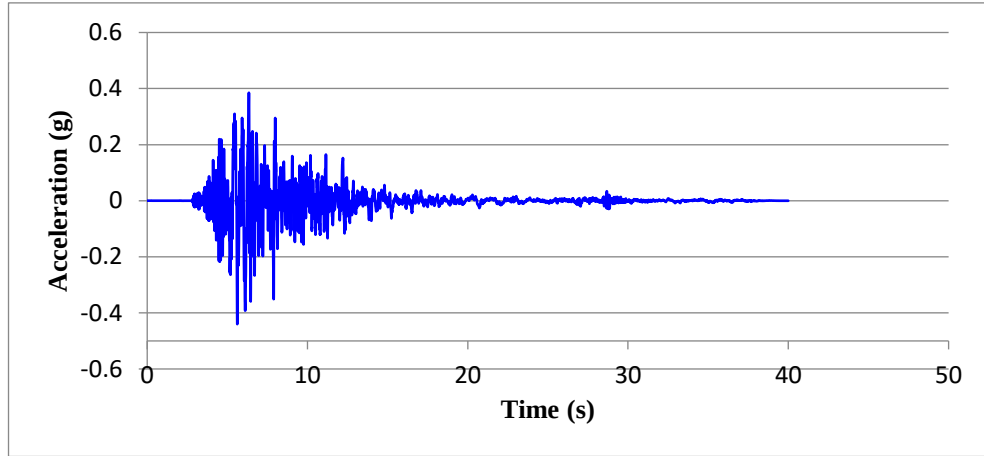


Figure 3.19 Unscaled acceleration time graph of L'Aquila Earthquake

Earthquakes are scaled based on the design acceleration spectra obtained from TBDY using least squares methods in time domain. Scaling in time domain does not affect the frequency content of the ground motion. Properties of the selected earthquakes and scaling factors for DD-1 and DD-2 earthquake levels are presented in Table 3.4 and Table 3.5, respectively. Selected earthquakes' amplitudes are multiplied by scaling factors to obtain scaled earthquakes.

Table 3.4 Selected earthquake records and scale factors for soil site class, ZB and earthquake ground motion level, DD-1 (PEER, 2006)

Result ID	Spectral Ordinate	Record Sequence Number	Earthquake Name	Year	Fault Type	R _{jb} (km)	R _{rup} (km)	Magnitude	V _{s30} (m/sec)	Scale Factor	Mean Squared Error
1	SRSS	145	"Coyote Lake"	1979	Strike Slip	5.3	6.13	5.74	561.43	4.8369	0.9099
2	SRSS	156	"Norcia_ Italy"	1979	Normal	1.41	4.64	5.9	585.04	5.1759	0.4611
3	SRSS	288	"Irpinia_ Italy-01"	1980	Normal	22.54	22.56	6.9	561.04	3.8348	0.4768
4	SRSS	302	"Irpinia_ Italy-02"	1980	Normal	22.68	22.69	6.2	574.88	4.2377	0.2125
5	SRSS	459	"Morgan Hill"	1984	Strike Slip	9.85	9.87	6.19	663.31	2.4825	0.6544
6	SRSS	1108	"Kobe_ Japan"	1995	Strike Slip	0.9	0.92	6.9	1043	1.5131	0.4562
7	SRSS	1612	"Duzce_ Turkey"	1999	Strike Slip	4.17	4.17	7.14	551.3	3.9782	0.109
8	SRSS	1787	"Hector Mine"	1999	Strike Slip	10.35	11.66	7.13	726	1.4919	0.171
9	SRSS	3954	"Tottori_ Japan"	2000	Strike Slip	15.58	15.59	6.61	967.27	2.4184	0.0788
10	SRSS	4097	"Parkfield-02_ CA"	2004	Strike Slip	1.6	2.99	6	648.09	1.9589	0.5908
11	SRSS	4482	"L'Aquila_ Italy"	2009	Normal	0	6.55	6.3	552	2.1663	0.6698

Table 3.5 Selected earthquake records and scale factors for soil site class, ZB and earthquake ground motion level, DD-2 (PEER, 2006)

Result ID	Spectral Ordinate	Record Sequence Number	Earthquake Name	Year	Fault Type	R _{jb} (km)	R _{rup} (km)	Magnitude	V _{s30} (m/sec)	Scale Factor	Mean Squared Error
1	SRSS	145	"Coyote Lake"	1979	Strike Slip	5.3	6.13	5.74	561.43	2.4456	0.9066
2	SRSS	156	"Norcia_ Italy"	1979	Normal	1.41	4.64	5.9	585.04	2.617	0.4534
3	SRSS	288	"Irpinia_ Italy-01"	1980	Normal	22.54	22.56	6.9	561.04	1.9389	0.468
4	SRSS	302	"Irpinia_ Italy-02"	1980	Normal	22.68	22.69	6.2	574.88	2.1426	0.217
5	SRSS	459	"Morgan Hill"	1984	Strike Slip	9.85	9.87	6.19	663.31	1.2552	0.6492
6	SRSS	1108	"Kobe_ Japan"	1995	Strike Slip	0.9	0.92	6.9	1043	0.765	0.4653
7	SRSS	1612	"Duzce_ Turkey"	1999	Strike Slip	4.17	4.17	7.14	551.3	2.0115	0.1069
8	SRSS	1787	"Hector Mine"	1999	Strike Slip	10.35	11.66	7.13	726	0.7543	0.1779
9	SRSS	3954	"Tottori_ Japan"	2000	Strike Slip	15.58	15.59	6.61	967.27	1.2228	0.0789
10	SRSS	4097	"Parkfield-02_ CA"	2004	Strike Slip	1.6	2.99	6	648.09	0.9905	0.6
11	SRSS	4482	"L'Aquila_ Italy"	2009	Normal	0	6.55	6.3	552	1.0953	0.6637

Selected earthquakes after applying scale factors given in Table 3.4 and DD-1 target spectrum are shown in Figure 3.20.

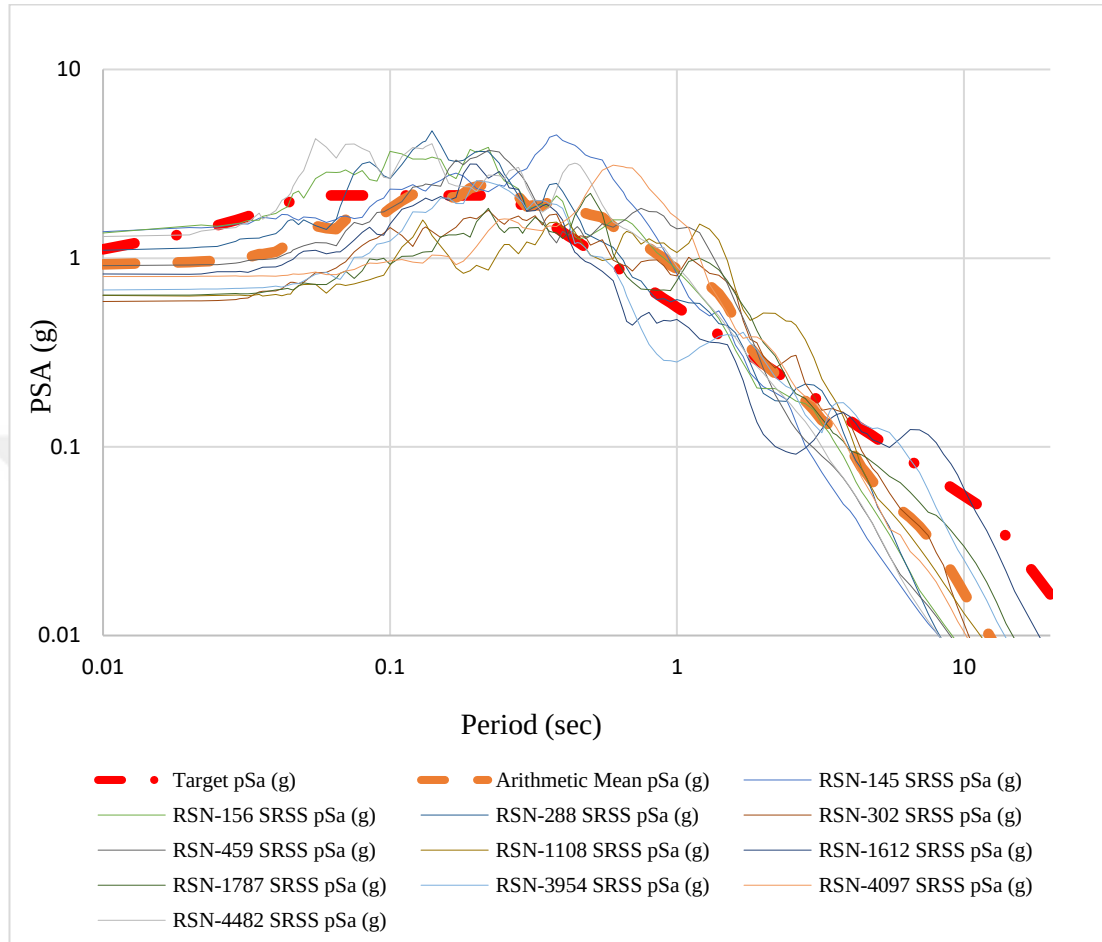


Figure 3.20 Scaled arithmetic mean PSA based on Target PSA of DD-1 earthquake level (Log Scale)

Selected earthquakes after applying scale factors given in Table 3.5 and DD-2 target spectrum are shown in Figure 3.21.

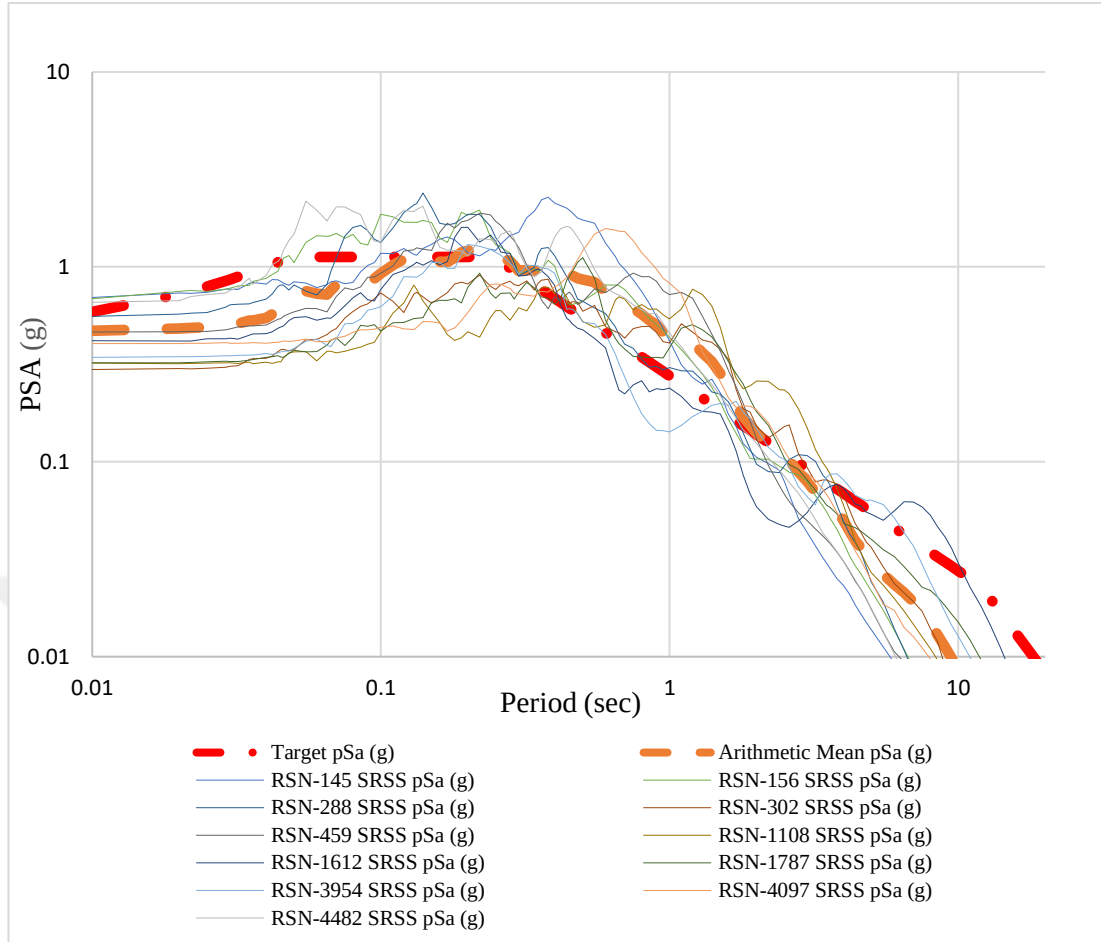


Figure 3.21 Scaled arithmetic mean PSA based on Target PSA of DD-2 earthquake level (Log Scale)

As can be seen in the Figure 3.20 and Figure 3.21, arithmetic mean of the selected scaled earthquakes and target spectral accelerations are very close, therefore acceptable.

To define a strong ground motion; amplitude, frequency content and duration of motion are enough. Amplitude parameters of selected earthquakes (PGA), frequency content (T_p , predominant period) and duration of motion of selected earthquakes are given in Table 3.6. Also, PGAs after scaling are given in this table.

Table 3.6 The amplitude parameter, frequency content parameters and duration of selected earthquake motions

Result ID	PEER #	5-95% Duration (sec)	Arias Intensity (m/sec)	T ₀ (sec)	T ₁ (sec)	T ₂ (sec)	PGA (Unscaled)	PGA (Scaled) DD1	PGA (Scaled) DD2
1	145	8.5	0.4	0.367	0.100	0.253	0.25	1.21	0.61
2	156	5.7	0.3	0.197	0.120	0.106	0.21	1.09	0.55
3	288	13.2	0.5	0.145	0.224	0.083	0.22	0.84	0.43
4	302	22.9	0.3	0.325	0.224	0.100	0.1	0.42	0.21
5	459	7.3	0.9	0.224	0.878	0.568	0.29	0.72	0.36
6	1108	7	1.2	0.416	0.128	0.775	0.31	0.47	0.24
7	1612	15	0.4	0.238	0.128	0.345	0.15	0.60	0.30
8	1787	11.7	1.9	0.502	0.210	0.325	0.33	0.49	0.25
9	3954	12.8	0.5	0.210	0.345	0.164	0.23	0.56	0.28
10	4097	6.4	1.5	0.643	0.253	1.057	0.35	0.69	0.35
11	4482	7.6	1.7	0.057	0.443	0.269	0.44	0.95	0.48

T₀ is predominant period, T₁ and T₂ are respectively second and third period at the earthquake.

3.5 Location of the Study Area

Project location is in Halkapınar district of Konak sub-province of İzmir city. İzmir is located in the west of Turkey. City is in an active tectonic zone. Due to rivers located nearby, İzmir Bay's upper soil levels are composed of alluvial material. Due to proximity to active faults, seismicity of the city is high. Project location is shown in Figure 3.22.



Figure 3.22 Area of investigation location (Google earth, 2018)

3.6 Soil Profile of the Project Site

To understand the ground conditions, geological and geophysical investigations are performed in the project site. Also, to determine the physical and mechanical parameters of the soil, site and laboratory experiments are performed. Laboratory tests are also performed on disturbed and undisturbed soil samples obtained from the site during boreholes. SK-18 borehole is 240m deep, hence SK-18 borehole is used in analyses. Consistency with other boreholes are also checked.

3.7 Standard Penetration Test (SPT) Results

Standard Penetration Test (SPT) gives a rough idea about the rigidity of the soils and layer boundaries. Moreover, many researchers have correlated geotechnical parameters with SPT values. Soil's dynamic and static parameters are correlated with SPT values with promising results. Using the disturbed samples obtained from SPT tubes, laboratory tests are performed to understand physical and mechanical properties of the soil. In this study, in case of insufficiency of laboratory data, soil's dynamic and static parameters will be determined using widely used correlations in the literature.

Standard Penetration Tests are performed down to 60m depth in SK-18 borehole. Based on the SPT data, soil's strength and stiffness parameters were determined based on the many available correlations presented in the literature. SPT values obtained

from the project site can be used in foundation design with certain adjustments. Properties of the borehole that is considered for design is given in Table 3.7.

Table 3.7 Borehole properties

Borehole	X coordinate	Y coordinate	GWL (m)	Total Depth (m)
SK-18	4255615990	51430118	1.5	240

SPT results depend on many variables such as drilling equipment, values are used after certain corrections applied to raw data. Corrections can be listed as: energy correction (C_E), borehole diameter correction (C_B), rod length correction (C_R), liner correction (C_S) and overburden correction (C_N).

Energy correction factor, C_E (%) is standardized to 60% energy transfer rate, and therefore other types of systems that deliver energy different than 60% are corrected to standard energy. Since drilling machine that has been used in this project is full automatic, energy ratio is assumed as 70% and resulting energy correction is 1.17.

Borehole diameter between 60 and 120mm requires no correction (1.0) for borehole diameter. Rod length correction, C_R depends on the depth. For borehole depth smaller than 4m, $C_R=0.75$, for 4m-6m $C_R=0.85$, for 6m-10m $C_R=0.95$ and for depths deeper than 10m, it is 1.0 (Bowles, 1996). Liner correction, C_S , is recommended as 1.0 for samplers without liners by Bowles (1996). Lastly, overburden correction is performed for granular soils and calculated by the correlation proposed by Peck et al. (1974). To correct the raw SPT data (given in Table 3.8), N following correlations are used.

$$N_{60} = N * C_E * C_B * C_R * C_S \quad (3.7)$$

$$N_{1,60} = N * C_E * C_B * C_R * C_S * C_N \quad (3.8)$$

Table 3.8 Standard Penetration Test Result (N) and Corrected SPT- N_{60} and $N_{1,60}$ values

Depth (m)	SPT- N_{30}	SPT- N_{60}	SPT- $(N_1)_{60}$
1.8	6	5	7
3.3	9	8	10
4.8	16	16	19
6.3	29	32	36
7.8	24	27	29

Table 3.8 continues

Depth	SPT-N₃₀	SPT-N₆₀	SPT-(N₁)₆₀
(m)			
9.45	20	22	22
10.8	21	25	25
12.3	18	21	20
13.8	16	19	17
15.3	27	32	28
16.55	21	25	21
18.3	18	21	17
20.3	23	27	21
21.3	29	34	27
22.8	29	34	26
24.3	38	44	33
25.8	23	27	19
27.35	18	21	15
28.8	12	14	10
30.3	18	21	14
31.8	11	13	9
33.3	10	12	8
34.8	9	11	7
36.3	9	11	7
37.8	15	18	11
39.3	17	20	12
41.8	41	48	27
42.3	32	37	21
43.8	33	39	22
45.3	30	35	19
46.8	29	34	18
49.8	28	33	17
51.3	31	36	18
52.8	34	40	20
54.3	25	29	14
55.8	27	32	15
57.3	31	36	17
59.3	33	39	18
90	50	59	19

Figure 3.23 shows the raw SPT data with depth and Figure 3.24 presents the corrected data.

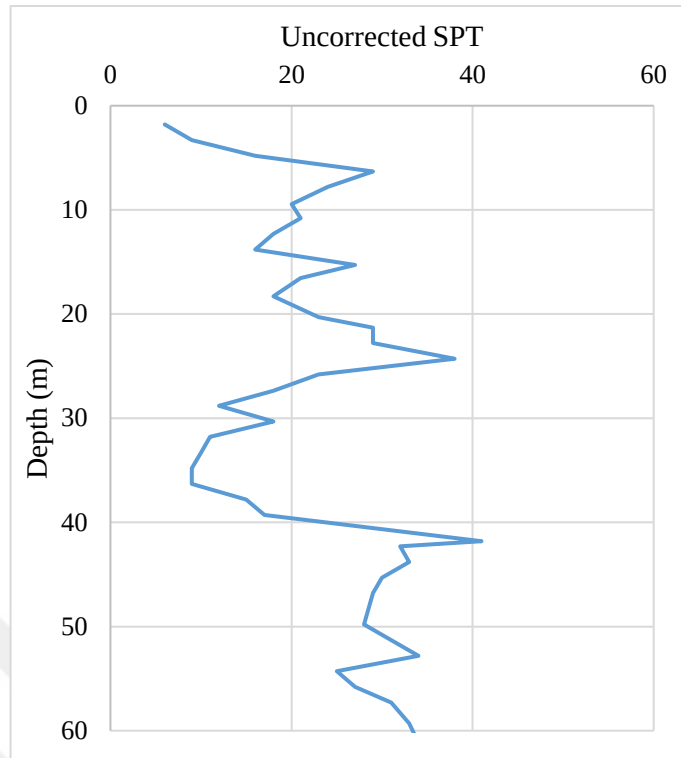


Figure 3.23 (Uncorrected) N-Depth relationship

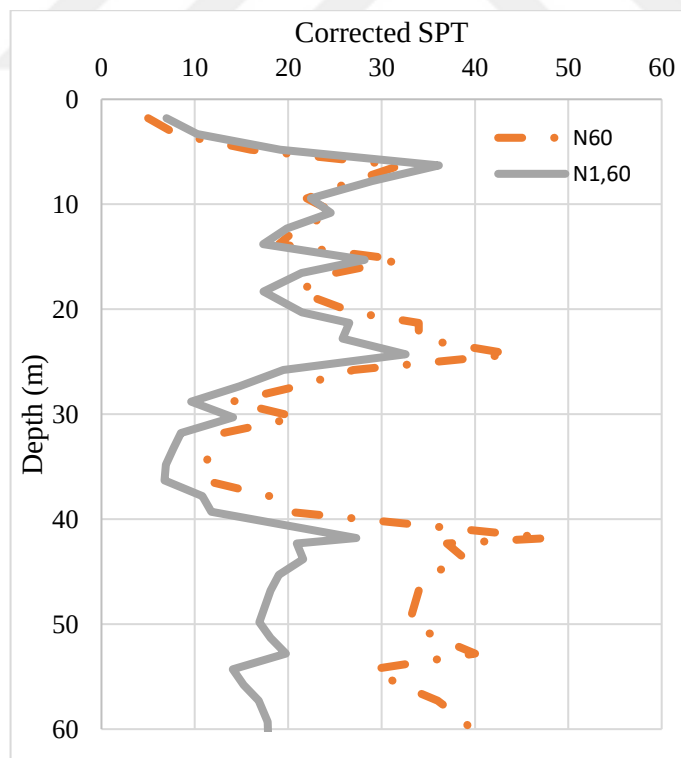


Figure 3.24 $N_{60} / N_{1,60}$ -Depth relationship

3.8 Laboratory Test Results

Laboratory experiments were performed on disturbed and undisturbed samples to determine the lithology and geotechnical properties of the soil layers. Wet sieve analysis, natural water content measurements, Atterberg limit tests, natural unit weight, consolidation, triaxial and uniaxial tests are performed on the samples, where applicable. Results of the laboratory experiments show that project site consists of alluvial soils such as CL, CH, SC and GC which are clay with low and high plasticity, clayey sand and clayey gravel, respectively. Results of the simple laboratory experiments are summarized Table 3.9.

Table 3.9 Sieve analysis test results, Atterberg consistency limit test results and soil classification

Depth (m)	+ #10	- #200	LL (%)	PL (%)	PI (%)	W _n (%)	USCS
1.50-1.95	3	77	38	11	27	25.8	CL
3.00-3.45	70	10		NP		10.5	GP-GM
4.50-4.95	59	10		NP		11.4	GW-GM
6.00-6.45	19	33		NP		15	SM
7.50-7.95	1	86		NP		31.8	ML
9.00-9.45	20	57	40	20	20	25.6	CL
10.50-10.95	11	68	40	20	22	30.5	CL
12.00-12.45	0	83	40	20	20	24.9	CL
13.50-13.95	4	86	41	20	21	27.7	CL
15.00-15.45	27	49	32	18	14	17.1	GC
15.50-16.00	0	86	39	21	18	27.8	CL
18.50-18.95	0	88	40	19	21	28.1	CL
19.50-20.00	0	90	40	20	20	26.5	CL
20.00-20.45	6	87	39	21	18	28.3	CL
21.00-21.45	41	36	33	20	13	11.4	GC
22.50-22.95	37	37	32	20	12	15.5	GC
24.00-24.45	35	22	33	19	14	17.5	SC
25.00-25.50	5	92	37	19	18	25.6	CL
25.50-25.95	37	39	32	18	14	14.3	GC
26.50-27.00	8	90	36	19	17	23.1	CL
27.00-27.45	19	38	33	19	14	15.1	SC
28.50-28.95	0	85	44	20	24	23.9	CL
29.50-30.00	0	81	55	25	30	35.9	CH
30.00-30.45	2	85	58	26	32	38.7	CH
31.50-31.95	0	94	58	28	30	36.7	CH
33.00-33.45	0	97	56	26	30	39.5	CH

Table 3.9 continues

Depth (m)	+#10	-#200	LL (%)	PL (%)	PI (%)	W _n (%)	USCS
33.50-34.00	0	96	57	26	31	38.7	CH
34.50-34.95	5	87	55	25	30	40.5	CH
36.00-36.45	1	96	38	20	18	25.7	CL
37.50-37.95	3	89	39	21	18	25.3	CL
39.00-39.45	2	91	40	20	20	13.2	CL
40.50-40.95	36	40	30	18	12	14.7	GC
42.00-42.45	28	40	32	17	15	14.9	SC
43.50-43.95	15	41	32	17	15	11.4	SC
45.00-45.45	16	68	38	20	18	21.5	CL
46.50-46.95	8	84	38	20	18	24	CL
47.00-48.00	6	76	37	20	17	23.6	CL
48.00-48.50	0	98	39	19	20	24	CL
48.50-48.95	34	48	31	19	12	15	GC
49.50-49.61	48	32	30	20	10	12.7	GC
51.00-51.45	32	36	32	18	14	21	SC
52.50-52.95	11	14	29	19	10	17.1	SC
54.00-54.45	11	45	33	18	15	9	SC
55.50-55.95	1	89	40	20	20	22.1	CL
56.00-57.00	0	86	40	20	20	26.4	CL
57.50-57.95	1	92	39	19	20	26.8	CL
60.00-61.00	0	94	38	20	18	27.3	CL
68.50-69.00	7	53	36	19	17	28	CL
78.00-79.00	8	64	37	19	18	28	CL
90.00-91.00	0	80	37	19	18	24.7	CL
108.00-110.00	57	26	30	19	11	11.8	GC
115.00-116.00	6	64	36	21	15	20.5	CL
119.00-120.00	33	34	33	16	17	8.4	SC
124.00-125.00	81	7				4	GP-GC
132.00-133.00	25	46	37	20	17	17.9	CL
140.00-141.00	1	71	38	20	18	19.8	CL
151.00-152.00	1	83	38	20	18	19.1	CL
155.00-156.00	42	30	33	18	15	12.2	GC
160.00-161.00	20	61	35	19	16	24.7	CL
172.00-173.00	1	86	37	19	18	12.66	CL
173.00-174.00	3	80	37	19	18	12.73	CL
178.00-179.00	21	65	35	20	15	11.88	CL
184.00-185.00	2	89	38	19	19	15.41	CL
190.00-191.00	1	82	39	19	20	13.57	CL
196.00-197.00	2	95	40	20	20	18.3	CL
204.00-205.00	0	93	40	19	21	14.36	CL
207.00-208.00	3	90	40	20	20	20.9	CL

Table 3.9 continues

Depth (m)	+#10	-#200	LL (%)	PL (%)	PI (%)	W _n (%)	USCS
213.00-214.00	13	82	38	18	20	11.3	CL
217.00-218.00	4	90	41	21	20	23.53	CH
223.00-224.00	3	95	52	24	28	13.92	CH
227.00-228.00	8	91	55	25	30	18.72	CH
231.00-232.00	0	95	53	23	30	19.8	CH
236.00-237.00	2	97	54	24	30	14.09	CH
239.00-240.00	3	92	52	23	29	20.73	CH

Uniaxial compressive tests, triaxial compressive tests and consolidation tests are performed on undisturbed samples, and results are presented in Table 3.10, Table 3.11 and Table 3.12.

Table 3.10 Unconfined compression test result

Borehole	Sample	Depth (m)	Unconfined Compressive Strength, q_u (kg/cm ²)	Density (gr/cm ³)	Soil Class
SK-18	Core	15.50-16.00	0.78	1.89	CL
SK-18	Core	25.00-25.50	1.84	1.98	CL
SK-18	Core	29.50-30.00	1.34	2.02	CH
SK-18	Core	33.50-34.00	1.70	2.02	CH
SK-18	Core	56.00-57.00	2.18	2.03	CL
SK-18	Core	60.00-61.00	2.3	2.04	CL
SK-18	Core	68.50-69.00	2.4	2.04	CL
SK-18	Core	78.00-79.00	2.6	2.05	CL
SK-18	Core	90.00-91.00	3.15	2.06	CL
SK-18	Core	115.00-116.00	2.65	2.06	CL
SK-18	Core	140.00-141.00	3.56	2.07	CL
SK-18	Core	151.00-152.00	2.89	2.06	CL
SK-18	Core	160.00-161.00	5.44	2.09	CL
SK-18	Core	172.00-173.00	9.57	2.11	CL
SK-18	Core	178.00-179.00	6.72	2.11	CL
SK-18	Core	184.00-185.00	7.08	2.15	CL
SK-18	Core	187.00-188.00	16.6	2.18	Claystone
SK-18	Core	190.00-191.00	6.73	2.11	CL

Table 3.10 continues

Borehole	Sample	Depth (m)	Unconfined Compressive Strength, q_u (kg/cm ²)	Density (gr/cm ³)	Soil Class
SK-18	Core	196.00-197.00	6.61	2.12	CL
SK-18	Core	204.00-205.00	6.41	2.1	CL
SK-18	Core	207.00-208.00	6.65	2.12	CL
SK-18	Core	213.00-214.00	7.5	2.16	CL
SK-18	Core	217.00-218.00	6.44	2.14	CH
SK-18	Core	223.00-224.00	4.59	2.15	CH
SK-18	Core	227.00-228.00	4.31	2.15	CH
SK-18	Core	231.00-232.00	4.34	2.16	CH
SK-18	Core	236.00-237.00	4.43	2.14	CH
SK-18	Core	239.00-240.00	4.59	2.15	CH

Table 3.11 Triaxial compression test result

Borehole	Sample	Depth (m)	ϕ (°)	c (kg/cm ²)	Density (gr/cm ³)	Soil Class
SK-18	UD	19.50-20.00	7	0.60	1.93	CL
SK-18	Core	26.50-27.00	11	0.90	2.00	CL
SK-18	Core	47.00-48.00	12	1.06	2.03	CL
SK-18	UD	48.00-48.50	11	1.10	2.04	CL
SK-18	Core	132.00-133.00	13	1.10	2.10	CL

Table 3.12 Consolidation test result

Borehole	Sample	Depth (m)	Natural Void Ratio	Other Void Ratio
SK-18	UD	19.50-20.00	0.828	0.707
SK-18	UD	48.00-48.50	0.796	0.527
SK-18	UD	90.00-91.00	0.618	0.536

Coefficients of consolidation obtained from oedometer tests performed on clayey soils are presented in Table 3.13. For clays in site, average coefficient of consolidation is determined as $1.5 \times 10^{-8} \text{ m}^2/\text{sec}$. Coefficient of consolidation is used in nonlinear effective stress analyses in DEEPSOIL. Also, by comparing the overburden stress with pre-consolidation stress, it is determined that clays in the site are normally consolidated.

Table 3.13 Coefficient of consolidation and description of over consolidation ratio

Borehole No	Depth (m)	Effective Stress (kPa)	Soil Class	c_v (m^2/s)	OCR
SK18	20	193.5	CL	4.16667E-09	OCR<1; NC
	48.5	450	CL	2.75E-08	OCR<1; NC

3.9 Geophysical Test Results

To determine dynamic parameters and determine bedrock levels, geophysical tests are performed. Also, shear wave profile of the site is determined. Using the shear wave velocities, dynamic parameters of the site are calculated. Following geophysical experiments are performed in the project site:

- MASW (Multichannel Analysis of Surface Waves) Methods
- Downhole Methods (Borehole Seismic Method)
- SPAC (Spatial Auto-Correlation) Methods

S-wave in the first 50m is determined using MASW method. In this method, using 10 different geometric layout, MASW records were taken. Layout of MASWs are shown in Figure 3.25. In this figure, recordings are separated as 1. Alan (Area-1) and 2. Alan (Area-2). In this study; P6, P7, P8, P9 and P10 were considered which are readings in Area-2. Moreover, due to proximity to Area-2, P5 is also considered. Shear wave profile for first 50m is determined using average of recordings from P5 to P10 and resultant average is presented in Figure 3.26.

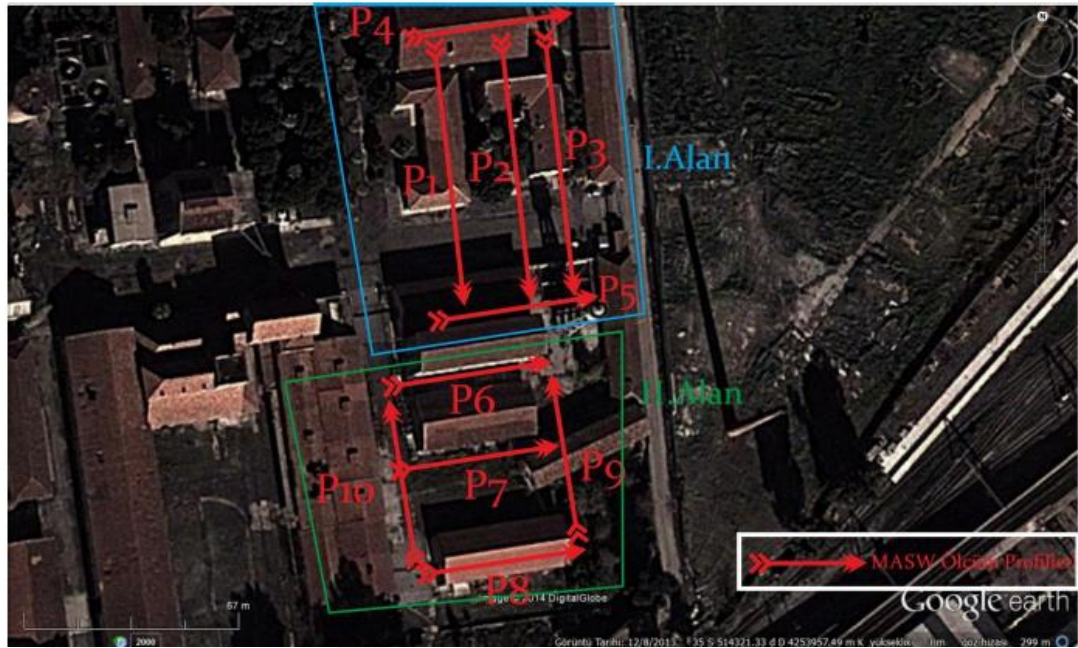


Figure 3.25 MASW (multichannel analysis of surface waves) methods (Google earth, 2014)

Second method to determine the depth vs. shear wave velocity relationship is downhole method which is one of borehole seismic tests. Downhole test is performed using receiver put inside previously drilled borehole and source at the surface. By changing the depth of the receiver, data from any depth can be obtained. Downhole test performed in SK-18 (which is the same borehole that SPTs are considered). Location of the borehole is shown in Figure 3.27. In this study, shear wave velocities down to 80m depth are obtained using downhole method and presented in Figure 3.28. Results was investigated and since 300 m/s at the first 10m is misleading, it was not included in the analyses. To ease the calculations, the raw form of the shear wave velocity profile was divided into three zones and formulated. Zones are divided as: Zone-1 from 0-25m, Zone-2 from 25-50m and Zone-3 from 50-80m. Equations and corrected shear wave velocity profile are presented below in Figure 3.29.

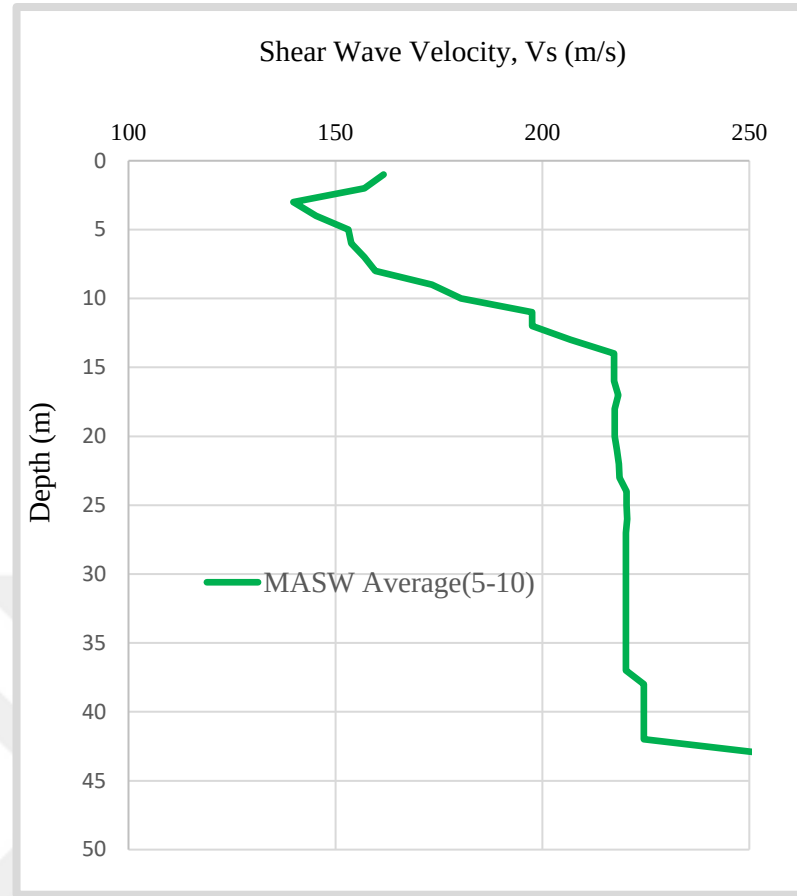


Figure 3.26 Average shear wave velocity with depth from MASW methods

Last method to determine shear wave profile of the ground is SPAC – array based microtremors. SPAC method determines the shear wave profile of the site using spatial autocorrelation method applied to results of array based microtremors. In this study, measurements with three different radii (27, 52 and 150m) were performed in single point. Test location and diameters are shown in Figure 3.30. With this method, shear wave velocities down to 250-300m depth are determined. Shear wave profile down to 250m and constant wave velocity after 250m were determined using SPAC method and presented in Figure 3.31.



Figure 3.27 Measurement point of downhole methods (Google earth, 2014)

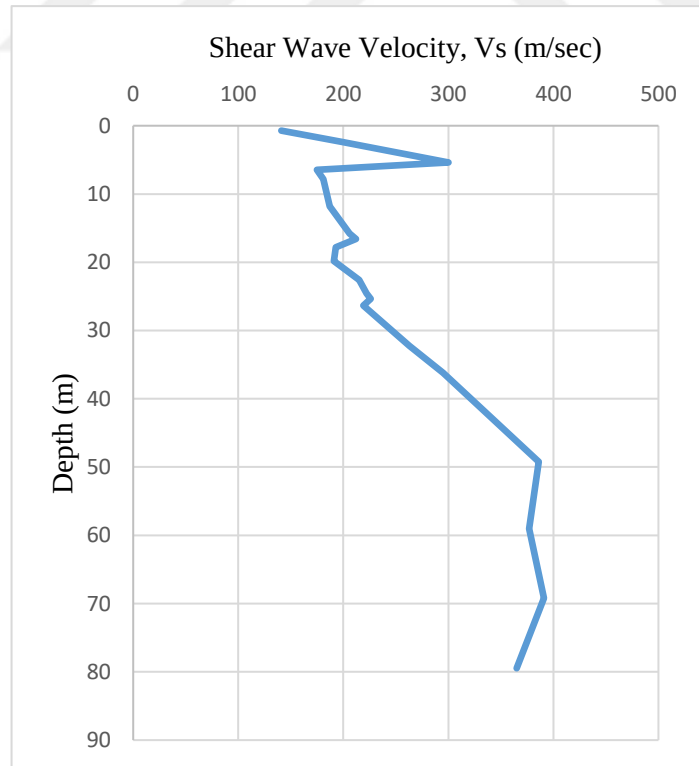


Figure 3.28 Original shear wave velocity with depth from downhole methods

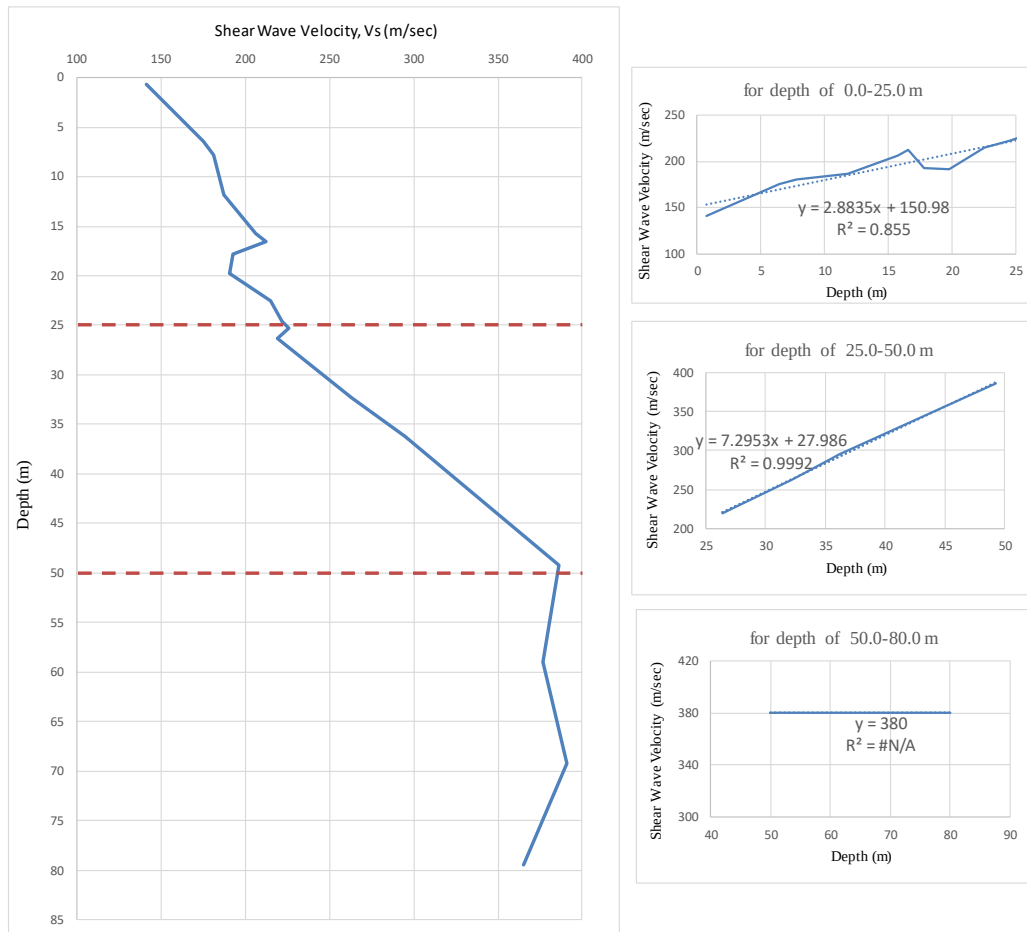


Figure 3.29 Corrected shear wave velocity with depth from Downhole Methods and velocity versus depth correlation graphs

Determining shear wave velocity profile accurately carries great importance for dynamic analyses. To do that, different methods are compared, and resultant average shear wave velocity profile is determined. MASW and Downhole methods resulted in similar values down to 30m and small differences between 30 and 50m. Therefore, average of MASW and Downhole methods are used for first 50m. Between 50 and 80m, average of Downhole and SPAC methods and after 80m, only SPAC method results are used. For each layer, shear wave velocities obtained from each method and average design shear wave velocities are presented in Table 3.14. Also, graphical representation is presented in Figure 3.32.

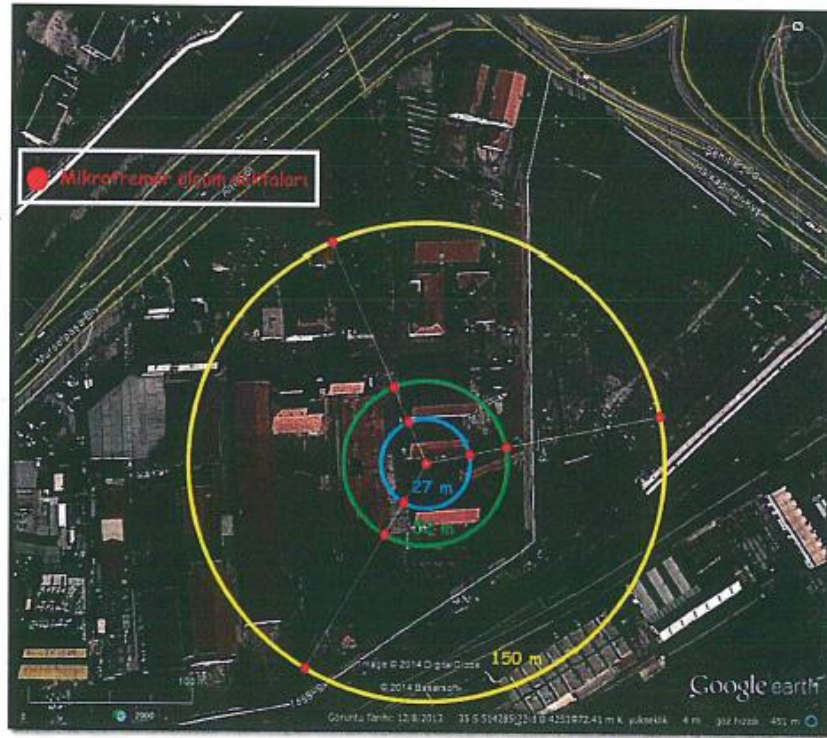


Figure 3.30 Measurement points of Spatial Auto-Correlation (SPAC) Method (Google earth, 2014)

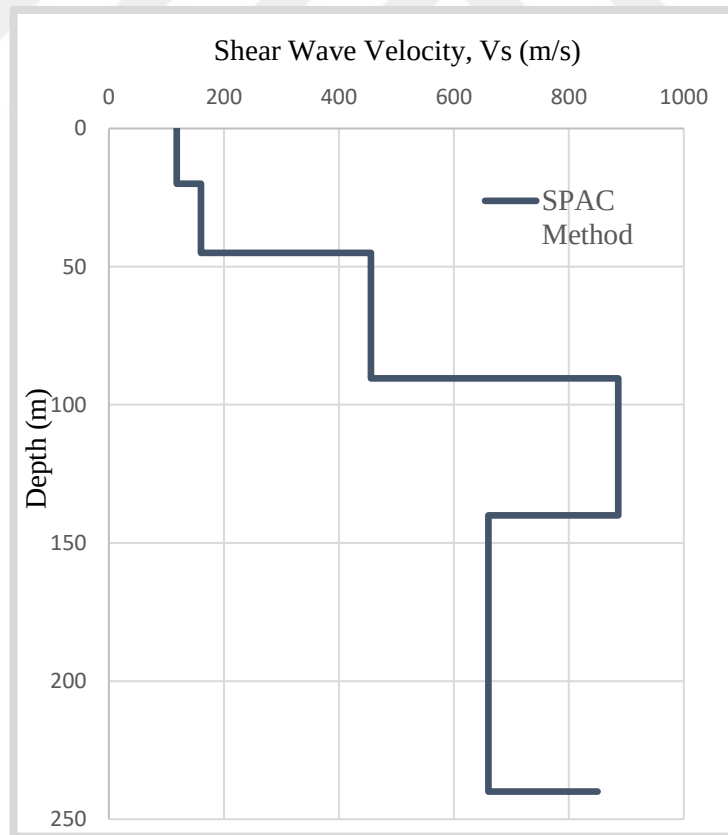


Figure 3.31 Shear wave velocity with depth from Spatial Auto-Correlation (SPAC) Method

Table 3.14 Idealized shear wave velocity from field test

Layer Number	Soil Type	Mid Depth (m)	MASW-Shear Wave Velocity, V_s (m/s)	Downhole-Shear Wave Velocity, V_s (m/s)	SPAC-Shear Wave Velocity, V_s (m/s)	Design Shear Wave Velocity, V_s (m/s)
1	Clay	1.5	151	157	118	154
2	Gravel	5.6	156	168	118	162
3	Sand	9.3	187	180	118	184
4	Clay	15.4	216	197	118	207
5	Gravel	23.0	220	217	160	219
6	Clay	32.3	221	265	160	243
7	Gravel	42.0	239	338	160	289
8	Clay	47.0	253	375	456	314
9	Sand	51.5	-	380	456	418
10	Clay	72.3	-	380	456	418
11	Gravel	107.5	-	-	886	886
12	Clay	132.3	-	-	886	886
13	Clay	190.0	-	-	660	660
14	Engineering Bedrock	617.0	-	-	850	850

Engineering bedrock's (250-1000m) shear wave velocity was assumed as 850 m/s, while shear wave velocity of the seismic bedrock (>1000m) was assumed as 3000 m/s.

In addition to shear wave velocity results obtained from geophysical tests, correlations between field tests and shear wave velocity were investigated. Correlation between SPT value and shear wave velocity of granular and cohesive soils are presented below (Wair, Dejong, & Shantz, 2012).

Granular Soils:

$$V_s = 30(N_{60})^{0.23}(\sigma'_v)^{0.23} \quad (3.9)$$

Cohesive Soils:

$$V_s = 26(N_{60})^{0.17}(\sigma'_v)^{0.32} \quad (3.10)$$

Correlation based shear wave velocities are compared with the geophysical results in Table 3.15.

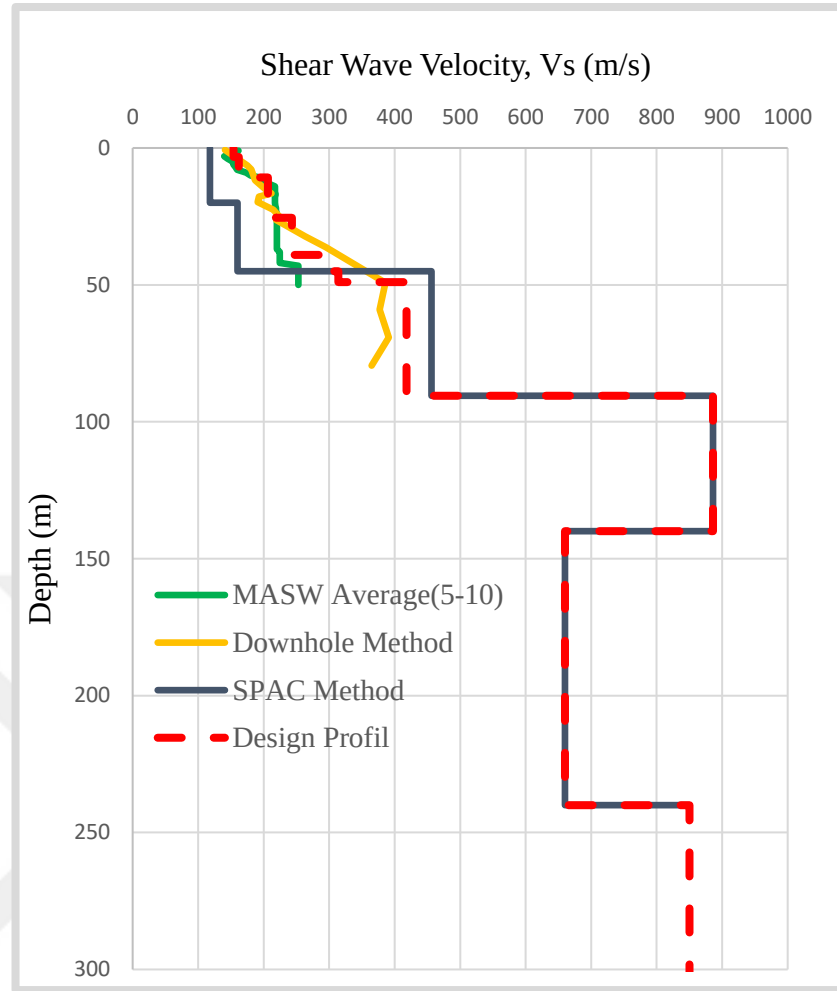


Figure 3.32 Design shear wave velocity and results of geophysical methods vs. depth

Table 3.15 Empirical shear wave velocity based on Wair et. al. (2012)

Layer Number	Soil Type	Mid Depth (m)	$N_{60,avr}$	Effective Stress σ'_{v0} (kPa)	Design Field Test-Shear Wave Velocity, Vs (m/s)	Empirical - Shear Wave Velocity, Vs (m/s)
1	Clay	1.5	6.5	28.5	154	104
2	Gravel	5.6	25.0	68.9	162	167
3	Sand	9.3	23.5	108.0	184	182
4	Clay	15.4	24.2	164.0	207	229
5	Gravel	23.0	34.8	233.9	219	238
6	Clay	32.3	15.7	318.9	243	263
7	Gravel	42.0	39.8	408.5	289	279
8	Clay	47.0	33.5	454.4	314	335
9	Sand	51.5	35.0	495.8	418	283
10	Clay	72.3	41.5	707.2	418	400
11	Gravel	107.5	>50	1066.4	886	-
12	Clay	132.3	>50	1318.6	886	-
13	Clay	190.0	>50	1907.1	660	-
14	Engineering Bedrock	617		6170.0	850	-

Generally, shear wave velocities obtained using geophysical tests and empirical correlations are very similar to each other as can be seen in Figure 3.33 except for 50-70m which is still acceptable.

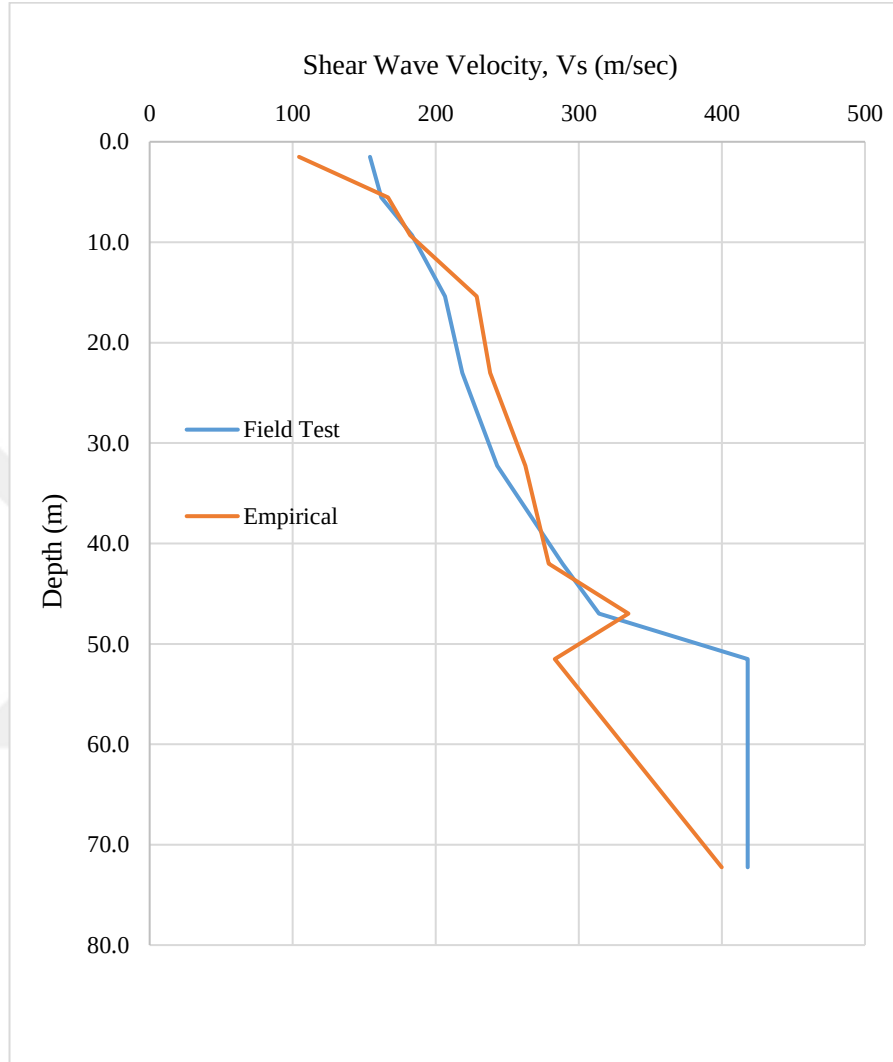


Figure 3.33 Comparison of the shear wave velocity values obtained by field tests and empirical relations

3.10 Seismic Site Classification

In this chapter, soil class to create response spectrum based on Turkish Earthquake Code (TBDY, Earthquake Code for Buildings, 2018) was determined. Soil class can be determined using SPT and shear wave velocity averages in upper 30m. Weighted average of shear wave velocity for upper 30m is 195 m/s, while weighted average of SPT is 18. Based on these results and definitions presented in TBDY soil class of the project site can be selected as ZD. Also, because the soil profile includes very thick

(>35m) soft or medium clays, the soil profile can be determined as ZF. In this case, because ZF presented more critical condition, soil profile was selected as ZF.

Table 3.16 Soil class from TBDY

Soil Class	Soil Type	Average in the top 30 meters		
		$(V_s)_{30}$ (m/s)	$(N_{60})_{30}$	$(C_u)_{30}$ (kPa)
ZA	Very strong rocks	> 1500	-	-
ZB	Slightly weathered, strong rocks	760 - 1500	-	-
ZC	Very dense sands, gravels and hard clays, and very jointed weak rocks	360 - 760	> 50	> 250
ZD	Medium dense, dense sands, gravels or very stiff clays	180 - 360	15 - 50	70 - 250
ZE	Loose sands, gravels or soft-medium clays or $PI > 20$ and $w > 40\%$ soft clays ($c_u > 25$ kPa) thicker than 3m at the surface	< 180	< 15	< 70
ZF	Soils that require site specific research and evaluation: 1) Soils that have risk of collapse or failure during earthquake (liquefiable soils, very sensitive clays, collapsible cemented soils etc.) 2) Peat and/or highly organic soils with total thickness over 3m 3) Highly plastic clays ($PI > 50$) with total thickness over 8m 4) Very thick (>35m) soft or medium clays.			

TBDY doesn't propose local soil effect coefficient for ZF soil class, therefore specific site response analyses are proposed by TBDY for ZF soil class.

Also, in very weak soil profiles, soil class are determined as Z4 according to DBYBHY (2007).

3.11 Definition of Soil Properties

Soil properties are determined using field and laboratory tests. Physical properties of the soils give information about strength and also dynamic properties. Soil classification and clay's plasticity indices are presented in Table 3.17.

To calculate lateral pressure on the soil skeleton, coefficient of lateral earth pressure (K_0) should be calculated. To calculate coefficient of lateral earth pressure, internal

friction angle should be calculated. For cohesive soils, Sorensen and Okkels (2013) proposed a correlation for normally consolidated clays. Correlation is presented below:

$$\phi'(\text{deg}) = 43 - 10 \log I_p \quad (3.11)$$

Internal friction angles of granular soils are calculated as average of several correlations. Correlations are presented below.

Peck, Hanson and Thornburn (1974) have proposed a chart to correlate ϕ' and N_{60} ; and Wolff (1989) have formulized this relation as below:

$$\phi'(\text{deg}) = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2 \quad (3.12)$$

Kulhawy and Mayne (1990) have proposed SPT-based correlation for friction angle with consideration of effective overburden stress:

$$\phi' = \tan^{-1} \left[\frac{N_{60}}{12.2 + 20.3 \left(\frac{\sigma'_0}{pa} \right)} \right]^{0.34} \quad (3.13)$$

Hatanaka and Uchida (1996) also proposed a correlation for internal friction angle with overburden corrected SPT value.

$$\phi' = \sqrt{20(N_1)_{60}} + 20 \quad (3.14)$$

Calculated friction angles and average of these correlations are presented below in Table 3.18.

Table 3.17 Physical Properties of Soil Layers

No	Type of Soil	Soil Class	PI (%)	Plasticity Degree	Type of Soil
1	Clay	CL	27	Plastic	Loose-Soft
2	Gravel	GP-GM	-	Nonplastic	Loose-Soft
3	Sand	SC	-	Nonplastic	Loose-Soft
4	Clay	CL	20	Plastic	Loose-Soft
5	Gravel	GC	-	Nonplastic	Loose-Soft
6	Clay	CH	30	Plastic	Loose-Soft
7	Gravel	GC	-	Nonplastic	Loose-Soft
8	Clay	CL	18	Plastic	Dense-Stiff
9	Sand	SC	-	Nonplastic	Dense-Stiff
10	Clay	CL	18	Plastic	Dense-Stiff
11	Gravel	GC	-	Nonplastic	Very Dense-Stiff
12	Clay	CL	18	Plastic	Very Dense-Stiff
13	Clay	CL	20	Plastic	Dense-Stiff

Table 3.18 Estimated values of friction angle, ϕ from SPT-N values

No	Soil Type	Wolff (1989) ϕ (°)	Hatanaka and Uchida (1996) ϕ (°)	Kulhawy and Mayne (1990) ϕ (°)	ϕ_{avr} (°)
1	Clay	-	-	-	29
2	Gravel	34	44	45	41
3	Sand	34	42	41	39
4	Clay	-	-	-	30
5	Gravel	37	43	40	40
6	Clay	-	-	-	28
7	Gravel	38	41	37	39
8	Clay	-	-	-	30
9	Sand	37	38	34	36
10	Clay	-	-	-	30
11	Gravel	41	52	31	41
12	Clay	-	-	-	30
13	Clay	-	-	-	30

Using the calculated internal friction angles, coefficients of lateral earth pressure were calculated. Using these coefficients, lateral earth pressures on the layers were calculated. Using effective vertical pressure and calculated lateral earth pressure effective confining pressures are calculated using the equations below. Calculated pressures are also presented in Table 3.19.

$$\sigma'_1 = \sigma'_v = \gamma' * h \quad (3.15)$$

$$\sigma'_2 = \sigma'_3 = \sigma'_h = K_0 * \sigma'_v \quad (3.16)$$

$$\sigma'_0 = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \quad (3.17)$$

Table 3.19 Pressures acting on soil layers

No	Soil Type	Mid-Depth of the Layer (m)	Vertical Effective Stress $\sigma'_{v,0}$ (kPa)	Horizontal Effective Stress $\sigma'_{h,0}$ (kPa)	Effective Confining Stress σ'_0 (kPa)
1	Clay	1.7	31.35	16.30	21.32
2	Gravel	5.6	68.90	23.86	38.87
3	Sand	9.3	109.30	40.49	63.42
4	Clay	15.4	165.36	82.71	110.26
5	Gravel	23.0	235.21	84.65	134.84
6	Clay	32.3	320.21	168.75	219.24
7	Gravel	42.0	409.82	154.24	239.43
8	Clay	47.0	455.77	224.81	301.79
9	Sand	51.5	497.12	202.02	300.39
10	Clay	72.3	708.56	349.50	469.19
11	Gravel	107.5	1067.76	366.44	600.21
12	Clay	132.3	1319.96	651.08	874.04
13	Clay	190.0	1908.44	954.52	1272.49

3.12 Idealized Soil Profile

Idealized soil profile was determined using laboratory and field test results. Soil profile consists of mostly clays and gravel-sand layers. There was clayey soils between 0 and 3.0m, in first 40m soft clays and sandy, gravelly clays, between 40 and 250m, again, mostly clays and occasional gravel and sand layers. Since boreholes were drilled down to 240m, from 240 to 1000m ground was assumed as engineering bedrock. Idealized soil profile is given in Figure 3.34.

0.0	SOIL SURFACE
-1.5	G.W.L
-3.3	CL (Low Plasticity Clay) $\gamma_n=19 \text{ kN/m}^3$ $N_{60}=6.5$ $N_{1,60}=9$ $V_s=154 \text{ m/s}$ $G_{\max}=48.4 \text{ MPa}$ $\phi=29$ $W_n=25$ $I_p=27$ $+No10=3$ $-No200=77$
-7.8	GP-GM (Poorly Graded Silty-Gavel) $\gamma_n=19 \text{ kN/m}^3$ $N_{60}=25$ $N_{1,60}=28$ $V_s=162 \text{ m/s}$ $G_{\max}=50.8 \text{ MPa}$ $W_n=10$ $\phi=41$ $+No10=65$ $-No200=10$
-10.8	SC (Clayey-Sand) $\gamma_n=19 \text{ kN/m}^3$ $N_{60}=23.5$ $N_{1,60}=24$ $V_s=184 \text{ m/s}$ $G_{\max}=65.2 \text{ MPa}$ $W_n=15$ $\phi=39$ $+No10=19$ $-No200=33$
-20.0	CL (Low Plasticity Clay) $\gamma_n=20 \text{ kN/m}^3$ $N_{60}=24$ $N_{1,60}=21$ $V_s=207 \text{ m/s}$ $G_{\max}=86.9 \text{ MPa}$ $W_n=26$ $I_p=20$ $c_u=45 \text{ kPa}$ $e_0=0,83$ $\sigma'_c=130 \text{ kPa}$ $c_c=0,31$ $c_s=0,05$ $+No10=0$ $-No200=85$
-25.5	GC (Clayey-Gravel) $\gamma_n=19,5 \text{ kN/m}^3$ $N_{60}=35$ $N_{1,60}=26$ $V_s=219 \text{ m/s}$ $G_{\max}=94.9 \text{ MPa}$ $W_n=15$ $\phi=40$ $+No10=40$ $-No200=36$
-39.0	CH (High Plasticity Clay) $\gamma_n=20 \text{ kN/m}^3$ $N_{60}=16$ $N_{1,60}=10$ $V_s=243 \text{ m/s}$ $G_{\max}=120.4 \text{ MPa}$ $W_n=38$ $I_p=30$ $c_u=65 \text{ kPa}$ $+No10=0$ $-No200=90$
-45.0	GC (Clayey-Gravel) $\gamma_n=19,5 \text{ kN/m}^3$ $N_{60}=40$ $N_{1,60}=22$ $V_s=289 \text{ m/s}$ $G_{\max}=165.4 \text{ MPa}$ $W_n=15$ $\phi=39$ $+No10=32$ $-No200=40$
-49.0	CL (Low Plasticity Clay) $\gamma_n=21 \text{ kN/m}^3$ $N_{60}=33.5$ $N_{1,60}=18$ $V_s=314 \text{ m/s}$ $G_{\max}=211.1 \text{ MPa}$ $I_p=18$ $W_n=23$ $c_u=90 \text{ kPa}$ $e_0=0,8$ $\sigma'_c=240 \text{ kPa}$ $c_c=0,24$ $c_s=0,04$ $+No10=7.5$ $-No200=80$
-54.0	SC (Clayey-Sand) $\gamma_n=19 \text{ kN/m}^3$ $N_{60}=35$ $N_{1,60}=17$ $V_s=418 \text{ m/s}$ $G_{\max}=338.4 \text{ MPa}$ $W_n=19$ $\phi=36$ $+No10=20$ $-No200=30$
-90.5	CL (Low Plasticity Clay) $\gamma_n=21 \text{ kN/m}^3$ $N_{60}=41.5$ $N_{1,60}=17$ $V_s=418 \text{ m/s}$ $G_{\max}=374 \text{ MPa}$ $W_n=26$ $I_p=18$ $c_u=130 \text{ kPa}$ $c_c=0,09$ $c_s=0,03$ $e_0=0,6$ $+No10=2$ $-No200=80$
-124.5	GC (Clayey-Gravel) $\gamma_n=21 \text{ kN/m}^3$ $V_s=886 \text{ m/s}$ $G_{\max}=1680 \text{ MPa}$ $W_n=8$ $+No10=70$ $-No200=16.5$
-140.0	CL (Low Plasticity Clay) $\gamma_n=21 \text{ kN/m}^3$ $V_s=886 \text{ m/s}$ $G_{\max}=1680 \text{ MPa}$ $W_n=19$ $I_p=18$ $+No10=25$ $-No200=50$
-240.0	CL (Low Plasticity Clay) $\gamma_n=21 \text{ kN/m}^3$ $V_s=660 \text{ m/s}$ $G_{\max}=932 \text{ MPa}$ $W_n=19$ $I_n=20$ $+No10=1$ $-No200=75$
-1000	ENGINEERING BEDROCK $V_s=850 \text{ m/s}$

Figure 3.34 Idealized soil profile

3.13 Soil-Bedrock Model

Soil profile is distinction between soil layers and bedrock level. Many researchers have distinguished engineering bedrock and seismic bedrock based on their shear wave velocities. Ambrasey et al. (1996) have defined soils as where shear wave velocity below 760 m/s. Nath (2007) and Morikawa et al. (2008) defined seismic bedrock as where shear wave velocity is over 3000 m/s. Anbazhagan and Sitharam (2009) have defined weathered rock as where shear wave velocity is over 330 ± 30 m/s as where shear wave velocity is over 760 ± 60 m/s. Lastly, Akgün et al. (2013) have defined engineering bedrock shear wave velocity range as 760 to 3000 m/s and seismic bedrock as shear wave velocities over 3000 m/s. This thesis deals with comparison of site response analyses performed at engineering bedrock and seismic bedrock. In this study, Akgün et al.'s (2013) assumptions were accepted. Soil-Engineering bedrock and soil-seismic bedrock profiles are presented in Figure 3.35 and Figure 3.36.

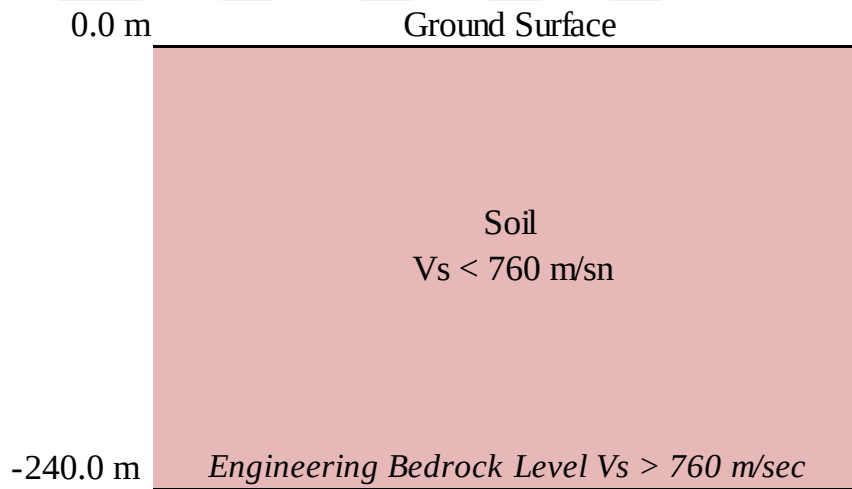


Figure 3.35 Soil-Engineering bedrock model (Model-1)

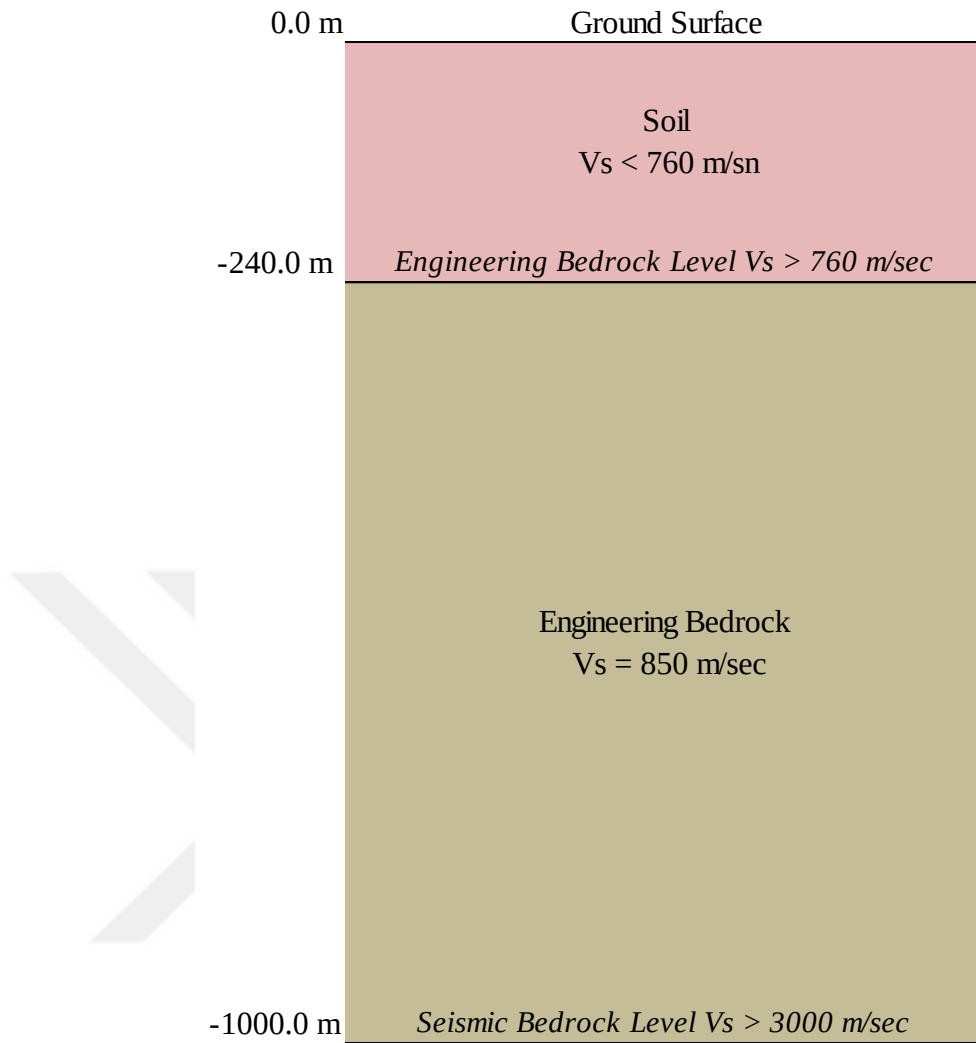


Figure 3.36 Soil-Seismic bedrock model (Model-2)

3.14 Modulus Reduction and Damping Ratio for Soil

Dynamic soil parameters are calculated using the approach proposed by Ishibashi and Zhang (1993). These correlations consider plasticity indices and effective confining pressure into account. Details of this approach were presented in the previous chapters. Input parameters for this method are presented in Table 3.20.

Table 3.20 Input parameters for Ishibashi-Zhang method

No	Soil Type	Soil Class	Mid-Depth (m)	PI (%)	Effective Confining Stress σ'_{o} (kPa)
1	Clay	CL	1.7	27	21.32
2	Gravel	GP-GM	5.6	-	38.87
3	Sand	SC	9.3	-	63.42
4	Clay	CL	15.4	20	110.26
5	Gravel	GC	23.0	-	134.84
6	Clay	CH	32.3	30	219.24
7	Gravel	GC	42.0	-	239.43
8	Clay	CL	47.0	18	301.79
9	Sand	SC	51.5	-	300.39
10	Clay	CL	72.3	18	469.19
11	Gravel	GC	107.5	-	600.21
12	Clay	CL	132.3	18	874.04
13	Clay	CL	190.0	20	1272.49

3.14.1 Modulus Reduction and Damping Ratio for Granular Layers

Modulus reduction and damping curves based on the approach of Ishibashi and Zhang (1993) for granular layers are presented Figure 3.37 and Figure 3.38. These curves were input to DEEPSOIL software.

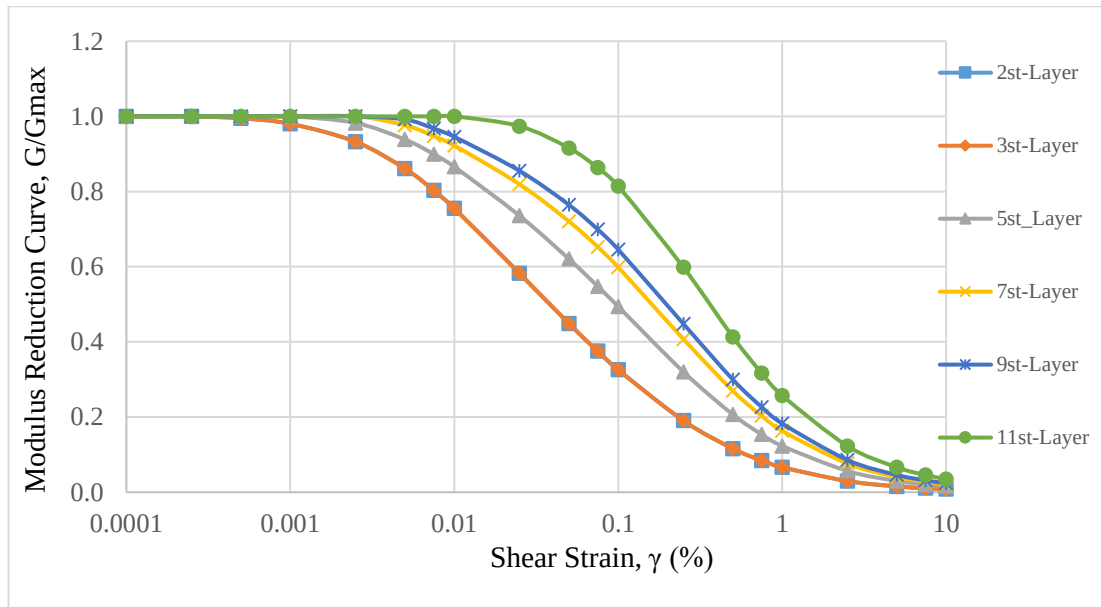


Figure 3.37 Modulus reduction curves of granular layers

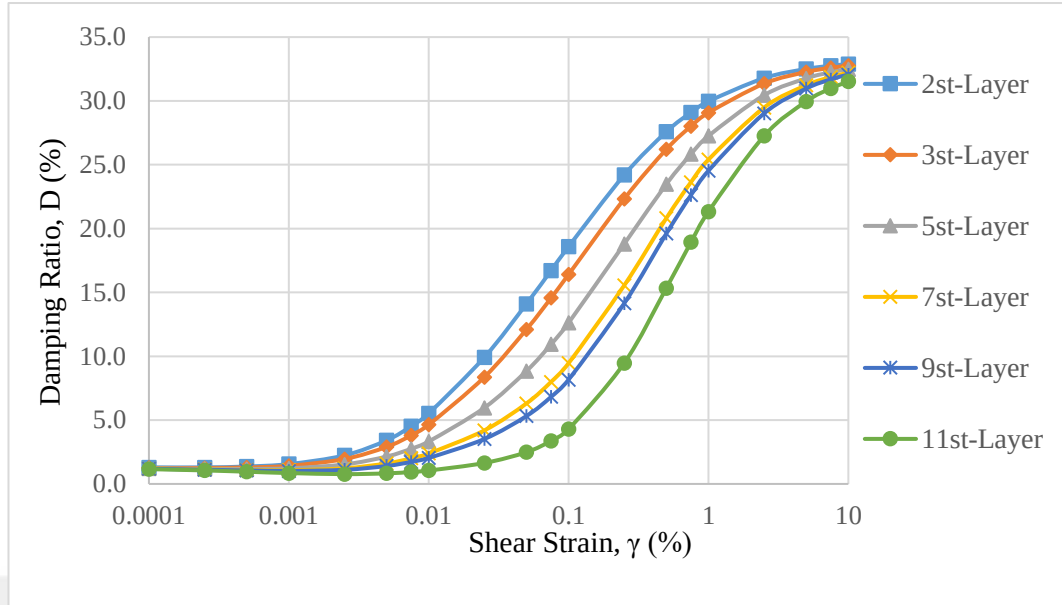


Figure 3.38 Damping curve of granular layers

3.14.2 Modulus Reduction and Damping Ratio for Clay

Modulus reduction and damping curves based on the approach of Ishibashi and Zhang (1993) for clay layers are presented Figure 3.39 and Figure 3.40. These curves were input to DEEPSOIL software.

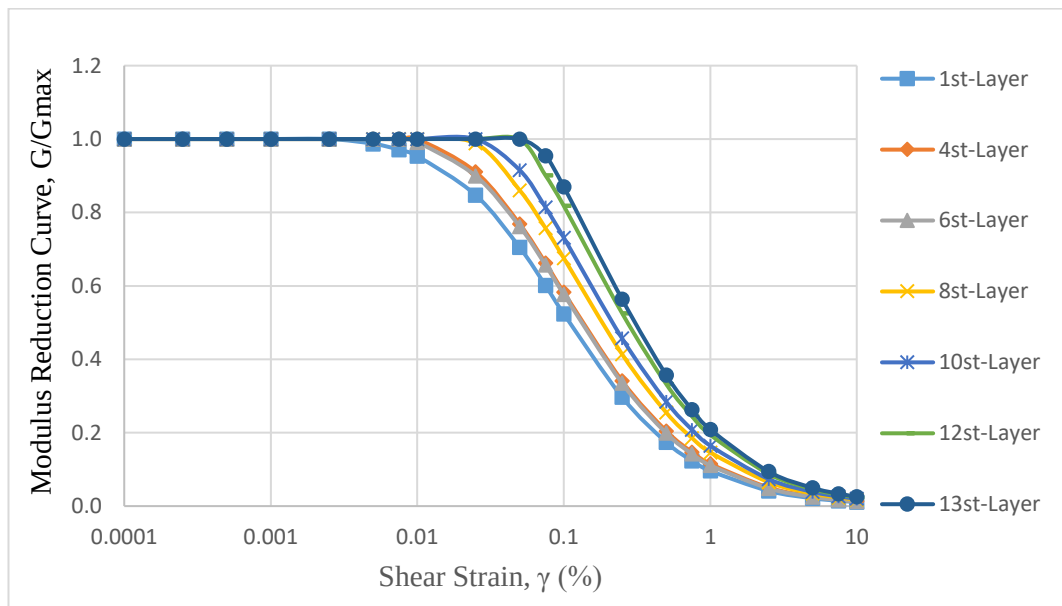


Figure 3.39 Modulus reduction curve of clay layers

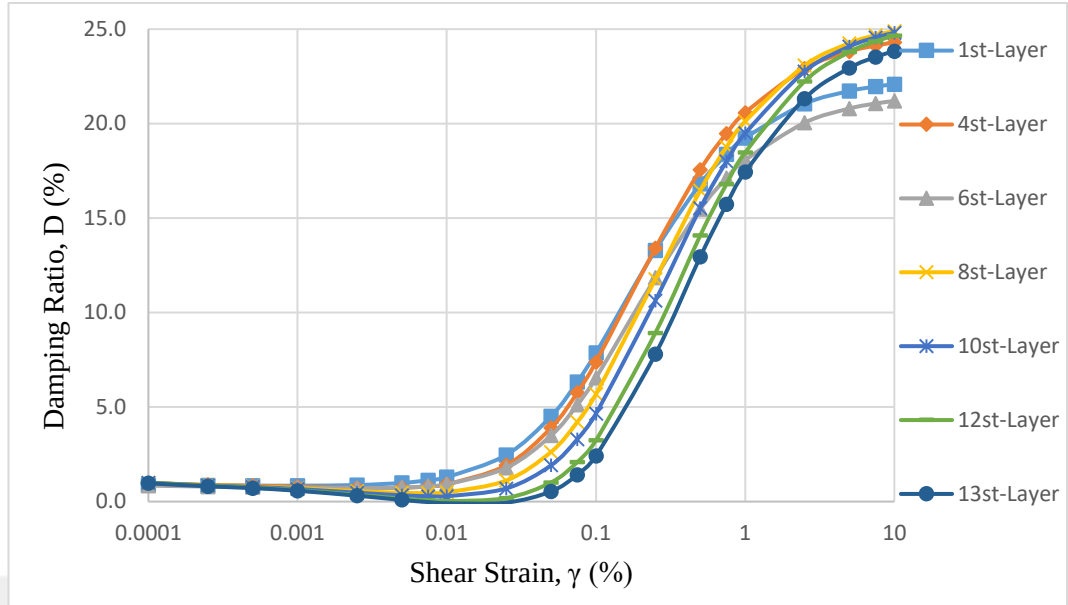


Figure 3.40 Damping curve of clay layers

3.14.3 Modulus Reduction and Damping Ratio for Rock

In this study, modulus reduction and damping curves based on EPRI (1993) for rock were used for rock layers. Weathered rock were assumed after 240m in the project site. EPRI (1993) recommendations for rock depths between 150-300, 300-600 and 600-1500 are presented in Table 3.21, Table 3.22 and Table 3.23, respectively. Also, graphical representations of these curves are given in Figure 3.41 and Figure 3.42.

Table 3.21 Shear modulus reduction and damping ratio for rock depth of 150-300 m

Shear Strain %	G/Gmax	Damping Ratio %
0.0001	1	3
0.001	1	3
0.003	0.960	4
0.01	0.875	5
0.03	0.675	8
0.1	0.410	14
0.3	0.200	20
1	0.080	26

Table 3.22 Shear modulus reduction and damping ratio for rock depth of 300-600 m

Shear Strain %	G/G _{max}	Damping Ratio %
0.0001	1	3
0.001	1	3
0.003	1	3
0.01	0.920	5
0.03	0.750	7
0.1	0.500	12
0.3	0.250	18
1	0.100	24

Table 3.23 Shear modulus reduction and damping ratio for rock depth of 600-1500m

Shear Strain %	G/G _{max}	Damping Ratio %
0.0001	1	3
0.001	1	3
0.003	1	3
0.01	0.950	4
0.03	0.825	6
0.1	0.600	10
0.3	0.325	15
1	0.150	22

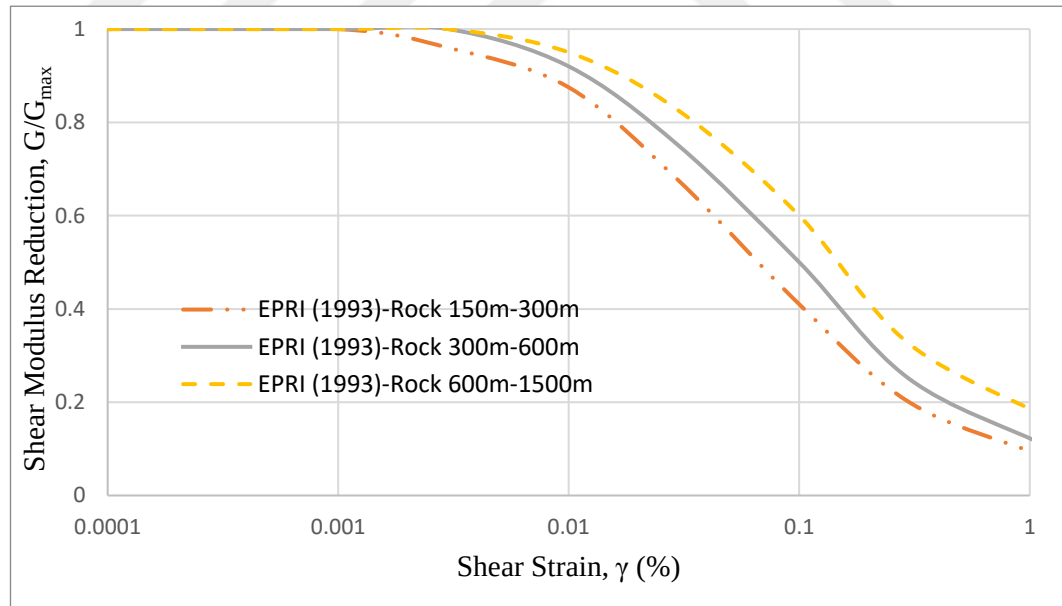


Figure 3.41 Modulus reduction curves of rock layers based on EPRI (1993)

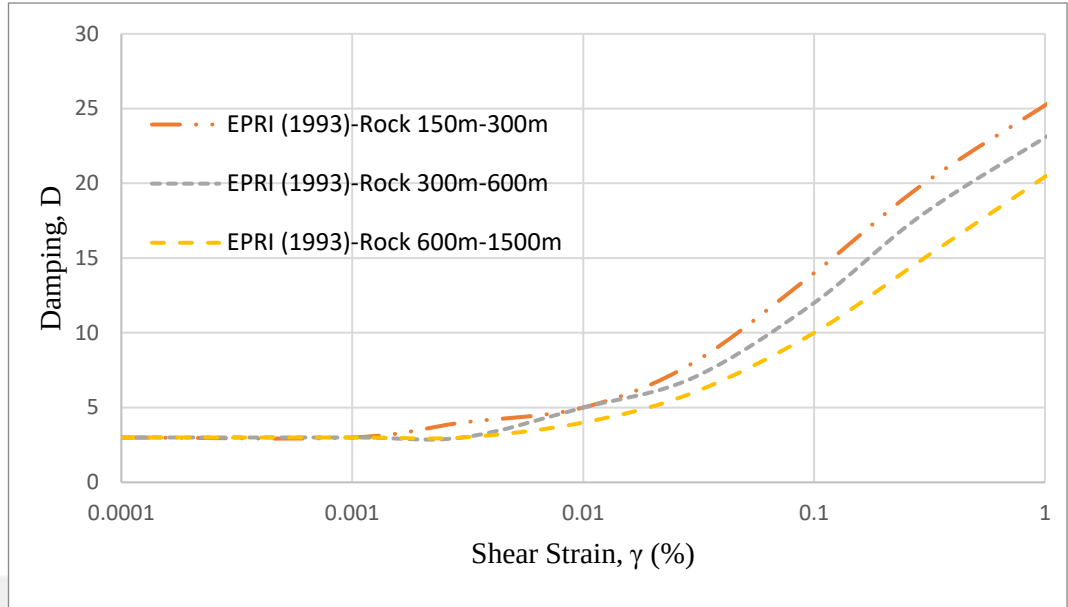


Figure 3.42 Damping ratio curves of rock layers based on EPRI (1993)

3.15 Maximum Layer Thickness

A significant consideration in nonlinear analysis is the thickness of layers when decoupled the soil column. In general, the maximum cut-off frequency (f_{max}) is calculated as shown in following equation:

$$f_{max} = \frac{v_s}{4H} \quad (3.18)$$

where V_s is the shear wave velocity and H is the thickness of the given layer. Hashash et. al. (2010) recommends cut-off frequency values above 30 Hz. Therefore, assuming f_{max} is 30 Hz, maximum layer thicknesses were calculated and presented in Table 3.24.

Table 3.24 Maximum layer thicknesses of soil layers for 30 Hz cut-off frequency

Layer Number	Thickness Layer (m)	Shear Wave Velocity, V_s (m/s)	Frequency (HZ)	γ (kN/m ³)	ρ (kN*s ² /m)	G_{max} (MPa)
1	1.3	154	30	20.0	2.0	48.4
2	1.4	162	30	19.0	1.9	50.8
3	1.5	184	30	19.0	1.9	65.2
4	1.7	207	30	20.0	2.0	86.9
5	1.8	219	30	19.5	2.0	94.9
6	2.0	243	30	20.0	2.0	120.4
7	2.4	289	30	19.5	2.0	165.4
8	2.6	314	30	21.0	2.1	211.1
9	3.5	418	30	19.0	1.9	338.4
10	3.5	418	30	21.0	2.1	374.0
11	7.4	886	30	21.0	2.1	1680.4
12	7.4	886	30	21.0	2.1	1680.4
13	5.5	660	30	21.0	2.1	932.5
14	7.1	850	30	22.0	2.2	1620.3

3.16 Porewater Pressure Parameters of Soil

DEEPSOIL (Hashash, et al., 2017) performs nonlinear excess pore pressure analyses using the models described in Chapter Two. Moreover, software asks for coefficient of consolidation, c_v . For low plasticity clays, based on the results of consolidation tests, average coefficient of consolidation is determined as 1.5×10^{-8} m²/sec. Coefficient of consolidation can be also calculated using the simple formula $c_v = k/m_v\gamma_w$ where m_v is coefficient of volume compressibility, k is hydraulic conductivity and γ_w is unit weight of water.

Since determining coefficient of consolidation for clayey sands and clayey gravels using consolidation tests is not easy, Shelly (1993) investigated on effects of gravel content on the hydraulic conductivity of clays. As can be seen in Figure 3.43, for gravel contents below 60%, clay governs the behavior, and hydraulic conductivity stays same. For gravel contents above 60%, hydraulic conductivity increases exponentially.

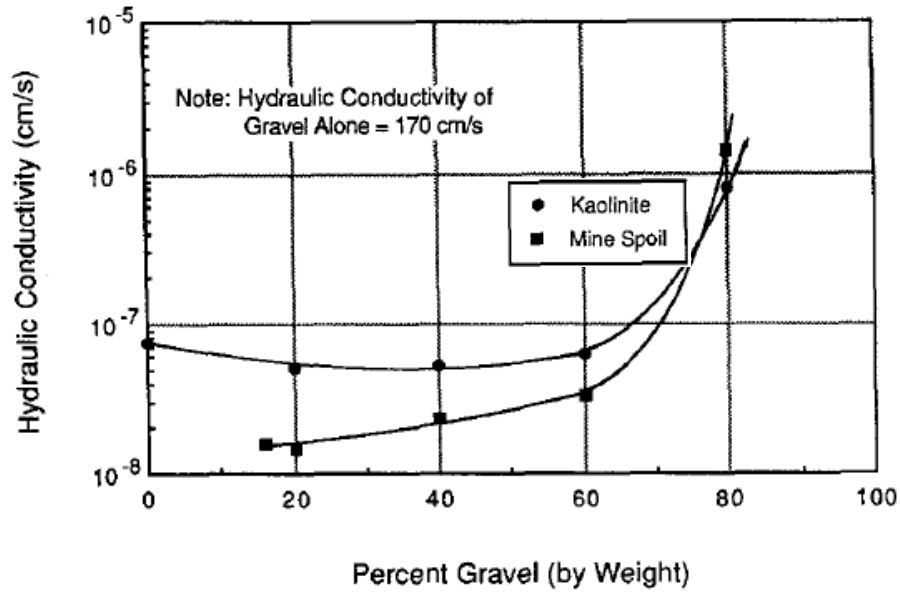


Figure 3.43 Hydraulic conductivity of gravelly clays for different gravel percent (Shelley, 1993)

Percent weights of soils that retained on 4.76mm sieve (#No4) and corresponding hydraulic conductivities based on Shelly' (1993) are given in Table 3.25.

Coefficient of volume compressibility can be calculated as:

$$m_v = \frac{1}{E_u} \quad (3.19)$$

Butler (1975) correlates undrained Young's (E_u) modulus with SPT-N value as below.

$$\frac{E_u}{N_{60}} = (1 - 1.2) \text{ (MN/m}^2\text{)} \quad (3.20)$$

Hence, undrained Young's modulus is calculated as $1 \times N_{60}$ equation. Based on the correlations presented above, coefficient of consolidation was calculated and presented in Table 3.25.

Table 3.25 Determining hydraulic conductivity and coefficient of consolidation

Layer Number	Unified Soil Classification USCS	N _{60,avr}	+ #No4	Hydraulic Conductivity k (m/s)	Coefficient of volume compressibility m _v (m ² /kN)	Coefficient of Consolidation c _v (m ² /s)
1	CL	6.5	23	-	1.54E-04	1.5E-08
2	GP-GM	25	90	1E-07	4.00E-05	2.5E-04
3	SC	23.5	67	1E-09	4.26E-05	2.4E-06
4	CL	24.2	15	-	4.13E-05	1.5E-08
5	GC	34.75	64	8E-10	2.88E-05	2.8E-06
6	CH	15.7	10	-	6.37E-05	1.5E-08
7	GC	39.75	60	7E-10	2.52E-05	2.6E-06
8	CL	33.5	20	-	2.99E-05	1.5E-08
9	SC	35	70	1.5E-09	2.86E-05	5.3E-06
10	CL	41.5	20	-	2.41E-05	1.5E-08
11	GC	50	83.5	2E-08	2.00E-05	7.5E-05
12	CL	50	50	-	2.00E-05	1.5E-08
13	CL	50	25	-	2.00E-05	1.5E-08

Required input parameters to be used in excess pore water pressure model parameter calculations are given in Table 3.26. Using these information, excess pore pressure model parameters were calculated and presented in Table 3.27 and Table 3.28.

Table 3.26 The selected coefficient of consolidation and soil parameters

Layer Number	Soil Type	Unified Soil Classification USCS	FC %	PI (%)	Idealized Field Test-Shear Wave Velocity, V _s (m/s)	c _v (m ² /s)
1	Clay	CL	77	27	154	1.5E-08
2	Gravel	GP-GM	10	0	162	2.5E-04
3	Sand	SC	33	0	183.5	2.4E-06
4	Clay	CL	85	20	206.5	1.5E-08
5	Gravel	GC	36	0	218.5	2.8E-06
6	Clay	CH	90	30	243	1.5E-08
7	Gravel	GC	40	0	288.5	2.6E-06
8	Clay	CL	80	18	314	1.5E-08
9	Sand	SC	30	0	418	5.3E-06
10	Clay	CL	80	18	418	1.5E-08
11	Gravel	GC	16.5	0	886	7.5E-05
12	Clay	CL	50	18	886	1.5E-08
13	Clay	CL	75	20	660	1.5E-08
14	Engineering Bedrock				850	1.5E-08

Table 3.27 The excess pore pressure parameters for sand layers

Soil Layer Properties		Exc. Pore Water Pressure Parameters for Granular Layers					
Layer Number	Soil Type	f	p	F	s	γ_{tvp} (%)	v
1	Clay	-	-	-	-	-	-
2	Gravel	1	1	1.43	1.35	0.02	3.8
3	Sand	1	1	1.18	1.56	0.02	3.8
4	Clay	-	-	-	-	-	-
5	Gravel	1	1	0.90	1.57	0.02	3.8
6	Clay	-	-	-	-	-	-
7	Gravel	1	1	0.59	1.59	0.02	3.8
8	Clay	-	-	-	-	-	-
9	Sand	1	1	0.33	1.54	0.02	3.8
10	Clay	-	-	-	-	-	-
11	Gravel	1	1	0.10	1.43	0.02	3.8
12	Clay	-	-	-	-	-	-
13	Clay	-	-	-	-	-	-
14	Engineering Bedrock	1	1	0.11	1.00	0.02	3.8

Table 3.28 The excess pore pressure parameters for clay layers

Soil Layer Properties		Exc. Pore Water Pressure Parameters for Cohesive Layers (OCR=1)						
Layer Number	Soil Type	s	r	A	B	C	D	γ_{tvp} (%)
1	Clay	0.116467	0.545114	7.6451	-14.714	6.38	0.6922	0.1
2	Gravel	-	-	-	-	-	-	-
3	Sand	-	-	-	-	-	-	-
4	Clay	0.148159	0.563917	7.6451	-14.714	6.38	0.6922	0.1
5	Gravel	-	-	-	-	-	-	-
6	Clay	0.10703	0.538662	7.6451	-14.714	6.38	0.6922	0.1
7	Gravel	-	-	-	-	-	-	-
8	Clay	0.161223	0.570671	7.6451	-14.714	6.38	0.6922	0.1
9	Sand	-	-	-	-	-	-	-
10	Clay	0.161223	0.570671	7.6451	-14.714	6.38	0.6922	0.1
11	Gravel	-	-	-	-	-	-	-
12	Clay	0.161223	0.570671	7.6451	-14.714	6.38	0.6922	0.1
13	Clay	0.148159	0.563917	7.6451	-14.714	6.38	0.6922	0.1

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Introduction

In this study, a site located in İzmir Bay which is composed of deep alluvial soils has been analyzed. Site specific response analyses were carried out using nonlinear effective and total stress analyses using eleven earthquakes acquired from PEER (2006) database considering the soil class and seismicity of the site. The main aim of this study is to investigate the effect of the thickness of the layer between the engineering bedrock and the seismic bedrock using total and effective stress-based site response analysis methods. Thus, analyses were carried out using different bedrock levels. These soil models were previously (i.e. Chapter 3) described as Soil-Engineering Bedrock (Model-1, total profile depth: 240m) and Soil-Seismic Bedrock (Model-2, total profile depth: 1000m).

4.2 Site Response Analysis Results

In this section, results of site response analyses for Model-1 and Model-2 using effective and total stress analyses are presented. Properties of analyses models are given in Table 4.1.

Table 4.1 General characteristics of Model-1 and Model-2

Soil Profile	Model - 1	Model - 2
Total Profile Depth (m)	240	1000
Natural Frequency (Hz)	0.472	0.176
Natural Period (sec)	2.121	5.697
Shear Wave Velocity of Eng./Seis. Bedrock (m/s)	850	3000

In order to properly present results and to make an orderly discussion, earthquakes are represented by abbreviations such as EQ-S6-1 meaning the earthquake is the 6th one in the table and its magnitude is scaled with respect to DD-1 level as defined in TBDY code. Short name of each earthquake is given in Table 4.2.

Table 4.2 Abbreviations assigned to the earthquakes

Result ID	Record Sequence Number	Earthquake Name	Earthquake Short Name for DD1 Earthquake Level	Earthquake Short Name for DD2 Earthquake Level
1	145	"Coyote Lake"	EQ-S1-1	EQ-S2-1
2	156	"Norcia_ Italy"	EQ-S1-2	EQ-S2-2
3	288	"Irpinia_ Italy-01"	EQ-S1-3	EQ-S2-3
4	302	"Irpinia_ Italy-02"	EQ-S1-4	EQ-S2-4
5	459	"Morgan Hill"	EQ-S1-5	EQ-S2-5
6	1108	"Kobe_ Japan"	EQ-S1-6	EQ-S2-6
7	1612	"Duzce_ Turkey"	EQ-S1-7	EQ-S2-7
8	1787	"Hector Mine"	EQ-S1-8	EQ-S2-8
9	3954	"Tottori_ Japan"	EQ-S1-9	EQ-S2-9
10	4097	"Parkfield-02_ CA"	EQ-S1-10	EQ-S2-10
11	4482	"L'Aquila_ Italy"	EQ-S1-11	EQ-S2-11

4.3 Total Stress Analysis Results

In this section, results of total stress ground response analyses for DD-1 and DD-2 earthquake levels are presented. For each selected earthquake, 5% damping response spectra were determined for Model-1 and Model-2 soil profiles.

4.3.1 Total Stress Analysis Results for DD-1 Earthquake Level

While earthquake waves travel through layered soils, changes occur in their predominant periods and amplitudes. These variations depend on maximum acceleration of the earthquake at the bedrock level, predominant period of soil layers and earthquakes along with dynamic soil parameters. One may investigate the causes of such variations by comparing parameters being peak ground acceleration, strain and deformation graphs are investigated through different soil layers which were given in Figure 4.2 and Figure 4.1. Accelerations, strains and displacement of the selected earthquakes at DD-1 level are presented in Figure 4.3.

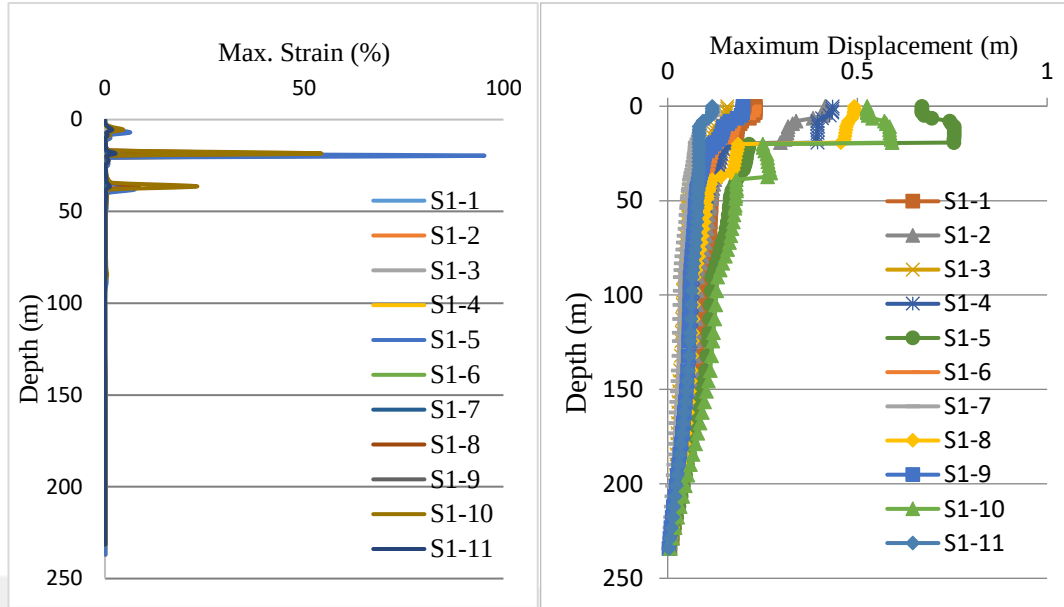


Figure 4.1 Max. strain and max. displacement for Model-1 with DD-1 earthquake level

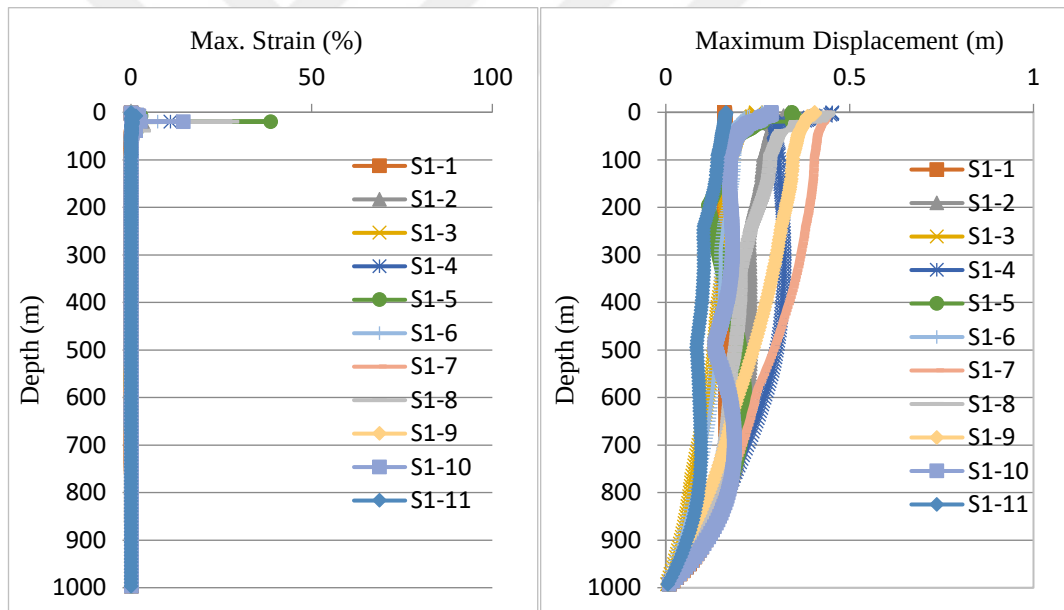


Figure 4.2 Max. strain and max. displacement for Model-2 with DD-1 earthquake level

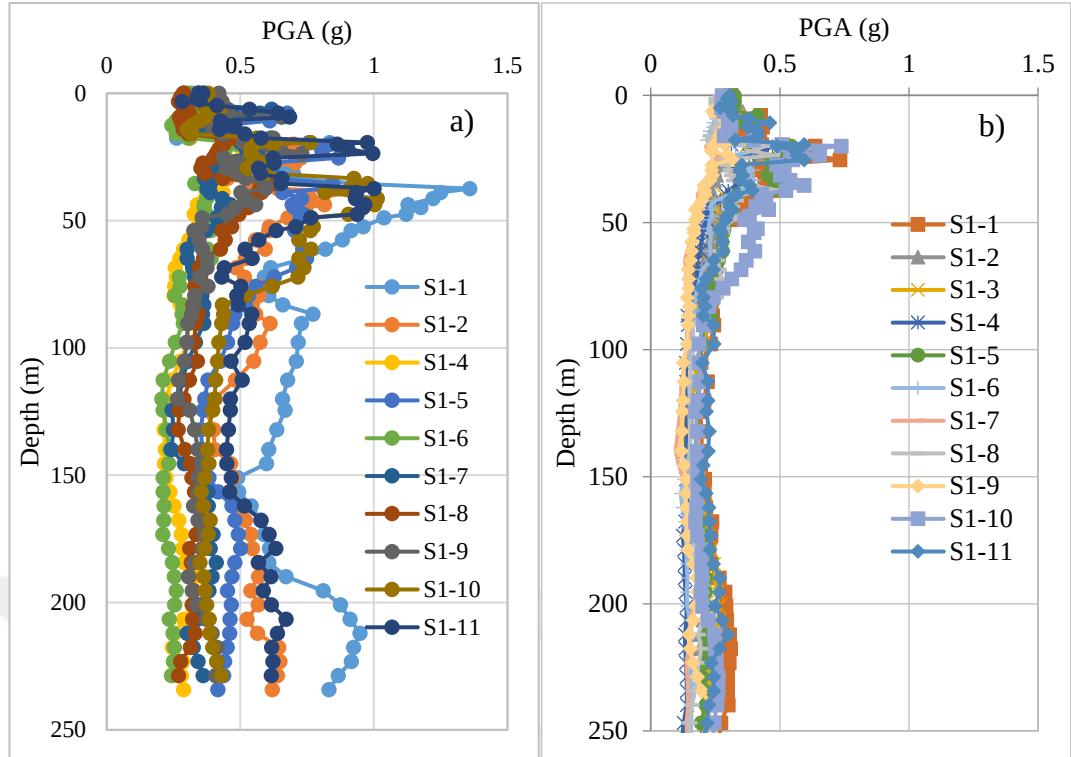


Figure 4.3 PGA value versus depth for a) Model-1 and b) Model-2 with DD-1

Maximum strain and maximum displacement graphs show that due to increasing displacements at the upper 20m, earthquakes are deamplified while propagating from defined bedrock level to the surface. Increasing strains absorb the earthquake waves, therefore response spectrum and PGAs decrease.

Acceleration spectrum of single degree freedom system with 5% damping under DD-1 earthquake level for Model-1 and Model-2 soil profiles are obtained. Results are presented in Figure 4.4 and Figure 4.5.

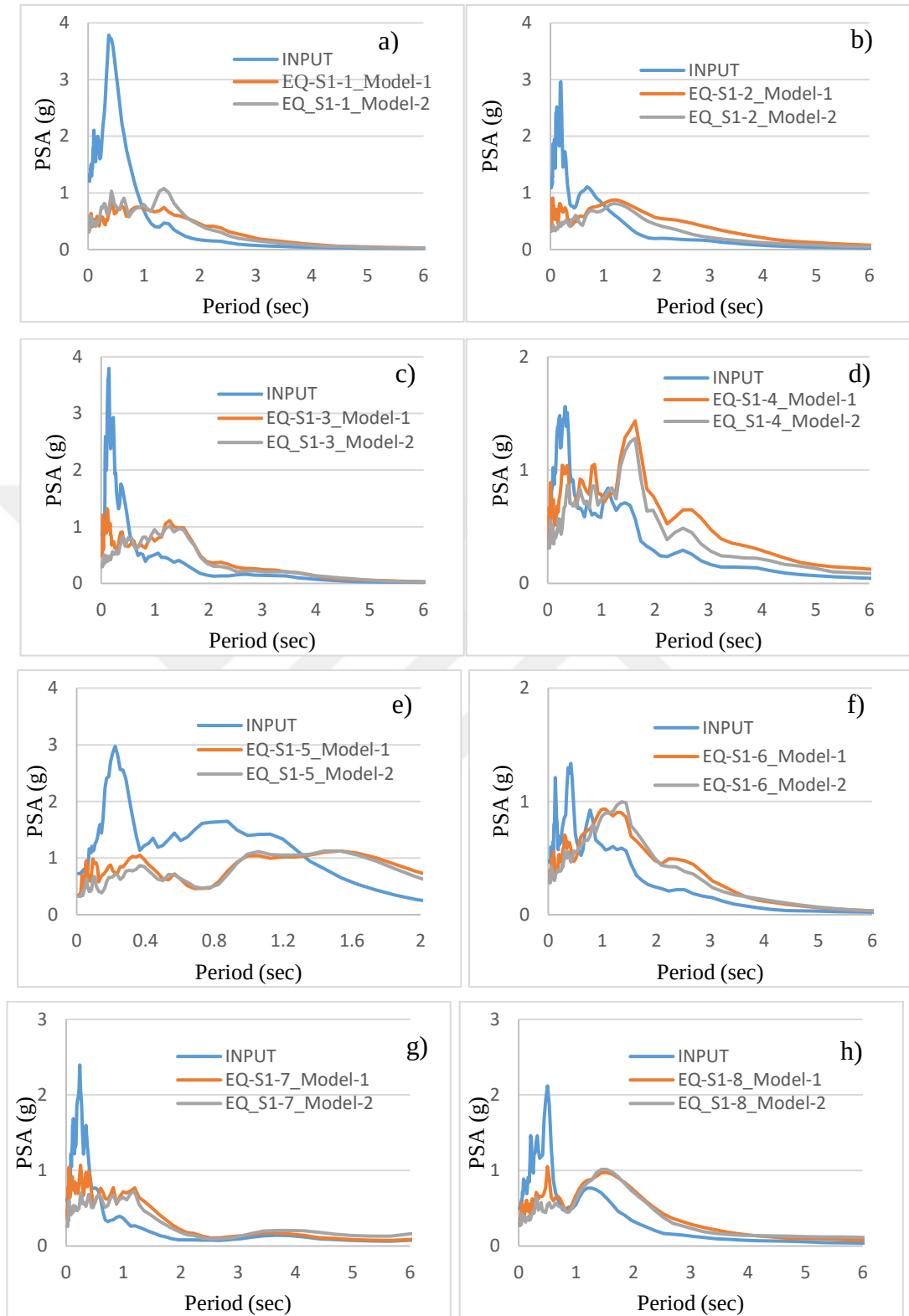


Figure 4.4 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S1-1, b) EQ-S1-2, c) EQ-S1-3, d) EQ-S1-4, e) EQ-S1-5, f) EQ-S1-6, g) EQ-S1-7, h) EQ-S1-8

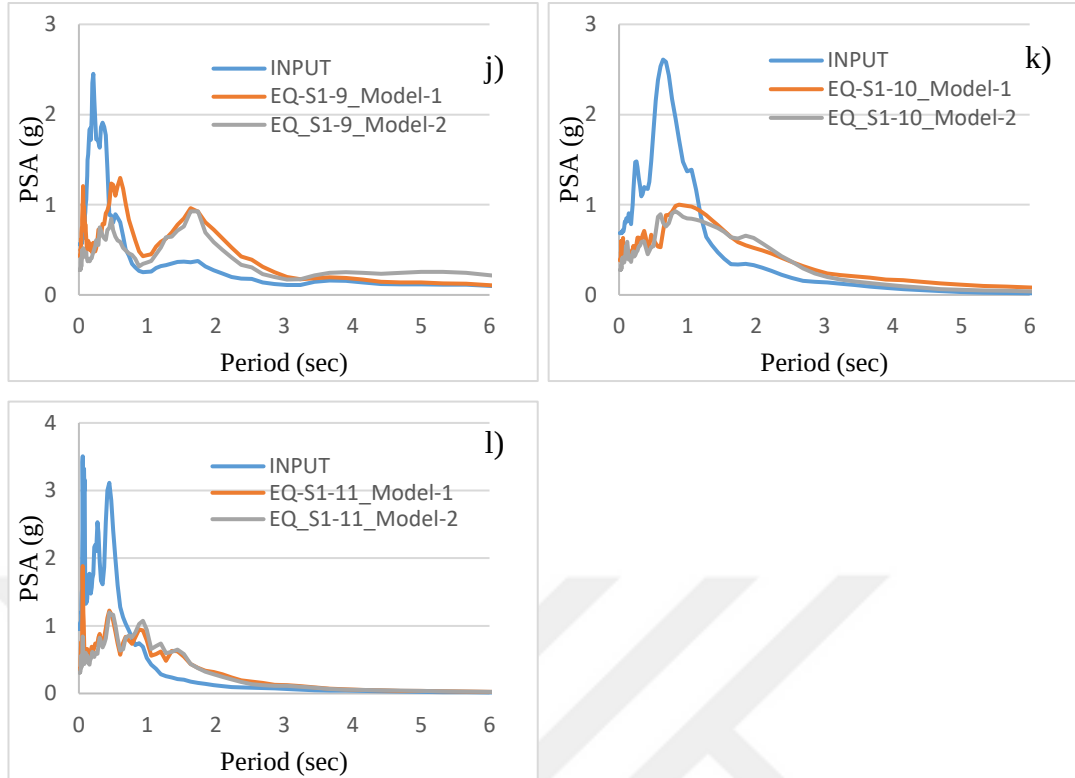


Figure 4.5 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for j) EQ-S1-9, k) EQ-S1-10, l) EQ-S1-11.

Based on the results presented above, accelerations at bedrock level are deamplified while propagating through soft alluvial soils. Acceleration spectrum of Model-1 and Model-2 with average of all 11 earthquakes are given in Figure 4.6 and Figure 4.7, respectively.

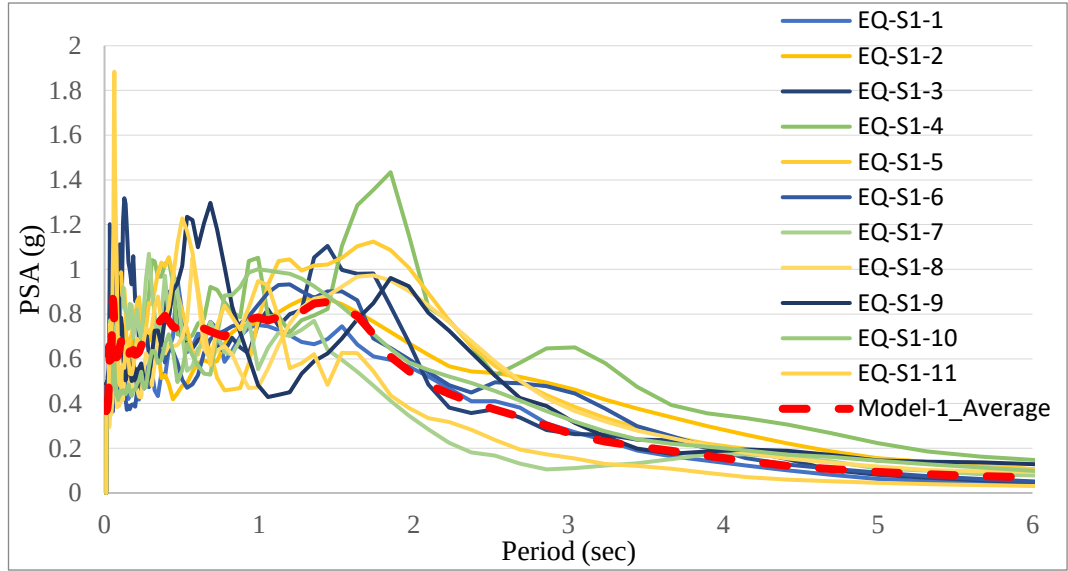


Figure 4.6 Average PSA curve for Model-1 (depth of 240m) with DD-1 earthquake level

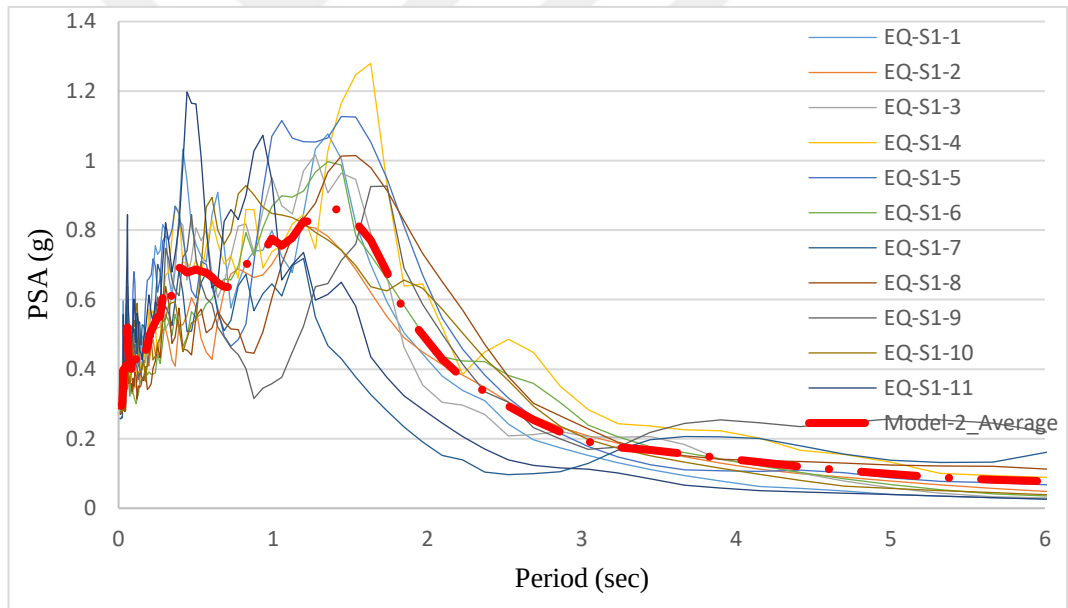


Figure 4.7 Average PSA curve for Model-2 (depth of 1000m) with DD-1 earthquake level

4.3.2 Discussion of Total Stress Analysis Results for DD-1 Earthquake Level

Comparison of Model-1 and Model-2 for DD-1 earthquake level is shown in Figure 4.8. Results show that Model-1 reaches higher accelerations at short period region than Model-2. The difference in short period reaches up to 40%, but average difference about 20%. The difference of two models in short period region is shown in Figure 4.9.

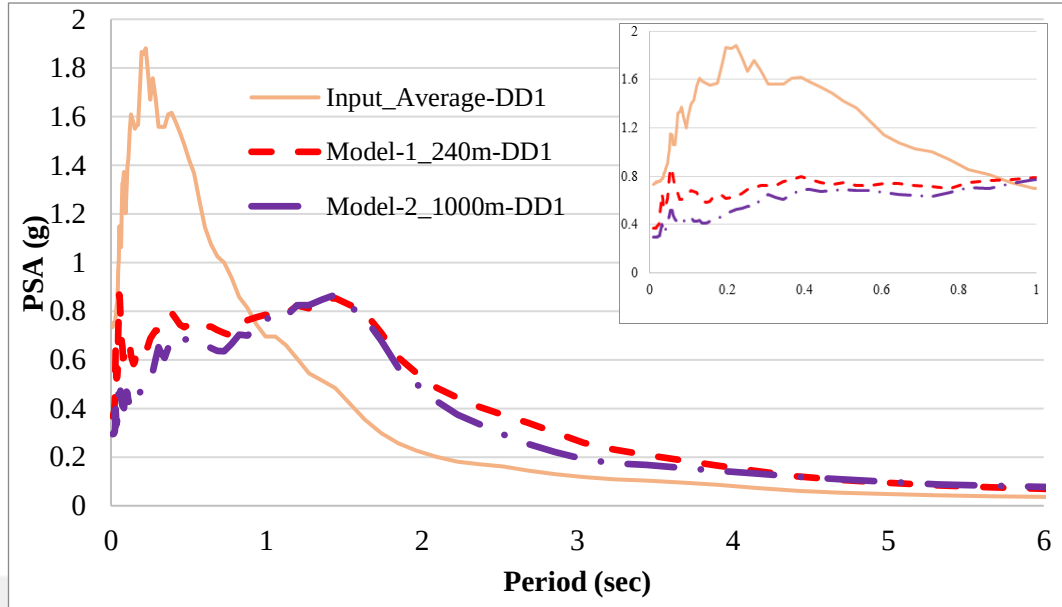


Figure 4.8 Comparison of Model-1 and Model-2 for DD-1 earthquake level

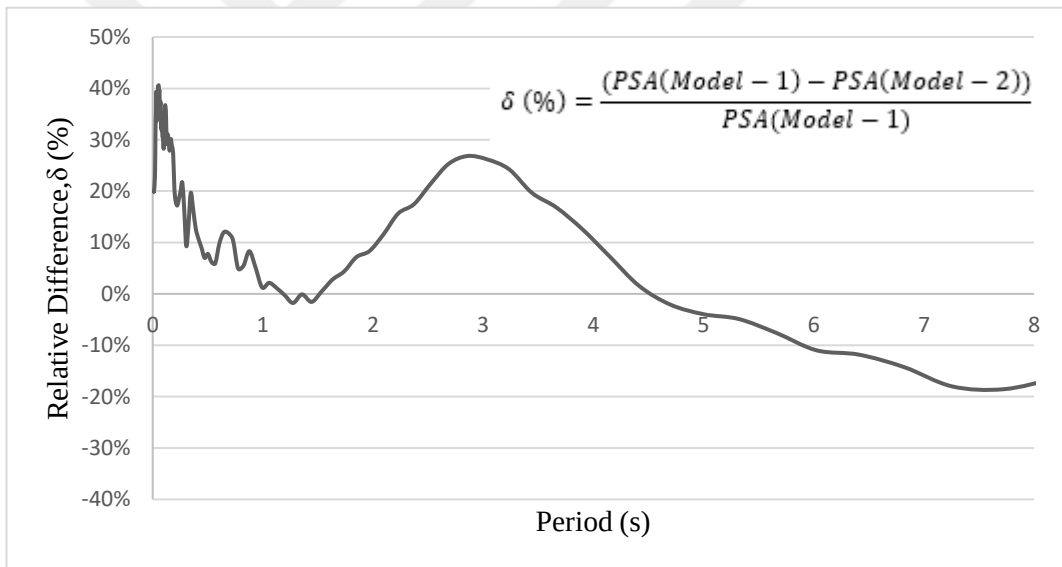


Figure 4.9 Relative difference of between Model-1 and Model-2 for DD-1 earthquake level

4.3.3 Total Stress Analysis Results for DD-2 Earthquake Level

Similar analyses were performed for DD-2 level earthquakes too. The strains, deformations and PGA of 11 earthquakes at DD-2 level are presented in Figure 4.10, Figure 4.11 and Figure 4.12 respectively.

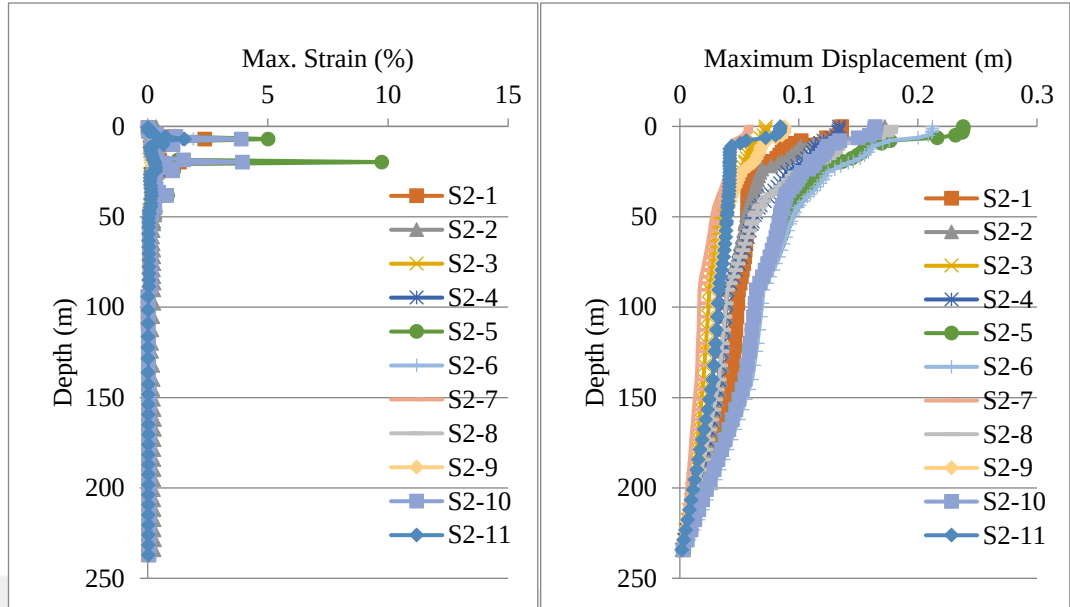


Figure 4.10 Max. strain and max. displacement for Model-1 with DD-2 earthquake level

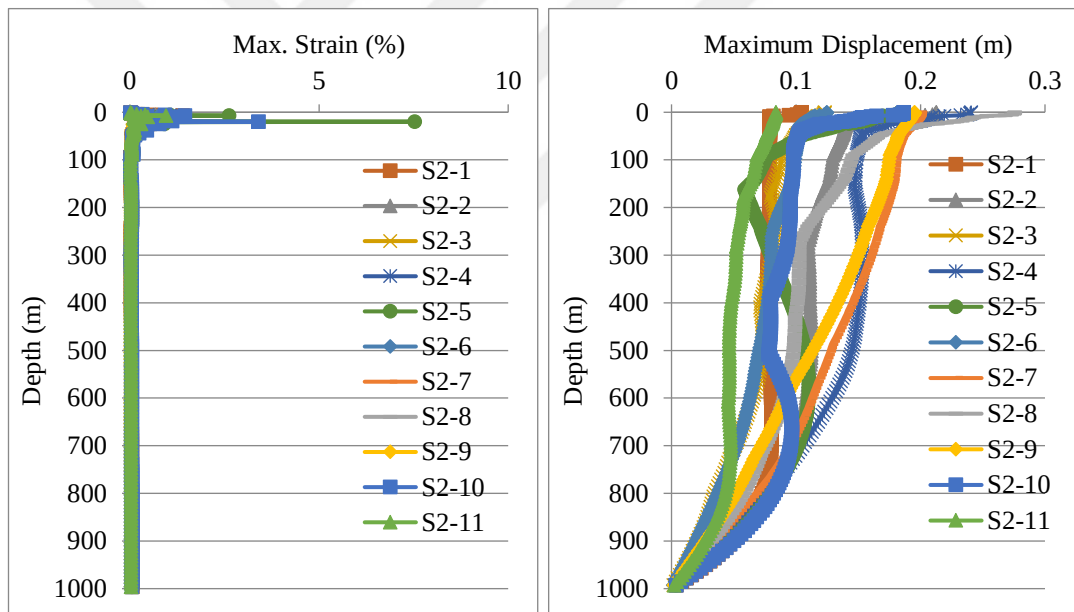


Figure 4.11 Max. strain and max. displacement for Model-2 with DD-2 earthquake level

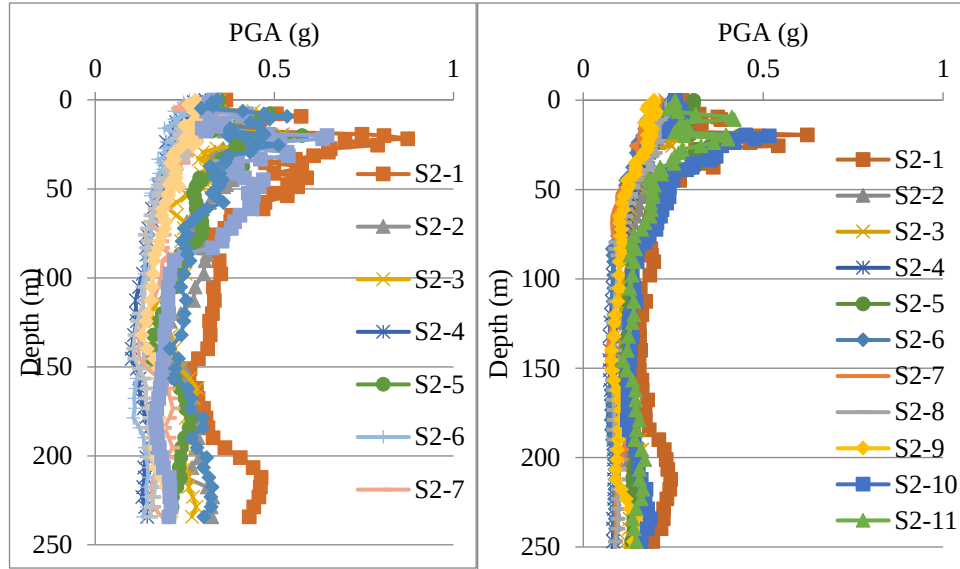


Figure 4.12 PGA value versus depth for a) Model-1 and b) Model-2 with DD-1

Acceleration spectrum of single degree freedom system with 5% damping under DD-2 earthquake level for Model-1 and Model-2 soil profiles are obtained. Results are presented in Figure 4.14 and Figure 4.14.

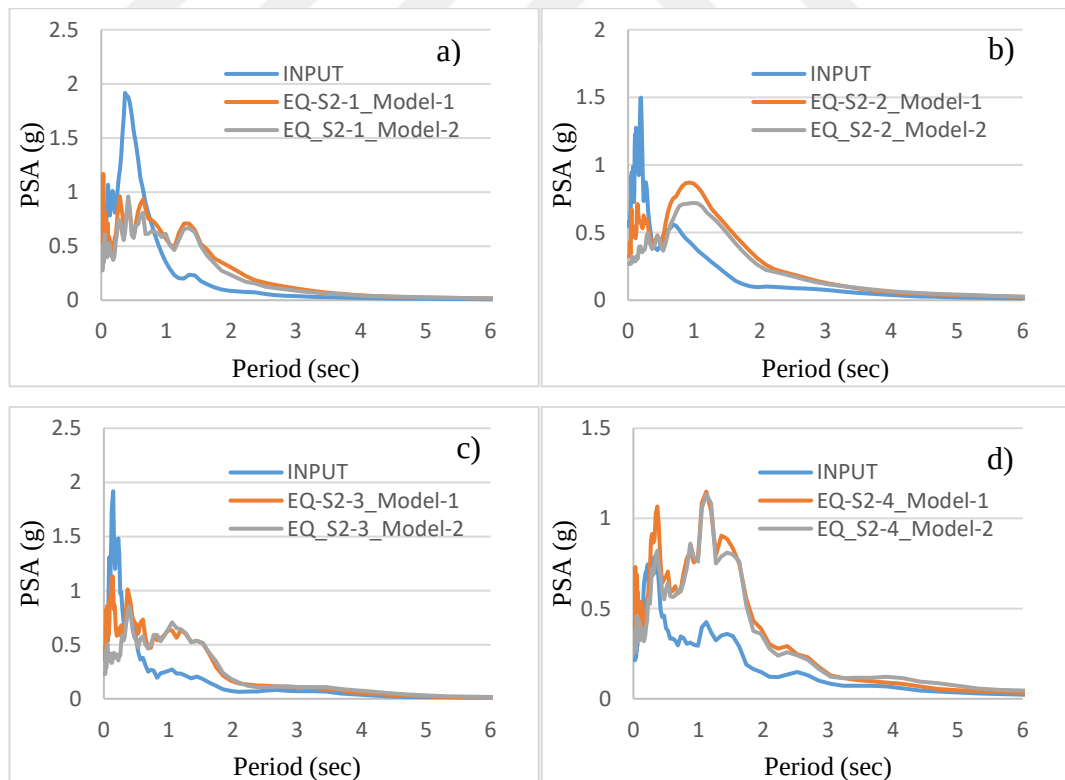


Figure 4.13 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S2-1, b) EQ-S2-2, c) EQ-S2-3, d) EQ-S2-4

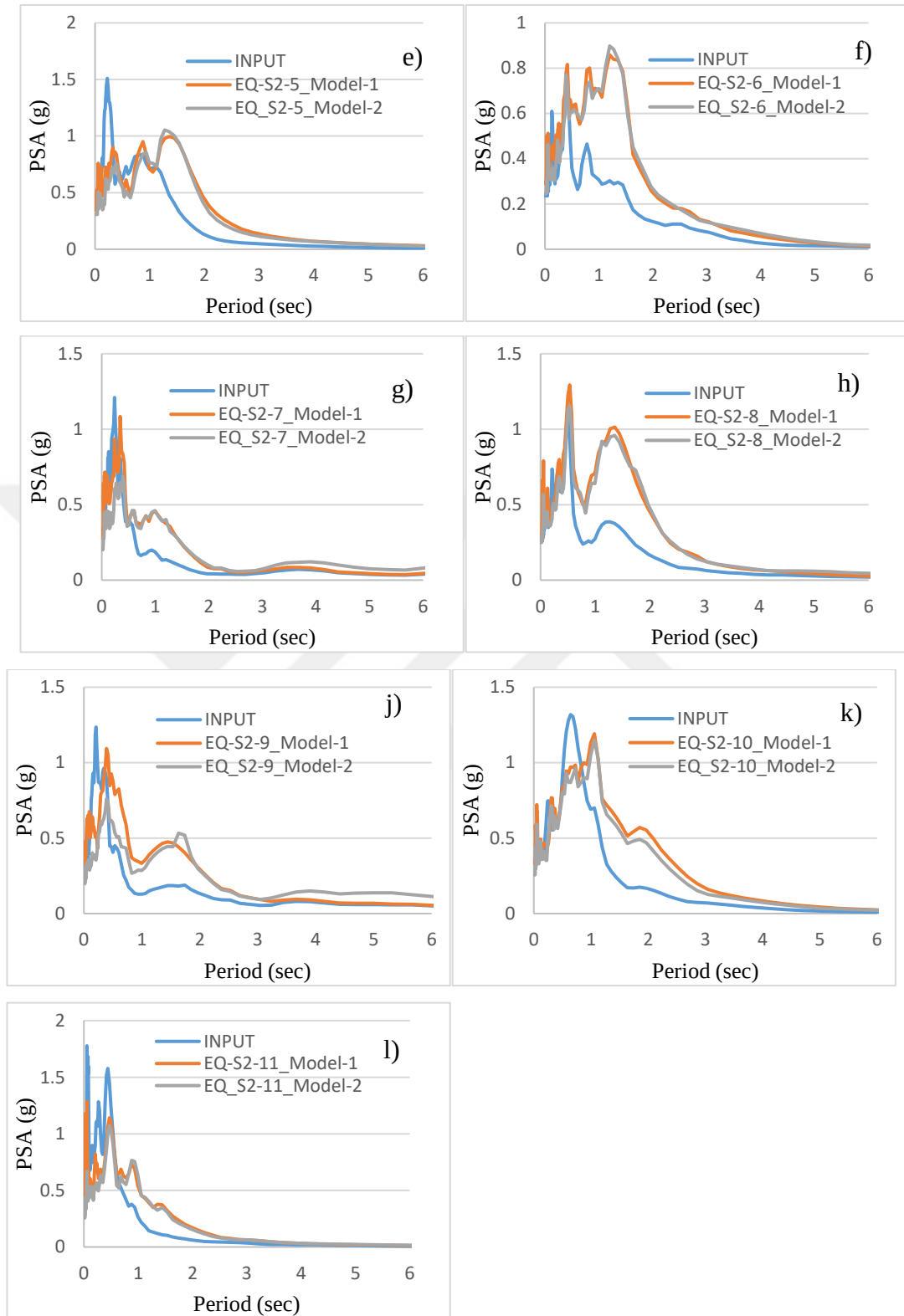


Figure 4.14 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for e) EQ-S2-5, f) EQ-S2-6, g) EQ-S2-7, h) EQ-S2-8, j) EQ-S2-9, k) EQ-S2-10, l) EQ-S2-11

Acceleration spectrum of Model-1 and Model-2 with average of all eleven earthquakes are given in Figure 4.15 and Figure 4.16, respectively.

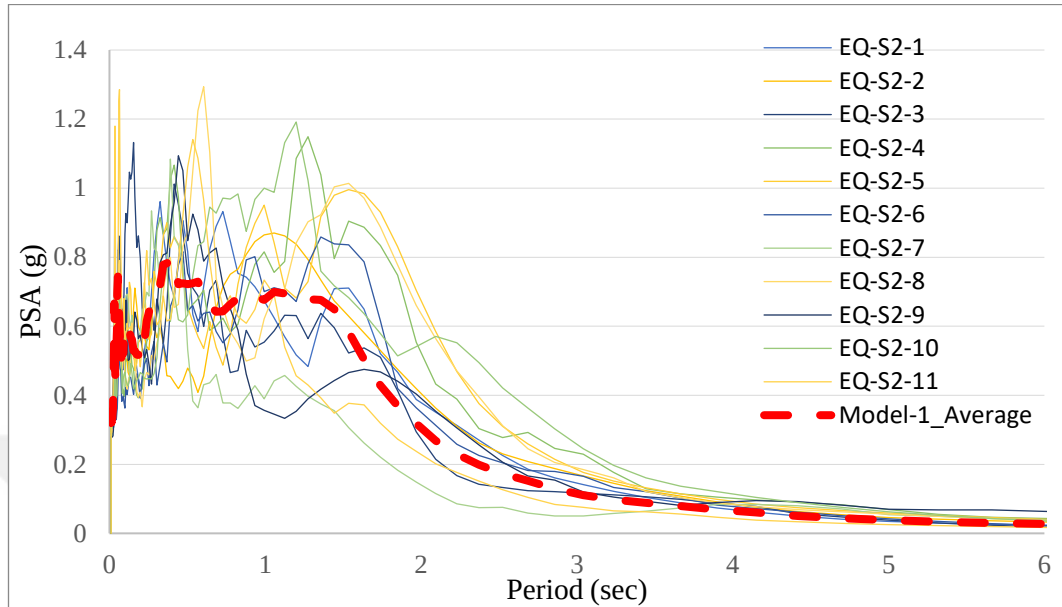


Figure 4.15 Average PSA curve for Model-1 (depth of 240m) with DD-2 earthquake level

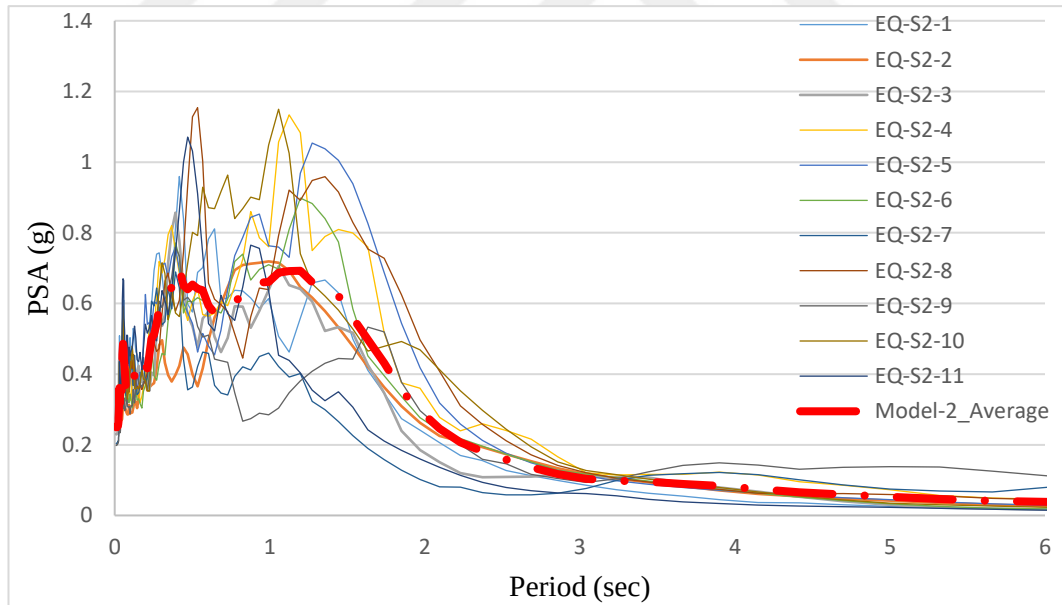


Figure 4.16 Average PSA curve for Model-2 (depth of 1000m) with DD-2 earthquake level

4.3.4 Discussion of Total Stress Analysis Results for DD-2 Earthquake Level

Comparison of Model-1 and Model-2 for DD-2 earthquake level is shown in Figure 4.17. Results show that Model-1 reaches higher accelerations at short period region than Model-2. The difference in short period reaches up to 45%, but average difference

is about 25%. The difference of two models in short period region is shown Figure 4.18. The difference is between Model-1 and Model-2 is due to higher length that waves propagate in Model-2, therefore exposed to higher de-amplification.

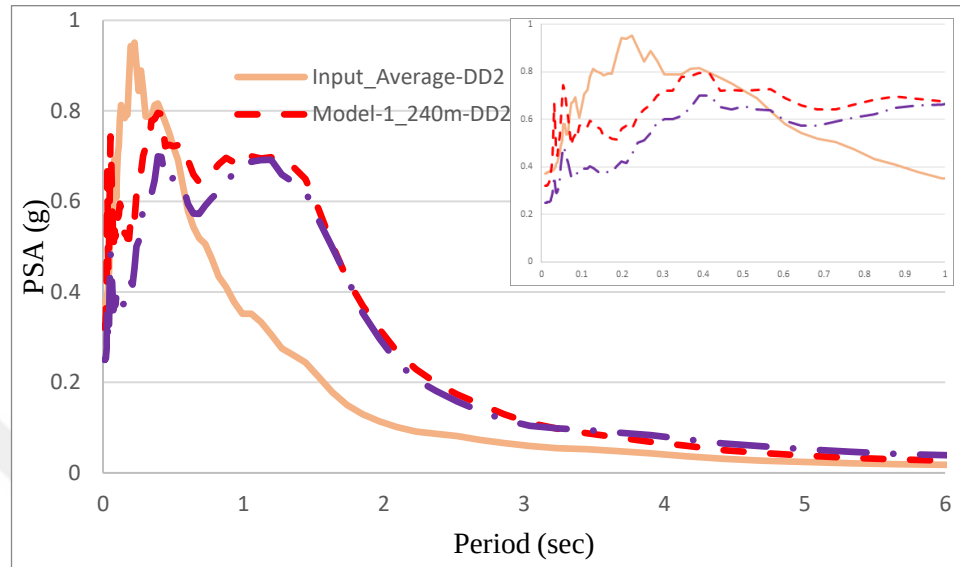


Figure 4.17 Comparison of Model-1 and Model-2 for DD-2 earthquake level

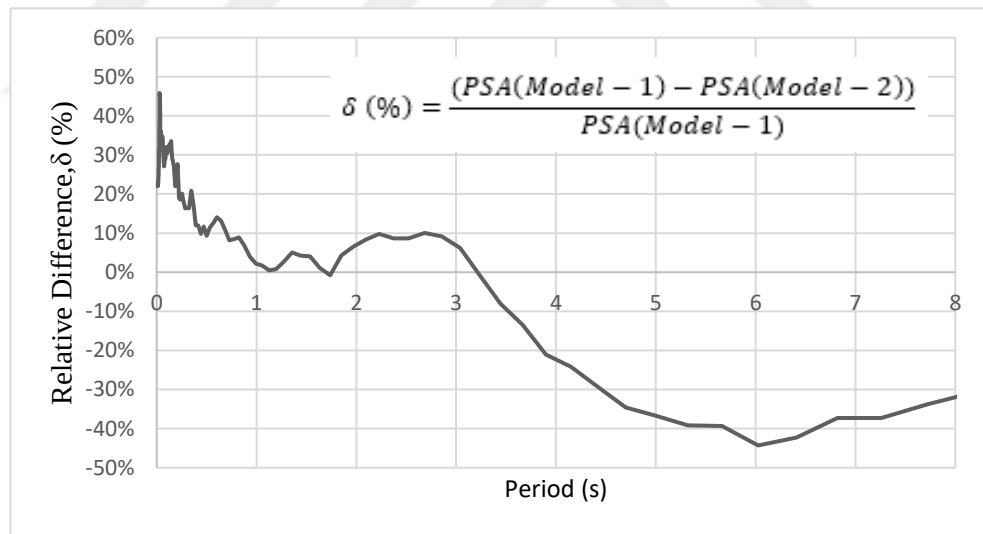


Figure 4.18 Relative difference of between Model-1 and Model-2 for DD-2 earthquake level

4.4 Effective Stress Analysis Results

Required parameters for effective stress analysis are presented in previous chapters. These parameters are used for calculation of excess pore pressure, hence by subtracting the excess pore pressure from total pressure, reaching to effective stress on the soil. Analyses were carried out for Model-1 and Model-2 soil profiles.

4.4.1 Effective Stress Analysis Results for DD-1 Earthquake Level

Maximum PWP ratios (u/σ'_v) and corresponding maximum strains, displacements and accelerations for DD-1 earthquake level are presented for Model-1 and Model-2 in Figure 4.19 and Figure 4.20, respectively.

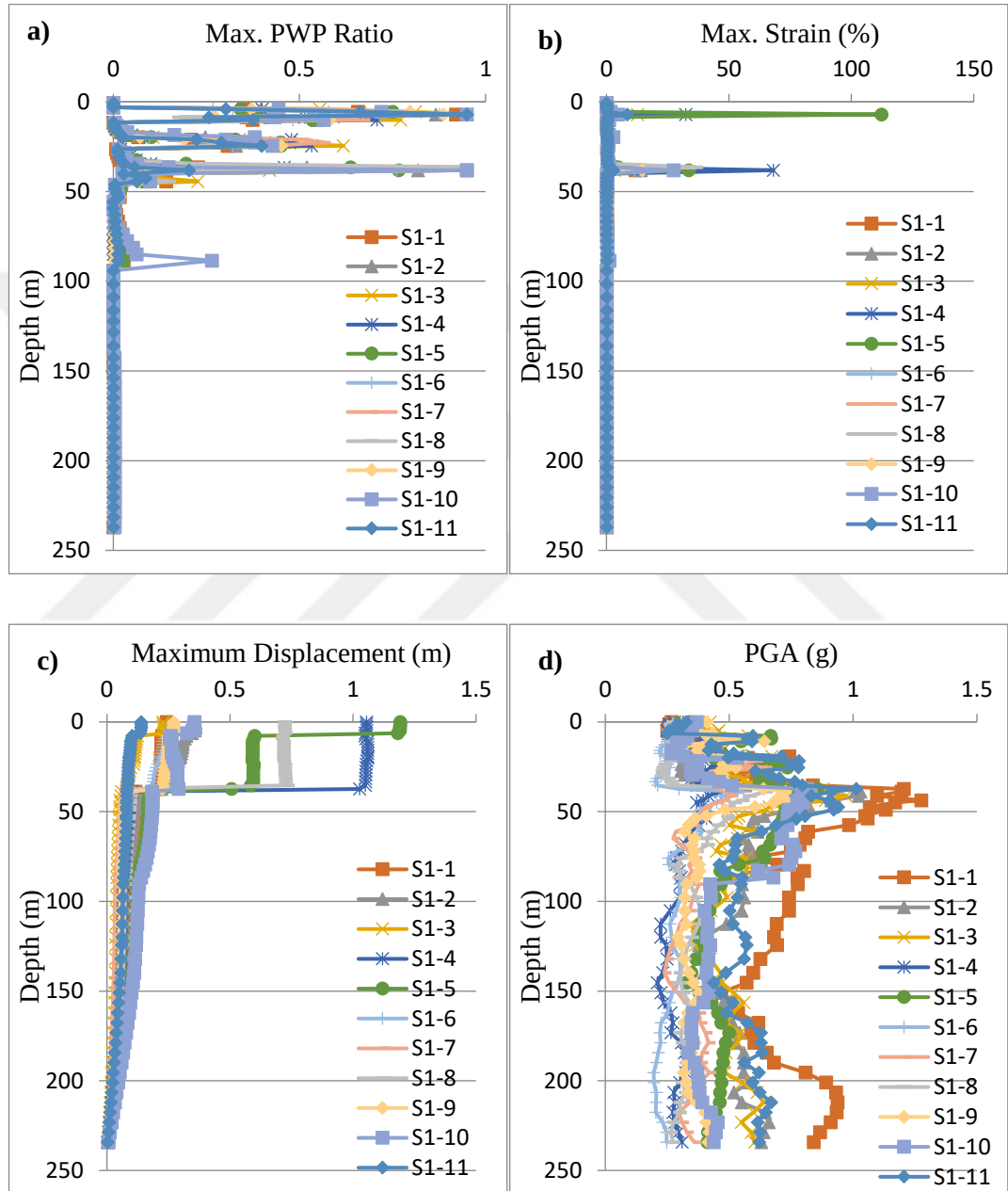


Figure 4.19 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-1 with DD-1 earthquake level

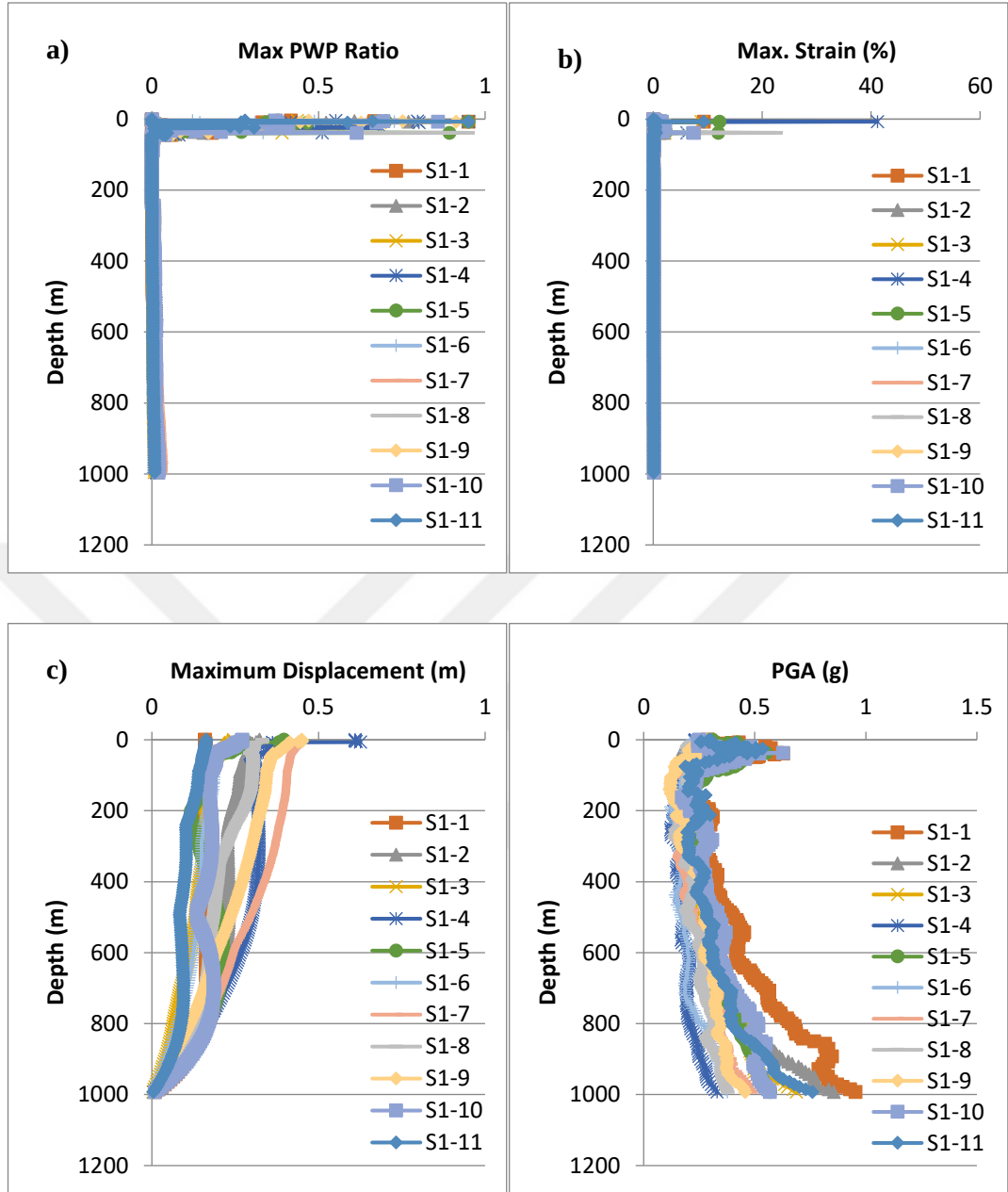


Figure 4.20 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-2 with DD-1 earthquake level

Acceleration spectrum of single degree freedom system with 5% damping under DD-1 earthquake level for Model-1 and Model-2 soil profiles are obtained. Results are presented in Figure 4.21 and Figure 4.22.

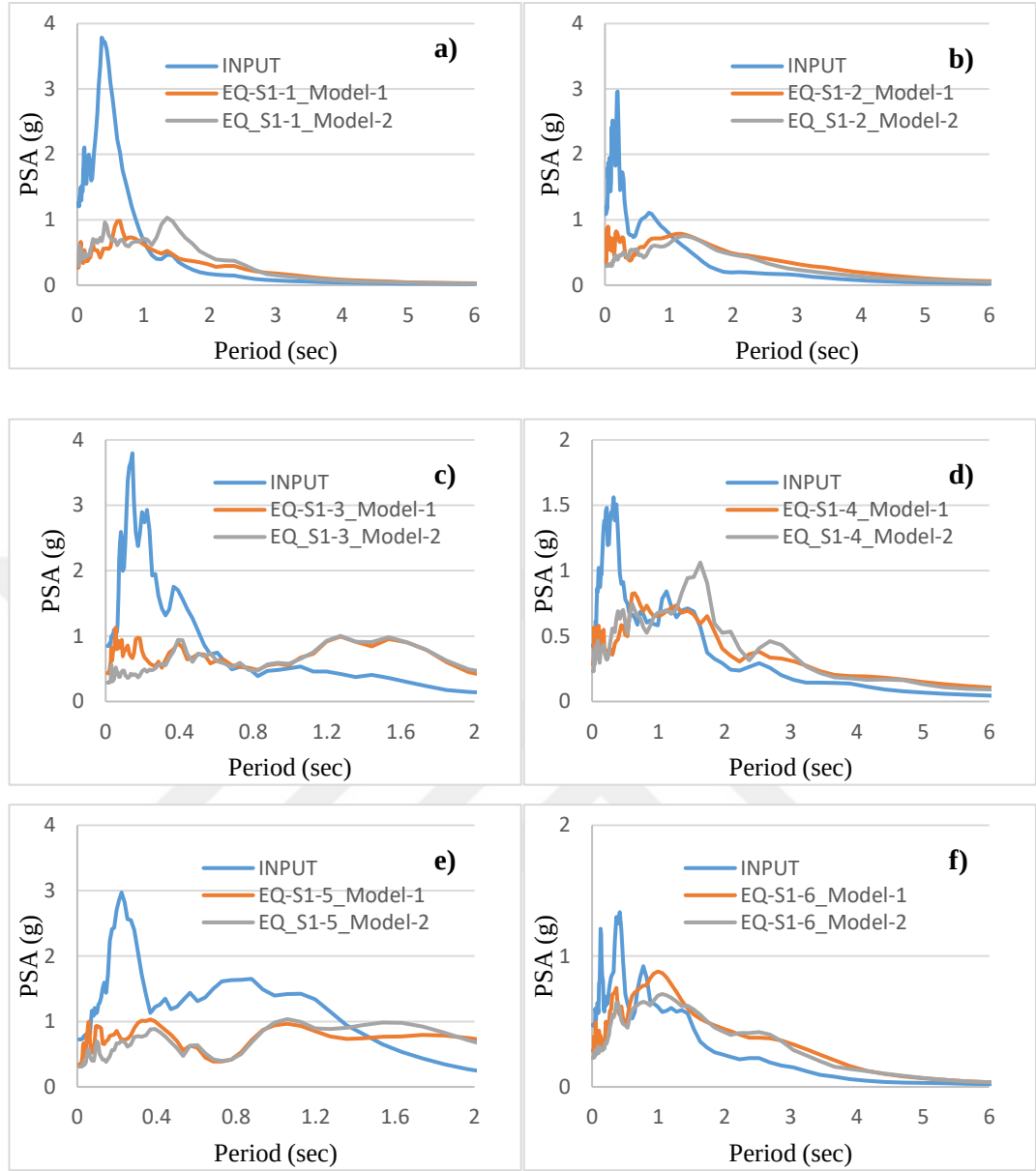


Figure 4.21 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S1-1, b) EQ-S1-2, c) EQ-S1-3, d) EQ-S1-4, e) EQ-S1-5, f) EQ-S1-6

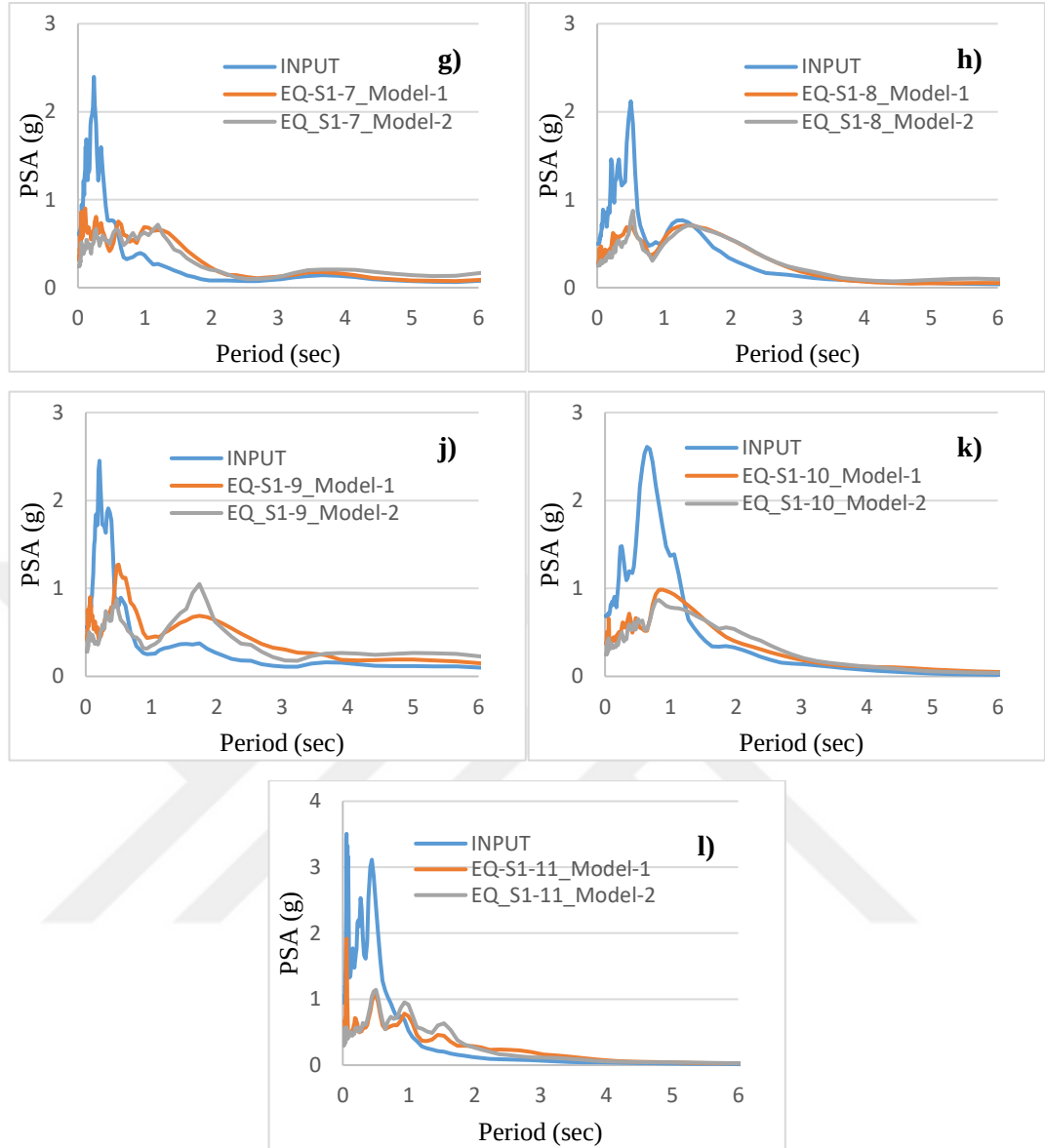


Figure 4.22 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for g) EQ-S1-7, h) EQ-S1-8, j) EQ-S1-9, k) EQ-S1-10, l) EQ-S1-11

Acceleration spectrum of Model-1 and Model-2 with average of all eleven earthquakes at DD-1 level are given in Figure 4.23 and Figure 4.24.

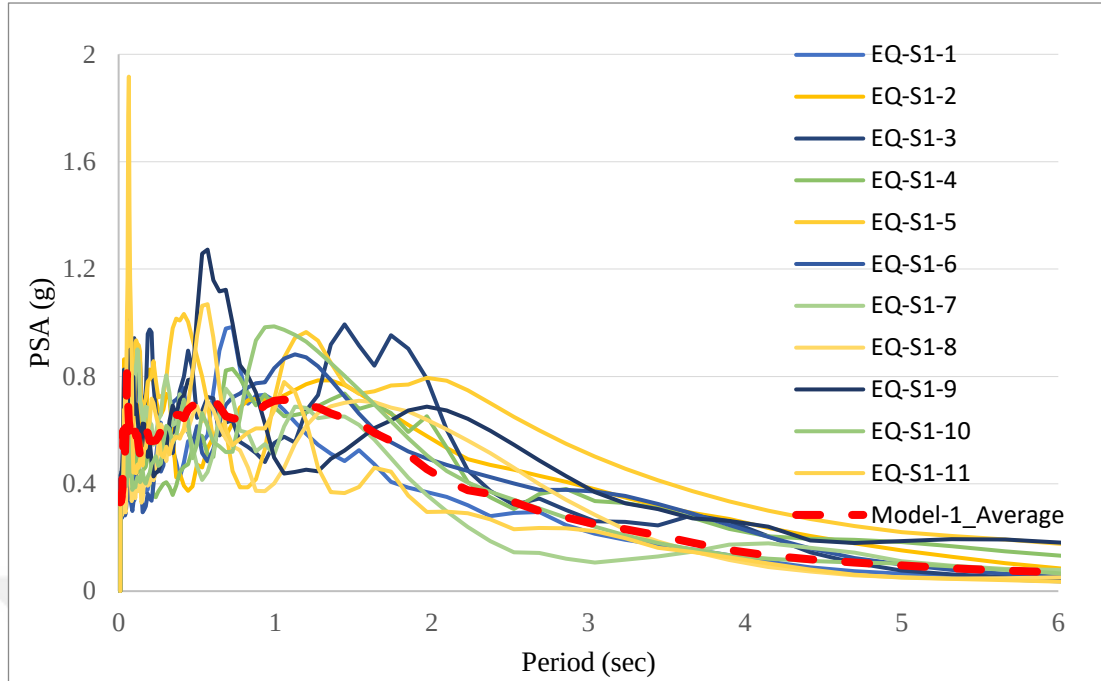


Figure 4.23 Average PSA curve for Model-1 (depth of 240m) with DD-1 earthquake level

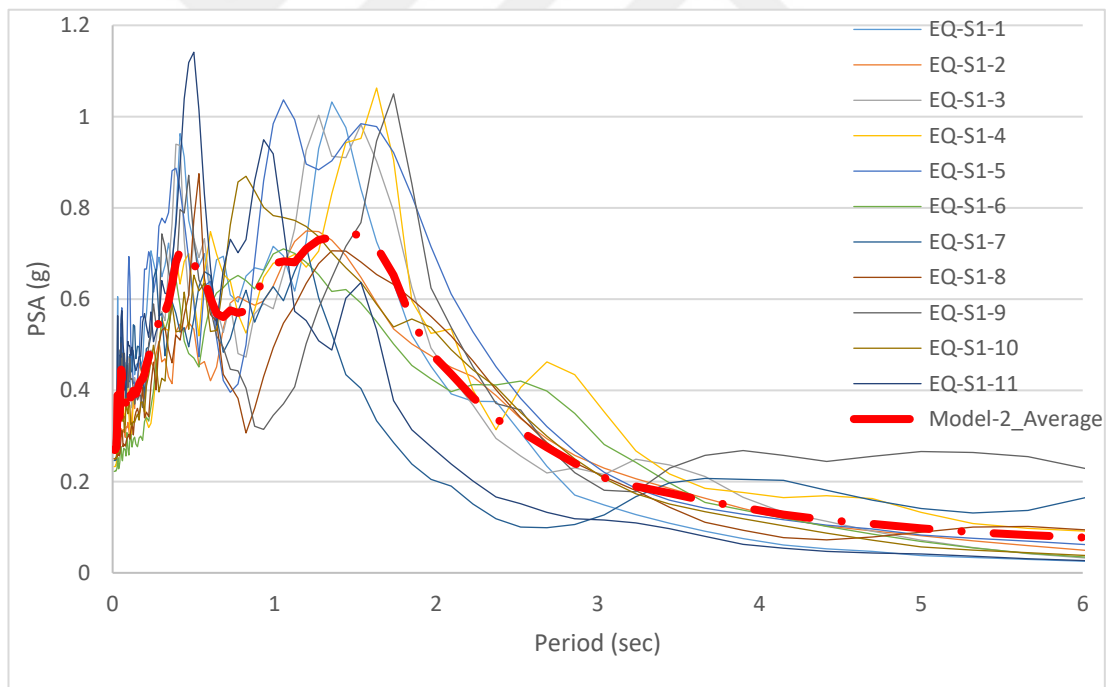


Figure 4.24 Average PSA curve for Model-2 (depth of 1000m) with DD-1 earthquake level

4.4.2 Pore Water Pressure Generation Near Ground Surface for DD-1

Pore water pressure generation results for DD-1 earthquake level is given Figure 4.25 which is for Model-1 (depth of 250m) and Figure 4.26 which is for Model-2

(depth of 1000m). Results show that pore water pressure generation near the ground surface reach the effective stress value in the other words, maximum PWP ratios (u/σ'_v) reaches 1 value. This means that soil loose its shear strength or pore water pressure is equal to effective stress. Consequently, liquefaction occur sand and gravel layers. As a result of the liquefaction, high shear strains occur The obtained most important result from the comparison between Model-1 and Model-2 is strain difference, model-1 has higher shear strain than model-2. Because model-1 is a shallow soil model, has higher PSA value on the surface, so model-1 has higher shear strain than model-2.

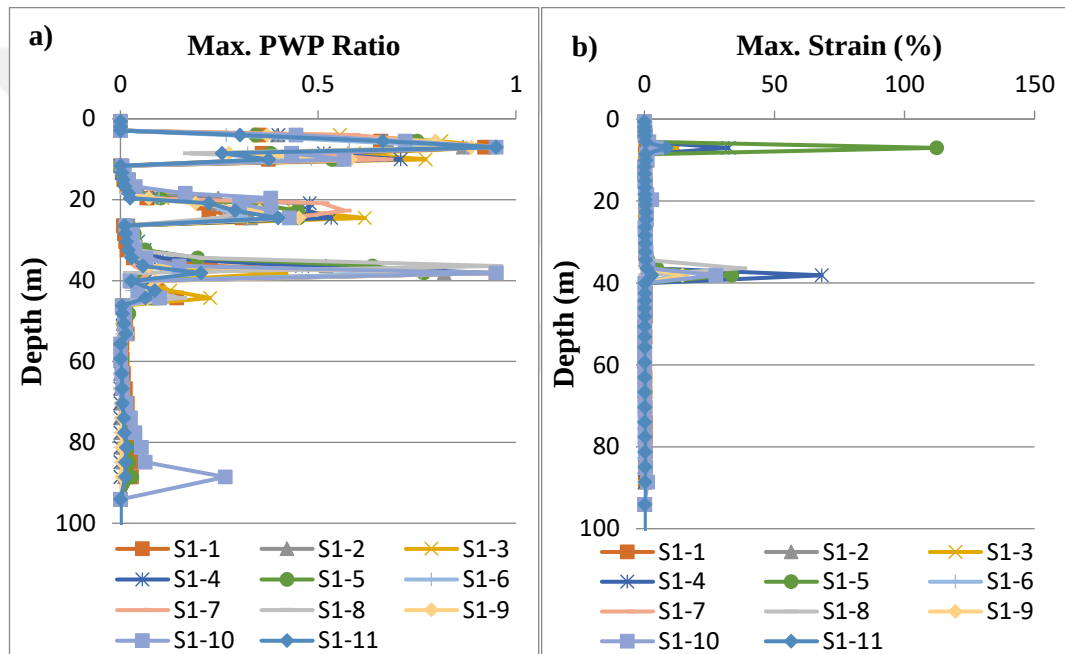


Figure 4.25 Pore water pressure generation near the ground surface for Model-1 (240m)

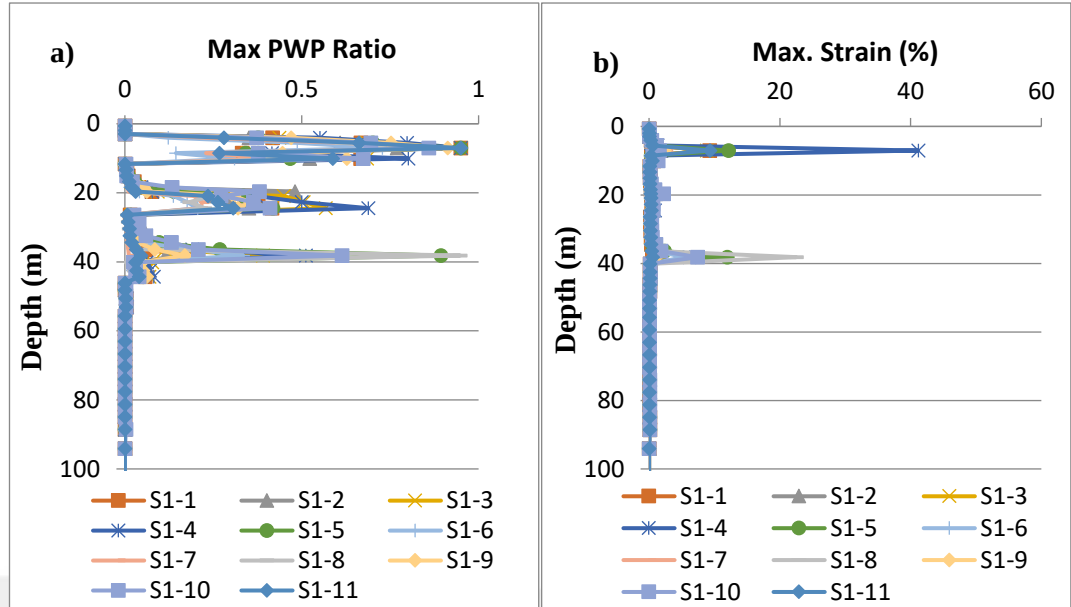


Figure 4.26 Pore water pressure generation near the ground surface for Model-2 (1000m)

4.4.3 Discussion of Effective Stress Analysis Results for DD-1 Earthquake Level

Comparison of Model-1 and Model-2 for DD-1 earthquake level is shown in Figure 4.27. Results show that Model-1 reaches higher accelerations at short period region than Model-2. The difference in short period reaches up to 45%, but average difference is about 20%. Results are similar in long period region. The difference of two models in short period region is shown Figure 4.28. Similar to previous results, due to higher soil column length of Model-2, waves are deamplified more than Model-1, resulting in smaller accelerations.

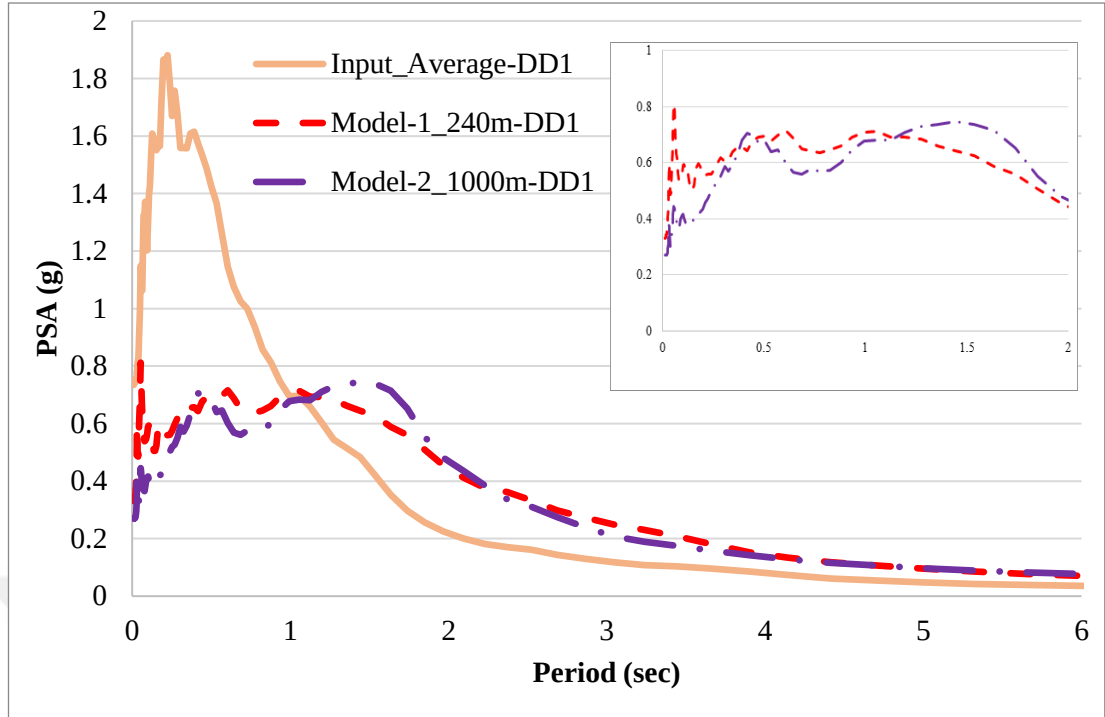


Figure 4.27 Comparison of Model-1 and Model-2 for DD-1 earthquake level

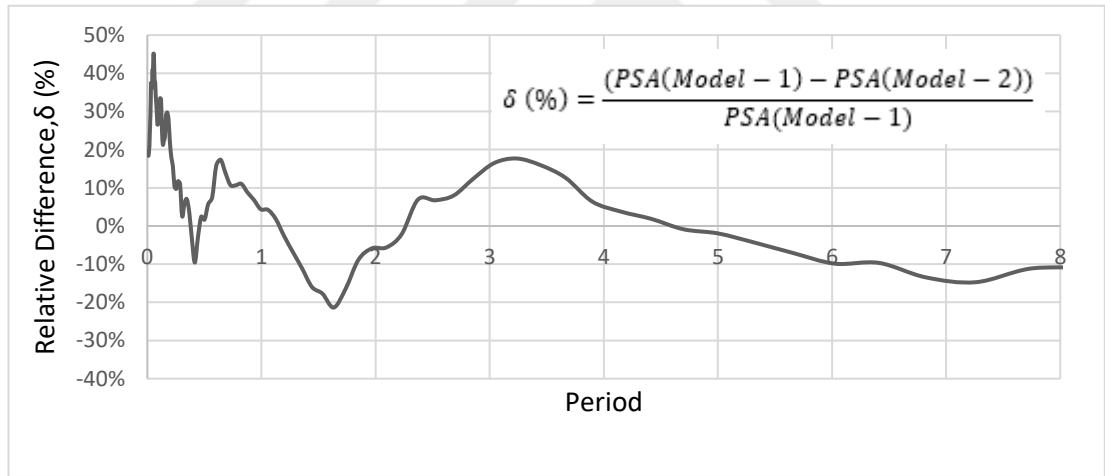


Figure 4.28 Relative difference of between Model-1 and Model-2 for DD-1 earthquake level

4.4.4 Effective Stress Analysis Results for DD-2 Earthquake Level

Maximum PWP ratios (u/σ'_v) and corresponding maximum strains, displacements and accelerations for DD-2 earthquake level are presented for Model-1 and Model-2 in Figure 4.29 and Figure 4.30, respectively.

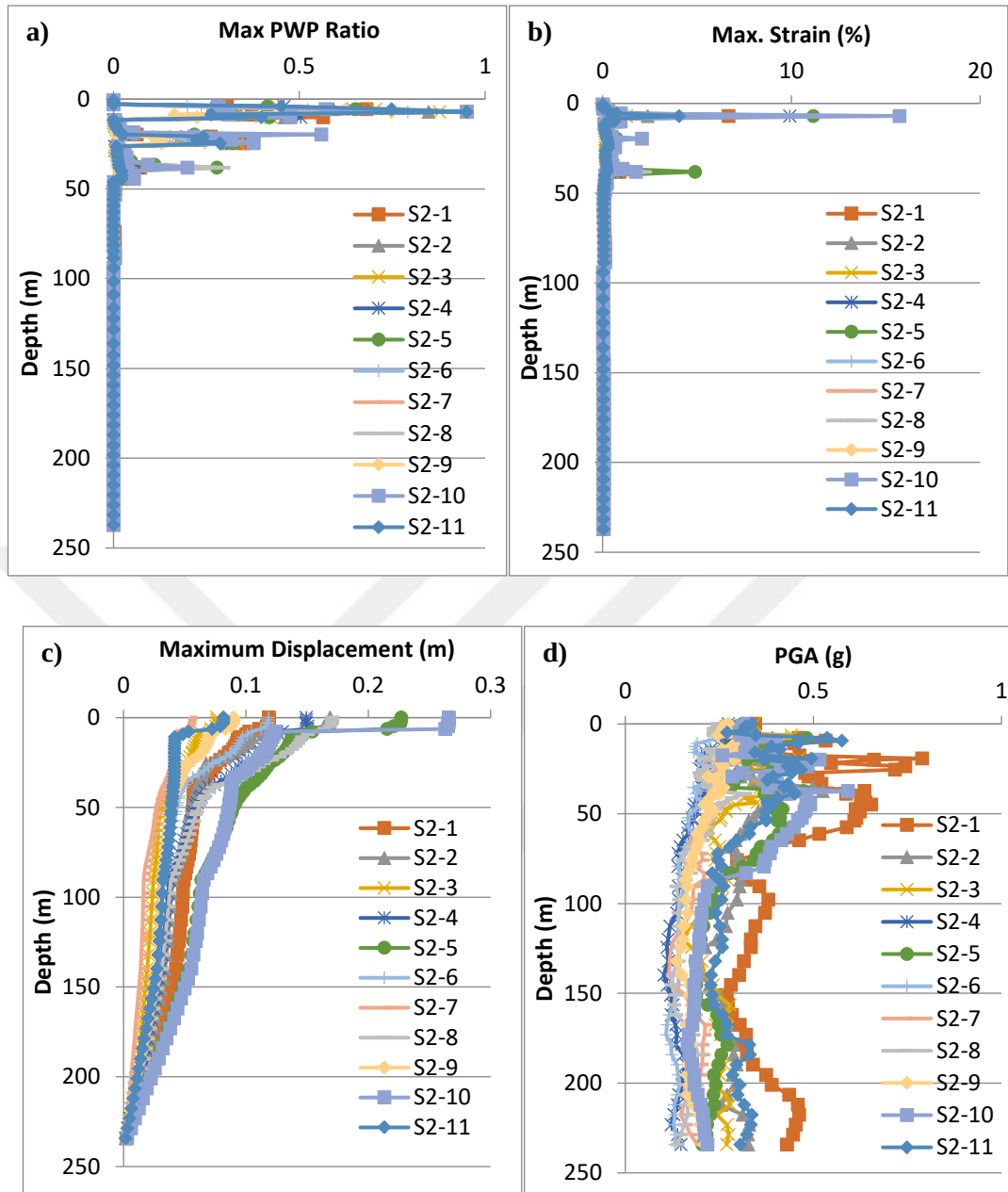


Figure 4.29 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-1 with DD-2 earthquake level

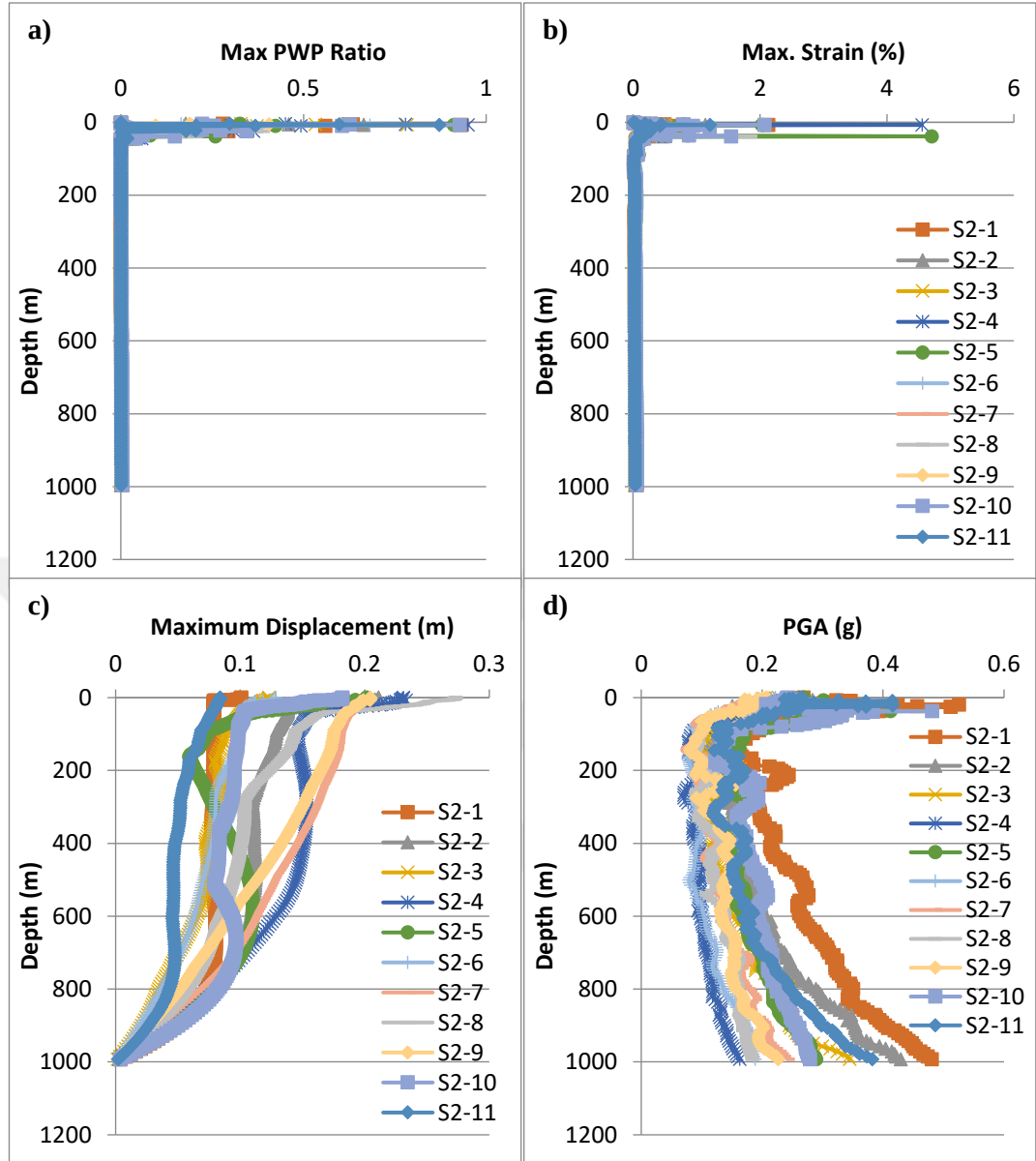


Figure 4.30 a) Max. PWP, b) Max. Strain c) Max. Displacement, d) PGA value for Model-2 with DD-2 earthquake level

Acceleration spectrum of single degree freedom system with 5% damping under DD-2 earthquake level for Model-1 and Model-2 soil profiles were obtained. Results are presented in Figure 4.31 and Figure 4.32.

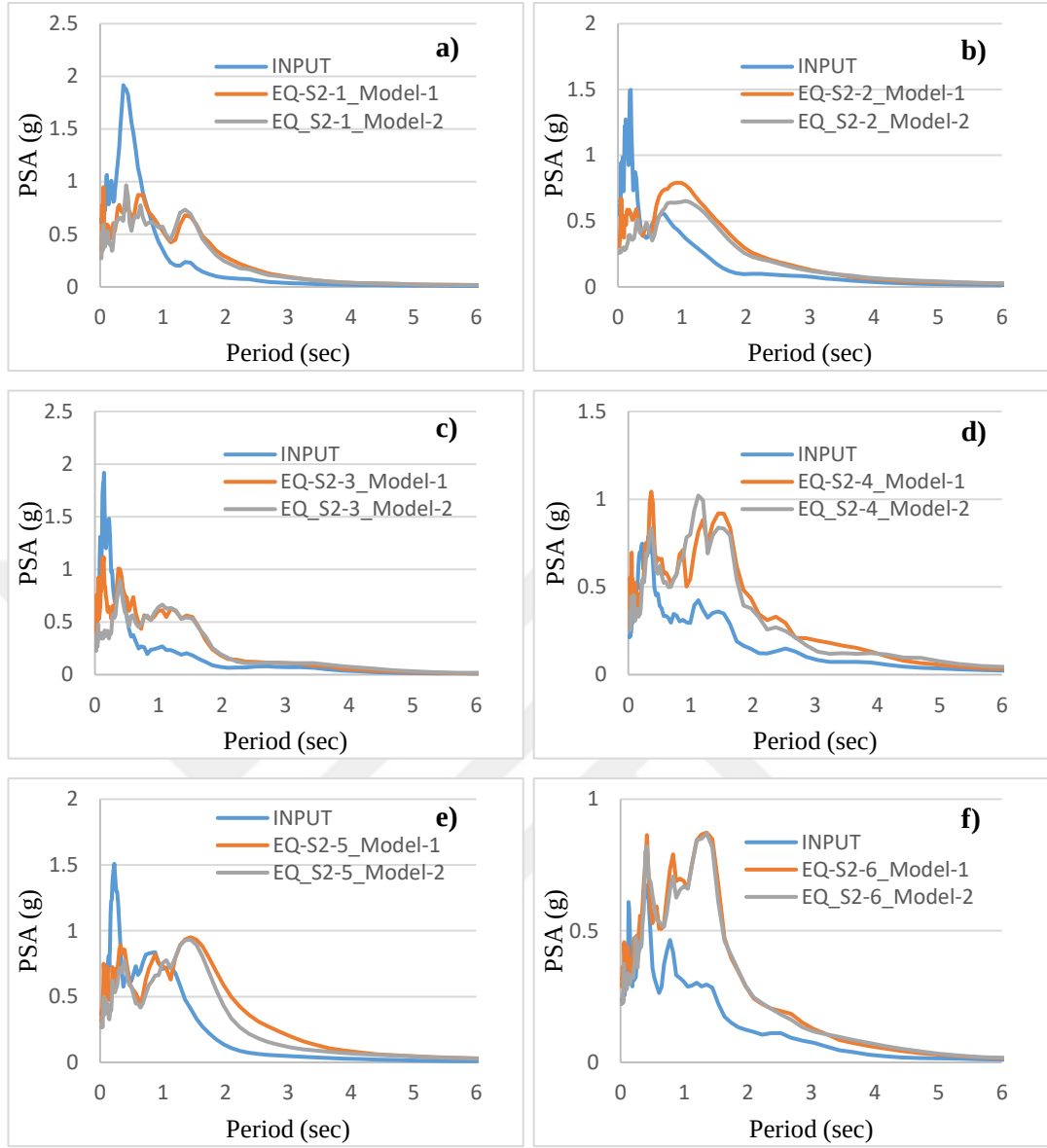


Figure 4.31 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for a) EQ-S2-1, b) EQ-S2-2, c) EQ-S2-3, d) EQ-S2-4, e) EQ-S2-5, f) EQ-S2-6

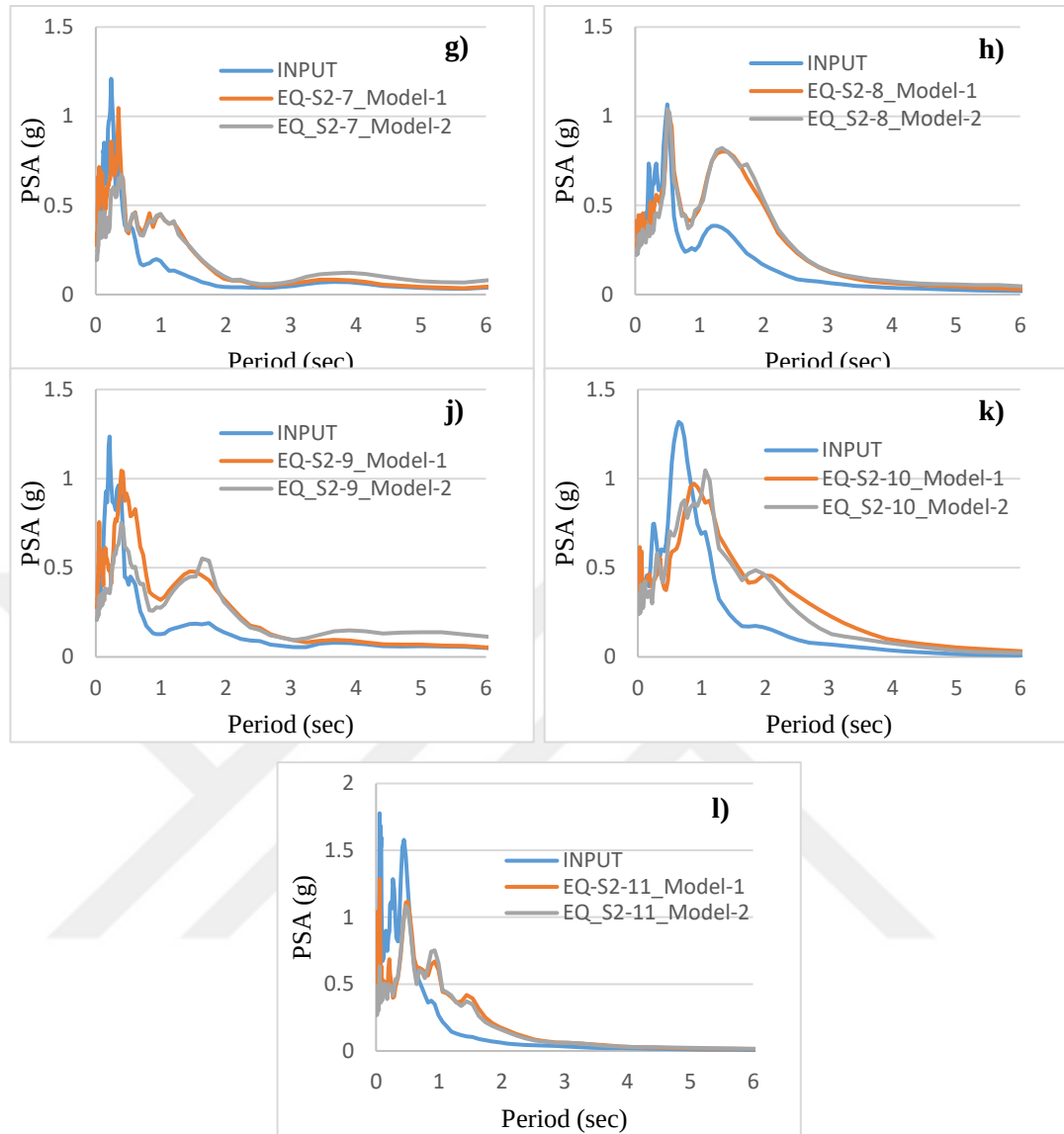


Figure 4.32 PSA Comparison of Model-1 (profil depth of 240m) and Model-2 (profil depth of 1000m) with each of selected earthquake record. PSA spectrum for g) EQ-S2-7, h) EQ-S2-8, j) EQ-S2-9, k) EQ-S2-10, l) EQ-S2-11

Acceleration spectrum of Model-1 and Model-2 with average of all 11 earthquakes at DD-2 level are given in Figure 4.33 and Figure 4.34.

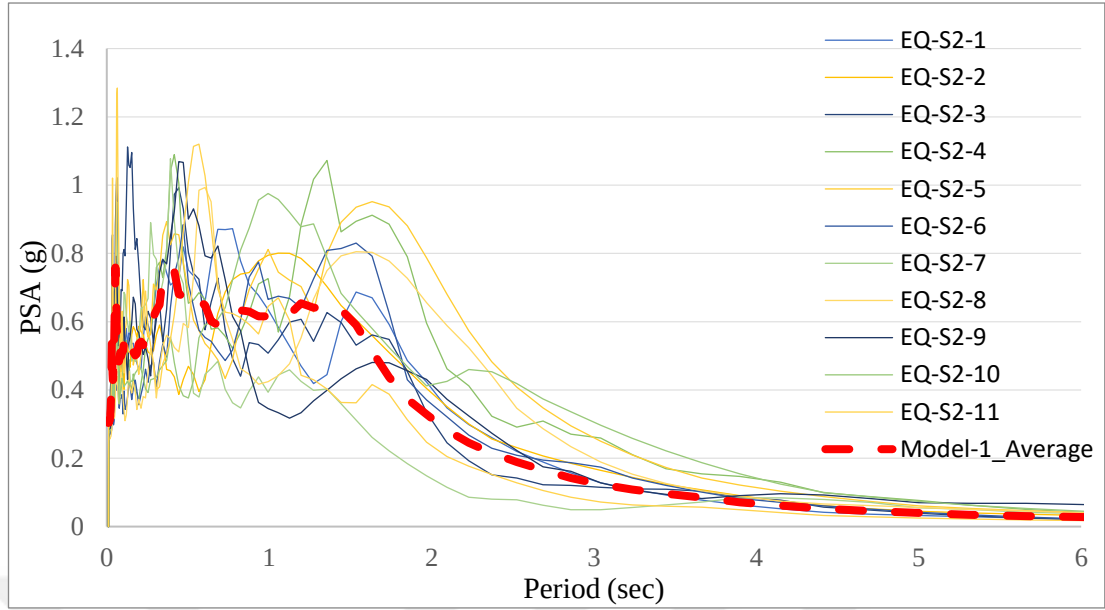


Figure 4.33 Average PSA curve for Model-1 (depth of 240m) with DD-2 earthquake level

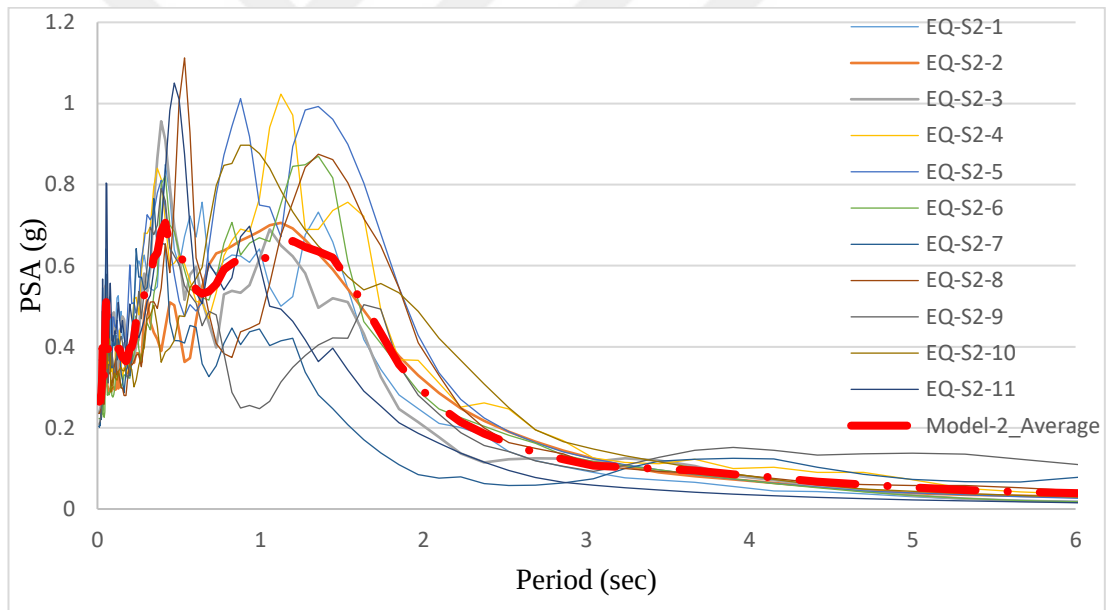


Figure 4.34 Average PSA curve for Model-2 (depth of 1000m) with DD-2 earthquake level

4.4.5 Pore Water Pressure Generation Near Ground Surface for DD-2

Pore water pressure generation results for DD-2 earthquake level is given Figure 4.25 which is for Model-1 (depth of 250m) and Figure 4.26 which is for Model-2 (depth of 1000m). Results show that pore water pressure generation near the ground surface reach the effective stress value in the other words, Maximum PWP ratios (u/σ'_v) reaches 1 value. This means that soil loose shear strength or pore water pressure

is equal to effective stress. Consequently, liquefaction occurs at sand and gravel layers. As a result of the liquefaction, high shear strains occur. The obtained most important result from the comparison between Model-1 and Model-2 is shear strain difference, model-1 has higher shear strain than model-2. Because model-1 is shallow soil model, has higher PSA value on the surface, so model-1 has higher shear strain than model-2. DD-1 earthquake level is the most dangerous earthquake level and has higher PSA value. Hence the obtained most important result from the comparison between DD-1 and DD-2 is strain difference, DD-1 has higher shear strain than DD-2 earthquake level.

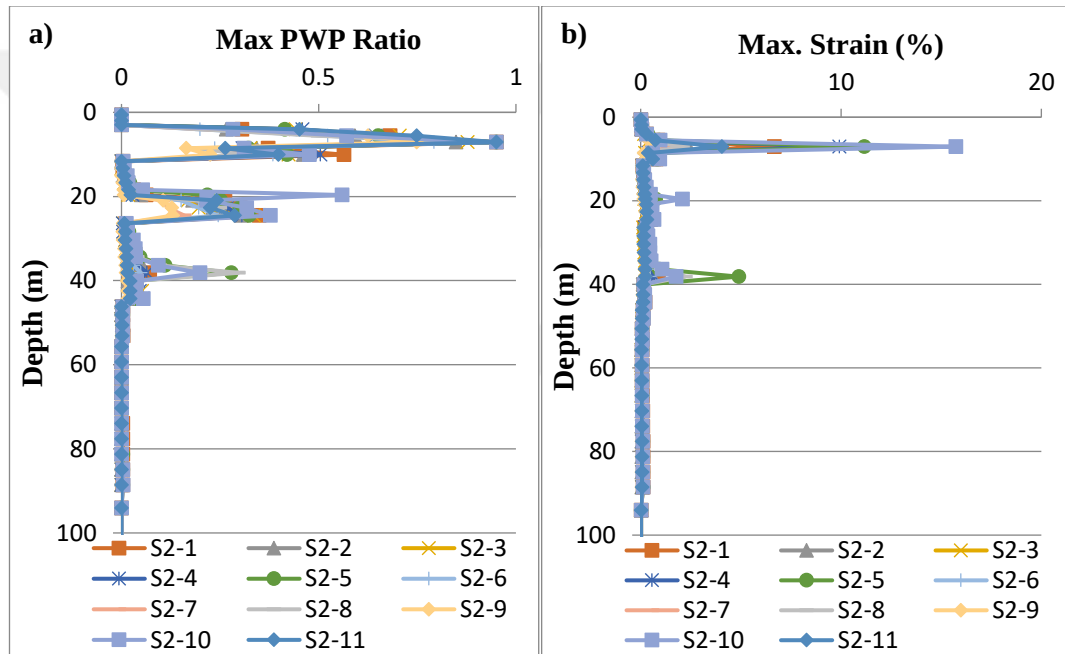


Figure 4.35 Pore water pressure generation near the ground surface for Model-1 (240m), DD-2

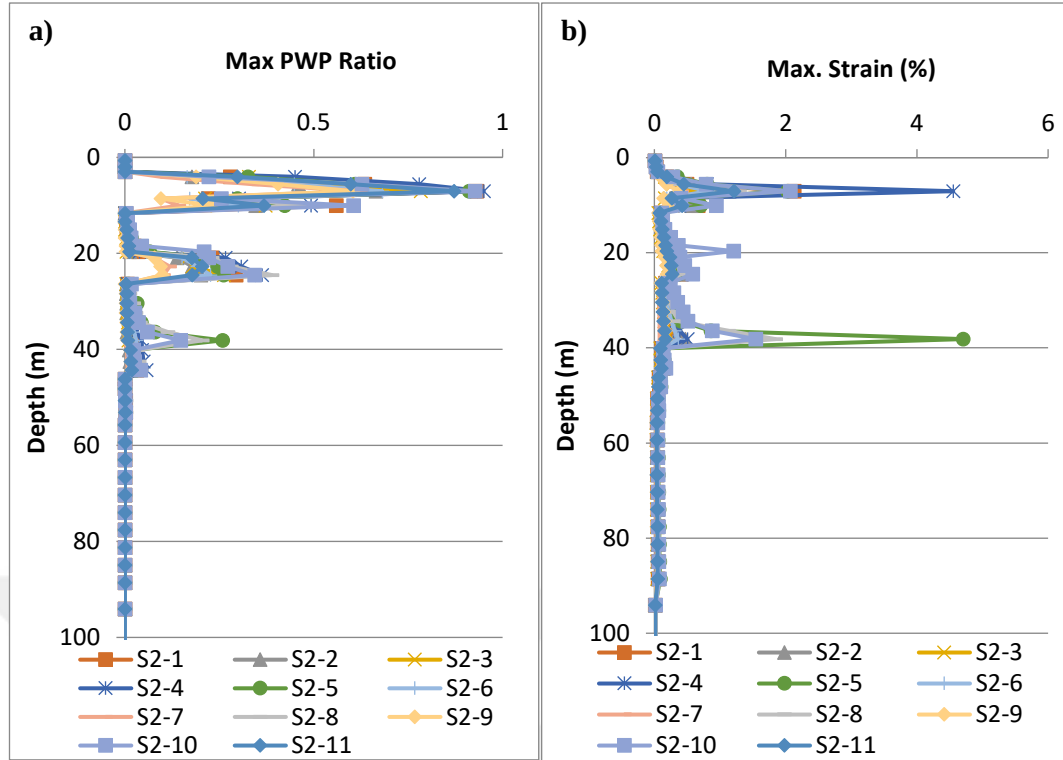


Figure 4.36 Pore water pressure generation near the ground surface for Model-2 (1000m), DD-2

4.4.6 Discussion of Effective Stress Analysis Results for DD-2 Earthquake Level

Comparison of Model-1 and Model-2 for DD-2 earthquake level is shown in Figure 4.37. Results show that Model-1 reaches higher accelerations at short period region than Model-2. The difference in short period reaches up to 40%, but average difference is about 2%. The results are similar in long period region. The difference of two models in short period region is shown Figure 4.37. Similar to previous results, due to higher soil column length of Model-2, waves are deamplified more than Model-1, resulting in smaller accelerations.

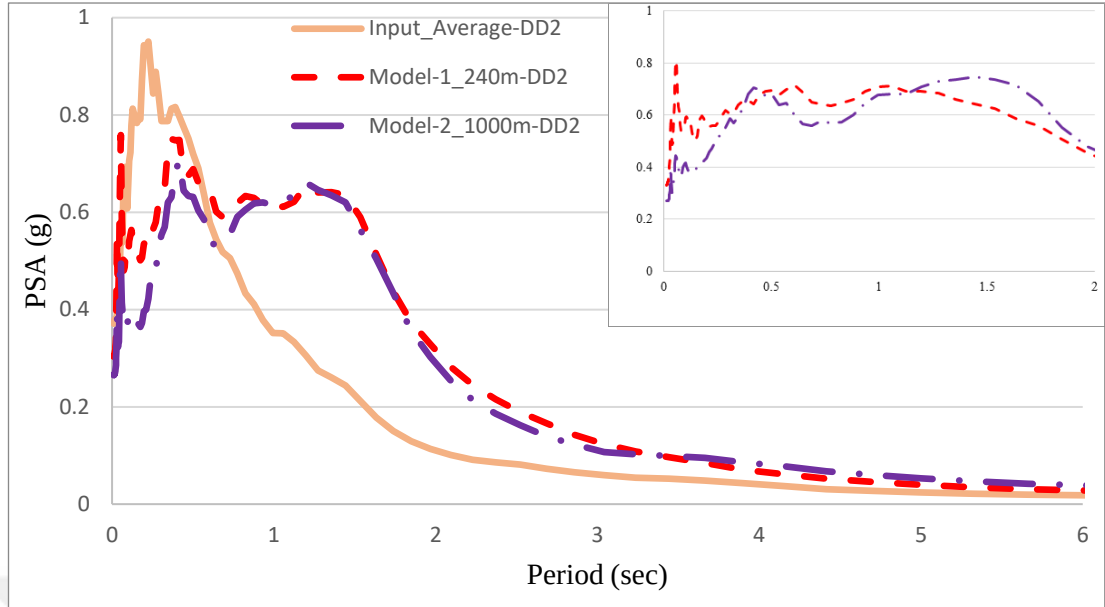


Figure 4.37 Comparison of Model-1 and Model-2 for DD-2 earthquake level

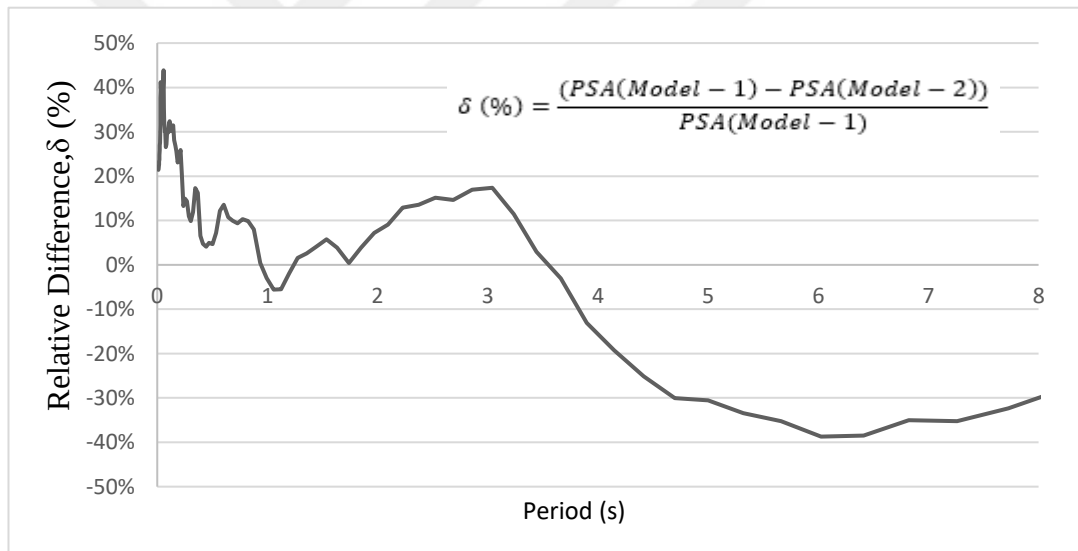


Figure 4.38 Relative difference of between Model-1 and Model-2 for DD-2 earthquake level

4.5 Discussions

In this section, site response analyses considering different methods and bedrock levels are compared which each other. In scope of the study, eleven earthquakes are used with different scaling factors for DD-1 and DD-2 earthquake levels. Effective and total stress based were made. Models with different bedrock levels are compared for effective stress analysis and both earthquake levels.

Comparison of effective stress and total stress analysis methods for Model-1 for two different earthquake levels is given in Figure 4.39. Results show that total stress analysis yields higher accelerations than effective stress analysis. In effective stress analysis, the pore water pressure occurs with the earthquake motion and thus the strength of the soil decreases. The shear strain of soil increase with decreasing strength of the soil. Consequently, in effective stress analysis the earthquake wave is damped more than in the total stress analysis because the damping ratio of the soil increases with increasing shear strain. To present the differences more accurately, relative difference of each analysis method is given in Figure 4.40. Results show that relative differences reach up to 40% and also, relative differences are higher at higher earthquake levels. Therefore, it can be concluded that difference between total and effective stress analysis increases as earthquake levels increase.

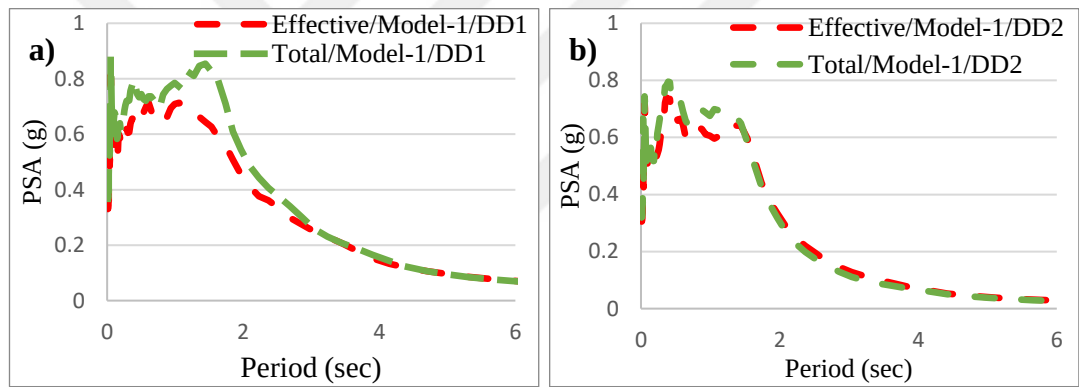


Figure 4.39 Comparison of effective and total stress analysis for Model-1 (240m) with a) DD-1 and b) DD-2 earthquake level

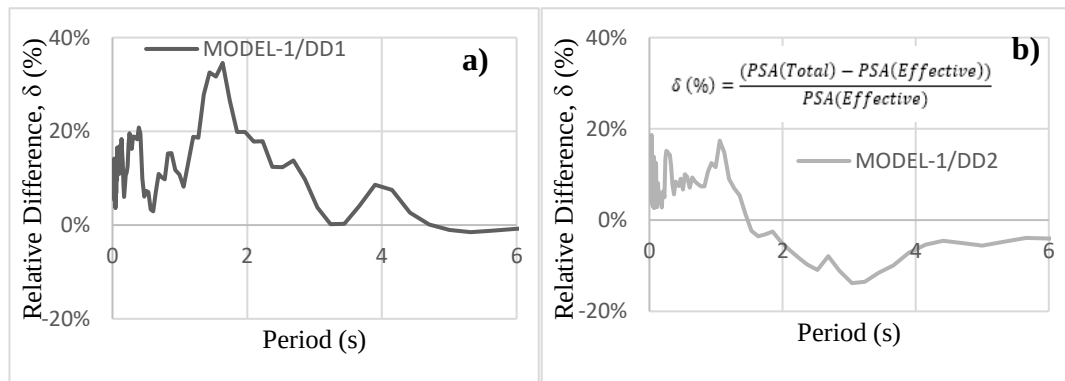


Figure 4.40 Relative difference effective and total stress analysis for Model-1 (240m) with a) DD-1 and b) DD-2 earthquake level

Similar comparisons were performed for Model-2 with bedrock level at 1000m were performed in Figure 4.41 and Figure 4.42. Similar to Model-1 results, total stress analyses yield higher results than effective stress analyses. Same as Model-1, for Model-2 differences were higher for more severe accelerations and earthquake levels.

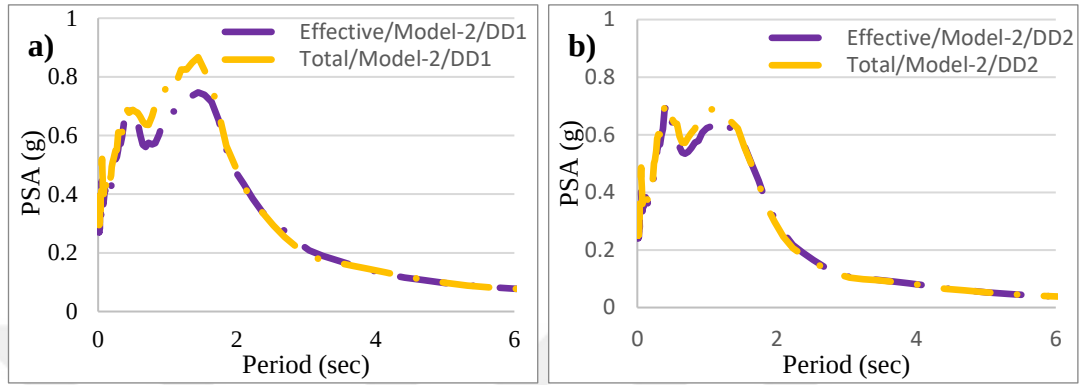


Figure 4.41 Comparison of effective and total stress analysis for Model-2 (1000m) with a) DD-1 and b) DD-2 earthquake level

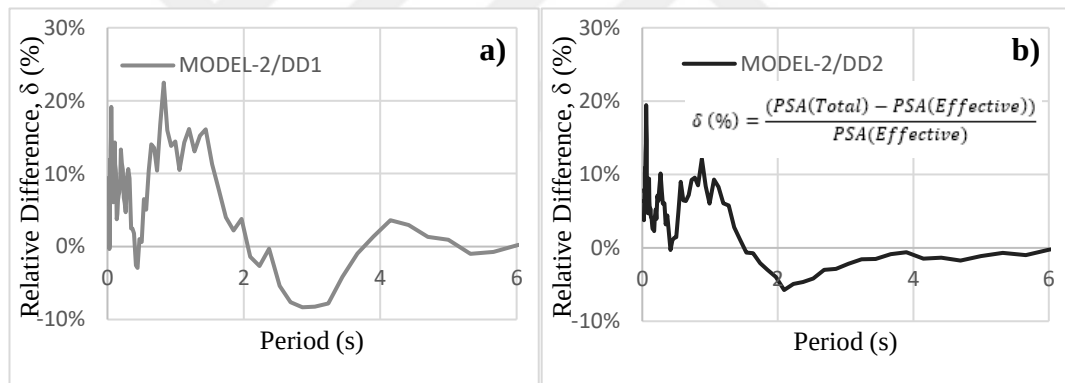


Figure 4.42 Relative difference effective and total stress analysis for Model-2 (1000m) with a) DD-1 and b) DD-2 earthquake level

Pore water pressure generation results is given. Results show that pore water pressure generation near the ground surface reach the effective stress value in the other words, Maximum PWP ratios (u/σ'_v) reaches 1 value. This means that soil loose shear strength or pore water pressure is equal to effective stress. Consequently, liquefaction occur, where the liquefaction occur, soil type is sand and gravel layer. As a result of the liquefaction, high shear strains are occurred. However, clayey layers (38.0m, above of sand and gravel layers) also experienced liquefaction in some depths for DD-1 earthquake level, these results are not consistent with literature on liquefaction and should be approached with caution.

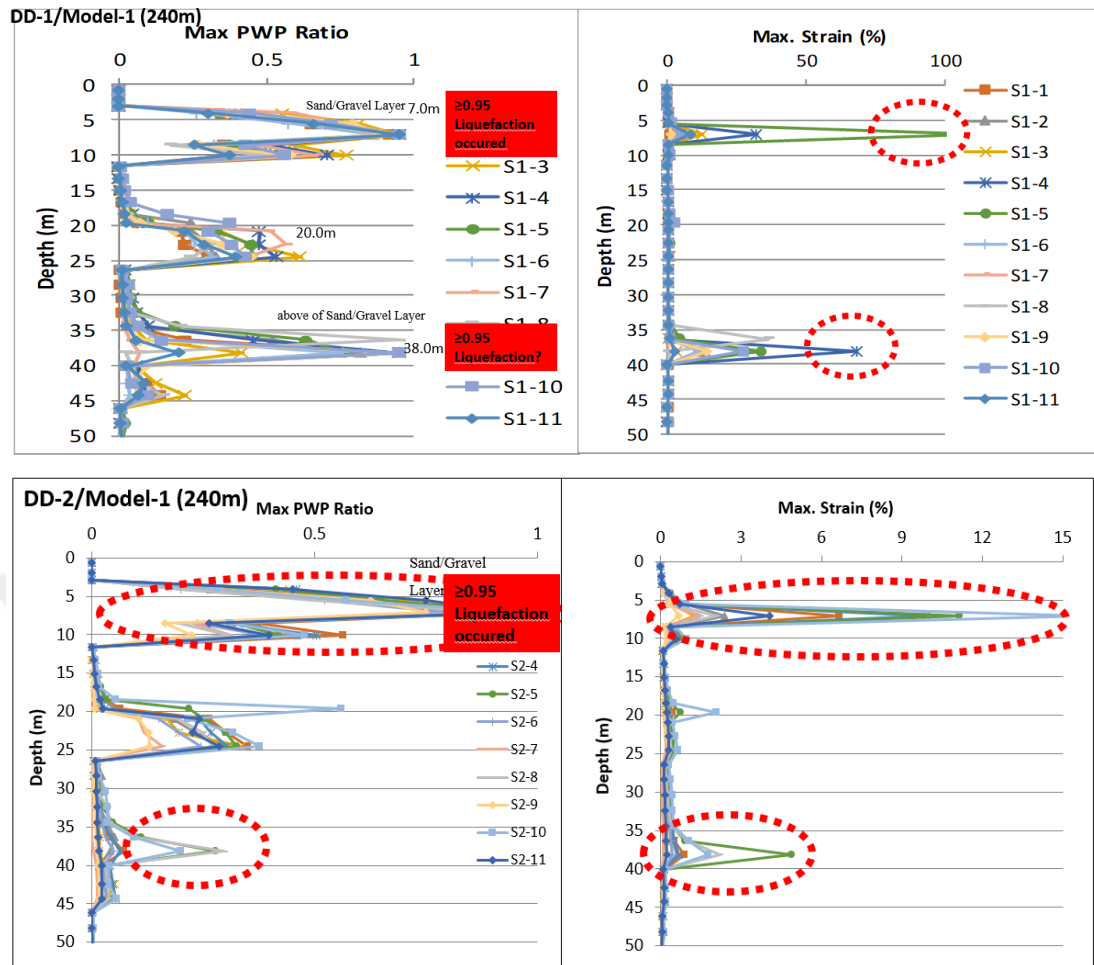


Figure 4.43 Effective stress analysis results for DD-1 and DD-2 earthquake level

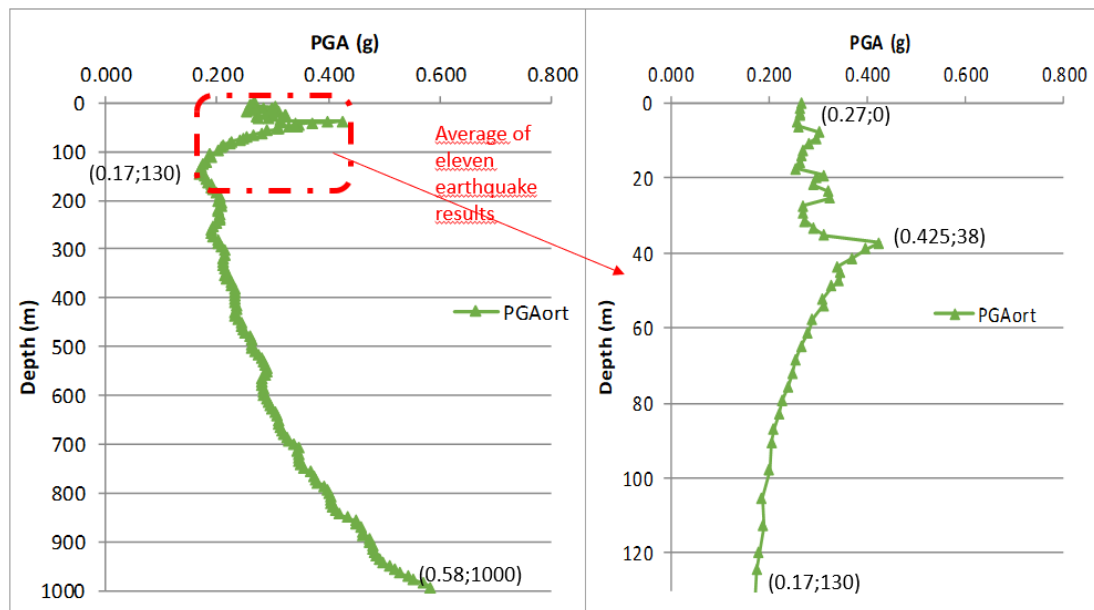


Figure 4.44 The variation of PGA value with depth using effective stress analysis for Model-2 and DD-1 earthquake level

The variation of PGA value with depth using effective stress analysis for Model-2 and DD-1 earthquake level was given Figure 4.44, results show that small accelerations are amplified by soft (low rigidity) soils. However, higher accelerations are deamplified due to softer response and nonlinear behavior of soft soils.

Another aspect of this thesis is to study the effect of different bedrock levels on the response spectrum. In order to fulfill this task, effective stress analyses result of Model-1 and Model-2 are compared for various earthquake levels. Comparison of Model-1 and Model-2 for DD-1 earthquake level using effective stress analyses are presented in Figure 4.45 and relative differences of the models are presented in Figure 4.46. Similar comparisons for DD-2 earthquake level are also presented in Figure 4.47 and Figure 4.48.

Results show that in the low period range, Model-1 with 240m bedrock level yields higher accelerations than Model-2 with 1000m bedrock level. However, in large range period Model-2 yields higher accelerations. Firstly, it can be observed that natural periods of the models are not effective in the response spectrum. For Model-1's natural period 2.121 sec, Model-2's spectral acceleration is 5% higher than Model-1's acceleration. Also, the relative differences between each model change significantly for period ranges. Periods can be separated into 4 zones for DD-1 earthquake level. Between 0 to 1.15sec, Model-1's accelerations are higher than Model-2's mostly. From 1.15 to 2.23 sec and after 4.69 sec, Model-2 governs. For DD-2 earthquake level, two different behavior are observed. Between 0-3.6 sec, Model-1 with shallow bedrock level governs, while for longer periods, Model-2 governs. Therefore, basic rules of thumbs cannot be applied here. However, it can be observed that for skyscrapers and high-rises, where natural period the building is over 4.7 sec, deeper soil column results in higher accelerations. However, for short period range, shallow soil columns yield higher response spectrum.

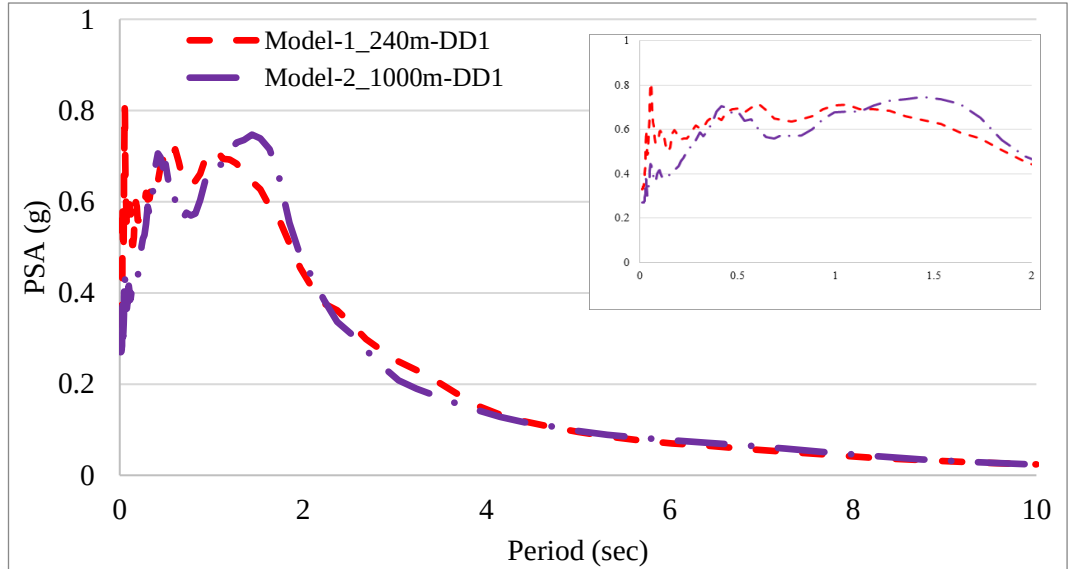


Figure 4.45 Comparison of Model-1 and Model-2 for DD-1 earthquake level using effective stress analysis

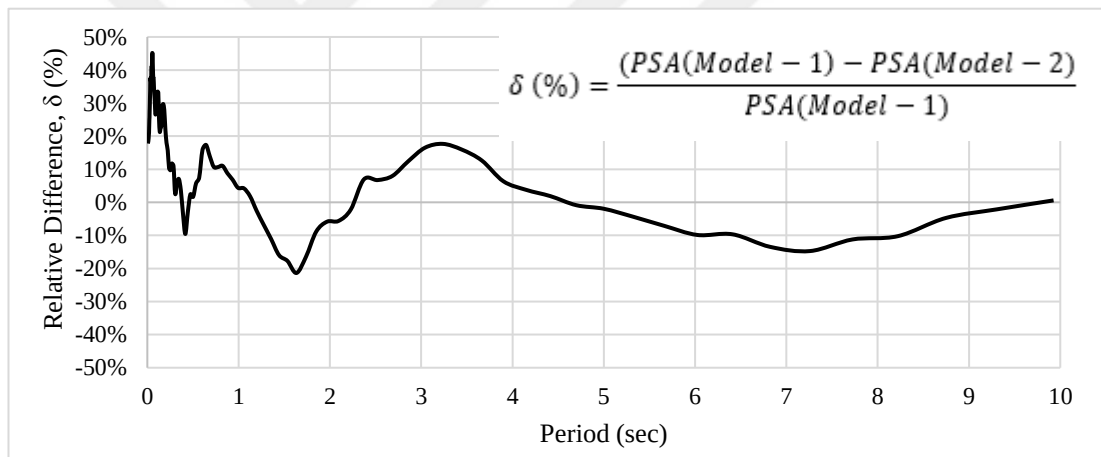


Figure 4.46 Relative difference of between Model-1 and Model-2 for DD-1 earthquake level using effective stress analysis (+ difference means Model-1 results in higher acceleration and vice versa.)

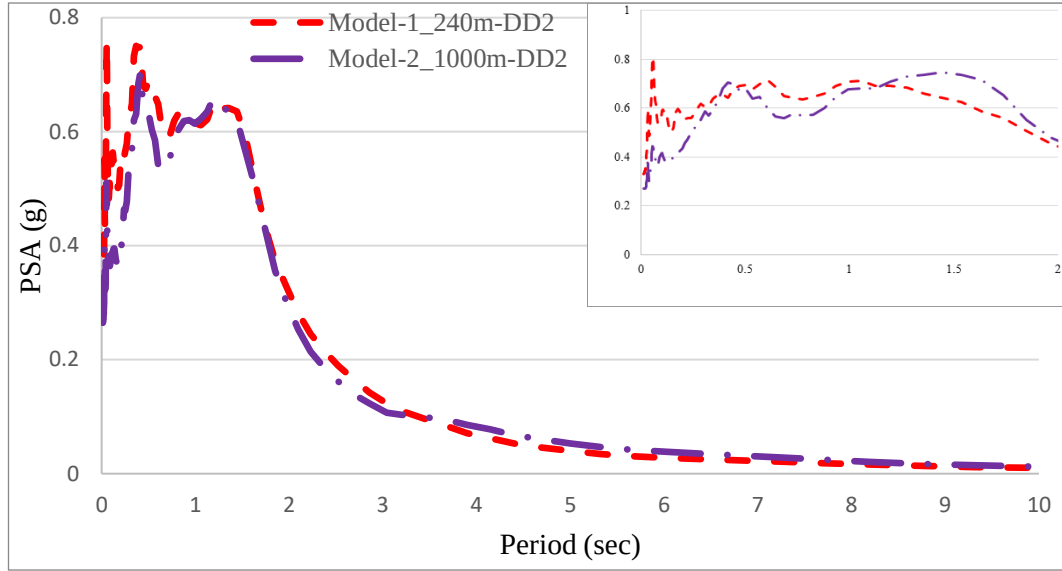


Figure 4.47 Comparison of Model-1 and Model-2 for DD-2 earthquake level using effective stress analysis

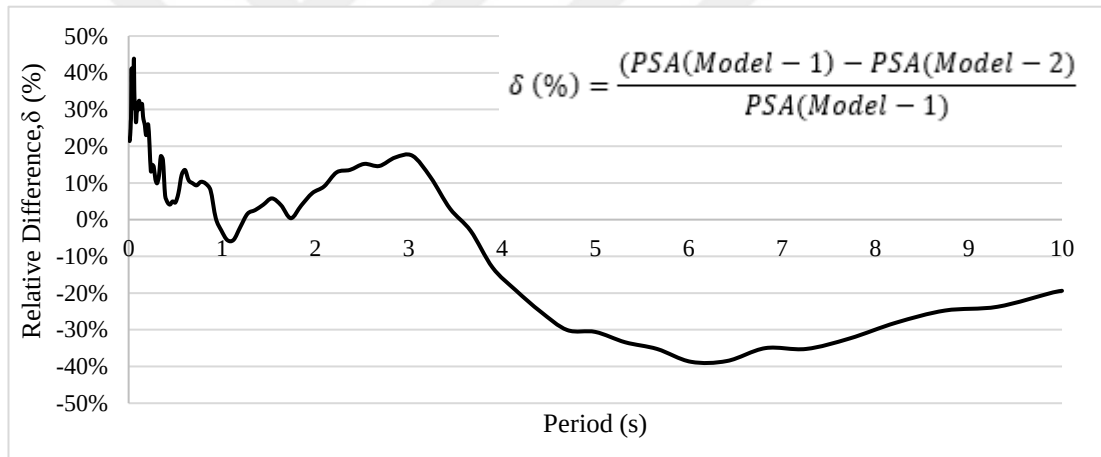


Figure 4.48 Relative difference of between Model-1 and Model-2 for DD-2 earthquake level using effective stress analysis (+ difference means Model-1 results in higher acceleration and vice versa.)

4.6 Comparison of Code-Based Design and Specific Site Response Analyses Results

The design response spectra can be obtained using two different methods, one of them is specific site response analysis and the other one is determined design spectra based on TBDY (2018) and DBYBHY (2007). One of the aims of this study is to find out the differences between of two methods. According to code-based design, design response spectra can be obtained easily using some table and correlation associated with soil class effect and map spectral acceleration coefficients.

The obtained design response spectra based on code-based and specific site response analysis results were compared and presented in Figure 4.49. According to the results, the obtained peak spectral acceleration values according to DBYBHY (2007) was more conservative than the obtained according to specific site response analysis. Also, the period lengthening have been found in extended period region. This means that site response results have given higher period range than design response spectra based on DBYBHY (2007) and TBDY (2018). Especially in high building design, period lengthening is important, because high buildings have high period. According to TBDY (2018), soil effect coefficients aren't suggested for ZF soil class, specific site response is proposed, since specific site response analyses eliminate the inconvenient results, it provides a more appropriate solution for ZF soil class.

The comparisons of the obtained design response spectra results without soil coefficients (the soil coefficients aren't suggested for ZF soil class) ZF soil class according to TBDY (2018) were given Figure 4.50 and Figure 4.51. As a beginning, The obtained design response spectra without soil coefficients based on TBDY (2018) and specific site response analysis results were compared and then according to the positive standard deviation of the specific site response analyses results, the soil effect coefficients have been obtained and given Table 4.3 for the deep alluvial deposits (ZF soil class) in the Izmir Bay.

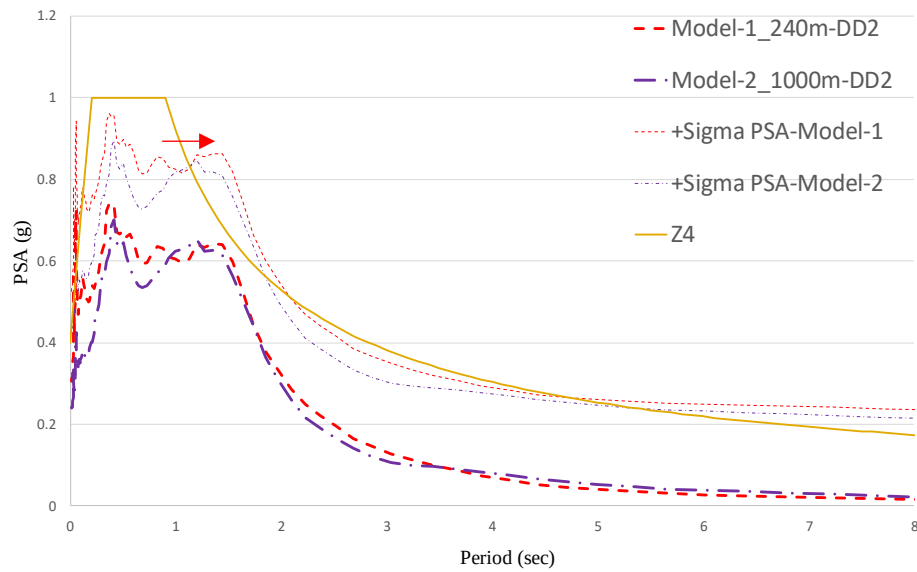


Figure 4.49 The comparison of the obtained design response spectra based on DBYBHY and specific site response analyses

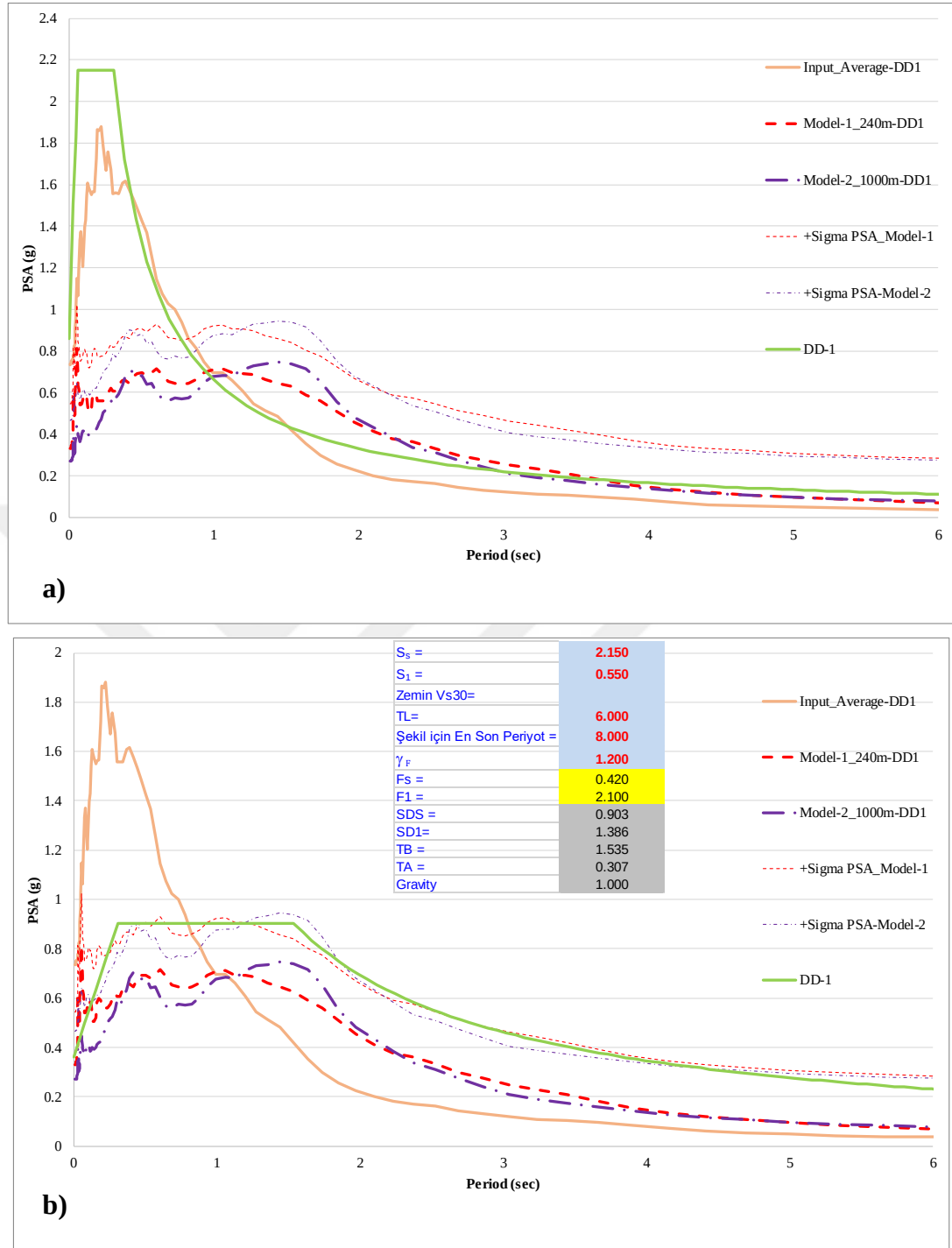


Figure 4.50 For DD-1 earthquake level at the surface, a) the comparison of design response spectra results, b) determining local soil coefficients for the deep alluvial deposits in Izmir Bay

As can be seen from the results, the soil effect coefficients can't be predicted for ZF soil class, it should be obtained according to specific site response analyses results for the investigation site.

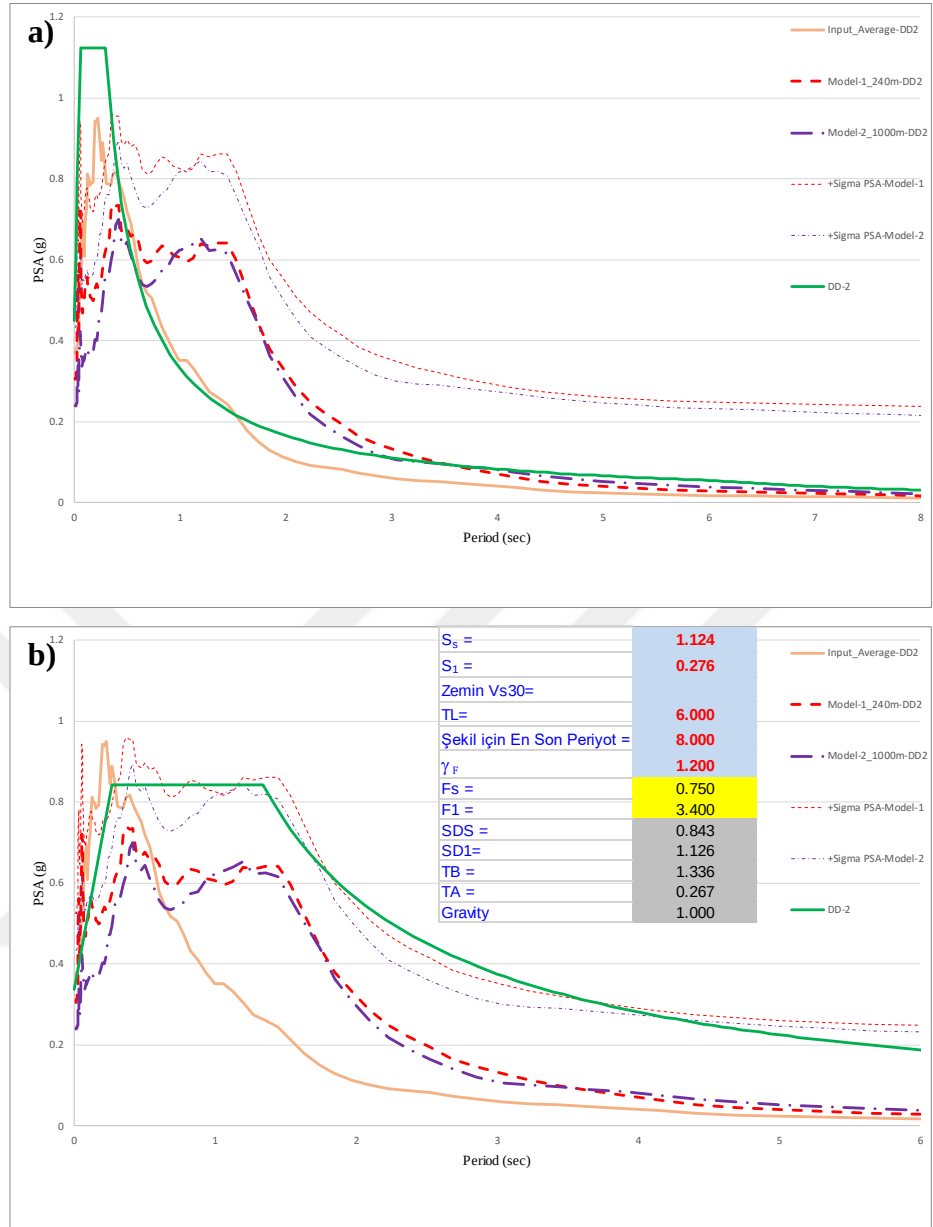


Figure 4.51 For DD-2 earthquake level at the surface, a) the comparison of design response spectra results, b) determining local soil effect coefficients for the deep alluvial deposits in Izmir Bay

Table 4.3 The obtained local soil effect coefficients for the ZF soil class with deep alluvial deposits in the Izmir Bay

Earthquake Level	S_s	S_1	F_s	F_1
DD-1 (2475 years)	2.15	0.55	0.42	2.1
DD-2 (475 years)	1.124	0.276	0.75	3.4

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

Both level of earthquakes were deamplified at short period range and amplified at higher periods. Therefore, especially for higher structures with larger natural periods, amplification of accelerations should be carefully inspected.

Moreover, comparison of Model-1 and Model-2 shows that for higher periods, deeper bedrock level in Model-2 amplifies the acceleration at higher periods.

Based on these results, it can be stated that for high-rise buildings, especially for structures over 40-50 stories amplification plays an important role and soil column should be modelled completely by including engineering bedrock in the analyses. Assumption of engineering bedrock as the ground motion source may result in unconservative results for higher periods.

Soil conditions changed the frequency content of the ground motion. In soft soils, soil amplification occurred at low frequencies.

Effective stress analysis results showed that maximum pore water pressure ratio (u/s'_v) became unity in sand and gravel layer. This means that the soil either lose shear strength or pore water pressure is equal to effective stress. Consequently, liquefaction occurred where the soil type is sand and gravel layer. However, clayey layers also experienced liquefaction at some depths, these results are not consistent with literature on liquefaction and should be approached with caution. Such large excess pore pressure ratios in the clay could have been due to pore pressure migration from neighboring cohesinless layers or it could be due to numerical/modelling errors.

In nonlinear ground response analyses, total stress analyses have resulted in higher accelerations than effective stress analyses. For both earthquake levels (DD-1 and DD-2) relative differences reached up to 40%. Based on the earthquake levels, average relative differences are higher for DD-1. Therefore, it can be concluded that as the accelerations increase, differences between total stress analyses and effective stress analyses increases.

The most important result deduced based on the comparison of site response spectra and design response spectra based on DBYBHY (2007) is period lengthening for Z4 soil class. The 2007 Code has become out of date as of January 2019 since the new earthquake code has become mandatory since then. The soil profile that was studied within the scope of this thesis belongs to ZF class for which the Code does not suggest any design spectra and require development of site specific dynamic analyses. This is a formidable task, however, for many low rise structures. Therefore, an attempt was made in order to check whether suggestions could be made for F_s and F_1 coefficients to be used in generating spectra for F Class soils. As shown on the figures and tables of the previous section this attempt has proven to be successful indicating that further studies are worthwhile in this manner.

The material models employed in site response analyses shall be improved especially for loose saturated silty sands and very soft to soft clayey soils of İzmir Bay area by determining corresponding model parameters via dynamic triaxial testing program. Besides, dynamic models shall be calibrated using the data of already available strong ground motion recorder array system installed in İzmir City.

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APPENDICES

Appendix A: Deepsoil Software Description

DEEPSOIL (Hashash, et al., 2017) software, developed by Youssef Hashash and Duhee Park at University of Illinois at Urbana-Champaign, is a one-dimensional site response analysis program. This Program can perform both 1-D nonlinear and equivalent linear analyses at the same time.

DEEPSOIL (Hashash, et al., 2017) software graphical user interface is composed 5 equivalent linear and 6 nonlinear stages. First step is selection of analysis types which are Linear, Equivalent Linear and Nonlinear. After selection of analysis type, the user should select solution types which are Frequency Domain and Time Domain. Selection of soil model and hysteretic re/unloading formulation are also performed in this step. First step is shown in Figure A.1. As can be seen in figure, in this study, nonlinear time domain analysis with pressure dependent modified Kondner Zelasko (MKZ) soil model is selected.

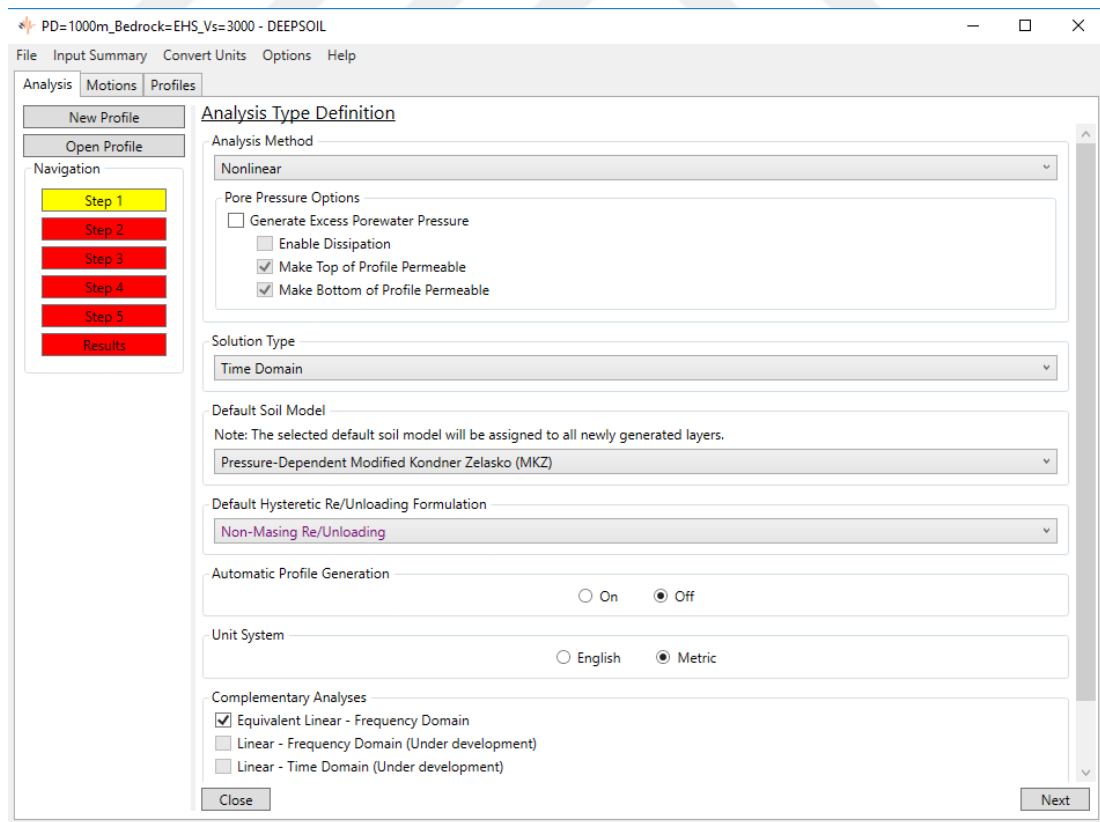


Figure A.1 First step window of DEEPSOIL software (Hashash, et al., 2017)

Second step is definition of soil profile and soil properties. Required soil properties depend on the selected analysis type and soil model. Water table is also defined in this section. Second step is shown in Figure A.2 which includes usual soil properties such as unit weight, shear wave along with nonlinear soil properties of MKZ model. Moreover, shear reduction and damping ratio curves are also described in this chapter and shown in Figure A.2.

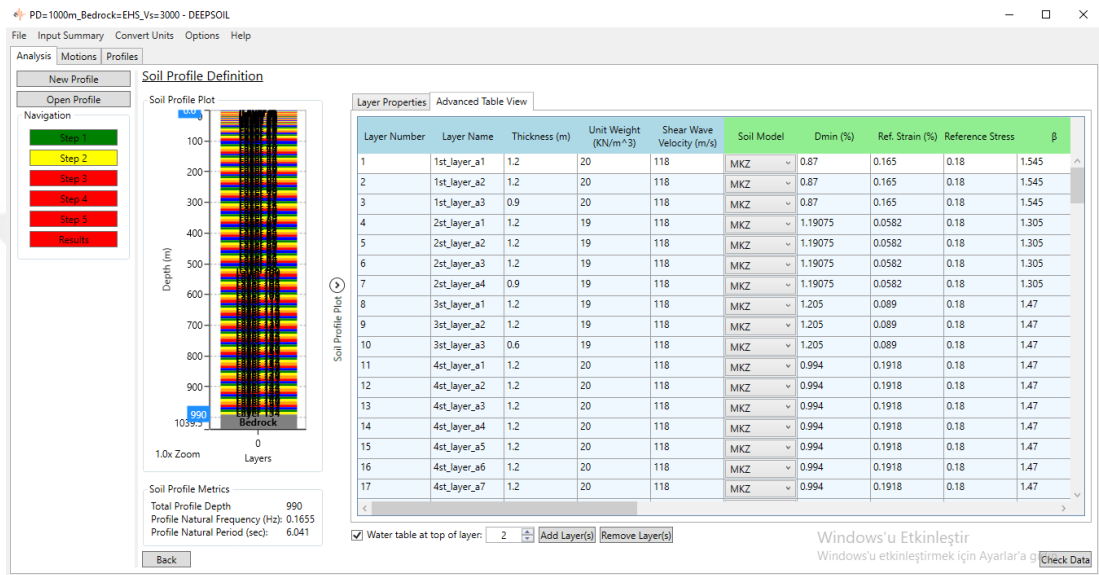


Figure A.2 Second Step – Soil Layer and Properties Definitions in DEEPSOIL

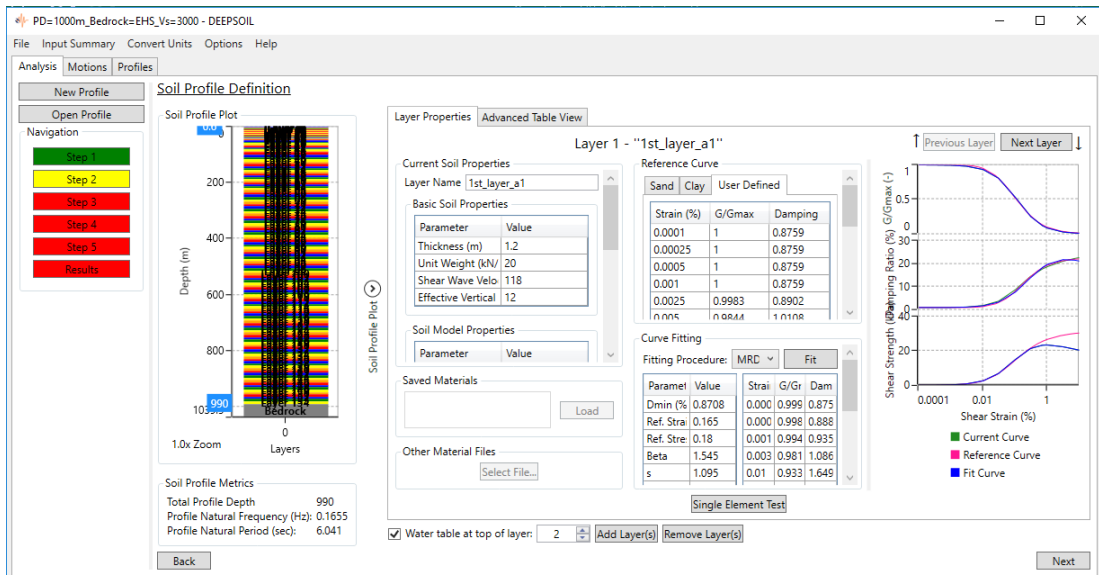


Figure A.3 Second Step – Shear Reduction and Damping Ratio Definition in DEEPSOIL

Third step is selection input motion. Previously selected earthquakes to use in site response analysis were defined in the DEEPSOIL software. The input motions to use in the analyses should be selected before running the analysis as shown in Figure A.4.

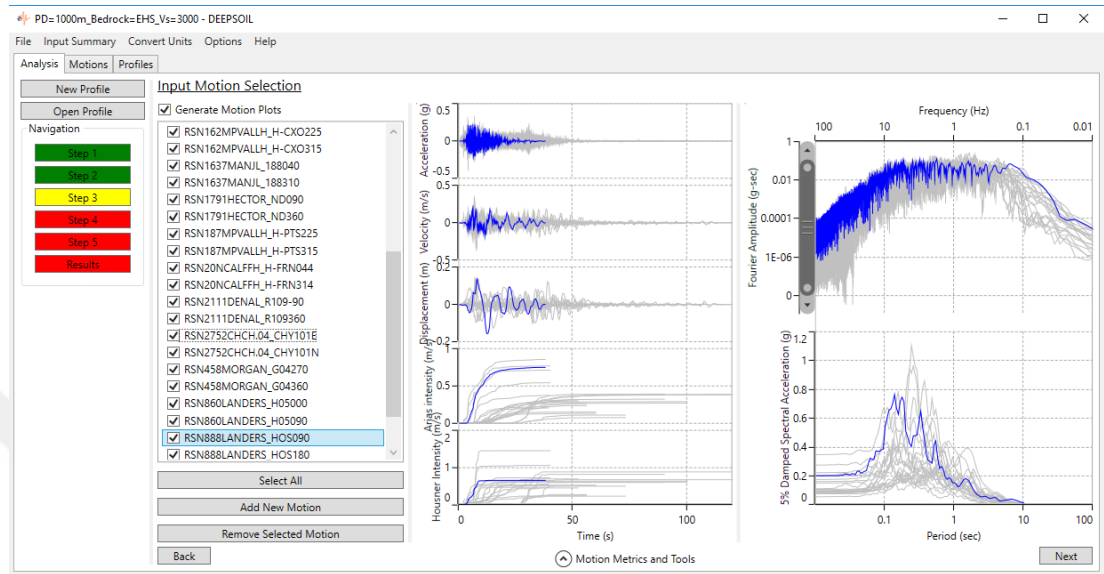


Figure A.4 Definition of earthquakes in third step

Fourth step is viscous/small-strain damping definition. This stage is only for time domain analysis. This window helps controlling the introduction of numerical damping through frequency independent nature of the viscous damping formulation. The fourth step has been shown in Figure A.5.

Finally, fifth step includes analysis control definitions and after this step, the analysis starts as shown in Figure A.6. Also, request for outputs of each layer can be made in this step.

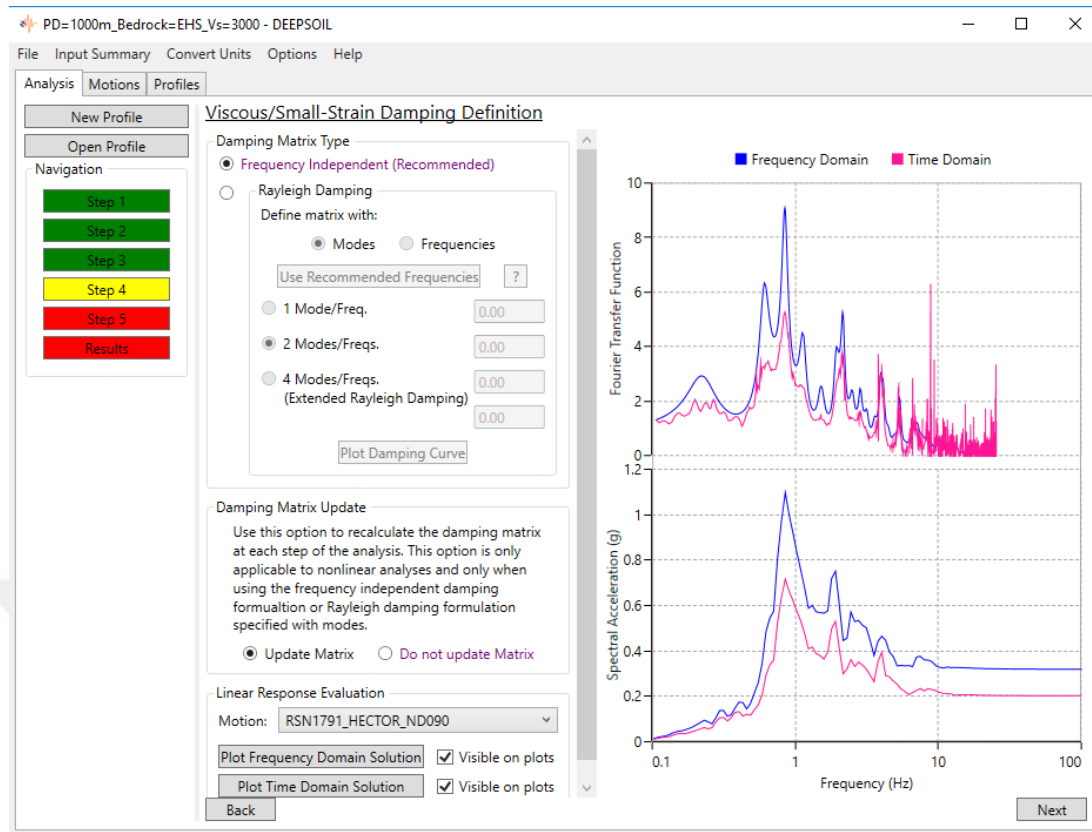


Figure A.5 Fourth step window of DEEPSOIL

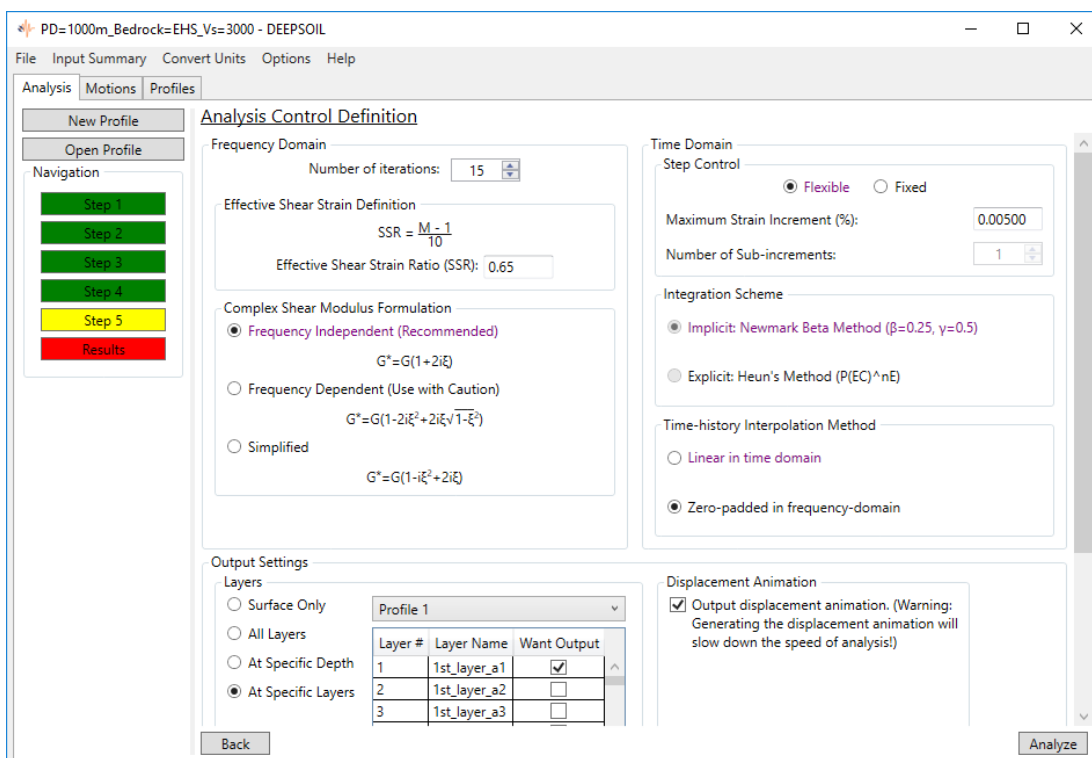


Figure A.6 Fifth step window of DEEPSOIL

After the analysis complete, results step generates time history of earthquake output, stress-strain behaviour of soil, spectral acceleration, soil profile, mobilized strength, displacement animation, and response spectra summary for selected soil layer and earthquake input motion. The results step is shown in Figure A.7.

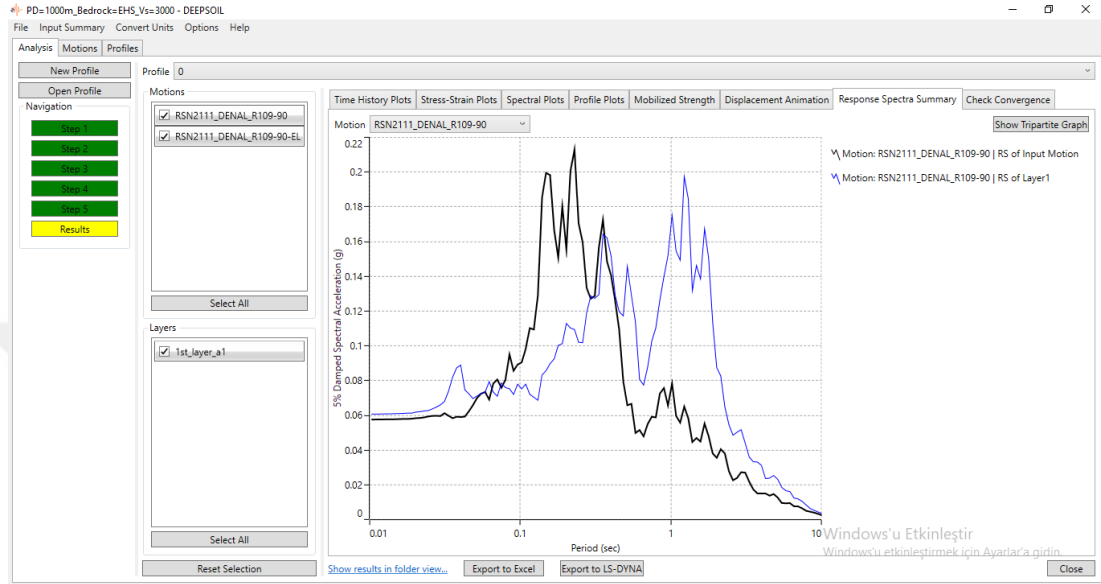


Figure A.7 Results step window of DEEPSOIL

Appendix B: Input Parameters In Deepsoil

The following tables involve the DEEPSOIL input parameters for both nonlinear total stress and nonlinear effective stress analyses.

Table B.1 Input parameters for selected site total stress analysis (Ref. Stress=0.18; b=d=0 all layers)

Layer Number	Layer Name	Thickness (m)	Unit Weight (kN/m ³)	Shear Wave Velocity (m/s)	Dmin (%)	Ref. Strain (%)	β	s	P1	P2	P3
1	CL	1.3	20	154	0.8703	0.17	1.55	1.10	0.58	0.28	2.75
2	CL	1.3	20	154	0.8703	0.17	1.55	1.10	0.58	0.28	2.75
3	CL	0.7	20	154	0.8703	0.17	1.55	1.10	0.58	0.28	2.75
4	GP-GM	1.5	19	162	1.1870	0.064	1.46	0.84	0.97	0.24	3.25
5	GP-GM	1.5	19	162	1.1870	0.064	1.46	0.84	0.97	0.24	3.25
6	GP-GM	1.5	19	162	1.1870	0.064	1.46	0.84	0.97	0.24	3.25
7	SC	1.5	19	184	1.1664	0.08	1.37	0.84	0.95	0.22	3.25
8	SC	1.5	19	184	1.1664	0.08	1.37	0.84	0.95	0.22	3.25
9	CL	1.7	20	207	0.9650	0.202	1.47	1.17	0.60	0.30	2.80

Table B.1. continues

Layer Number	Layer Name	Thick. (m)	Unit Weight (kN/m ³)	Shear Wave Velocity (m/s)	Dmin (%)	Ref. Strain (%)	β	s	P1	P2	P3
10	CL	1.7	20	207	0.9650	0.202	1.47	1.17	0.60	0.30	2.80
11	CL	1.7	20	207	0.9650	0.202	1.47	1.17	0.60	0.30	2.80
12	CL	1.7	20	207	0.9650	0.202	1.47	1.17	0.60	0.30	2.80
13	CL	1.7	20	207	0.9650	0.202	1.47	1.17	0.60	0.30	2.80
14	CL	0.7	20	207	0.9650	0.202	1.47	1.17	0.60	0.30	2.80
15	GC	1.8	19.5	219	1.1659	0.152	1.47	0.84	0.95	0.21	3.25
16	GC	1.8	19.5	219	1.1659	0.152	1.47	0.84	0.95	0.21	3.25
17	GC	1.8	19.5	219	1.1659	0.152	1.47	0.84	0.95	0.21	3.25
18	CH	2	20	243	0.8405	0.165	1.20	1.16	0.58	0.32	2.25
19	CH	2	20	243	0.8405	0.165	1.20	1.16	0.58	0.32	2.25
20	CH	2	20	243	0.8405	0.165	1.20	1.16	0.58	0.32	2.25
21	CH	2	20	243	0.8405	0.165	1.20	1.16	0.58	0.32	2.25
22	CH	2	20	243	0.8405	0.165	1.20	1.16	0.58	0.32	2.25
23	CH	2	20	243	0.8405	0.165	1.20	1.16	0.58	0.32	2.25
24	CH	1.5	20	243	0.8405	0.165	1.20	1.16	0.58	0.32	2.25
25	GC	2.4	19.5	289	1.1700	0.249	1.50	0.89	0.91	0.23	3.25
26	GC	2.4	19.5	289	1.1700	0.249	1.50	0.89	0.91	0.23	3.25
27	GC	1.2	19.5	289	1.1700	0.249	1.50	0.89	0.91	0.23	3.25
28	CL	2.6	21	314	0.9972	0.239	1.22	1.26	0.59	0.33	2.85
29	CL	1.4	21	314	0.9972	0.239	1.22	1.26	0.59	0.33	2.85
30	SC	3.5	19	418	1.1708	0.307	1.53	0.93	0.88	0.25	3.20
31	SC	1.5	19	418	1.1708	0.307	1.53	0.93	0.88	0.25	3.20
32	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
33	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
34	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
35	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
36	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
37	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
38	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
39	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
40	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
41	CL	3.65	21	418	0.9976	0.33	1.47	1.32	0.59	0.35	2.55
42	GC	7.4	21	886	1.1553	0.346	0.90	1.14	0.77	0.34	3.20
43	GC	7.4	21	886	1.1553	0.346	0.90	1.14	0.77	0.34	3.20
44	GC	7.4	21	886	1.1553	0.346	0.90	1.14	0.77	0.34	3.20
45	GC	7.4	21	886	1.1553	0.346	0.90	1.14	0.77	0.34	3.20
46	GC	4.4	21	886	1.1553	0.346	0.90	1.14	0.77	0.34	3.20
47	CL	7.75	21	886	0.9980	0.329	1.05	1.46	0.60	0.40	1.70

Table B.1. continues

Layer Number	Layer Name	Thick. (m)	Unit Weight (kN/m ³)	Shear Wave Velocity (m/s)	Dmin (%)	Ref. Strain (%)	β	s	P1	P2	P3
48	CL	7.75	21	886	0.9980	0.329	1.05	1.46	0.60	0.40	1.70
49	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
50	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
51	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
52	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
53	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
54	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
55	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
56	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
57	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
58	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
59	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
60	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
61	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
62	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
63	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
64	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
65	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
66	CL	5.55	21	660	0.9677	0.3	0.75	1.58	0.58	0.40	0.90
67	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
68	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
69	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
70	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
71	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
72	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
73	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
74	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
75	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
76	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
77	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
78	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
79	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
80	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
81	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
82	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
83	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77

Table B.1 continues

Layer Number	Layer Name	Thick. (m)	Unit Weight (kN/m ³)	Shear Wave Velocity (m/s)	Dmin (%)	Ref. Strain (%)	β	s	P1	P2	P3
84	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
85	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
86	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
87	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
88	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
89	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
90	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
91	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
92	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
93	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
94	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
95	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
96	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
97	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
98	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
99	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
100	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
101	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
102	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
103	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
104	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
105	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
106	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
107	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
108	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
109	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
110	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
111	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
112	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
113	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
114	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
115	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
116	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
117	Rock	7.06	22	850	2.9829	0.146	1.49	1.01	0.86	0.40	0.77
118	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
119	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
120	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
121	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27

Table B.1 continues

Layer Number	Layer Name	Thick. (m)	Unit Weight (kN/m ³)	Shear Wave Velocity (m/s)	Dmin (%)	Ref. Strain (%)	β	s	P1	P2	P3
122	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
123	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
124	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
125	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
126	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
127	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
128	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
129	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
130	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
131	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
132	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
133	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
134	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
135	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
136	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
137	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
138	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
139	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
140	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
141	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
142	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
143	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
144	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
145	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
146	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
147	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
148	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
149	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
150	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
151	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
152	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
153	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
154	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
155	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
156	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
157	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
158	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
159	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27

Table B.1 continues

Layer Number	Layer Name	Thick. (m)	Unit Weight (kN/m ³)	Shear Wave Velocity (m/s)	Dmin (%)	Ref. Strain (%)	β	s	P1	P2	P3
160	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
161	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
162	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
163	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
164	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
165	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
166	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
167	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
168	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
169	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
170	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
171	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
172	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27
173	Rock	7.14	22	850	2.9917	0.219	1.52	1.04	0.76	0.40	1.27

Table B.2 Additional input parameters for selected site effective stress analysis (Ru=0.95 all layers)

Layer Number	Layer Name	PWP Model	Cv (m ² /sec)	PWP Input 1 f s α α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - -	PWP Input 6 v C v v v
1	CL	Clay - Matasovic	1.5E-08	0.116	0.545	7.645	- 14.714	6.380	0.692
2	CL	Clay - Matasovic	1.5E-08	0.116	0.545	7.645	- 14.714	6.380	0.692
3	CL	Clay - Matasovic	1.5E-08	0.116	0.545	7.645	- 14.714	6.380	0.692
4	GP-GM	Sand - Vucetic/Dobry	0.00025	1.000	1.000	1.430	1.350	0.020	3.800
5	GP-GM	Sand - Vucetic/Dobry	0.00025	1.000	1.000	1.430	1.350	0.020	3.800
6	GP-GM	Sand - Vucetic/Dobry	0.00025	1.000	1.000	1.430	1.350	0.020	3.800
7	SC	Sand - Vucetic/Dobry	0.0000025	1.000	1.000	1.180	1.560	0.020	3.800
8	SC	Sand - Vucetic/Dobry	0.0000025	1.000	1.000	1.180	1.560	0.020	3.800
9	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
10	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
11	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692

Table B.2 continues

Layer Number	Layer Name	PWP Model	Cv (m2/sec)	PWP Input 1 f s α α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - - -	PWP Input 6 v C v v v
12	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
13	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
14	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
15	GC	Sand - Vucetic/Dobry	0.0000028	1.000	1.000	0.900	1.570	0.020	3.800
16	GC	Sand - Vucetic/Dobry	0.0000028	1.000	1.000	0.900	1.570	0.020	3.800
17	GC	Sand - Vucetic/Dobry	0.0000028	1.000	1.000	0.900	1.570	0.020	3.800
18	CH	Clay - Matasovic	1.5E-08	0.107	0.539	7.645	- 14.714	6.380	0.692
19	CH	Clay - Matasovic	1.5E-08	0.107	0.539	7.645	- 14.714	6.380	0.692
20	CH	Clay - Matasovic	1.5E-08	0.107	0.539	7.645	- 14.714	6.380	0.692
21	CH	Clay - Matasovic	1.5E-08	0.107	0.539	7.645	- 14.714	6.380	0.692
22	CH	Clay - Matasovic	1.5E-08	0.107	0.539	7.645	- 14.714	6.380	0.692
23	CH	Clay - Matasovic	1.5E-08	0.107	0.539	7.645	- 14.714	6.380	0.692
24	CH	Clay - Matasovic	1.5E-08	0.107	0.539	7.645	- 14.714	6.380	0.692
25	GC	Sand - Vucetic/Dobry	0.0000026	1.000	1.000	0.590	1.590	0.020	3.800
26	GC	Sand - Vucetic/Dobry	0.0000026	1.000	1.000	0.590	1.590	0.020	3.800
27	GC	Sand - Vucetic/Dobry	0.0000026	1.000	1.000	0.590	1.590	0.020	3.800
28	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
29	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
30	SC	Sand - Vucetic/Dobry	0.0000053	1.000	1.000	0.330	1.540	0.020	3.800
31	SC	Sand - Vucetic/Dobry	0.0000053	1.000	1.000	0.330	1.540	0.020	3.800
32	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
33	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
34	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
35	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
36	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692

Table B.2 continues

Layer Number	Layer Name	PWP Model	Cv (m2/sec)	PWP Input 1 f s α α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - - -	PWP Input 6 v C v v v
37	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
38	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
39	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
40	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
41	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
42	GC	Sand - Vucetic/Dobry	0.000075	1.000	1.000	0.100	1.430	0.020	3.800
43	GC	Sand - Vucetic/Dobry	0.000075	1.000	1.000	0.100	1.430	0.020	3.800
44	GC	Sand - Vucetic/Dobry	0.000075	1.000	1.000	0.100	1.430	0.020	3.800
45	GC	Sand - Vucetic/Dobry	0.000075	1.000	1.000	0.100	1.430	0.020	3.800
46	GC	Sand - Vucetic/Dobry	0.000075	1.000	1.000	0.100	1.430	0.020	3.800
47	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
48	CL	Clay - Matasovic	1.5E-08	0.161	0.571	7.645	- 14.714	6.380	0.692
49	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
50	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
51	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
52	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
53	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
54	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
55	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
56	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
57	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
58	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
59	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
60	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
61	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692

Table B.2 continues

Layer Number	Layer Name	PWP Model	Cv (m2/sec)	PWP Input 1 f s α α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - - -	PWP Input 6 v C v v v
62	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
63	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
64	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
65	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
66	CL	Clay - Matasovic	1.5E-08	0.148	0.564	7.645	- 14.714	6.380	0.692
67	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
68	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
69	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
70	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
71	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
72	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
73	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
74	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
75	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
76	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
77	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
78	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
79	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
80	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
81	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
82	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
83	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
84	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
85	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
86	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800

Table B.2 continues

Layer Number	Layer Name	PWP Model	Cv (m2/sec)	PWP Input 1 f s α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - - -	PWP Input 6 v C v v v
87	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
88	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
89	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
90	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
91	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
92	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
93	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
94	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
95	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
96	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
97	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
98	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
99	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
100	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
101	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
102	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
103	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
104	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
105	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
106	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
107	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
108	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
109	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
110	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
111	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800

Table B.2 continues

Layer Number	Layer Name	PWP Model	Cv (m2/sec)	PWP Input 1 f s α α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - - -	PWP Input 6 v C v v v
112	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
113	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
114	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
115	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
116	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
117	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
118	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
119	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
120	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
121	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
122	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
123	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
124	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
125	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
126	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
127	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
128	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
129	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
130	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
131	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
132	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
133	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
134	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
135	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
136	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800

Table B.2 continues

Layer Number	Layer Name	PWP Model	Cv (m2/sec)	PWP Input 1 f s α α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - - -	PWP Input 6 v C v v v
137	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
138	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
139	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
140	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
141	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
142	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
143	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
144	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
145	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
146	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
147	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
148	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
149	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
150	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
151	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
152	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
153	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
154	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
155	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
156	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
157	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
158	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
159	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
160	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
161	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800

Table B.2 continues

Layer Number	Layer Name	PWP Model	Cv (m2/sec)	PWP Input 1 f s α α α	PWP Input 2 p r Dr (%) β β	PWP Input 3 F A FC (%) - Dru=1.0	PWP Input 4 s B - - CSRt	PWP Input 5 Y C - - -	PWP Input 6 v C v v v
162	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
163	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
164	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
165	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
166	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
167	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
168	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
169	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
170	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
171	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
172	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800
173	Rock	Sand - Vucetic/Dobry	1.5E-08	1.000	1.000	0.110	1.000	0.020	3.800