

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

SENSITIVITY OF DYNAMIC SITE RESPONSE
ANALYSES TO EXCESS PORE PRESSURE
GENERATION PARAMETERS

by

Doğa ALTUNEVLEK

June, 2022

İZMİR

SENSITIVITY OF DYNAMIC SITE RESPONSE ANALYSES TO EXCESS PORE PRESSURE GENERATION PARAMETERS

**A Thesis Submitted to the Graduate School of Natural and Applied Sciences of
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by

Doğa ALTUNEVLEK

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M.SC THESIS EXAMINATION RESULT FORM

We have read the thesis entitled “**SENSITIVITY OF DYNAMIC SITE RESPONSE ANALYSES TO EXCESS PORE PRESSURE GENERATION PARAMETERS**” completed by **DOĞA ALTUNEVLEK** under supervision of **PROF. DR. GÜRKAN ÖZDEN** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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SENSITIVITY OF DYNAMIC SITE RESPONSE ANALYSES TO EXCESS PORE PRESSURE GENERATION PARAMETERS

ABSTRACT

Geotechnical earthquake engineering studies the problems arising from the behavior of soils under earthquake loading. The biggest uncertainties in these problems are the selection of the earthquake record and the determination of the soil parameters. In this study, the behavior of the soils of the Izmir region under earthquake loading has been examined using 1-dimensional ground response analyses. Acceleration records recorded at the stations during the 30 October 2020 Samos earthquake were used in the analyses. In addition to outcrop records, ground surface records are also available at the stations. Accordingly, the ground surface station was defined as the input motions, and the rock record in the region was obtained by 1-dimensional analysis. The obtained rock record was given to another station site whose soil profile is known, and the results were compared with the recorded motions. When the exposure records at different stations were applied as input in a certain area, results that were not close to the ground surface were obtained. MKZ and GQ/H models were used in nonlinear analyses. The effects of these models on the results were demonstrated by comparing the response spectra obtained on the ground surface. Overall, the MKZ model has been shown to damped the motion reaching the ground surface. In the GQ/H model, on the other hand, high spectral accelerations were obtained in the high period region of the response spectrum. In addition, the analyses were repeated by increasing the shear wave velocity by 20%. Accordingly, the closest result to the acceleration recorded in the station was obtained when the MKZ model and 20% increased shear wave velocity profile were used. These results revealed the effect of the errors caused by the inability to model the soil behavior on the results obtained.

Keywords: Soil, response, nonlinear, earthquake, one dimensional analysis

SENSITIVITY OF DYNAMIC SITE RESPONSE ANALYSES TO EXCESS PORE PRESSURE GENERATION PARAMETERS

ÖZ

Geoteknik deprem Mühendisliği, zeminlerin deprem yüklemesi altındaki davranışından kaynaklı problemleri incelemektedir. Bu problemlerdeki en büyük belirsizlikler deprem kaydının seçilmesi ve zemin parametrelerinin belirlenmesidir. Bu çalışmada, İzmir bölgesi zeminlerinin deprem yüklemesi altındaki davranışı 1 boyutlu yer tepki analizleri ile irdelenmiştir. Analizlerde 30 Ekim 2020 Samos depreminde istasyonlarda kaydedilmiş ivme kayıtları kullanılmıştır. İstasyonlarda mostra kayıtlarının yanı sıra, zemin yüzeyi kayıtları da mevcuttur. Buna göre, zemin yüzeyi istasyonu girdi hareketi olarak tanımlanarak 1 boyutlu analizlerle bölgedeki kaya kaydı elde edilmeye çalışılmıştır. Elde edilen bu kaya kaydı zemin profili bilinen başka bir istasyona verilerek, kaydedilen kayıt ile karşılaştırmalar yapılmıştır. Farklı istasyonlardaki mostra kayıtları belirli bir sahada girdi olarak uygulandığında zemin yüzeyinde yakın olmayan sonuçlar alınmıştır. Nonlineer analizlerde MKZ ve GQ/H modelleri kullanılmıştır. Bu modellerin sonuçlara etkisi zemin yüzeyinde elde edilen tepki spektrumları karşılaştırılarak ortaya konmuştur. Genel olarak, MKZ modelinin zemin yüzeyine ulaşan hareketi sönümlediği gösterilmiştir. GQ/H modelinde ise tepki spektrumunun yüksek periyot bölgesinde yüksek spektral ivmeler elde edilmiştir. Ayrıca, analizler kayma dalgası hızı %20 arttırılarak sonuçlar elde edilmiştir. Buna göre İstasyonda kaydedilen ivme kaydına en yakın sonuç MKZ modeli ile %20 arttırılmış kayma dalgası hızı profili kullanıldığında elde edilmiştir. Bu sonuçlar, zemin davranışının tam olarak modellenememesinden kaynaklanan hataların, elde edilen sonuçlara etkisini ortaya koymuştur.

Anahtar kelimeler: Zemin, tepki, nonlineer, deprem, bir boyutlu analiz

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CHAPTER ONE

INTRODUCTION

Geotechnical earthquake engineering deals with soil related problems under earthquake loading in seismically active regions. Earthquake waves due to the fault rupture propagate along a path, and finally reach the ground surface. Seismology is the branch mainly concerns about the source mechanism of the rupture and path of an earthquake. Strong ground motion stations record the earthquake data on rock or soil surfaces. On the one hand, rock outcropping records are very useful for seismologists to define the characteristics of the earthquake. On the other hand, the stations founded on a soil surface and records the motions are important for geotechnical earthquake engineering in terms of the soil effect. The different motions obtained on the rock and soil surfaces demonstrates the site amplification concept. Structures on the soil and rock subject different earthquake loadings, and design spectrums were developed to be used in the codes to design the structures to withstand the earthquake loading. Apart from the unknowns due to the seismology, there are uncertainties in the soil-structure interaction problem, due to the soil behavior under earthquake loading. Historically, soil effect on the earthquake motion was investigated either by comparing the surface motions recorded on the rock and soil surface. The computer techniques were developed by researchers and one-dimensional (1D) site response analyses techniques were implemented to approach the problem more elaborately. The most important issue in the soil behavior under earthquake loading is the estimation of soil modulus and damping properties. The maximum shear modulus (or shear wave velocity) and the minimum damping ratio are the properties for linear analysis at the very small strain range. However, stress-strain curve of soils is a hysteresis loops under cyclic loading, in which the initial modulus reduces and damping increases with the increasing strain. 1D analysis were performed linearly by an iterative procedure that is called equivalent linear method to take the soil nonlinearity into consideration. Modulus reduction and damping ratio curves suggested by researchers can be used to define the soil behavior in the equivalent linear method. More

comprehensive analysis is the fully nonlinear method in time-domain. True nonlinear behavior can be considered in the fully nonlinear methods. The modulus reduction behavior can be defined in the fully nonlinear analysis and the damping consisting with the experimental results is controlled by the unloading/reloading rule.

In this study, the effect of soil behavior on the site response analyses was investigated. The problematic sites of İzmir City in Turkey was selected for this purpose. The equivalent linear and fully nonlinear methods were employed and the 1D ground response analysis results were compared for a given site and earthquake. The maximum shear modulus of soils was estimated by the shear wave velocity measurements and using the basic soil mechanics laboratory tests. Considering the probable errors to be made in the prediction, the analyses were repeated with the 20% increased shear wave velocities. Modulus reduction and damping curves given in the literature were studied thoroughly, and the most proper ones were used in the equivalent linear analyses. Two well-known nonlinear soil models were implemented in the fully nonlinear analyses: Modified Kondner&Zelasko (MKZ) and Generalized Quadratic/Hyperbolic model (GQ/H). Besides, the deconvolution analysis procedure was adopted for the motions recorded on soil surface, to obtain the top of rock (within) and rock outcropping motions. The effect of the input motion on the site response analyses was examined by comparing the results for within and outcrop motions.

The chapters of the thesis is organized to cover the topics described above in a progressive fashion. In chapter Two, the background of the problem was introduced. Chapter Three describes the seismicity and faults in İzmir City. The difference in the equivalent linear and nonlinear analysis methods and a case study in Mavişehir region is given in Chapter Four. Seismic response of the alluvial and deep soft soil deposits is investigated by a case study in Bayraklı region in Chapter Five. Chapter Six presents the results for sensitivity of dynamic site response analyses to excess pore pressure generation parameters. Finally, conclusion and recommendations for future studies are presented in Chapter 7.

CHAPTER TWO

BACKGROUND AND LITERATURE REVIEW

During an earthquake, local site conditions affect all components of strong ground motions, and this effect can be understood by case studies on both free field conditions and structure behavior. Historical resources on the relationship between earthquake damage and local site conditions go back 200 years. The first large-scale findings to relate ground shaking damage to ground conditions were obtained with the 7.9 magnitude San Francisco earthquake of April 18, 1906. When the 1906 San Francisco earthquake occurred, scientists in California began collecting observations about the earthquake and its effects. The report on the earthquake in 1908 showed that the most damage was in artificially filled ground and soft soil areas. However, it is known that these reports did not have a major impact on the redevelopment of the Marina area in San Francisco and the areas south of Market Street after the earthquake, and the same areas were heavily damaged in the Loma Prieta earthquake of 1989.

It is clear that the ground surface motions are affected by local site conditions. Local site conditions strongly influence all of the important characteristics of strong ground motion, including amplitude, frequency content, and duration. In fact, these local geological conditions, can increase or decrease seismic ground motion and can also make a difference in shaking intensity (Bard et al., 1988; Bard and Bouchon, 1985).

Many modern earthquake codes include the need to consider local soil conditions in the creation of elastic design acceleration spectra to be used in earthquake resistant building design. Studies (Kramer, 1996; Iida, 1998; Hayashi et al., 1999) show that strong ground motion parameters—amplitude, frequency, duration—are affected by local site conditions. It has been determined that the boundaries of this effect change depending on the material and geometrical properties of the soil layers and the characteristics of the earthquake wave, based on theoretical, instrumental and observational.

As the seismic waves propagate from the earthquake source, they encounter heterogeneities and discontinuities through the soil domain. Earthquake waves travel faster in hard rocks than soft rocks and sediments. The waves traveling at high speed along the bedrock, break and slow down dramatically as they enter soft ground due to the sharp stiffness contrast. The waves slow down and their amplitudes get larger as seismic energy builds up, while the waves move from deeper, harder rocks to shallower soft ground. This amplitude effect also depends on the topography, geometry, soil properties and input motion characteristics of the site. The more weathered rock or the softer the ground beneath an area, the greater the wave amplitude and soft/weak layers increase the ground movement.

Findings from theoretical ground response analyzes and measured surface motions confirm soil amplification in soft soils and deep alluvial soils. It should be noted, however, that in relation to the direct amplification effects of soft soil materials, the geometry of soft sediments can further disrupt ground motion in soft rock sites. Seismic waves entering valleys with soft ground can be reflected and refracted repeatedly from the valley walls, trapping seismic energy in the valley. This can result in both higher amplitudes and longer shaking times. Alluvial valleys that carry seismic energy, limit the body waves (P and S) and cause extreme Rayleigh and Love surface waves. Since such effects are geometric in nature, they depend on the characteristics of the incident wave and it can be very difficult to predict the approach direction. Because of their size, these deep geological structures can affect the shaking over a wide area.

In order to accurately predict how the seismic energy is changed and how the engineering structures will behave during the strong ground shaking, it is necessary to examine the local site conditions, the frequency content of the surface motions, and therefore the response spectra they produce on the site surface. It is certain that it would be wrong to evaluate the effects of destructive earthquakes on soft and deep alluvial soils on structures only on the free field situation. As a result of soil-structure interaction, strains occurring under the foundation base and in free field conditions will be different. The

main factor in this interaction is the earthquake intensity. Different responses will occur in different earthquakes under the same foundation for the same structure and these responses will be transmitted to the structure and different inertia effects will occur in the structure. For this reason, defining it as earthquake-soil structure interaction is seen as the most appropriate description to evaluate the building behavior.

As a result of the earthquake, seismic waves trying to reach the surface are trapped between weak, loose and soft soil layers, causing the development of non-linear behavior in the soil. This behavior can cause an increase in the period, large strains and permanent deformations in the soil. In fact, in addition to the period lengthening due to the flexible foundation, the nonlinear behavior that develops during an earthquake in loose and soft soils with low shear wave velocities can also increase the dominant period of a soil profile. When these period extensions catch up with the building periods, the resonance effects become pronounced.

The consequences of local site effects are clearly visible in the response spectra. However, at this stage, the conceptual difference between the response spectrum and the design spectrum should be kept in mind. The response spectrum is obtained by calculating the maximum response values of single degree-of-freedom systems with different natural vibration periods under the influence of strong ground motion, for a certain damping ratio. The design spectra, on the other hand, are obtained by fitting a smoothed envelope curve to the response spectra calculated from different ground motion acceleration records in order to take into account the maximum earthquake effects that may occur by taking into account the seismic characteristics and soil conditions of the region.

Moreover, in cases where more special design is required, design spectra can be obtained from the seismic hazard analysis result. However, as a result of the process of idealizing and smoothing the response spectra specific to a certain region in the design spectra, the amplification and filtering effects seen in the soil layers are neutralized in certain periods. It is seen that the spectra are below the design spectra in some cases.

Considering the effects of soil-structure interaction instead of free field conditions, it causes the periods to lengthen in the design spectrum.

Previous research on the effect of soil properties on damage distribution in Adapazarı has shown that in the southern parts of the city, there are three to four storey buildings on the stiff soils, unlike those built on shallow, hard residual profiles on bedrock. These building probably suffered significantly more damage due to soil amplification (Bakır et al., 2002, 2005).

It is claimed that the soft soils form a kind of base isolation. As a result, buildings built on such soils suffered less structural damage, but overturning, over-settlement and insufficient bearing capacity were common. Trifunac and Todorovska (1998) also claimed that soft soils around the earthquake zone act as passive insulators. The fact that earthquake damage is high in some areas and relatively low in others indicates that local soil effects are significant.

After an earthquake occurs, the seismic waves emanating from the earthquake focus are refracted through the environments they travel and reach the earth by being reflected. As seismic waves travel, their amplitudes change due to impedance differences between the layers they pass through. Accordingly, an increase is observed in the intensity, period and duration of the earthquake movement. Alluvial basins with young sediments trap body waves and cause surface waves to form in the alluvium (Kramer, 1996).

Structural response of a building under the design earthquake can be predicted by the free field ground surface acceleration records obtained from the site response analyses. In this study site response analyses are performed using the fully nonlinear methods. In the analyses dynamic soil properties and the soil models representing dynamic soil behaviour are required. In this chapter the past studies about the dynamic properties of soils, soil models and soil structure interaction are shared.

2.1 Dynamic Soil Properties

There are two main parameters that affect the behavior of the soil under cyclicloading. These are the shear modulus and damping ratio. Both parameters change depending on the strain level. The shear modulus reduction with strain can historically be represented by either a stress-strain curve or a modulus reduction curve, as shown in figure 1. (Kramer, 1996)

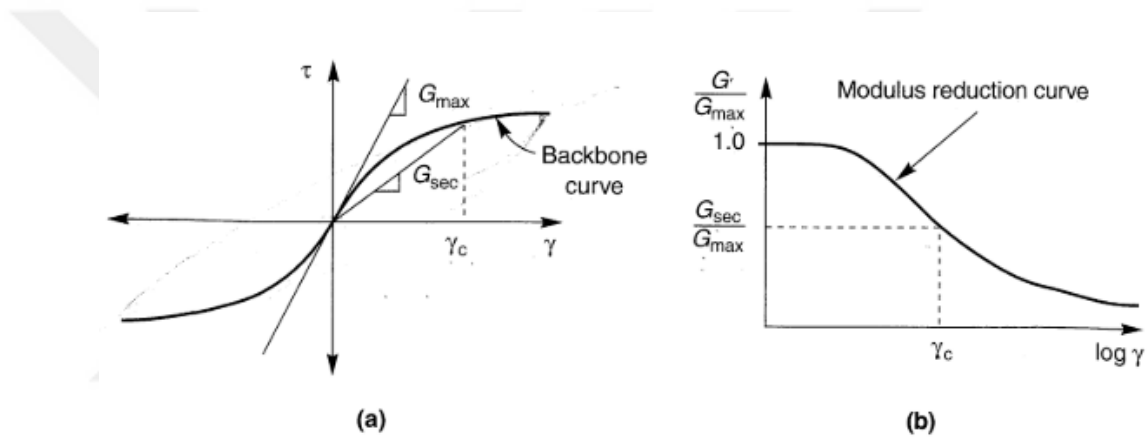


Figure 2.1 The soil behavior a) shear stress-strain and b) modulus reduction curve

Variation of the maximum shear modulus depending on soil properties, loading conditions and geological factors has been given by Vucetic and Dobry (1987). According to the study, the shear modulus of the soil increases with the increase in the effective stress and decreases with the increase in the void ratio. While the effect of plasticity index on the shear modulus is not significant in normally consolidated clay soils, the shear modulus increases as the plasticity index increases in over consolidated clay. Table 1 summarizes the effect of certain factors on the maximum shear modulus of soils.

Table 2.1 The effect of certain factors on $G_{\max}$ (after Vucetic and Dobry, 1987)

Increasing Factor	$G_{\max}$
Confining stress	Increases with σ'_m
Void ratio, e	Decreases with e
Geologic age, t_g	Increases with t_g
Cementation, c	Increases with c
Overconsolidation ratio, OCR	Increases with OCR
Plasticity index, PI	Increases with PI if $OCR > 1$; Stays about constant if $OCR = 1$
Strain rate	Not effect for non-plastic soils; increases with strain rate for plastic soils
Number of loading cycles, N	Decreases after N cycles of large γ_c ; but recovers later with time in clays; increases with N for sand

The shear modulus reduction with strain in cohesive soils is largely dependent on the plasticity index. As the plasticity index increases, the soil approaches the linear soil behavior and its nonlinearity decreases. This behavior is given in Figure 3 by Vucetic and Dobry (1991). Similarly, the variation of damping ratio with strain depends on the plasticity index. As the plasticity index increases, the damping ratio decreases. This behavior is given in Figure 4 by Vucetic and Dobry (1991).

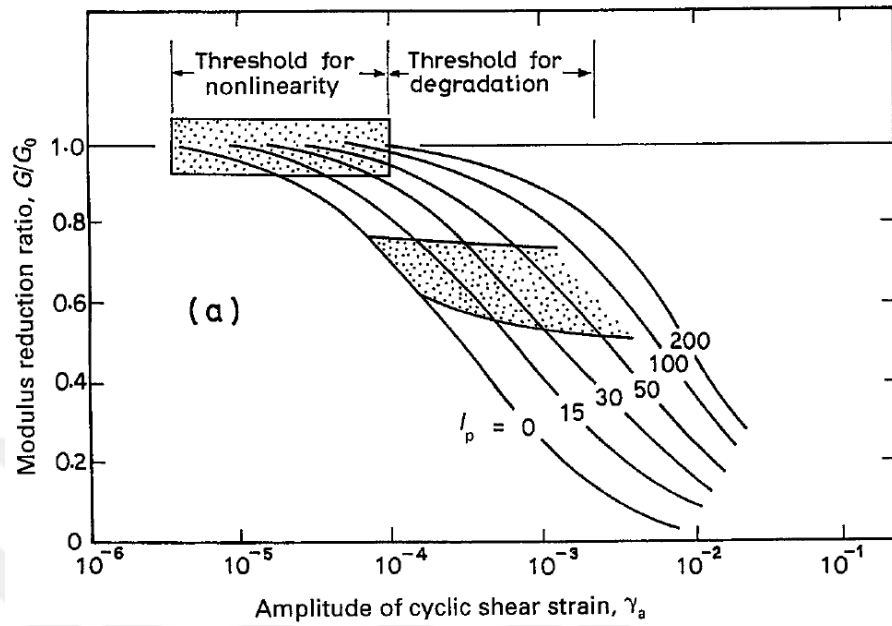


Figure 2.2 The effect of PI on the modulus degradation curves given by Vucetic and Dobry, 1991 (after Ishihara, 1996)

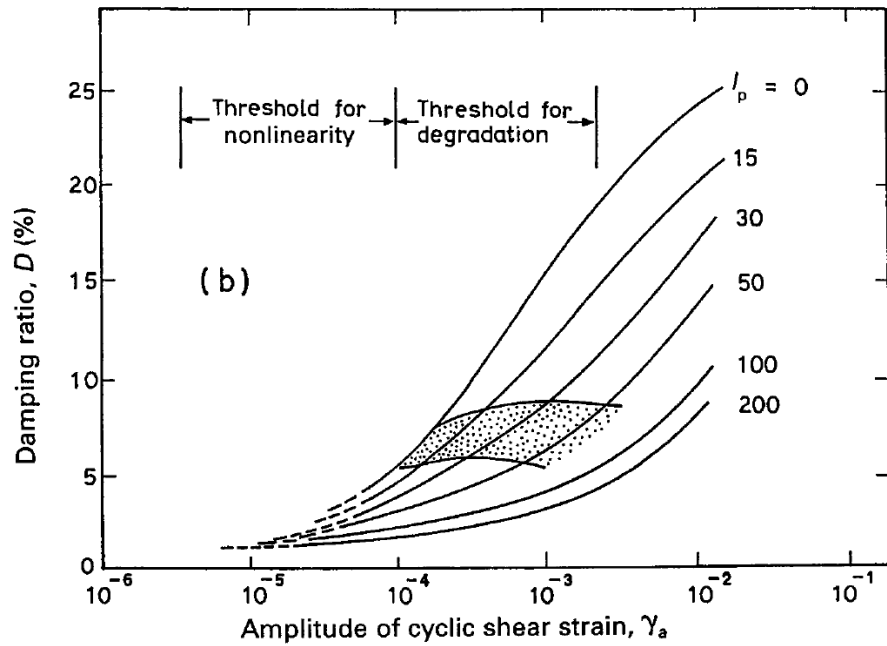


Figure 2.3 The effect of PI on the damping ratio curves given by Vucetic and Dobry, 1991 (after Ishihara, 1996)

Shear modulus degradation curves in cohesionless soils depend on the effective confining stress. The effective stress increase causes the cohesionless soils to show more linear behavior (Ishibashi, 1992; Darendeli, 2001 and Zhang, 2006). Figure 5 shows the variation of the curves under different effective stresses. Figure 5a shows that the shear modulus of cohesionless soils (where the $PI=0$) under low confining stresses degrades more as compared to the soils subjecting higher stresses. Similarly, the damping ratio of soils increase near the ground surface, where the confining pressure is low. However, the effect of confining stress is not that significant in the cohesive soils in which the degradation curves are similar (Figure 5b). The study shows the importance of the depth (confining stress) on the cohesionless soils.

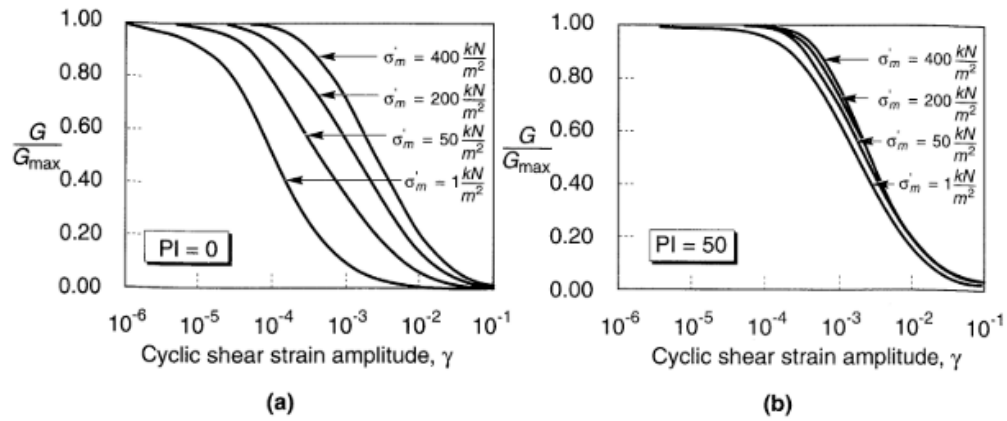


Figure 2.4 The effect of effective confining stress on the modulus degradation curves for a) cohesionless soil ($PI=0$), and b) cohesive soils ($PI=50$) (after Ishibashi, 1992)

Several researchers have studied the effect of certain factors on the modulus degradation curves (G/G_{max}). These factors and the effects are tabulated by Dobry and Vucetic (1987), and given in Table 2. Similarly, the variation of the damping ratio with certain factors are given in Table 3.

Table 2.2 Effect of environmental and loading conditions on modulus degradation (after Dobry and Vucetic, 1987)

Increasing Factor	$G/G_{\max}$
Confining stress	Increases with σ'_m ; the effect decreases with increasing PI
Void ratio, e	Increases with e
Geologic age, t_g	May increase with t_g
Cementation, c	May increase with c
Overconsolidation ratio, OCR	Not affected
Plasticity index, PI	Increases with PI
Cyclic strain, γ_c	Decreases with γ_c
Strain rate	G increases with strain rate but $G/G_{\max}$ probably not affected if G and $G_{\max}$ are measured at same strain rate
Number of loading cycles, N	Decreases after N cycles of large γ_c for clays; for sands, can increase (under drained conditions) or decrease (under undrained conditions)

The general framework about the effect of some factors on the shear modulus degradation and damping ratio has given so far. Vucetic and Dobry (1991) suggested the modulus reduction and damping relation for cohesive soils depending on the plasticity index as presented above. The most commonly used curves for sands were suggested by Seed et al. (1986) based on the empirical studies. The study has suggested the mean, upper and lower bound curves (Figure 5) and the confining stress effect can only be considered by using the lower bound or mean curves for shallow and upper bound curves for greater depths. The confining stress effect has been studied later by Ishibashi (1993), Darendeli (2001) and Zhang (2005).

Table 2.3 Effect of environmental and loading conditions on damping ratio (after Dobry and Vucetic, 1987)

Increasing Factor	Damping ratio
Confining stress	Decreases with σ'_m ; the effect decreases with increasing PI
Void ratio, e	Decreases with e
Geologic age, t_g	Decreases with t_g
Cementation, c	May Decreases with c
Overconsolidation ratio, OCR	Not affected
Plasticity index, PI	Decreases with PI
Cyclic strain, γ_c	Increases with γ_c
Strain rate	Stays constant or may increase
Number of loading cycles, N	Not significant for moderate cyclic strain and N

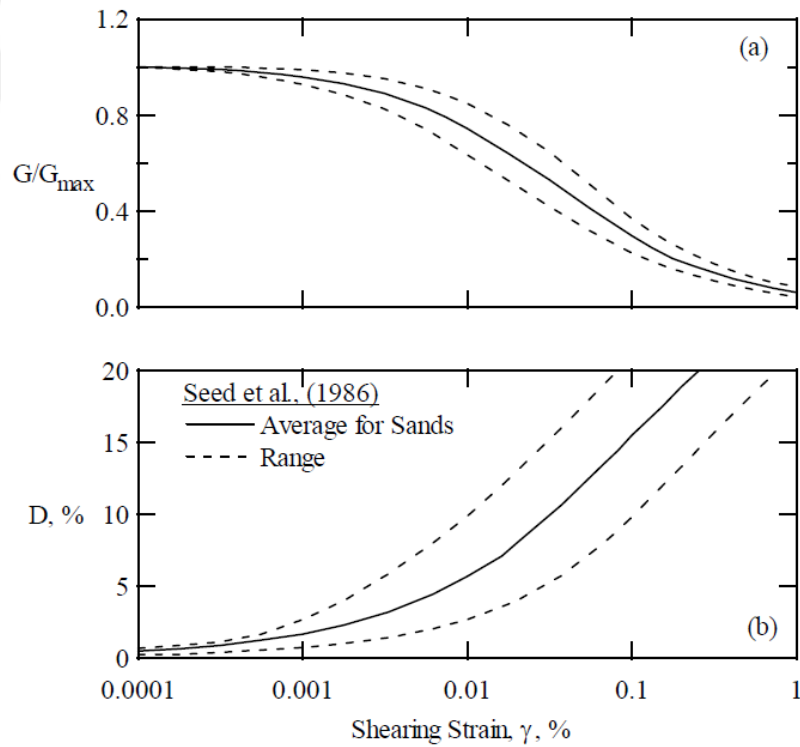


Figure 2.5 The curves proposed by Seed et al. (1986) for a) modulus reduction and b) damping ratio (taken from Darendeli, 2001)

Darendeli (2001) conducted some laboratory tests to formulate the modulus reduction and damping ratio curves depending on the confining stress for sands and plasticity index for clays. The results of the calibrated model for cohesionless soil is given in Figure 6. As it is discussed before, the confining stress increase cause the soil to behave more linearly and less reduction in the modulus is observed, which may significantly affect the site response analyses especially in the deep soil deposits.

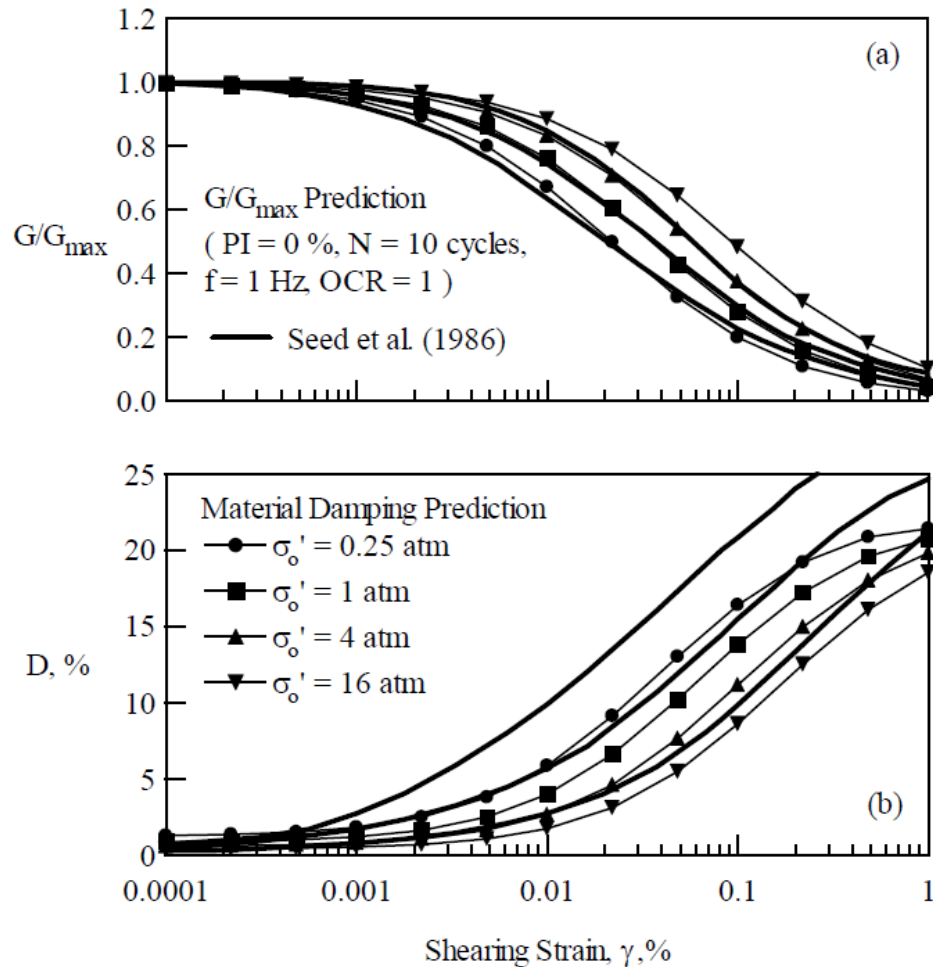


Figure 2.6 Comparison of the Darendeli (2001) and Seed et al. (1986) models for soil nonlinear behavior

Similar to the confining stress effect, Darendeli (2001) performed laboratory tests on cohesive soils to consider the effect of PI on the modulus reduction and damping ratio

curves. Figure 7 shows his model results. As it was given before, the increase in PI results in less damping.

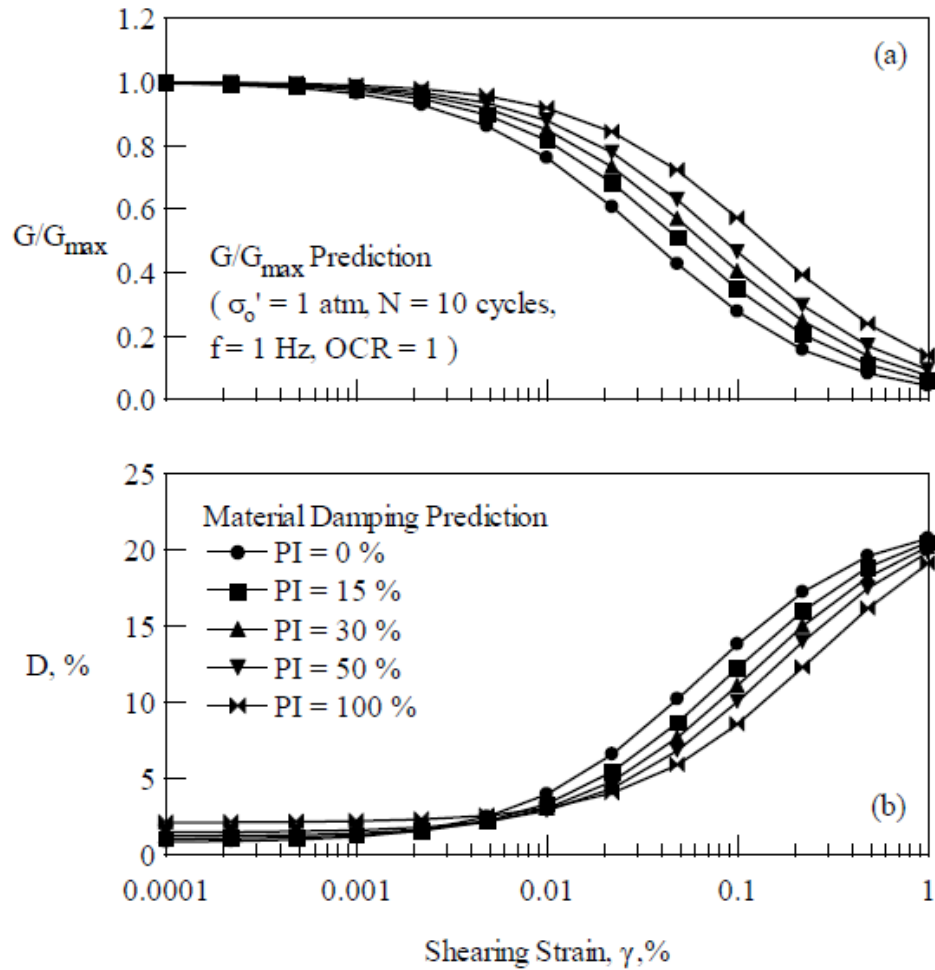


Figure 2.7 Effect of PI on modulus reduction and damping ratio curves (Darendeli 2001)

Parameters	Impact on shear modulus		Impact on material damping	
	Clean sands	Cohesive soils	Clean sands	Cohesive soils
Strain amplitude	***	***	***	***
Mean effective confining pressure	***	***	***	***
Void ratio	***	***	***	***
Number of loading cycles	+	*	***	***
Degree of saturation	*	***	**	—
<i>OCR</i>	*	**	*	**
Effective strength envelope	**	**	**	**
Octahedral shear stress	**	**	**	**
Frequency of loading (above 0.1 Hz)	*	*	*	**
Other time effects (thixotropy)	*	**	*	**
Grain characteristics, size, shape, gradation, mineralogy	*	*	*	*
Soil structure	*	*	*	*
Volume change due to shearing strain below 0.5%	—	*	—	*

***: Very important

**: Less important

*: Relatively unimportant

+: Relatively unimportant except for saturated sand

—: Unknown

Figure 2.8 Parameters affecting the soil behavior and their relative importance on the initial shear modulus and material damping presented by Hardin and Drnevich (taken from Jia, 2018)

Parameters	Impact on normalized shear modulus reduction curve	Impact on material damping curve (Sect. 2.3.3)
Strain amplitude	***	***
Mean effective confining pressure	***	***
Soil type and plasticity	***	***
Number of loading cycles	*+	***++
Frequency of loading (above 1 Hz)	*	**
<i>OCR</i>	*	*
Void ratio	*	*
Degree of saturation	*	*
Grain characteristics, size, shape, gradation, mineralogy	*	*

***: Very important

**: Important

*: Less important

+: On competent soils included in the study by Darendeli [144]

++: Soil type dependent

Figure 2.9 Parameters affecting the soil behavior and their relative importance on the shear modulus reduction and damping curves presented by Hardin and Drnevich (taken from Junbo Jia, 2018)

2.2 Soil Models

The soil behavior under cyclic loads have been investigated by several researchers (Hardin and Drnevich, 1972, Seed et al., 1986; Vucetic and Dobry, 1991, Darendeli, 2001; Zhang, 2005). The behavior obtained from the experimental studies must be written in a model form to be used in a ground response analysis. Kramer (1996) defined the stress-strain behavior of cyclically loaded soils by three classes: equivalent linear models, cyclic nonlinear models, and advanced constitutive models. According to Groholski et al. (2007), nonlinear soil behavior can be modelled either by Ramberg&Osgood (1943) or Kondner&Zelasko, 1963 (KZ), where the latter is used in 1D ground response analyses. The stress-strain curve of KZ model is given in Figure 8 and the shear stress-strain equation can be written as follows:

$$\tau = \frac{G_{max}\gamma}{1 + \frac{G_{max}}{\tau_{max}}\gamma} \quad (2.1)$$

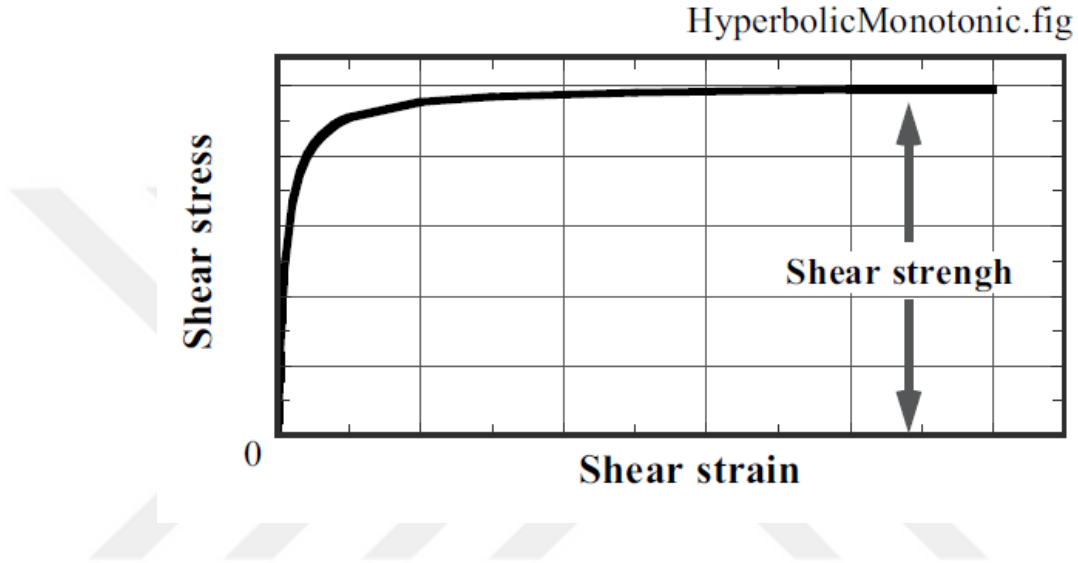


Figure 2.10 Stress-strain curve of Kondner&Zelasko model (Towhata, 2008)

2.2.1 Modulus Reduction

The above equation is rewritten by introducing the reference strain concept (after Hardin and Drnevich, 1972) and this form enables to obtain the modulus reduction curves using the reference strain.

$$\frac{G}{G_{max}} = \frac{1}{1 + \frac{\gamma}{\gamma_r}} \quad \gamma_r = \frac{\tau_{max}}{G_{max}} \quad (2.2)$$

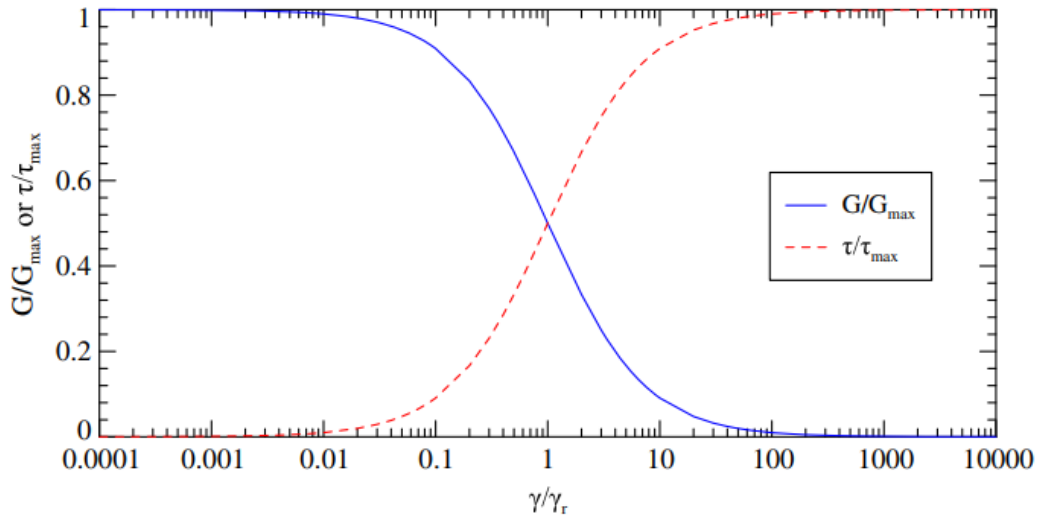


Figure 2.11 $G/G_{\max}$ and $\tau/\tau_{\max}$ in Kondner&Zelasko model (1963) (given by Groholski et al., 2006)

Hardin and Drnevich (1972) model is simple, yet efficient to implement, since the model parameters are the initial shear modulus and shear strength, which are the key parameters of a soil material. However, the two parameters are not enough to capture the soil nonlinear behavior in all strain levels. A similar model with an additional parameter “a”, has been suggested by Darendeli (2001) to overcome this problem. The parameter “a” is used for and the value of “a” is constant, 0.919. The reference strain, γ_r depends on the effective confining stress, plasticity index and overconsolidation ratio as it is given in the following equations, since these are the most important factors for the modulus reduction curves.

$$\frac{G}{G_{\max}} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_r}\right)^a} \quad (2.3)$$

$$\gamma_r = (\phi_1 + \phi_2 * PI * OCR^{\phi_3}) * \sigma'_0{}^{\phi_4} \quad (2.4)$$

where $\phi_1 = 0.0352$; $\phi_2 = 0.001$; $\phi_3 = 0.3246$; $\phi_4 = 0.3483$

2.2.2 Modified Kondner&Zelasko (MKZ) Model

A modification to the original Kondner and Zelasko (1963) model was made by Matasovic (1993) with the two additional parameters. These parameters were inserted into the original model and the new model was called as modified hyperbolic model.

$$\tau = \frac{G_{max}\gamma}{1 + \beta \left(\frac{\gamma}{\gamma_r}\right)^s} \quad (2.5)$$

β and s parameters allow the soil modulus reduction curve to better fit the experimental curves.

The Matasovic (1993) model further extended by site response analysis software, DEEPSOIL (Hashash and Park, 2001).

$$\gamma_r = \text{Reference strain} \left(\frac{\sigma'_v}{\text{Reference stress}} \right)^b \quad (2.6)$$

where σ'_v is the vertical effective stress and b is curve fitting parameter. The modified form of Kondner & Zelasko model given above is called as pressure-dependent hyperbolic model (or MKZ model by DEEPSOIL).

2.2.3 Generalized Quadratic/Hyperbolic (GQ/H) Model

The laboratory tests conducted by Darendeli (2001) shows the shear stress-strain behavior of soils. However, the resonant column and torsional shear tests are suitable for small and medium strain levels, respectively. The curves of Darendeli (2001) were extrapolated for larger strains. According to Phillips and Hashash (2009), the suggested curves can not sustain the shear strength of soils at large strains and a correction is required. Groholski (2016) proposed a method to consider the soil shear strength in conjunction with the modulus reduction curves. Groholski (2016) model results and its comparison with the modulus reduction curve of Darendeli (2001) is shown in Figure 10.

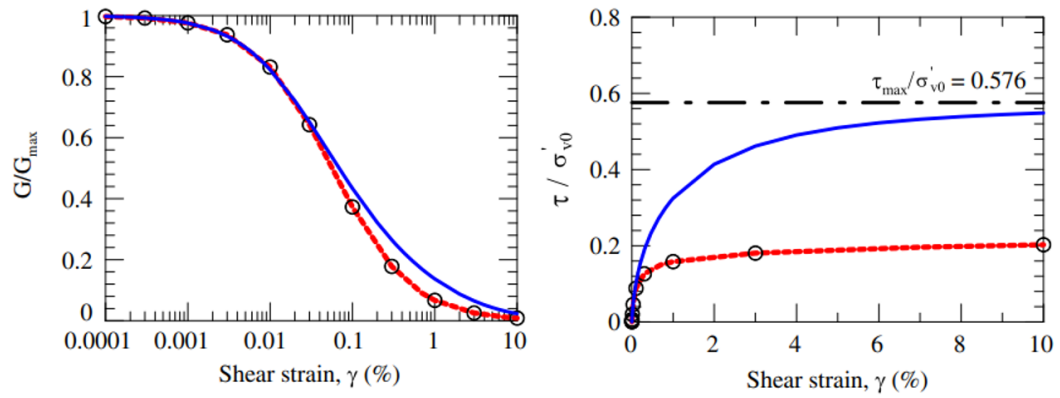


Figure 2.12 Comparison of the Darendeli (2001) curve at a constant effective stress and the proposed method by Groholski (2016)

According to Figure 10, the curves of Darendeli cause the $G/G_{\max}$ values to reduce significantly at large strains. Results of the reduction curves show that the shear stress, especially for strains greater than 1%, is significantly underestimated. The GQ/H model proposed by Groholski (2016) sustain the ratio of ultimate shear stress to initial effective stress to be 0.576, which corresponds to internal angle of friction value of 30° . This difference is more important at greater depth for sands. The effect of the shear strength on the shear stress-strain (or modulus reduction curves) behavior of soils is shown in Figure 2.13

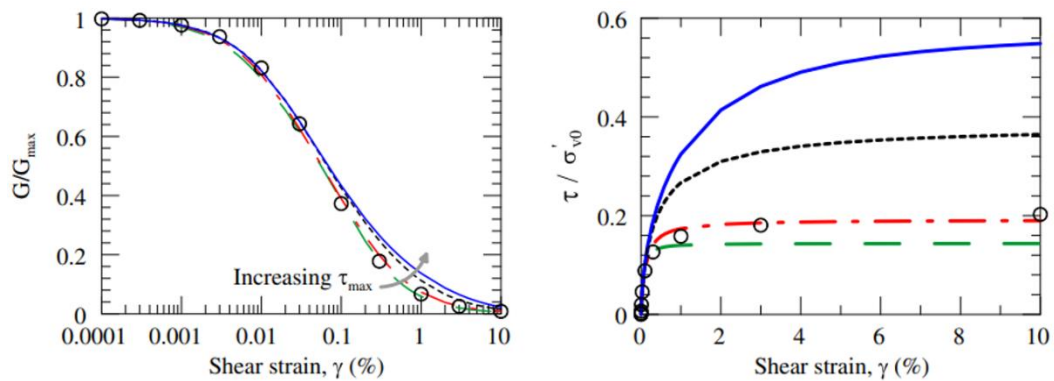


Figure 2.13 Effect of shear strength on the modulus reduction and stress-strain curves in GQ/H model.

2.2.4 Damping Ratio

The soil behavior under cyclic loading can be determined by hyperbolic model as shown in Figure 2.14. The damping ratio of the soil is determined using the area of hysteresis loop. The laboratory tests were performed and the damping ratio variation with the shear strain curves were presented by several researchers. The modulus reduction and damping curves are used in site response analyses with the equivalent linear method.

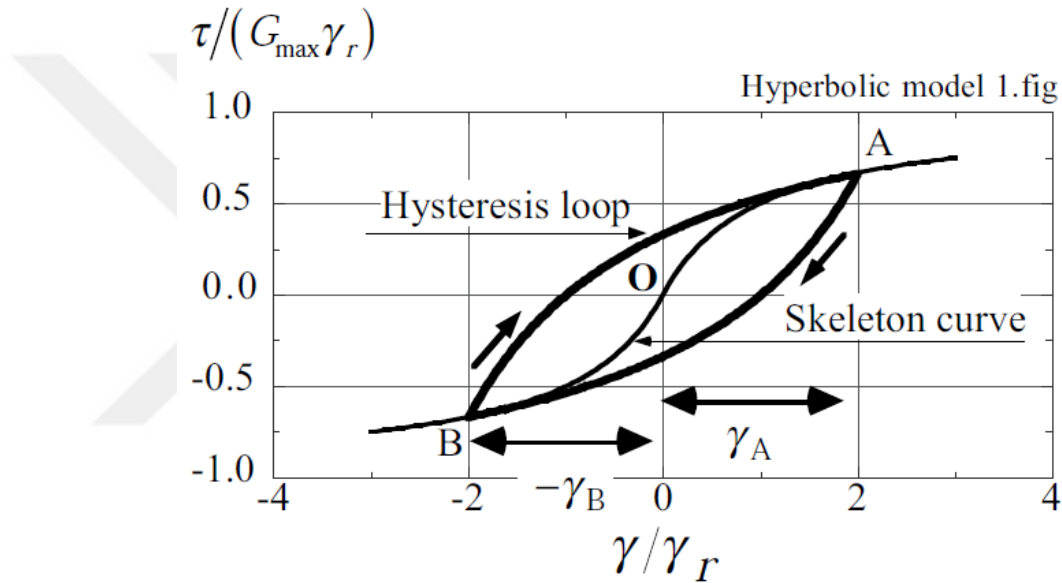


Figure 2.14 Hyperbolic stress-strain model for cyclic loading (Towhata)

Historically, the site response analyses are performed using the equivalent linear method. If the shear strains are greater than 1%, the method is no longer suitable according to Kramer (1996) and fully nonlinear analysis method must be used for high intensity loading (or soft soils subjected to moderate earthquakes). However, the unloading/reloading rule must be defined for the fully nonlinear analyses. The most commonly used rule for unloading/reloading behavior is the Masing rule. The details about the equivalent linear and fully nonlinear methods will be given in Chapter 3.

The main problem in the fully nonlinear analysis is that the damping ratio must represent experimental results. However, the Masing rule leads the soil to have unrealistically larger damping ratios than the experimental results. Groholski (2016) proposed a method to take this damping ratio difference into account. Figure 13a shows the difference in damping ratio for GQ/H model with the proposed approach and Masing rule. The hysteretic loops for each method are shown in Figure 2.15.

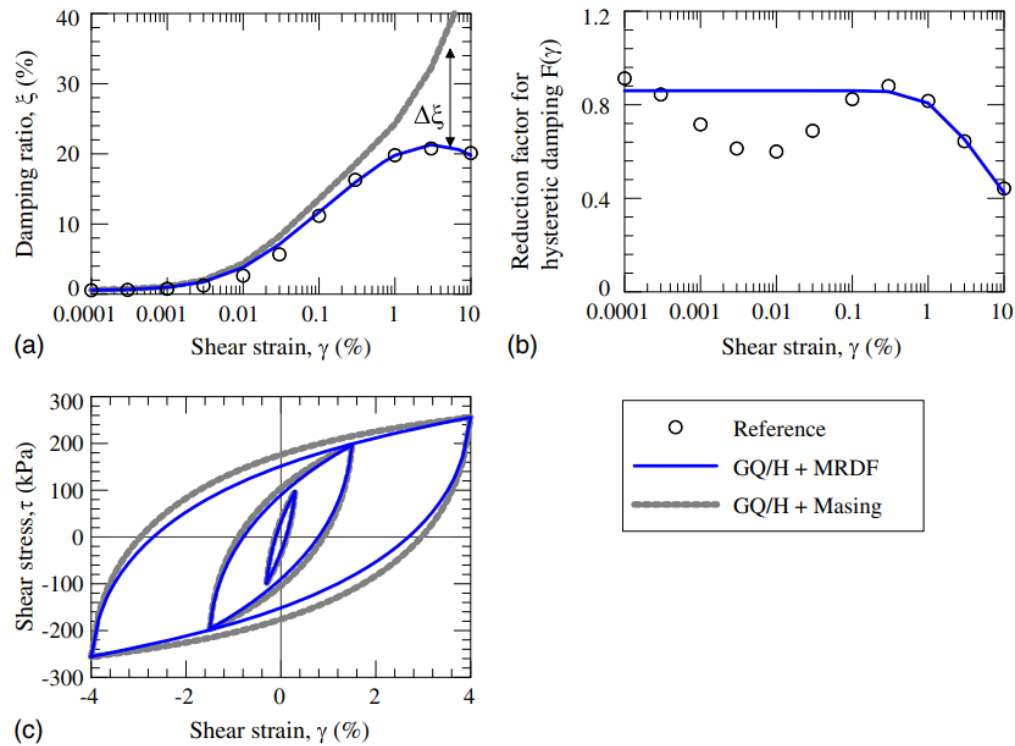


Figure 2.15 The hysteretic damping behavior in GQ/H model (Groholski, 2016)

2.3 Ground Response Analysis

Ground response analyses are performed to predict the soil stresses and strains under the given earthquake loading. Apart from the soil behavior, the acceleration records reaching to the ground surface must be predicted. The travel of waves from the rupture to the site is shown in Figure 14. Further picture of the problem is given in Figure 15 showing how the seismic loads on structures are subjected by the underlying effects.

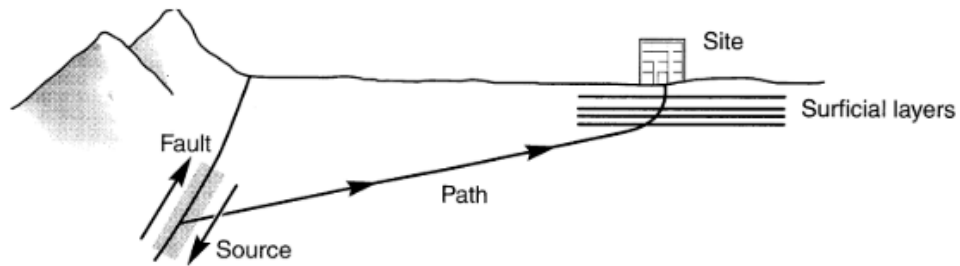


Figure 2.16 Wave propagation process (Kramer, 1996)

The rupture mechanism at the source is the first issue to consider in the estimation of earthquake records. The path in which the waves pass through the elastic domain also affects the recorded motion. To take the source mechanism and the path into account is not an easy task. The recorded strong ground motions are used for prediction and either deterministic or probabilistic methods are employed for this purpose, but these topics are not covered in this thesis.

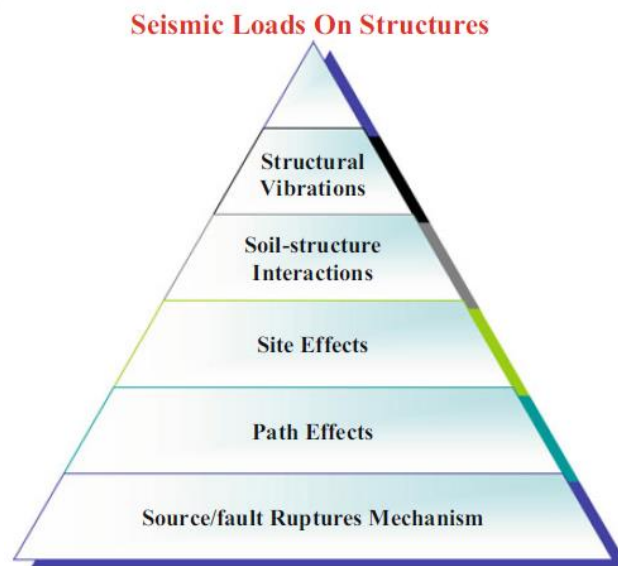


Figure 2.17 Seismic wave transmission from the source to seismic loading on structures (Jia, 2018)

Strong ground motion stations are founded on soil or rock outcrop surfaces and the earthquake records can be obtained either on the soil surface or rock outcropping as shown

in Figure 16. The recorded motions at the rock outcropping is different than the free surface motion, and the comparison of them may indicate the site effect.

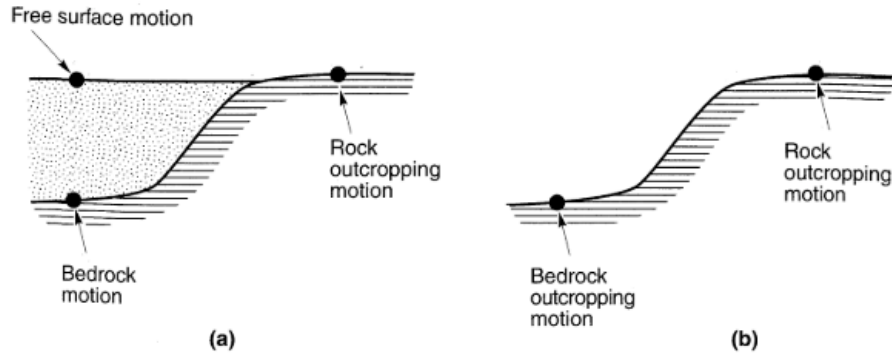


Figure 2.18 Strong ground motions recorded on a) the soil overlying a bedrock and b) no soil overlying bedrock (or rock outcropping)

Figure 17 shows how the recorded earthquake motions can be implemented to obtain the surface motion at another site. Rock outcropping motions can be directly used in a site response analysis as an outcrop motion. If the record is obtained at a soil surface, reverse site response analysis is required to back calculate the motion at top of rock or at rock outcrop, and this process is called as deconvolution.

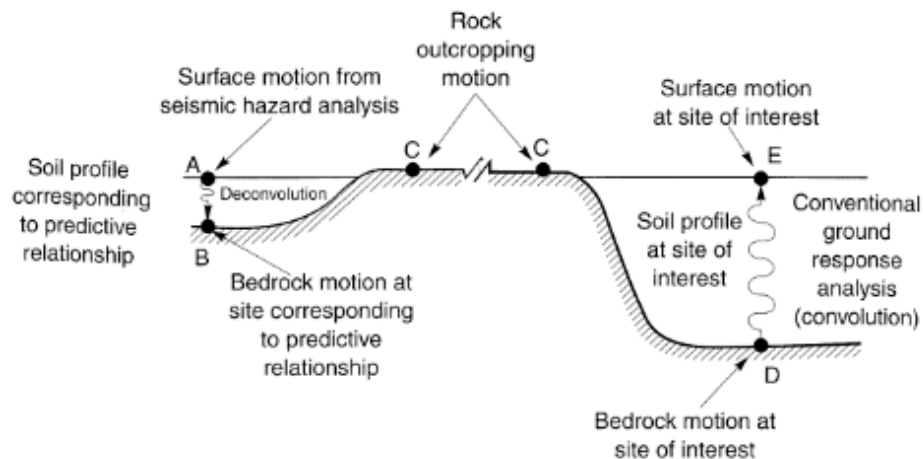


Figure 2.19 Procedure of the methods for ground motion prediction considering the local site conditions (Kramer, 1996)

2.4 Site Effect

The motions recorded on rock and soil has different characteristics. The comparison of acceleration response spectrum of the motions recorded on different soil conditions is shown in Figure 18. The period at the peak spectral accelerations (PSA), in which corresponds to the predominant period of the motion is lower in the rock site ($T_{p,rock}=0.2$ sec in Figure 18). However, the period is greater as the stiffness of soils increase. The curve of soft to medium clay in Figure 18 shows a wide range of predominant period due to the low stiffness of the soil.

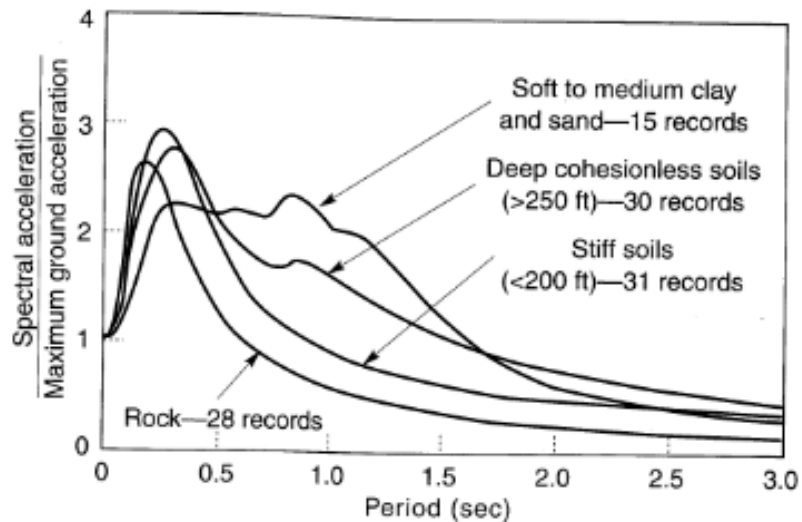


Figure 2.20 Normalized response spectra for different site conditions (Seed et al., 1976)

The site effect can be observed in real earthquake records. Dobry (2014) showed a cross section view of the Mexico City clay. Figure 19 shows acceleration spectrum at a dense cemented sand/gravel where the shear wave velocity is 500 to 900 m/s. The peak spectral acceleration is as low as 0.1g at that location. However, the site with $V_s=75$ m/s having 37m thickness amplifies the peak ground acceleration to about 0.2g and the PSA is as high as 0.7g. The natural period of a soil profile can be estimated by $4H/V_s = 2$ sec, that almost equal to the period of PSA. The same soil with greater thicknesses has damped the motion at the site as shown in Figure 19.

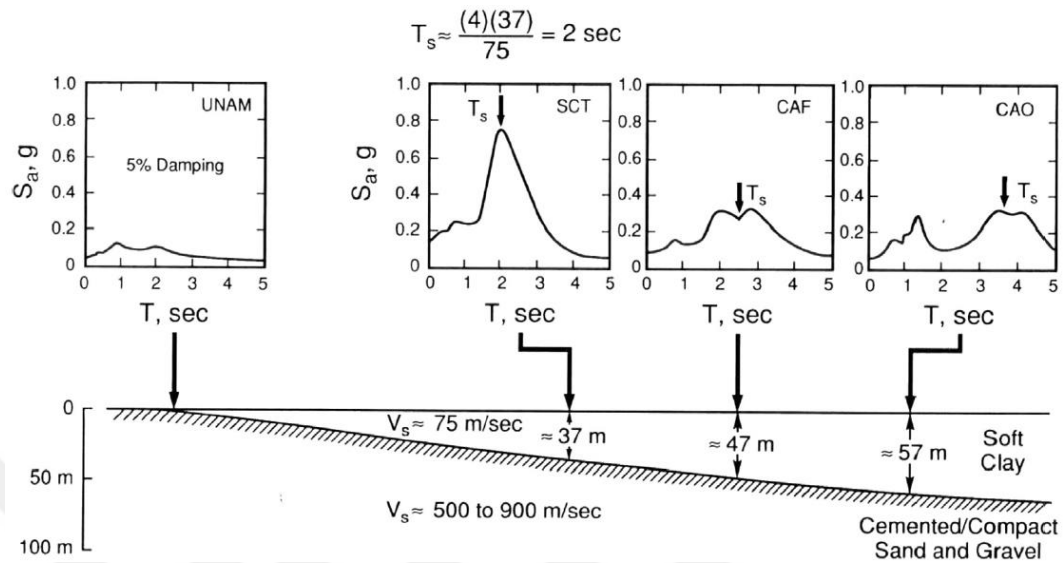


Figure 2.21 Earthquake amplification on the Mexico City soft clay in 1985 (Dobry, 2014)

San Francisco earthquake (1957) is another example to the site effect. A cross section given by Kramer (1996) shows the change of acceleration responses at the surface for different sites. The surface records may amplify or de-amplify depending on the frequency content of the input motion and dynamic properties of soils.

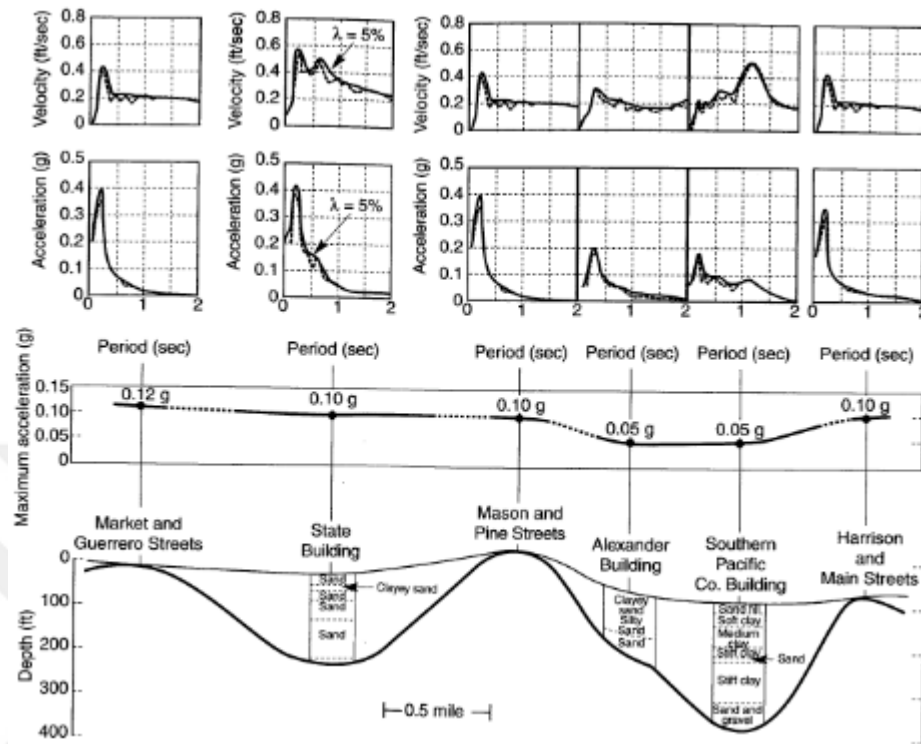


Figure 2.22 Variation of spectral acceleration along a section through San Francisco in the 1957 San Francisco earthquake (Idriss and Seed, 1968)

2.5 Soil-Structure Interaction

Structures are analyzed and designed by assuming the foundations to be rigidly connected to the underlying soil. However, the response of soil under an earthquake event may vary the structure response, and the vibration of the structure changes the soil response, which is called as soil-structure-interaction (SSI). One of the reasons of damages on the structures under earthquake loading is the SSI phenomenon. The basic modeling techniques for SSI are shown in Figure 21. Rigid base model in Figure 21a could not consider the soil effect. In Figure 21b, the structural elements are connected to the ground by springs which must represent the soil behavior under the considered loading event. Linear solutions are suggested by several researchers for the spring constants (Gazetas, Pais Kause etc.). The soil nonlinearity can be considered in these analyses by selecting the appropriate shear modulus. Strain-compatible modulus values are proper for SSI analyses

which can be determined by the site response analysis results. Figure 21c and 21d further increase the complexity of the problem by taking the damping ratio into account, which enables to model the system for fully dynamic analysis.

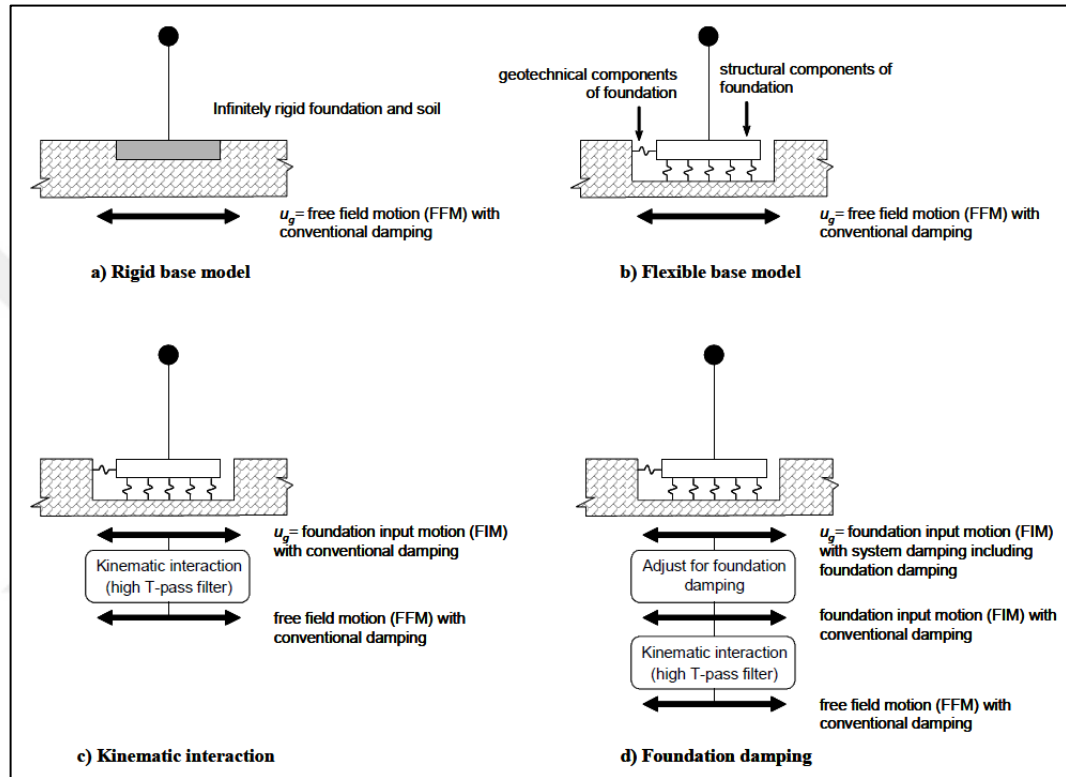


Figure 2.23 Soil-structure-interaction analyses methods (FEMA 440)

Historically, the effects of considering the SSI in the analyses has two results. The increase in the period and greater damping ratios. This behavior is shown in Figure 22. Period lengthening takes place due to the SSI effect, since the flexibility of the soil and foundation cause the system stiffness to reduce. This increase may result in lower or greater base shear depending on the period of the structure. For low-period structures, such nuclear power plants or short (rigid) buildings, the increase in the period lead greater base shear. However, medium to high period structures demand less base shear in the SSI case. From structural point of view, SSI analysis are less conservative for flexible structures and SSI is not required for safer design. The role of SSI on the medium to high

buildings are beneficial as the fixed base period is greater. Therefore, SSI analyses are highly popular in the nuclear power plant industry as the presence of sensitive elements requires rigid structures.

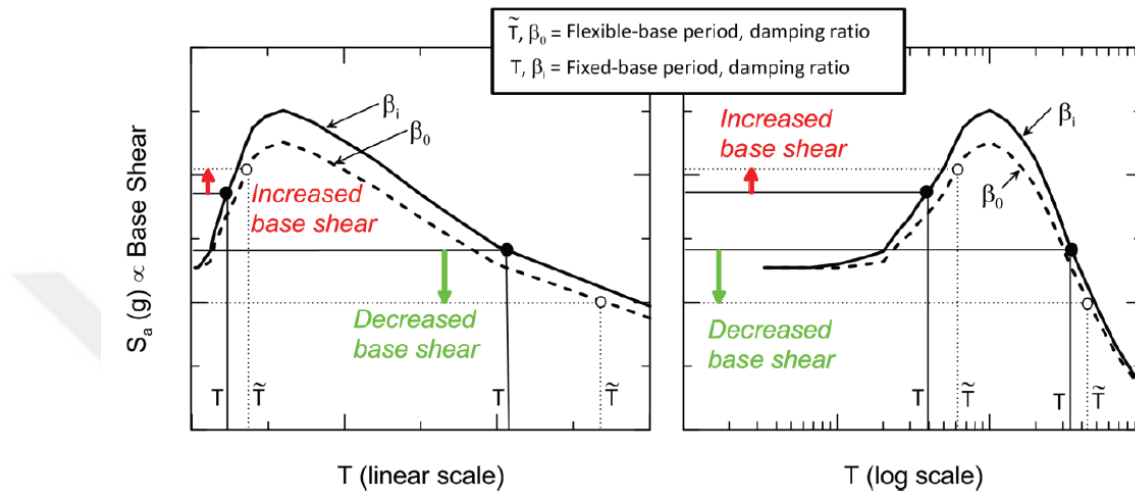


Figure 2.24 The effect of SSI on period lengthening on the structure response (NEHRP, 2012)

SSI concept has become more popular recently, since the behavior of soft soil conditions may vary the response significantly. Moreover, Mylonakis and Gazetas (2000) stated that the role of SSI on the structures may not be beneficial in some cases. Figure 22 shows the effect of period lengthening using the design response spectrum, but the responses are higher at greater periods for soft soil conditions and this may cause an oversimplification and unsafe designs according to Mylonakis and Gazetas (2000). They compare the design spectrum of NEHRP (1997) and actual spectra from the selected earthquakes in Figure 23.

The records of Kobe at Takatori station (1995), Bucharest (1977) and Mexico City (1985) earthquakes show the higher amplifications at the periods greater than 1.0 sec. The reason is that the high strains reduce the period of the soil profile and results in higher spectral accelerations as the period increase, which shows the significance of the soil on the SSI analyses, especially in soft and deep conditions.

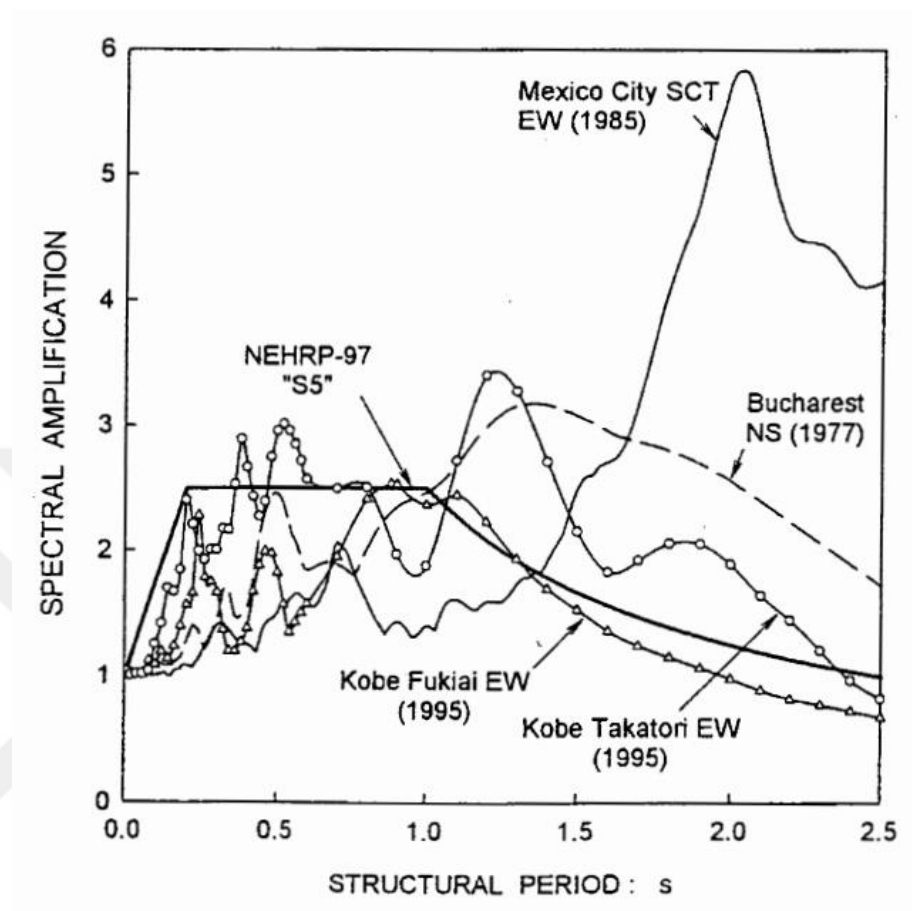


Figure 2.25 Comparison of a typical seismic code design spectrum and actual spectra from earthquakes (Mylonakis and Gazetas, 2000)

CHAPTER THREE

STRUCTURAL GEOLOGY OF İZMİR AND SURROUNDING REGIONS

In Western Anatolia, four main paleotectonic units developed during the historical period. These units are the Sakarya continent, the İzmir-Ankara suture (Mesozoic) zone, the Menderes massif, and the Lycian nappes, from north to south, respectively (Şengör, 1982; Şengör et al., 1985; Şengör, 1987; Yılmaz, 1997).

Sakarya continent is the basement unit formed by the combination of metamorphic and non-metamorphic units. Mesozoic and Cenozoic units overlie this base unit. The İzmir-Ankara suture zone is located between the Sakarya continent and the Menderes massif. This region generally consists of ophiolitic *mélange* and peridotites. It is estimated that some parts of the İzmir-Ankara suture zone underwent metamorphism between the Upper Cretaceous and Lower Miocene periods. This geological unit is called the Bornova schist or *mélange* near İzmir (Kaya, 1981; Yılmaz, 1997). The *mélange* consists of deformed volcanic sedimentary units. These units include basic lavas, shales and sandstones. Limestones, serpentines, and marble blocks can also be observed within the *mélange* unit. The age of the *mélange* was estimated as Upper Cretaceous from the examination of the red colored limestones.

The Menderes massif is the most important metamorphic unit in Western Anatolia. This unit is located between the İzmir-Ankara suture zone and the Lycian nappes. The Menderes massif has a complex lithology and structure. The age of the massif is estimated to be 35 ± 5 million years (Şengör et al., 1984). The Lycian nappes lie between the Menderes massif and the Bey Dağları autochthonous unit. It was developed by the thrusting of the oceanic crust on the Anatolide-Tauride platform during the subduction and subduction along the İzmir-Ankara suture zone during the Upper Cretaceous period. Later, during the continental collision in the Upper Paleocene-Lower Eocene, this

complex unit was changed again and the original complex unit of the Lycian nappes was formed (Dewey and Şengör, 1979; Şengör and Yılmaz, 1981; Şengör et al., 1984; Erdoğan, 1990; RADIUS, 1999; Özer et al., 2001; Bozkurt and Sözbilir, 2004; Emre and Sözbilir, 2007). The main paleotectonic units united before the Middle-Miocene period and formed the geological structure of Western Anatolia. The basic geological units in Western Anatolia are shown in Figure 3.1.

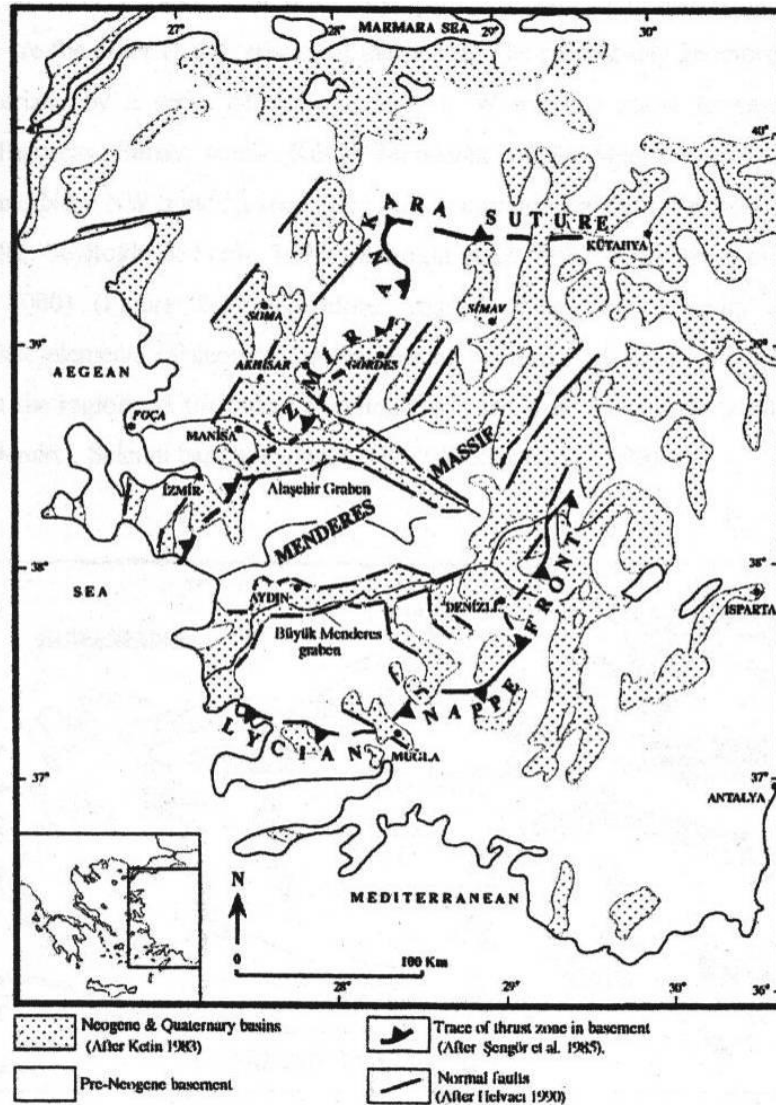


Figure 3.1 Basic geological units in Western Anatolia (Seyitoğlu and Scott, 1996)

Western Anatolia is one of the fastest deformation regions in the world. The speed of the north-south oriented opening in the region is approximately 30-40 mm/year. Continuing seismicity today is the most important indicator of this movement. Today's geomorphology consists of east-west oriented main graben systems. These are the Bakırçay, Simav, Gediz, Küçük Menderes, Büyük Menderes, and Gökova grabens from north to south (Westaway, 1990; Taymaz et al., 1991; Patton, 1992).

The study area is under the influence of gravitational forces today. With the effect of gravitational forces in the region, dip-slip normal faults have occurred. The fault systems forming the Izmir Bay are examples of this (McKenzie, 1978; Şengör, 1982; Şengör, 1987; Taymaz et al., 1991; Barka and Reilinger, 1997; Emre and Barka, 2000; Sözbilir, 2001 and 2002; Bozkurt and Sözbilir, 2004; Emre et al., 2005; Akyol et al., 2006). The age of the faults is Oligo-Miocene (Şengör, 1982).

In Horst regions, it is obvious that compressive forces will be effective and therefore reverse faults have occurred (Şengör, 1982; Şengör et al., 1985; Şengör, 1987; Taymaz et al., 1991; Barka and Reilinger, 1997). N-S, NE-SW and E-W trending faults affect the study area. Among the mentioned faults, EW trending ones are mostly dip-slip normal faults (Şengör, 1987; Sözbilir, 2001 and 2002; Bozkurt and Sözbilir, 2004; Emre et al., 2005; Akyol et al., 2006; Emre and Sözbilir, 2007). Fault planes offer slopes of 45-50 degrees. In addition to these, there are also NS, NE-SW oriented strike-slip faults in the region (Dewey and Şengör, 1979; Şengör et al., 1985; Westaway, 1990; Taymaz et al., 1991; Seyitoğlu & Scott, 1991; Seyitoğlu et al., 1992; Seyitoğlu & Scott, 1996; Akyol et al., 2006; Uzel & Sözbilir, 2008; Sözbilir et al., 2008; Sözbilir et al., 2009). İzmir Province in general and Aliğa and Menemen Districts in particular are located in the First Degree Earthquake Zone (RADIUS, 1999).

There are three different tectonic belts in the region. These belts are the Menderes Massif in the east, the İzmir-Ankara Zone in the middle and the Karaburun Belt in the west. The Menderes Massif and the Karaburun Belt have been a stable platform area where

carbonate deposition developed from the Triassic to the end of the Campanian. The first lowering of the İzmir-Ankara Zone developed in the region between Bornova and Seferihisar during the Campanian-Early Maestrichtian (Şengör et al., 1984; Erdoğan, 1990; Erdoğan and Güngör, 1992; Yılmaz, 1997; Özer et al., 2001; Sözbilir, 2001). and 2002; Bozkurt and Sözbilir, 2004; Emre et al., 2005; Akyol et al., 2006; Emre and Sözbilir, 2007; Uzel and Sözbilir, 2008).

In this first deformation, which started with the movement of the platform into the basin in the form of nappes during the flysch deposition between the Maestrichtian and Danian, the Bornova Complex was pushed over the Menderes metamorphics by large-scale tectonic movements (Erdoğan and Güngör, 1992; Yılmaz, 1997). This deformation is in the form of shears that spread over the entire volume of the Bornova Complex and present a fish scale geometry (Şengör, 1981; Şengör et al., 1984; Şengör et al., 1985; Erdoğan and Güngör, 1992; Yılmaz, 1997; Özer et al., 2001).

The Bornova Complex, which rose in the Miocene, was fragmented in the NE-SW direction in accordance with its internal structure, and the pitted areas were covered with lakes in accordance with the paleo geography of that period (Kaya, 1981; Yılmaz, 1997). In the second phase of the neotectonic period, as the uplift tension in Western Anatolia receded towards the Southern Aegean subduction zone, west-east faulting was effective and today's horst-graben morphology began to develop (Şengör, 1987; Westaway, 1990; Yılmaz, 1997; Akyol et al., 2006; Özkaymak et al., 2011). This movement also affected the sediments accumulated in the Miocene lake basins, and the rising and collapsing blocks underwent various deformations. The parts of the Miocene lake successions rising with the foundation formed thresholds between the old mountain masses. Sabuncubeli in the north of İzmir and Buca-Cumaovası in the south are typical examples of this (Seyitoğlu & Scott, 1991; Seyitoğlu et al., 1992; Seyitoğlu & Scott, 1996; Yılmaz, 1997; Uzel & Sözbilir, 2008; Özkaymak & Sözbilir, 2008; Özkaymak et al., 2011).

The tectonic trough (graben) stretching from the Gediz Valley to Kemalpaşa and from here to the İzmir Bay section via Bornova is one of the most distinctive structural elements in the region. The Belkahve Threshold, which separates the Kemalpaşa and Bornova plains from each other, is the part of the underlying flysch-carbonate unit that has been raised by striking the SW, in a long depression groove. The presence of hot water outlets in the southern parts of İzmir Bay indicates that active faults are active especially on the southern slopes. A similar situation exists in the northern coasts of the gulf, especially in Aliğa, Menemen and Karşıyaka regions (Tubitak 106G159, 2012).

3.1 Alluvial Geomorphology of İzmir and Its Surroundings

The crustal movements that developed in the Neotectonic period around İzmir formed the basis of the geomorphological units. In the first phase of this period, the İzmir-Ankara suture zone was fragmented, and NE-SW and NW-SE trending extensions occurred. This stage was followed by the uplift of the Menderes massif and Miocene lakes filled the depressions between the rising blocks. During this period, there was also a severe volcanic activity in the region. Beginning from the end of the Miocene, the valleys between the uplifted blocks deepened and newly fragmented formations in east-west direction were formed. Today's morphology has been shaped in this faulting and fragmentation process. The Kemalpaşa-İzmir branch of the Gediz graben is one of the most important of these young depressions. The floor of the valley was filled with flood type terrestrial deposits in Post-Miocene and the alluvial cover covered the terrestrial unit (Kayan, 2000).

The alluvial plains on the shores of İzmir Bay have covered the terrestrial unit formed by the tectonic movements that have been going on since the Pliocene. While the coarse material carried by the rivers from the high levels of the mountains to the plain was stored in the form of accumulation cones, the accumulated material was raised together with the mountain blocks by tectonic movements. The finer materials were collected in the middle part of the plain (Tubitak 106G159, 2012).

As a result of the subsequent erosion of the rising blocks, eroded material was carried to the lower floor. These erosional areas can be seen on the hills north of Karşıyaka and Bornova. There are sediments with coarse volcanic blocks or limestone clastics in the north of İzmir Bay. These formations cannot be encountered in the south of the Gulf. The fact that this unit is seen towards the north indicates that the Bornova plain has deepened by sliding towards the south (Kayan, 2000). Bornova is not a typical delta plain. This is because there is no river crossing the Bornova plain. However, there are three main streams flowing into the sea from the Bornova plain. They arise from the mountains surrounding the plain. These streams are Kocaçay, Gökdere (Arabian Stream) and Manda Stream. It forms the Kocaçay Bornova alluvial deposit cone. Gökdere (Arabian Stream) forms alluvial deposits south of Bornova. The Manda Stream (Kavaklıdere) shapes the sediments in the eastern part of the Bornova plain. The eastern part of the plain is higher than the western part, as there is not enough material to be transported from the high region in the east to the sea in the west. This is the reason why typical delta development cannot be observed along the Bornova coast (Kayan, 2000).

Despite being shaped on the same terrestrial deposits, the alluvial plains surrounding İzmir Bay show differences depending on their geomorphological development. Karşıyaka is a developed delta mouth in front of the skirts of Yamanlar Mountain. The Old Gediz River Delta, located in the northwest of the Karşıyaka/Bostanlı region, was shaped by the accumulation of alluvium carried by the Gediz River in the Quaternary period. The bed of the Gediz River was oriented towards the northwest mouth of the İzmir Bay at the end of the nineteenth century. This area has been one of the densely populated areas of the city for 30 years. High-rise buildings, apartments and industrial facilities are built on alluvial delta floors. There are generally continental origin, brown, overconsolidated solid-very solid clays in the region. Quaternary alluvial deposits carried by the Gediz River cover clays of terrestrial origin. These alluvial deposits generally consist of normally consolidated or incompletely consolidated soils. Under the terrestrial origin clays, andesite-based bedrock is located. The complex geological and geomorphological structure of the delta soils caused horizontal and vertical discontinuities

in the soil stratification. These discontinuities, which can be observed even at close distances, are the main reason for the emergence of total and differential settlement problems in high-rise buildings and apartments in the region (Kayalar, 1991; Özden, 2000).

3.2 Change of Engineering Bedrock Depth in İzmir

The bedrock levels throughout the study area vary in coastal districts such as Çiğli, Mavişehir, Karşıyaka, Bayraklı and Balçova. The general trend in this variability is that the bedrock depth increases as one approaches the coast. Knowing the bedrock depth is of particular importance in terms of foundation design, but it is one of the influential factors in dynamic soil behavior analysis.

The level of the bedrock cannot generally be determined from the drilling logs in the geotechnical reports prepared for regions with deep alluvial layers. Information about the depth of the rock can be obtained from a limited number of soil survey reports prepared in areas where the bedrock is close to the surface. For example, it was determined that the bedrock is 23 m deep around Alaybey (Çakıcı, 1997) and 54 m deep near Karşıyaka Stadium (Çakıcı, 1998). Information about the bedrock level can also be obtained from groundwater exploration drilling logs. Although the depths of these drillings are higher than the engineering drillings in the ground survey reports, it is not possible to determine the soil profile up to the bedrock in these drillings, since detailed logs are not kept in water drillings. In order to eliminate the stated deficiencies, deep drillings are planned within the scope of Work Package No.4. According to the logs at hand, the andesite pebbles reached at the base of the delta area are characterized as engineered bedrock in terms of dynamic soil behavior. Andesite pebbles are reached in the AOSB in the range of 200-250 m. Towards the north from Mavişehir, the depth of the rock gradually decreases, while it is 270 m on the coast, it becomes 17 m towards the skirts of Yamanlar Mountain. It has been determined that andesite pebbles are at a depth of 138 m in the parcels to the north of Dudayev Boulevard, which is still the scene of intense construction. The bedrock depth

gradually decreases in the West-East direction from Bostanlı to Karşıyaka. Andesite pebbles found at 136 m levels in the Forest Nursery just behind Karşıyaka Stadium are seen as andesite in the Alaybey (Naldöken) section. The andesite depth in this section is 23 meters. However, it is stated that the bedrock in the coastal area of Alaybey shows a sudden decrease and descends to a depth of 50-60 m. The bedrock (andesite), which is found to be 9 m deep in a location very close to the andesite formation outcropping in Bayraklı, first descends to 23 m when going south in the Bayraklı coastal area, and dives down quite abruptly at the Directorate of Public Works. In the sounding made for the Strong Ground Motion Station at the aforementioned location, the bedrock was not cut even though it was descended to 63.5 m. The claystone was reached at a depth of 216 m in the drilling carried out in Manavkuyu.

The bedrock (weathered tuff) was reached at a depth of 97 m during the deep sounding carried out in the Alsancak district (İzmir Fair Marriage Office) in the coastal part to the south of the İzmir Bay. During the drilling in Güzelbahçe, the rock (claystone) depth was determined at 11.5 m. In Balçova, on the other hand, the bedrock depth increases in the east-west direction and deepens from 61 m towards the sea at the Kipa Shopping Center location. Base rock could not be reached in well 01-BLC (İnciraltı DEU Marine Sciences Institute), and sandstone pebbles were detected in the range of 94-113 m.

During the engineering drillings in Menemen and Aliğa regions, the bedrock (weathered tuff) was found at a level of 69.5 m, within 3.0 km of the south of Çanakkale-İzmir highway, near Ulukent. As in the Mavişehir example, the rock level deepens towards the sea in the Menemen Plain. The Menemen Plain alluvial successions formed by the new bed of the Gediz River can be clearly observed through the bedrock depths. In high altitude districts of İzmir (Buca, Yeşilyurt, parts of Bayraklı outside the coastline), the bedrock depth is quite shallow. In these districts, the bedrock is entered within the first 3.0 m depth from the surface. Similarly, in Urla and Aliğa district centers, the rock level is just above the surface. The depth from the surface of the tuff formation found between Menemen and Aliğa is variable in the range of 12~30 m.

3.3 Earthquake Hazard for İzmir and Its Surroundings

As mentioned in the previous sections, Western Anatolia, one of the regions with high seismic activity in our country, is under the influence of north-south extensional tectonics, and many east-west directional graben systems have developed in the region resulting from this tectonic structure. The main graben systems in the central part of Western Anatolia are the Gediz and Büyük Menderes grabens. Normal faults have developed in parallel with the main graben systems in the region.

The closest seismic activity area around İzmir is located at the western end of the Gediz graben system. The general tectonic structures in this graben system are normal faults. Neotectonic period structures outside the Gediz graben system have strike-slip fault characteristics. There are normal faults in the south and east of the İzmir Bay. The main ones of these faults are İzmir Fault, Manisa Fault and Bornova Fault. The northeast-southwest trending Tuzla Fault and Foça-Bergama Fault Zone in the north and south of İzmir Bay are strike-slip and potential earthquake sources. The NNE-SSW trending Karaburun Fault at the western end of İzmir Bay has been evaluated as a strike-slip fault. Active tectonic resources in İzmir city center are shown in Figure 4.32. These are considered as faults that produce earthquakes with close focus in the city center of İzmir and are expected to produce earthquake motions containing high-frequency harmonics. In addition to these faults, earthquake movements generated by active tectonic sources located 50~80 km from the city center are expected to be rich in low-frequency harmonics and pose a greater threat to high-period structures located on ground profiles consisting of thick alluvial layers. Therefore, active tectonic sources producing near and far focused earthquakes should be considered together, especially for the design acceleration spectra to be developed for İzmir City Center. In this context, the critical faults around İzmir are generally given in Figure 4.32. Although detailed examination findings about these faults are available in various sources (Emre, Ö. et al., 2005), a summary of the subject is presented below in order to explain the earthquake hazard of İzmir.

CHAPTER FOUR

EQUIVALENT LINEAR AND FULLY NONLINEAR METHODS

The past earthquakes show the difference in the motions recorded at the soil surface and rock outcrop. Idriss and Seed (1968) stated the local soil conditions may increase the amplitude of the rock motion which is called soil amplification. Ground response analysis are performed to predict the soil response and the surface motions under a specific earthquake motion defined at the top of rock or outcropping rock. The first software for 1-dimensional ground response analysis was developed by Schnabel et al. in 1972 (SHAKE). The program was improved several times and modified versions were released (SHAKE91 and SHAKE2000). EERA is the another software for 1D ground response analysis which was written in FORTRAN 90 and released in 1998. These programs employ the equivalent-linear analysis method for the stress-strain relation of soil material. More comprehensive approach in the ground response analysis (GRA) is the fully nonlinear methods. DMOD and DEEPSOIL are the most common softwares for the nonlinear GRA. The details of the equivalent linear and fully nonlinear approaches are given in the following section.

The input earthquake motion in GRA must be defined. Strong ground motion stations can be located at the soil surface, rock outcropping and top of rock (bedrock). In the GRA, it is important to define the location of the input motion. If the station is on the soil surface, the motion at the top of rock or rock outcropping must be found by reverse site response which is deconvolution analysis. Bedrock (top of rock) motion can also be obtained in the vertical arrays. Figure 24 shows the difference of these motions recorded at different locations. The motions recorded at the soil surface includes the properties of the soil profile. The top of rock stations are scarce as compared to surface ones. Therefore, it is important to have rock outcropping motions if available, or it can be obtained by deconvolution analysis and the result can be employed directly in another site.

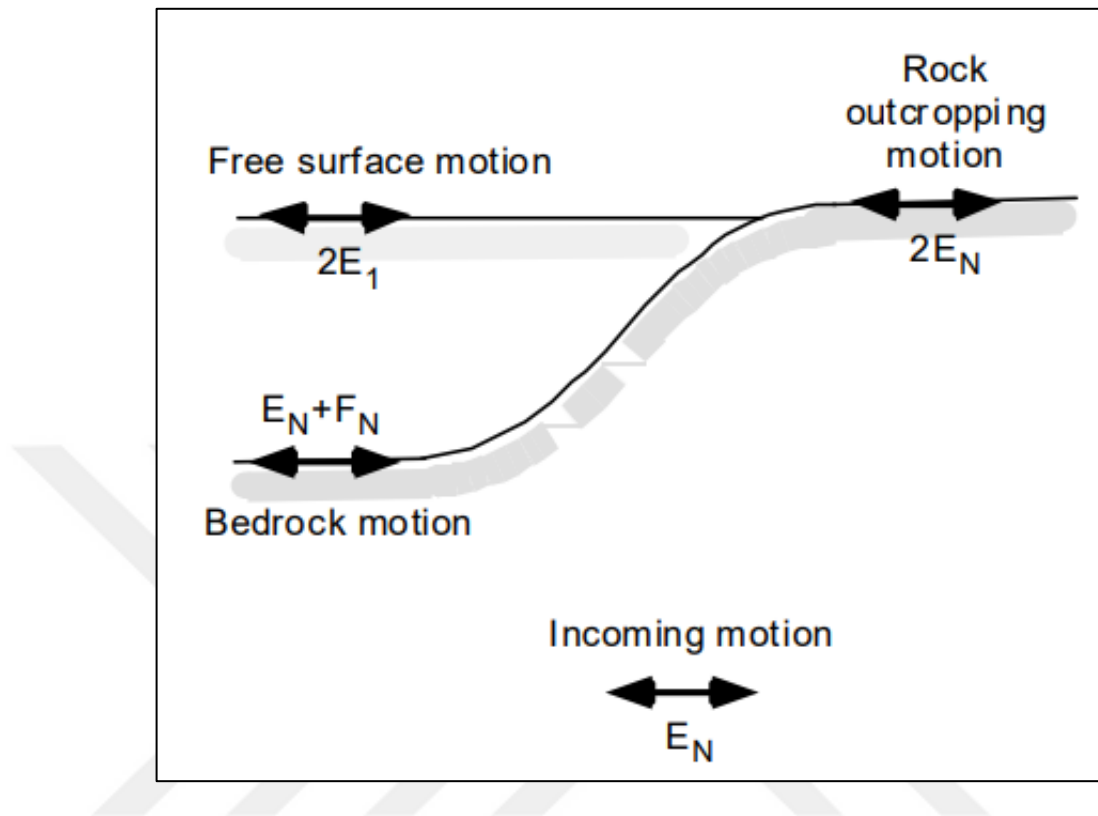


Figure 4.1 Terminology of the motions in ground response analysis (EERA Manual)

4.1 Equivalent-Linear Analysis in Frequency Domain

Historically, ground response analyses are performed using the equivalent-linear approach. Stress-strain behavior of soils is nonlinear in nature, and the nonlinearity can be achieved by iterative approach. In reality, the stress-strain curve at very small strain is almost linear in which the behavior can be represented by the maximum shear modulus. The soils tested in the laboratory show hysteretic loops under cyclic loading. However, the hysteretic behavior could not be considered in the dynamic analyses in the linear approach. The reduction of secant modulus can be predicted using the stress-strain curve of soils material. Another important feature of the dynamic response is the damping ratio. Damping of the soil depends on the area inside the cycle of a stress-strain loop, which can be predicted as a strain-dependent property. Figure 25a shows the true nonlinear behavior of soils under cyclic loading with the hysteretic loop. This behavior can be approximated

by the equivalent linear approach as shown in Figure 25b. The strain-dependent secant modulus and damping ratio values are predicted and used in the analyses.

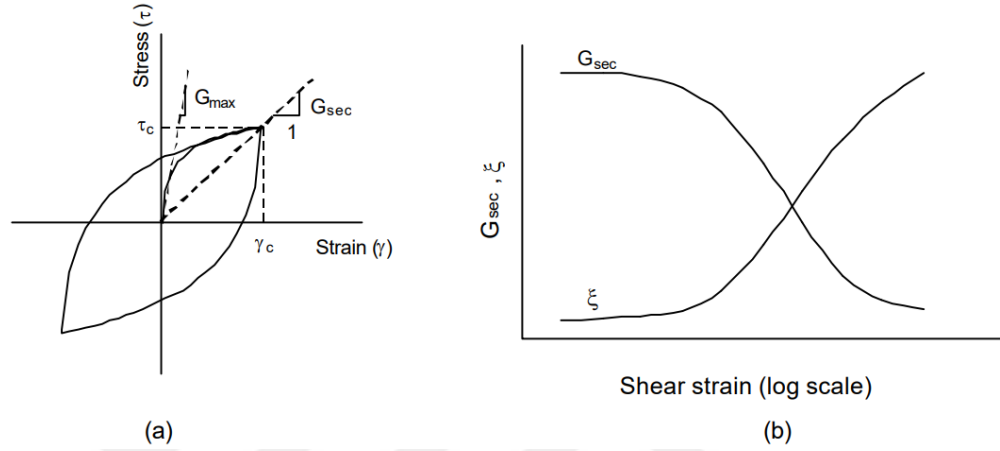


Figure 4.2 Equivalent-linear model: a) Hysteresis stress-strain curve, and b) Variation of secant shear modulus and damping ratio with shear strain

The procedure of the equivalent linear approach was summarized by EERA Manual as follows.

- The first iteration is performed using the small strain modulus and damping ratio.
- The maximum shear strain is obtained in the each soil layer.
- Effective strain is calculated by multiplying the maximum shear strain with R , the ratio defined depending on the earthquake magnitude as follows:

$$\gamma_{eff} = \gamma_{max} * R$$

$$where R = \frac{M - 1}{10}$$

- Effective strain value calculated above in the previous step is used to predict the modulus reduction (G/G_{max}) and damping ratio values for each layer.
- The second linear analysis is performed using the new shear modulus and damping ratio values.

- The iteration is repeated until the calculated shear modulus and damping ratio values calculated in the previous iteration converges to the new one with a specified.

The procedure explained above is shown in Figure 26.

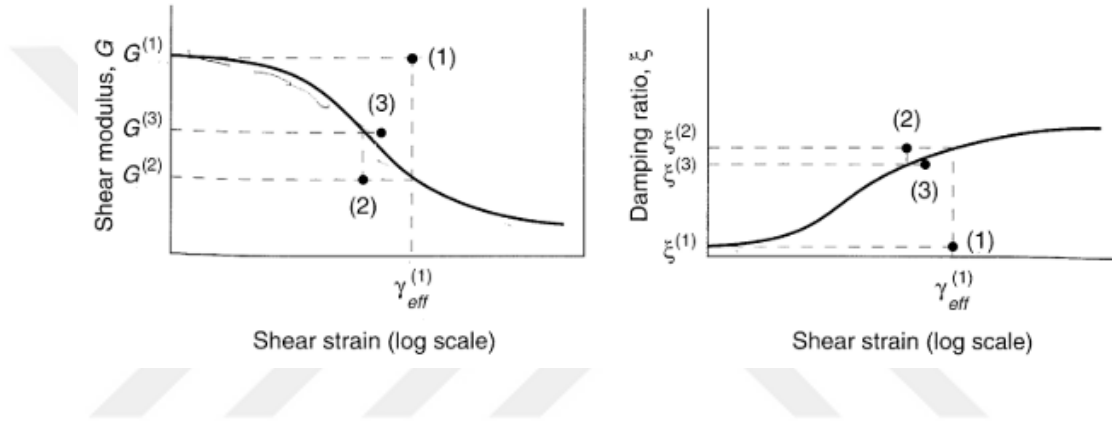


Figure 4.3 The iteration procedure for strain-compatible shear modulus and damping ratio in equivalent linear analysis (Kramer, 1996)

The procedure of the equivalent linear method described above is summarized by Towhata with a flow chart as shown in Figure 28. The analysis consists of two parts: The initial estimates for soil behavior and the selection of earthquake motion. For soil behavior, the maximum shear modulus of soils must be defined by the shear wave velocity as follows:

$$G_{max} = \rho V_s^2$$

Then, the G/G_{max} and damping curves are defined depending on the soil types. For cohesionless soils, the curves must be properly selected to consider the effective confining pressure. For cohesive soils, the curves depend on the plasticity index. The strain under the considered earthquake is calculated as a first guess and strain-compatible modulus and damping values are determined for the second iteration.

The earthquake motions recorded at the stations are the time histories. However, the equivalent linear analysis is performed in frequency domain, not in time-domain. Therefore, the time history of the motion is transformed to the harmonic components by Fast Fourier Transformation (FFT). The analysis results are calculated for each harmonics, and then the reverse FFT is applied to obtain the outputs in time-domain.

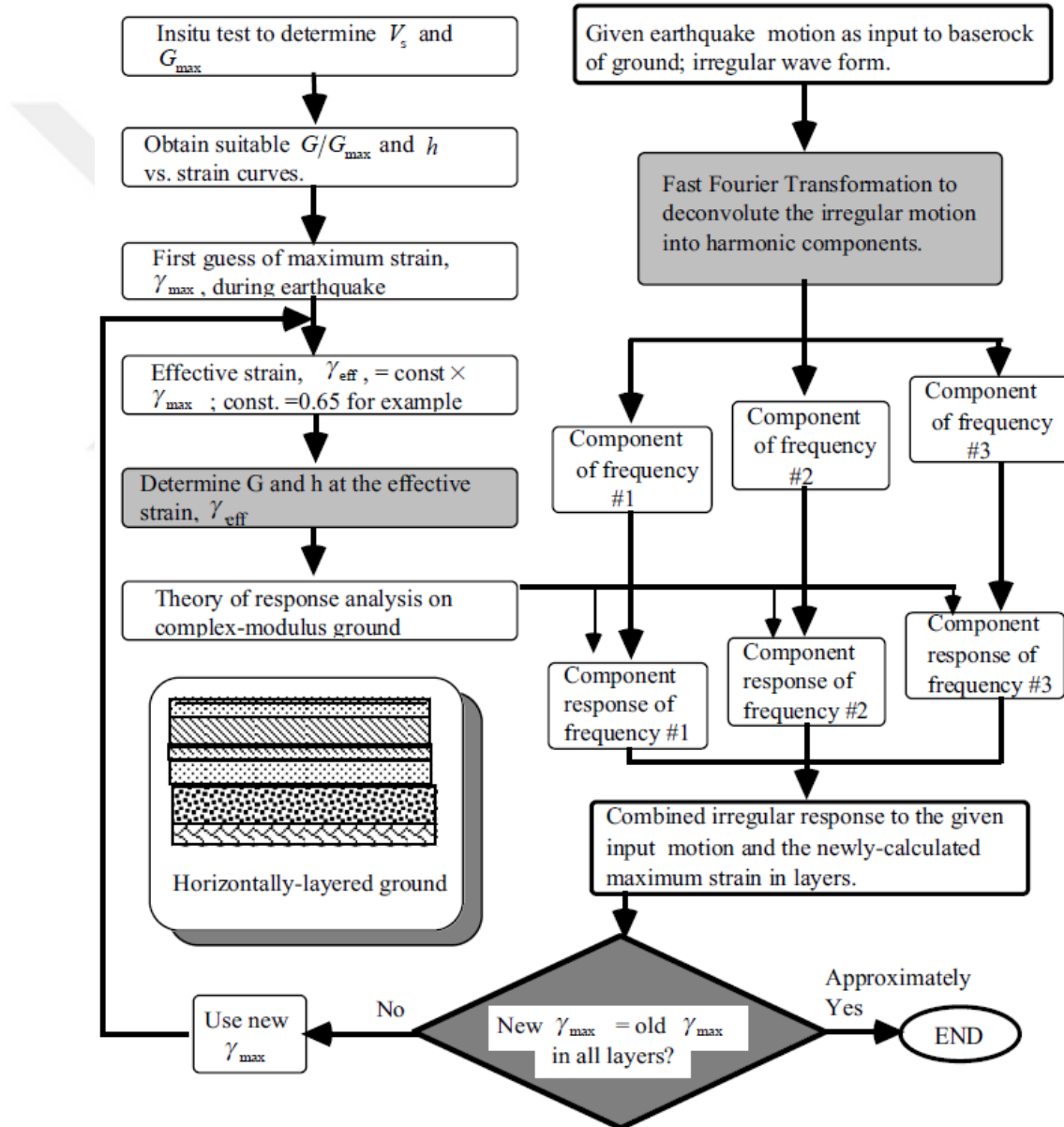


Figure 4.4 Flowchart of the equivalent linear analysis approach (Towhata)

4.2 Fully Nonlinear Analysis in Time Domain

Fully nonlinear analysis requires to solve the following equation of motion in time domain.

$$m \frac{d^2 u}{dt^2} + c \frac{du}{dt} + ku = R(t)$$

where u : displacement, du/dt and d^2u/dt^2 are the velocity and acceleration, respectively. $R(t)$ in the equation is the external load which is changing with time, t . The solution of the above differential equation of motion can be solved with any numerical techniques such as the Newmark's β method.

The stress-strain response of soils under cyclic loading is hysteretic in nature. In time domain analyses, the response of the soil in any time step must be calculated under the irregular input motion. Figure 29 shows the shear stress-time history of a soil response and the corresponding shear stress-strain curves. Since the external load in the earthquake analysis consists of loading and unloading, the soil behavior must include the first loading (backbone curve) and unloading/reloading behavior. The Masing's rule is the most common method in the geotechnical earthquake engineering as shown in Figure 29b. The rule for unloading/reloading is that the initial modulus is the same with the backbone curve, but enlarge it by a factor of 2. The simple equation for the backbone curve is defined as:

$$\tau = \frac{G_{max} \gamma}{1 + \frac{\gamma}{\gamma_r}} \quad (4.1)$$

The Equation 4.1 is scaled by factor 2, for unloading/reloading behavior in Masing's rule as follows:

$$\frac{\tau - \tau_A}{2} = \frac{G_{max} \frac{\gamma - \gamma_A}{2}}{1 + \left| \frac{\gamma - \gamma_A}{2\gamma_r} \right|} \quad (4.2)$$

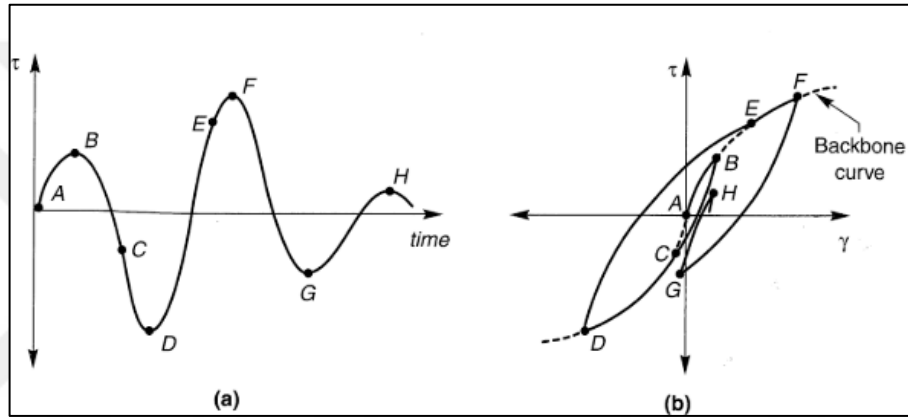


Figure 4.5 The stress-strain behavior in time history analysis using the Masing's unloading/reloading rule

The stiffness reduction definition in time domain analysis is the same with the equivalent linear analysis method. The main problem in the fully nonlinear methods is the damping, since it is highly dependent on the unloading/reloading function. Several methods have been proposed so far for the unloading/reloading curve to account the damping more realistically (Pyke-1979, Darendeli-2001, and Groholski-2016)

4.3 Case study: Method Comparison for Site Response Analysis in a Deep Alluvial Soil Profile

In this section, the results of ground response analyzes performed in the time domain and frequency domain of a soil profile in the Mavişehir region are discussed. Mavişehir profile contains deep soft alluvial deposits. The average shear wave velocity in the first 30 meters is 151 m/s. According to TBDY-2018 earthquake code, it is ZF soil class. The Mavişehir soil profile analyzed is given in Figure 1.

The ground profile is 300 m and reaches the engineering bedrock. Since the bedrock depth is known in the profile, site response analyzes were performed by giving the outcrop records of the Samos 2020 earthquake directly from the profile base.

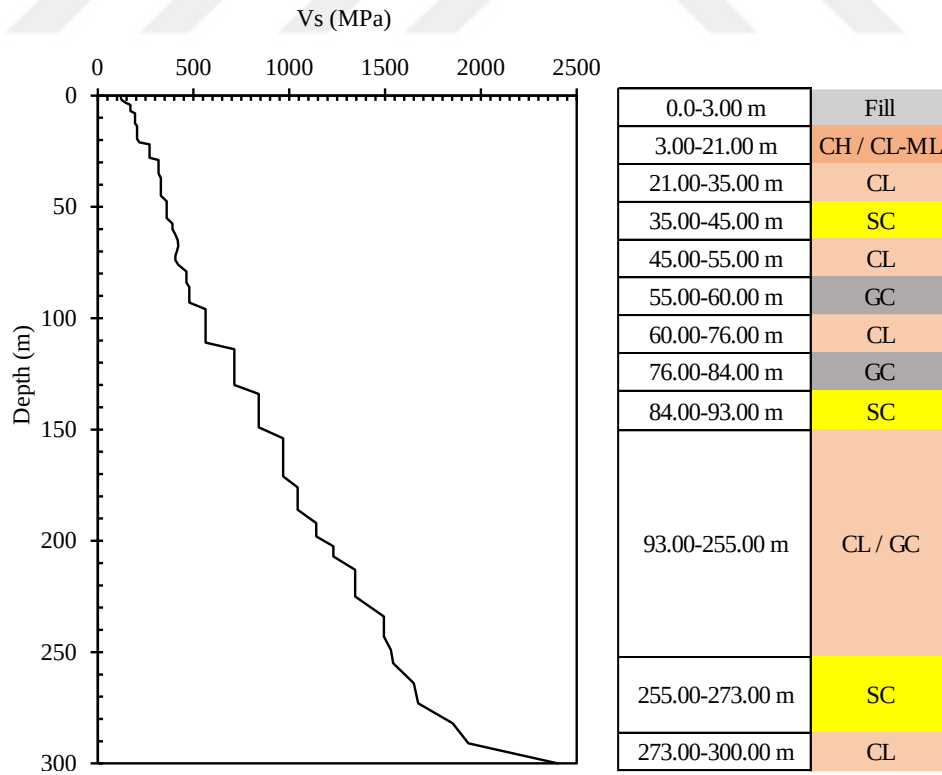


Figure 4.6 Soil profile data for Mavişehir region

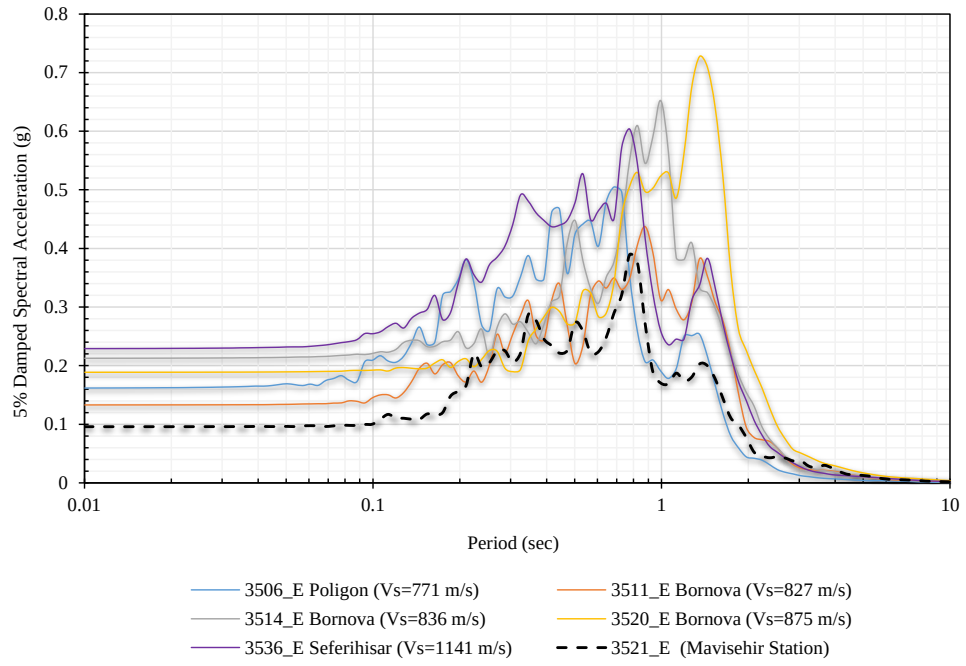


Figure 4.7 Equivalent linear analysis results in Mavisehir region (East direction)

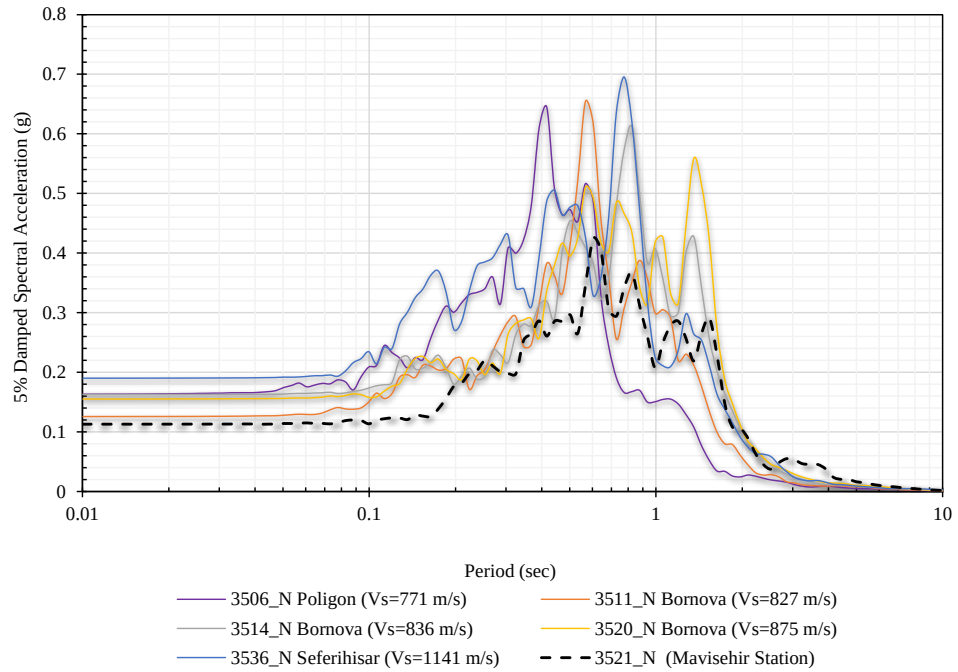


Figure 4.8 Equivalent linear analysis results in Mavisehir region (North direction)

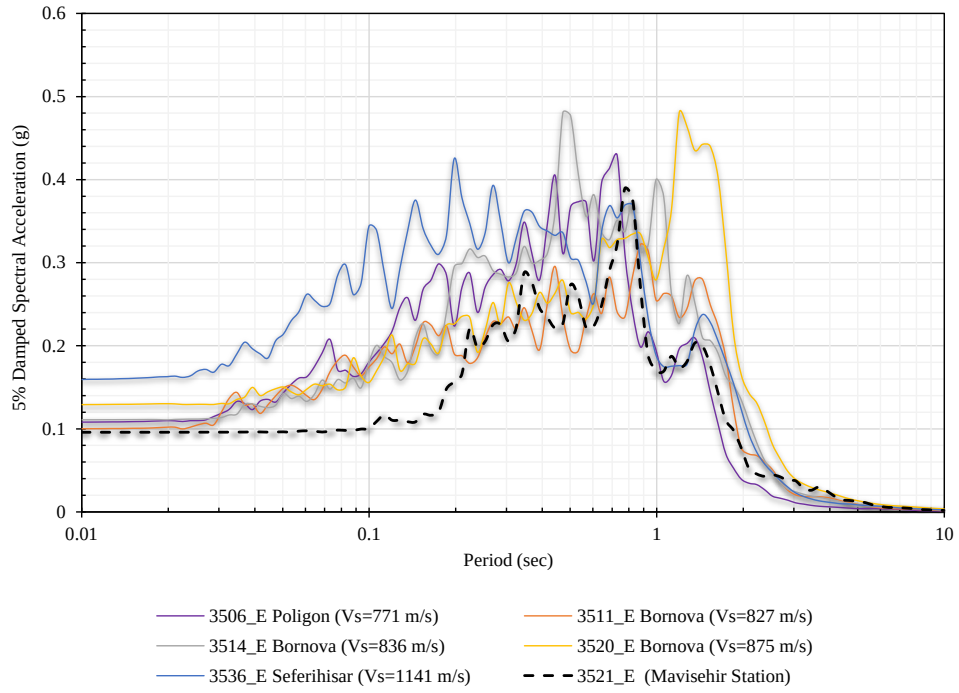


Figure 4.9 Total nonlinear analysis results in Mavisehir region (East direction)

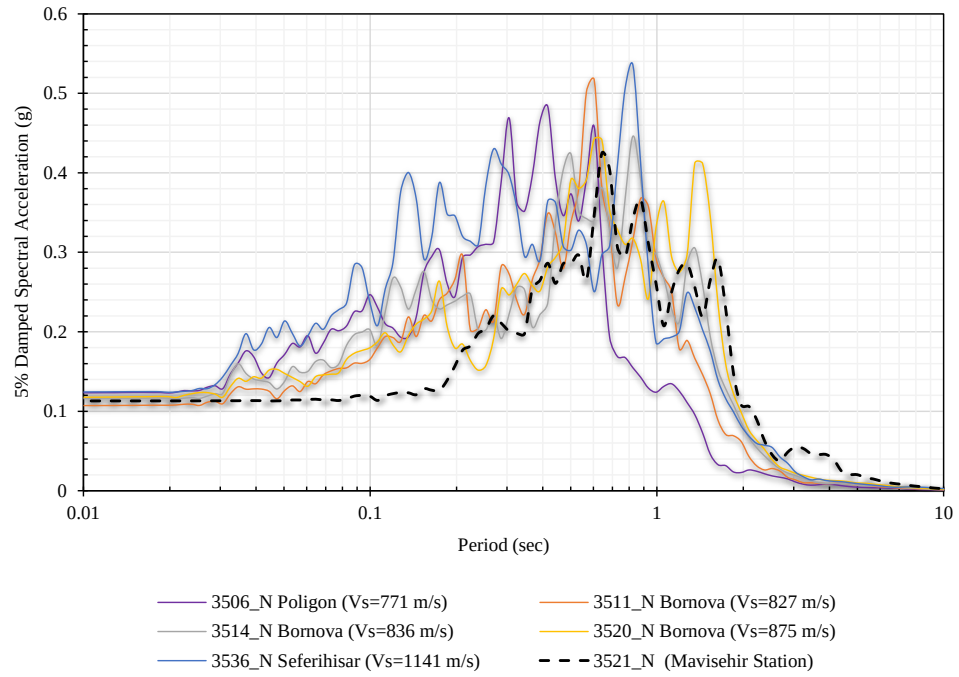


Figure 4.10 Total nonlinear analysis results in Mavisehir region (North direction)

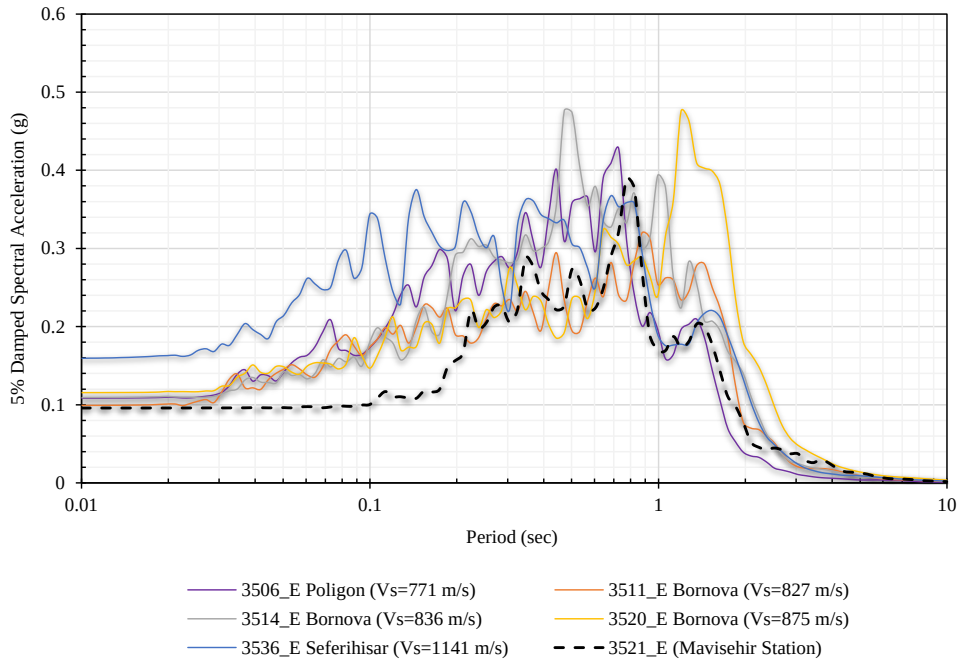


Figure 4.11 Effective stress analysis results in Mavisehir region (East direction)

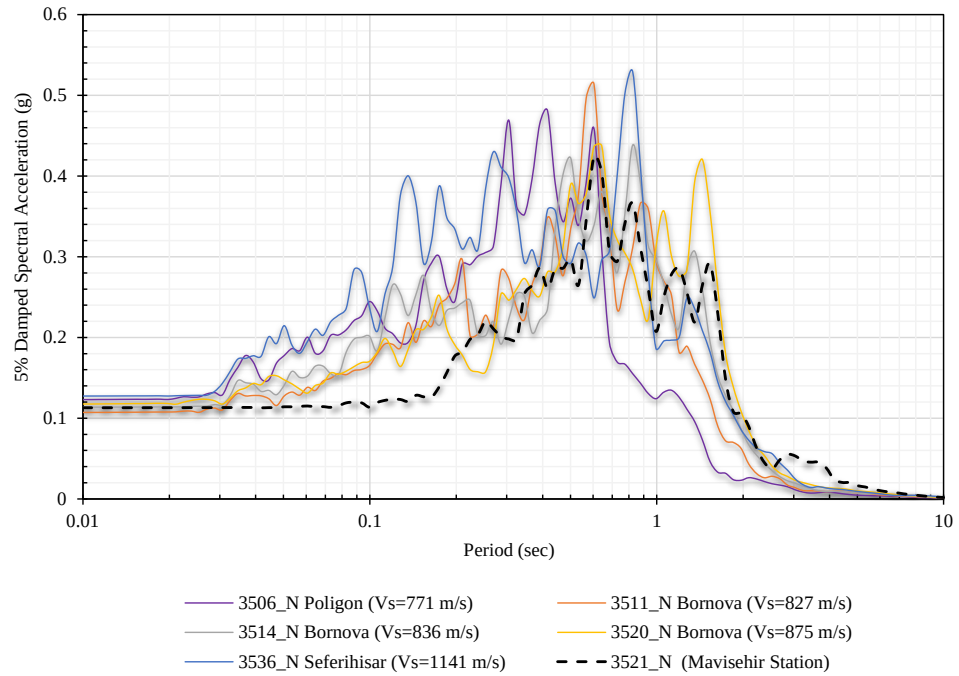


Figure 4.12 Effective stress analysis results in Mavisehir region (North direction)

The PGA value of the Samos earthquake measured at the Mavişehir strong motion station is 0.096 g in the E direction and 0.113 g in the N direction. Considering both components of the earthquake, the maximum spectral accelerations dictate different periods. The maximum spectral acceleration in the E component of the station was 0.389 g, and the maximum spectral acceleration in the N component was 0.425 g. These accelerations occurred in 0.77 and 0.60 sec periods, respectively. In the E direction, 4 distinct peaks, one of which is maximum, are observed between the 0.1-1.0 s periods, and a 0.203 g peak occurs in the 1.35 s period. In the N direction, there are 2 distinct peaks, one of which is maximum, between the 0.1-1.0 s periods, 0.285 g peaks in the 1.19 s period and 0.291 g peaks in the 1.53 s periods. The natural period of the field is 3.33s. and Mavisehir station recording especially in N direction field resonance period amplification is observed.

Equivalent linear analysis results are shown in Figure 1 and Figure 2. When the analyzes in both directions are compared, it is seen that the peak ground accelerations are higher in the equivalent linear analyzes than in the nonlinear analyzes. However, in the equivalent linear analysis, the spectral magnitudes in the high frequency region move steadily up to 0.1s and are unresponsive, but after 0.03 s in nonlinear analyses, more magnification occurs compared to the equivalent linear analysis. Rathje and Kottke (2011) stated that due to the inconsistency in the ground motion phase produced by the nonlinear stress-strain response, it will be subjected to greater magnification for periods of less than 0.04 seconds than would occur in the equivalent linear analysis. They predicted that between 0.04 s and 0.2, when the nonlinear analysis has stress reversal, more magnification will occur than the equivalent linear analysis due to the sudden stiffness change of the soil. Relevant opinions are consistent with the analysis results.

Between the 0.2 s and 1.0 period, spectral magnitudes are larger in equivalent linear analysis, whereas spectral magnitudes are damped in nonlinear analysis. Here, the real non-linear behavior of the soil creates the difference between the analyses. It is expected

that two different analysis methods will give closer results in stiff soil profiles with small strain deformations or in low-magnitude input earthquakes.

According to Kim (2016), equivalent linear coordinates will exceed nonlinear analysis due to equivalent linear analysis that creates a stronger resonance response associated with time-invariant soil properties near the resonance field frequency. The natural period of the soil column is 2.95 s and after the 1 s region, misleading resonance effects are seen in the equivalent linear analysis. The damping effect is seen in the nonlinear analysis.



CHAPTER FIVE

CASE STUDY: SEISMIC RESPONSE OF THE ALLUVIAL AND DEEP SOFT SOIL DEPOSITS IN İZMİR

5.1 Introduction

An earthquake with a magnitude of M_w : 6.9 occurred on October 30, 2020, at 14:51 (TSİ), with its epicenter in the Aegean Sea, between the offshores of Seferihisar (İzmir) and Samos (Samos) Island.

According to the information coming from national and international seismology centers after the earthquake, the magnitude of the earthquake according to KOERI (Kandilli Observatory and Earthquake Research Institute) is M_w : 6.9, $Z=10$ km; According to GFZ (GeoForschungs Zentrum, Helmholtz-Zentrum Postdam Deutsches) M_w : 7.0, $Z=15$ km; According to AFAD (Disaster and Emergency Management Presidency Earthquake Department), M_w : 6.6 $Z=16$ km; According to OCA (GéoAzur, Université de Nice Sophia-Antipolis, Valbonne, France), M_w is 7.2 $Z=10$ km. The main shock is 24.00 km away from Doğanbey Payamlı village of Seferihisar district of İzmir province, which is the closest settlement of the earthquake. At the same time, Seferihisar-Teos, Bornova and Bayraklı districts of İzmir province are approximately 40 km, 65 km and 70 km away, respectively. Samos earthquake was felt in İstanbul, especially in İzmir, Manisa Aydın, Muğla, Denizli. From the main shock to 11.11.2020, nearly 3000 aftershocks with magnitudes ranging from 1.0 to 5.3 were recorded (AFAD, 2020; EMSC, 2020).

According to the data of the main shock and aftershocks that occurred, the rupture of the fault was concentrated on the North Samos Fault, which is tectonically important with an approximately E-W trending normal fault, and the other aftershocks occurred near the

junction points of the Karliovası Fault and Tuzla Fault in the sea. The intensity of the earthquake in the settlement was calculated as MMI (Modified Mercalli Index) VII.

In the earthquake, 17 buildings were damaged at the level of destruction. In this section, a building that caused loss of life in Bayraklı region was selected and the building was evaluated in terms of soil-structure interaction.

5.2 Approach to Building Soil-Structure Interaction Assessment

The Bornova Plain was formed by the alluvium brought by the streams with short seasonal flow reaching the plain from the surrounding high masses, filling this depression. Among these streams, Manda Stream and Kocaçay are the largest streams flowing in the Bornova Plain and pouring into the Izmir Bay. Regardless of the epicenter of the earthquake, mostra records close to the study area are very important for the realism of the site-specific soil behavior analysis that should be done in the first step.

It is very difficult to predict the earthquake motion on the ground surface in areas where the engineering rock cannot be reached. Starting from this point, ground motion records measured on the ground surface were tried to be estimated by a series of deconvolution processes. The relevant results were compared with analyzes using mostra records.

5.3 Deconvolution of Surface Motions

In order to create site-specific design spectra, ground motions must be predicted. One-dimensional ground response analysis is a preferred method for the determination of earthquake-induced forces in terms of handling the problem in a simple way.

Ground motions need to be estimated to generate site-specific design spectra. One-dimensional ground response analysis is a preferred method for determining earthquake-

induced forces. Free field ground motion is obtained when SH waves propagating vertically from the source due to fault rupture pass through a semi-infinite space where all boundaries are horizontal and reach the surface. Conversely, in deconvolution analysis, the surface motion is reduced to a control point at a specified depth by passing through the soil layers. The reason for applying deconvolution analysis is to reach the bedrock motion in that region and thus to estimate the ground motion in the free field. Ground motion estimation can also be made through different earthquake attenuation relationships, taking into account the epicenter distance on the basis of spectral magnitudes. However, it is important to make a near-accurate estimation of bedrock motion specific to an earthquake, to understand the site-specific soil behavior, and to interpret the forces transmitted to the structure and the design spectra. It is a common approach in the literature to use outcropping motions for input motion in ground response analysis. However, an outcrop record is intended to use to estimate the ground motion, it must represent the site of interest. For example, if we do not perform a multi-dimensional analysis, it will be difficult for us to interpret the surface motions that occur due to the basin effect in large plains. In one-dimensional ground response analysis, which is especially important in terms of engineering practice, the uncertainties that may arise from the irregular field stratigraphy can be reduced by a near-accurate estimation of the ground motion and the accurate description of the soil properties by field tests (Electric Power Research Institute [EPRI],1986).

Silva (1988) tried to estimate the surface motion records of earthquakes of equivalent magnitude at low strain and high strain levels in soft alluvial soil profiles using a series of equivalent-linear deconvolution analysis results and examined their coherency with the observed records. Based on the positive results of the coherency analyses, a series of deconvolution processes have been proposed.

The areas where the mentioned process is applied are alluvial deposits, soft soil deposits and profiles with very deep engineering bedrock. In this section, the same

analysis steps are applied for the soil profile of the Camdibi strong ground motion station, which is an alluvial soil profile. The steps followed are presented below:

1. Low-pass filtering was applied at 15 Hz and scaled 0.87 times, surface motion recorded at Camdibi strong ground motion station.
2. The motion obtained in step 1 was reduced to a layer determined as the control point below the surface by equivalent deconvolution analysis.
3. As a result of iteration, shear modulus reduction ($G/G_{\max}$) and damping ratios (β) of each layer were obtained.
4. Linear deconvolution analysis was applied using the degraded soil data obtained in the 3rd step. For this analysis, motion recorded at a strong ground motion station at 15 Hz was applied low-pass filtering, not scaling.

Silva (1988) applied coherence analysis to alluvial deposited soil profiles with depths varying between 40-50 meters and profile base shear wave velocities between 500 m/s and 800 m/s. Theoretically propagated motions were compared with those obtained from the vertical array data. Silva realized that 75% of maximum power were the normally propagated shear waves. He stated that the remaining 25% might be the scattered waves or P-waves. Square root of power spectrum corresponds to acceleration response spectrum. hence the vertically propagated shear waves can be obtained with multiplying the amplitude by 86%.

Silva (1988) stated “The response spectra of the unfiltered deconvolved motion generally falls below that of the observed motion and follows the oscillations fairly well out to about 12-15Hz. Beyond this range, the deconvolved motion begins to diverge from the observed. Based upon results of the coherence analysis, this behavior is expected and indicates that filtering the surface motion at 15Hz prior to deconvolving reduced high frequency contamination at depth.”

Equivalent linear deconvolution analyzes were performed in DEEPSOIL V.7 software and linear deconvolution analyzes were performed in EERA software.

5.4 Camdibi Sites for Deconvolution

The Samos earthquake, which occurred on October 30, 2020 and whose epicenter was in Izmir, was recorded by the Camdibi health center station with the code TK 3522 and the surface motion records were used in deconvolution analyses. Camdibi station was chosen as a validation site to better understand the surface movement in Bayraklı and Manavkuyu regions, as will be seen later in the study. It has a thick alluvial ground profile that also represents Bayraklı and Manavkuyu region. Low plasticity clay, clayey sand layers with fine content exceeding 20% and clayey gravel layers are observed. The groundwater level is at 13 meters for the Camdibi profile, lower than that of the Bayraklı region. The average shear wave velocity for the first 30 meters is 258.2 m/s. According to the Turkish building seismic code 2018, the site class is ZD. TDY classification in these profiles complies with NEHRP and EuroCode-8 classes. It corresponds to class D in NEHRP and class C in EuroCode-8. The soil profile has data up to 40 meters deep and the engineering rock was inaccessible. It is seen that the shear wave velocity at 40 meters is 525 m/s. The engineering bedrock could not be reached and ended with a ZC soil class profile.

In the equivalent linear deconvolution analysis, the free field motion is the motion at the surface of the soil deposit. Bedrock motion is the motion at the base of the soil deposit. The outcropping motion is the motion that occurs where the rock exits the ground surface. Therefore, rock outcropping motion is not affected by soil amplification. In the deconvolution analysis, the x and y components of the strong ground motion station TK 3522 coded Camdibi Health Center (Camdibi Sağlık Ocağı) were used as the free field motion.

The shear wave velocity varying with depth along the idealized soil profile and the calculated maximum shear modulus values are presented in Figure 5.2. It is seen that there are low velocity zones in the soil profile at some depths. The SPT numbers and shear wave velocity values increase at the depths greater than 23 m due to the presence of gravel content.

For this analysis applied in the EERA software, the soil material was defined for each layer in the idealized soil profile. Normalized shear modulus reduction curves and small-strain material damping curves are produced according to Darendeli's curve-fitting approaches considering soil plasticity, over-consolidation ratio, loading frequency and number of loading cycle.



Figure 5.1 Camdibi Station idealized soil profile

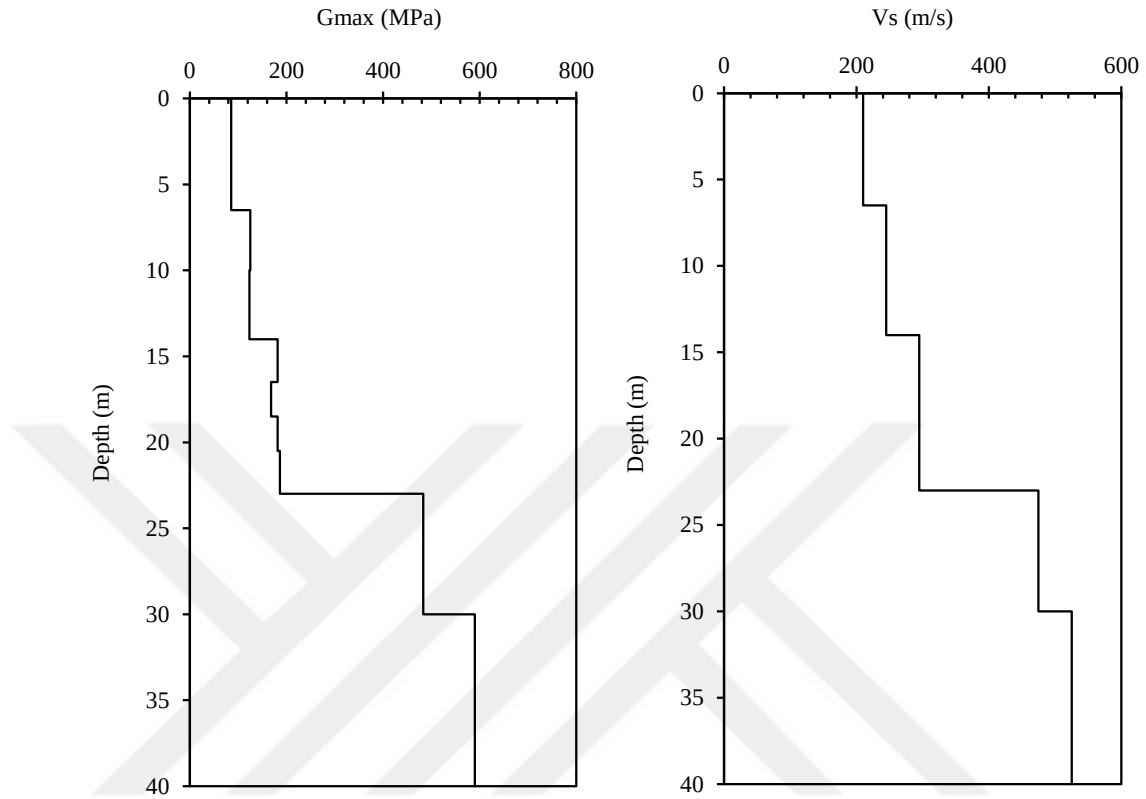


Figure 5.2. Maximum shear modulus and shear wave velocity profile for Camdibi Station

5.4.1 Equivalent Linear Deconvolution Analysis

The input motion to be used in the deconvolution procedure to be applied for the Camdibi station has been modified by filtering and scaling (step 1). The step 2 and step 3 part of the deconvolution procedure given in section 5.3 describe the equivalent linear deconvolution analysis. The closeness of the degradation curves for sand and clay soil is shown in Figure 5.3. Three sand and one clay soil material were chosen to simplify the analysis. Although this idealization was made by considering depth and plasticity indices, care was taken to select curves with different nonlinearity.

As it is known, the effect of effective stress increase on the nonlinear behavior of the soil in sandy soils is greater than in clay soils. When the curves of the selected materials are examined, it can be said that the sand profiles show more discrete behavior from each

other. Since the plasticity indices of the clay layers in the idealized profile are close to each other, it is seen that the curves of the clay materials almost overlap. In Figure 5.4, the normalized shear modulus curves of the 4 soil models are presented for the analysis case.

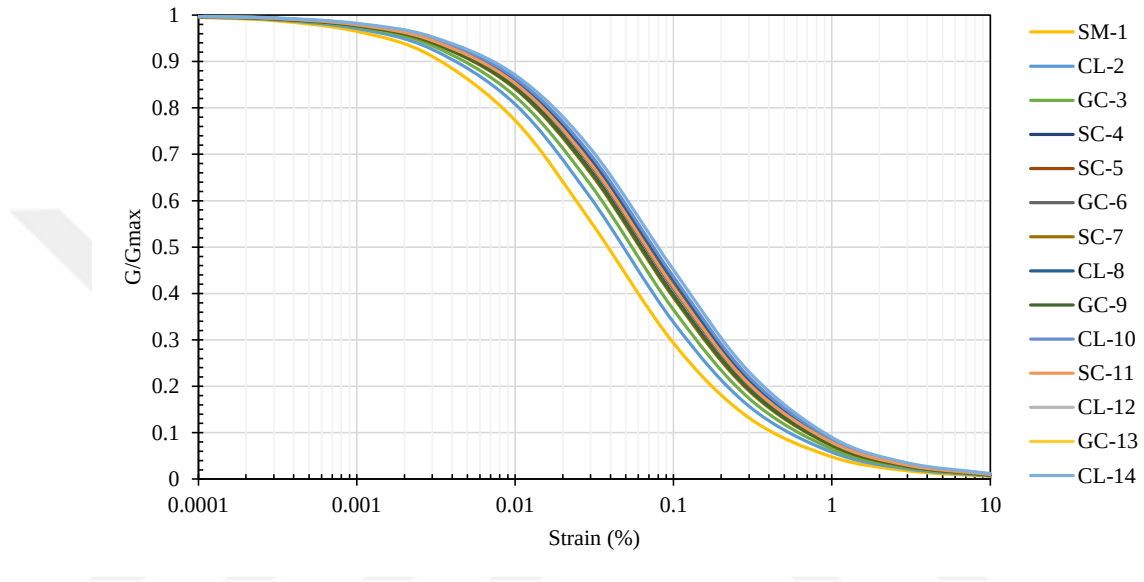


Figure 5.3 The modulus degradation curves of Darendeli (2001) for the equivalent deconvolution analysis

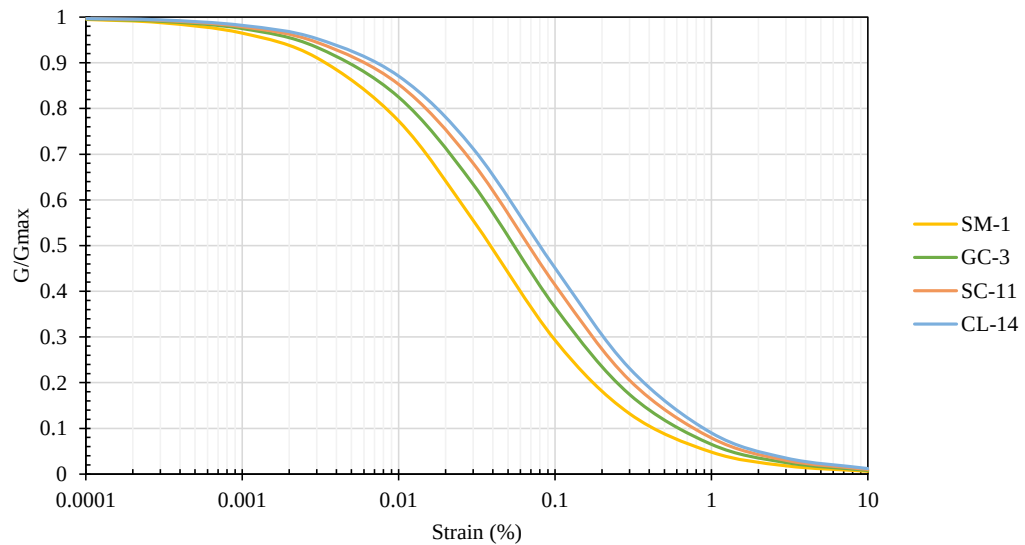


Figure 5.4 The modulus degradation curves used in the equivalent deconvolution analysis

Equivalent linear deconvolution was performed separately for the east-west and north-south directions of the earthquake recording. The analysis was completed with 8 iterations for each layer. The ratio of effective and maximum shear strain was taken as 0.6. The maximum shear strain, maximum stress, maximum acceleration, shear modulus reduction ratio and damping ratio were obtained for 40 meters deep Camdibi station soil profile.

The variation of the maximum shear strain and modulus reduction ratios obtained in the last iteration as a result of the analysis with depth is given in figure 5.5. Maximum shear strain occurred in the low-plastic clay layer (CL-8) located between 18.50 m and 20.50 m in the graphs. Similar behavior is observed in both components of the earthquake record. The maximum strain occurring in the north-south component is 0.016%. The shear modulus reduction ratio in the north-south component at the same depth was obtained as 0.76. When Figure 5.6 is examined, a maximum damping of 4.6% occurs in the clay layer between 18.5 and 20.50 m in the north-south component.

Soil softening and damping increase are compatible with the increase of shear strain. This low plasticity clay layer between clayey sand (SC-7) and clayey gravel layers (GC-9) falls between the stiffness contrast. For this clay layer, which is between cohesionless soils with much higher shear strength values, a decrease in strength occurs at strain levels of 0.016%.

Another dramatic damping and shear modulus reduction ratio variation occurs in the clayey sand layer (SC-4 and SC-5) at 10.00-14.00 between gravelly soil soils (GC-3 and GC-6). When figures 5.5 and 5.6 are examined, a decrease in strength occurs at 0.015% strain levels in the clayey sand layer.

The motion recording obtained as a result of the equivalent linear deconvolution analysis is not used in the other stages of the procedure. The analysis was carried out to obtain the damping ratios and modulus reduction ratios for the degraded soil.

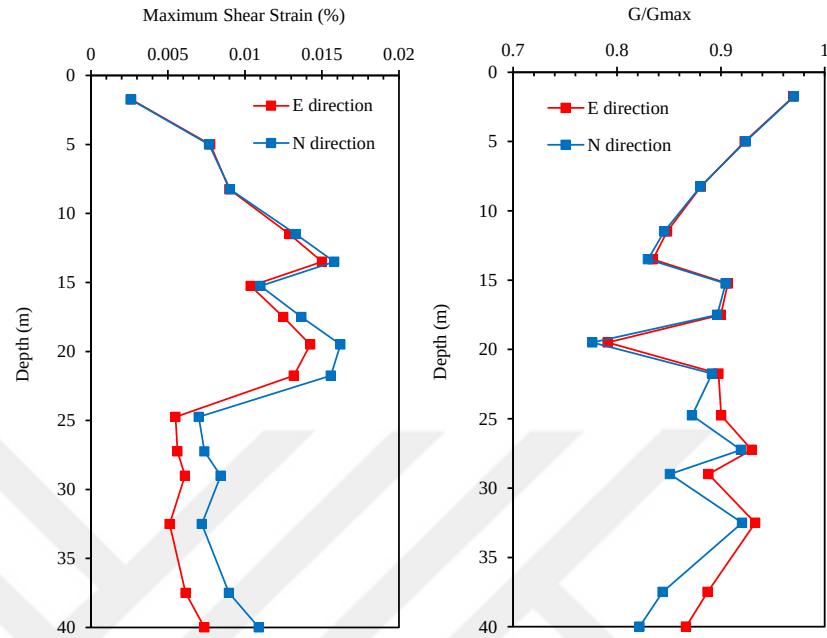


Figure 5.5 Maximum shear strain and G/Gmax profiles after the last iteration in deconvolution analysis

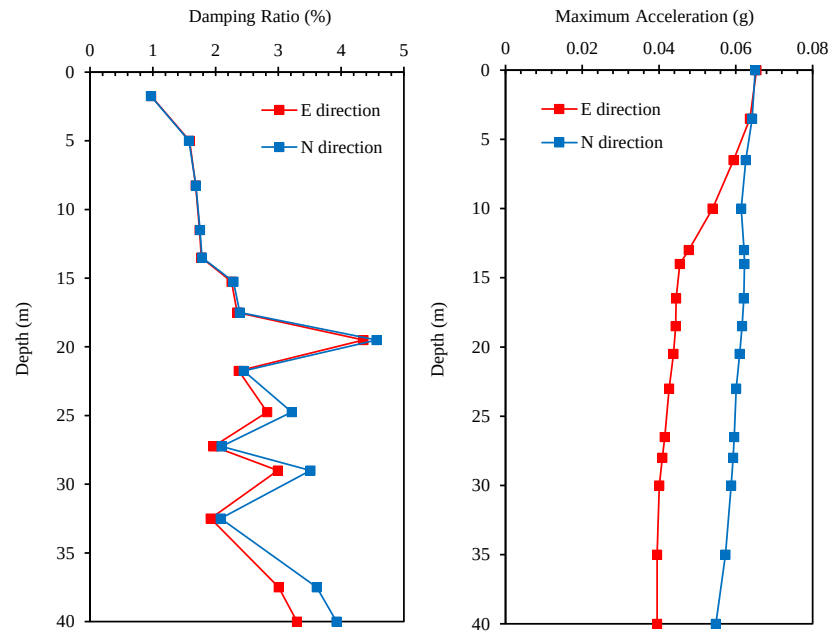


Figure 5.6 Damping ratio and maximum acceleration profiles after the last iteration in deconvolution analysis

5.4.2 Linear Deconvolution Analysis

The third stage of the deconvolution process is completed with the damping ratios and shear strength ratios obtained from the last iteration. Shear modulus ratios of the soils were recalculated according to the shear modulus reduction ratios of the degraded soil. Soil softening due to cyclic loading in the soil is taken into account by obtaining a new shear wave velocity for each layer.

The shear modulus reduction ratio obtained for each soil layer was calculated as a result of the equivalent-linear deconvolution analyzes for the East-West and North-South components of the earthquake at Camdibi station. Thus, shear wave velocities were recalculated for the degraded soil layers.

According to the results given in Table 5.1 and Table 5.2, depending on the maximum shear modulus obtained as a result of the deconvolution analysis, the shear wave velocities of the disturbed soil decrease between 2% and 12%. In addition, it is seen that the gravelly clay (CL-8) layer between 18.50-20.50 m, where the maximum shear modulus reduction ratio decreases abruptly, has the highest damping ratio.

Linear deconvolution was performed with the new shear wave velocity values obtained as a result of the equivalent-linear deconvolution analysis and the minimum damping values obtained as a result of the iteration.

For this analysis, motion recorded at a strong ground motion station at 15 Hz was applied low-pass filtering, not scaling.

Table 5.1 Equivalent linear deconvolution analysis results for East-West direction

Sublayer	Middle of Layer (m)	Maximum Strain (%)	$G/G_{\max}$	Damping (%)	$G_{\max}$ (MPa)	Shear Modulus (MPa)	γ (kN/m ³)	V_s (m/s)
SM-1	1.75	0.0026	0.970	0.97	85.41	82.88	19.00	207
CL-2	5	0.0077	0.924	1.58	85.41	78.91	19.00	202
GC-3	8.25	0.0090	0.880	1.68	125.43	110.43	20.50	230
SC-4	11.5	0.0133	0.845	1.75	123.60	104.49	20.20	225
SC-5	13.5	0.0158	0.830	1.78	123.60	102.56	20.20	223
GC-6	15.25	0.0110	0.904	2.28	181.86	164.49	20.50	281
SC-7	17.5	0.0137	0.896	2.39	168.55	151.10	19.00	279
CL-8	19.5	0.0162	0.776	4.56	181.86	141.16	20.50	260
GC-9	21.75	0.0156	0.892	2.45	186.29	166.09	21.00	279
CL-10	24.75	0.0070	0.872	3.21	482.99	421.32	21.00	444
SC-11	27.25	0.0074	0.919	2.09	482.99	444.10	21.00	455
CL-12	29	0.0084	0.851	3.51	482.99	411.07	21.00	438
GC-13	32.5	0.0072	0.920	2.08	590.02	543.03	21.00	504
CL-14	37.5	0.0090	0.844	3.61	590.02	497.99	21.00	482

Table 5.2 Equivalent linear deconvolution analysis results for North-South direction

Sublayer	Middle of Layer (m)	Maximum Strain (%)	$G/G_{\max}$	Damping (%)	$G_{\max}$ (MPa)	Shear Modulus (MPa)	γ (kN/m ³)	V_s (m/s)
SM-1	1.5	0.0022	0.974	0.92	85.41	83.21	19	207
CL-2	4.75	0.0074	0.927	1.54	85.41	79.15	19	202
GC-3	8	0.0087	0.884	1.68	125.43	110.88	20.5	230
SC-4	11.25	0.0126	0.850	1.74	123.60	105.10	20.2	226
SC-5	13.5	0.0150	0.835	1.77	123.60	103.15	20.2	224
GC-6	15.25	0.0109	0.864	2.87	181.86	157.05	20.5	274
SC-7	17.5	0.0133	0.846	3.11	168.55	142.52	19	271
CL-8	19.5	0.0142	0.791	4.35	181.86	143.89	20.5	262
GC-9	21.75	0.0140	0.841	3.18	186.29	156.60	21	270
CL-10	24.75	0.0055	0.901	2.81	482.99	435.09	21	451
SC-11	27.25	0.0056	0.923	2.06	482.99	445.89	21	456
CL-12	29	0.0061	0.889	2.98	482.99	429.33	21	448
GC-13	32.5	0.0051	0.932	1.94	590.02	549.86	21	507
CL-14	37.5	0.0061	0.887	3.00	590.02	523.62	21	495

5.5 Estimation of Outcrop Motion

With a series of deconvolution processes, the outcropping motion was tried to be estimated by using the recorded surface motion at the Camdibi strong ground motion station in the Samos 2020 earthquake. The east-west component of the motion recorded at the Camdibi strong motion station in the Samos 2020 earthquake is given in figure 5.7. In Figure 5.8, the estimated outcropping motion is given.

For the east-west component of the earthquake, when the acceleration-time history of the camdibi station is examined, it is seen that the maximum peak acceleration is 0.058 g. In the estimated outcrop acceleration-time history, the maximum acceleration was obtained as 0.093 g. For the north-south component of the earthquake, when the acceleration-time series of the camdibi station is examined, it is seen that the maximum peak acceleration is 0.075 g. In the estimated outcrop acceleration-time series, the maximum acceleration was obtained as 0.085 g. The acceleration-time history for the north-south directions is shared in the appendices.

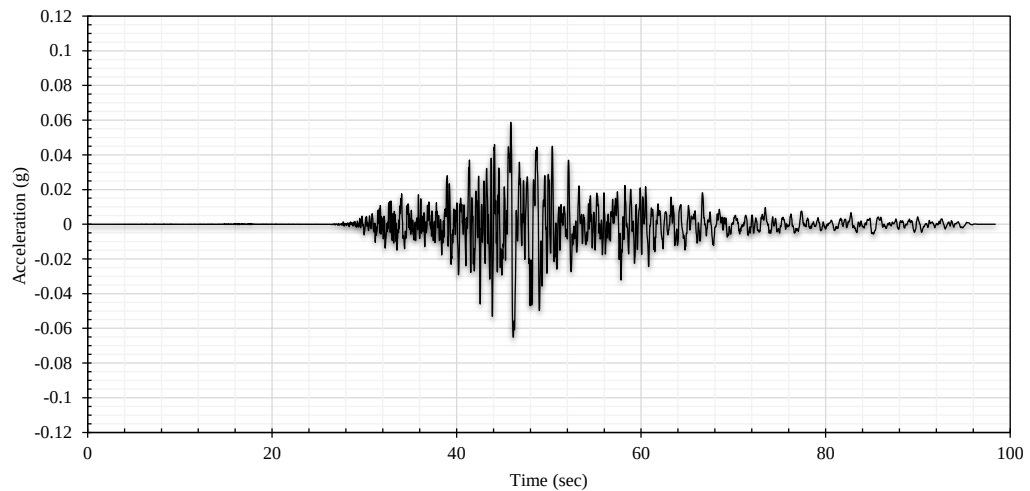


Figure 5.7 Strong ground motion record of Camdibi Station

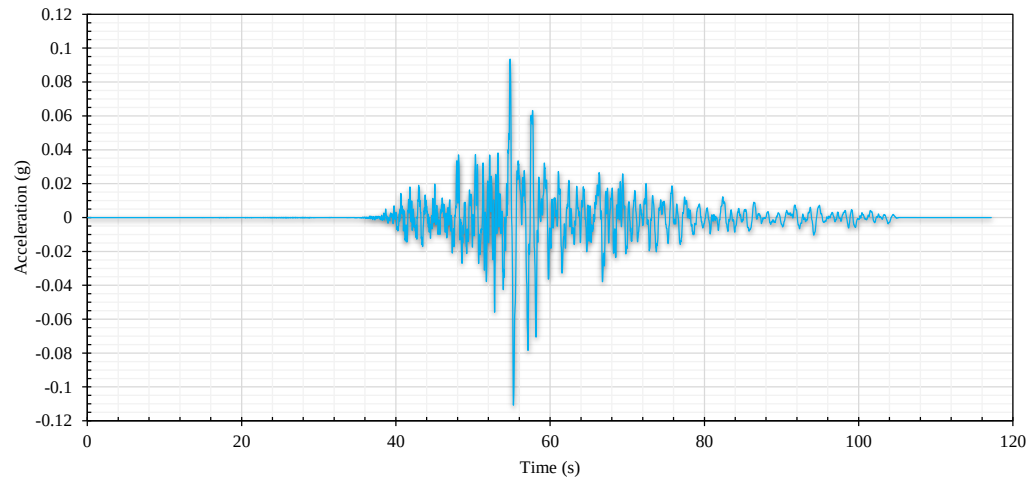


Figure 5.8 Outcrop motion obtained in the deconvolution analysis

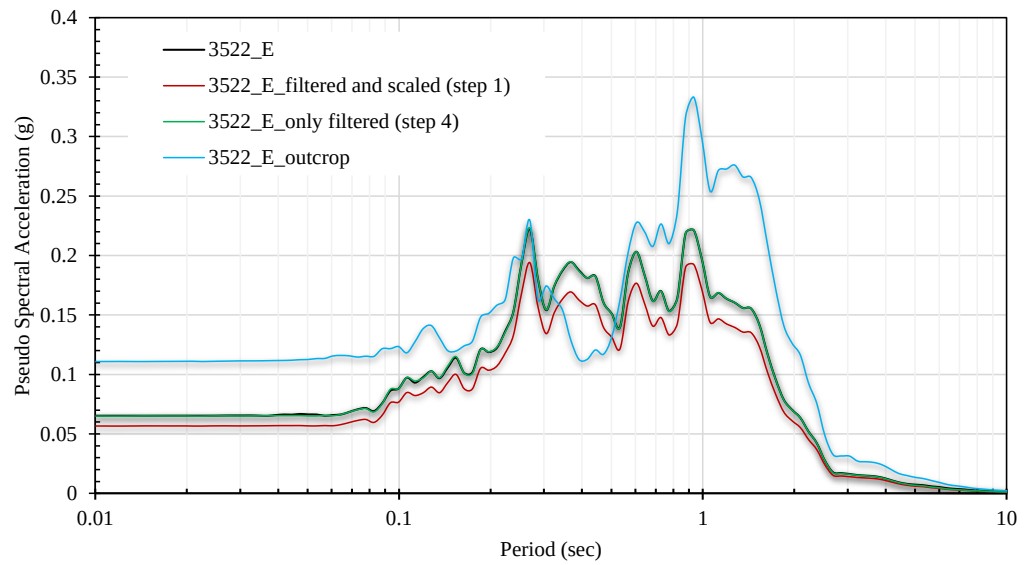


Figure 5.9 Variation of response spectra in the deconvolution process for North-South direction

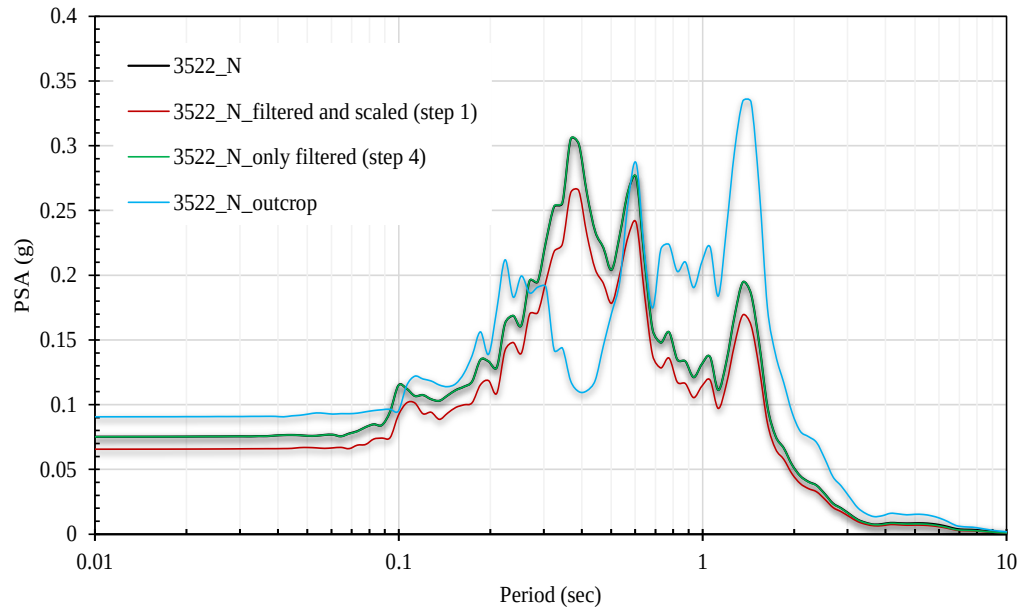


Figure 5.10 Variation of response spectra in the deconvolution process for East-West direction

With the result of a series of deconvolution processes applied in the Camdibi soil profile, the outcropping motion was tried to be estimated with the surface motion recorded at the strong ground motion station. The variation of the spectral amplitudes of the input and output motion during the deconvolution process is given in figures 5.9 and 5.10 for east-west and north-south directions, respectively.

The variation of spectral amplitudes occurring in the soil layers during the linear deconvolution analysis phase are given in figures 5.11 and 5.12 for east-west and north-south directions, respectively.

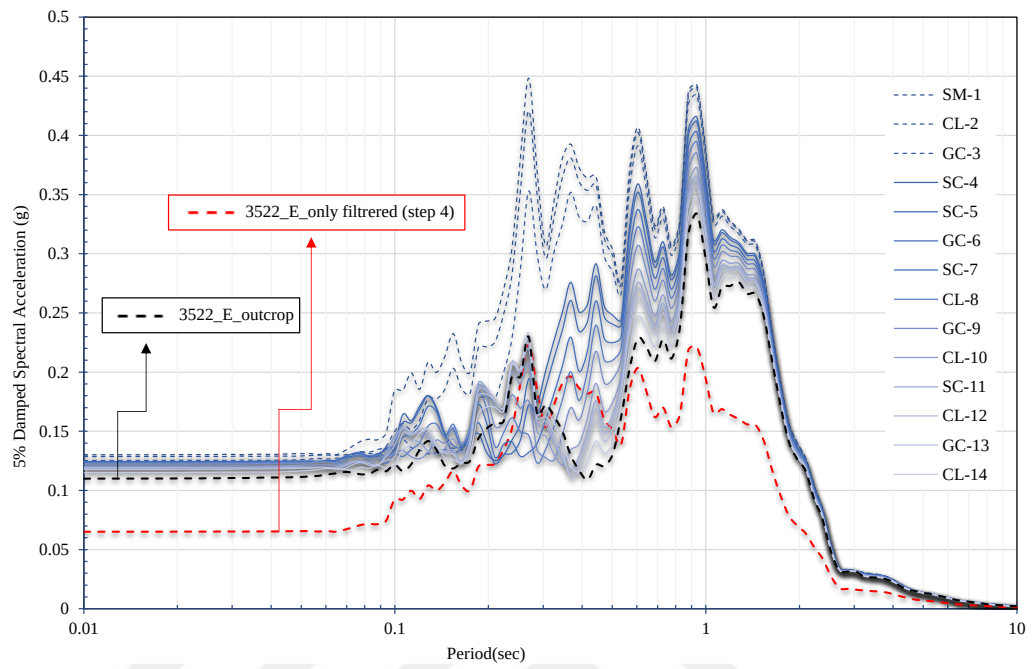


Figure 5.11 Linear deconvolution analysis results for East-West direction

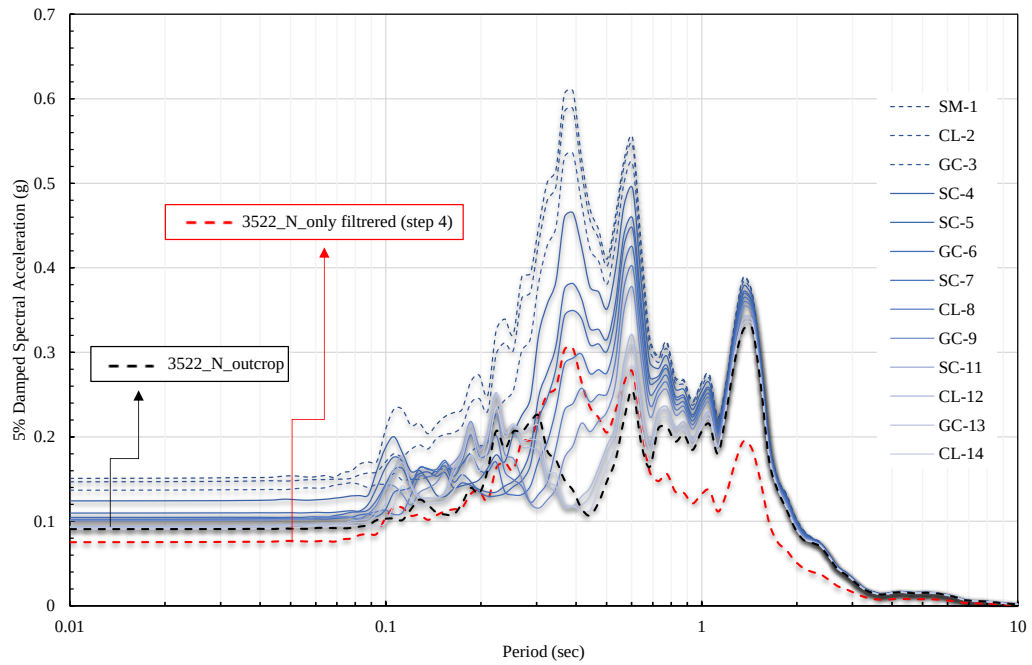


Figure 5.12 Linear deconvolution analysis results for North-South direction

5.6 Validation of the Estimated Ground Motion

Surface ground motion data recorded at Camdibi strong ground motion station were converted into outcropping motions to be used for both directions as a result of the deconvolution process. The suitability of the estimated data as a result of this series of analyzes was checked with the site response analysis performed at Bayraklı station. The results were compared with the data recorded at the Bayraklı station during the Samos 2020 earthquake.

The groundwater level is at 2.00 m and the average shear wave velocity is 185.9 m/s for the first 30. According to the Turkish building seismic code 2018, the site class is ZD. It corresponds to class D in NEHRP and class D in Eurocode-8. No engineering rock is encountered in the first 63.5 meters and the soil layer ends with ZC soil class. In Figure 5.13, some soil data of the relevant profile are plotted.

Effective stress-based non-linear site response analysis was performed in the deepsoil software and the MKZ soil model was used. Since the Camdibi profile, from which the estimated Outcropping motion record was obtained, ended at 525 m/s, the estimated motion from the layer with the same shear wave velocity was given to the Bayraklı profile. The results of the analysis were compared with the recorded at Bayraklı station.

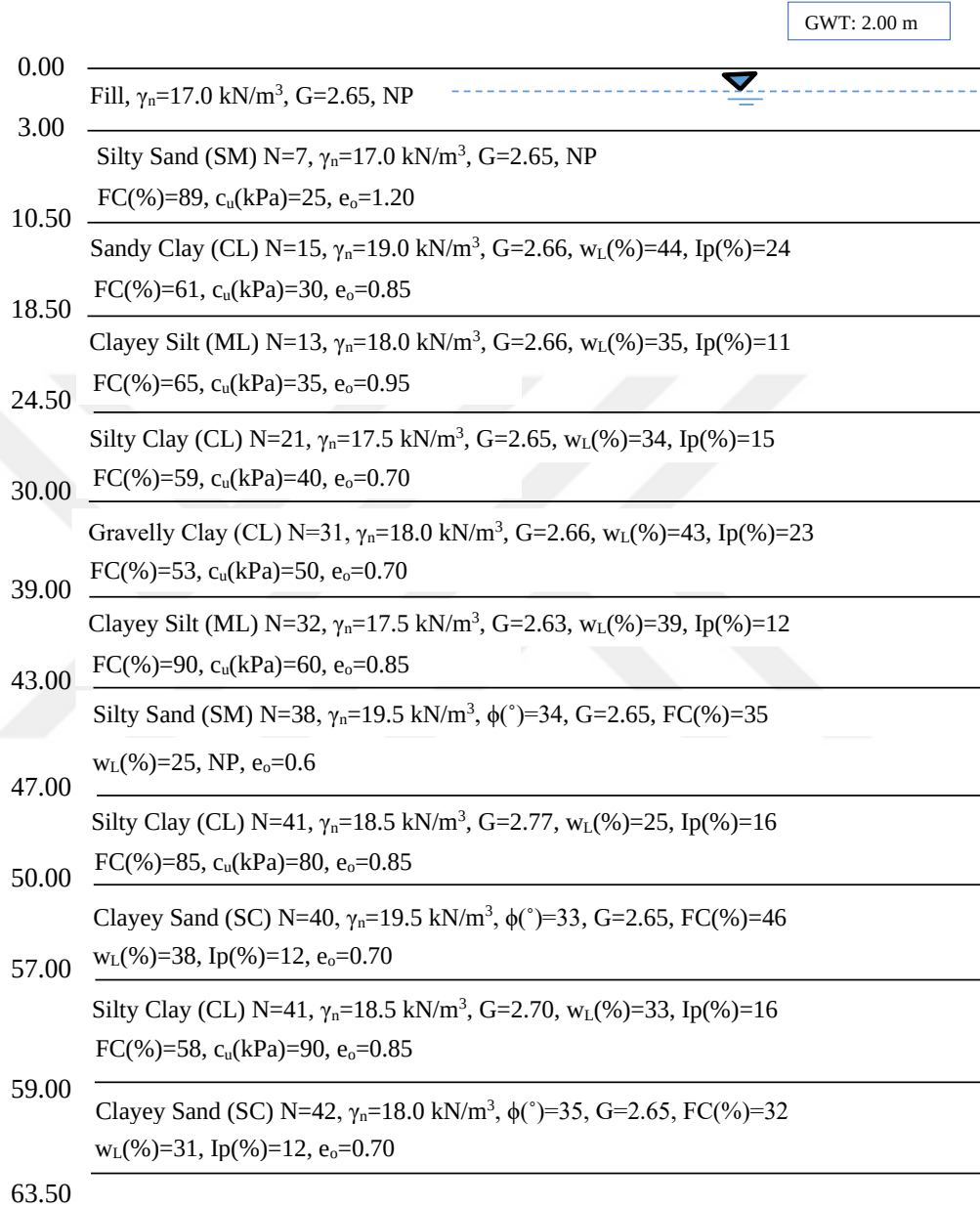


Figure 5.13 Idealized soil profile of Bayraklı strong ground motion station coded TK 3513

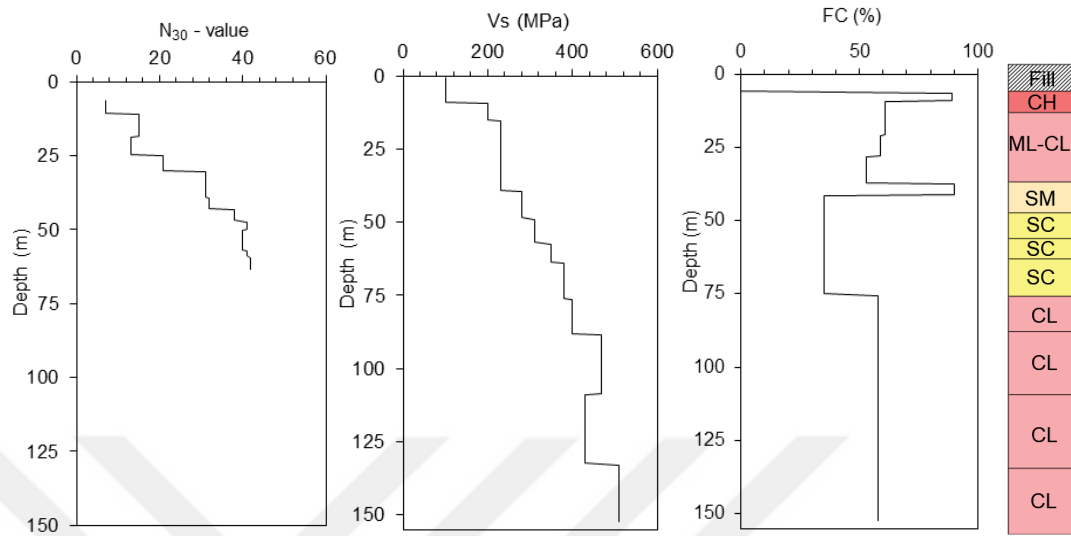


Figure 5.14 Soil profile data of Bayraklı strong ground motion station coded TK 3513

5.6.1 Comparison of the Estimated and Recorded Motion

Site response analyzes were performed on the Bayraklı strong ground motion station profile to test the validity of the predicted outcropping motion using the Camdibi soil profile. The results were compared with the surface motion recorded at Bayraklı strong ground motion station in the Samos 2020 earthquake. Analyzes were performed as effective stress-based nonlinear analyzes using MKZ and GQ/H soil models.

The results of the site response analysis performed with the GQ/H model are given in figures 5.15 and 5.16 for east-west and north-south directions, respectively. The PGA values obtained as a result of the analyzes are 10%-20% higher than the PGA values measured at Bayraklı station. The trend of spectral amplitudes was captured between the 0.5 s and 1.1 s periods. However, spectral amplifications are seen between the 0.1-0.3 sec period and in the high period region. The north-south and east-west directions of the earthquake show different spectral trends. Spectral characteristics specific to the earthquake direction were captured with the GQ/H model.

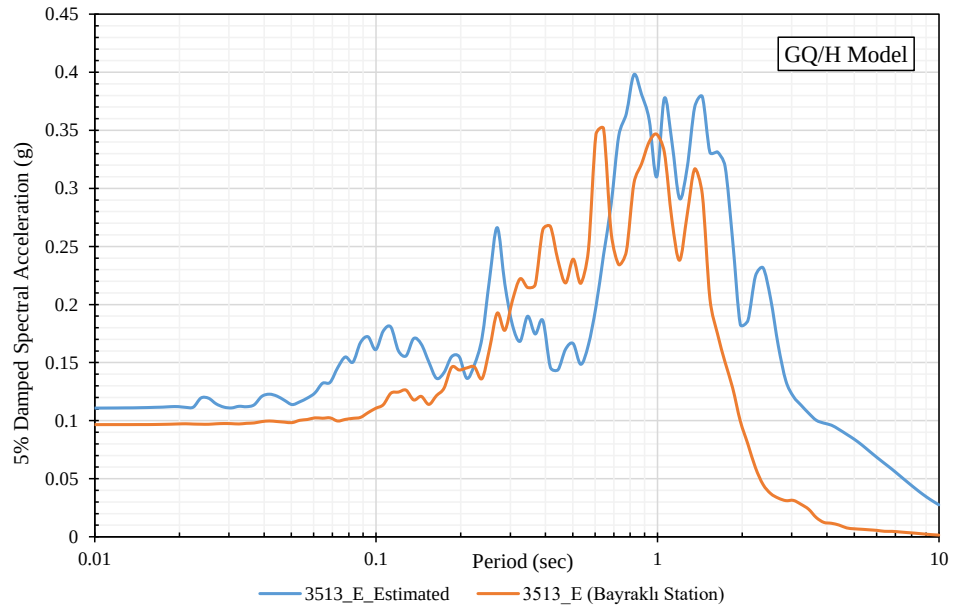


Figure 5.15 Comparison of the estimated and the recorded motion at Bayraklı station using GQ/H model (East-West direction)

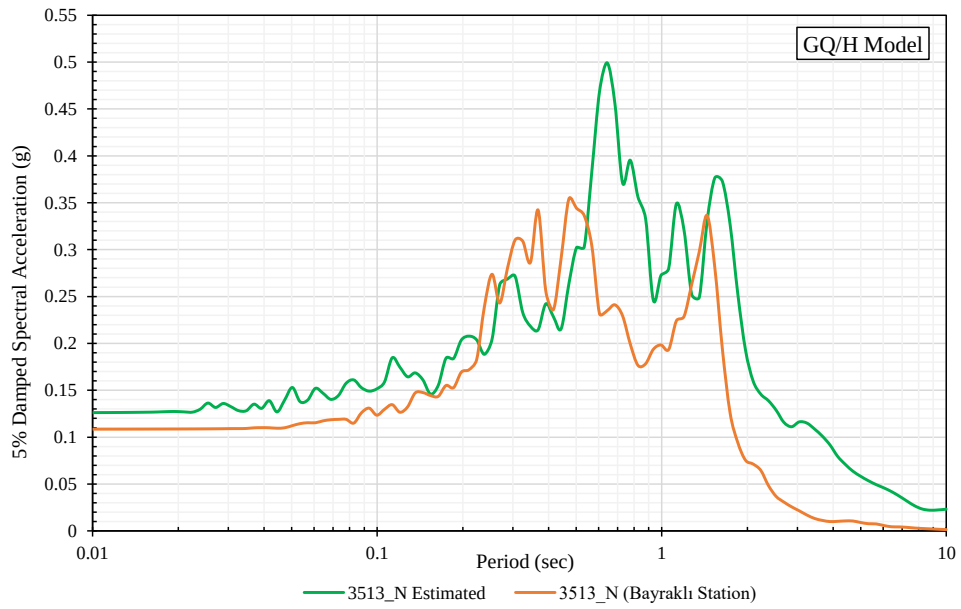


Figure 5.16 Comparison of the estimated and the recorded motion at Bayraklı station using MKZ model (North-South direction)

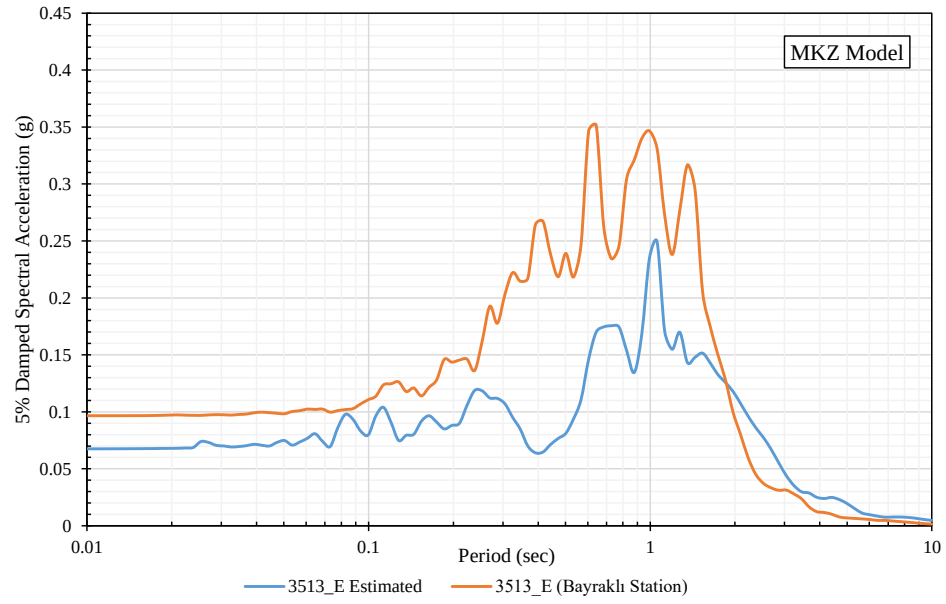


Figure 5.17 Comparison of the estimated and the recorded motion at Bayraklı station using MKZ model (East-West direction)

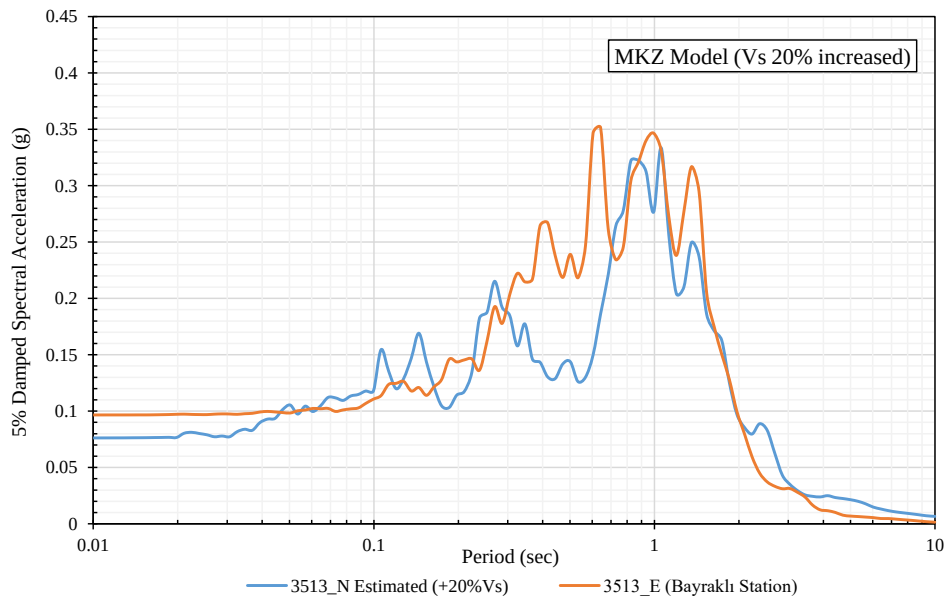


Figure 5.18 Comparison of the estimated and the recorded motion at Bayraklı station using MKZ model with increased Vs profile by 20% (East-West direction)

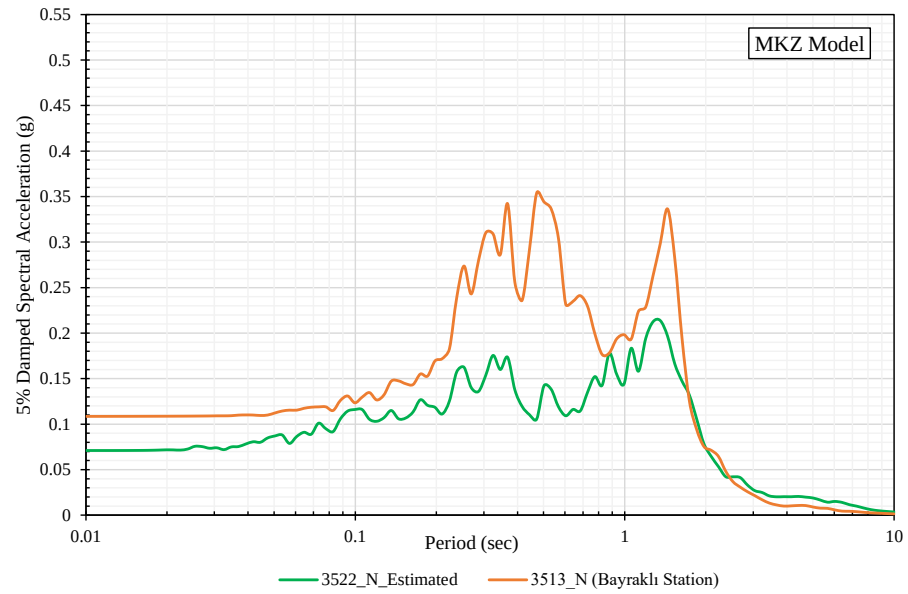


Figure 5.19 Comparison of the estimated and the recorded motion at Bayraklı station using MKZ model (North-South direction)

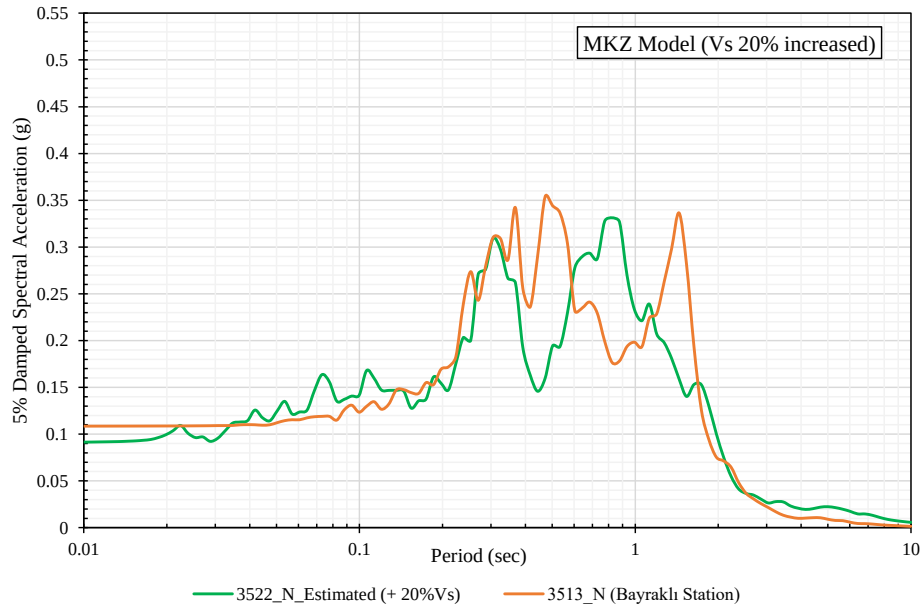


Figure 5.20 Comparison of the estimated and the recorded motion at Bayraklı station using MKZ model with increased Vs profile by 20% (North-South direction)

Spectral acceleration values recorded for the east-west and north-south directions of the earthquake during the Samos earthquake at Bayraklı station were compared with the estimated spectral acceleration values. In the non-linear analysis for the MKZ soil model, it was predicted that the shear wave velocity data could vary by $\pm 20\%$ and more consistent results were obtained with the shear wave velocities increased by 20%. As can be seen in figure 5.18 and figure 5.20, consistent results were obtained after 2 s period. On the other hand, filtering between 0.3-0.7 s draws attention and spectral accelerations could not be captured. Peak accelerations were achieved around the 1 second region.

Spectral characteristics specific to the earthquake direction were captured better with the GQ/H model compared to the MKZ model. According to figure 5.18, a single large peak appears between 0.8-1.0 sec periods for east-west direction in the MKZ model. Only two of the three peaks in the station record were captured, but the spectral accelerations of the peaks in the 1 sec region gave results close to the measured values. According to Figure 5.20, it is seen that the spectral trend is captured but there is a noticeable phase difference.

In the GQ/H model, the PGA values were estimated larger than those measured at Bayraklı station, while they were estimated smaller in the MKZ model. Although there are significant amplifications in the high period region in the GQ/H model, the spectral magnitudes in the MKZ model are compatible with the measured values.

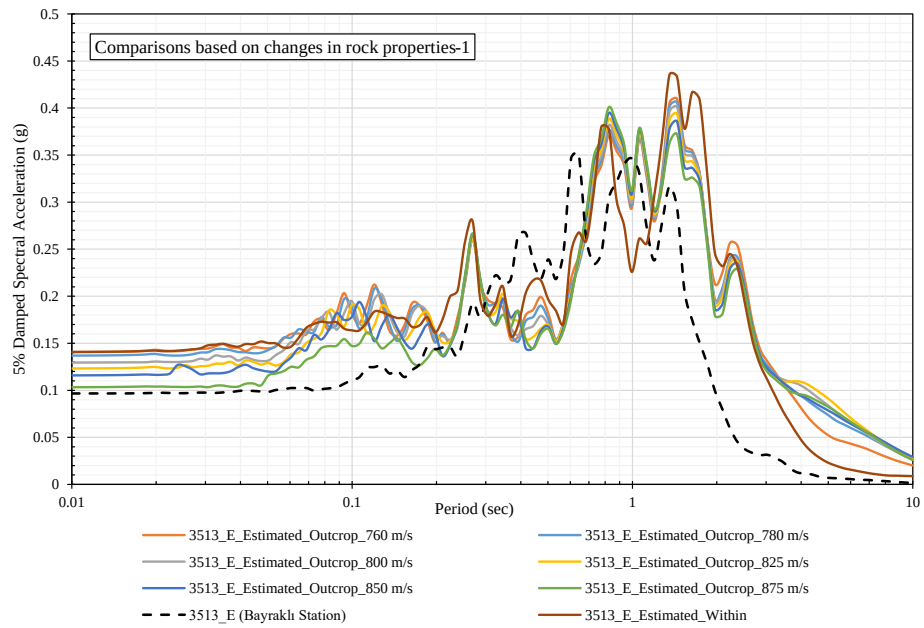


Figure 5.21 Comparison of elastic half-space and rigid half-space solutions in terms of rock properties

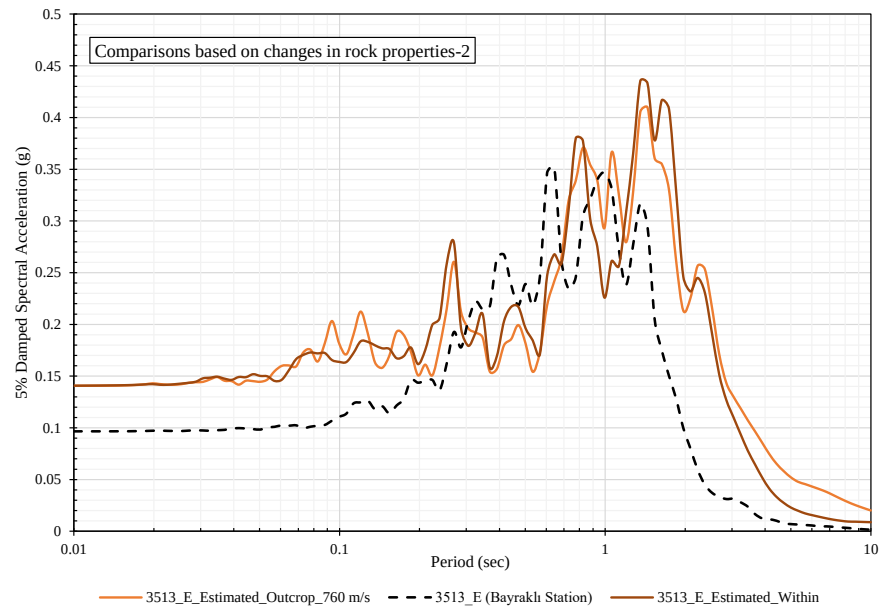


Figure 5.22 Comparison of elastic half-space and rigid half-space solutions

5.7 Case study: SSI Effect on a Nine Storey Building

Considering the compatible results obtained at Bayraklı strong ground motion station, site response analysis was performed for a building that collapsed in the 2020 Samos earthquake. In order to evaluate the analysis results, the building was considered as a single degree of freedom system, and the flexible-base foundation was accepted. While calculating the natural vibration period of the system, the rigidity of the system was calculated by taking into account the maximum displacement measured by the recorded motion at the Bayraklı station. With the assumption of flexible foundation, the horizontal, vertical and rotational stiffness of the foundation are calculated and the effects of soil flexibility are taken into account. Static stiffness of foundation Gazetas (1991); Mylonakis et al. (2006), dynamic stiffness were calculated using the relations of Pais and Kausel (1988).

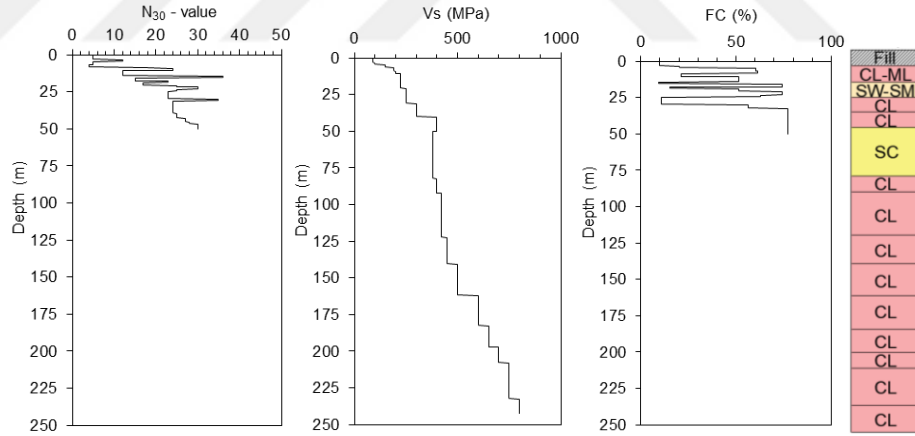


Figure 5.23 Soil profile data for the site of a collapsed building located in Bayrakli region

From this point of view, the period lengthening ratio of the system was calculated using Eq 5.1.

$$\frac{\tilde{T}}{T} = \sqrt{1 + \frac{k}{k_x} + \frac{k h^2}{k_{yy}}} \quad (5.1)$$

In addition, the period lengthening ratio, foundation damping and flexible-base foundation system damping were calculated by taking into account the degraded shear modulus and increased damping ratio for the 0.02% strain level, which is the maximum shear strain caused by the given input movement in the ground. Figure 5.24 shows the modulus reduction and the damping ratio curves with the expected shear strain level of 0.02%.

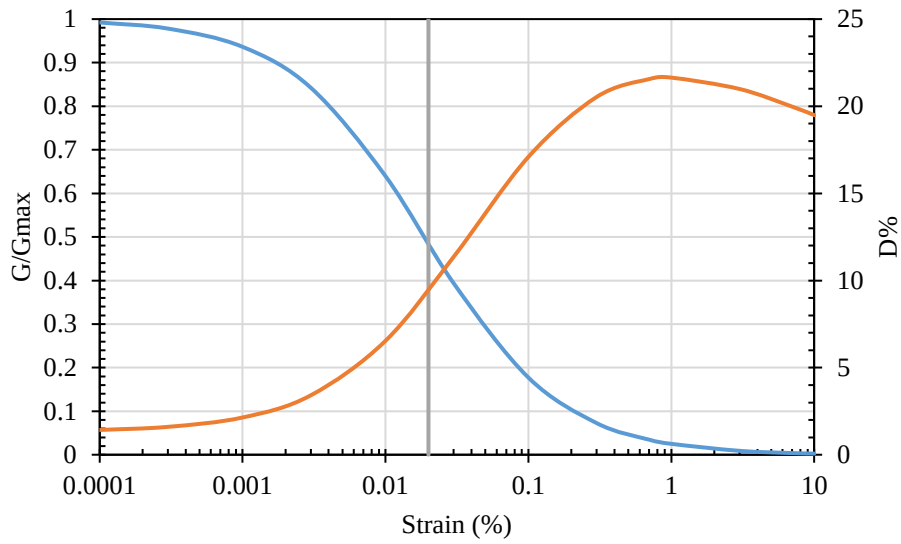


Figure 5.24 Shear modulus reduction and damping ratio curves

Equations 5.2 and 5.3 were employed to estimate the foundation damping and the damping of the system with the consideration of soil-structure interaction.

$$\beta_f = \left[\frac{\left(\frac{\tilde{T}}{\bar{T}}\right)^{n_s} - 1}{\left(\frac{\tilde{T}}{\bar{T}}\right)^{n_s}} \right] \beta_s + \frac{1}{\left(\frac{\tilde{T}}{\bar{T}_x}\right)^{n_x}} \beta_x + \frac{1}{\left(\frac{\tilde{T}}{\bar{T}_{yy}}\right)^{n_y}} \beta_{yy} \quad (5.2)$$

$$\beta_0 = \beta_f + \frac{1}{\left(\frac{\tilde{T}}{\bar{T}}\right)^n} \beta_i \quad (5.3)$$

The calculated values for the period lengthening and the variation of damping ratio due to the soil-structure interaction was summarized in Table 5.3.

Table 5.3 Soil structure interaction results

T (sec)	$\tilde{T}/T$	$\tilde{T}$ (sec)	β_s	β_i	β_f	β_o
0.84	1.65	1.38	0.089	0.050	0.061	0.072

Nonlinear ground response analysis was performed on the soil profile of a collapsed building with the estimated outcropping motions. The acceleration-response spectra obtained on the surface ground motion are presented in Fig. 5.25 for $D=5\%$, and the damping ratio increased as a result of soil-structure interaction $D=7.2\%$. In addition, spectral accelerations corresponding to $\tilde{T}=1.38$ s, increasing period due to the result of soil-structure interaction, are also shown. Accordingly, the spectral accelerations corresponding to the estimated elongated structure period were found to be 0.41g and 0.47g for the E and N directions, respectively.

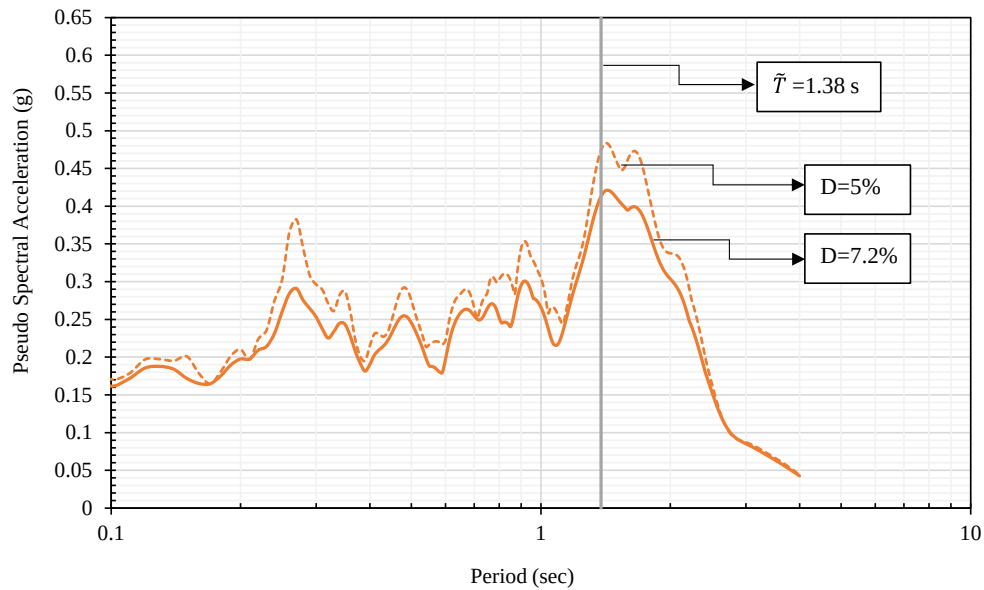


Figure 5.25 Acceleration response spectra for the surface motion obtained in the site (for East direction)

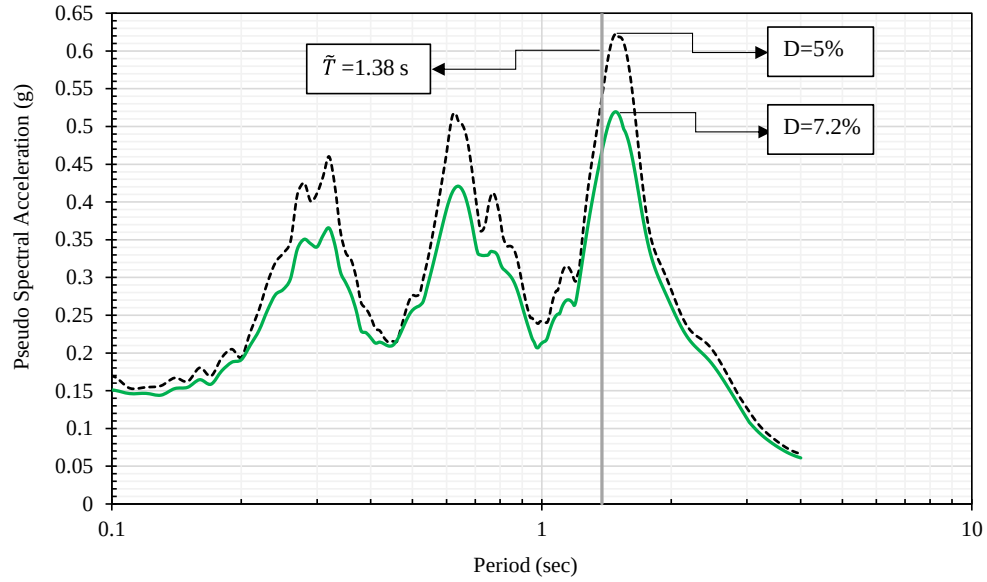


Figure 5.26 Acceleration response spectra for the surface motion obtained in the site (North direction)

In this section, the surface motion was estimated for the locations where there is no strong ground motion station in the Bayraklı region and which were destroyed during the earthquake. For this, the outcrop record was estimated by using the strong ground motion station record of the Bayraklı region, the engineering rock of which is at a shallower depth. This estimation was made by a series of deconvolution processes found in the literature (Silva, 1988). In addition, its validity was demonstrated by comparing it with the recorded surface motion at the Bayraklı strong ground motion station. From this point of view, ground-response analyzes were carried out in a ten storey building location in Bayraklı region. As a result of the ground response analysis, the accelerations that occur in the structure are estimated by considering the concept of structure-soil interaction. The horizontal design force corresponds to 10% of the building mass in the 1975 regulation to which the building is subject during construction. However, the predicted accelerations showed that the base shear force could be above the design value. This result showed the importance of the soil-structure interaction concept on a collapsed building.

CHAPTER SIX

SENSITIVITY OF DYNAMIC SITE RESPONSE ANALYSES TO EXCESS PORE PRESSURE GENERATION PARAMETERS

The purpose of this section is to show the effect of pore water pressure parameters on ground response analysis with free field analysis. For this purpose, ground-response analyzes were carried out in two sites with deep alluvial soil layers in the Bayraklı region. The shear wave velocities and soil profile of the two sites are given below.

Site response analyzes were performed using General Quadratic-Hyperbolic and Modified Kondner-Zelasko soil models to examine the variation of excess pore water pressure on the basis of soil model. As a result of the analysis, the normalized excess pore water pressure ratios were compared for the two models and the analyzes were continued with the GQ/H model in the subsequent analyzes. Matasovic (1992) model was used for pore water pressure parameters. The Matasovic (1992) pore water pressure generation parameters must be determined by a curve fitting procedure of cyclic undrained lab-test data. The following equation proposed by Matasovic and Vucetic (1993, 1995), can be used to determine the best-fit parameters to be used in the analysis.

$$u_N = \frac{p * f * N_c * F * (\gamma_c - \gamma_{tvp})^s}{1 + f * N_c * F * (\gamma_c - \gamma_{tvp})^s} \quad (6.1)$$

It is well known that the generation of excess pore-water pressure during dynamic loading causes degradation of the shear strength and shear stiffness of the soil.

$$\delta_G = \sqrt{1 - u^*} \quad (6.2)$$

$$\delta_\tau = 1 - (u^*)^v \quad (6.3)$$

Table 6.1 Pore water pressure generation parameters (Matasovic, 1992)

Variable	Description
u_N	Normalized excess pore pressure (r_u)
N_{eq}	Equivalent number of cycles
γ_c	The current reversal shear strain
γ_{tvp}	Threshold shear strain value
p	Curve fitting parameter
s	Curve fitting parameter
F	Curve fitting parameter
f	Dimensionality factor
s	Degradation parameter

Matasovic (1993) produced two constant functions to represent the degradation of the soil under cyclic loads in terms of shear modulus and shear strength. These two functions each correspond to a fixed degradation parameter. The u^* value is the excess pore pressure normalized to the effective stress. The value of v is an exponential curve fitting parameter defined to relate excess pore pressure and shear strength degradation parameter. The formulas given in equation 1 and equation 2 can be applied on other soils except clay soils. Equation 3 was proposed by Matasovic (1995) for clay soils.

$$\delta_G = \delta_\tau = N^{-1} \quad (6.4)$$

6.1 Effects of Soil Models

Effective stress based site response analysis was performed with the recordings of strong ground motion stations near the Bayraklı region, which recorded outcrop in the Samos 2020 Earthquake. The soil profile used for the analysis belongs to the site of a building

collapsed in the Samos earthquake. The soil profile data of the collapsed building site used for the analysis are given in figure 1.

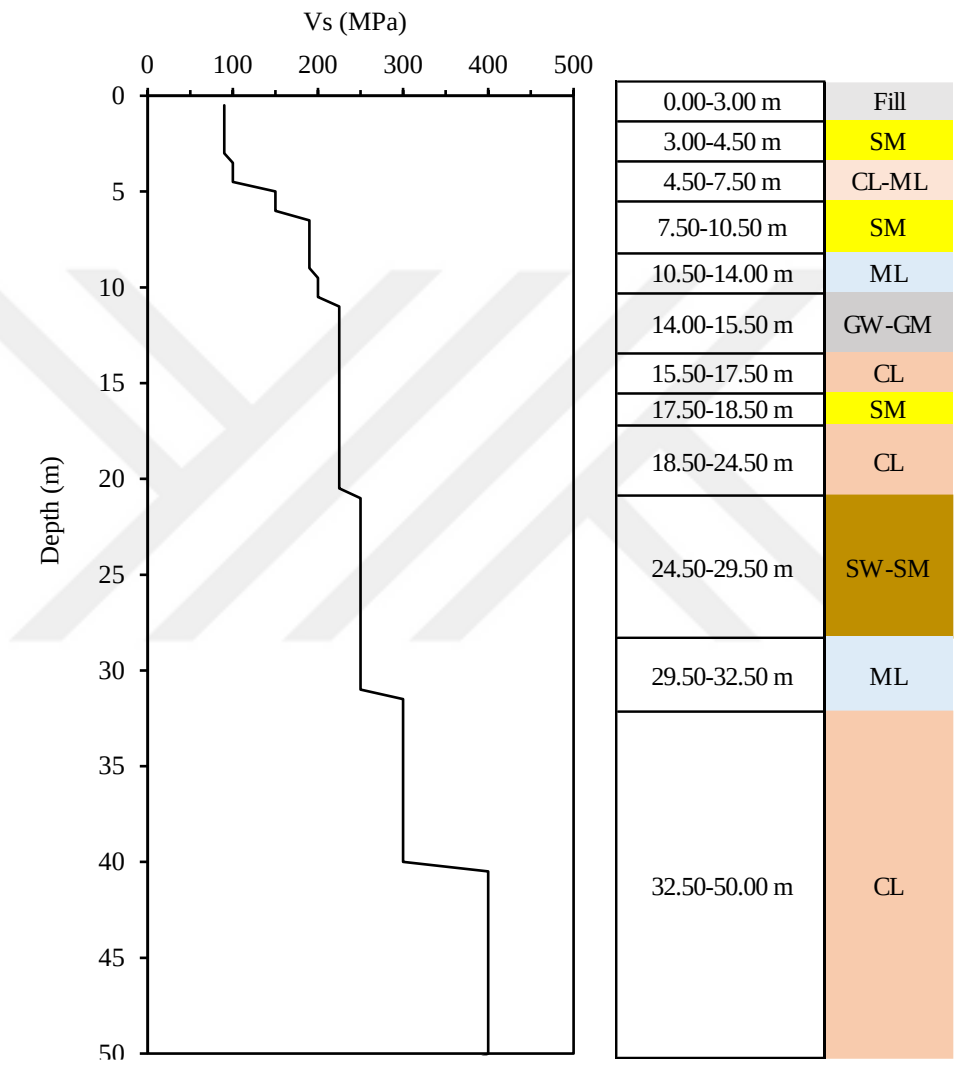


Figure 6.1 Damaged building site soil profile data for 50 m

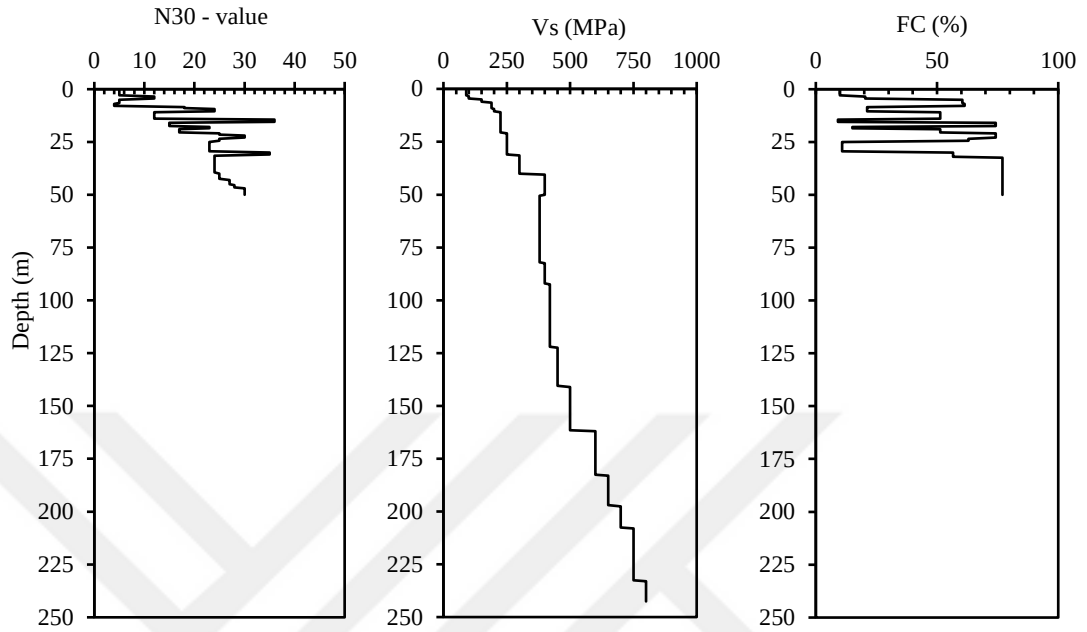


Figure 6.2 Damaged building site soil profile data

The variation of normalized pore water pressure with depth in the analyzes made in two different soil models is given in Figure 6.3, Figure 6.4 and Figure 6.5. In the sandy silt (SM) layer between 3.00-4.50 / 7.50-10.50 / 17.5-18.50 m, low plasticity silt (ML) between 10.50-14.00 m and well-graded silty sand (SW-SM) between 24.50-29.50 m, pore water pressure increase can be observed. It can be said that the pore water pressure development is more triggered in the analyzes made on the records of the TK-3514 and TK-3520 stations. However, since the normalized excess pore pressure values are quite low, liquefaction triggering is not possible. Therefore, it can be said that the Samos 2020 earthquake records, which are between DD-3 and DD-4 earthquake levels, do not trigger the development of excessive pore water pressure.

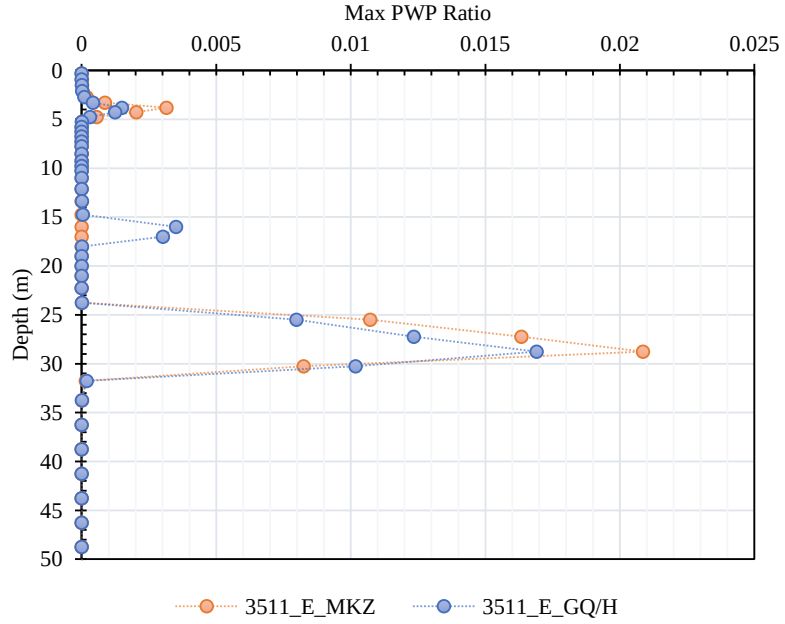


Figure 6.3 PWP results at the damaged building site under the Samos Earthquake (TK-3511 record)

Among the records used in the analysis, the most excessive pore pressure development occurred in the TK-3520 station record examined in Figure 6.5. It is seen that the MKZ model gives higher pore water pressure values than the GQ/H model, except for the low plasticity silt (ML) layer between 10.50-14.00 m.

In Figures 6.6 and Figure 6.7, the 1995 Kobe earthquake records were used for site response analyzes in the site where the collapsed buildings were located. For the records corresponding to the DD-2 earthquake level, pore water pressure generation was examined. The normalized pore water pressure ratio (r_u) exceeds 0.8 in the SM layer between 7.50-10.50 m and in the SW-SM layer between 24.50-29.50 m. In the mentioned SW-SM layer, both soil models have reached the same maximum r_u value.

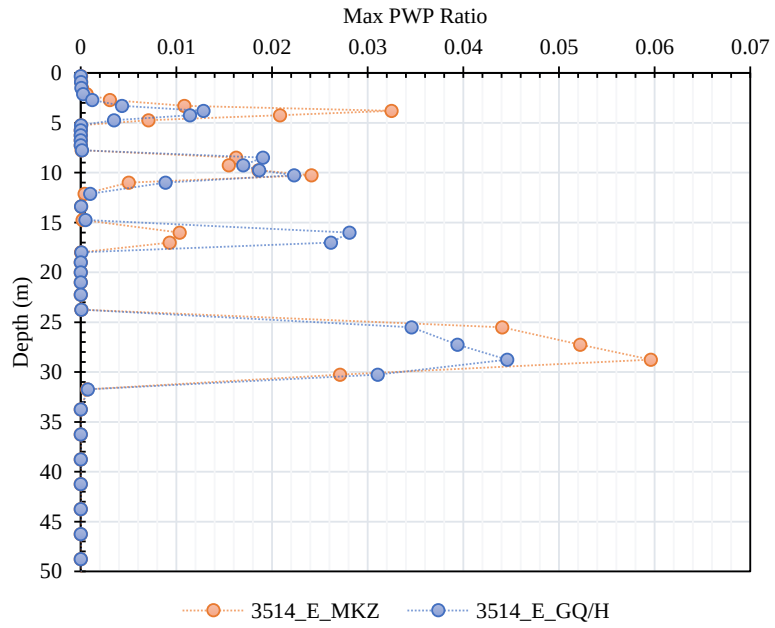


Figure 6.4 PWP results at the damaged building site under the Samos Earthquake (TK-3514 record)

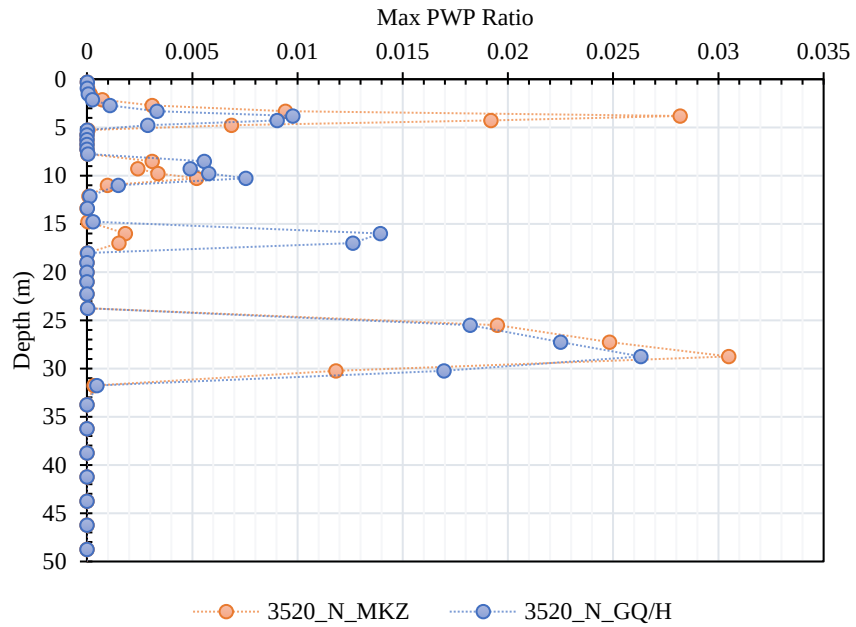


Figure 6.5 PWP results at the damaged building site under the Samos Earthquake (TK-3520 record)

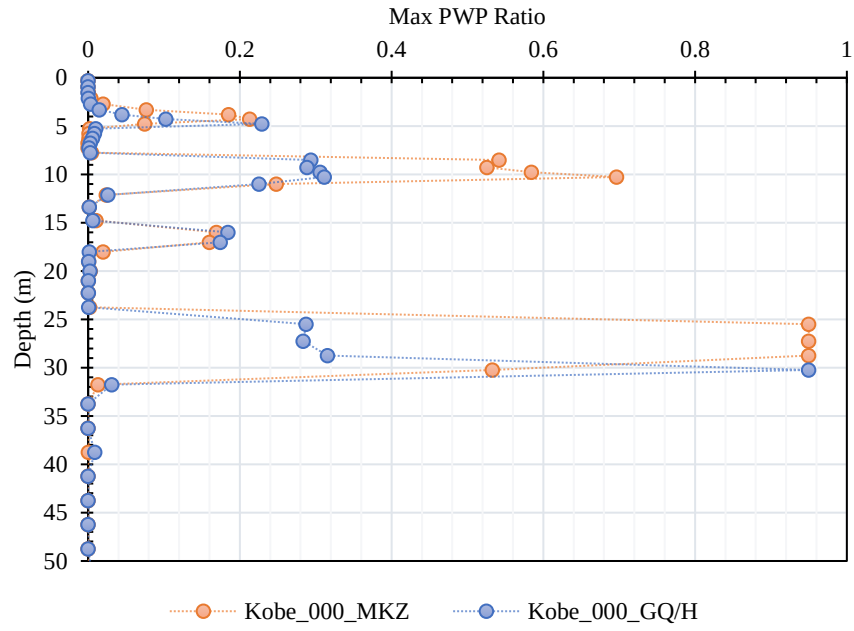


Figure 6.6 PWP results at the collapsed building site under the Kobe Earthquake (000 direction)

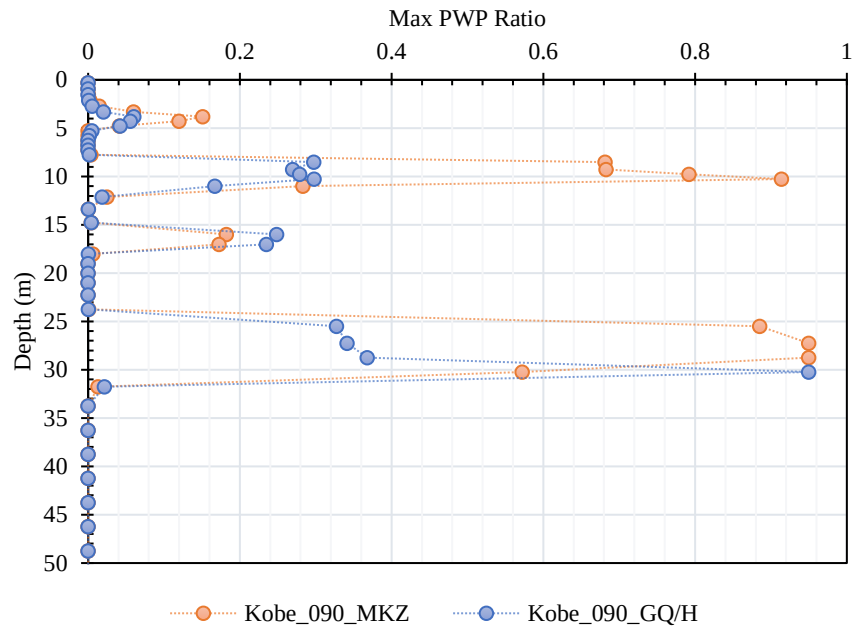


Figure 6.7 PWP results at the collapsed building site under the Kobe Earthquake (090 direction)

6.2 Effect of Pore Water Pressure Parameters

The soil profile of a deep alluvial soil profile in the Bayraklı region is given in figure 6.8. Total stress and effective stress-based nonlinear ground response analysis was performed and the variation of strain level and PWP with depth is given in figure 6.9.

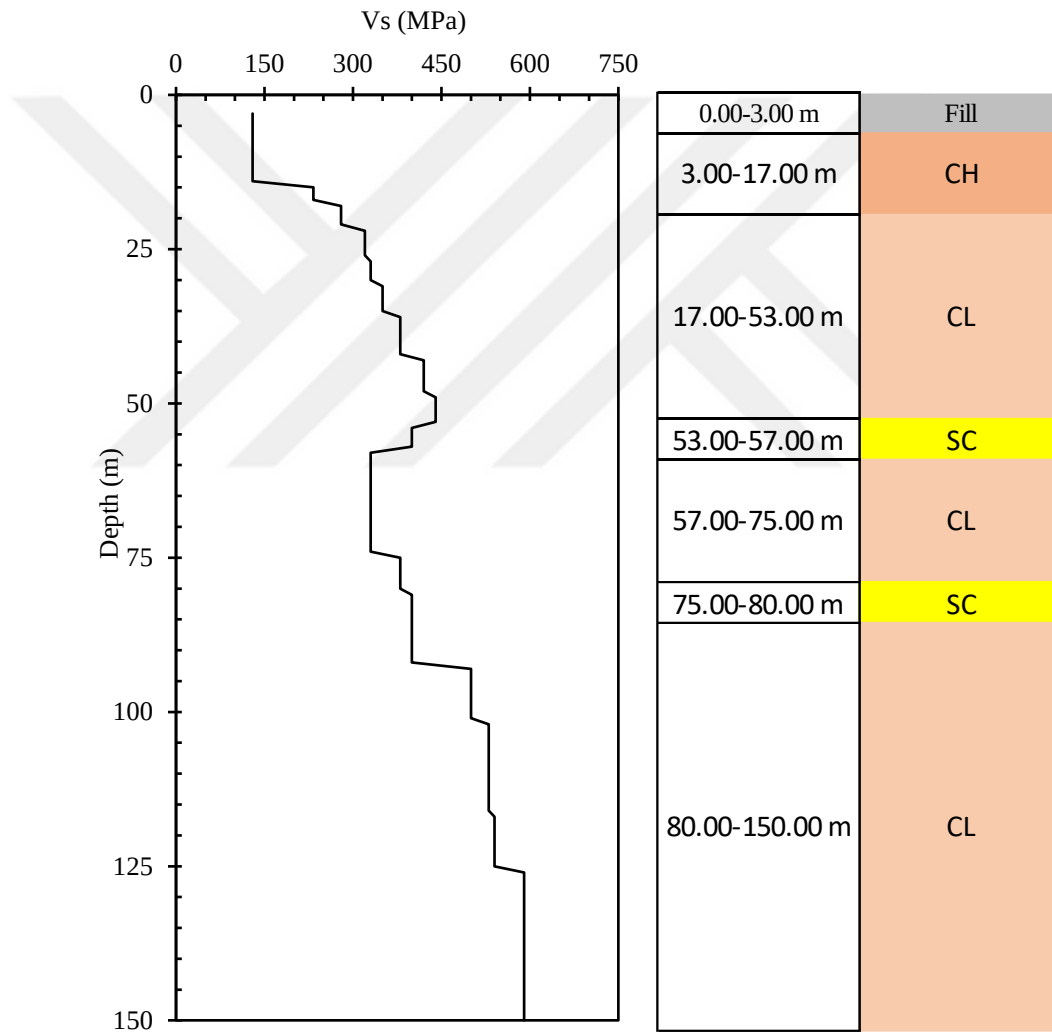


Figure 6.8 Soil profile of a site in Bayraklı Region

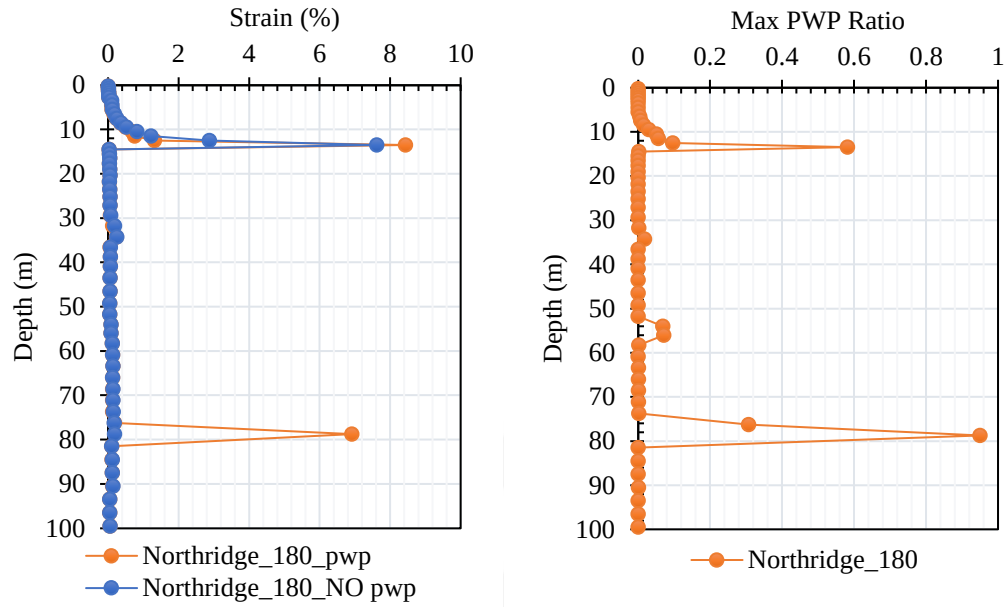


Figure 6.9 Strain and PWP results under Northridge Earthquake

Hysteresis loop for total stress and effective stress based nonlinear site response analyzes are given in Figure 6.10 and Figure 6.11.

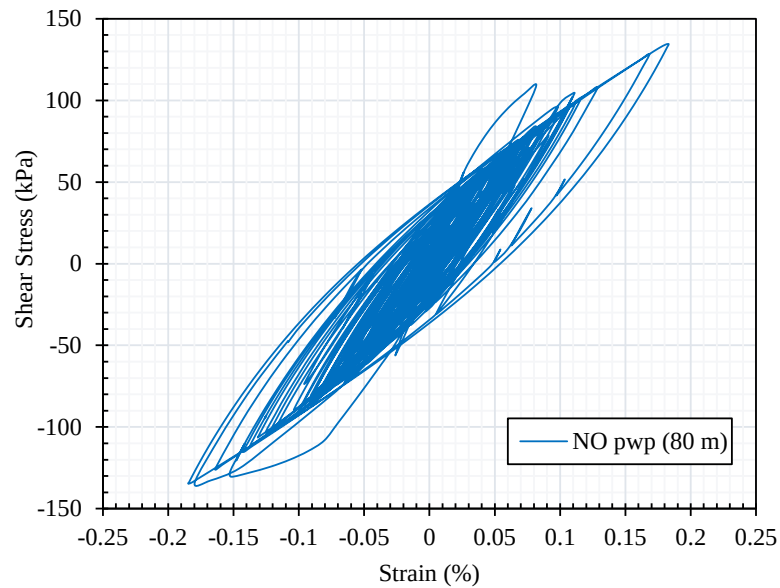


Figure 6.10 Stress-strain plot for Northridge Earthquake

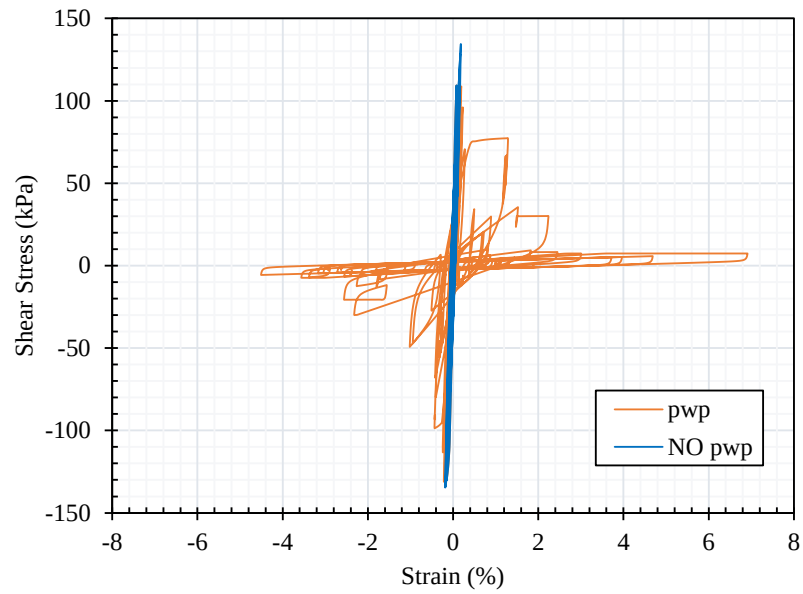


Figure 6.11 Stress-strain plot for Northridge Earthquake with pore water pressure model

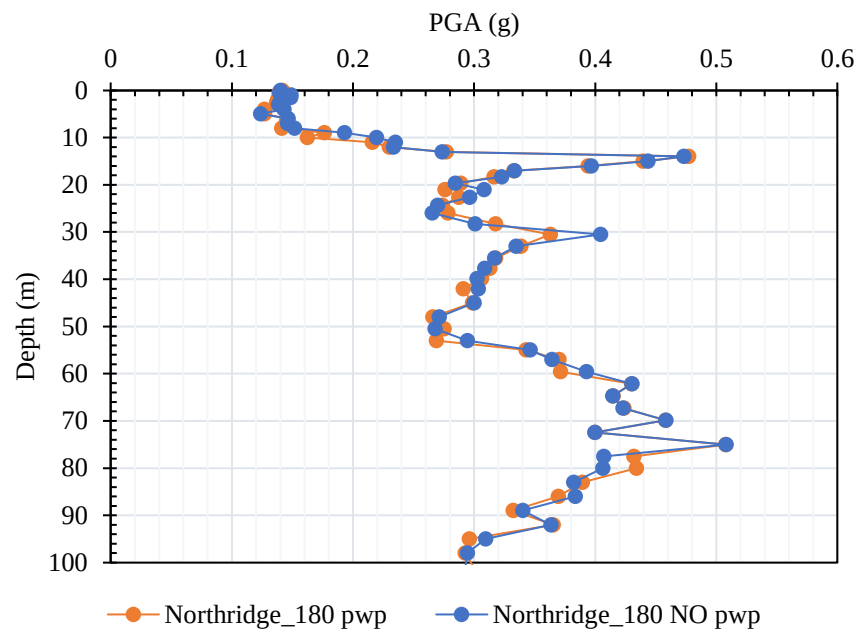


Figure 6.12 Peak ground acceleration results under Northridge Earthquake

In total stress based and effective stress stress-based nonlinear analyzes, varying strain levels did not affect the peak ground acceleration values. As can be seen in figure 6.12, there are no significant differences in PGA values varying with depth.

In the effective stress-based nonlinear analyzes performed by selecting a series of earthquakes belonging to the DD-2 earthquake level, it was observed that excess pore water pressure generation also occurred at very deep depths. In soil layers SC between 53-57 m and SC between 75-80 m, r_u is above 0.8. In Figure 6.13 and Figure 6.14, it is seen that when the strain level is increased, the r_u value increases accordingly. In addition, r_u values emphasize that deep-seated liquefaction behavior can be observed.

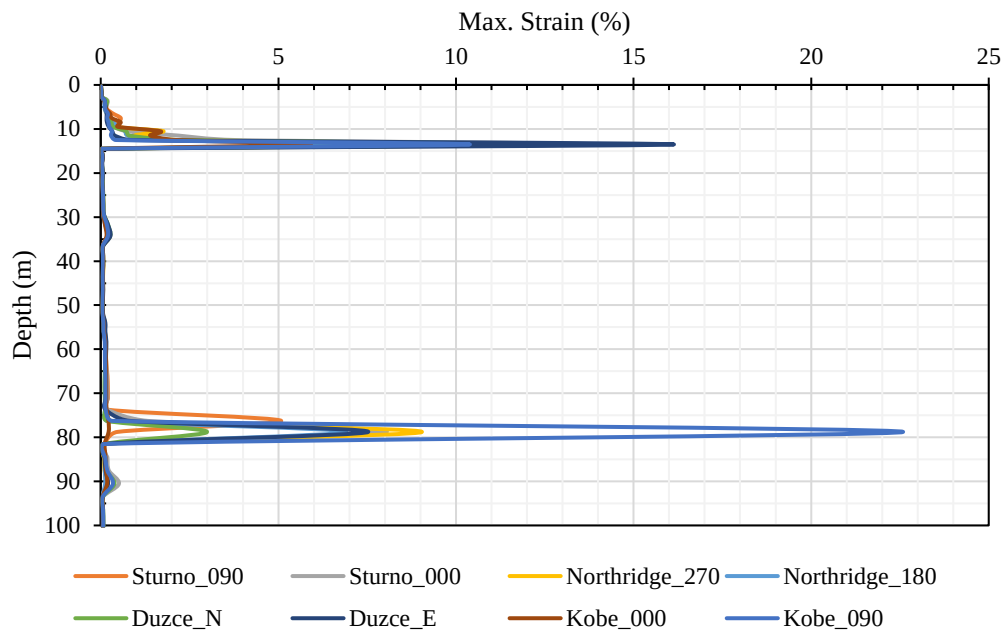


Figure 6.13 Shear strain results for DD-2 earthquake level

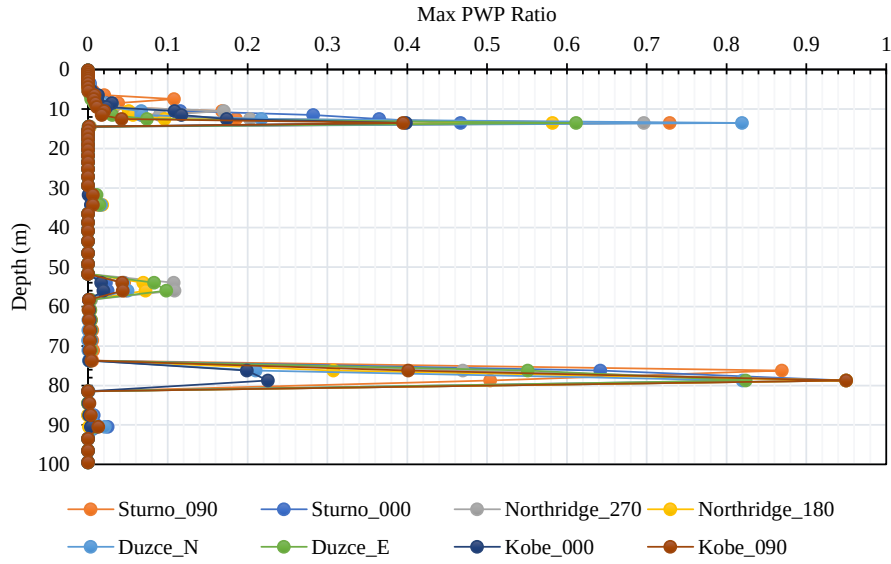


Figure 6.14 PWP results for DD-2 earthquake level

6.3 Effect of degradation parameter $\nu=1$ and $\nu=3.8$

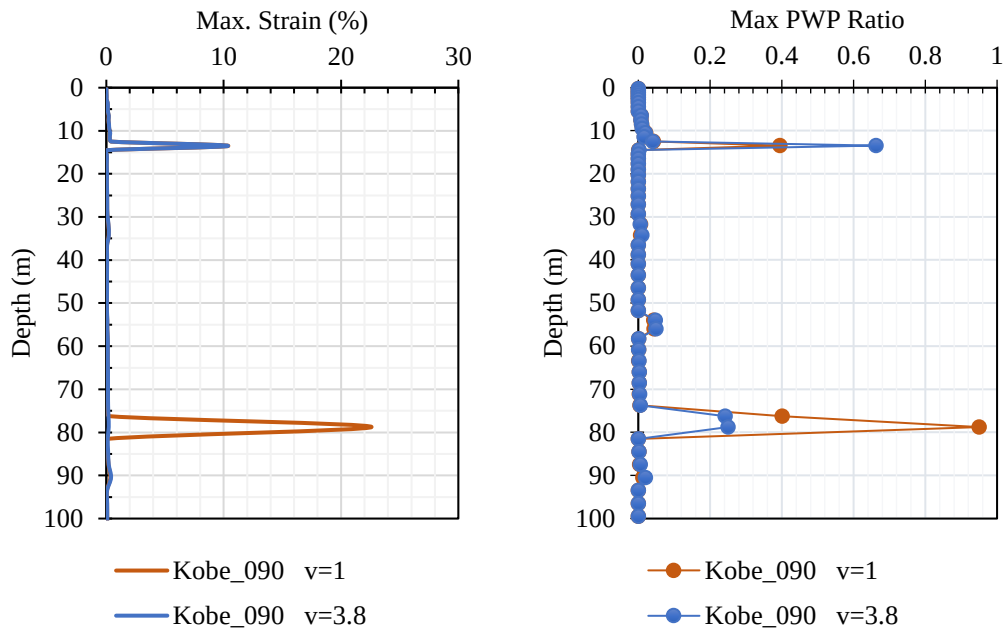


Figure 6.15 Effect of degradation parameter (ν) on maximum strain and PWP (Kobe earthquake)

The results obtained by changing the degradation parameters are given in figure 6.15. When the degradation parameter is $v=1$, high strain is seen in the SC layer between 75-80 m. Consistent with high strain, the r_u value exceeds 0.8. When the degradation parameter 3.8 is selected, the higher strain in the deep layers occurred in the shallower layers. The analyzes made with the Kobe Earthquake, which was recorded at the DD-2 earthquake level, are repeated for the Düzce Earthquake in figure 6.16. When the degradation parameter is selected as 3.8 in the analysis of the Düzce Earthquake, the r_u parameter exceeds 0.8 at shallow depths.

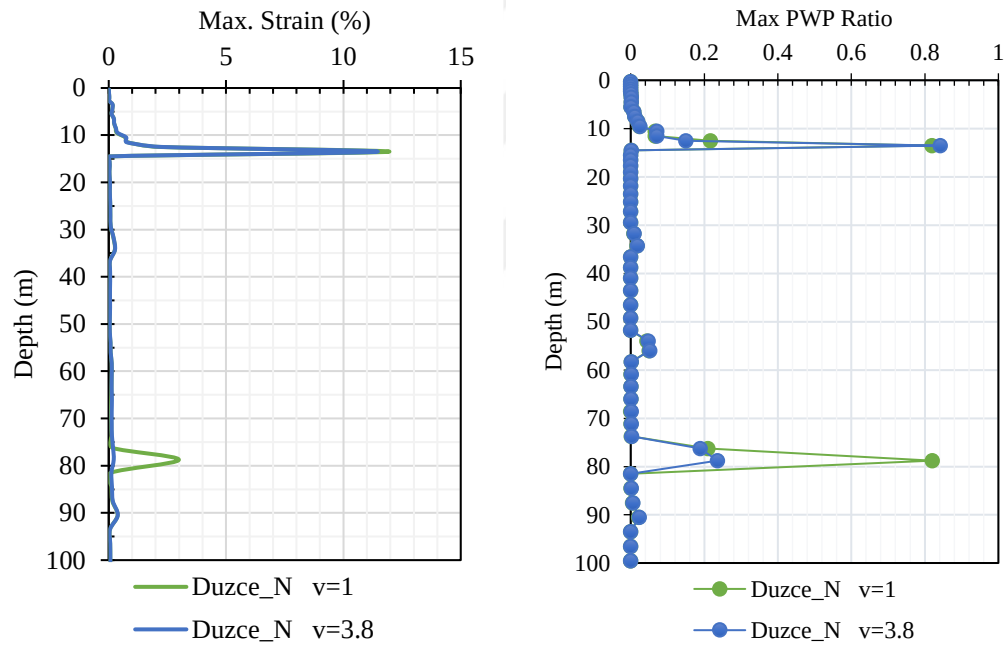


Figure 6.16 Effect of degradation parameter (v) on maximum strain and PWP (Duzce earthquake)

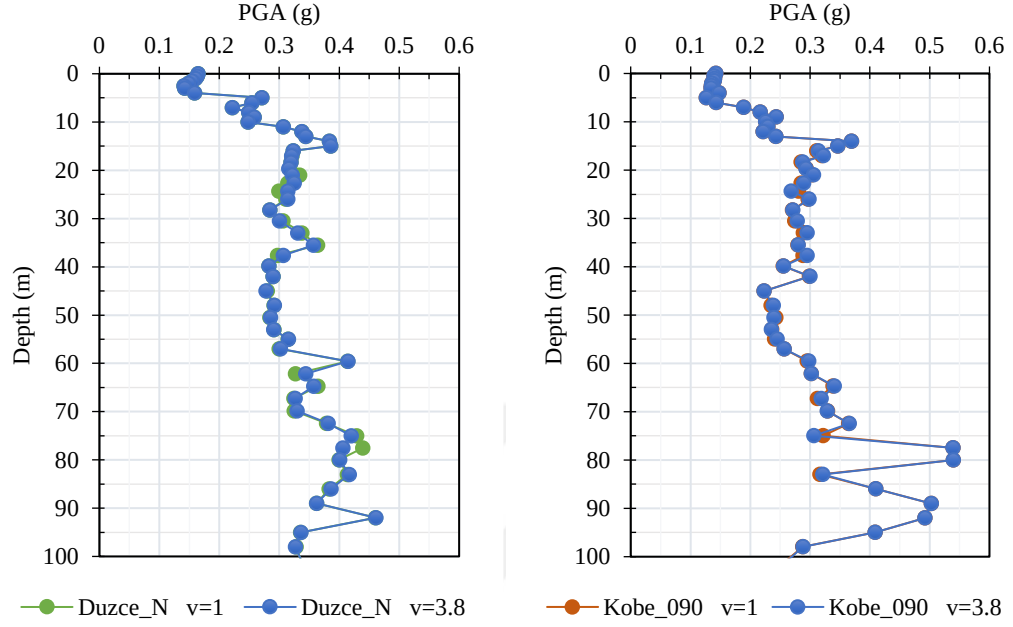


Figure 6.17 Peak ground acceleration results under Kobe and Duzce Earthquake

The effect of the degradation parameter on the strain level and the developing pore water pressure value is given in Figure 6.18 and Figure 6.19. The variation of the excess pore pressure value with time with increasing strain level is shown. As the degradation parameter increases, the strain level decreases and the r_u value decreases accordingly.

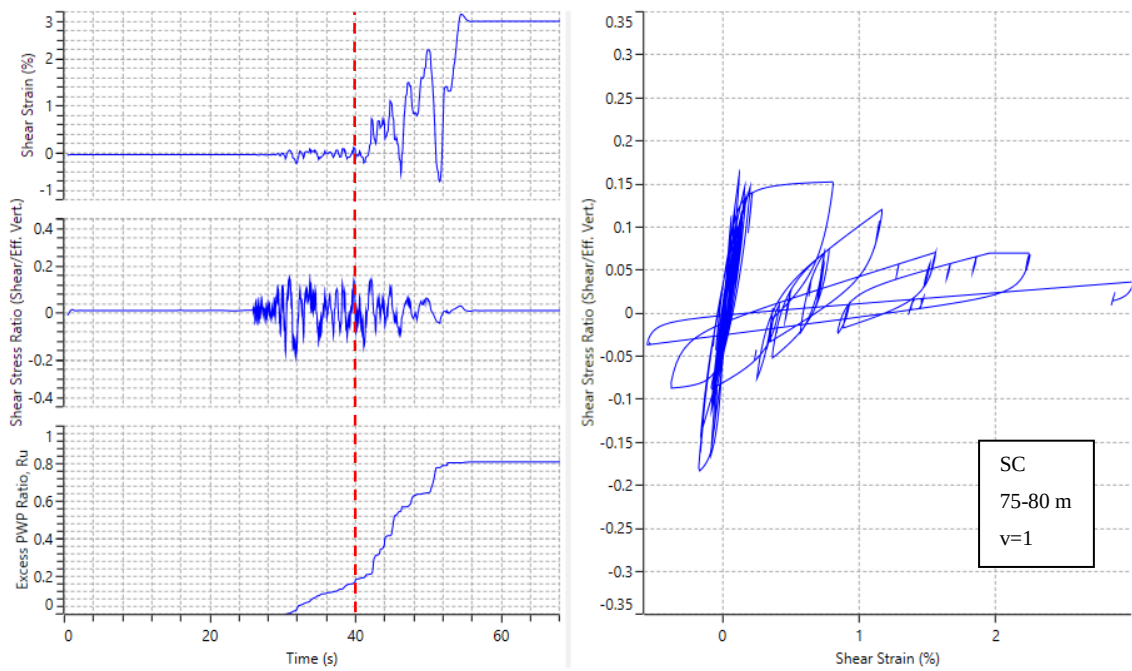


Figure 6.18 Shear strain, stress ratio, PWP and stress-strain plots at 75-80 m for $v=1$

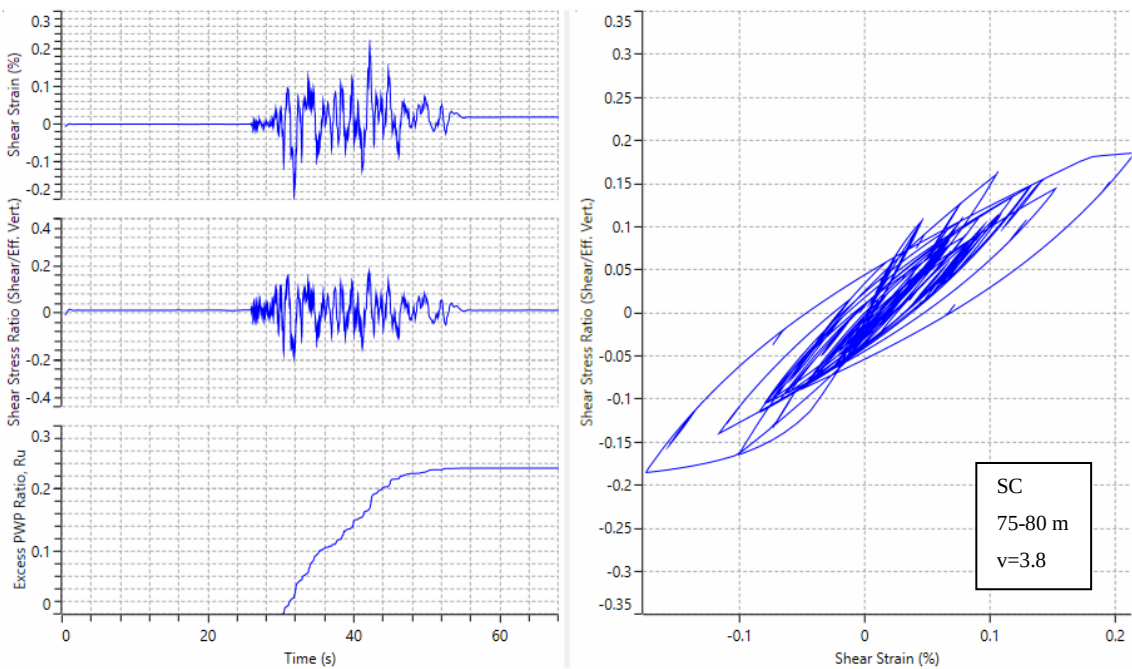


Figure 6.19 Shear strain, stress ratio, PWP and stress-strain plots at 75-80 m for $v=3.8$

CHAPTER SEVEN

CONCLUSIONS

Ground response analyses are carried out to predict the soil strains and motions on the ground surface. Equivalent linear and fully nonlinear analyses were performed for a site in Mavisehir/İzmir, and the results were compared by presenting the response spectra of the motions on the ground surface. The input was the strong ground motions of the Samos earthquake, which occurred on October 30, 2020. There were three strong ground motion stations in Bayraklı region, being on the rock outcrop. One record was on the soil surface in Camdibi/Bornova. The soil profile on the station was taken from a Tubitak project (KAMAG), and the record was used to predict the top of rock (or within) motion by deconvolution analyses. A detailed procedure of Silva (1988) was adopted in the deconvolution process to obtain the within and outcropping motions. Then, the motions were applied to the soil profile of Bayraklı Station, where the ground motion was recorded. The analyses were performed in time domain using the 1D constitutive models, namely MKZ and GQ/H. Besides, considering the uncertainty in the shear wave velocities, the analyses were repeated using the increased shear wave velocities by 20%.

The results showed that the MKZ model damped the peak ground and the spectral acceleration on the surface motion, significantly. Whereas, the response spectrum on the ground surface at the Bayraklı profile was better captured by GQ/H model. However, there was difference in the spectral accelerations at the larger periods. The analysis with the increased shear wave velocities by 20% in MKZ model gives better results for both the peak ground and the spectral acceleration at all periods.

The site of a collapsed building during the Samos Earthquake in Bayraklı region was used for the 1D analysis. Three strong ground motions recorded on the rock outcrop and the estimated rock motion (within and outcrop) from the deconvolution analyses were employed in the ground response analysis. The obtained surface motions were compared

to investigate the effect of input motions. Since the motion reaching to the ground surface at the location is not recorded, the accuracy of the results could not be tested. The results of the analyses with the different input motions does not provide a single consistent spectrum. The results showed the effect of the input motions on the soil surface records.

The effect of pore water pressure parameters on the site response analyses were investigated using Matasovic (1992) model. The key parameter for pore water pressure generation was the degradation parameter (v). Depending on the earthquake level, it may possible to observe high pore water pressures at the depth where alluvial soil layers exist. This result may confirm the phenomenon called as deep-seated liquefaction.

The following conclusions are obtained in this study:

- Dynamic soil properties are the key parameters in the analyses, and the selection of the soil models affects the ground response analyses (GRA) results remarkably.
- In general, the spectral accelerations in the MKZ model is lower than the GQ/H model that more damping takes place.
- Shear wave velocity profile has a significant effect on the results of GRA.
- The rock outcrops recorded on the stations were used in a soil profile in Bayrakli region, and it was observed that the results were not close to each other.
- Pore water pressure ratio (r_u) could be as high as 0.80 at depths where the stiffness contrast is high.

The thesis show that the ground response analysis results are highly sensitive to the soil modulus and damping parameters, and these properties must be better understood on the local scale. Besides, as the effect of the input motions is very important on the analyses, the uncertainties due to the selection of the records must be reduced. Excess pore water pressure generation may takes place in the deep alluvial soil layers, therefore, the future studies are required to better determine the model parameters.

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APPENDICES

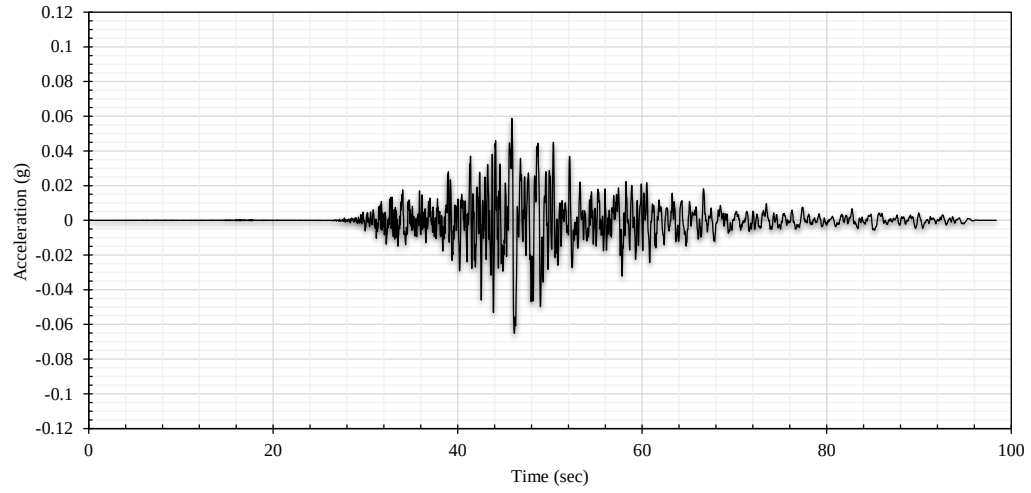


Figure A.1 Strong ground motion record of Çamdibi Station (3522 east-west direction)

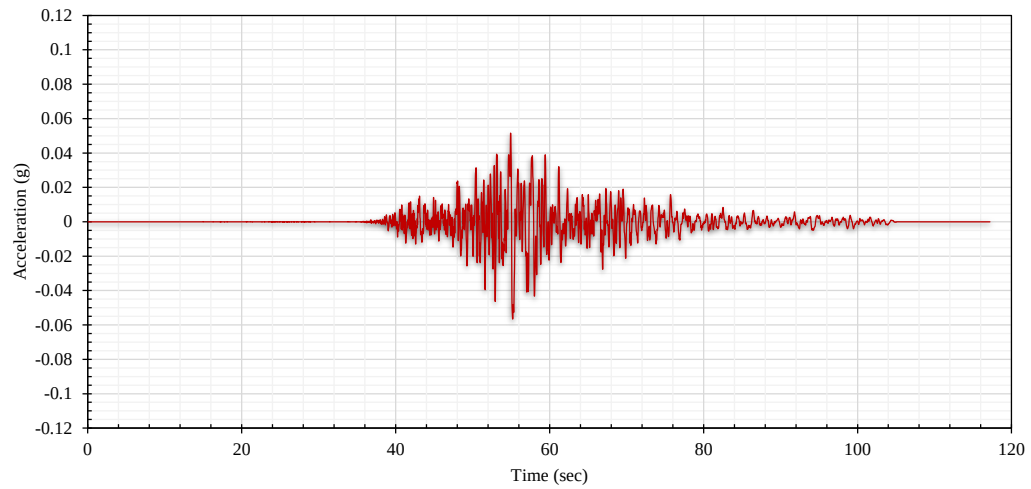


Figure A.2 Filtered and scaled motion of Çamdibi Station for deconvolution analysis (3522 east-west direction)

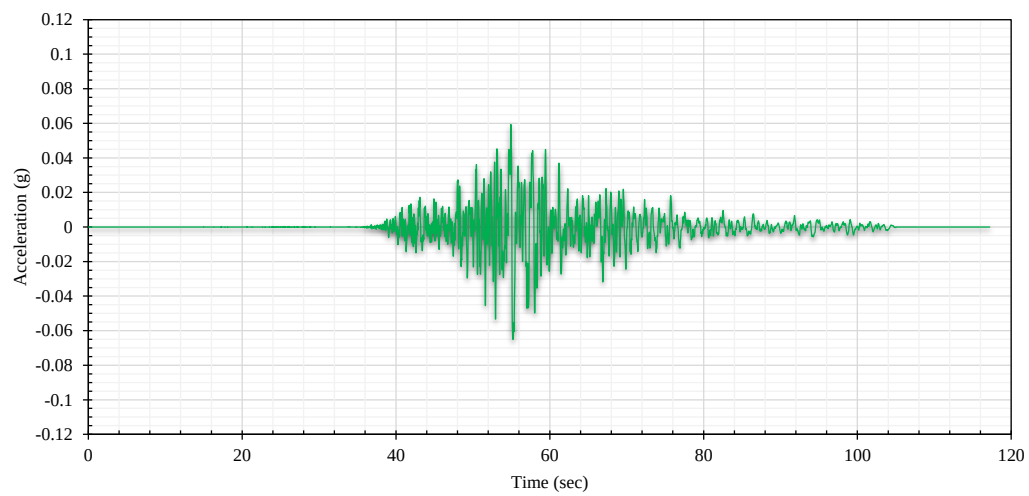


Figure A.3 Filtered record of Camdibi Station (3522 east-west direction)

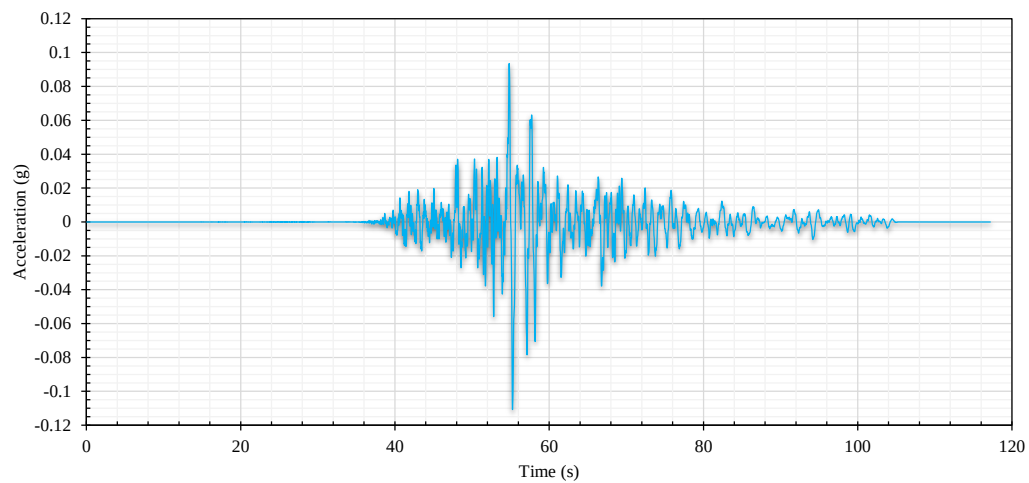


Figure A.4 Outcrop motion obtained in the deconvolution analysis (3522 east-west direction)

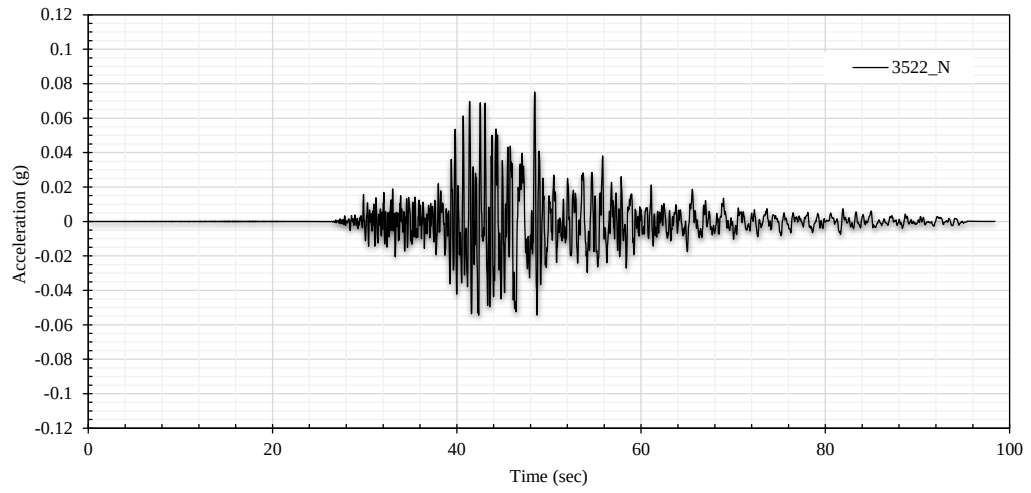


Figure A.5 Strong ground motion record of Çamdibi Station (3522 north-south direction)

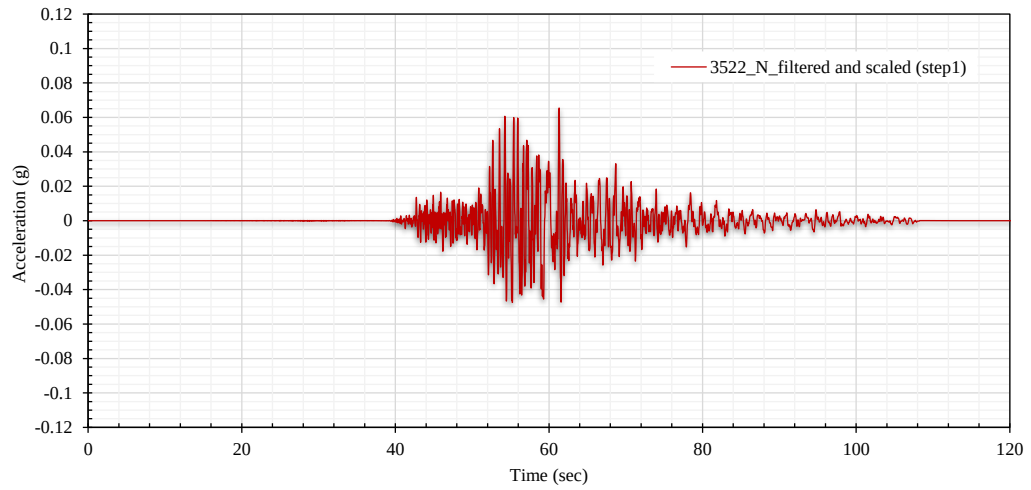


Figure A.6 Filtered and scaled motion of Çamdibi Station for deconvolution analysis (3522 north-south direction)

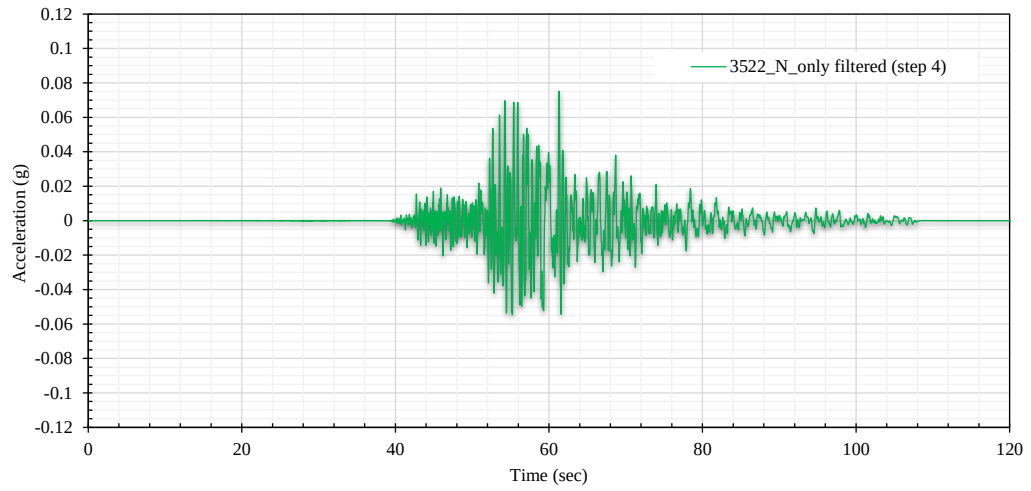


Figure A.7 Filtered record of Camdibi Station (3522 north-south direction)

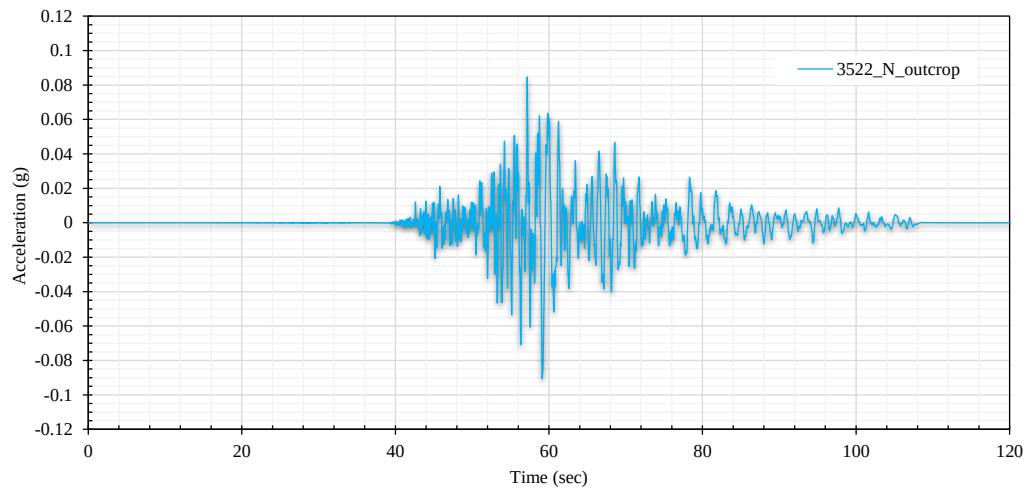


Figure A.8 Outcrop motion obtained in the deconvolution analysis (3522 north-south direction)