A COMPARISON STUDY BETWEEN 1D AND 2D SITE RESPONSE ANALYSES BASED ON OBSERVED EARTHQUAKE ACCELERATION RECORDS

A Thesis

by

Shima Shamekhi

Submitted to the

Graduate School of Sciences and Engineering
In Partial Fulfillment of the Requirements for
the Degree of

Master of Science

in the
Department of Civil Engineering

Özyeğin University
July 2022

Copyright © 2022 Shima Shamekhi

A COMPARISON STUDY BETWEEN 1D AND 2D SITE RESPONSE ANALYSES BASED ON OBSERVED EARTHQUAKE ACCELERATION RECORDS

Approved by:

Professor M. Atilla Ansal, Advisor,
Department of Civil Engineering
Özyeğin University

Professor S. Feyza Çinicioğlu,
Department of Civil Engineering
Özyeğin University

Professor Gökçe Tönük
Department of Civil Engineering
Mef University

Date Approved: 28 July 2022

To my father, whose determination is as strong as the mountain,

To my mother, whose heart is as big as the ocean,

To my sisters whose supports are as much as a shining sun,

To my brother whose courage is as high as the blue sky, and

To my infinite love, my dear husband "Amin" to whom I dedicate not only my thesis

But also, my whole life.

ABSTRACT

Dynamic site response analysis is a technique that uses site characteristics and dynamic movements to attempt to predict the response of the specific site to ground motions in the form of response spectra. The main objective of the present work is to evaluate the difference between the one-dimensional and two-dimensional site response analysis, based on the comparison among the peak ground and spectral accelerations recorded by Istanbul Rapid Response Network (IRRN) and three Istanbul vertical array stations (IGVA) including Ataköy, Fatih and Zeytinburnu stations, during the M_w=6.5, 24.5.2014 Gökçeada and M_w=5.9, 19.5.2011 Kütahya earthquakes with the calculated accelerations by the 1D and 2D site response analyses. In this study, shear wave velocity profiles were determined based on insitu geophysical, geotechnical measurements, and laboratory tests within the Istanbul Microzonation Project. The shear wave velocity values, which are subject to some uncertainties due to some measurement methods or environmental conditions, are re-evaluated using a revision scheme to obtain the best matches between the recorded and calculated spectral accelerations by 1D site response analysis. These modified shear wave velocity profiles are later used for 1D and 2D site response analyses which in North-South and East-West directions to model peak ground and spectral accelerations on the ground surface. The results show the improvement of the calculated PGA and SA values. Finally, as another focus of this study, by modelling different symmetric spacing around the main boreholes separately in two main directions of some IRRN stations, the influence of horizontal distance on site response was investigated for 2D analyses. The results show that, in general, no unique relationship can be defined between the horizontal distance and the site response. There is an optimal spacing for each soil profile to obtain the best agreement between the calculated SA and the recorded value.

ÖZET

Dinamik site tepki analizi, belirli sitenin yer hareketlerine tepkisini tepki spektrumları biçiminde tahmin etmeye çalışmak için site özelliklerini ve dinamik hareketleri kullanan bir tekniktir. Bu çalışmanın temel amacı, İstanbul Acil Müdahale ve İstanbul düşey kayıt ağları (Ataköy, Fatih ve Zeytinburnu) ivme istasyonları tarafından, Mw=6.5, 24.5.2014 Gökçeada ve Mw=5.9, 19.5.2011 Kütahya depremlerinde kaydedilen en büyük yatay ve spektral ivmeler arasındaki karşılaştırmaya bağlı olarak tek-boyutlu ve iki-boyutlu saha davranış analizleri arasındaki farkı değerlendirmektir. Bu çalışmada, İstanbul Mikrobölgeleme Projesi kapsamında yerinde jeofizik, jeoteknik ölçümler ve laboratuvar deneylerine dayalı olarak kayma dalgası hız profilleri belirlenmiştir. . Ölçüm yöntemleri ve çevresel koşullar nedeniyle belirsizlikler içeren bulunmuş kayma dalgası hız değerleri, 1 boyutlu saha tepki analizi ile kaydedilen ve hesaplanan spektral ivmeler arasında en iyi eşleşmeleri elde etmek için bir iyileşyirme şeması kullanılarak yeniden değerlendirilmiştir. Bu yenilenmiş kayma dalgası hızı profilleri daha sonra Kuzey-Güney ve Doğu-Batı yönlerinde zemin yüzeyindeki en büyük ve spektral ivmeleri modellemek için 1B ve 2B saha tepki analizleri için kullanılmıştır. Sonuçlar, hesaplanan PGA ve SA değerleri modellenmesinde iyileşme olduğunu göstermektedir. Son olarak, bu çalışmanın bir diğer odak noktası olarak, bazı IRRN istasyonlarının iki ana yönünde ana sondaj kuyularının etrafındaki farklı simetrik boşluklar ayrı ayrı modellenerek, 2B analizler için yatay mesafenin saha tepkisi üzerindeki etkisi araştırılmıştır. Sonuçlar, genel olarak, yatay mesafe ile saha tepkisi arasında hiçbir ilişki tanımlanamayacağını göstermektedir. Hesaplanan SA ve kaydedilen değerler arasında en iyi anlaşmayı elde etmek için her kuyu için optimal bir ara mesafe bulunmaktadır.

ACKNOWLEDGMENTS

I would like to express my deepest gratitude to my supervisor Prof. Dr. Mustafa Atilla Ansal for his sincere encouragement, continuous support, and admirable advice throughout the research and for giving me the opportunity to explore various aspects of the field of geotechnical earthquake engineering.

Very special thanks go to Dr. Asli Kurtulus for her guidance, encouragement, and excellent advice throughout the research.

My special thanks to my dear friends and supporters during these years, Shima Sadeghzadeh, Maryam Massahi Fard, Saba Hajizadeh, Araz Salimnezhad,

I am very grateful to my parents and husband for their immense love and care. They have always encouraged me to discover my potential and motivated me to pursue my dreams. They have sacrificed a lot for me to reach this stage in my life.

I really acknowledge the financial support provided by the Scientific and Technological Research Council of Turkey (TUBITAK) under the grant No: 116M232.

Finally, I would like to thank Ozyegin University team for their support in allowing me to conduct this research over the past years.

TABLE OF CONTENTS

ABSTRACT ..... **iv**

ÖZETÇE ..... **v**

ACKNOWLEDGMENTS ..... **vi**

TABLE OF CONTENTS ..... **vii**

LIST OF TABLES ..... **xi**

LIST OF FIGURES ..... **xii**

1. INTRODUCTION **1**

1.1. Statement of the problem……..1

1.2. Site specific response analysis.....3

1.2.1. Dynamic site characterization ……...4

1.2.2. Selection of rock motion……....5

1.2.3. Site response analysis and design spectra……..5

1.3. Wave propagation analysis/ site amplification…….6

1.3.1. Methods of site response analysis…….....…….7

1.3.2. One dimensional wave propagation analysis…….7

1.3.2.1. linear site response analysis and review of past studies on one dimensional linear site response analysis…….....…9

1.3.2.2. Equivalent linear site response analysis and review of past studies on one-dimensional equivalent linear site response analysis……..…..10

1.3.2.3. Non-linear site response analysis and review of past studies on one-dimensional non-linear response analysis…….……..15

1.3.3. Two-dimensional wave propagation analysis…….…….16

1.3.3.1. Review of past studies on two-dimensional linear site response analysis……....17

1.3.3.2. Review of past studies on two-dimensional equivalent linear site response analysis…….…17

1.3.3.3. Review of past studies on two-dimensional non-linear site response analysis…….…...19

1.4. Model tests…….…….19

1.5. Downhole arrays…….……20

1.6. Istanbul Geotechnical vertical Arrays system…….……...…20

1.7. Istanbul Rapid Response Network…….…….22

1.8. Earthquake wave types and wave description…….……....25

1.8.1. Shear waves…….…….27

1.8.1.1. review of past studies on shear wave velocity…….……28

1.9. Investigation of current seismic provisions on the site response analyses…….29

1.9.1. Provisions in the Turkish earthquake code…….…30

1.10 Cyclic behavior of soil…….....32

2. METHODOLOGY…….....34

2.1. Introduction……34

2.2. Case study……..37

2.2.1. Site definition……..37

2.2.1.1 Ataköy vertical array……..38

2.2.1.2 Zeytinburnu vertical array……..39

2.2.1.3 Fatih vertical array……..41

2.2.2. Input motions……..42

2.2.2.1 Gökçeada earthquake……..42

2.2.2.2 Kütahya earthquake……48

2.2.3 Damping……53

2.2.4 Dynamic shear modulus reduction……54

2.2.5 Dynamic Shear modulus (G)……56

2.2.6 Poisson's ratio……57

2.2.7 Shear wave velocity……...57

2.2.8 Mesh dimension……58

2.2.9 Bedrock……..58

2.2.10 Boundary conditions……..59

2.2.10.1 Nodal force and displacement boundary condition……60

2.2.11 Frequency domain……..61

2.3. One-dimensional modeling……62

2.4. Improvement of shear wave velocity……63

2.4.1 Statement of problem……63

2.4.2 Past studies of the improvement of shear wave velocity……..64

2.4.3 Simulation for the improvement……...65

2.5. Two-dimensional modeling……...68

3. RESULTS AND DISCUSSIONS……...70

3.1 Discussion of results of 1D equivalent linear site response analysis……..70

3.2 Discussion of improving shear wave velocities……..78

3.3 Discussion of results of 2D equivalent linear site response analysis……..86

3.4 Discussion for results of analyzing different models of horizontal distances….90

4. CONCLUSIONS…….…95

4.1 Achievements against objectives …….....95

4.2 Gaps and future studies…….……...97

BIBLIOGRAPHY……..……...98

VITA……..……104

LIST OF TABLES

1 examples of ground types defined in the different of seismic codes 31

2 Soil types used in the soil profiles 54

LIST OF FIGURES

1 NAF locations 2

2 NAF structure and historical earthquakes with M> 7 in Turkey 2

3 The conditions and amplification variation in a natural site 16

4 The location of IRRN and IGVA stations in European side of Istanbul 22

5 The location of IRRN stations in Istanbul 23

6 The location of early warning system stations in Istanbul and around of it 24

7 The schematic of propagation of body waves of an earthquake into the particles of a soil or rock layer 26

8 A typical shape of the horizontal elastic design spectrum 31

9 General schematic of equivalent damping and stiffness 33

10 Soil classification and shear wave velocity profile of Ataköy vertical array 39

11 Soil classification and shear wave velocity profile of Zeytinburnu vertical array ... 40

12 Soil classification and shear wave velocity profile of Fatih vertical array 41

13 Acceleration time history records of Gökçeada earthquake by ATK station 44

14 Acceleration time history records of Gökçeada earthquake by FTH station 46

15 Acceleration time history records of Gökçeada earthquake by ZYT station 47

16 Acceleration time history records of Kütahya earthquake by ATK station 49

17 Acceleration time history records of Kütahya earthquake by FTH station 51

18 Acceleration time history records of Kütahya earthquake by ZYT station 52

19 Damping reduction curves for different soils used in this study 54

20 Modulus reduction curves for different soils used in this study 56

21 1D modeling for AKKIO station 63
22 The main roadmap for improving the shear wave velocities 66
23 Best 30 obtained shear wave velocity profile using enhancement procedure for AKKIO station 67
24 2D modeling for BAHHI station in EW direction 69
25 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in EW direction 71
26 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in NS direction 72
27 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in EW direction 72
28 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in NS direction 73
29 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in EW direction 74
30 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in NS direction 74

31 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in EW direction 75
32 Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in NS direction 75
33 Comparison of recorded SA at the bedrock level during Gökçeada earthquake for three ATK, FTH and ZYT IGVA stations in EW direction 76
34 Comparison of recorded SA at the bedrock level during Gökçeada earthquake for three ATK, FTH and ZYT IGVA stations in NS direction 77
35 Comparison of recorded SA at the bedrock level during Kütahya earthquake for three ATK, FTH and ZYT IGVA stations in EW direction 77
36 Comparison of recorded SA at the bedrock level during Kütahya earthquake for three ATK, FTH and ZYT IGVA stations in NS direction 78
37 Comparison of PGA values of 1D site response analyses calculated using the records from the ATK IGVA station in two directions of Gökçeada earthquake on the ground surface before the Vs revision 79
38 Comparison of PGA values of 1D site response analyses calculated using the records from the ATK IGVA station in two directions of Gökçeada earthquake on the ground surface after the Vs revision 80
39 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the ATAIO IRRN station based on the ZYT IGVA records in EW direction of the Gökçeada earthquake 81

40 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the AVIIO IRRN station based on the FTH IGVA records in NS direction of the Gökçeada earthquake 81
41 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BABAO IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake 82
42 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BABAO IRRN station based on the ATK IGVA records in NS direction of the Gökçeada earthquake 82
43 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BAVIO IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake 83
44 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BAVIO IRRN station based on the FTH IGVA records in NS direction of the Gökçeada earthquake 83
45 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the FBZMD IRRN station based on the ATK IGVA records in NS direction of the Gökçeada earthquake 84
46 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the ISKIB IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake 84

47 Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the ZYTAL IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake 85

48 Comparison of SA analyses between recorded, 1D and 2D analyses for AKUKO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction 86

49 Comparison of SA analyses between recorded, 1D and 2D analyses for ATAIO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction 87

50 Comparison of SA analyses between recorded, 1D and 2D analyses for AVIIO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction 87

51 Comparison of SA analyses between recorded, 1D and 2D analyses for BABAO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction 88

52 comparison of 1D and 2D calculated acceleration time history of AKKIO IRRN station in NS direction using ATK-NS records 90

53 comparison of 1D and 2D calculated acceleration time history of AKKIO IRRN station in EW direction using ATK-EW records 90

54 Comparison of 2D site response analyses between various horizontal distances from location of AKKIO-NS IRRN using the records of ATK IGVA station of Gökçeada earthquake in NS direction 91

55 Comparison of 2D site response analyses between various horizontal distances from location of ATAIO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction 92
56 Comparison of 2D site response analyses between various horizontal distances from location of AKKIO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction 92
57 Comparison of 2D site response analyses between various horizontal distances from location of AKUKO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction 93
58 Comparison of 2D site response analyses between various horizontal distances from location of BABAO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction 93

CHAPTER I

INTRODUCTION

1.1 Statement of the Problem

Istanbul (the cultural capital of Turkey) is one of the metropolises of the world located between the Sea of Marmara and the Black Sea, separated by the strait of the Bosphorus into the European side (in the west) and the Asian side (in the east), with a total area of 5343 km2 (Figure 1). Therefore, compared to the other cities in Turkey, it offers the best economic opportunities as it has more industrial and commercial areas. Therefore, the population in Istanbul is growing very fast, but Istanbul is located directly on the North Anatolian Fault (NAF). This fault structure extends along northern Turkey for more than 1400 km from 41˚E to 29˚E and provides a right-lateral displacement between the Eurasian and Anatolian plates of about 25 mm/year. Most of the strain accumulation that takes place in the northern part of the fault starts at the Gulf of Izmit and propagates into the Sea of Marmara (Figure 1,2). According to the scientific evidence, Istanbul is exposed to a high seismic risk, which also increases along the NAF [1,2].

On the European side of the city, there are more inhabitants, industries, and important centers than on the Asian side. In addition, this side is more vulnerable, due to soil conditions, wave amplification, and thick fill material, among other factors [1]. In the past, Istanbul has experienced some strong earthquakes, such as M=7.6 earthquakes

(1509, 1719, and 1766) and M=7 (1894) ground motion in the Sea of Marmara, which caused intensities up to X- XI [3]. Therefore, the main task of engineers is carefully studying the physical conditions related to geology and geotechnics as well as seismicity in order to be prepared against any possible seismic threat.

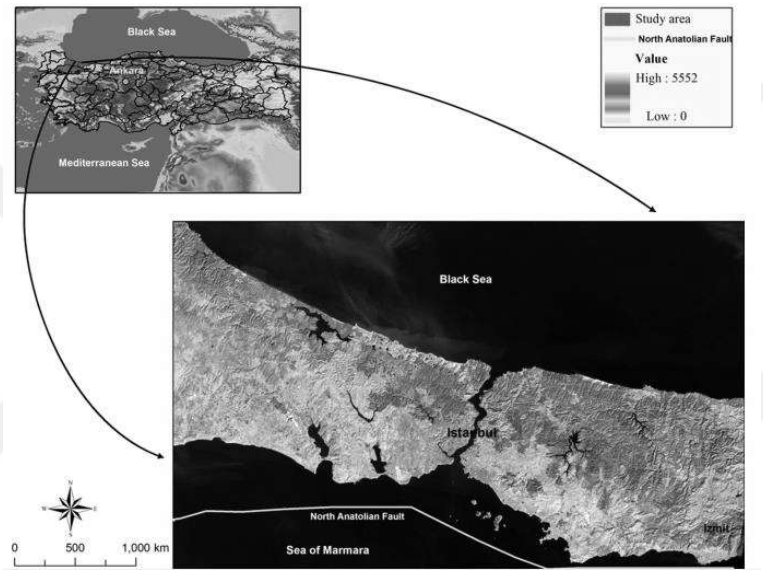


Figure 1: NAF locations [1].

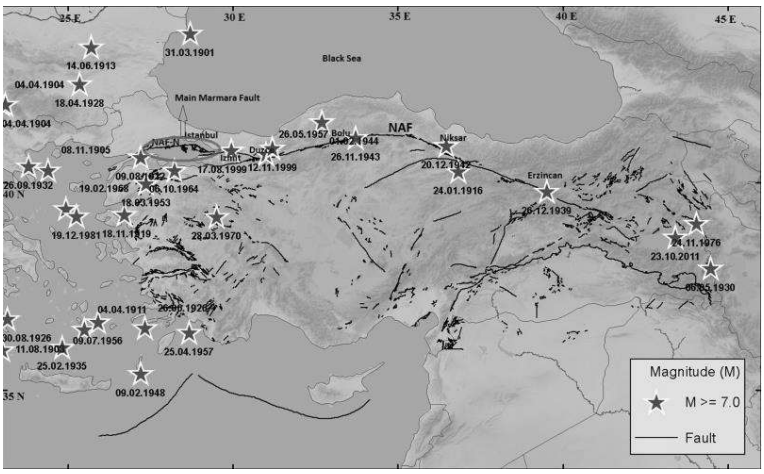


Figure 2: NAF structure and historical earthquakes with M>7 in Turkey [1].

1.2 Site-specific response analysis

The site and soil conditions have a very large impact on the propagation of damage in large earthquakes, so we need to know all the conditions of the site due to construction or other related investigations. There are three categories for seismic response, including theoretical, numerical and model tests. Theoretical and numerical site-specific response analysis or soil amplification study is a method of calculating the dynamic response of soil layers of a particular site or area where an earthquake hazard exists, using rock motion as input motion. This is usually the first phase of a seismic study that leads geotechnical engineers to calculate the natural periods of the site, calculate the amplification of ground motion, provide various parameters for structural engineering, investigate the liquefaction potential and stability of slopes and embankments at the site [4,5,6,7,8,9,10,11,12,13,14].

The different soil types behave differently in earthquakes and that this behavior should be taken into account when designing earthquake-resistant structures in earthquake-prone areas [15]. Important factors affecting this process include soil properties, geologic conditions, soil types and stratification, depth of bedrock, topographical environments, fault situation and epicenter of the earthquake, wave propagation, and the condition of the water table. Some of these factors introduce uncertainties, such as the rupture mechanism, the velocity of the subsurface layers, the attenuation characteristics, and the mode of energy transfer between the source and the site [4,7]. From a technical point of view, complex site impacts are expected in all urban areas [16]. However, there are some approaches to overcome the above problems more

or less, such as seismic hazard analysis (probabilistic or deterministic) to predict rock motion at the site or seismic hazard analysis based on empirical attenuation relationships to predict rock motion parameters [4,7]. According to the research of Pehlivan et al, [17] the above factors are modeled in engineering documents by an empirical ground motion prediction equation (GMPE) to predict ground shaking at a site. Once a ground shaking level has been determined for the rock conditions, the remaining task is to calculate the dynamic response of the soil deposit to this rock movement. Ansal et al [18] showed that the peak ground acceleration does not include the effect of differences due to the distance and magnitude of an earthquake, but the frequency content of the acceleration time history and thus its effect in the elastic acceleration response spectra can represent the effect of these parameters. They concluded that there are significant differences between the frequency content and the predominant site periods while magnitude and distance from the source increased or decreased. Moreover, the resulting frequency effect (the shift of predominant site periods to longer periods based on acceleration spectra) is independent of local site conditions. The following is a brief description of the analysis process.

1.2.1 Dynamic site characterization

This phase of the analysis process includes the measuring, calculating, and evaluating the below steps:

- Shear wave velocity profile with depth (via a geophysical testing technique such as surface wave method spectral analysis).

- Variation of shear modulus with strain (or modulus decrease curve).
- Variation of damping with strain (or damping ratio curve) [4,13,14].

1.2.2 Selection of rock motion

Natural or artificial acceleration time histories are selected as the appropriate bedrock motion for the site. This motion should be associated with exact seismotectonic constructions or source zones that cause the strongest displacement or the most intense motion. There are some conditions for using a natural time history containing duration of strong earthquake, peak parameters of ground motion and spectral content of response. Otherwise, if there are no natural motion parameters, synthetic motions can be generated using the concept of spectral compatible time histories. There are some methods to solve this concept, such as frequency-time domain generation, empirical Green's function technique, ARMA modeling, etc. [4,19,20].

1.2.3 Site response analysis and design spectra

There are several methods for calculating ground surface time histories by inputting the selected seismic force into the soil profiles of the desired site to obtain the design spectrum for the site. A large number of research papers introduce an empirical numerical modeling system or physically scaled modeling to study the site effects. Empirical studies focused on determining the amplitude of the transfer function linking input motion to ground surface displacements by using microtremor measurements and evaluating them by inversion of Fourier amplitude spectra or the like. On the other hand,

numerical methods were developed much earlier to study site effects, such as one-, two-, or three-dimensional analyzes. These methods are discussed below [4,5].

1.3 Wave propagation analysis/site amplification

Although seismic waves travel limited meters in soil layers, unlike rocks, they play an important role in analyzing the response characteristics of the site [15]. The passage of earthquake waves through the soil layers to the ground surface changes the parameters of ground motion, such as frequency content, which is called the ground amplification problem. In other words, the upper layers of the soil profile behave like a filter that, due to its geotechnical properties, amplifies or de-amplifies the effect of ground motion in the deeper layers [21]. In the study by Puri et al [15], site analyzes were performed for various locations in Haryana and amplification factors were determined for PGA ranges which showed that the soil types are capable of amplifying earthquake ground motions. There are two physical and mathematical viewpoints for this problem. The first is to predict the characteristics of ground motions at the surface. In the second case, it is the wave propagation in a continuous medium, because the released energy from the ground motion takes time to travel through the soil layers, and this is a continuous motion, so the behavior when these waves travel through is a function of the stiffness and attenuation characteristics of the soil layers. For this reason, geological materials can be evaluated as one-, two-, or three-dimensional wave propagation depending on the geometry and loading conditions [4]. Due to the complexity of soil behavior, we as engineers need to model soil profiles that describe the important features of cyclic behavior but should be simple and rational [15].

1.3.1 Methods of wave propagation analysis

The methods of site analysis are generally divided into three main categories: 1-linear 2-equivalent linear 3-nonlinear analysis, which differ in some areas, such as in the simplification of assumptions, the integration of the equation of motion, and the representation of the stress-strain relationship of the soil. The different types of these techniques have been developed due to changing search requirements and increasing information on the fundamental behavior of soils under cyclic loading obtained from in-situ or laboratory tests [4].

1.3.2 One dimensional wave propagation analysis

This procedure is commonly used for site response analysis, as there are many commercial programs with various soil models for use on personal computers. Most designed projects using this method in the past have survived earthquakes, so the method has proven itself and there is a belief that it provides conservative results. In addition, there are three assumptions for this theory: all ground layers are horizontal and extend to infinity, the ground surface is datum, and the input ground motions are spatially uniform, horizontally polarized shear waves and propagate vertically. By the way, there are some good reasons for using 1D analyzes, such as: Stress waves propagate vertically from the epicenter of ground motion toward the ground surface (Snell's law of refraction between solid and soil layers), while their velocity decreases. The main ground motions tend to be horizontal due to the structural design and the soil layers are considered to be semi-isotropic in the horizontal direction, so the properties of soil layers can change very rapidly in the vertical direction. However, several cases have shown that the effects of

location and variability of damage evaluated by one-dimensional analysis are not satisfactory [4,7,22]. The main objective of 1D analyzes of site response is to study the site response due to SH waves propagating vertically from bedrock [21,23,24].

Some of the mathematical principles of 1D site response analyses are presented. As the simplest case, consider a single soil overlying an infinite, approximately rigid half-space with linear elastic material properties. Thus, amplification function can be calculated using the equation below:

$$\mathrm{Ar}(\omega) = |Hr(\omega)| = \left|\frac{U(0,\omega)}{U(H,\omega)}\right| = \frac{1}{\cos\left(\frac{\omega_H}{Vs}\right)} \geq 1 \qquad (1)$$

Here $H_r(\omega)$ represents the complex transfer function, whereas $U(z, \omega)$ shows the magnitude of horizontal displacement at depth z, ω is angular frequency. In addition, H and Vs refers to the depth and shear wave velocity of the soil layer respectively [25,26].

The ratio of the Fourier amplitude of the motion at the ground surface $U_0(\omega)$ with respect to the Fourier amplitude of the base motion $U_H(\omega)$ is defined as amplification factor as:

$$Ar(\omega) = \frac{U_0(\omega)}{U_H(\omega)} \qquad (2)$$

If $\frac{\omega H}{Vs} = \frac{\pi}{2} + n\pi$ (n = 0, 1, 2, …) the amplification function inclines to infinity. Moreover, if n = 0, the natural frequency ω_r of the soil layer is shown by [25,26]:

$$\omega_r = \frac{\pi Vs}{2H} \text{ , } f_r = \frac{Vs}{4H} \tag{3}$$

If we include energy dissipation or, in other words, damping in its simplest form (viscous damping) in the above equation, it means that the displacement amplitudes associated with the resonant frequency are no longer infinite. Then equation with the damping is as the form below:

$$A^*r(\omega) = |H^*r(\omega)| = \frac{1}{\cos\sqrt{cos^2(KH) + (DKH)}} \tag{4}$$

$$(A^*r)_{max} \cong \frac{2}{(2n-1)\pi D} \text{ , } n = 1, \dots, \infty \tag{5}$$

Where K denotes wave number and D represents damping ratio [25,26].

1.3.2.1 Linear site response analysis and review of past studies on one-dimensional linear site response analysis

This method involves some ordinary mathematical functions when, in special cases where the use of analytical methods is not possible, numerical techniques such as the finite element method are used [4]. Volpini et al [25] found that the computer software Abaqus is faster than FIAC 3D for linear elastic analysis. Volpini et al [25] believe that 1D analyses of site response provide unrealistic results, especially when the site is subjected to high amplitude vibrations. The authors stated that in this case, nonlinear 3D analyses are the most complete approach.

James Kaklamanos et al [27] investigated a linear, equivalent linear and nonlinear 1D site response study using the SHAKE and DEEPSOIL and Abaqus/Explicit programs. They focused on well-modeled sites with 1D wave propagation. For all sites, they obtained a positive bias at short spectral periods (< 0.2). Another result they obtained is that fully nonlinear models of site response are an improvement over linear or equivalent linear models. According to the results obtained by James Kaklamanos et al [27], the equivalent linear and nonlinear methods for modeling ground motions with large loads achieve accurate predictions. An important finding of their team is that modeling with more input parameters in nonlinear or equivalent linear models can degrade the results of site response analysis. This is due to the fact that all these parameters do not match the actual characteristics of the natural site.

1.3.2.2 Equivalent linear site response analysis and review of past studies on one dimensional equivalent linear response analysis

This method was introduced by Schnabel et al [28], based on the model of the aggregated mass of a sand deposit located on a solid ground to which the seismic motions were applied. After generalizing this method for a wave propagation model with an energy-transferring boundary, the input motions could be applied at any level of the soil layers. The one- dimensional equivalent linear analysis of the site response is generally performed in the frequency domain [9,17] .Two important advantages of equivalent linear modeling are the low computational cost and few input parameters. There are some common computer programs for this purpose such as SHAKE, DEEPSOIL, ERRA, Pro-Shake and others. Since equivalent linear modeling is based on a representation of the

entire soil behavior, the nonlinear stress-strain loop is approximated by a single equivalent linear crossing of the shear modulus, which is a function of the shear moment [15]. In equivalent linear analyzes of soil behavior, G-γ-D (strain reduction-density-damping ratio reduction) curves are needed to describe the change in shear modulus and damping with increasing amplitude of shear strain. This curve is found from laboratory tests [22]. The equivalent linear analysis process is based on an iterative system in which the shear modulus and damping are first estimated for each soil layer. Then we can obtain the results by analyzing these linear-time invariant properties. After we obtain the shear strain history, the maximum shear strain for each layer is obtained. Then the effective shear strains are obtained as a fraction of the peak strains. The equivalent linear damping ratio (D) and shear modulus (G) are evaluated by using effective shear strain. This process is repeated until the properties consistent with strain reliably match those used for dynamic response analysis [15]. In one-dimensional equivalent linear site analyzes, the damping ratio is a key parameter because even small values of damping can significantly affect ground motion [29].

The 1D equivalent linear site response analyses have some weaknesses. This method neglects multi-directionality in the strain history of real wave propagation because it considers only a single strain level as representative of the entire strain history of a soil layer, without taking into account the plastic response of pore water pressure variations in the soil [30]. In general, equivalent linear analyses yield longer peak soil acceleration and shear stress values for large soil movements. On the other hand, this

method leads to low amplification in the high frequency range. This problem arises from the evaluation of the damping ratio based on the effective strain [29].

Govinda-Raju et al [4] investigated a case study of equivalent linear site response analyses during the Bhuj earthquake on January 26, 2001, at a site near the Sabarmati River belt in Ahmedabad city. In this study, the effects of soil amplification and settlement and their relationship with acceleration of soil layers were investigated. Ansal et al [18] performed 1D equivalent linear site response analysis using the computer software SHAKE based on the recorded ground acceleration of the Gökçeada earthquake recorded by vertical arrays for the East-West and the North-South directions (EW and NS). The result was that the general recorded PGA values are higher than the calculated ones because the 1D site response analysis method was simplified. After this investigation, the researcher shows that the spectral accelerations calculated by the 1D analysis, and the recorded spectral accelerations agree very well when the vertical array is very close to the relevant Istanbul Rapid Response Network (IRRN) station in the correct direction. He also examined the scaled input motion of the rock to the same and highest PGA recorded at the rock level and concluded that the differences between the calculated 1D site response analysis and the recorded spectral acceleration improved significantly. In addition, his team noted the differences in the frequency characteristics of the recorded acceleration time histories of the Gökçeada earthquake with respect to the EW and NS directions due to geotechnical characteristics of the site, such as the depth of the engineered bedrock, the shear wave velocity of the bedrock, etc.

In another study by Ansal et al [31], the differences of acceleration spectra at different depths for the Çınarcık, Kütahya and the Sea of Marmara earthquakes in two directions (EW and NS) were determined on the vertical arrays of Ataköy, Fatih and Zeytinburnu to show the effects of distance (frequency content) and magnitude. They present two ways to model the observed earthquake features at the ground surface. One way is the empirical approach in terms of the average shear wave velocity in the uppermost 30 m (V_{S30}). Either way, the V_{S30} values from the IRRN stations are approximately the same, but the peak ground accelerations are, contrary to expectation, very different due to differences in the depth of the engineered subsurface, so V_{S30} may be an inferior index of the site's amplification potential. The second option is to use 1D analyzes of site response. Their group examined 1D analyzes of ground motions from Çınarcık and Kütahya recorded for 20 IRRN stations using the SHAKE91 computer program. The comparison between the calculated ground motions and those recorded at these stations showed that 1D analyzes of ground motions using detailed soil profiles can provide a good prediction of local ground motions. In addition to the importance of the above two factors, it was found that the use of noise measurements cannot always provide reliable results in terms of the effects of local site conditions. Moreover, they checked the spectra of acceleration response using motion records at the Sea of Marmara, Kütahya and Büyük çekmece at the bedrock level in the vertical arrays of Ataköy, Fatih and Zeytinburnu in two components EW and NS and found out large differences between spectral acceleration of the bedrock and ground surface. This shows the importance of geological features of the bedrock construction. Different response patterns can be found in the soil layers along the anisotropic characteristics of these layers due to the increase

of spectral acceleration with different depths from the bedrock to the ground surface in two directions EW and NS. Moreover, changes in the predominant period in the same plane were observed for different directions. In another study, B. Setiawan [32] investigated the equivalent linear response of the Addaide Reqolite zone during the 1997 Burra earthquake using the ERRA program based on the SHAKE software. Another objective of this study was to understand how the actual seismic waves affect the founded structures in the area of interest. This study resulted in modifying the input parameters at the studied site due to local site effects. After comparing the results with references, the hazard spectra and amplification of the earthquake in question were derived.

In the other study conducted by Ansal et al [29], the modelling of the motion characteristics in terms of peak ground accelerations and acceleration response spectra is investigated. This attempt was made using two methods, empirical and 1D site response analysis. In this work, the site analyses provided more accurate results than the empirical approach due to the detailed characterization of the site. It was also found that the vertical arrays were a suitable option for modelling the properties of the soil layers to predict the ground motions at the site in question.

Kurtulus A. [33] conducted a study on site response analysis using information from IGVA and IRRN (Istanbul Rapid Response Network and Istanbul Geotechnical Vertical Arrays). She used two methods for her work: 1D analyses of site response using rock motions recorded in borehole arrays, and an empirical approach based on amplification factors proposed by Borcherdt [34] based on V_{S30} values. She used the SHAKE91 computer program for 1D analyses of site response. She concluded that the

empirical approach does not provide a good prediction of spectral acceleration. Therefore, V_{S30} values are not a good indicator for response analysis. Because she used a low excitation level of input motion, the equivalent linear 1D analyses modelled the PGA more accurately because the ground is assumed to exhibit elastic behavior during these motions.

1.3.2.3 Non-linear site response analysis and review of past studies on one-dimensional non-linear response analysis

It is proved that the stress-strain behavior of soil under dynamic loading is nonlinear [22]. For this type of analysis, it is important that the strain-stress characteristics of the specific soil be realistically modeled and carried out using a discrete model, such as finite element analyses, etc. The nonlinear stress-strain behavior of soil under dynamic loading integrates the equations of motion step by step in the time domain [4,22]. There are many scientific documents showing that soils under cyclic and dynamic shear conditions exhibit nonlinear behavior, such as a reduction and corresponding growth of material damping with increasing shear strain level. These variations result in significant changes in the amplitude, duration, and frequency content of a ground motion. In the case of strong ground motions, the accumulation of plastic strain and the buildup of excess pore water pressure should also be investigated to characterize the soil. Thus, here an engineer should use a realistic simulation of the wave propagation problem [30]. For nonlinear site behavior, analyses are performed when the time domain is chosen such that small strain and damping values diverge over the entire loading duration [27].

1.3.3 Two-dimensional wave propagation analysis

According to the scientific documents, some of the sites should be studied with a two-dimensional dynamic model. The reason for this is the formation of surface waves during the transition between non-horizontal layers [21,24,35]. The two-dimensional analysis of the site response takes into account the vertical and horizontal properties of the soil layers. This type of analysis is usually calculated by a dynamic finite element analysis in the frequency domain or in the time domain. The finite element method considers the two-dimensional case with discrete quadrilateral elements whose corners are characterized by nodal points [17]. In general, a 2D model overestimates the dynamic stiffness of the ground and the radiation attenuation due to the geometric distribution of the waves [25]. In 2D and 3D models, it can be observed that the interaction between solid and fluid is very well accounted for by a coupled effective stress formulation. The lack of adequate expertise is one of the main reasons why less 2D or 3D methods are used [30].

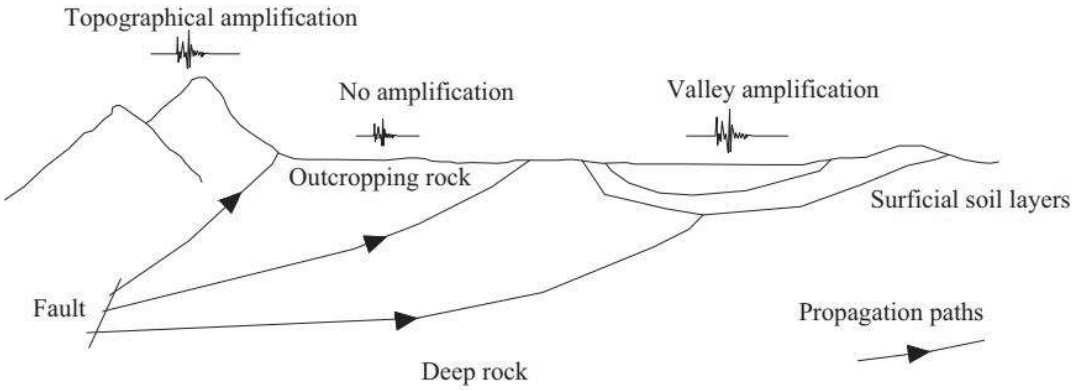


Figure 3: The conditions and amplification variation in a natural site [16].

The Figure 3 shows a real, natural site. However, if you use a 1D analysis-based computer program such as DEEPSOIL, the soil layers are considered as horizontal lines. This method reduces the effects of site conditions on shear wave velocity, soil layer properties, and soil classification. Therefore, the real features are no longer important in the 1D analysis [16].

1.3.3.1 Review of past studies on two-dimensional linear site responses analysis

Carolini et al. [21] examined a comparative study between 1D and 2D site analyses in terms of the transfer function. They concluded that for shallow soil classification, 1D site analyses provide accurate results, but in special cases, 2D modelling is required rather than time and information consuming. Two of the main disadvantages of 2D or 3D site analyses are the production of massive and unrealistic results when one or more detailed pieces of site information are not available. In addition, engineers tend to make the interaction and application of results as simple as possible, but the results of 2D and 3D models are not so in most situations [21].

1.3.3.2 Review of past studies on two-dimensional equivalent linear site responses analysis

The two-dimensional equivalent linear site analysis method is similar to the one-dimensional equivalent linear analysis technique in which the shear modulus and damping ratio are defined separately for each element. Raleigh damping is used to model viscous damping due to the time domain method, resulting in frequency dependent damping [17].

Stamati et al [22] presented a study on the comparison between 1D and 2D numerical analyzes based on equivalent and nonlinear methods. They showed the effects of soil properties on site response. Due to the nonlinear behavior of soil, their results based on the nonlinear method were much better than the results based on the equivalent linear 1D or 2D method. They also considered the effect of damping on site response. They also discussed peak ground acceleration (PGA) as one of the most commonly used parameters for strong motion in the time domain. They showed that the PGA can reveal the complexity of a seismic motion and the effects of frequency-dependent phenomena. They used a relationship to calculate the aggravation factor as follows:

$$\mathrm{AF} = \frac{spectral\ acceleration\ on\ the\ ground\ surface\ obtained\ from\ 2D\ analyses}{spectral\ acceleration\ on\ the\ ground\ surface\ obtained\ from\ 1D\ analyses} \qquad (6)$$

They also demonstrated that when the main excitation is increased, deamplification or amplification also increases.

In the study of A. Amorsi et al [30], a multidimensional numerical approach is presented to duplicate the observed response of a free-field site, considering each horizontal component in the north-south and east-west directions of the input motion of interest, using a hardening soil model with an elastic superposition model for small strains. By comparing the obtained accelegrams and spectra with those recorded in-situ in the free field downhole and at the surface, they aimed to demonstrate the role of the multi-directional influence of the input on the soil response. Finally, they investigated the validity of the presented method for back-prediction of seismic ground response and concluded that the HS (Hardening Soil) model is valuable and affordable for in situ

ground response analysis. This numerical method shows good agreement with recorded references.

1.3.3.3 Review of past studies on two-dimensional non-linear site responses analysis

Gaetano et al [16] performed a numerical analysis of the seismic ground response to investigate the role of the different gradients of the problem, such as the horizontal direction of the input motion and the shear wave velocity profiles. In this study, the researchers used nonlinear 1D, 2D, and 3D approaches. The 3D analysis in this study was performed using the PLAXIS3D program.

The three-dimensional method has not been investigated in this research because it is not related to the research topic. Enthusiasts can read about it in many sources.

1.4 Model tests

Since of the complexity of soil behavior, the results of theoretical and numerical analyzes are not as well validated as in nature, so a controlled model test such as a 1g shake table, may be useful at present, but this type of survey has some limitations, such as the scale effect limitation. Since 1980, as a result of the rapid development of computers, numerical methods have expanded in all areas, including the analysis of seismic ground response. Nowadays, for example, we can calculate the whole ground system using the finite element method. In fact, there are very extensive documents that have done research in this field [8]. The main objective of the research of Yonng Jin et al [8] is to compare the results of the 1g shaking table and the results of the numerical

response methods by a reliable process. For this purpose, they use the DEEPSOIL and ABAQUS programs to perform the numerical part of the research.

1.5 Downhole arrays

The nature of soils and the changing properties of these soils even at very short horizontal or vertical distances make site investigation one of the most important topics in the construction science. To date, the best method for creating a good model for estimating the behavior of the soil layers and the ground surface during a shaking event has been data collected using borehole logging. Since the first stage of in-situ and laboratory tests or their analysis is the borehole investigation, its quality is a very important issue. For this purpose, some accelerometers were installed in vertical arrays. These boreholes have provided very useful data on earthquakes during the last decades [6]. Based on the credible surface and borehole records of the vertical arrays, one objective of the vertical arrays is to validate the ground modeling of the site in question [27,33]. Three of which (Ataköy, Fatih, Zeytinburnu) in the European part of Istanbul were selected for our investigations.

1.6 Istanbul Geotechnical vertical Arrays system

In construction science, assessing the risk of existing structures and designing a new structure or retrofitting and rehabilitating an old structure is the most important issue. The first step to this process is to know the interested site and the geological and geotechnical properties of the soil. For this purpose, scientists obtain the evidence of the ground movements that occurred in the past to use in the numerical analysis and predict

a realistic behavior of the site. In the Istanbul metropolitan area, a network of borehole arrays is provided to study the amplification at different soil sites in the city [33].

During the Istanbul Microzonation Project (IMP) have supplied about 2900 boreholes were provided including 14 of which are deeper than 100 meters. These fourteen downhole are Istanbul Geotechnical Vertical Arrays (IGVA) system. The area where this project is carried out covers about 182 km^2. The decision on where to drill the most suitable boreholes was made after a careful study of the characteristics of the soil profiles, seismic susceptibility, and depth of the bedrock. The results obtained from these boreholes are used directly or indirectly in studies on seismic research [33].

Almost all of the downhole arrays in Istanbul were installed using the same methods, with some minor differences. We describe here in detail the Ataköy borehole system, which is representative of all of them. In the Ataköy zone, a drilling platform was constructed in 2005 with four boreholes at depths of 25, 50, 70 and 140 meters. PVC pipes were installed in the boreholes, and the space between the pipes and the borehole wall was impregnated with cement grout. Then disturbed or undisturbed samples were taken to perform laboratory tests on them such as SPT, classification of soil layers or depth of water table. Information that is more detailed can be found in Parolai et al [6] and Kurtulus [33].

In the Istanbul Microzonation Project, geotechnical data were obtained in two ways: 1. Drilling logs of at least 30 m depth using standard penetration tests and laboratory tests. 2. Deep drilling at 25 different locations to locate the engineered bedrock

using PS and seismic wave velocity measurements. Engineering bedrock in all wells was calculated by combining geotechnical data and seismic survey profiles. Shear wave velocity profiles for the soil layers between the bedrock and the ground surface were determined using SPT blow counts based on an empirical relationship proposed in the literature. However, there are modifications based on in-situ borehole measurements when this was required [29]. Figure 4 demonstrates a part of explained systems in the European side of Istanbul.

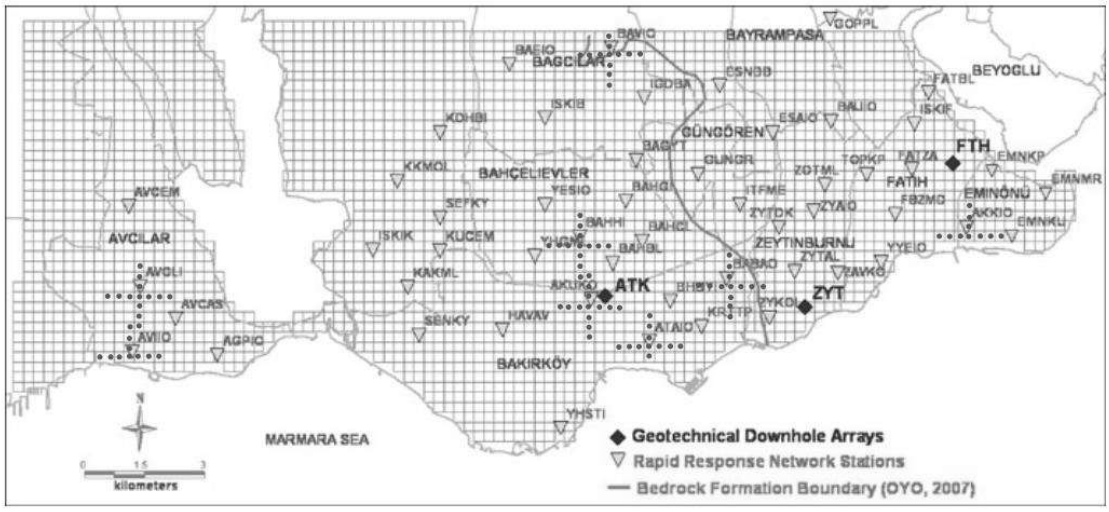


Figure 4: The location of IRRN and IGVA stations in European side of Istanbul [33].

1.7 Istanbul Rapid Response Network

Engineers and scientists can reduce the potential impact of large ground motions on urban areas by taking timely and proper action after catastrophic earthquakes. An earthquake early warning system enables clean emergency shutdown of damage-prone systems such as data centers, transportation, or power plants, especially in industrial

areas. Nowadays, some of the mentioned systems are either under construction or in the planning phase in some countries such as Mexico, Romania, California, Japan, Taiwan, Turkey and Greece [36].

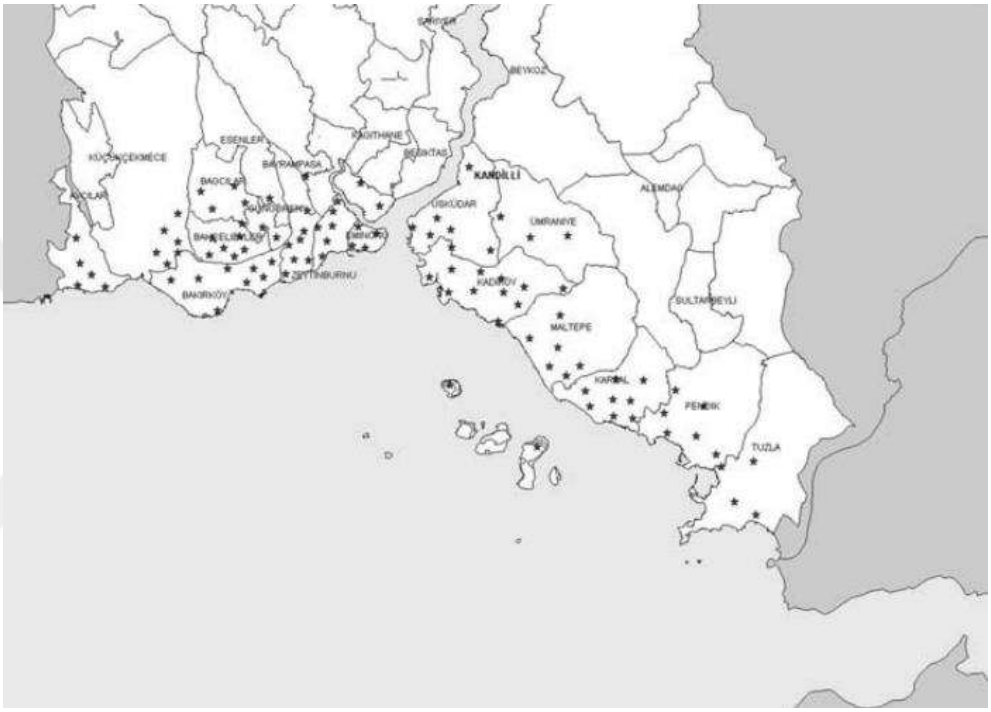


Figure 5: The location of IRRN stations in Istanbul [36].

Forty recorders were considered for strong motions that would occur on critical structures, as shown in Figure 5. The sum of these boreholes is Istanbul Rapid Response Network [36].

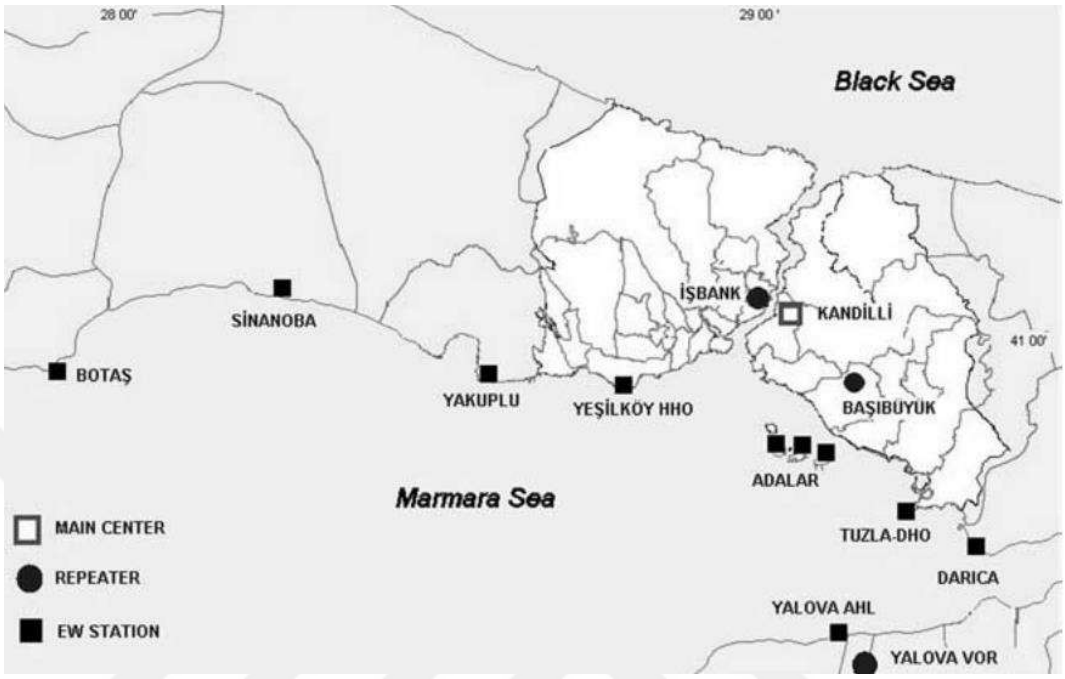


Figure 6: The location of early warning system stations in Istanbul and around of it [36].

As shown in the Figure 6, collectively, this network and its functions are referred to as the Istanbul Earthquake Rapid Response Network and Early Warning System (IERRNEWS) [36]. The IERREWS is planned and operated by Boğaziçi University with logistical support from the Istanbul government. The headquarters of the First Army and the construction of the system are provided by GeoSigInc, and communications are provided by ARIAGSM [36]. IERREWS consists of four main components:

1. monitoring system

2. communication link

3. data processing facilities

4. systems for issuing and transmitting rapid response information and early warnings [36].

The four main objectives of IERREWS are:

1. reliable data for accurate, effective characterization of ground motions and damage through other rapid maps such as shaking, damage, and casualty maps

2. recorded motions for subsequent analysis of earthquake effectiveness of structures

3. empirical basis for long-term improvements in seismic microzonation, seismic requirements of building codes, and design guidelines

4. seismological information for developing considerations of earthquake generation at the source and propagation of seismic waves [36].

IERREW has appropriate guidelines for information on stations and instruments [36]. In addition, the study by ERDID et al [36] provides many details about the distances between the boreholes and instruments and how they work. Two methods based on spectral displacements and instrumental intensities are used by the Department of Earthquake Engineering at Boghazici University to generate information on fast responses [36].

1.8 Earthquake waves types and wave description

The typical regions have significant differences in the wave velocities of the soil layers, soil classification, thickness and depth of the seismic bedrock, which consequently

play an important role in the analysis of the site response [22]. The body wave velocity of an earthquake includes two types as shown in Figure 7.

The velocity of body waves are sorted in two important groups:

1. The velocity of compressional waves (V_P).

2. The velocity of shear waves (V_S) [37].

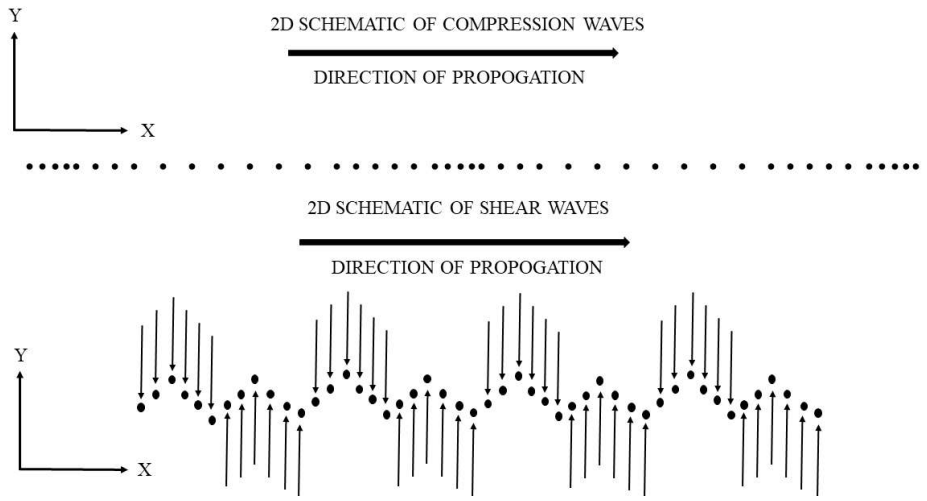


Figure 7: The schematic of propagation of body waves of an earthquake into the particles of a soil or rock layer.

An important feature resulting from the study of boreholes is the shear and compression wave velocities (V_S and V_P) of the soil layers from the ground surface to the bedrock. One method for determining these values is suspension PS velocity logging Due to the occurrence of incidence bodies or surface waves with non-vertical paths at sites underlain by irregular soil layers, the 1D site-response method is no longer valid in this case because it can result in complex wave fields that increase the duration of the motion

and its amplitude. A typical example is sedimentary valleys that are very soft against bedrock at the surface. In this case, body waves can become surface waves [25].

1.8.1 Shear waves

One of the earthquake body waves and at the same time the most important lithological parameter is the shear wave velocity. It has implications for all geological and geotechnical investigations [37]. Shear wave velocity is more important than V_P for studying the site response. Moreover, it is determined by rather difficult and costly methods such as core analysis. Therefore, it is important that we can predict the shear wave velocity using a low-cost, fast, and reliable system [37,38]. There are two ways to measure V_S:

1. In-situ geophysical tests, unlike laboratory tests, do not require undisturbed samples.
2. In the laboratory, we can measure G_{max} values using a resonant column device or flexure elements.

However, laboratory test requires high quality, undisturbed samples, so this is an expensive task. Moreover, this is only possible for discrete soils, while it cannot provide realistic results for entire soil layers. One of the most important parameters for soil research is the shear modulus for small strains, or G_{max} for short, which refers to strains on the order of 10^{-3}% or less. According to elastic theory, G_{max} is related to the shear wave velocity V_S by the following equation:

$$G_{max}=\rho.Vs^2 \quad (7)$$

Where G_{max} is the shear modulus in Pa, V_S is the shear wave velocity in m/s, and ρ is the soil density in kg/m^3 [39]. Moreover, the response of a large volume of soil can be measured under in-situ loading conditions. Therefore, this method is the preferred option for engineers to measure low strain shear characteristics. However, when estimating V_S profiles by indirect methods such as in-situ tests, correlations, and shear strength, the uncertainty of V_S values increases [39]. In this study, we have attempted to improve the shear wave profiles and minimize these errors.

1.8.1.1 Review of past studies on shear wave velocity

In recent years, many researchers have studied this topic and tried to present empirical relations and intelligent methods to predict shear wave velocity [37]. Overall, intelligent relations are more popular and reliable than empirical equations among engineers. They are also easier to learn, such as ANN, LLD. Some of the intelligent methods are a combination of others, so they provide more efficient results, but also require more time and technologies [37]. For example, Mehrgini et al [37] presented a new neural Elman network-based method to predict V_S using gamma ray (GR), compressional wave velocity (V_P), resistivity (LLD), bulk density (RHOB), neutron porosity (NPHI) and water saturation (S_ω). To verify the performance of the presented method, they compared it with other well-known empirical methods, which mentioned in that study. In this context, they used an optimization procedure to obtain the best shear wave velocities, the Levenberg-Marquadt and Paticlen Swarm Optimization (LMPSO)

algorithm. They concluded that their relationship is more reliable and accurate compared to the empirical relationships. It should be noted that some of the empirical and intelligent relations are valid for a specific domain [37].

Manakou et al [40] studied a 3D model to investigate the mechanical properties and geometry of the Mygdonian Basin using independent and different field data. Their main focus was on the shear wave velocity because it plays an important role in the response of the area to ground motions. They considered both vertical and horizontal variations in the sediments to increase accuracy. In this study, the researchers used the spatial auto-correlation method to evaluate shear wave velocity at some sites. This method is inexpensive and non-invasive. At the end, they validated the obtained shear wave velocity values by 1D recorded shear wave velocity measurements, indicating that their 3D models correctly represent the properties of the overlying rock [40]. L'Heureurix et al [39] presented some guidelines for estimating V_S profiles in Norwegian clays. They rely on the index properties of the soils and correlation with cone penetration data, undrained shear strength, and 1D compression parameters [39].

1.9 Investigation of current seismic provisions on the site response analyses

Earthquake codes have been written all over the world for predicting the response of sites and designing earthquake-resistant structures, such as the Turkish Earthquake Code (TEC), the Euro Code 8 (EC8), the Uniform Building Code (UBC), and the International Building Code (IBC). Many of studies have shown that soil properties have

a very important influence on the extent and magnitude of damage during ground motion, leading to phenomena such as liquefaction, landslide, loss of bearing capacity, etc. [41].

1.9.1 Provisions in the Turkish code

Seismic codes classify the wide range of soil materials into different groups and assign a design spectrum to each. Soil response analyses are the primary basis for developing the design spectrum [22]. Soil properties are considered in seismic codes in design spectra or constraints for soil types or controls for shear wave velocities, etc. For example, TEC has a manual for design spectra. This code has been studied for different types of structures such as concrete or steel structures. This code has been developed according to the site conditions of each country or area. Therefore, they differ significantly in their limitations or values of the corresponding parameters. However, it is designed to show the important role of soil and site characteristics and properties as the basis for all structures or non-structures [13,24,41,42].

In an overview, site conditions are divided into three categories in the earthquake codes. These classes are called local site classes or subsoil classes, ground types, and soil profile types. Table 1 shows some examples of the ground types and shear wave velocities given in some codes [41]. These soil classifications were made based on standard penetration test blow-count representative V_S and some others. There are many other parameters, which affect the response spectra, such as the magnitude of the ground motion, the distance between the source and the site, the classification of the site, and... .

In general, there are two types of response spectra in codes, based on acceleration or displacement. In addition, they are divided into elastic and in-elastic categories[41].

Table 1: examples of ground types defined in the different of seismic codes [41].

TEC			UBC, IBC & FEMA 368		EC8	
	Ground types	Description	Ground types	Description	Ground types	Description
Z1	–	Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks $V_s >$ 1000 m/s; Very dense sand, gravel V_s >700 m/s; Hard clay, silty clay $V_s >$ 700 m/s	S_A^*	Hard rock V_s >1500 m/s		
	$h_1^+ \leq 15$ m	Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity $V_s \approx 700 - 1000$; Dense sand, gravel $V_s \approx 400 - 700$; Very stiff clay, silty clay $V_s \approx 300 - 700$	S_B	Rock $V_s \approx 760 - 1500$	A	Rock or rock-like geological formation including most 5 m weaker material at the surface $V_{s,30}$ >800 m/s
Z2	$h_1 > 15$ m	Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity $V_s \approx 700 - 1000$; Dense sand, gravel $V_s \approx 400 - 700$ Very stiff clay, silty clay $V_s \approx 300 - 700$	S_C	Very dense soil or soft rock $V_s \approx 360 - 760$	B	Deposit of very dense sand, gravel or very stiff clay, at least several tens of m in thicknesses, characterized by a gradual increase of mechanical properties with depth $V_{s,30} \approx 360 - 800$
	$h_1 \leq 15$ m	Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity $V_s \approx 400 - 700$; Medium dense sand and gravel $V_s \approx 200 - 400$; Stiff clay, silty clay $V_s \approx 200 - 300$				
Z3	$15\text{ m} < h_1 \leq 50$ m	Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity $V_s \approx 400 - 700$; Medium dense sand and gravel $V_s \approx 200 - 400$; Stiff clay, silty clay $V_s \approx 200 - 300$	S_D	Stiff soil $V_{s,30} \approx 180 - 360$	C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m $V_s \approx 180 - 360$
	$h_1 \leq 10$ m	Soft, deep alluvial layers with high water table $V_s < 200$; Loose sand $V_s \approx 200$; Soft clay, silty clay $V_s < 200$				

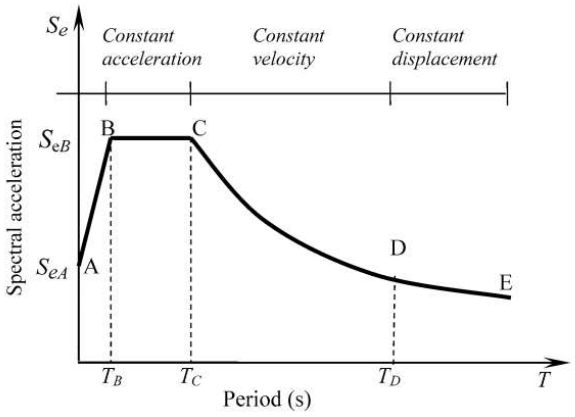


Figure 8: A typical shape of the horizontal elastic design spectrum.

In Figure 8, T the periods of the structures S_{eA} and S_{eB} show the elastic design spectra. T_B and T_C show the lower and upper limits of the period of the constant spectral acceleration branch and T_D the value defining the beginning of the range of the constant displacement response of the spectrum. There are some differences in the spectral shapes recommended by the earthquake codes. Therefore, the differences and similarities in the spectra used by the considered seismic codes have been described in several articles [41]. For more details that are interesting, you can find more and useful data in the new versions of the earthquake codes or design provisions.

1.10 Cyclic behavior of the soil

The objective of soil response analyses is to evaluate the effects of confining pressure of soil layers under a ground motion [29]. As mentioned earlier, all geotechnical studies are based on soil properties. These studies, especially the dynamic studies of soil behavior, such as earthquakes, ocean waves, etc., are cyclic and result in a cyclic response or behavior of soil types. Most dynamic properties used in dynamic site response analysis are stiffness and damping ratio (shear modulus and low strain ratio) [43,44].

There are numerous documents on the quantification of these parameters. There are also a variety of methods to measure them. Nowadays, many numerical methods are developed to calculate these parameters and the relationship between them. Many computer programs for calculating soil responses directly use these parameters or curves representing the cyclic behavior of soil under dynamic loading [43,44]. However, the

small strain and damping curves used in site response analyses should be realistic or well represent the dynamic properties of the soil of the site or area in question[43,44].

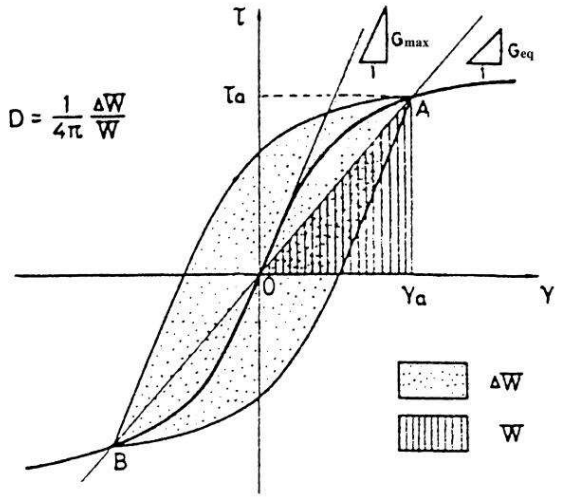


Figure 9: General schematic of equivalent damping and stiffness [43].

Figure 9 illustrates the general schematic of equivalent linear cyclic parameters and their positions corresponding to each other in general soil.

These parameters may change under different soil conditions, such as drained or undrained soils, so attention should be paid to these aspects in a site response analysis, or they should be considered as a general condition [43,44].

CHAPTER II

METHODOLOGY

2.1 Introduction

In this chapter, we explain the methodology of the site response analysis, the definition of the computer programs used in this study, the site of interest and its materials, and the definition of the ground motions used. Before we begin describing the modeling, it is important to remember that all solutions are presented in the SI system. For this study, we used numerical methods to analyze the site response. In the first part, we used the computer program DEEPSOIL V6.1for the 1D site analysis and in the second part, we used the program GeoStudio- QUAKE/W 2021.3 for the 2D analysis. Below is a brief introduction to these computer programs and how they work.

DEEPSOIL V6.1

The computer program DEEPSOIL version 6.1 was used in the 1D site analysis we performed. The DEEPSOIL V6.1 program was developed by Youssef Hashash and Duhee Park at the College of Illinois at Urbana Champaign. The DEEPSOIL V6.1 program is known as a unified one-dimensional equivalent linear and nonlinear site response analysis program that operates in either the time domain or frequency domain. The DEEPSOIL V6.1 program uses an infinite half-space with the property of convolution or deconvolution in the equivalent linear option. The program consists of

several steps, such as selection of analysis type, definition of soil profiles and soil properties, analysis control, motion control, and output. The program has its limitations, so the modeling is performed only in the flat ground models [8, 45]. The procedure in the simplest equivalent linear form of site response analysis by the DEEPSOIL V6.1 program consists of the following steps:

1- Definition of the soil profile according to the type of analysis and the general information about it, such as the analysis in the time domain or in the frequency domain using shear modulus or shear wave velocity values or pore water pressure generation.

2- Then all the soil properties such as unit weight, thickness, shear wave velocity values, etc. should be defined.

3- In the next step, the properties of the half-space should be defined, including the half-space type and the properties of the bedrock layer.

4- In the next step, the ground motion data should be imported according to the special format of the program and applied to the defined soil profile.

5- At the end, when the equivalent linear analysis type is selected, the frequency domain properties should be defined and then the profile should be analyzed, and the results exported [45].

GeoStudio- QUAKE/W 2021.3

In the 2D site response analysis research, the computer program QUAKE/W version 2021.3 was used. QUAKE/W 2021.3 is a part of the computer program

GeoStudio 2021.3. This program provides a very extensive multidimensional space and options for the analysis of any structure under any situation. This part of the program can be used alone or integrated with other parts of the GeoStudio 2021.3 collection. The program QUAKE/W 2021.3 was introduced as a geotechnical finite element program for structures subjected to dynamic and static loads [46]. Briefly, the main steps of modeling and analysis with this program are as follows:

1. The first step is the selection of geometry, dimension, and type of analysis.

2. Define the geometry and scale of the project according to the required dimensions, step by step, from point to region. Then determine the size of the mesh for each element of the structure.

3. The next step is to define the characteristics of the soil layers or other structures at each stage, including the initial situation and dynamic state.

4. The next step is to draw the boundaries of the project. Realistic boundary conditions should be defined for each condition to model the project as well as possible.

5. Import the ground motions according to the special format of the program and scale or modify them if necessary.

6. In the last step after the analysis, the results can be retrieved and visualized [46].

2.2 Case Study

What brings modeling closer to reality is the correct acquisition, use and evaluation of real-world information. Typically, the input data needed for site behavior analysis are divided into four categories:

1-Site characterization: geologic or topographic configuration of soil profiles.

2-Dynamic characterization: dynamic properties of soil layers detailed by laboratory tests or by using standard curves and correlations.

3-Input of earthquake motion: A suitable time history is selected that corresponds to the probable earthquake hazard of the site in problem.

4-Analysis type: the parameters to control the procedure of the computer program [15].

2.2.1. Site definition

In the introduction part, detailed information is provided about the location of the research site and the processing and acquisition of information from it according to IRRN and IMP. A brief explanation is provided here again. The basic information about the boreholes and soil stratification investigated in this study is obtained from IMP and IRRN. The detailed geotechnical investigations conducted on the European side of Istanbul are the first phase of the Istanbul Microzonation Project (IMP). The region of interest was divided into cells of 250×250 m in a grid system and investigated using boreholes, laboratory tests, and seismic parameters measured in situ for soil profiles in

each cell, such as shear wave velocity values extending to the engineered bedrock [18,47].

The Istanbul Rapid Response Network (IRRN) stations are scattered throughout the city. There are 55 IRRN stations in IMP, and there are also three Istanbul Geotechnical Vertical Arrays (IGVA) in the area of interest, extending down to the engineered bedrock. Many earthquakes that occurred nearby were recorded by IRRN and IGVA stations, e.g., Çınarcık (12.03.2008) ML = 4.8, Kütahya (19.05.2011) ML = 5.9 or Gökçeada (24.05.2014) ML = 6.5 etc. [18,47]. Three vertical arrays and their data are used for this research. Below is some important information about them.

2.2.1.1 Ataköy vertical array

The Ataköy vertical array site is located near Atatürk International Airport. The general soil classification of the site is stiff soil and rocky. The bedrock and soil formation are of Miocene age and are referred to as the Güngören Formation, which is underlain by the Ceylan Formation of Eocene age. The Ceylan Formation is defined as a bedrock formation at a depth of 100-150 m in the corresponding part of the city with sandstone and clay stone soil classification. In addition, the Güngören formation is mainly composed of hard clay and sand layers including limestone. In this vertical arrangement at 110 m depth, dense clay or hard sandy clay was found at the first depth, followed by a sandstone layer. After the first 5 meters of depth, there is a 32 m thick limestone layer. Using PS logging tests and geotechnical soil investigations, the Vs profile was established. The average shear wave velocity profile for the upper 30 meters is 287 m/s. Ground motions are recorded in all vertical arrays using internal three-component

accelerometers. The border of the separation of the bedrocks in European side of Istanbul with the red line can be seen in Figure 4. The Figure 10 shows a brief soil classification and shear wave velocity profile of this vertical array [33].

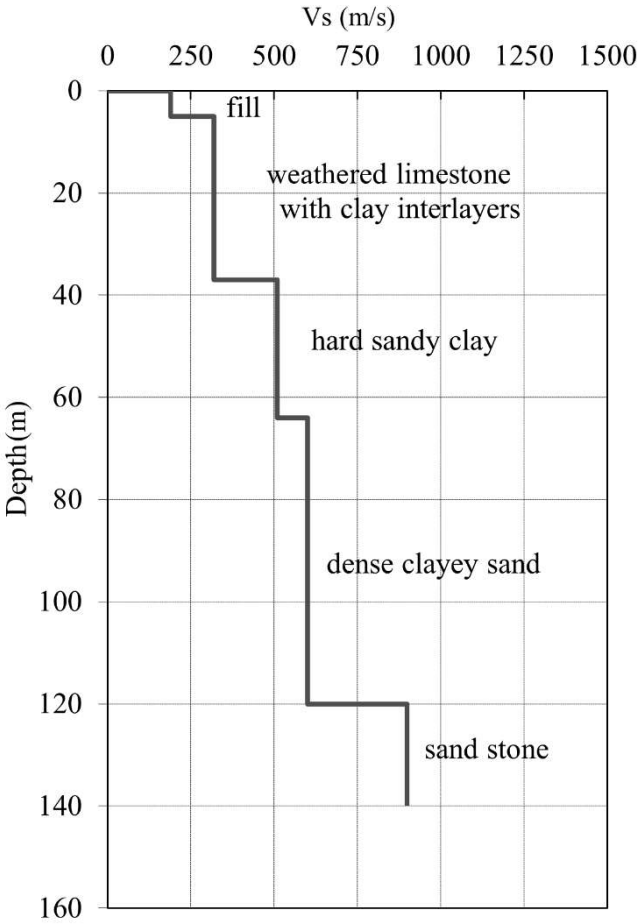


Figure 10: Soil classification and shear wave velocity profile of Ataköy vertical array.

2.2.1.2 Zeytinburnu vertical array

The Zeytinburnu vertical array site is located near the coastline in southwest Istanbul. The general soil classification of the site is soft soil/rock. The soil profile consists of Miocene age units known as Güngören and Bakırköy. The soil profile shows that the Bakırköy Formation covers the Güngören Formation. It is associated with

limestone and clay layers. The Trakya Formation is defined as a basement formation at 200-250 m depth with a combination of sandstone and shale (greywacke). The profile overview shows alternating clay and silt layers up to 240 m. Between 5 and 34 m depth there is a limestone layer. The average shear wave velocity was determined using PS and on-site geotechnical investigations. This value for the Zeytinburnu array equals to 197 m/s. Figure 11 shows a brief soil classification for this vertical array [33].

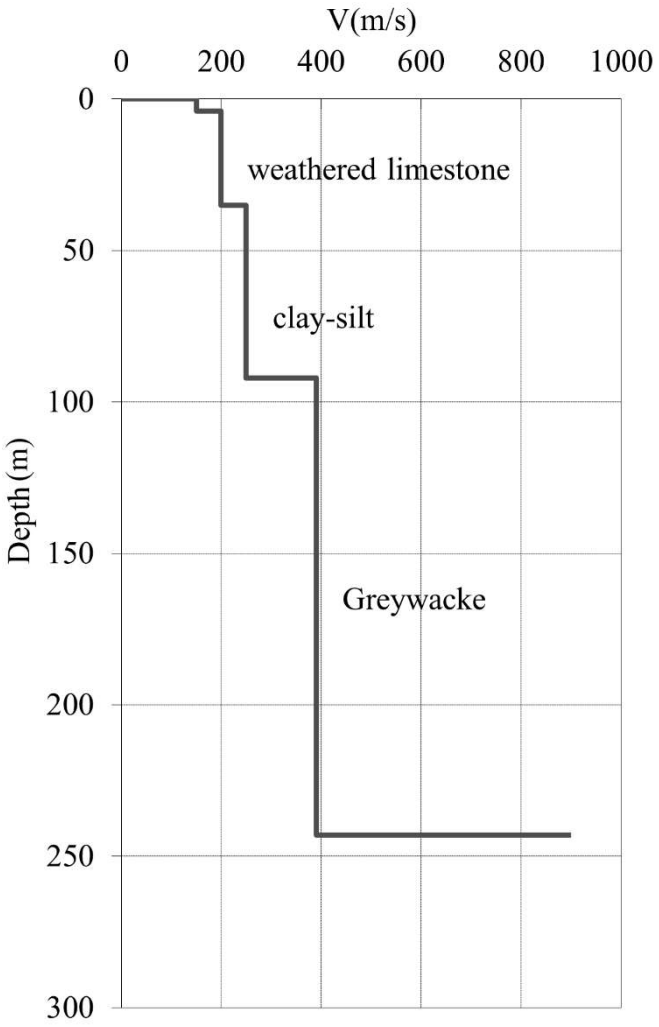


Figure 11: Soil classification and shear wave velocity profile of Zeytinburnu vertical array.

2.2.1.3 Fatih vertical array

The Fatih Vertical Array site is located near the historic mosque. The geological characteristics of the soil profile of the array are similar to those of the Zeytinburnu vertical array but there is one important difference. The depth of the bedrock is about 60-80 m. The profile overview corresponds to the combination of alternating clay, silty sand to bedrock. Another important point is that the upper 40 m of the greywacke is highly weathered. The VS30 for this vertical configuration is 335 m/s. Figure 12 illustrates a brief soil classification of this vertical array [33].

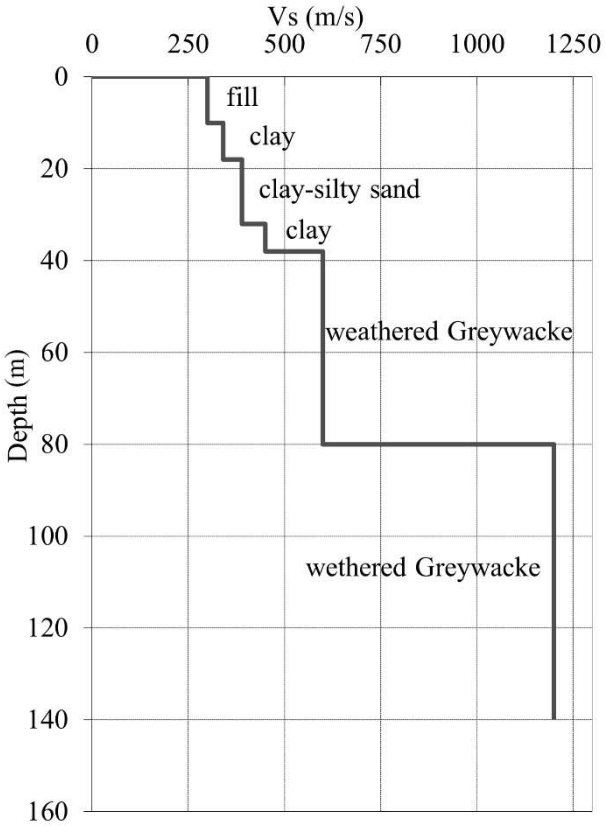


Figure 12: Soil classification and shear wave velocity profile of Fatih vertical array.

These vertical arrays recorded both earthquakes (Gökçeada and Kütahya) at the bedrock level and in the directions used in this study.

2.2.2 Input motions

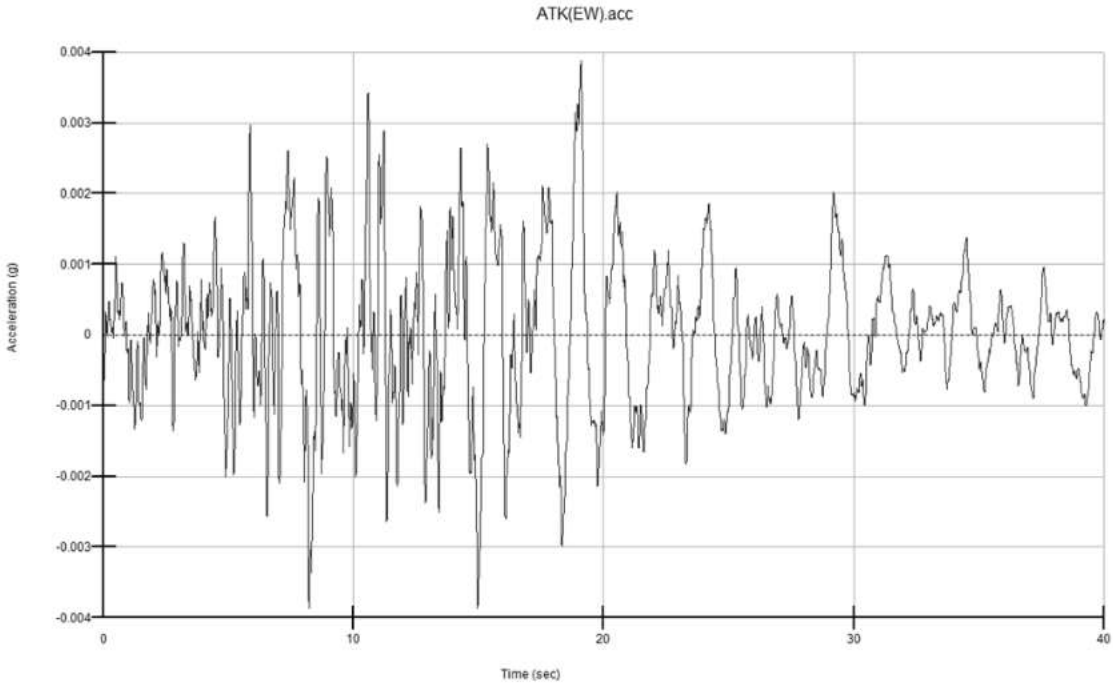
There are two types of input motions: External and Internal. In this study, we only consider the external type of motions applied to the bedrock of the model. In addition, the rock can be flexible or rigid. In the flexible type, the input motion can be applied by a traction force on the base of the model. However, if the base is rigid, the motion is forced as an acceleration or displacement time history [25].

In the case of ground response, the analysis using the DEEPSOIL V6.1 program can add or import an acceleration time history compatible with the maximum dynamic load expected in the area of interest [15]. Also in the QUAKE/W 2021.3 program, there is a direct option in the dynamic conditions to add or import the acceleration time history of the considered ground motion in the specific format of the program. The acceleration time history in the 2D program can be added to the program in two ways: horizontal and vertical. In this study, we used both of them to investigate the response of the site to the ground surface [46]. In addition, there are some options in both programs to modify or scale the ground motion inputs to a specific value [45,46].

2.2.2.1 Gökçeada earthquake

The Gökçeada earthquake of 24.05.2014 with $M_L = 6.5$ was a shallow event with a focal depth of 23 km and an epicenter distance of 166 km, which resulted in the highest PGA value for both IRRN and IGVA stations. The geographic coordinates of this event

are latitude = 40.2108 N and longitude = 25.3073 E. This ground motion was felt in an extensive area, even in Istanbul. The type of fault that caused this movement was a strike-slip rupture. This event was recorded by 23 IRRN stations and three IGVA stations (ATK, FTH and ZYT) in different components. The list of IRRN stations that recorded this earthquake is shown in the Figure 25. The corresponding documents were obtained from OYO organizations [18,47,44]. Figures 13,14,15 shows the time history of acceleration in the frequency domain of the vertical ATK array in the three components EW, NS, and the vertical component.



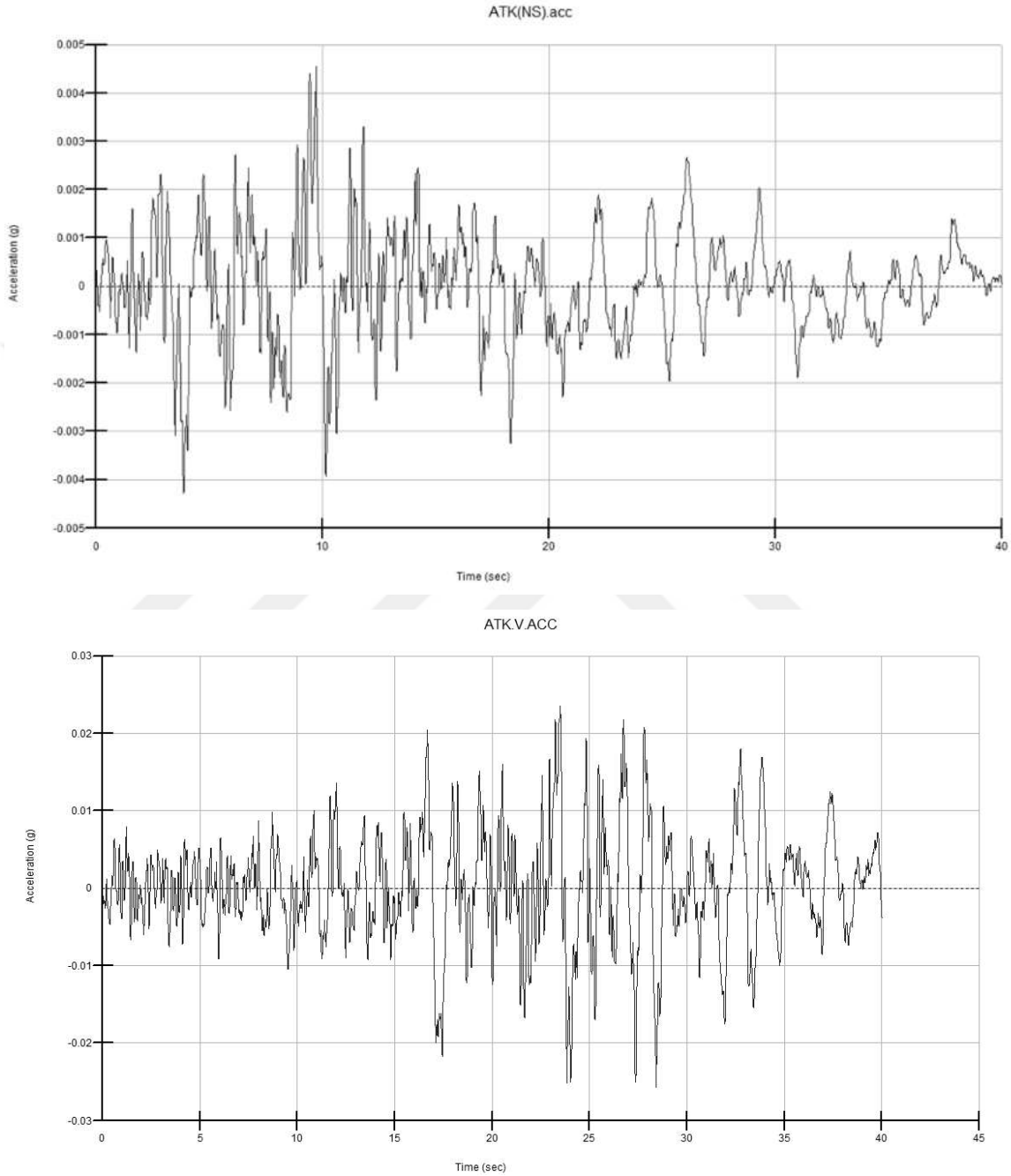
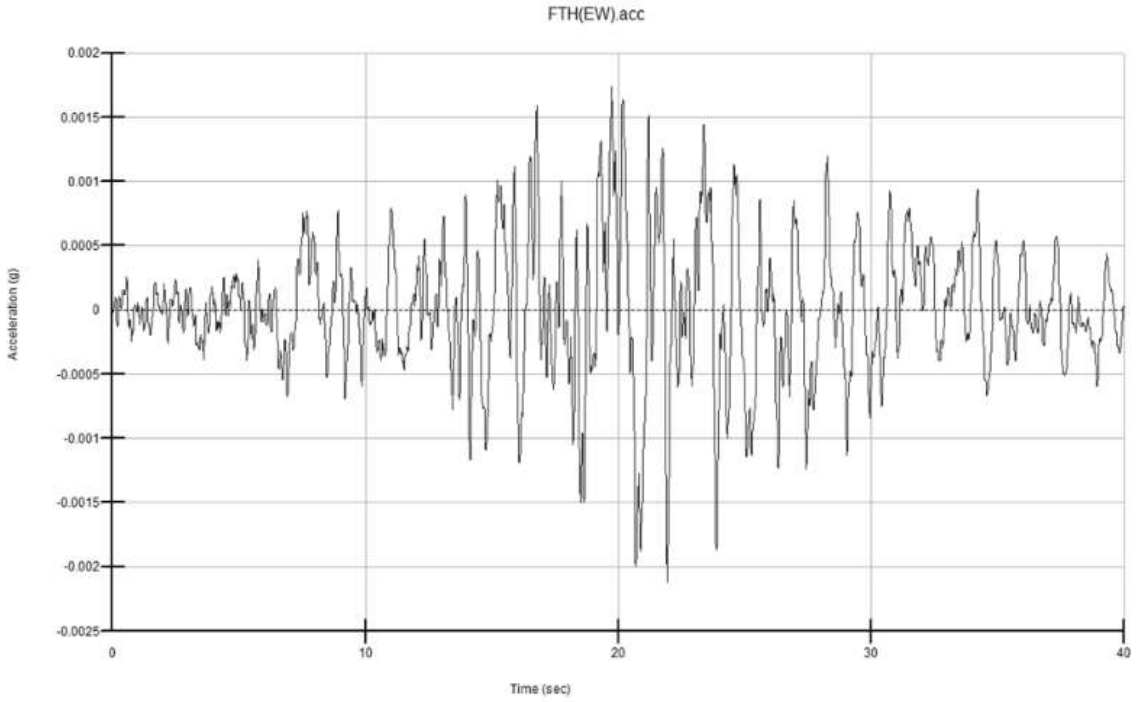
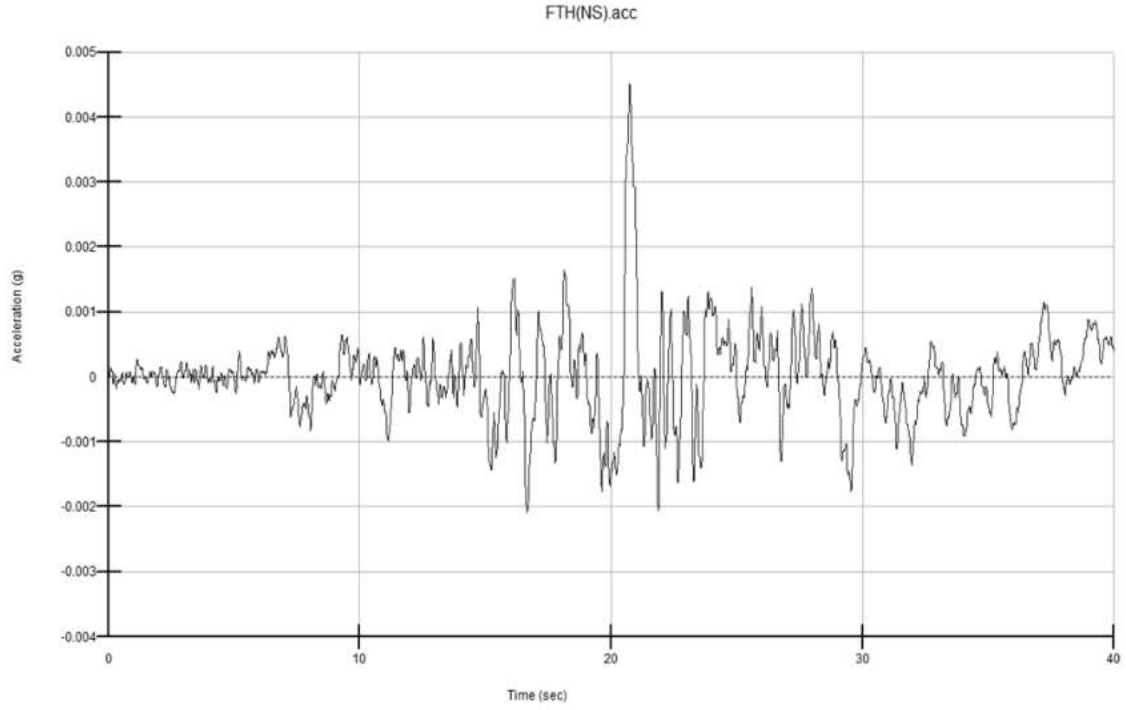


Figure 13: Acceleration time history records of Gökçeada earthquake by ATK station.


FTH(EW).acc
Acceleration (g)
0.002
0.0015
0.001
0.0005
0
-0.0005
-0.001
-0.0015
-0.002
-0.0025
0
10
20
30
40
Time (sec)


FTH(NS).acc
Acceleration (g)
0.005
0.004
0.003
0.002
0.001
0
-0.001
-0.002
-0.003
-0.004
0
10
20
30
40
Time (sec)

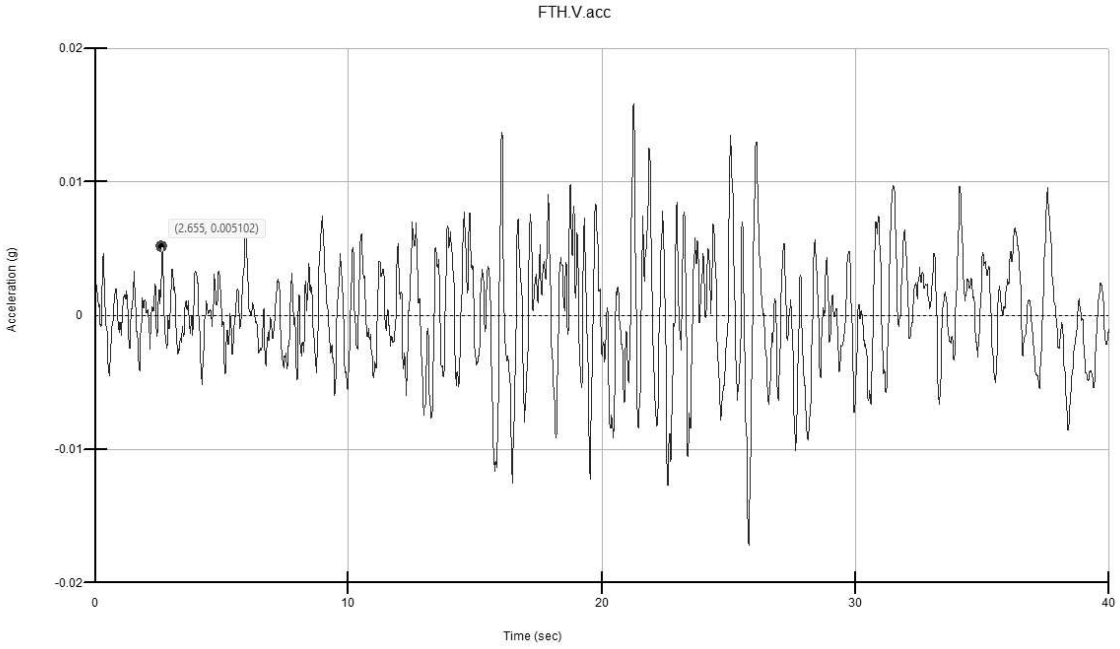
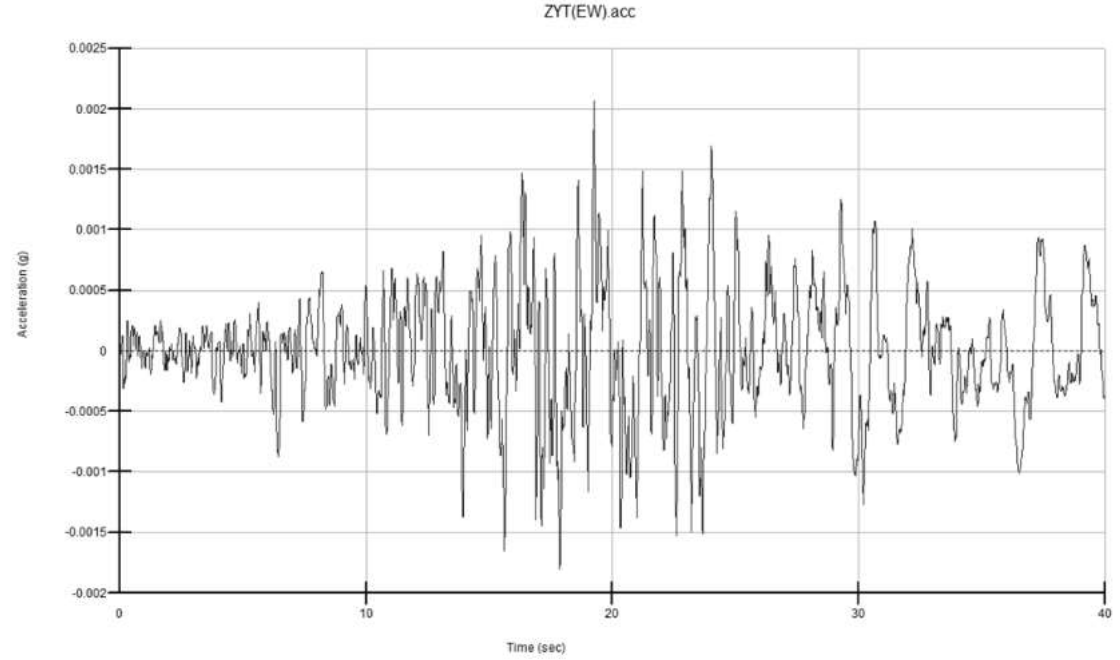


Figure 14: Acceleration time history records of Gökçeada earthquake by FTH station.



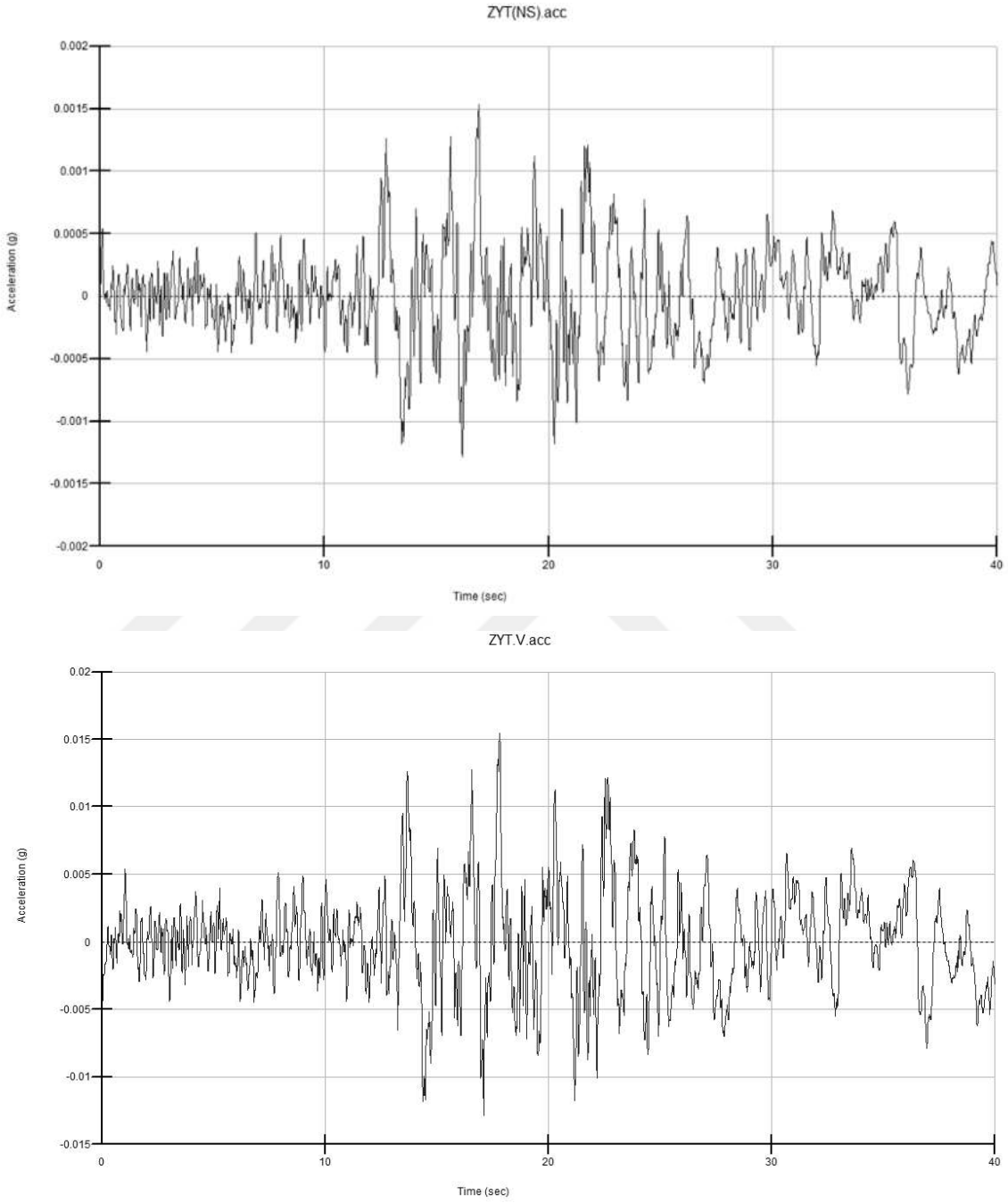
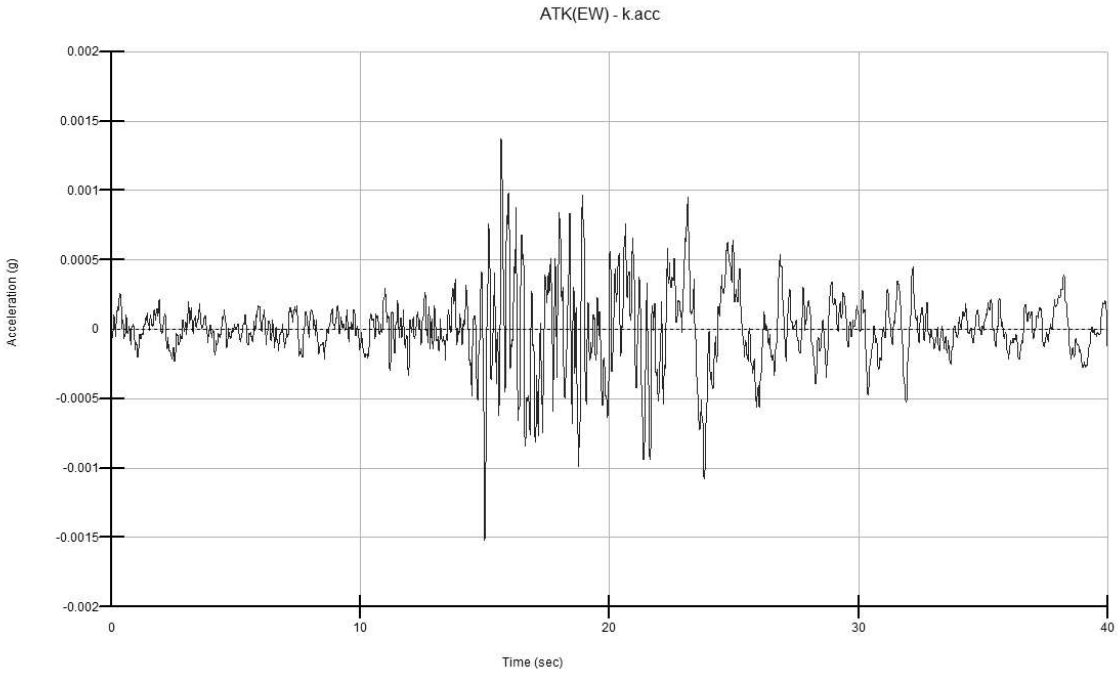


Figure 15: Acceleration time history records of Gökçeada earthquake by ZYT station.

2.2.2.2 Kütahya earthquake

Another earthquake recorded by the three IGVA stations (ATK, FTH, ZYT) was the Kütahya earthquake of 19.05.2011 with M_L = 5.7, a focal depth of 24.5 km and an epicentral distance of 205 km. The geographic coordinates of the epicenter of the ground motion are 39.1328 N and 29.0820 E. This earthquake was recorded by 22 IRRN stations in three EW, NS, and vertical components. This earthquake was felt in a wide area of the territory. All necessary station records were obtained from the OYO organization. This rupture was the result of strike-slip faulting [18,31,47,48]. Figure 31 shows the common stations of the IRRN that recorded the Kütahya earthquake. The following Figures 16,17,18 show the time history of acceleration in the frequency-time domain recorded by three vertical arrays of the Kütahya earthquake.



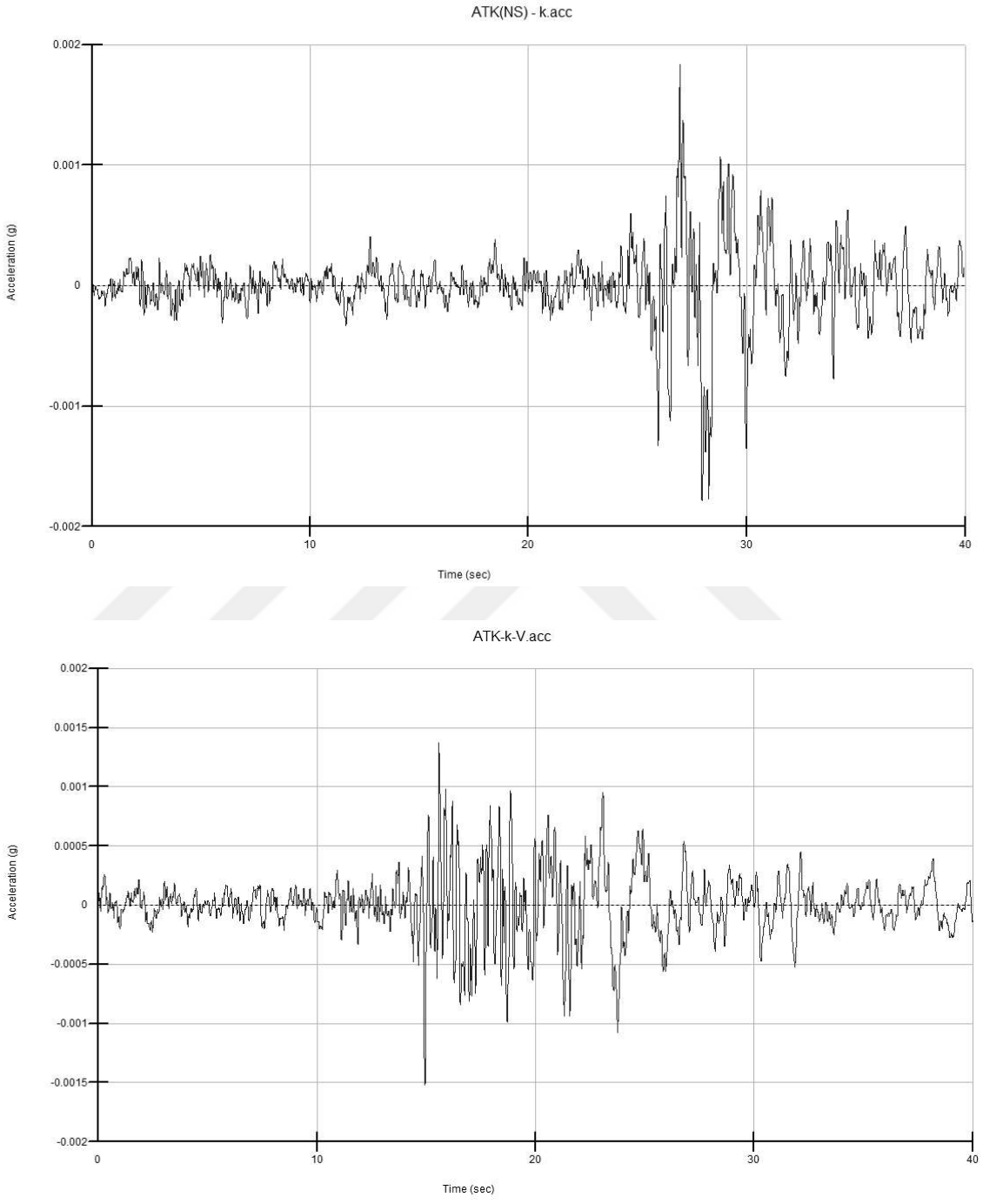
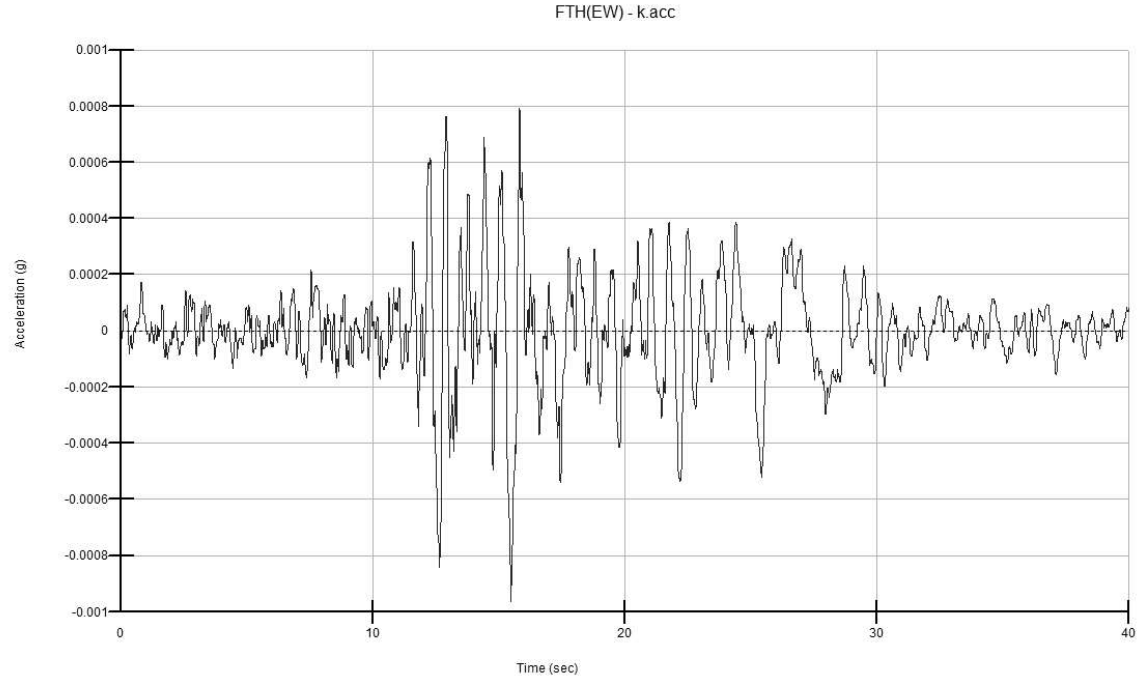
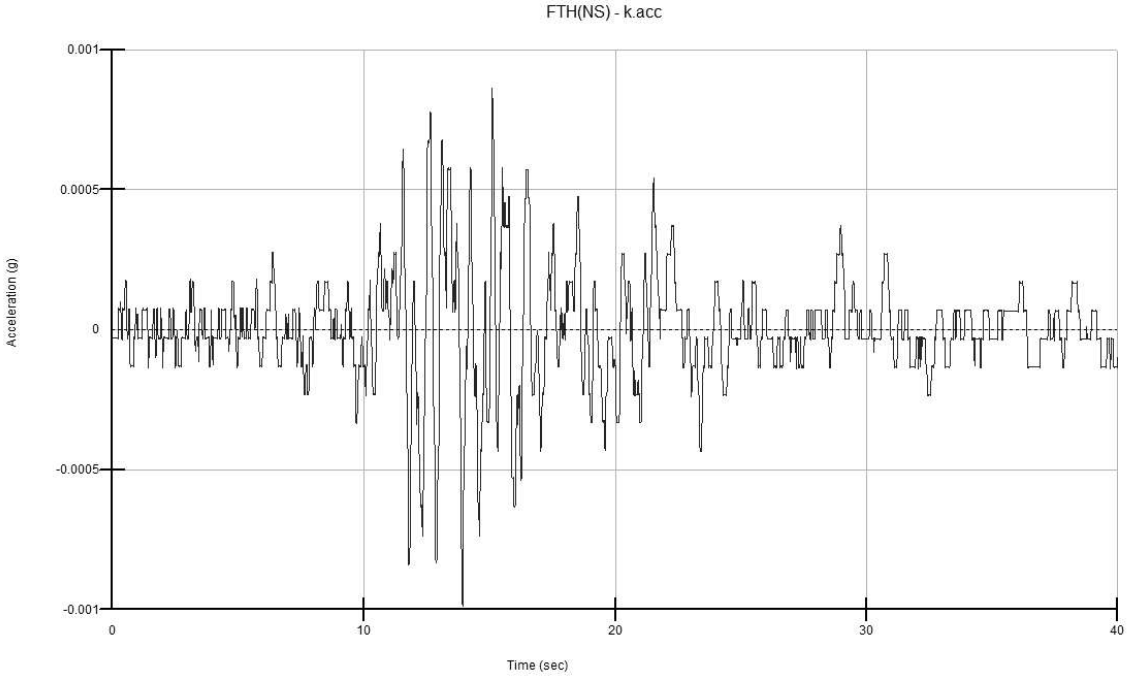


Figure 16: Acceleration time history records of Kütahya earthquake by ATK station.


FTH(EW) - k.acc
0.001
0.0008
0.0006
0.0004
0.0002
0
-0.0002
-0.0004
-0.0006
-0.0008
-0.001
Acceleration (g)
0
10
20
30
40
Time (sec)


FTH(NS) - k.acc
0.001
0.0005
0
-0.0005
-0.001
Acceleration (g)
0
10
20
30
40
Time (sec)

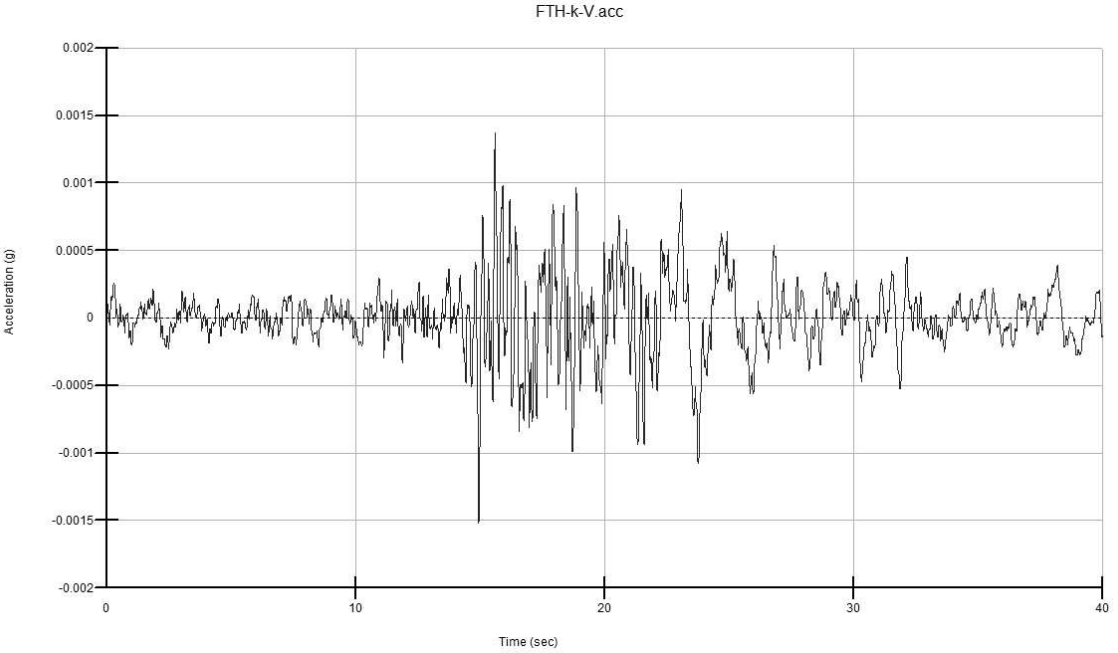
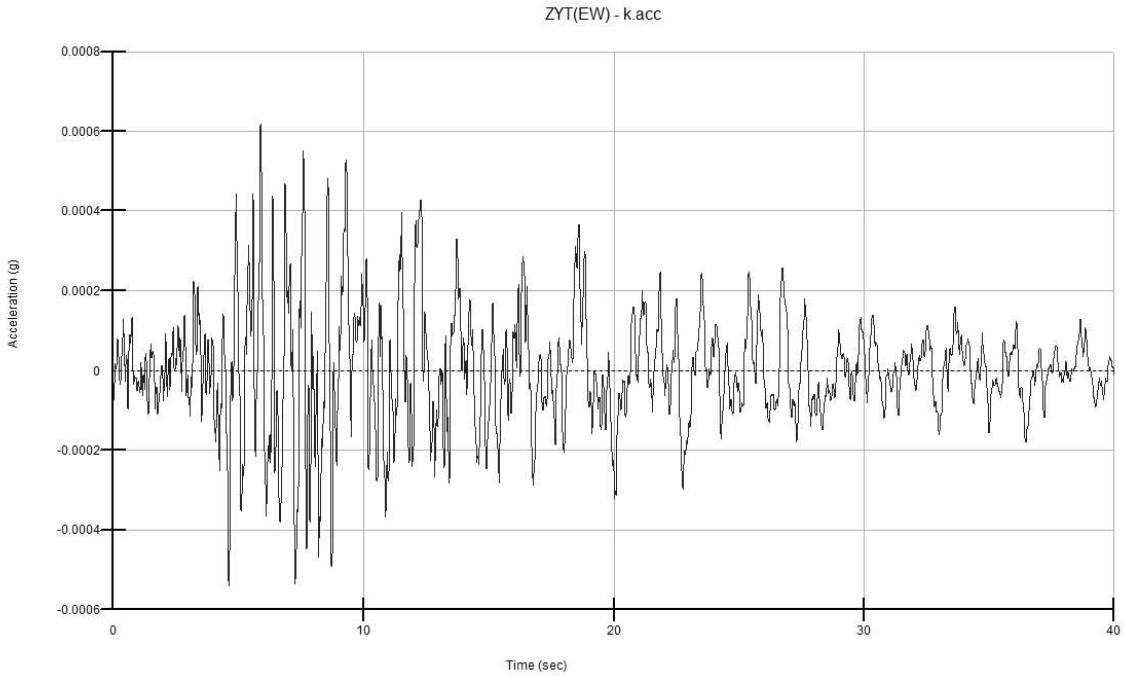


Figure 17: Acceleration time history records of Kütahya earthquake by FTH station.



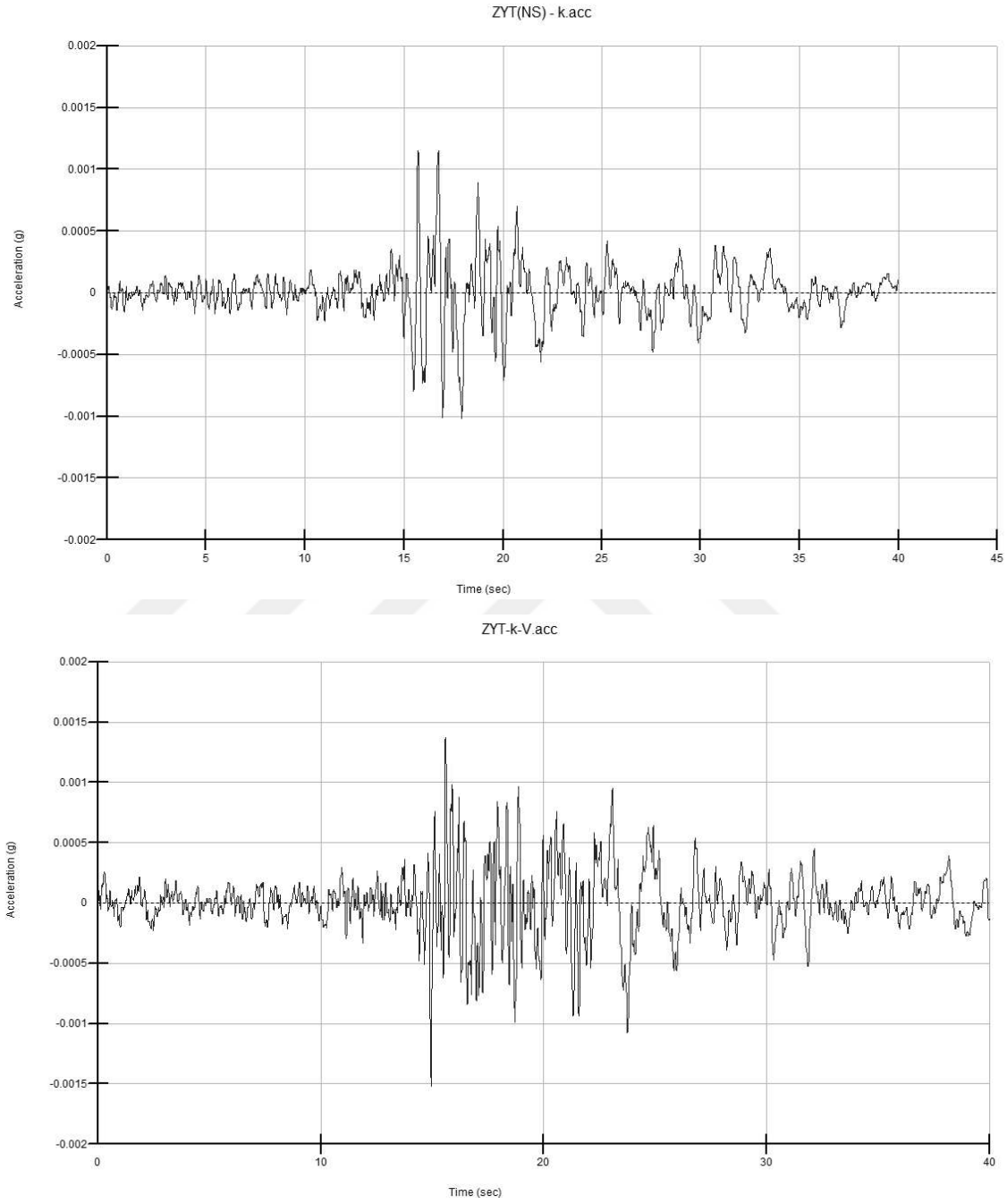


Figure 18: Acceleration time history records of Kütahya earthquake by ZYT station.

2.2.4 Damping

As mentioned earlier, damping is a value of a dissipative property of the material that has been experimentally shown to be independent of frequency. Therefore, the linear viscoelastic frequency domain may be preferred over the time domain analysis, which is why it has been easier to construct frequency independent in the past [25]. For this study, we determined the damping ratio for both analysis programs from discrete points as a function of shear strain in percentage values corresponding to the G-reduction function. For equivalent linear analyzes, there is only one-way to calculate the damping curve. As described above the relationship between damping ratio and cyclic shear strain is shown below:

$$\xi = 0.333\frac{1+\exp\left(-0.0145PI^{1.3}\right)}{2}\left[0.586\left(\frac{G}{G_{max}}\right)^2\right] - 1.547\frac{G}{G_{max}} + 1 \qquad (8)$$

The background formulation for both programs is the relationship presented. In this equation, PI represents the plasticity index of the soil. G indicates the shear modulus reduction and $\frac{G}{G_{max}}$ is a candidate for the confining pressure. In the above equation, all values should be given in units of kPa. The parameters of computing damping, such as$\frac{G}{G_{max}}$, are calculated for certain PI values, the sum of which then gives the damping ratio function [45,46].

The following Figure19 shows the reduction curves of the different soils used in this study. Table 2 presents the soil types regarding the Figure 19,20.

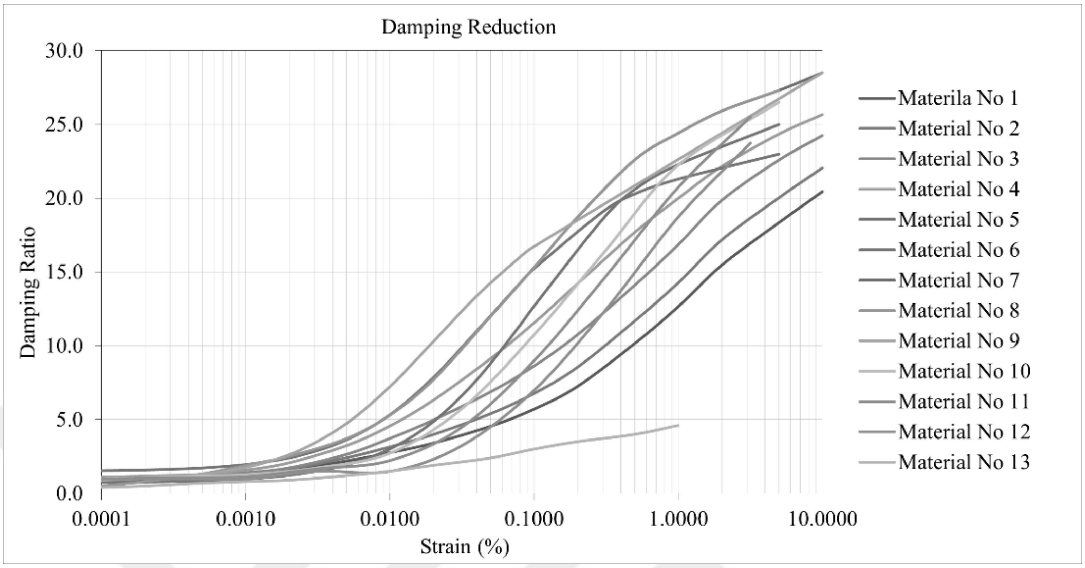


Figure 19: Damping reduction curves for different soils used in this study.

Table 2: Soil types used in the soil profiles.

Material No	Soil type
1	Vucetic&Dobry PI=60
2	Vucetic&Dobry PI=45
3	Vucetic&Dobry PI=30
4	Vucetic&Dobry PI=15
5	Darendeli Silt
6	Darendeli Sand with fines
7	Seed&Idriss Sand
8	Seed Gravel
9	Menq Gravel
10	EPRI Rock 15-37m
11	EPRI Rock 37-76m
12	EPRI Rock 76-152m
13	Idriss Rock

2.2.4 Dynamic shear modulus reduction

When a soil is subjected to dynamic loads, cyclic shear stresses develop in it. This leads to softening phenomena between the soil particles. In soil mechanics, this

phenomenon is associated with the G_{max} and confining pressure values. This parameter is a prerequisite for the finite element method [45,46].

Kramer [26] in his book presented a relationship of $\frac{G}{G_{max}}$ for different soil layers as follows:

$$\frac{G}{G_{max}} = K(\gamma, PI)(\boldsymbol{\sigma'}_x)^{m(\gamma.PI)-m_0} \tag{9}$$

$$K(\gamma, PI) = 0.5\left\{1 + tan\left[ln\left(\frac{0.000102 + n(PI)}{\gamma}\right)^{0.492}\right]\right\} \tag{10}$$

$$m(\gamma.PI) - m_0 = 0.272\left\{1 - tan\left[ln\left(\frac{0.000556}{\gamma}\right)^{0.4}\right]\right\}\exp\left(-0.0145PI^{1.3}\right) \tag{11}$$

In the above equations, γ represents a range of cyclic shear strain, $\boldsymbol{\sigma'}_x$ is a candidate for confining pressure, and PI is the plasticity index of the soil layer.

There are specific values for n (PI) in the reference handbook but in present research we used from direct data given by IMP information, so we did not use from these relationships. In the DEEPSOIL V6.1 and QUAKE/W 2021.3 programs, there is a data point option to define the properties of the soil. This allows the data to be imported directly into the program [26,45,46].

The Figure 20 presents the Shear Modulus reduction curve for soil types, which are used in this study.

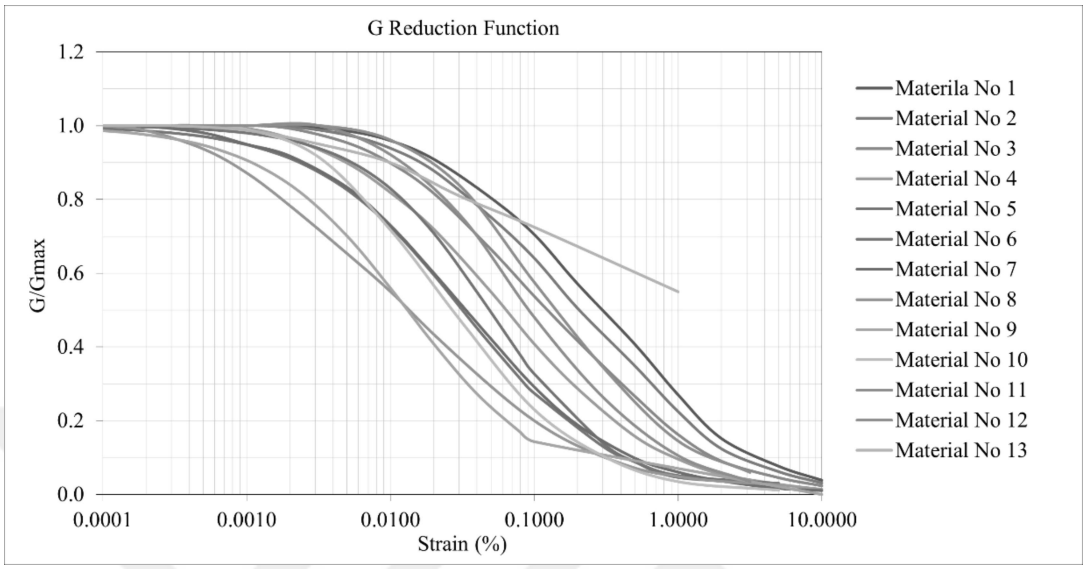


Figure 20: Modulus reduction curves for different soils used in this study.

2.2.5 Dynamic Shear modulus (G)

The shear modulus is one of the most important factors in site investigation due to site response studies is shear modulus. There are several documents in which SPT'N' values are correlated with dynamic shear modulus (G) of soil types [15].

Shear modulus is a parameter that relates the stiffness of a soil to stress state values. Overall, soil stiffness is directly related to confining stress. In general, the shear modulus of small strain, i.e. the maximum shear modulus of a soil type, is defined as a function. In the manual, there are some relations to calculate the Gmax value according to the soil classification. We declined to comment on these because they are irrelevant to this study [46].

In the 1D computer program, the user does not define this parameter, but in the 2D program, it should be defined for each soil layer. In this study, we used equation 7 to calculate the shear modulus for each layer.

2.2.6 Poisson's ratio

We calculated the Poisson's ratio (ν) for the soil layers using the following formula:

$$K_0 = \frac{\nu}{1-\nu} \tag{12}$$

Here, K_0 represents the earth pressure coefficient at rest when the soil layers are bounded by a horizontal and semi-infinite surface. There is another relationship between the velocity of the soil layers and Poisson's ratio as follows:

$$\nu = \frac{v_P^2 - 2v_S^2}{2v_P^2 - 2v_S^2} \tag{13}$$

However, since we did not deal with the compression velocity v_p of the soil layers, we did not use this relation. Moreover, v_s is the shear wave velocity in this equation [27]. The direct Poisson's ratio is only used in the 2D program (QUAKE/W 2021.3).

2.2.7 Shear wave velocity

As explained in the introductory chapter, the shear wave is one of the body waves that propagates during an earthquake. This wave can be quantified by in-situ tests or empirical or formulas. All of the previously mentioned methods have improved greatly

in recent decades. Nowadays, for example, engineers use Dipole Sonic Log (DSL) or SPT in-situ tests. However, the determination of shear wave velocity by in-situ tests is usually more time consuming and costly [49]. In the 1D program, we can compile the shear wave values directly in the soil properties option, but in the 2D program QUAKE/W 2021.3, we should include them in the Gmax value as described in the introductory part in equation 7.

2.2.8 Mesh dimension

The discretization of space and how this is done for a 2D or 3D model is very important because, unlike the 1D approach (where there are only the height dimensions of the column), there is also the width in the 2D domain (and the width for the 3D domain). The maximum frequency transmitted by a model can be estimated based on the largest elements or zones within the material [25]. For 1D program, meshing cannot be designed by the operator, but it can be for 2D program. However, in the QUAKE/W 2021.3 manual, the number of elements is considered better to limit the 2000. In this study, 2D modeling is performed with a 10 m meshing, so the number of elements depends on the shape and size of the soil profile. In addition, the type of mesh pattern is a mixed unstructured square and triangular mesh. The details of mesh types and patterns are described in the programs' manuals [45,46].

2.2.9 Bedrock

According to NEHRP provisions, bedrock is usually assumed to be the deepest layer with a shear wave velocity (Vs) ≥760 m/s of the soil profile [15]. In IMP, the

bedrock layer and its shear wave velocity values are determined using the blow count method [47]. In DEEPSOIL V6.1, a 1D program, the bedrock of each soil profile was modeled as an elastic half-space with 5% damping, considering the specific unit weight and shear wave value of each IRRN profile [45]. In the 2D modeling using the QUAKE/W 2021.3 program, based on the assignment of the specific, shear wave velocity, damping ratio, and characterization of the bedrock layer, a layer was defined as a bedrock layer in the soil profile modeling, and then all the mentioned properties were specified in the material definition option. However, since this layer behaves like a rigid rock layer, it is designed with a fixed bearing node boundary condition at the bottom of the layer. If the last layer of the boreholes is not on the rock, the rock of the borehole of the central station was continued to the bottom of the other stations according to properties of their profiles.

2.2.10 Boundary conditions

A ground motion affects a large zone of the earth's surface, but engineers are sympathetic to the response of interested small study area or only a single structure because of some reasons such as location distance from the epicenter earthquake, time of calculation and complexity of their, ... sympathized. For this reason, engineers consider supposed boundaries around the model or area of interest. In all geological computer programs, these artificial boundaries are assumed to be semi-infinite. In this case, an appropriate boundary condition has been created for the site or model of interest. Boundary conditions can be divided into two main groups: 1. elementary boundary condition (EBC), 2. absorbing boundary condition (ABC). There are two types of EBCs:

free or fixed. Due to the need for a large numerical model, which mirrored all the waves at the boundaries, have no time to return to the central province of the model. This type of boundary conditions are not suitable for small models. ABCs due to the absence of the above limitation has been welcomed by engineers. ABCs are divided into two main categories: global or local. The differences between global and local ABCs are that in the global method, each boundary node is fully connected to all other boundary nodes in both time and space, while in a local scheme, only one node is considered in the current time and in each analysis step, and perhaps a few neighboring points in time and space. From a theoretical point of view, global bounds are careful, but local bounds are approximate, which is more attractive for numerical implementation [25].

In both programs used in this study, the programs automatically apply the boundary condition for different situations into the meshing.

2.2.10.1 Nodal force and displacement boundary condition

Nodal force boundary condition usually specifies a known value for the force of a node, and nodal displacement boundary condition usually fixes a known value for the displacement of a node. Nodal displacement is more popular than nodal force boundary condition because the force at various locations is unknown [46]. The boundary conditions in the 1D site response program are defined automatically, so user verification is not required [45]. In the 2D modeling program, the internal nodes and elements are subject to the boundary conditions defined or applied by program during the analysis, but the boundary conditions in the base and the user, if desired should define lateral sides of the structure. As well, there are options to define or assign the desired nodal displacement

boundary condition, such as fixation in horizontal or vertical displacement, rotation in different directions. For the initial condition of modeling, we chose the fixed displacements in horizontal and vertical directions on the bedrock plane and the fixed displacement only in horizontal direction on the lateral boundaries. However, for the dynamic situation, we chose the option of fixed displacement in horizontal direction for the bedrock layer and in vertical direction for the lateral boundaries of the model [46].

2.2.11 Frequency domain

It is obvious that there are relatively more studies dealing with frequency formulation than with time domain, but time domain is in some cases the best or only method for solving problems [50]. However, since we perform equivalent linear analysis in this study, we used the frequency domain method. There are three methods for estimating the response spectra of Single Degree Of Freedom (SDOF) systems: the frequency domain solution, the Duhamel integral solution, and the Newmark-β method. Here we have focused on the frequency domain because of the equivalent linear analyses. Fourier Amplitude Spectra (FAS) modify the input motion by a transfer function, as shown below:

$$H(f) = \frac{-f_n^2}{(f^2 - f_n^2) - 2i\xi f f_n} \tag{14}$$

In the above equation, f_n represents the natural frequency of the oscillator as the following function:

$$f_n = \frac{1}{2\pi}\sqrt{k/m} \tag{15}$$

The ξ is a candidate for the damping ratio, which is calculated as follows:

$$\xi = \frac{c}{2\sqrt{km}} \tag{16}$$

In the above equation, m, c, k respectively are mass, viscous damping and stiffness of the system. The use of the Fast Fourier Transfer function is necessary to move between the time domain and the frequency domain (where the oscillator transfer function is put and peak oscillator is used and the peak response of the oscillator is estimated accordingly). This is proved above the frequency periphery of the ground motion, this solution works exactly [45,46]. An iteration number of fifteen is chosen for both (1D and 2D) programs. The effective shear strain ratio (SSR) is chosen to be 0.65. The acceleration due to gravity is assumed to be 9.81 m/s [45,46].

2.3 One-dimensional modeling

In one-dimensional modeling, all input materials are defined in the previous sections. To achieve the best modeling for each station, we considered the center of the 250×250 m of each cell of IMP. In this step, the initial values of shear wave velocity were used, which were obtained from the information of IMP. In addition, the pore water pressure was not considered during the 1D analysis because it is an equivalent linear analysis. After the analysis, the program provides results in the form of stress-strain diagrams, time history diagrams, spectral plots, animation of convergence and other useful results [45]. Figure 21 shows an example of modeling in 1D program for AKKIO station.

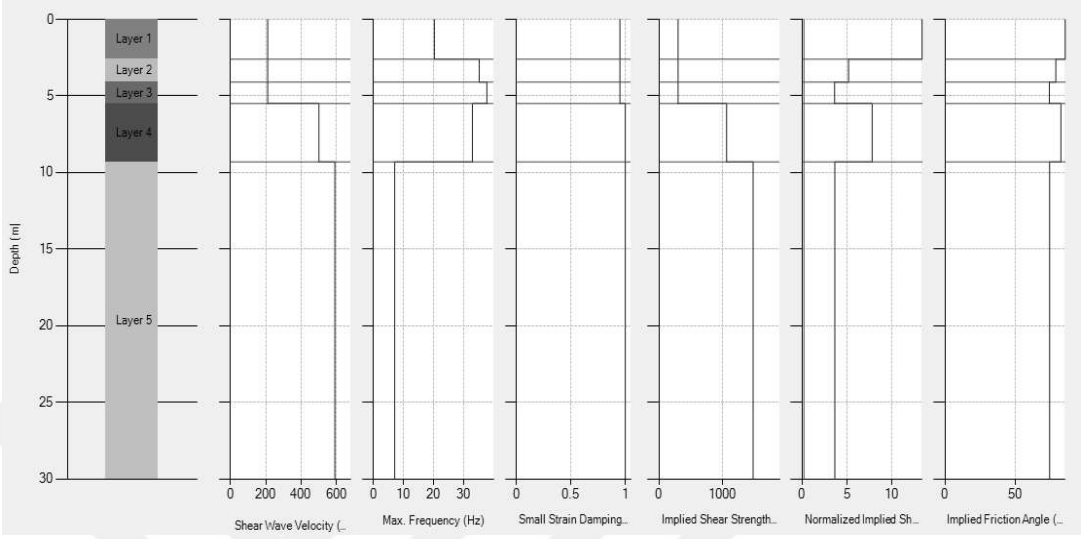


Figure 21: 1D modeling for AKKIO station.

2.4 Improvement of shear wave velocity

2.4.1 Statement of problem

As mentioned above, shear wave velocity has a very significant influence in geotechnical engineering and research about site response for construction of structures on the other hand, in the evaluation of the stability of the soil against landslides and in other case studies. Therefore, accurate measurement, calculation, and modeling of velocity are very vital for accuracy of research in any case. Due to field constraints such as machine errors, disturbed samples, insufficient operator accuracy, etc., shear wave velocity is subject to many errors and deviates from its true value. In addition, empirical relationships and approaches have inaccuracies at different locations. In numerical methods, these errors are caused by insufficient accuracy of modeling or coding, weak correlation between factors affecting V_S, and engineers' tendency to use computer programs that require less time and parameters to save time and money and for other

reasons. Therefore, from the very beginning, scientists tend to obtain a prediction and simulation method with minimum parameters and low cost with maximum realistic results. In this case, various methods of prediction, optimization, or modification have emerged to date using different sciences such as mathematics and physics, etc. [22,25,37,39,49].

There are two main databases for predicting shear wave values, including modeling based on typical log data and modeling using seismic information. Each group has advantages and disadvantages, and engineers should choose one of the two methods depending on their requirements [37]. The first group has already been explained. In this study, we use the second method to modify and define a more accurate shear wave velocity for IRRN boreholes due to improve the site response analysis and its results.

2.4.2 Past studies of the improvement of shear wave velocity

As mentioned earlier, many scientists have tried to correlate other soil parameters such as soil type, depth, overburden pressure, etc. with shear wave velocity. Then they can obtain or predict the V_S values according to the relationships and equations without spending time and money. There are many examples of methods or relationships in the literature [23,24,35,42,51,90].

Gholami et al [49] defined a methodology for determining shear wave velocity from seismic information. Their method consists of four steps including some optimization approaches such as Fuzzy Inference System, Neural Network, Support Vector Regression to obtain the best and more realistic values for shear wave velocities.

Upon and his team [51] worked on a method to predict shear wave velocities by using a Train set of information, selecting them randomly and assigning them to some optimization methods. They then combined and compared them, trying to find the best method with the least error. Wang et al [52] discussed a new hybrid prediction model for computing shear wave values based on LSTM neural network optimized by PSO algorithm and compared with some other models. His team obtained a satisfactory result on this topic.

2.4.3 Simulation for the improvement

For the modification process, we first collected all the information about the IRRN stations that recorded both the Gökçeada and Kütahya earthquake. At this point, we developed an algorithm to improve the recorded shear wave velocity profiles of the stations based on the spectral acceleration curves. The following algorithm in Figure 22 represents the main roadmap for improving the shear wave velocities. First, the shear wave velocity profile of each IRRN station is defined for the enhancement system. Then, a data volume of ±30% of the respective value is included in the respective profile. Randomly, one of them is selected and forms the new profile V_S. Then this new profile should go through some filters and checks to get the best result. First of all, the shear wave velocity values of each layer of the top 30 meters should be below 500 m/s. Once this filter is passed, the system should check the differences of shear wave velocity between the interconnected layers. It should be less than 500 m/s. The next step should be to check the shear wave velocity of the bedrock layer. It should be equal or greater than 760 m/s.

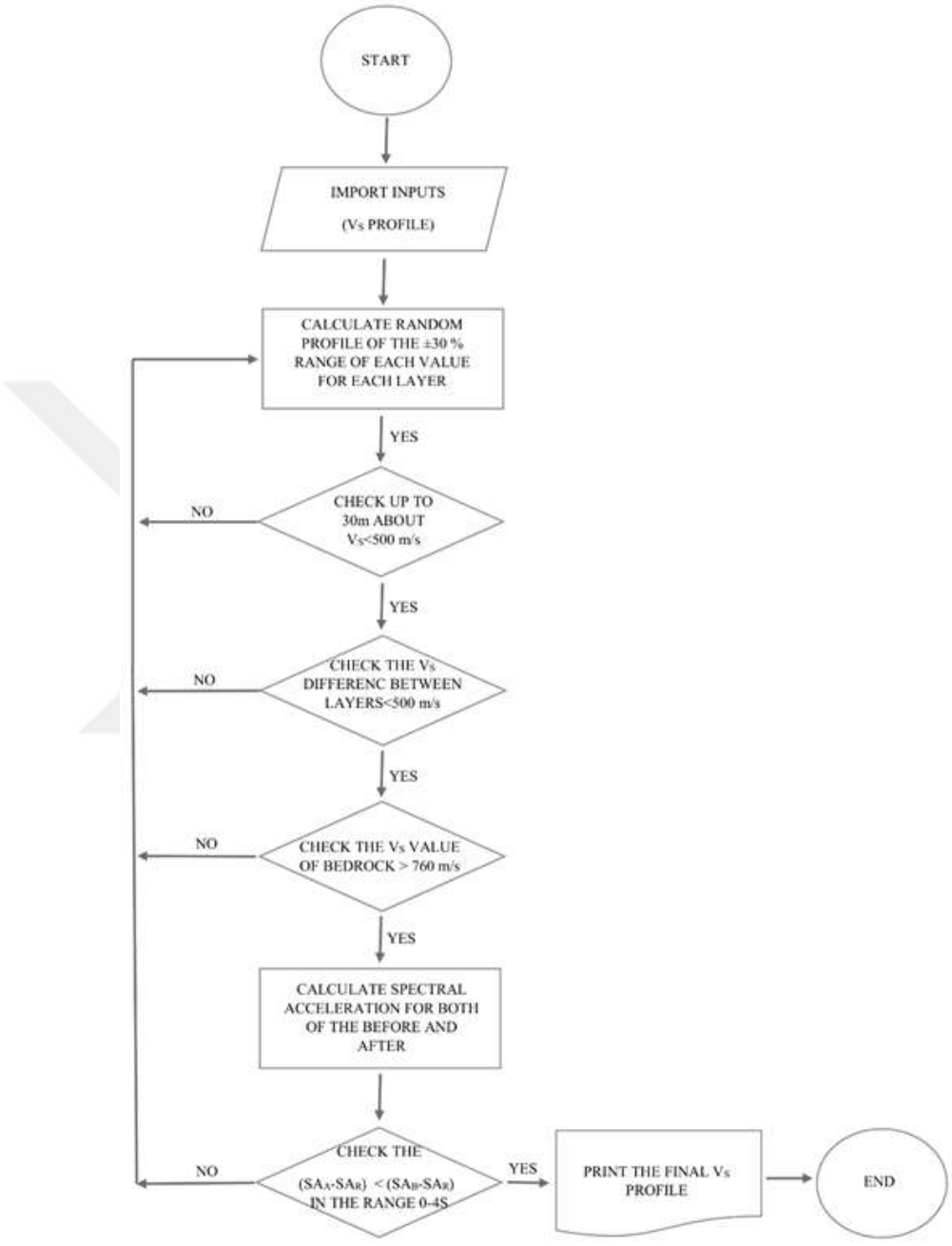


Figure 22: The main roadmap for improving the shear wave velocities.

When all the above conditions are confirmed, the calculated shear wave velocity profile is imported into the 1D site analysis computer program (DEEPSOIL V6.1) and the analysis is performed to obtain the spectral acceleration at the ground surface. The

squared differences of the new and the recorded spectral values at the ground surface in the range of 0-4 seconds should be calculated. If this value for any point on the graphs is greater than the initial values (input data given by IRRN Statins), then this process should go back to the first step and create a new shear wave velocity profile and check the above conditions for it.

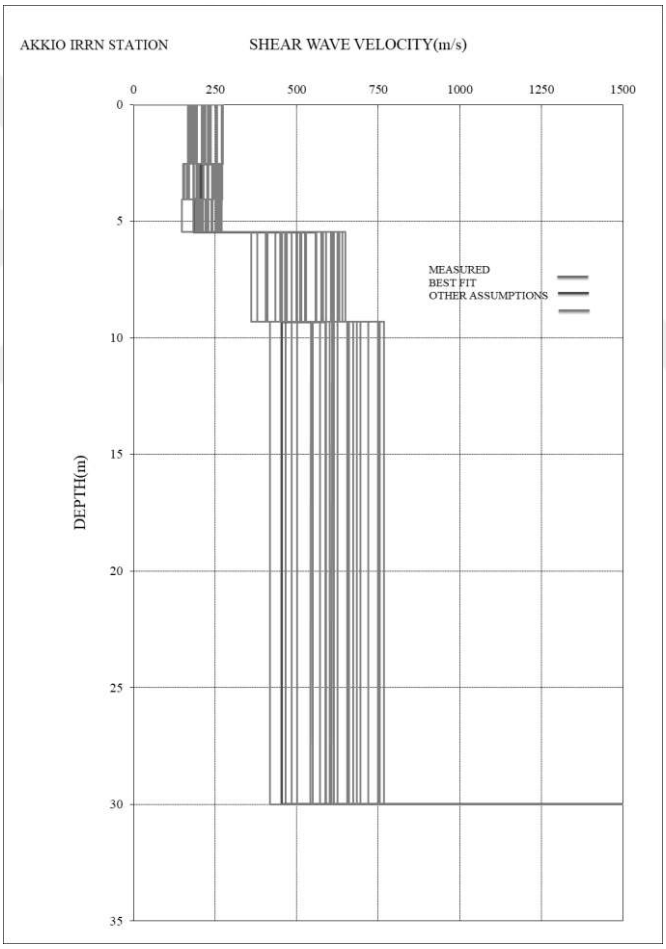


Figure 23: Best 30 obtained shear wave velocity profile using enhancement procedure for AKKIO station.

This process was continued until the best 30 profiles were obtained. Figure 23 presents best 30 obtained shear wave velocity profile using enhancement procedure for AKKIO station. Then the best of them, based on the minimum difference between the

recorded and the improved spectral acceleration, is selected for the rest of the study. To validate the calculated values, we used and analyzed the data from the Kütahya earthquake in the same stations.

2.5 Two-dimensional modeling

In two-dimensional modeling, all input materials are defined in the previous sections. Using the side boreholes of the main borehole, an attempt was made to design the soil stratification as accurately as possible, and if accurate information is not available somewhere, some values for the thickness of the strata are assumed by linear interpolation considering the shear wave velocity and kind of the strata. For the design in two directions EW and NS, we used three boreholes from east and west and two boreholes north and south of the main borehole (Figure 4), respectively. Therefore, the length of the model for each station separately is 1500 m. In addition, pore water pressure was not considered in the 2D analysis because it is an equivalent linear analysis.

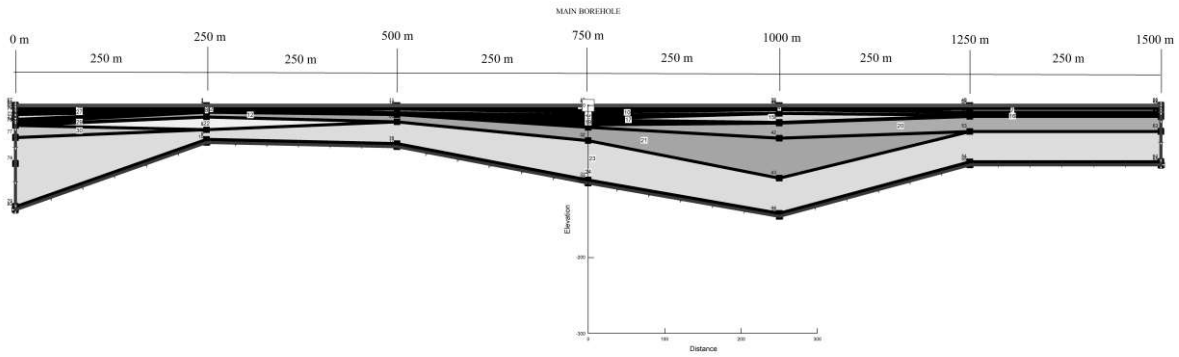


Figure 24: 2D modeling for BAHHI station in EW direction.

Almost all existing studies are concerned with the effect of vertical soil classification and epicentral distance on site response at the ground surface [10,35, ,53,54,55,56,57,58,59,60]. Therefore, there is no realistic and impressive research on the effect of horizontal distance classification on the site response of a specific area. Considering the effects of soil profiles at horizontal distances on site response, the authors selected an IRRN station with approximately uniform soil types at a distance of about 1250 m at NS and EW from the center of the station. This distance was then divided into 250, 500, 750, 1000, and 1250 meters. Similar to 1D modeling, the analysis program provides results in the form of stress-strain plots, time history plots, spectral plots, animations of convergence, and other useful results [45]. Figure 23 shows an example of modeling in the 2D program for the BAHHI station with a length of 1500 m in EW direction, i.e., five boreholes in the direction of EW. After improving the shear wave velocity, the calculated shear wave profiles are used for 2D modeling.

CHAPTER III

RESULTS and DISCUSSIONS

3.1 Discussion of results of 1D equivalent linear site response analysis

In the first step of this study, a general site response analysis is performed for 31 stations of the IRRN. At these stations, two of the major earthquakes that occurred previously were taken in consideration. In this step, all information used for the analysis is obtained from in-situ or laboratory tests during IMP. Therefore, no specific operations were performed on the information, and it is used directly. The one-dimensional analysis throughout the study is performed using the DEEPSOIL V6.1 1D computer program for site response analysis. For both earthquakes according to the previous chapter, the corresponding soil profiles were prepared and the analyzes were performed. The calculated 1D results were obtained using the time history of acceleration at the bedrock level of three IGVA stations of the Gökçeada and Kütahya earthquakes before revising the shear wave velocities. The results and the data were obtained using different related vertical arrays and directions.

In the DEEPSOIL V6.1 program, the ground motions are imported only in one direction. Therefore, we compared only the same direction in the comparison. To get an overview of the whole analysis, we first compared the PGA calculated in the direction EW and NS with the recorded ones. Figures 25-28 show the comparison between EW and NS corresponding to the recorded PGA at the ground surface by using Gökçeada

earthquake data. These figures show good agreement with the recorded data in some stations for example AKUKO and FBZMD, but for more stations the calculated 1D are lower than the recorded ones. This problem can occur for several reasons. First, the simplification of the process during the one-dimensional site response analysis causes some errors or some information is not used during the site response analysis. The second reason could be that the information is not accurate enough. The second reason may occur due to some reasons that were discussed in detail in the previous chapter. Another result considered in this diagram is that the stations that have the same rock formation as the vertical arrays and whose recorded acceleration history is used in the analysis have good agreement, e.g., AKUKO station, which has the same bedrock as ATK IGVA, and the results of the analysis using the records of ATK IGVA have very good agreement in both main directions (Figure 25,26).

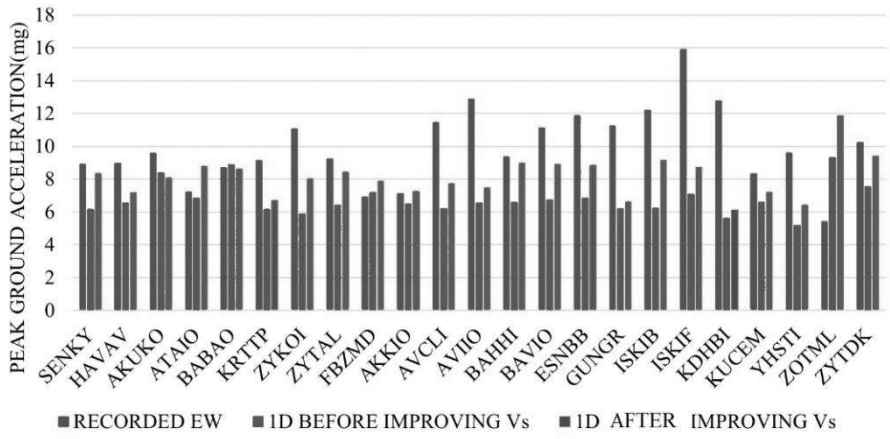


Figure 25: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in EW direction.

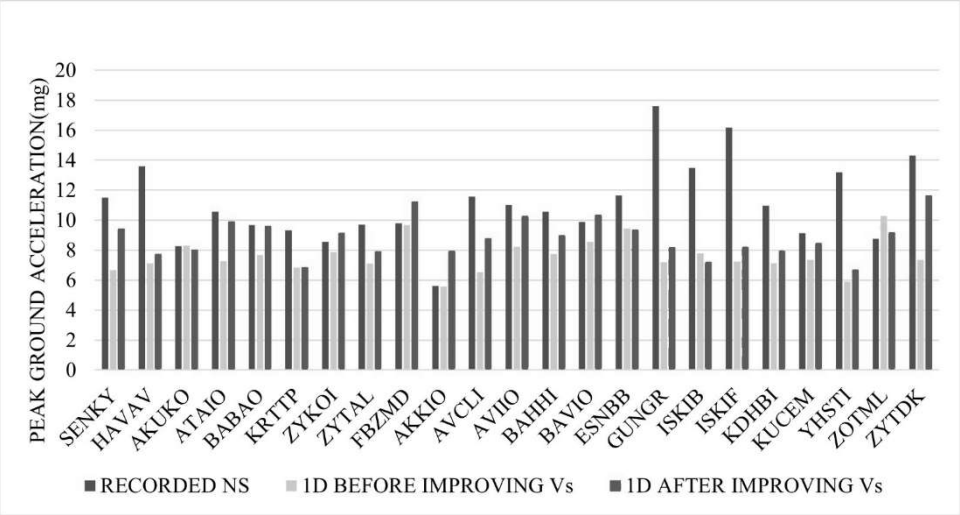


Figure 26: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in NS direction.

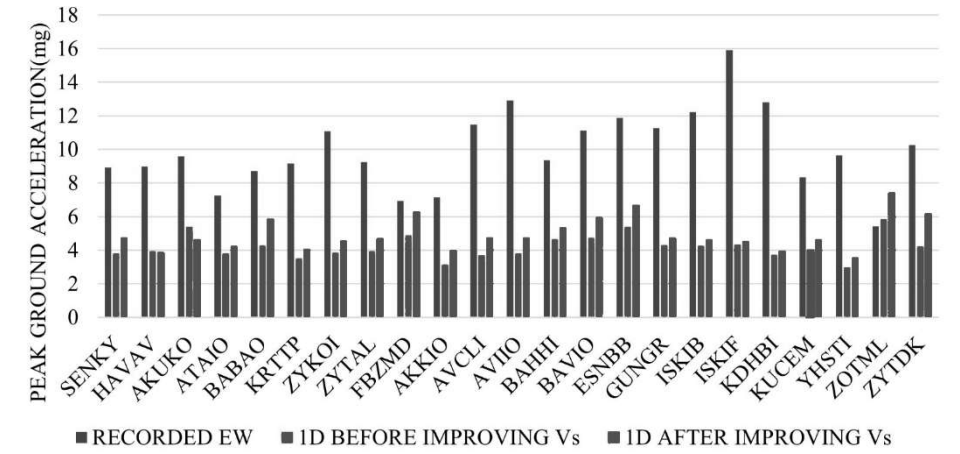


Figure 27: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in EW direction.

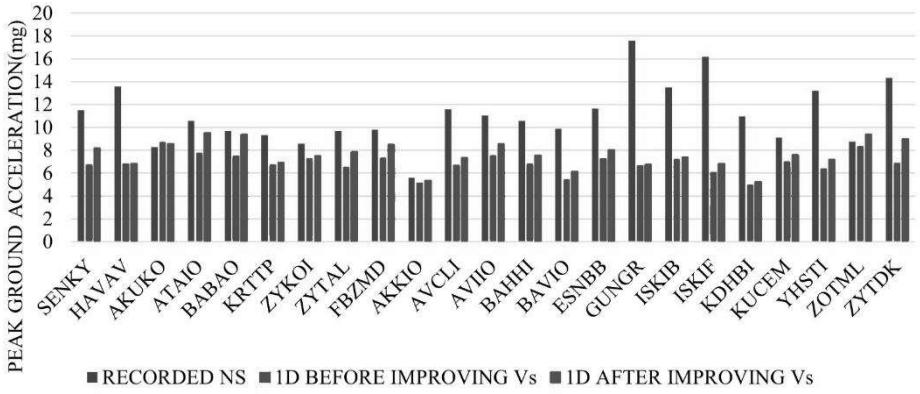


Figure 28: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Gökçeada earthquake, on the ground surface before and after shear wave velocity revision in NS direction.

In addition, one-dimensional analyses were performed for some IRRN stations that recorded the Kütahya earthquake as well as Gökçeada earthquake. As can be seen in Figures 29-32, for these stations, the differences between the values recorded at the ground surface and those calculated by 1D analyses are significant different for example in ESNBB and ISKIF stations. The views we reviewed for the first earthquake are more or less true for this earthquake as well. Generally, at some stations, good agreement was obtained by using records of a particular IGVA direction for one or both of earthquakes for example the results of ESNBB station in Figure 30 that shows the NS direction for this station is the direction with higher PGA values with comparison results of other Figures. This issue clarifies the intensity changes in the area.

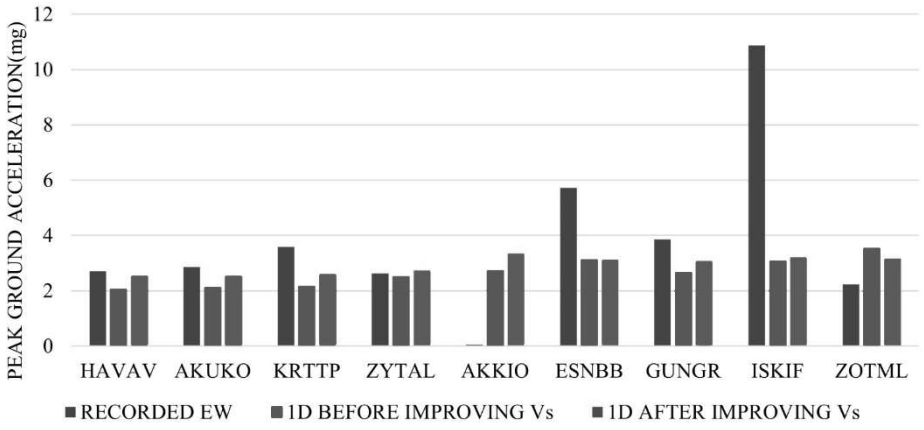


Figure 29: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in EW direction.

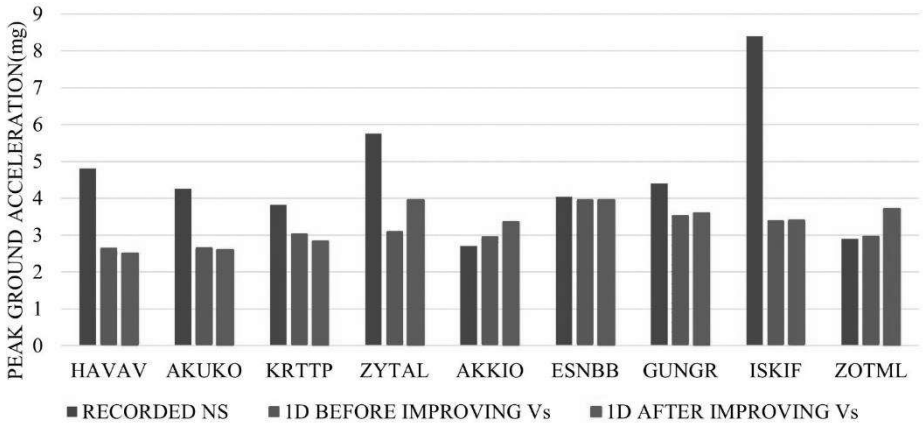


Figure 30: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at ATK IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in NS direction.

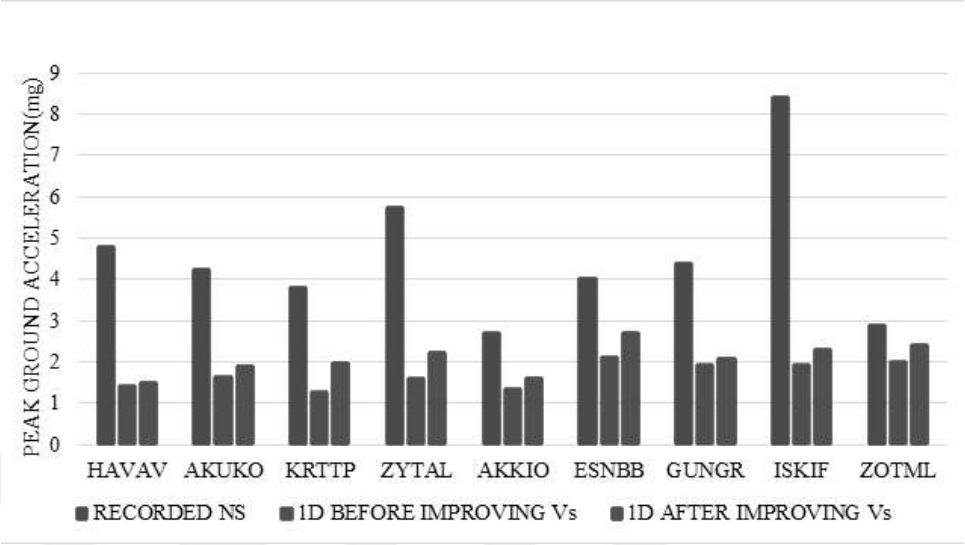


Figure 31: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in EW direction.

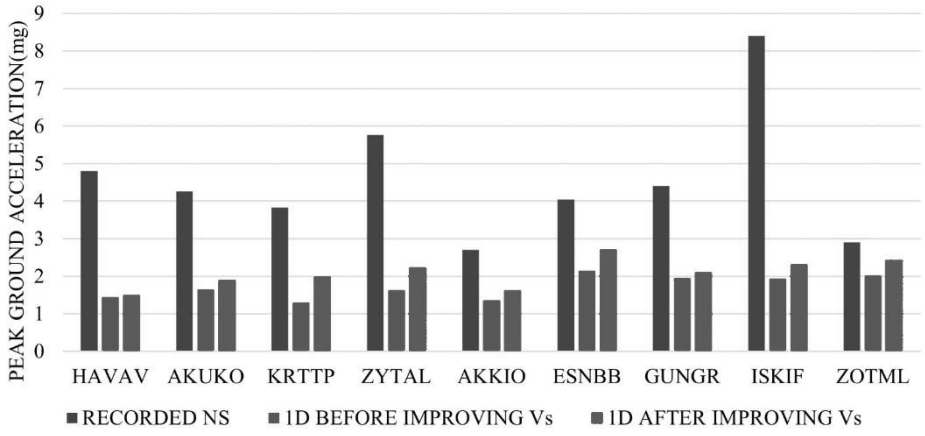


Figure 32: Comparison of PGA recorded and calculated by 1D-site response analyses using the bedrock record at FTH IGVA station for Kütahya earthquake, on the ground surface before and after shear wave velocity revision in NS direction.

Figures 39-47 show the results of spectral acceleration analysis for some stations by using the Gökçeada earthquake records. In these figures, the recorded values and the values calculated by the one-dimensional method are compared, and it becomes clear that the analysis method needs to be improved. It can be seen from Figures 33-36 that the differences between the spectral accelerations of the three rock-level IGVA stations are very large for both earthquakes. This is one of the main issues causing the significant differences in the PGA and SA values in FTH and ZYT between the recorded values and those calculated by 1D analyses. When these responses are included, one gets the feeling that it is necessary to correct or modify the ground profiles to approximate the results or to predict as accurately as possible the site response compared to the recorded ones, which is the only way to improve the results for a geotechnical engineer.

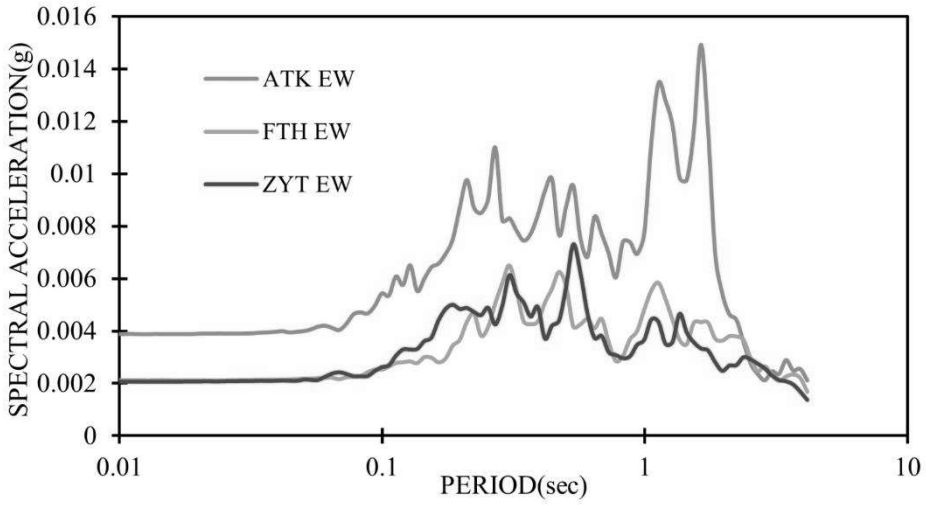


Figure 33: Comparison of recorded SA at the bedrock level during Gökçeada earthquake for three ATK, FTH and ZYT IGVA stations in EW direction.

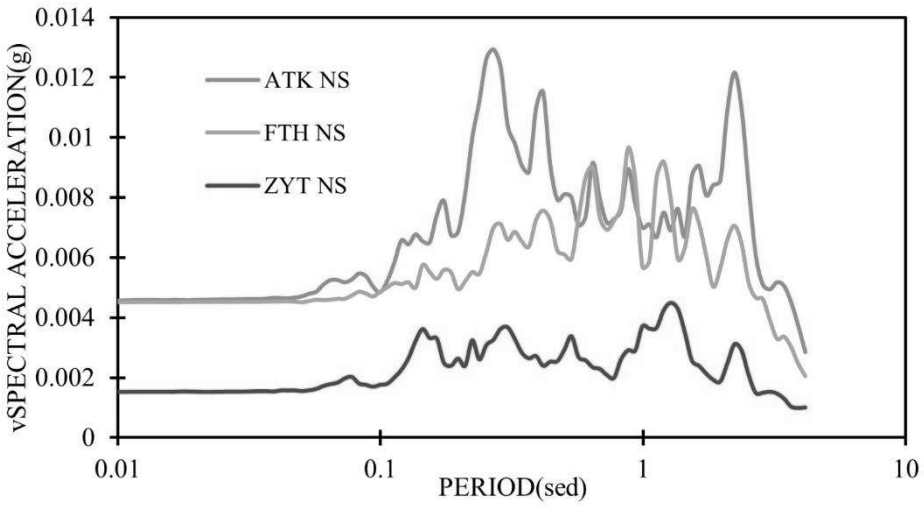


Figure 34: Comparison of recorded SA at the bedrock level during Gökçeada earthquake for three ATK, FTH and ZYT IGVA stations in NS direction.

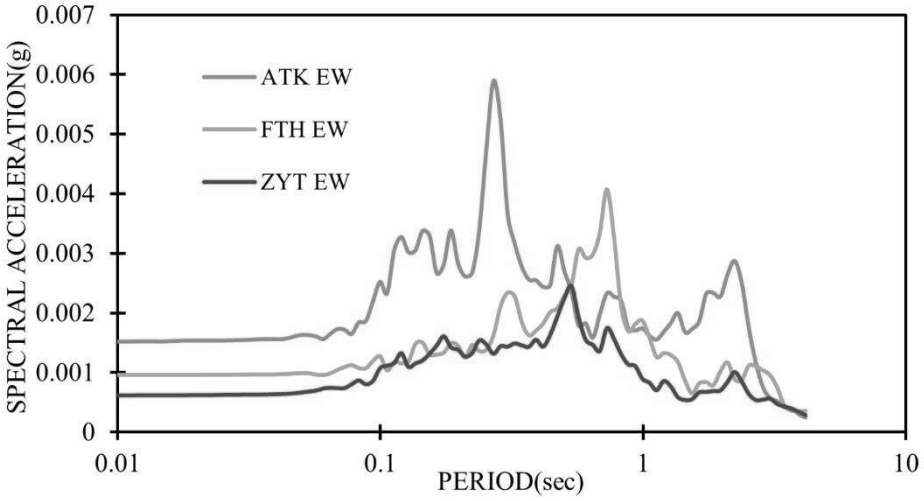


Figure 35: Comparison of recorded SA at the bedrock level during Kütahya earthquake for three ATK, FTH and ZYT IGVA stations in EW direction.

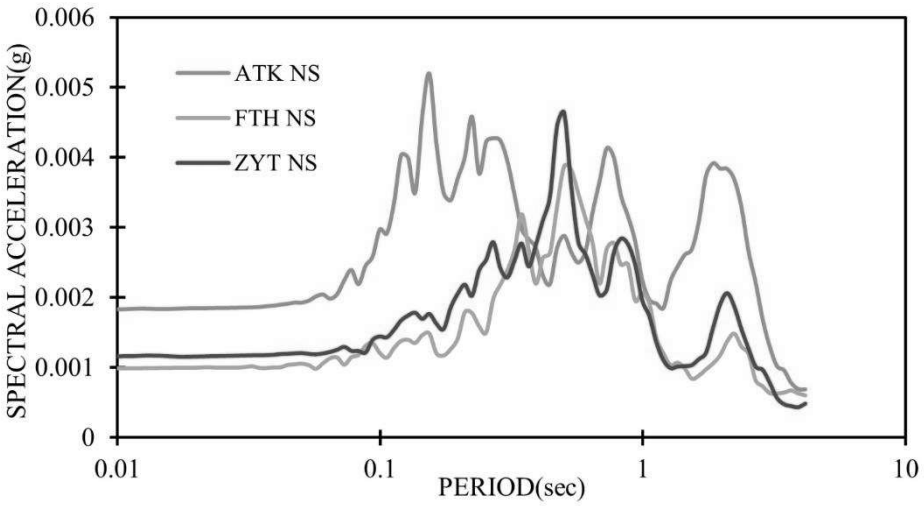


Figure 36: Comparison of recorded SA at the bedrock level during Kütahya earthquake for three ATK, FTH and ZYT IGVA stations in NS direction.

3.2Discussion of improving shear wave velocities

After completing the one-dimensional analyses for all IRRN stations that recorded the Gökçeada and Kütahya earthquakes, as a main goal of this research, we attempted to improve the shear wave velocity values. This was done following the procedure explained in the methodology chapter.

In this study, due to decreasing the difference between comparison factors, the main direction to improve the shear wave velocity is considered as east-west direction. Then, the direction NS is modelled according to the new shear wave velocity and the obtained results. Figure 37,38 illustrates the differences between the EW and the NS directions in the PGA values for IRRN stations before and after improving shear wave velocity profiles, which recorded the Gökçeada earthquake using ATK IGVA inputs.

According to Figures 37,38 for both earthquakes in the same IRRN stations, the NS direction is the predominant direction because it has more PGA values than EW. However, in some stations the predominant direction changes after applying improving method. This problem shows that the seismic studies in NS direction in the Istanbul area, especially on the European side, should be studied more carefully. However, in general, the seismic studies should be considered as multidirectional studies of the site response because the classification of the ground layers in Istanbul changes at very short intervals in each direction. Nevertheless, peak ground acceleration is not a general representative of the results of site response analyses, and the other factors such as spectral acceleration, displacement, velocity, arias intensity, duration, etc. should be studied.

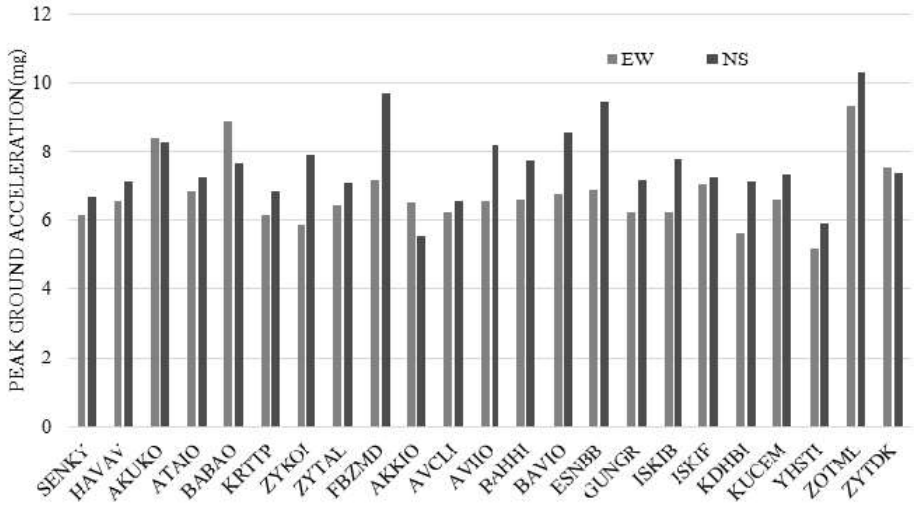


Figure 37: Comparison of PGA values of 1D site response analyses calculated using the records from the ATK IGVA station in two directions of Gökçeada earthquake on the ground surface before the Vs revision.

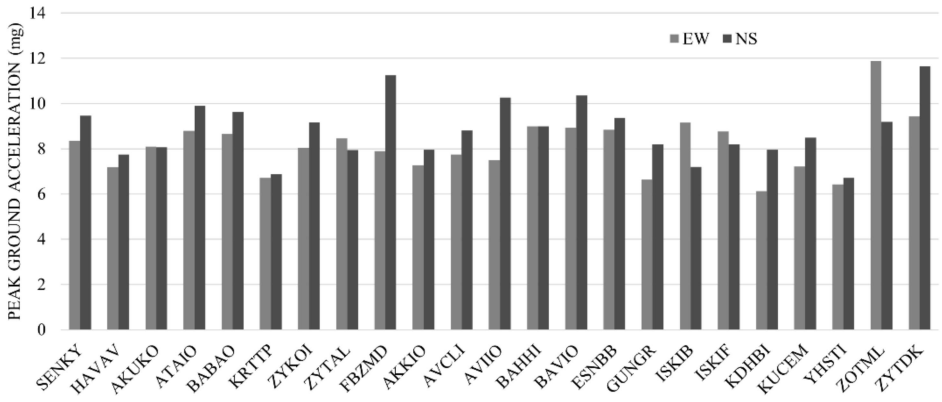


Figure 38: Comparison of PGA values of 1D site response analyses calculated using the records from the ATK IGVA station in two directions of Gökçeada earthquake on the ground surface after the Vs revision.

After applying the new improved shear wave velocities to the analysis, the results gained are compared with those obtained before the improvement and with the recorded results. In Figures 25-28, it can be observed that the peak ground acceleration values were very well improved by using the Gökçeada earthquake inputs in some stations and the differences between the recorded and calculated values decreased specially in case of using ATK IGVA station records. For example in BAHHI, ZYTDK station in EW direction by using ATK IGVA records in Figure 25.

In the sequel, we compared the spectral acceleration of the improved profiles with previous ones and with recorded profiles. Figures 39-47 show the comparison of SA analyses between the recorded, 1D analyses for the different IRRN stations based on the various IGVA records of the Gökçeada earthquake in two main directions. For the comparison between spectral acceleration values, the best results are selected. In these figures, it can be clearly seen that the spectral acceleration values have improved

especially in some stations that have the same rock formation as the geotechnical vertical array used for example BAVIO station in Figure 43 by using ATK IGVA records.

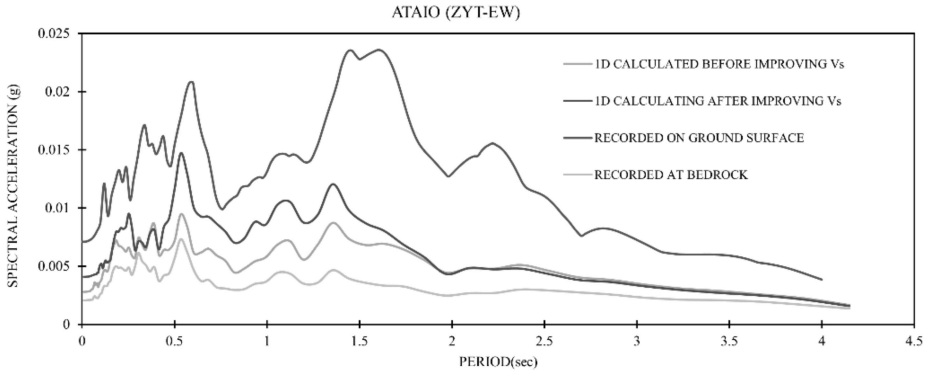


Figure 39: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the ATAIO IRRN station based on the ZYT IGVA records in EW direction of the Gökçeada earthquake.

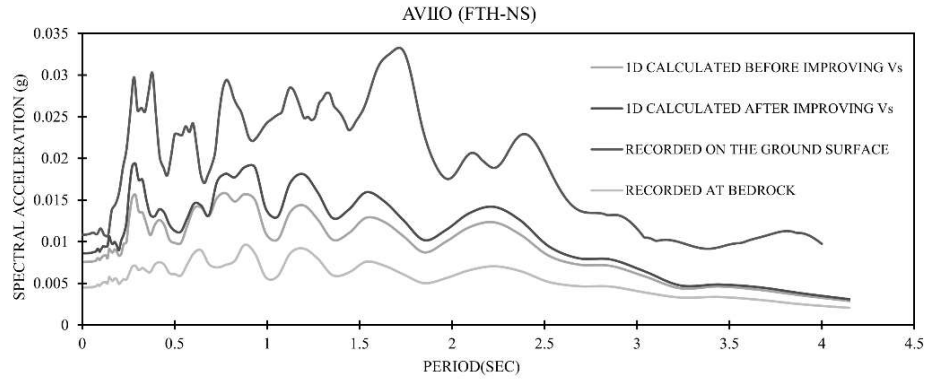


Figure 40: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the AVIIO IRRN station based on the FTH IGVA records in NS direction of the Gökçeada earthquake.

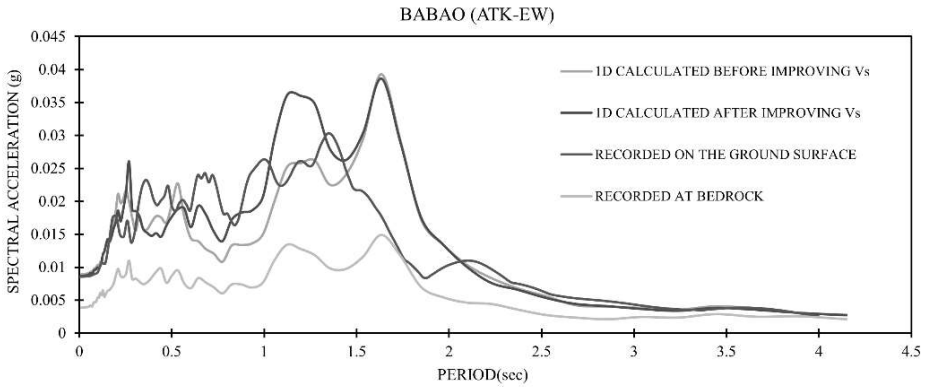


Figure 41: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BABAO IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake.

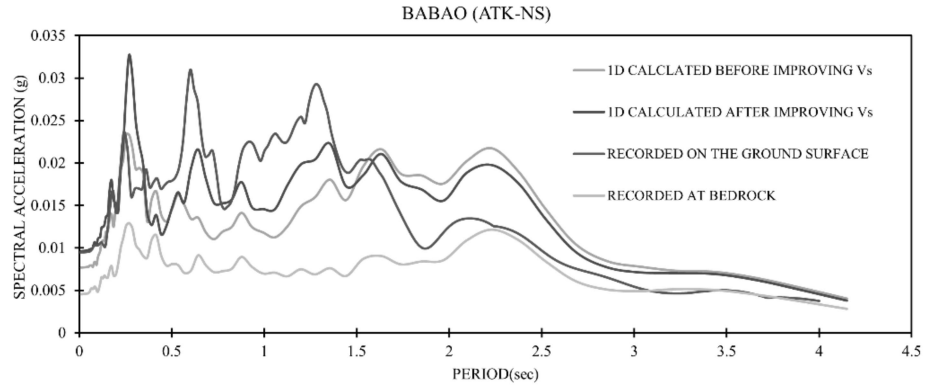


Figure 42: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BABAO IRRN station based on the ATK IGVA records in NS direction of the Gökçeada earthquake.

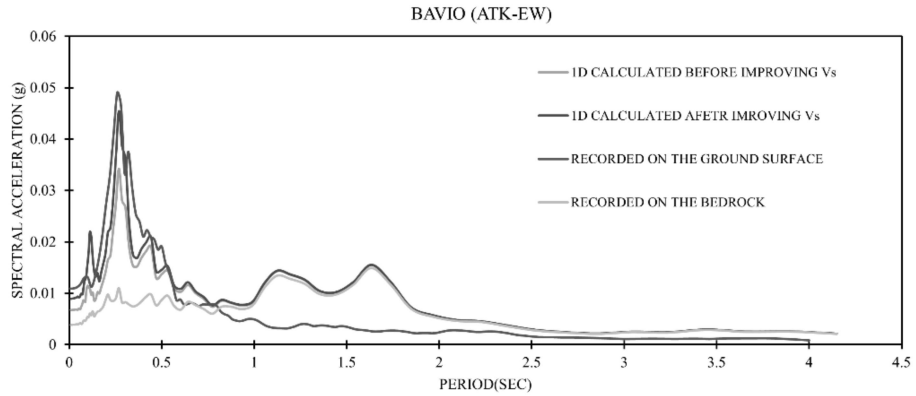


Figure 43: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BAVIO IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake.

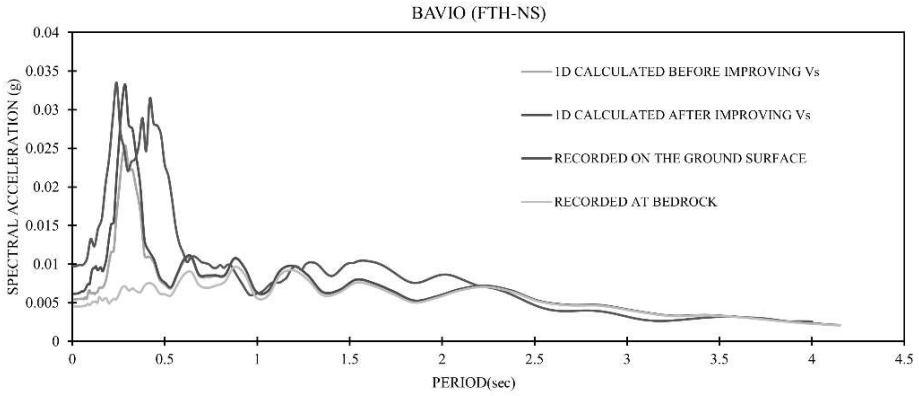


Figure 44: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the BAVIO IRRN station based on the FTH IGVA records in NS direction of the Gökçeada earthquake.

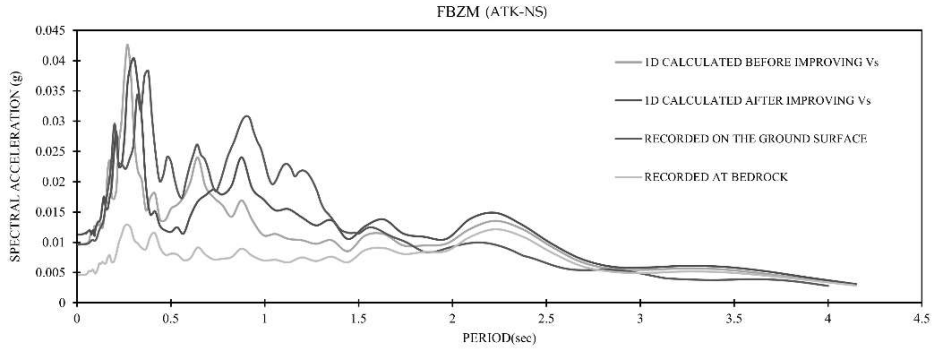


Figure 45: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the FBZMD IRRN station based on the ATK IGVA records in NS direction of the Gökçeada earthquake.

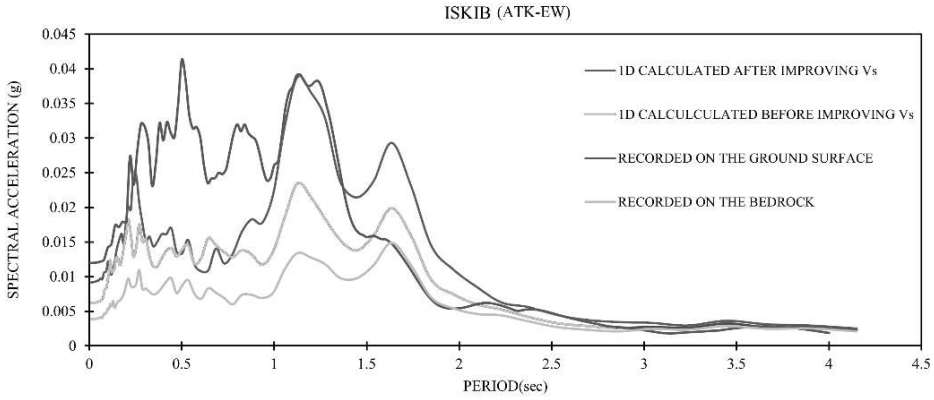


Figure 46: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the ISKIB IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake.

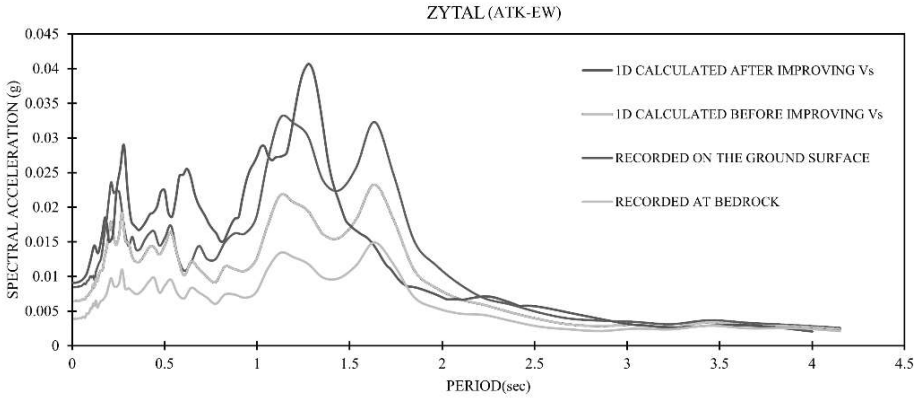


Figure 47: Comparison of SA analyses between recorded, 1D calculated before and after improving Vs analyses for the ZYTAL IRRN station based on the ATK IGVA records in EW direction of the Gökçeada earthquake.

Figures 39-47 demonstrate the improvement in the spectral acceleration values of the one-dimensional calculations after applying the revision procedure. Thus, the uncertainty of the shear wave velocity of the IRRN stations is demonstrated. In addition, these figures compare the values recorded at the ground surface and in the bedrock. These comparisons show the amplification of ground motion between soil layers in all soil profiles of the IRRN stations. This proves that the investigation of amplification in Istanbul should be considered based on the soil classification.

For some stations, the improvement in shear wave velocities is not very good. This is due to the effects of difference between recorded value at IGVA stations, the other parameters and their uncertainties or errors. The improving values of peak ground acceleration and spectral acceleration show the errors present in the shear wave velocity values. These errors may have been included in the V_S values during measurement or due to machine errors or insufficient operator measurement accuracy.

3.3 Discussion of results of 2D equivalent linear site response analysis

The next step of our study was to calculate the two-dimensional site response analyses for selected IRRN stations to investigate the differences between 1D and 2D. Following the explanations in the Methodology chapter, the two-dimensional profiles were modelled and then using the vertical and one of the east-west or north-south directions, the 2D analyses were performed. All 32 IRRN stations that recorded the Gökçeada and Kütahya earthquakes are modelled in the same way. Finally, the spectral acceleration curves obtained from the QUAKE/W 2021.3 program are compared with 1D curves. In the Figures 48-51 one can observe the differences between 2D and 1D and those recorded on the ground surface for modeling in two main directions and analyzing using inputs in different categories.

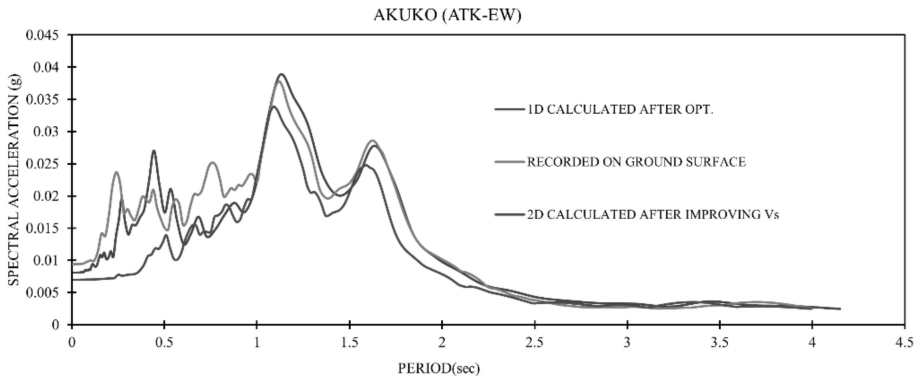


Figure 48: Comparison of SA analyses between recorded, 1D and 2D analyses for AKUKO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction.

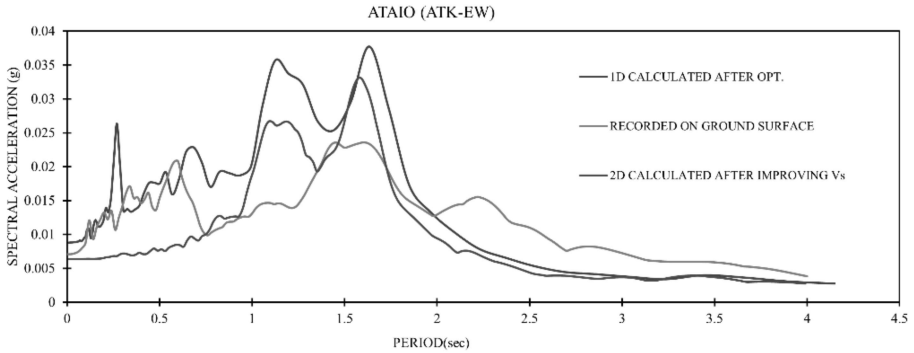


Figure 49: Comparison of SA analyses between recorded, 1D and 2D analyses for ATAIO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction.

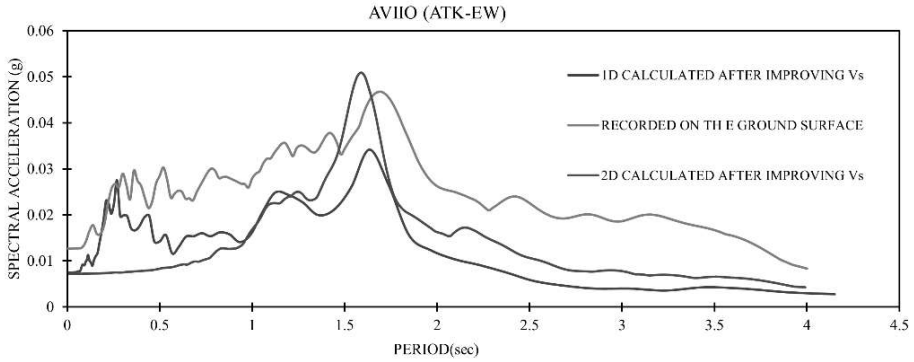


Figure 50: Comparison of SA analyses between recorded, 1D and 2D analyses for AVIIO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction.

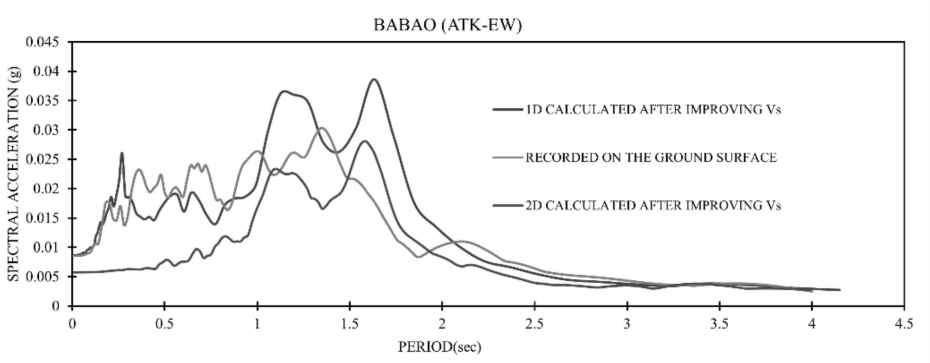


Figure 51: Comparison of SA analyses between recorded, 1D and 2D analyses for BABAO-EW IRRN using the ATK IGVA station records of the Gökçeada earthquake in EW direction.

However, for most stations, the two-dimensional analysis is lower than the 1D analysis. This problem may be due to a few reasons. First reason is the input of the vertical component of earthquakes. This leads to some resultant when entering the different directions of ground motions. Another reason can be other special factors, such as the poison ratio of the soil types or K_0 of the soil layers, which should be defined directly by the user for each soil layer. Nevertheless, in the whole soil layer, there are impurities or mixing with other soil types, all of which lead to a decrease in the accuracy of the results. In this study, we paid special attention to model the soil profiles as accurately as possible, in terms of both the dimensions and depth of the soil layers and in terms of the material properties of the soil layers. In the most figures, it can be seen that the SA in the short period (0-0.5 sec) are lower than the recorded values. This is due to the equivalent linear method, since this method at the high frequencies leads to low amplification due to the evaluation of the damping ratio based on the effective strain values. The second reason is the absence of the third dimension of the soil profiles in modeling, namely width. Also,

in the middle period (0.5-2 sec), the SA values of the 2D results tend to match better with the recorded values. However, in the long period (2-4 sec), the SA values get more or less a good agreement with the recorded values because of the convergence of the spectral acceleration graph. In overall because of the much completing of two-dimensional than one-dimensional modeling, where the 2D results are very good, the results of the two-dimensional site analyzes are satisfactory.

Figures 52-53 illustrate the time histories of acceleration at the ground surface from 1D and 2D analyses after enhancement of the AKKIO station shear wave velocity profiles in two main directions using the ATK IGVA station records based on the EW and NS directions. The maximum and minimum values of the 1D results are larger and smaller, respectively, than those of the 2D calculations. However, the pattern of the 1D and 2D results in the graphs is almost the same. As a result, the acceleration on the ground surface after 1Ds is larger than 2D, so the amplification ratio of the results of the 1D analyses is larger than 2D. This may be due to the fact that the 2D analyses use the vertical component of the recorded acceleration time histories, which causes the resultant between the horizontal and vertical components of the ground motion.

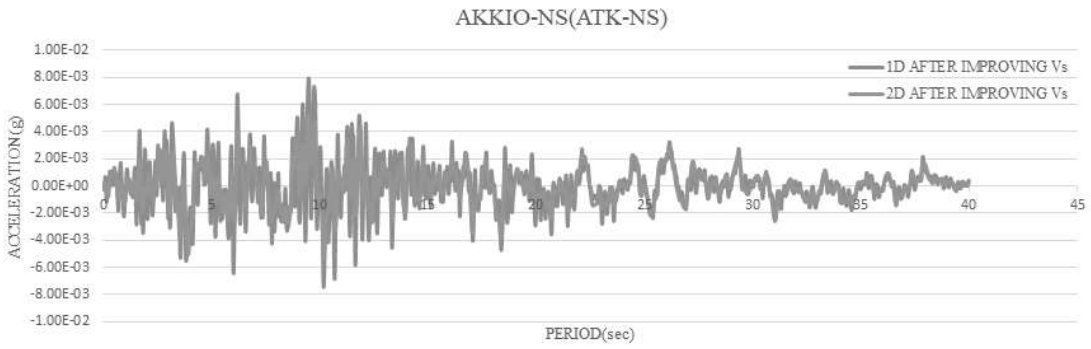


Figure 52: Comparison of 1D and 2D calculated acceleration time history of AKKIO IRRN station in NS direction using ATK-NS records.

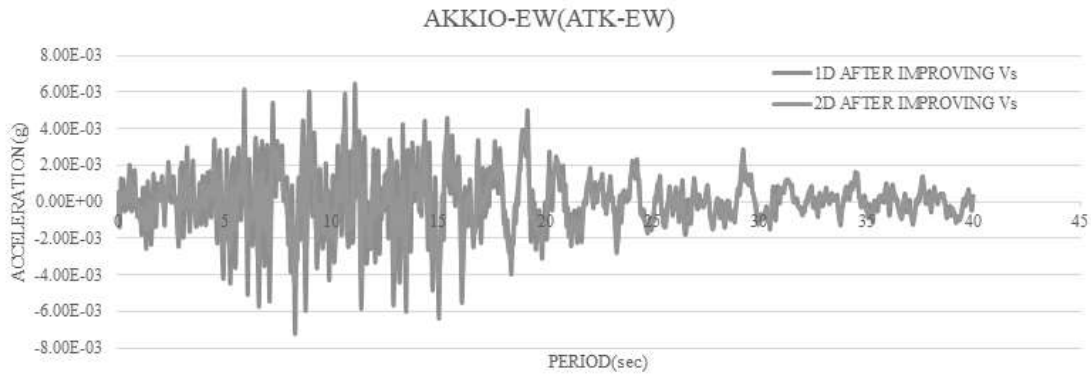


Figure 53: Comparison of 1D and 2D calculated acceleration time history of AKKIO IRRN station in EW direction using ATK-EW records.

3.4 Discussion for results of analyzing different models of horizontal distances

As the main objective of this study, we modeled the soil profiles of 8 IRRN (shown in Figure 4) stations with different symmetric lengths in the horizontal direction to investigate the effect of horizontal distance on site response on the European side of

Istanbul. According to the methodology chapter, some of the boreholes were connected to the main borehole in the main directions around the main boreholes to obtain soil profiles with different symmetric lengths such as 250,500,750,1000,1250 and 1500 meters.

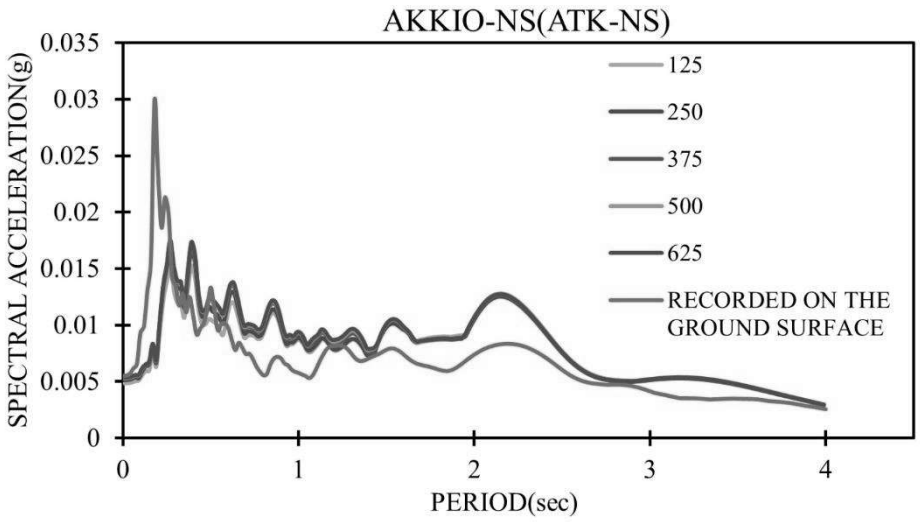


Figure 54: Comparison of 2D site response analyses between various horizontal distances from location of AKKIO-NS IRRN using the records of ATK IGVA station of Gökçeada earthquake in NS direction.

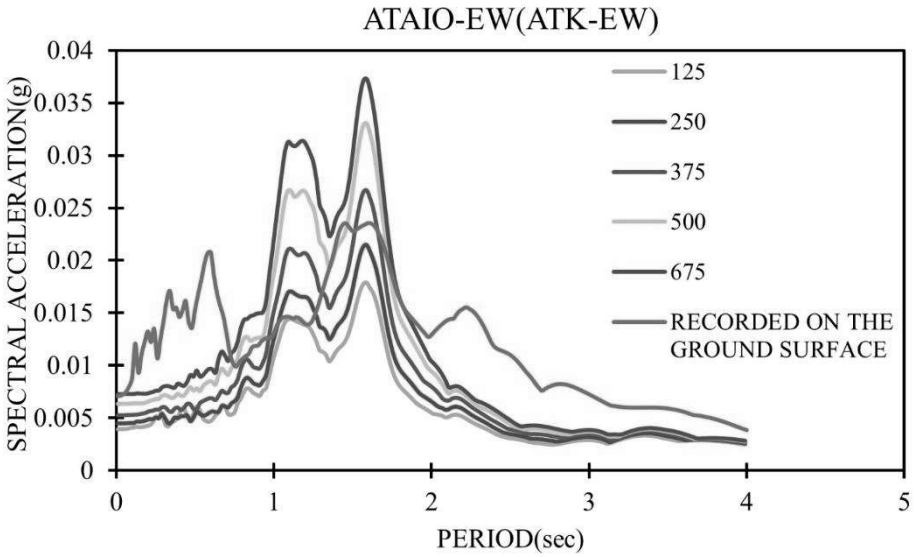


Figure 55: Comparison of 2D site response analyses between various horizontal distances from location of ATAIO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction.

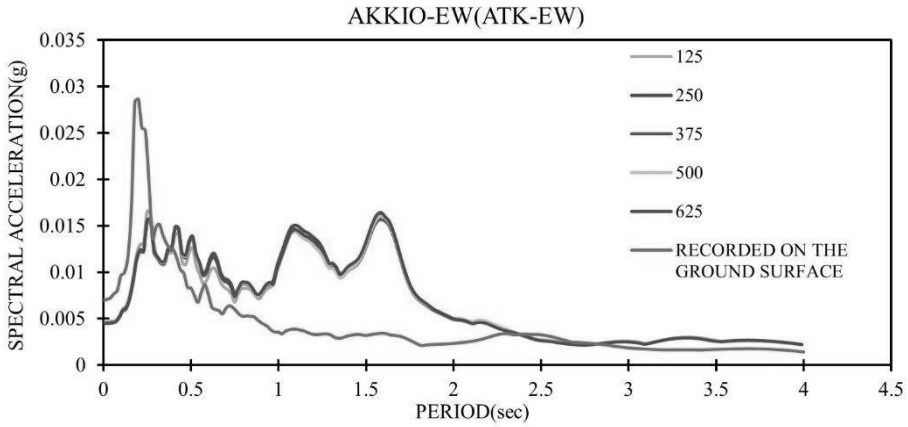


Figure 56: Comparison of 2D site response analyses between various horizontal distances from location of AKKIO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction.

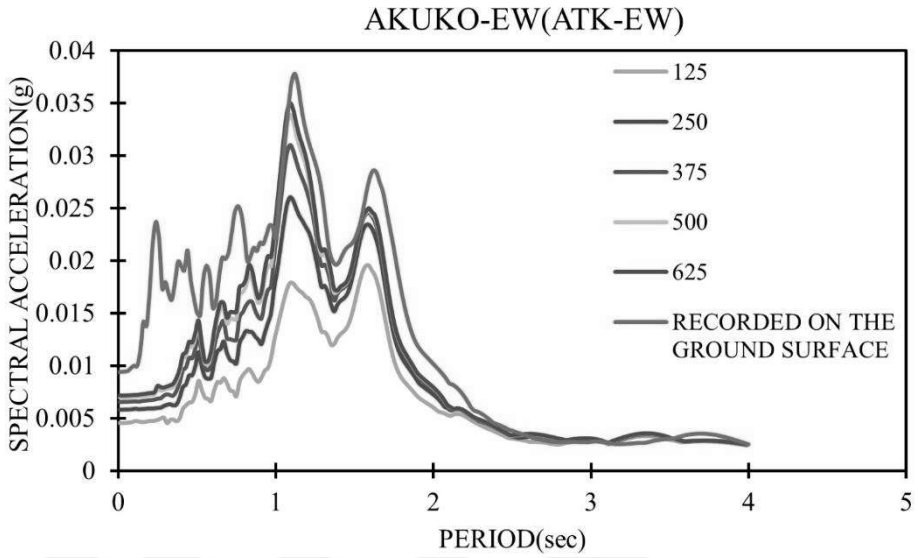


Figure 57: Comparison of 2D site response analyses between various horizontal distances from location of AKUKO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction.

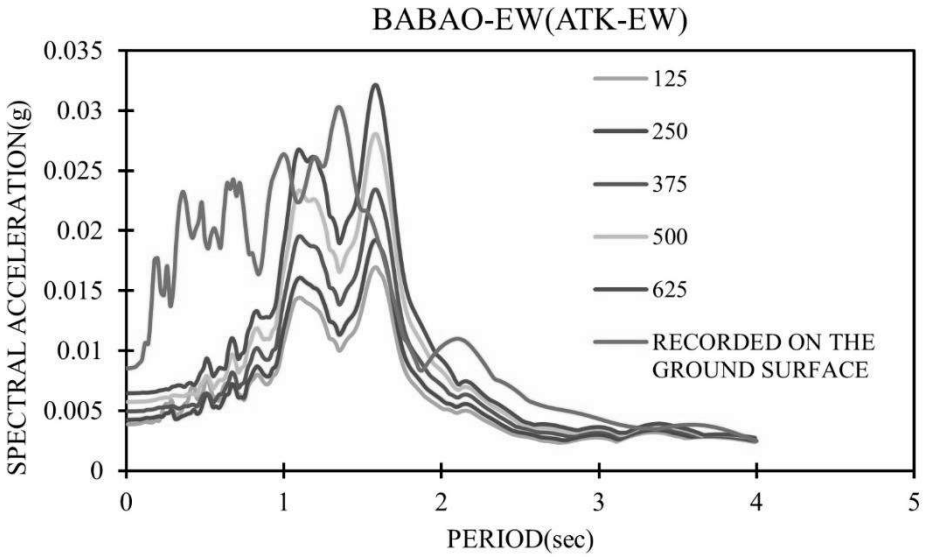


Figure 58: Comparison of 2D site response analyses between various horizontal distances from location of BABAO-EW IRRN using the records of ATK IGVA station of Gökçeada earthquake in EW direction.

In the Figures 54-58, it can be seen spectral acceleration according to the Gökçeada earthquake inputs of different distances around the main borehole in some example stations. The suitable distance to obtain the best results for matching with the recorded data in Figure 55 is 375m (totally 750 m length).In the stations of Figures 55,57,58 it can be observed a significant amplification with increasing the distances from main boreholes. This issue may be is because of the increasing the thickness of the fine soils or addition a vertical fine soil layer that should be studied the three-dimensional modelling very well in these areas. Figures 54,56 present the horizontal distance around the main borehole has not effect on the response of the site. It is due to the approximately uniform soil classifications around the main boreholes. Overall defining a constant equation for site response whole over the world is a very problematic task that this is also true for the city of Istanbul. It is difficult to assign a relationship to associate the horizontal distance with response of the area regarding to the results and the heterogeneity of soil layers in European side of Istanbul.

CHAPTER IV

CONCLUSIONS

4.1 Achievements against objectives

The main objectives of the present research are to compare one-dimensional with two-dimensional site response analysis and to improve the shear wave velocity profiles to obtain the best site response using 1D and 2D analysis. Another objective of this research is to study the effect of horizontal distance on site response and try to calculate a relationship between them. During the above process, some important results and conclusions were obtained that are consistent with the research objectives and are listed below:

1- To achieve the goal of one-dimensional equivalent linear site response analysis as a numerical method, we determined the dynamic soil behavior using the computer program DEEPSOIL. This program is a one-dimensional program that uses only the horizontal component of the earthquake recorded in one direction. The ground profiles used are part of the IRRN system, which recorded two major earthquakes, including: Gökçeada and Kütahya earthquake, at this stage the obtained results during the analyzes. The acceleration time histories used as inputs were obtained from three IGVA stations named: Ataköy, Fatih, and Zeytinburnu in three principal components (horizontal and vertical). The records at these stations were used as references in the format of PGA and SA to compare the results of the analyzes. The results of the one-dimensional analyzes showed good agreement at some stations, but at most stations the values obtained are still lower than those recorded.

Therefore, in addition to the simplicity of the 1D method, the uncertainty of the soil properties used was also considered.

2- In the second phase of the research, due to the uncertainties and errors in the measurement and calculation of shear wave velocity, an algorithm was developed to reduce the differences between the values from SA and the calculated values. After applying the scheme, the results of the 1D analyses were again compared with the references and an improvement in the SA and PGA values is undeniable. However, there are some stations where the calculated and plotted graphs do not agree very well due to other uncertainties and simplifications.

3- In the third phase, as the main objective of the research, two-dimensional equivalent linear analyses were performed for the same stations by modelling them in two horizontal directions with the 2D computer program: Quake/W performed. The revised values of shear wave velocities were used for the analyses in this part. In addition to one horizontal direction and the vertical sake recordings were also used for this purpose. The results are compared with the recorded and revised 1D results. The need to use two-dimensional site response analyses is illustrated, particularly for some stations. However, it was found that for some stations, more accurate modelling and use of more correct inputs are needed to obtain better results.

4- Finally, to understand the effects of horizontal distance on site response analysis, the two-dimensional models were progressively extended with a symmetric value in each direction. The results of the analyses were collected and compared with each other and with the recorded references. The results showed that the effects of soil heterogeneity and geotechnical features in the horizontal extent are very important for the site response results.

4.2 Gaps and future studies

1- In the literature, there are few studies that have investigated the nonlinear two-dimensional or the linear equivalent linear or nonlinear three-dimensional in the Istanbul metropolis. Therefore, these types of analyzes can be further considered using numerical computer programs to obtain high quality site response results.

2- According to literature, researchers around the world have studied some powerful numerical optimization methods for numerical values. However, these methods require more time and effort than master research, so they were not considered as a main strategy in this research. Therefore, they can be used for response parameters such as shear wave velocity, thickness, etc. to increase their accuracy as much as possible.

3- The effect of horizontal distance on the response of a particular site is not widely considered in the literature. Therefore, this problem can be considered in the future research.

BIBLIOGRAPHY

[1] Uysal, C., Maktav, D. and Small, C., 2018. Mapping urban growth and its relation to seismic hazards in Istanbul. Journal of the Indian Society of Remote Sensing, 46(8), pp.1307-1322.

[2] Beyhan, G., Keskinsezer, A. and Beyhan, S., 2017. A comparative study on Soil Properties and Applications Review with EERA and NERA in İstanbul-MARMARAY Project between Kazlıçeşme to Sirkeci. Periodicals of Engineering and Natural Sciences (PEN), 5(1).

[3] Ansal, A., Akinci, A., Cultrera, G., Erdik, M., Pessina, V., Tönük, G. and Ameri, G., 2009. Loss estimation in Istanbul based on deterministic earthquake scenarios of the Marmara Sea region (Turkey). Soil Dynamics and Earthquake Engineering, 29(4), pp.699-709.

[4] GovindaRaju, L., Ramana, G.V., HanumanthaRao, C. and Sitharam, T.G., 2004. Site-specific ground response analysis. Current Science, pp.1354-1362.

[5] Makra, K., Chávez-García, F.J., Raptakis, D. and Pitilakis, K., 2005. Parametric analysis of the seismic response of a 2D sedimentary valley: implications for code implementations of complex site effects. Soil dynamics and earthquake engineering, 25(4), pp.303-315.

[6] Parolai, S., Ansal, A., Kurtulus, A., Strollo, A., Wang, R. and Zschau, J., 2009. The Ataköy vertical array (Turkey): insights into seismic wave propagation in the shallow-most crustal layers by waveform deconvolution. Geophysical Journal International, 178(3), pp.1649-1662.

[7] Amoroso, S., Gaudiosi, I., Tallini, M., Di Giulio, G. and Milana, G., 2018. 2D site response analysis of a cultural heritage: the case study of the site of Santa Maria di Collemaggio Basilica (L'Aquila, Italy). Bulletin of Earthquake Engineering, 16(10), pp.4443-4466.

[8] Jin, Y., Kim, H., Kim, D., Lee, Y. and Kim, H., 2021. Seismic response of flat ground and slope models through 1 g shaking table tests and numerical analysis. Applied Sciences, 11(4), p.1875.

[9] Kaklamanos, J., Bradley, B.A., Thompson, E.M. and Baise, L.G., 2013. Critical parameters affecting bias and variability in site-response analyses using KiK-net downhole array data. Bulletin of the Seismological Society of America, 103(3), pp.1733-1749.

[10] Zhang, W., Taciroglu, E., Restrepo, D., Taborda, R., Kurtulus, A. and Ansal, A., 2019. An investigation of the effects of surface topography on the seismic structural demands for a region of Istanbul. In Earthquake Geotechnical

Engineering for Protection and Development of Environment and Constructions (pp. 5873-5880). CRC Press.

[11] Ansal, A., 2011. Site specific response analysis and vertical arrays. In 1st International Conference on Urban Construction in Vicinity of Active Faults, Tebriz.

[12] Tönük, G. and Ansal, A., 2007. SITE RESPONSE NONLINEARITY BASED ON CASE STUDIES.

[13] Stewart, J.P., Afshari, K. and Goulet, C.A., 2017. Non-ergodic site response in seismic hazard analysis. Earthquake Spectra, 33(4), pp.1385-1414.

[14] Tönük, G., Ansal, A., Kurtuluş, A. and Çetiner, B., 2014. Site specific response analysis for performance based design earthquake characteristics. Bulletin of Earthquake Engineering, 12(3), pp.1091-1105.

[15]] Puri, N., Jain, A., Mohanty, P. and Bhattacharya, S., 2018. Earthquake response analysis of sites in state of Haryana using DEEPSOIL software. Procedia Computer Science, 125, pp.357-366.

[16] Falcone, G., Boldini, D. and Amorosi, A., 2018. Site response analysis of an urban area: A multi-dimensional and non-linear approach. Soil Dynamics and Earthquake Engineering, 109, pp.33-45.

[17] Pehlivan, M., Rathje, E.M. and Gilbert, R.B., 2012, September. Influence of 1D and 2D spatial variability on site response analysis. In The 15th World Conference on Earthquake Engineering, September (Vol. 24).

[18] Ansal, A., Fercan, Ö., Kurtuluş, A. and Tönük, G., 2017. 2D site response analysis of the Istanbul rapid response network. Theme Lecture, PBD III, Vancouver.

[19] Ansal, A., Tönük, G., Kurtuluş, A. and Çetiner, B., 2012. Effect of spectra scaling on site specific design earthquake characteristics based on 1D site response analysis. Proceedings of 15WCEE, Lisbon, Portugal.

[20] Tönük, G., Ansal, A., Kurtuluş, A. and Çetiner, B., 2014. Site specific response analysis for performance based design earthquake characteristics. Bulletin of Earthquake Engineering, 12(3), pp.1091-1105.

[21] Volpini, C. and Douglas, J., 2019. An accessible approach for the site response analysis of quasi-horizontal layered deposits. Bulletin of Earthquake Engineering, 17(3), pp.1163-1183.

[22] Stamati, O., Klimis, N. and Lazaridis, T., 2016. Evidence of complex site effects and soil non-linearity numerically estimated by 2D vs 1D seismic response

analyses in the city of Xanthi. Soil Dynamics and Earthquake Engineering, 87, pp.101-115.

[23] Stewart, J.P., Afshari, K. and Hashash, Y.M., 2014. Guidelines for performing hazard-consistent one-dimensional ground response analysis for ground motion prediction. Peer Rep, 16, p.117.

[24] Fercan, O., Kurtulus, A., Ansal, A. and Safak, E., 2016. Site response evaluations through vertical arrays in Istanbul.

[25] Volpini, C., Douglas, J. and Nielsen, A.H., 2021. Guidance on conducting 2D linear viscoelastic site response analysis using a finite element code. Journal of Earthquake Engineering, 25(6), pp.1153-1170.

[26] Kramer, S.L., 1996. Geotechnical earthquake engineering. Pearson Education India.

[27] Kaklamanos, J., Baise, L.G., Thompson, E.M. and Dorfmann, L., 2015. Comparison of 1D linear, equivalent-linear, and nonlinear site response models at six KiK-net validation sites. Soil Dynamics and Earthquake Engineering, 69, pp.207-219.

[28] Schnabel, P.B., 1972. SHAKE: A computer program for earthquake response analysis of horizontally layered sites. EERC Report 72-12, University of California, Berkeley.

[29] Ansal, A., Tönük, G., Kurtuluş, A., Erdik, M. and Parolai, S., 2010, May. Modeling the observed site response from Istanbul strong motion network. In Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics (pp. 24-29).

[30] Amorosi, A., Boldini, D. and Di Lernia, A., 2016. Seismic ground response at Lotung: Hysteretic elasto-plastic-based 3D analyses. Soil Dynamics and Earthquake Engineering, 85, pp.44-61.

[31] Ansal, M.A., Kurtuluş, A. and Tonuk, G., 2014. Site response from Istanbul vertical arrays and strong motion network. In Proceedings of the 10th National Conference in Earthquake Engineering. Earthquake Engineering Research Institute (EERI).

[32] Setiawan, B., 2017. Site specific ground response analysis for quantifying site amplification at a regolith site. Indonesian Journal on Geoscience, 4(3), pp.159-167.

[33] Kurtuluş, A., 2011. Istanbul geotechnical downhole arrays. Bulletin of Earthquake Engineering, 9(5), pp.1443-1461.

[34] Borcherdt, R.D., 1994. Estimates of site-dependent response spectra for design (methodology and justification). Earthquake spectra, 10(4), pp.617-653.

[35] Raptakis, D.G., Manakou, M.V., Chávez-García, F.J., Makra, K.A. and Pitilakis, K.D., 2005. 3D configuration of Mygdonian basin and preliminary estimate of its site response. Soil Dynamics and Earthquake Engineering, 25(11), pp.871-887.

[36] Erdik, M., Fahjan, Y., Ozel, O., Alcik, H., Mert, A. and Gul, M., 2003. Istanbul earthquake rapid response and the early warning system. Bulletin of earthquake engineering, 1(1), pp.157-163.

[37] Mehrgini, B., Izadi, H. and Memarian, H., 2019. Shear wave velocity prediction using Elman artificial neural network. Carbonates and Evaporites, 34(4), pp.1281-1291.

[38] Anbazhagan, P. and Sitharam, T.G., 2008. Site characterization and site response studies using shear wave velocity.

[39] L'Heureux, J.S. and Long, M., 2017. Relationship between shear-wave velocity and geotechnical parameters for Norwegian clays. Journal of geotechnical and Geoenvironmental engineering, 143(6), p.04017013.

[40] Manakou, M.V., Raptakis, D.G., Chávez-García, F.J., Apostolidis, P.I. and Pitilakis, K.D., 2010. 3D soil structure of the Mygdonian basin for site response analysis. Soil Dynamics and Earthquake Engineering, 30(11), pp.1198-1211.

[41] Doğangün, A. and Livaoğlu, R., 2006. A comparative study of the design spectra defined by Eurocode 8, UBC, IBC and Turkish Earthquake Code on R/C sample buildings. Journal of Seismology, 10(3), pp.335-351.

[42] Ansal, A., Tönük, G. and Kurtuluş, A., 2018, June. Implications of site specific response analysis. In European Conference on Earthquake Engineering Thessaloniki, Greece (pp. 51-68). Springer, Cham.

[43] Puzrin, A.M. and Shiran, A., 2000. Effects of the constitutive relationship on seismic response of soils. Part I. Constitutive modeling of cyclic behavior of soils. Soil Dynamics and Earthquake Engineering, 19(5), pp.305-318.

[44] Elçin, G.Ö.K., 2018. The Gokceada Island (Northwest of Turkey) Earthquake of Mw 6.5 on 24 May 2014: Strong-Motion Examinations. Dokuz Eylül Üniversitesi Mühendislik Fakültesi Fen ve Mühendislik Dergisi, 20(60), pp.982-999.

[45] Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Groholski, D.R., Phillips, C.A. and Park, D., 2016. DEEPSOIL 6.1, user manual. Urbana, IL, Board of Trustees of University of Illinois at Urbana-Champaign.

[46] Krahn, J., 2004. Dynamic modeling with QUAKE/W: an engineering methodology. Calgary: GEO-SLOPE.

[47] IMM, I.M.M., 2007. Production of Microzonatıon Report and Maps–European Side (South).

[48] Dogangün, A., Ural, A., Sezen, H., Güney, Y. and Firat, F.K., 2013. The 2011 earthquake in Simav, Turkey and seismic damage to reinforced concrete buildings. Buildings, 3(1), p.173.

[49] Gholami, A., Seyedali, S.M. and Ansari, H.R., 2020. Estimation of shear wave velocity from post-stack seismic data through committee machine with cuckoo search optimized intelligence models. Journal of Petroleum Science and Engineering, 189, p.106939.

[50] Alielahi, H. and Adampira, M., 2018. Evaluation of 2D seismic site response due to hill-cavity interaction using boundary element technique. Journal of Earthquake Engineering, 22(6), pp.1137-1167.

[51] Upom, M.R.A., Alel, M.N.A., Ab Kadir, M.A. and Yuzir, A., 2019, May. Prediction of shear wave velocity in underground layers using Particle Swarm Optimization. In IOP Conference Series: Materials Science and Engineering (Vol. 527, No. 1, p. 012012). IOP Publishing.

[52] Wang, J., Cao, J. and Yuan, S., 2020. Shear wave velocity prediction based on adaptive particle swarm optimization optimized recurrent neural network. Journal of Petroleum Science and Engineering, 194, p.107466.

[53] Woolery, E.W. and Street, R., 2002. 3D near-surface soil response from H/V ambient-noise ratios. Soil Dynamics and Earthquake Engineering, 22(9-12), pp.865-876.

[54] Castellaro, S. and Mulargia, F., 2014. Simplified seismic soil classification: The Vfz matrix. Bulletin of earthquake engineering, 12(2), pp.735-754.

[55] Pitilakis, K., Gazepis, C. and Anastasiadis, A., 2004, August. Design response spectra and soil classification for seismic code provisions. In Proceedings of the 13th world conference on earthquake engineering, Vancouver, BC, Canada.

[56] Luzi, L., Puglia, R., Pacor, F., Gallipoli, M.R., Bindi, D. and Mucciarelli, M., 2011. Proposal for a soil classification based on parameters alternative or complementary to Vs, 30. Bulletin of Earthquake Engineering, 9(6), pp.1877-1898.

[57] García-Jerez, A., Piña-Flores, J., Sánchez-Sesma, F.J., Luzón, F. and Perton, M., 2016. A computer code for forward calculation and inversion of the H/V spectral ratio under the diffuse field assumption. Computers & geosciences, 97, pp.67-78.

[58] Sohrabi-Bidar, A., Kamalian, M. and Jafari, M.K., 2009. Time-domain BEM for three-dimensional site response analysis of topographic structures. International journal for numerical methods in engineering, 79(12), pp.1467-1492.

[59] Forcellini, D., Tanganelli, M. and Viti, S., 2018. Response site analyses of 3D homogeneous soil models. Emerging Science Journal, 2(5), pp.238-250.

[60] Gupta, R.K., Agrawal, M., Pal, S.K. and Das, M.K., 2021. Seismic site characterization and site response study of Nirsa (India). Natural Hazards, 108(2), pp.2033-2057.