EFFECTS OF SOIL STRUCTURE INTERACTION AND BASE ISOLATED SYSTEMS ON SEISMIC PERFORMANCE OF FOUNDATION SOILS

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ABSTRACT

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In this thesis primarily structural induced liquefaction potential was aimed to be analyzed. Also the effect of base isolation systems both on structural performance and liquefaction potential was studied. FLAC software was chosen for the analyses so that structure and soil could be modeled together. By these means the soil structure interaction effects were also examined. Four different structures and three different sites were analyzed under two different input motions. All the structures were also analyzed as base isolated. It was mainly found that depending on the structural type and for a certain depth the liquefaction potential could be higher under the structure than the one in the free field. Also it was concluded that base isolation systems were very effective for decreasing the story drifts, shear forces in the structure and liquefaction potential in the soil. It was also found that the interaction took place between structure, soil and input motions.

Keywords: Structural Induced Liquefaction, Cyclic Stress Ratio, Soil Structure Interaction, Base Isolation Systems.

ÖZ

ZEMIN YAPI ETKILESIMI VE IZOLATOR SISTEMLERININ TEMEL ZEMININ SISMIK PERFORMANSLARI ÜZERINE ETKISI

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Bu tezde birincil olarak yapi kaynakli sivilasma potansiyelinin incelenmesi amaçlanmistir. Ayrica temel izolatörlerinin yapi performansina ve sivilasma potansiyeline etkisi çalisilmistir. Yapi ve zeminin birlikte modellenebilmesi için FLAC programi seçilmistir. Böylece yapi zemin etkilesimi de incelenmistir. Dört farkli yapi ve üç farkli zemin profili iki farkli deprem altinda incelenmistir. Bütün yapilar ayrica temel izolatörleri göz önüne alinarak analiz edilmistir. Genel olarak yapi tipine bagli olarak ve belirli bir derinlige kadar sivilasma potansiyeli yapi altinda bos sahadakine göre daha yüksek bulunmustur. Ayrica temel izolatörlerinin yapida katlar arasi deplasmanlari ve kesme kuvvetlerini, zeminde ise sivilasma potansiyelini düsürdügü bulunmustur. Ayrica etkilesimin yalnizca zemin ve yapi arasinda degil deprem kaydiyla da oldugu görülmüstür.

Anahtar Kelimeler: Yapi Kaynakli Sivilasma, Tekrarli Gerilme Orani, Zemin Yapi Etkilesimi, Temel Izolator Sistemleri

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INTRODUCTION

1.1 Statement of the Problem

Liquefaction engineering is one of the challenging areas in geotechnical earthquake engineering. Especially after urban areas struck by big earthquakes which caused considerable damage in structures due to liquefaction, it has been realized that more effort should be given to understand the interaction between structural performance and geotechnical aspects.

It was the Niigata, Japan earthquake in 1964 that first dramatically illustrated the importance of soil liquefaction under building foundations. Several hundred structures were damaged during this earthquake due to liquefaction of foundation soils. Since then, reliable methods have been developed for analyzing the liquefaction problem for the free field stress conditions, but the influence of structures on liquefaction triggering and post liquefaction performance has not been well understood.

The current state of the practice in assessing the potential for liquefaction beneath a structure is to treat the soil as if it were in the free field and ignore any effects of the building. This practice has developed since it is believed to be a conservative approach and easy to perform. In addition, it does not require structural properties. However it was observed, after recent earthquakes (e.g. Kobe,1995 and Kocaeli,1999), that there could be no signs of liquefaction in the free field but once one gets closer to foundation structure-induced liquefaction could be observed.

In very simple terminology liquefaction can be triggered by shear stresses introduced to the soil and can be reduced by the vertical effective stress on the soil. There are correlation charts between liquefaction resistance and SPT blow count proposed by Seed et al. (1984). Using these charts the cyclic stress ratio $\frac{t_{ave}}{s'}$ required for the liquefaction can be determined for different values of SPT blow counts. t_{ave} is the equivalent uniform cyclic shear stress induced by the earthquake and s' is the vertical effective stress. The existence of the structure affects both the shear force and vertical stresses imposed to the soil relative to the ones in the free field. So before understanding these static and dynamic stress distributions in the soils under structures discussing the potential of liquefaction will be premature.

Another challenging and widely recognized area in earthquake engineering is structural control. Especially the passive control of structures (i.e. the use of base isolators and dampers) are now considered to be a dependent and very effective ways to decouple the structure and soil. Doing so will prevent the adverse effects of structure and soil mutually.

The main purpose of base isolators is to reduce the level of acceleration imposed to the structure consequently the shear forces and relative displacements in the super structure. Based on the short discussion on liquefaction above, it can be seen that the reduction in the level of shear stresses will not only prevent structural damage but also behave in the favor of liquefaction potential.

The effect of soil structure interaction on both structural and soil performance is worth to be taken into consideration not only for special type of structures such as power plants but also for conventional structure for which the response values can be different than the decoupled analyses results.

1.2 Aim and Scope of the Study

Based on the discussions so far, soil liquefaction potential of free field soils is different than that ones under structures mainly due to variabilities in stress conditions these soils are subjected to. It is aimed to examine the effects of structure on cyclic stress ratio, a measure of soil liquefaction demand, in foundation soils.

As part of this aim, it can be seen that the use of base isolators reduces the shear stresses in the underlying soils and overlying structures. This does not only prevent the structural damage but also reduces the stress level introduced to the soil by the superstructure. Based on case histories where no indication of liquefaction was observed in the free field but under the structures. The advantages of isolators from both structural point of view, by reducing the acceleration levels and drift ratios; and from geotechnical point of view, by reducing the shear stress levels, consequently liquefaction potential, are aimed to be studied.

In this thesis, structure and soil was modeled together so that the soil structure interaction could be taken into account. To model the whole system, the program FLAC which implements the finite difference method, was used. Two input motions were selected as Kocaeli (1999) and El-Centro (1979) Earthquakes.

Three different types of soil strata were modeled. For this purpose the borehole data obtained from the Sakarya City were used. The sites were chosen so that each one had different periods.

Four different types of structures were modeled to stand for three different types of structural periods. The structures were modeled as three, four and six story structures. Also one more four story structure was modeled having the same structural period with the original four story structure with an additional 50% weight. All these types of structures were analyzed also by adding an isolation layer.

As a result 54 different scenarios were analyzed and the results for the free field and structural induced cyclic stress ratio obtained. Also the effects of base isolation from both structural and geotechnical point of view were examined. Lastly the soil structure interaction was discussed.

1.3 Scope of the Thesis

This thesis mainly composed of five main sections. Contents of each chapter are summarized as follows:

Chapter 1 The general framework of the thesis was stated. Also the aim and the scope of the study were defined.

Chapter 2 Past studies mainly on the effects of structures on the potential of liquefaction, the base isolation systems and soil structure interaction were reviewed.

Chapter 3 The characteristics of the input motions were given in this chapter. Also the soil profiles were constructed by using correlations to the field tests and the responses of different soil profiles were examined. The basic advantages of the base isolation were given and the structural properties were introduced. Lastly the effects of mesh size and boundary conditions were explained which are one of the primary importance in numerical methods.

Chapter 4 In this chapter the results of the analyses were examined. Firstly the results of the soil response which were obtained by plastic analyses were given. Afterwards the liquefaction potential was studied on the basis of cyclic stress ratio approach. The effect of the structure on liquefaction potential and the advantages of base isolation on the basis of cyclic stress ratio were shown. The effect of base isolation from structural point of view was also given in this chapter. Finally, the effect of soil structure interaction on response spectrum was studied.

Chapter 5 Summary and conclusion of the results were given. Lastly, possible future extensions were discussed.

CHAPTER II

REVIEW OF PAST STUDIES

2.1 Introduction

Numerous analytical and experimental studies has been performed on the three topics mentioned in the previous chapter, namely; effect of structure on liquefaction potential, base isolation systems, soil structure interaction. Consequently, simplified procedures and computer codes have been developed. However it was the last decades in which big earthquakes happened so that it turned out to be possible to compile performance case history database. Many advances have been made on the topics mentioned above, some of which are summarized in the following three sections.

2.2 Past Studies on the Effect of Structures to the Cyclic Stress Ratio

As part of common practice, the potential for liquefaction below a building is often evaluated by assessing the soil under free field stress conditions. However a summary of available field case histories, shaking table model tests and centrifuge model tests is presented by Rollins and Seed (1990). This study indicates that the liquefaction potential can be much different near a building than in the free field.

There are some cases in the literature where it appears that the presence of a structure prevented liquefaction. Two cases involving oil-storage tanks founded on sand deposits were reported. Watanabe (1966) indicated different performances of oil storage tanks that were subjected to 1964 Niigata Earthquake. While liquefaction was observed in the free field near the tanks, the tanks themselves experienced little or no damage because of liquefaction. A second field case concerns the performance of three oil tanks that were subjected to the Miyagiken-Oki earthquake of magnitude 7.4 in 1978. A study performed by Ishihara et al (1980) indicated that liquefaction would have been expected without the presence of the tank, however no settlement of the tank was observed. Water spouts and sand boils were observed in the areas adjacent to the tanks.

Although the two previous cases indicate that liquefaction might be less likely to trigger under structures as compared with the free field, some investigations suggest that the conditions for liquefaction are worse near a structure than in the free field. Liu and Qiao (1984) indicated that this conclusion was supported by a damage investigation following the Tangshan, China earthquake of magnitude 7.8 in 1976. During the investigation it was often found that more sand boils and fissures were concentrated near structures than in the free field. The very similar field investigations were done after the Kocaeli earthquake of magnitude 7.4 in 1999.

It is often difficult to draw firm conclusions based on the limited field evidence available. Additionally, the case histories also have some uncertainties like the acceleration levels or the soil properties. In order to overcome the difficulties, many investigators have performed experimental studies to simulate the field conditions for the purpose of studying the influence of structures on liquefaction potential.

While many of tests showed that pore pressures were lower beneath the building than at equivalent depths in the free field, some did indicate that there was a zone near the model structure which was more susceptible to pore pressure generation than the free field. After an experimental study, Liu and Qiao (1984) found that sand boils developed first near the buildings before progressing towards the free field.

Rollins and Seed (1990) stated that if the spectral acceleration ratio, (S_a/a_{max}), corresponding to the building period is higher than 2.75, the induced cyclic stress ratio would be higher beneath the building than in the free field. If the ratio (S_a/a_{max}) is less than about 2.75, then the induced cyclic stress ratio will be lower beneath the building and the potential for liquefaction will tend to be decreased, although some change in liquefaction resistance will occur because of the increased overburden pressure. Additionally they made an assumption for the increase in the vertical stress with depth and adopted an average value. After this averaging, according to Rollins and Seed (1990), liquefaction would occur below the building before it occurs in the free field for (S_a/a_{max}) ratios greater than about 2.4.

Rollins and Seed (1990) goes further and uses this limit value and compares it with the spectral values obtained by using more than 100 strong ground motion data. According to their findings, for the buildings with periods greater than about 0.75 sec the liquefaction potential will be lower beneath the building than in the free field. For the buildings with periods greater than about 1 sec, the liquefaction potential may be 60% less than the free field value. However; for the buildings with a period between 0.1 and 0.5 sec the liquefaction potential might be worse than in the free field.

2.3 Past Studies on Soil Structure Interaction

During an earthquake, foundation soils filter and transmit the shaking to the building and at the same time it has the role of bearing the building vibrations and transmitting them back to the depths of the ground. In other words the ground and the building interact with each other. This interaction has been attracting the interest of researchers for the last half-century.

One of the first comprehensive studies belongs to Seed and Lysmer (1975). The drawbacks and the advantages of two methods which are still being used were examined. These methods are 1) representing the effect of the soil on the structural response by a series of springs and dashpots or 2) modeling the soil-structure system by finite element method. They pointed out the lack of rigorous numerical modeling and again the lack of database obtained from field cases which are no more concern thanks to powerful PC's and recorded response data.

One of the ways to evaluate natural period of a soil structure system is to use micro tremor data. Similarly, Ohba (1992) proposed a correlation between natural period of a structure as a function of its height which is a commonly used in practice. He also included the effect of stiffness of the soil on the natural period of the structure. The standard penetration test results were used to account for the stiffness of the soil. He concludes that increase of the height makes natural period of the structure longer, also this value gets longer as the stiffness of the soil gets smaller. Putting aside the changes in the level of acceleration because of the existence of the structure and considering the response spectra obtained from the free field motions and from the ones underneath the structure being equal, even this observation itself is enough to emphasize the effect of soil structure interaction.

A report was published by Architectural Institute of Japan after the Kobe earthquake, 1995 (1997). During this earthquake very many strong ground motion data were obtained in and around city of Kobe. Among them, there were some records which were simultaneously obtained at the foundation level and at the ground surface. After comparing them, it was concluded that the maximum accelerations on the foundation level are smaller. It was revealed in this report that the maximum accelerations on the foundation levels were 30% smaller than the ones in the free field.

In soil structure interaction field, few empirical studies have been performed due to limited availability of strong motion data from sites with instrumented structures and free field accelerometers. Recently, a comprehensive study was conducted by Stewart et al (1999) using 77 strong motion data sets at 57 building sites which encompass a wide range of structural and geotechnical conditions.

It was observed firstly in this study that there was nearly no reduction in spectral acceleration values obtained from free field and surface foundation motions, which was the primary important parameter controlling the structural response. However it is worth to note that there are cases for which there is a considerable reduction or sometimes increases in spectral accelerations. Also for the same site and same structure, different response of the structures and the level of acceleration were obtained under different input motions. For one earthquake the free field value of the peak ground acceleration was recorded to be greater than the one obtained from the surface foundation motions. For another input motion, the peak ground acceleration obtained from the surface foundation motions turns out to be greater. These kinds of observations lead to a conclusion that the interaction takes not only between soil and structure but also with the input motion itself.

However Stewart et al (1999) indicates that there is a high correlation between the lengthening ratio of structural period (T'/T) due to the flexibility of the foundation and structure to soil stiffness ratio. Typical soil structure interaction effects occur for the values of around 0.1–0.3 of stiffness ratios. For these typical values the lengthening in the period is around 1.1–1.5. However there are again some cases for which the stiffness ratio is around 1.5 and consequently lengthening in the period are around 4. Such a big difference in natural period results in completely different level of accelerations. As a general trend when the structure is stiff and the underlying soil is soft the soil structure effect gets important, on the other hand as the structural period gets longer and the stiffness of the soil under the structure gets higher soil structure interaction loses its importance. This extreme case can be a base isolated structure founded on a rock site which can be found in the data sets compiled by Stewart et al (1999). For these kinds of structures, it can be observed that there is hardly any soil structure interaction effect.

2.4 Past Studies on Base Isolation Systems

Decoupling of the superstructure from the soil, consequently from the adverse effects of the earthquake motion by using base isolation systems has become a very popular method. Also for special structures, such as historical buildings, police departments, hospitals, structures for high technology equipments, it is the only way of seismic protection. Although the effectiveness of the base isolation systems has been proved by experimental and numerical studies, there was a debate on their effectiveness during earthquakes. However all the base isolated structures performed well during two big earthquakes: Norhtridge, 1994 and Kobe, 1995.

The very recent earthquakes which have occurred in Italy have highlighted the particular vulnerability of historic masonry buildings and the need of properly defining retrofitting measures. An experimental test program on a full scale model representing a sub assemblage of the cloister façade of the Sao Vicente de Fora monastery, retrofitted through base isolation, has been carried out at the European Laboratory for Structural Assessment of the Joint Research Centre. The results are provided by Luca et al (2001). The experimental results suggested a particularly improved behavior of the base isolated specimen. Depending on the earthquake and the type of isolators, the masonry part above the isolators experiences displacements from 2.8 up to 24 times smaller than the ones registered in the fixed base test model. The forces are reduced from 1.5 up to 15 times. No further cracking and damage in the masonry elements of the specimen, which extensively occurred during the previous fixed base test, have been observed following the base isolated tests.

Nagarajaiah et al (1999) performed a study on the performance of base isolated USC hospital building during Northridge Earthquake. The seismic response and performance evaluation of the hospital building shows that base isolation was effective in reducing the response and providing earthquake protection. Nagarajaiah et al (1999) made conclusions on different aspects of base isolation obtained from the sensors on the building. The period lengthening and the dominant fundamental mode response are shown to be the main reasons for the effectiveness of base isolation. The fundamental period was nearly 1.3 seconds in the base isolated case. This value was beyond the predominant periods of the ground motion spectral accelerations. The estimated 15% damping in the fundamental mode reduced the base displacement further.

In base isolated structures, the contribution of the second mode is negligible or orthogonal to the earthquake input and the fundamental mode dominates the response Kelly (1993). For the USC hospital building case, it was shown by Nagarajaiah et al (1999) that a significant amount of response occurred in the fundamental mode (effective modal mass was 93% for the fundamental mode). The period of the second mode and the predominant period of the ground motion was close to each other 0.4 and 0.5 sec; however the second mode (effective modal mass of 6%) had smaller participation when compared with the fixed base case (effective modal mass of 24%). So a general conclusion for the higher mode effects can be drawn for this individual case as the Northridge earthquake had significant percent of its energy in the higher mode range but could not transmit the energy because the participation of the higher modes was reduced due to base isolation.

Base isolation reduced the base shear, accelerations, and story drifts. The peak roof acceleration was reduced to nearly 50% of the peak ground acceleration due to the effectiveness of base isolation. It is worth to put the figures; the free field acceleration was 0.49g and the peak foundation/ground acceleration was 0.37g. By comparing these two values again the effect of soil structure interaction can be seen for this specific case. The peak roof acceleration was reduced to 0.21g thanks to base isolation systems for the USC hospital building.

Comparison of the computed response in the base isolated and fixed base cases indicates that the response would have been three times larger in the fixed base case. For example, the peak drift was <30% of the code specification, and the superstructure remained in the elastic range which would have not been the case if it were a fixed base structure.

It was also shown by Nagarajaiah et al (1999) that the bearing displacement during the Northridge earthquake was only 10% of the design displacement (25 cm). This is because ground spectral acceleration at the fundamental period was only 30% of design spectral acceleration. In this study the computed peak base displacement in the case of an earthquake with spectral accelerations at 2.2 seconds equal to the design spectra is 15 cm, which is less than the design displacement. So, the building is expected to perform well in future earthquakes similar to those used in the original design.

Although isolation systems were proved to be effective under normal scenarios, excessive research on different aspects of base isolation systems such as its adaptability to developing countries, effectiveness under near fault motions and the use of additional damping have been carried out.

CHAPTER III

DETERMINATION AND CHARACTERISTICS OF INPUT MOTIONS, SOIL CONDITIONS AND STRUCTURAL PROPERTIES

3.1 Introduction

In this chapter the characteristics of the input motions used in the analyses were examined. The determination of soil properties and the response of different sites were given. Also the structural properties were explained. Additionally, a section was devoted to explain briefly the behavior of base isolated structures. Lastly the commercial program used in the analyses was discussed.

3.2 Characteristics and Selection of the Input Motion

Three properties are mainly used to define a strong ground motion record. These are namely the amplitude, duration and the frequency content. The first characteristic can be easily visualized; the damage level in the soil or structure is directly proportional with the amplitude of the earthquake. The amplitude of the motions can be examined simply by the response quantities. The acceleration response values have been widely used because the damage can be easily correlated with the acceleration; however recently also velocity response values have become popular. This is supported by the case history performance data collected after the Chi-chi, 1999 and Kobe, 1995 earthquakes and it was found out that the correlation between the velocity and damage level could be better than that one with acceleration level, especially for near field cases.

The duration of the motion is mainly related with the plastic deformations in the soil and in the structure. The plastic deformations can be caused by a short duration or a long duration motion mainly depending on the amplitude; however if this yielding occurs early in earthquake shaking then the duration of each yielding will affect the overall response.

The frequency component can be best understood by the fourier amplitude spectrum. This topic will be discussed more in detail in the following sections. However it is worth to say a few words at this stage. Basically the earthquakes can be rich in high, medium or low frequencies depending on the mechanism of the earthquake. There will be two kinds of amplification in a whole system; firstly because of the soil strata and secondly because of the structure. It is very important that if the earthquake contains any considerable energy for a range of frequency for which there will be an amplification because of the two sources mentioned above, the resulting spectrum will have a very big energy in that range.

The acceleration time histories and response values of the Kocaeli, 1999 and El-Centro, 1979 Earthquakes for 0.05 damping which were used in the numerical analyses in this thesis, were plotted in Figure 3.1. Kocaeli Earthquake has strong pulses nearly for 10 seconds; however El-Centro Earthquake has comparably small pulses but they continue nearly for 30 seconds. In Figure 3.1 it can be seen that response values for Kocaeli Earthquake are greater than the ones for El-Centro Earthquake. The frequency content of the motions can also be compared; in acceleration response spectra Kocaeli Earthquake has its peak values in 0.2-0.3 seconds and El-Centro Earthquake has its peak values in 0.4-0.5 seconds which is because Kocaeli Earthquake is also rich in higher frequencies.

Kocaeli Earthquake, M_w=7.4, 1999 El-Centro Earthquake, M_w=6.5, 1979

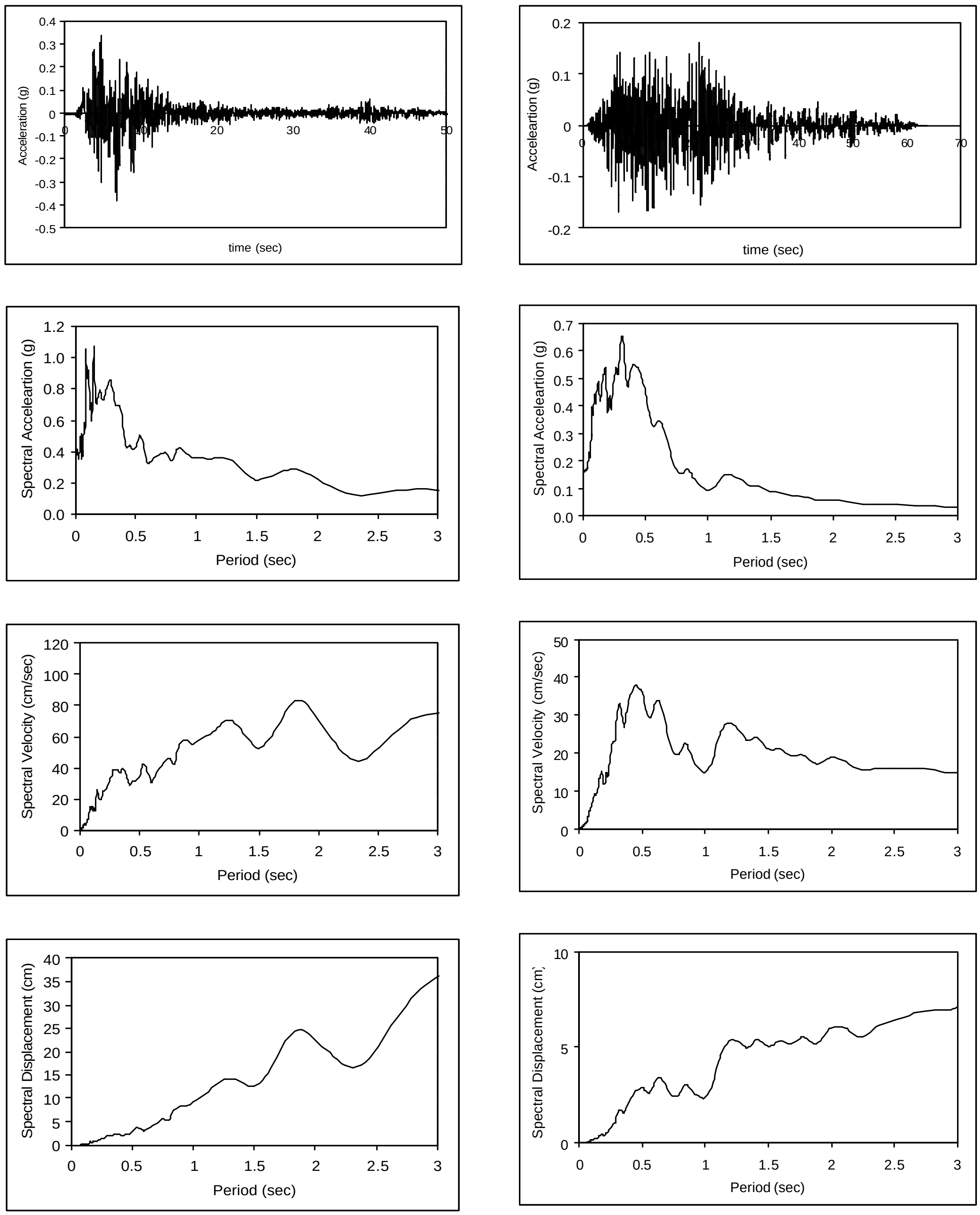


Figure 3.1 Acceleration Time Histories and Response Values of the Input Motions Used in the Analyses

3.3 Soil Characteristics

In this chapter the procedure to determine the soil parameters were discussed along with the effect of different soil strata on the response. Two bore-logs from boreholes in Adapazari, were adopted as the basis to determine the soil parameters. The results discussed in this chapter were obtained from the SHAKE analyses. The aim to perform SHAKE analyses was to examine simply the free field response.

3.3.1 Determination of Soil Parameters

In the following two sub sections firstly the determination of shear wave velocity and secondly the other parameters were discussed. These parameters were used to define the site characteristics.

3.3.1.1 Determination of Shear Wave Velocity

For soil site response analyses the most important parameter is the shear wave velocity of the soil layers. Although there are some more parameters controlling the soil behavior when it is modeled as a plastic material, shear wave velocity still stands to be the most important parameter. There are number of correlations between the V_s (shear wave velocity) vs. the field test as SPT, CPT, etc. results in the literature; however being widely used, SPT correlation studies are the still most popular ones in the common practice.

In the SPT, a standard split barrel sampler is driven into the soil at the bottom of a bore hole by repeated blows (30 to 40 blows per minute) of a 63.6 kg hammer released from a height of 76 cm. The sampler is usually driven 46 cm. The blows are counted for each 15 cm, considering the first 15 cm would have been disturbed, it is omitted and the number of blows required to achieve last 30 cm. of penetration is taken as the standard penetration resistance, N. This N value stands for the soil type, confining pressure and soil density.

Because the test is affected by a series of reasons it is a common practice to normalize the N value to an overburden pressure (100 kPa) and to correct it to an energy ratio of 60% as follows:

$$(N_1)_{60} = N_m . C_N \cdot \frac{E_m}{0.6.E_{ff}} \tag{3.1}$$

where N_m is the measured penetration resistance, C_N is overburden correction factor, E_m is the actual hammer energy and E_{ff} is the theoretical free fall energy.

In this study V_s estimates from the procedure proposed in Seed et al. (1984) were considered. It was proposed in this study that the shear modulus, G_{max} can be correlated to mean effective stress and the SPT blow counts as:

$$G_{\max} = 20000.(N_1)_{60}^{0.333}.\sqrt{s'_m} \tag{3.2}$$

where G_{max} is in psf, s'_m is the mean effective stress.

After the determination of the G_{max}, V_s can be found as:

$$V_S = \sqrt{\frac{G_{\max}}{r}} \tag{3.3}$$

where ? is the mass density of the soil.

3.3.1.2 Determination of Other Soil Parameters

Although the comparison between the results obtained from two different programs; SHAKE and FLAC will be made afterwards. In this section only the procedure to obtain the soil properties which were used in the programs was explained.

In both programs, the soil mass density is taken as an average value of 1.7 t/m^3 and the water level was chosen to be 1.5 m below the ground surface.

In site response analysis by SHAKE91 (Schnabel et al., 1972) the equivalent linear model is used. For this purpose, firstly the shear modulus or shear wave velocity and the hysteretic damping need to be defined. The degrading effects of large ground strains on dynamic soil behavior are often quantified by relationships for modulus degradation (G/G_{max}) and damping vs. shear strain. Average degradation values are used for clays, sands and rock for the analyses. Below are the relations which were used in the analyses.

For damping relationship:
Sand: “Average” proposed by Seed and Idriss, 1970
Clay: “PI=30 OCR=1-8” proposed by Vucetic and Dobry, 1991
Rock: “Rock” proposed by Schanabel, 1973

For modulus degradation relationship:
Sand: “Average” proposed by Seed and Idriss, 1970
Clay: “PI=20-40” proposed by Sun et al., 1988
Rock: “Rock” proposed by Schanabel, 1973

In FLAC analyses, the mohr-coulomb failure criterion in conjunction with elastic perfectly plastic model was implemented. For this model in addition to shear modulus and mass density, bulk modulus, K, tension cutoff, t, cohesion, c, and internal friction angle, Φ are needed.

The bulk modulus can be correlated to shear modulus by:

$$K = \frac{G.(2.(1+n))}{3.(1-2n)} \tag{3.4}$$

where n is the poisson's ratio. It was taken to be 0.3 as an average value. Inserting this value into Equation 3.4, K value turns out to be twice of G value.

The tension cutoff value can be determined by:

$$t = \frac{c}{\tan \Phi} \tag{3.5}$$

where Φ is the internal friction angle and c is the cohesion.

In this study, Φ and c values were determined from Equation 3.6 and Table 3.1:

$$c = 0.29.N^{0.72}.100 \tag{3.6}$$

where N is the SPT blow count vales. The cohesion values were used for clayey materials. For sands these values were taken much lower than the ones used for clays.

Table 3.1 Correlation between SPT Blow Counts and Internal Friction Angle

N	Φ
5-10	30-35
10-30	35-40
30-50	40-45
>50	>45

3.3.2 Comparison of Soil Response Obtained From Different Sites

As stated before, the analyses were performed by FLAC; however a preliminary study was performed by SHAKE to have a better idea on the selection of soil strata, structural types and the input motions. It was known that the results obtained from FLAC analyses will be different because of different reasons. These reasons are mainly due to the elastic-perfectly plastic modeling of soil and the soil structure interaction.

However the main idea was to choose three different sites having their own site periods. So the amplification of the three sites for the input motions was known to be different. The structures were chosen so that their natural periods were close to the natural periods of the sites.

In Figure 3.2 the soil profiles used in the analyses with their own site periods can be seen. In Figure 3.3 and Figure 3.4, the comparison of response spectra for within motions, and the ground surface motions were presented for Kocaeli and El-Centro Earthquakes.

To obtain within motions, original surface outcrop motions of Kocaeli and El-Centro records were deconvolved to a depth of 20 m. for the first and second site for which the engineering bedrock was assumed to start. For the third case another 40 m. was assumed to exist under the top 20 m. The original outcrop motion was first deconvolved to 60 m. after which the engineering bedrock was assumed to start and the within motion obtained at a depth of 20 m. below the surface level.

So within motions derived after these analyses and for which the response spectra were shown in Figure 3.3 and 3.4 are selected as the input motion at the base of the soil profiles presented in Figure 3.2. As a general idea it was aimed to have sites with three different periods namely; 0.3, 0.4 and 0.7 seconds. These periods match with the natural periods of the structures explained in the following section.

For the site with the period 0.7 seconds, within motion was obtained not in the 60 m. but in the 20 m. because modeling the site as 60 m would have required much more time in FLAC analyses. Another way would have been constructing the site as 20 m. and with very low shear wave velocities so that again it had a period of 0.7 seconds; however this would have been unrealistic from the soil properties point of view.

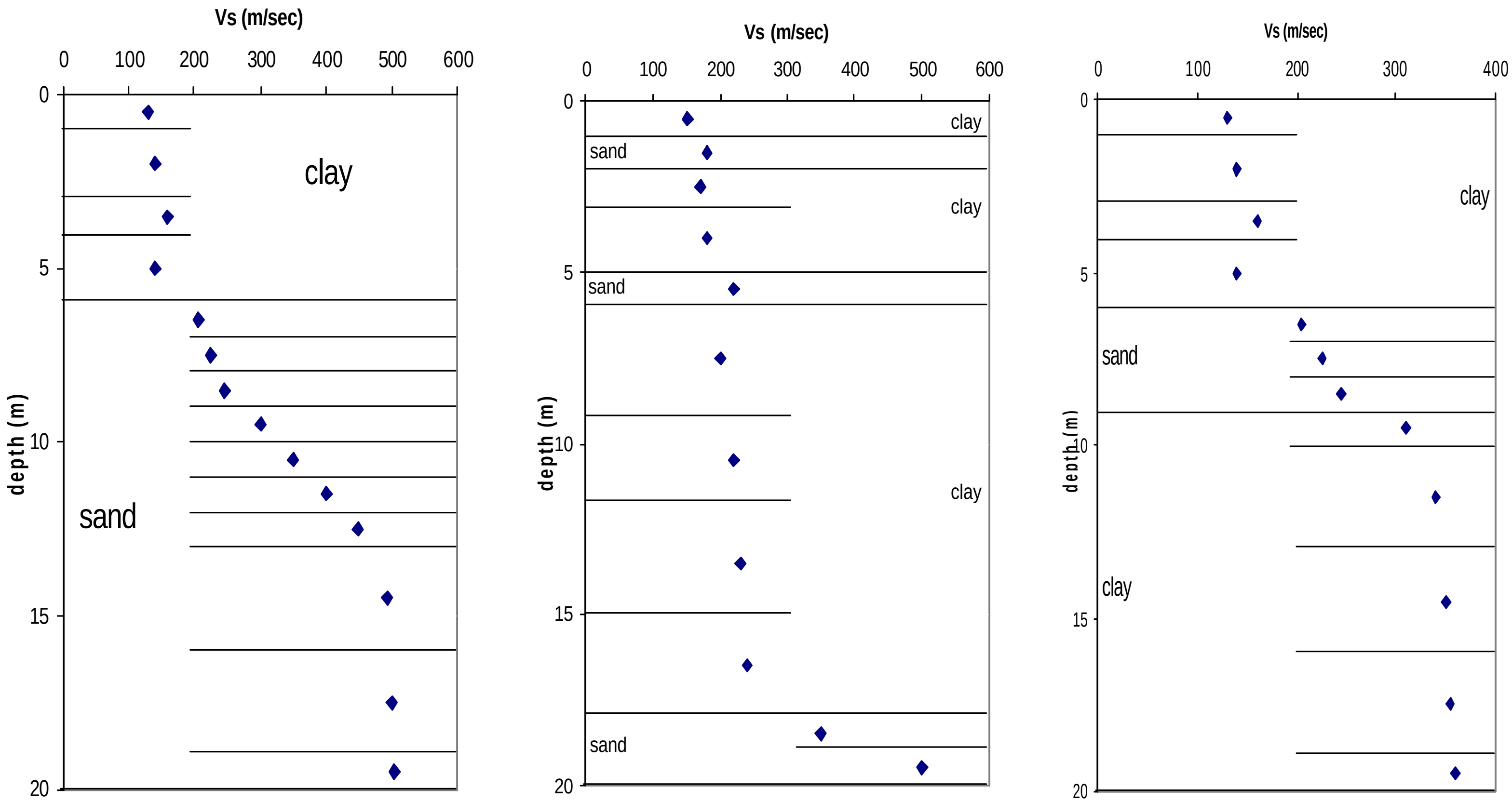
The natural period of the sites can be roughly estimated as:

$$T = \frac{4H}{V_e} \tag{3.7}$$

where H is the total thickness of site and V_e is the equivalent shear wave velocity of the site. V_e can be estimated as:

$$V_e = \frac{\sum_i h_i . V_i}{\sum_i h_i} \tag{3.8}$$

where h_i is the thickness of the each layer and V_i is the shear wave velocity of that layer.



First Site
T_p=0.29 sec

Second Site
T_p=0.42 sec

Third Site
T_p=0.7 sec

Figure 3.2 Shear Wave Profiles of Three Sites

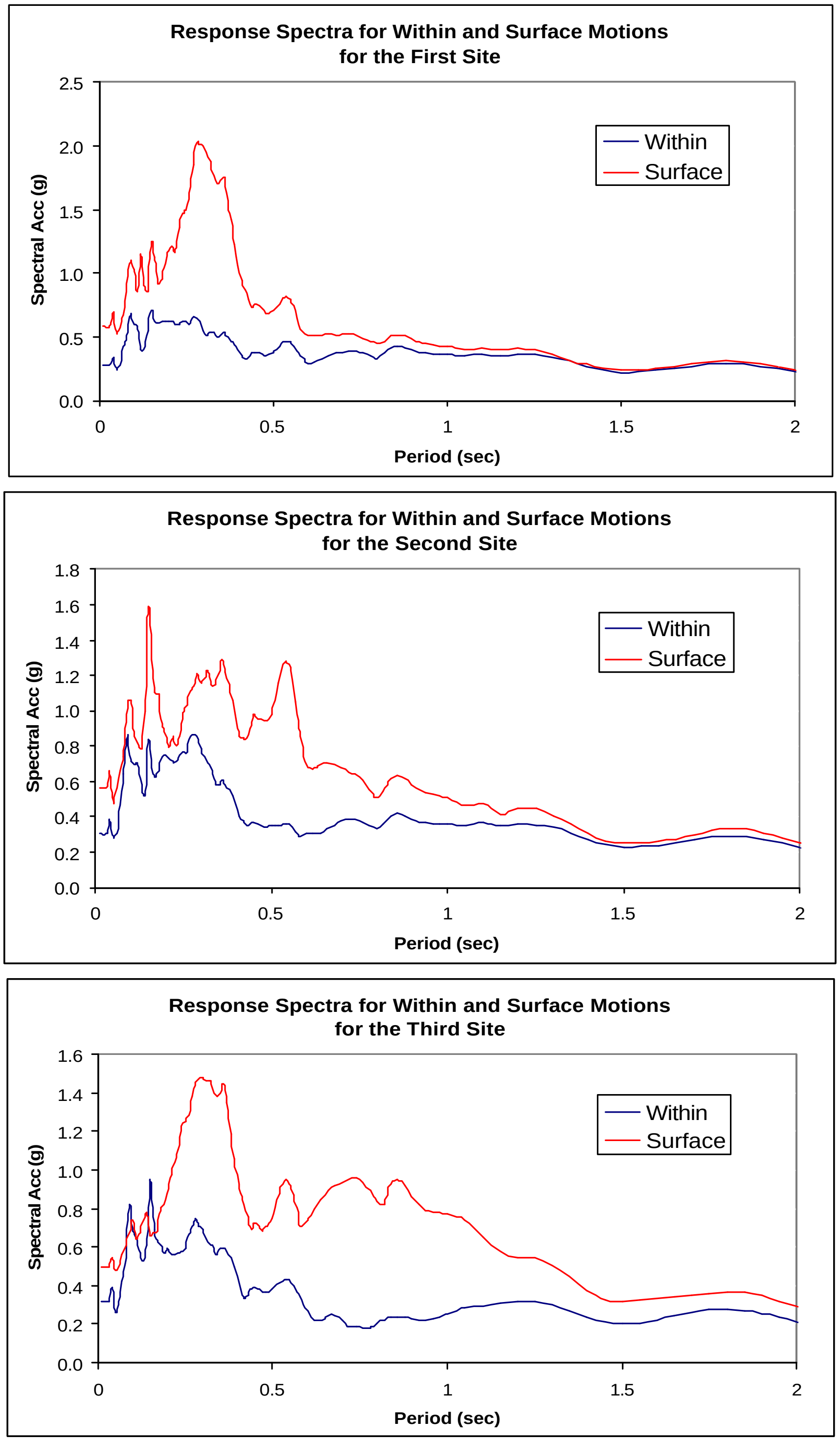


Figure 3.3 Response Spectra under Kocaeli Earthquake, 1999

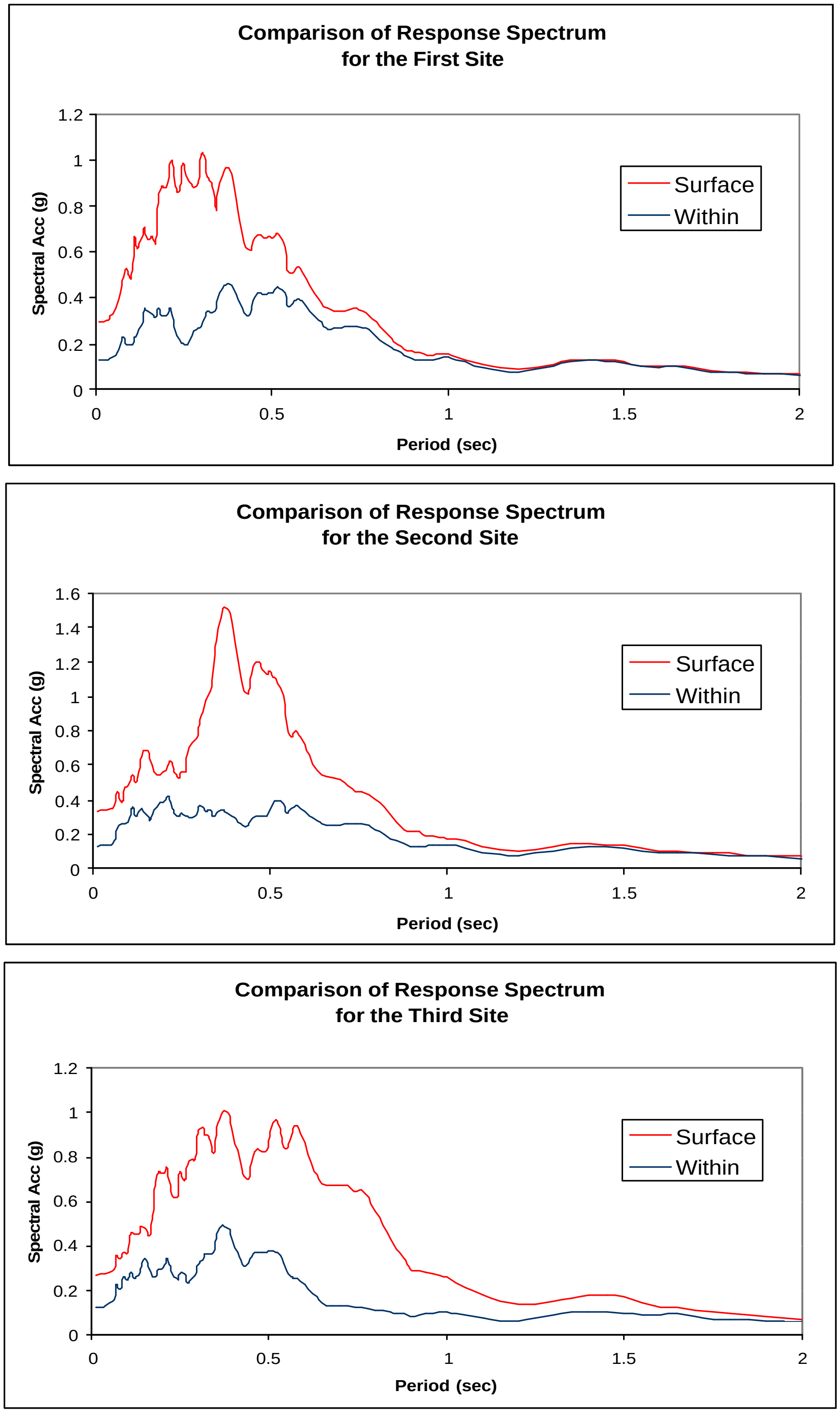


Figure 3.4 Response Spectra under El-Centro Earthquake, 1979

As can be seen from Figure 3.3 and 3.4, for the first site the amplification in the surface motion is around 0.3 sec, for the second site the amplification is around 0.4 sec and for the last site it is around 0.7 sec, all of which well corresponds with the site periods.

As discussed before in simple manner, the first site can be anticipated to be more dangerous from the resonance point of view for the structures having a natural period of 0.3 seconds, the second site seems to be more important for the structures with a natural period of 0.4 seconds, and for the last one a structure with a natural period of 0.7 sec would be more critical. However it is worth to state once more that there are some reasons because of which it is impossible to make a conclusion from this kind of a simplified study.

3.4 Definition of Structures

In this section an overview for the structural properties was given. Also the comparison between the non-isolated and isolated structures was done.

3.4.1 Structural Properties

As in common practice, to find out the forces and deformations, system matrices are obtained from the structural properties in FLAC. For structural elements young's modulus, second moment of area, cross sectional area and the mass density are to be defined. What is different in FLAC from other commercial programs from structural point of view is that the structure is not modeled as it was fixed base. Because the structure is modeled including the soil under itself, soil structure interaction and the flexible base effects can be taken into account.

The mass per unit length was taken to be 1.5 t/m as an average value and considering other structural properties defined above, the periods of the structures were found to be 0.32 sec. for three story structure, 0.43 sec. for four story one, 0.65 sec for the six story one, it they are considered to be fixed base structures. The fourth type is again a four story structure having the same structural period of the original four story structure; however additional 50% mass is included to stand for the special type of structures such as hospitals.

The foundation was modeled having the same structural properties of the beams. So the foundation can be visualized as a beam resting on the soil mainly responsible of distributing the structural load uniformly to the soil.

To model the isolators a much less stiff structural elements were used and a structural period of 2-3 seconds were achieved for the base isolated structures. These values are very close to typical natural periods of isolated structures. In common practice 3-4 seconds of a natural period are tried to be obtained to isolate the structure well enough. Also a damping value of 0.05 was used for both the non isolated and isolated structures which is rather a low value for isolated ones. These last two issues; modeling isolators as perfectly elastic materials and using a low damping value brought some simplifications for the modeling of the base isolated structures.

The isolated structures are discussed in the following section however it is worth to mention these two restrictions at this stage. Modeling an isolator elastically is always true for only some kind of isolators; for some, a bilinear behavior is to be modeled depending on the type of isolator. However this kind of bilinear behaviors can also be modeled implementing the equivalent linear model. So using linear model for all the types of base isolators will be a simplification.

Also it is a common practice to model the isolated structures with a much higher damping value to reduce the large amount of isolator displacements. This additional damping can be achieved by external dampers. However after numerous trials in FLAC, it was seen that for very high damping values, there would be some numerical instabilities. So it was decided to use low damping values.

Modeling the isolated structure with a low damping will only result in large base displacements but again the level of structural control can be achieved. Also considering the isolators behavior as an equivalent linear one will only cause small errors from the engineering point of view.

Four different types of structures considered in this study can be seen in Figure 3.5. The bay widths and the floor heights were chosen close to conventional type of structures. As a result the overturning moments were taken into account realistically. However because of the runtime limitations the number of the bays was limited.

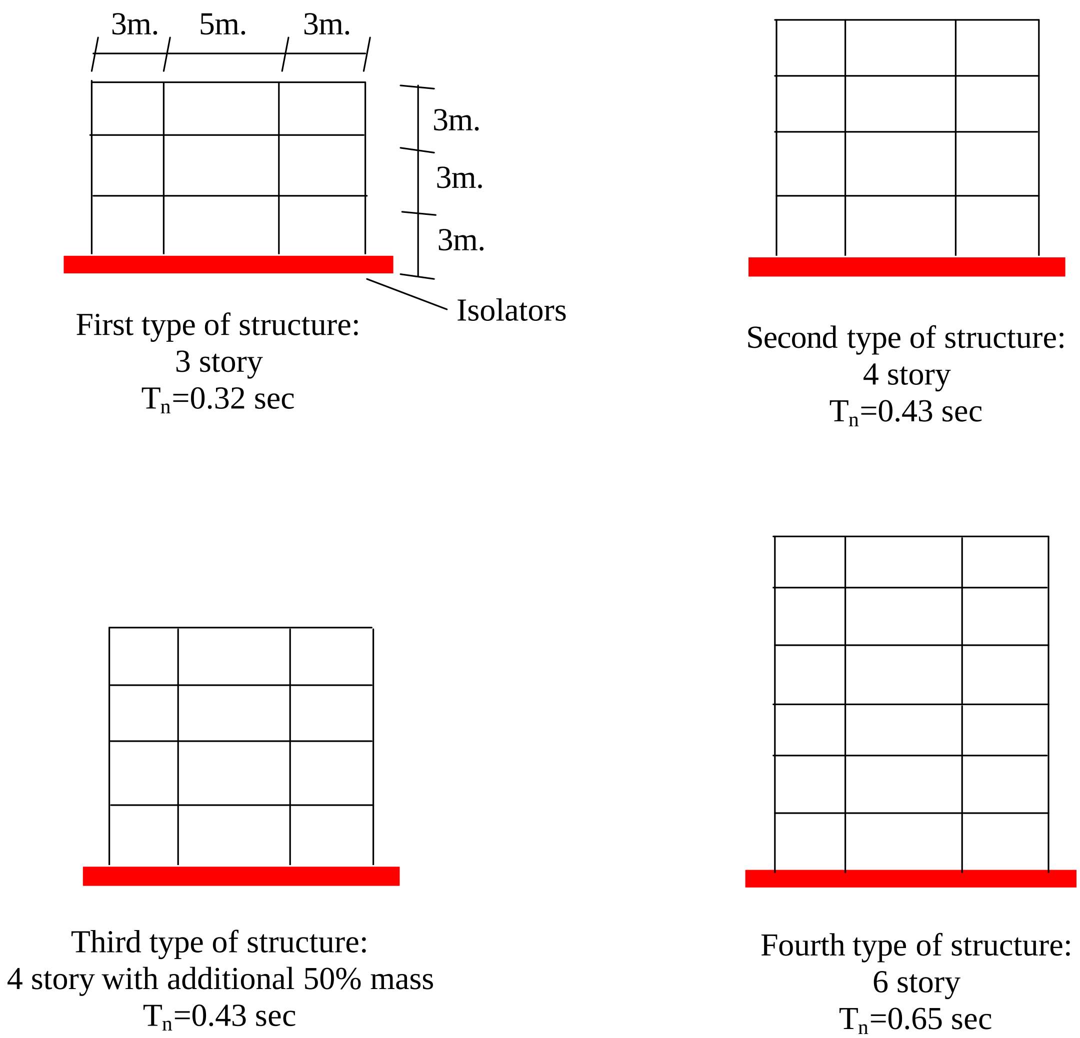


Figure 3.5 Structural Types Used in the Analyses

3.4.2 Base Isolated Structures

In this section the concept of base isolated structures was defined and some numerical examples were given to underline the effectiveness of isolation systems. Also short information was given on isolator devices.

Sometimes the level of the ground motions is considerably high causing severe structural damage. Many aseismic designs and technologies have been developed over the years to control the effects of earthquakes on structures. Seismic isolation is relatively recent and evolving technology compared to conventional asesmic methods.

The concept of protecting the structure from the damaging effects of an earthquake shaking by de-coupling its base from the ground is an attractive solution. This can be achieved using flexible supports. Even though the first proposals were made 100 years ago, it is the last several decades in which the base isolation systems have become a rational strategy for earthquake-resistant design. As a result; especially in Japan, USA, Italy and New Zealand base isolation has now advanced to a point where it is often considered for both new and existing structures.

In seismic isolation, the protection of structure itself and the contents against earthquake is achieved by de-coupling the structure from ground by using some flexible bearings and appropriate damping. In general, but not in all the cases, these bearings are mounted just between the base of the structure and the ground.

Seismic isolation technology is applicable to a wide range of civil structures. Some examples for the use of this technology in different types of structures can be given as follows: "Wellington Central Police Station" in New Zealand which is supposed to remain functional just after even during the earthquake. "The High-Tech R&D Centre, Obayashi Corporation", Japan, in which there are key equipment including super computers, for such a building the acceleration level must be kept under an acceptable level. "Salt Lake City and County Building", USA, in this kind of structures the seismic isolation technology suppress the conventional aseismic technologies because the structure should be kept as it is to protect its historical value. "The Mortaiolo Bridge", Italy, there is a huge number of isolated bridges especially in Italy. The use of rubber bearing for bridges is an already accepted technology to overcome the thermal effects so just taking the appropriate damping into consideration it turns out to be a simple and effective way to use seismic isolation technologies for bridges.

Despite in the conventional methods some local failures are permitted, in seismically isolated structures the superstructure is expected to remain in elastic zone which also allows architects to make more free designs with large openings, slender columns etc. In seismically isolated structures the floor accelerations are decreased incomparably which is the main concern for some structures. All these advantages are obtained at the expanse of high horizontal displacements concentrated in the base isolation layer. So extreme care must be taken to keep this clearance gap always operational.

The following quote from the Earthquake Engineering Research Institute (EERI 1996) is worth noting: “Workers in a large supply room within the USC hospital reported a gentle swaying motion with no disruption of supplies or materials falling off shelves as a result of the earthquake. In contrast, a pharmacy in an adjacent fixed base medical building reported substantial disruption of the supplies that were placed on the shelves.”

3.4.2.1 Nature of Isolated Structures

In this section it was aimed to perform some parametric studies on base isolated structures to give an insight on these systems.

The general idea in seismic isolation is shifting the period to a value which is larger than both the dominant period of non-isolated structure and ground motion. Typical earthquake motions have their peaks in the range of 0.2-0.6 seconds which coincide with the predominant period range of many ordinary structures. So shifting the period above 2-4 seconds, the acceleration imposed to the system can be decreased at the expanse of large horizontal displacement, in the order of 5-40 cm. However; it is possible to decrease these displacements by using additional damping. All these aspects of base isolation can be seen in the Figure 3.6. A sample input motion was chosen to be Northridge Earthquake, 1994. The response spectrum was constructed for three different damping values; 0.02 and 0.05 standing for the non isolated and 0.3 standing for the isolated structures.

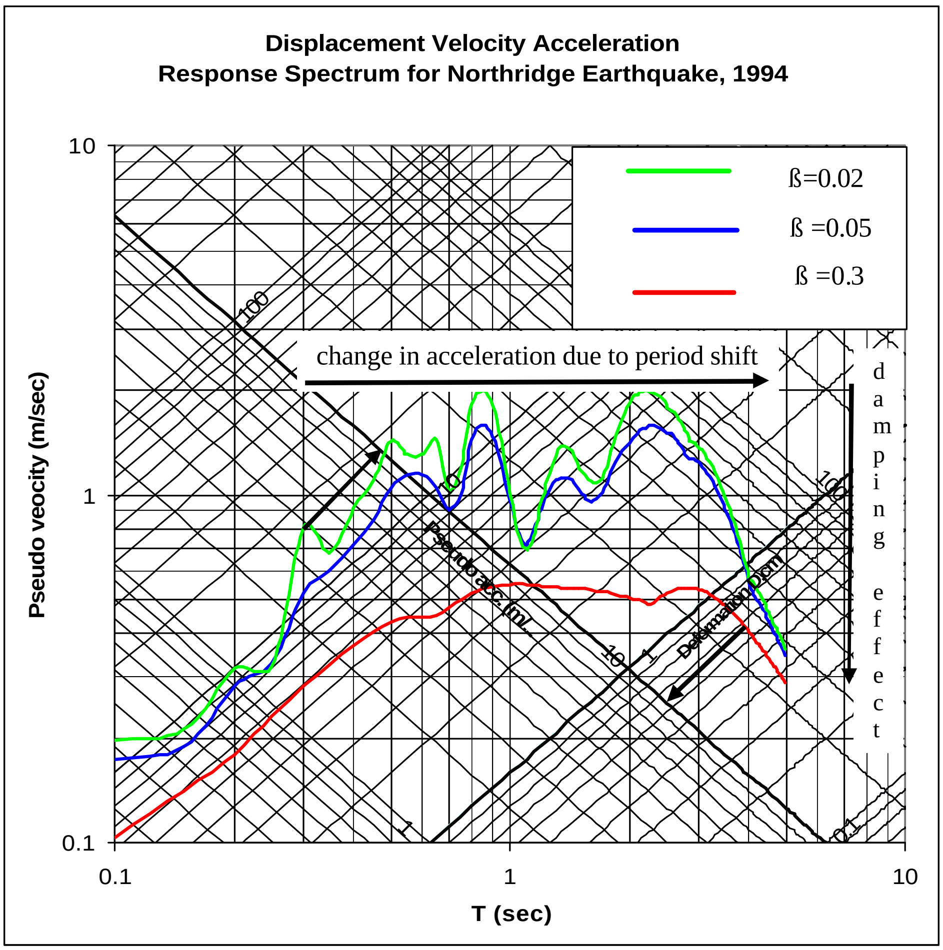
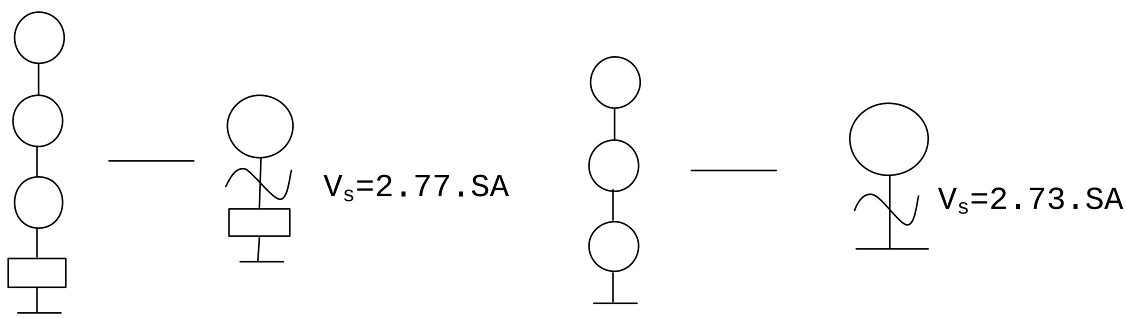


Figure 3.6 Displacement-Velocity-Acceleration Response Spectrum for Northridge Earthquake, 1994

3.4.2.2 Comparison of Isolated and Non-Isolated Structural Response

If two multi degree of systems are taken; one for fixed base and one for base isolated, and are simplified to two degree of freedom system for base isolated case and first mode single degree of system for fixed based, we can obtain some results to see the difference between base isolated and fixed based systems.



Base Isolated System
T=2 sec, ß=0.3

Fixed Based System
T=0.45 sec, ß =0.05 and 0.02

Figure 3.7 Lumped Mass Systems for Base Isolated and Fixed Base Cases

For response spectrum analyses 4 different earthquake motions were used which can be thought to have some own characteristics like; Kocaeli, 1999 and Northridge, 1994 motions are near field motions, Mexico, 1985 motion is rich in long period component so it specially turns out to be important for structures like base isolated. The response values for two systems (Figure 3.7) are summarized in Table 3.2.

Table 3.2 Response Values for Simplified Base Isolated and Fixed Based Systems

Name of EQ	El Centro, 1979	Northridge,1994	Kocaeli,1999	Mexico,1985
Base Displacement(cm)	7	15	13	21
Acc(m/sec^2), ß=0.3	0.7	1.5	1.3	1.2
Acc (m/sec^2), ß=0.05	5	13	8	2.2
Acc (m/sec^2), ß=0.02	8	15	10	2.5

From Figure 3.7, it can be seen that the forces which will be introduced to the superstructure have nearly the same coefficients; 2.77 and 2.73, so the force will be directly proportional to the spectral acceleration values. Then it can be concluded that especially for the near field earthquakes the reduction is nearly 8-10 times; however for a very special case like Mexico earthquake the reduction coefficient is only 2. Even for this special case if the period of the base isolated system is elongated up to 3-4 sec the reduction coefficient will be considerably high.

3.4.2.3 Isolator Devices

Isolator devices are used to provide adequate horizontal flexibility, centering force and damping. The main function of an isolation system is to support structure while providing a high horizontal flexibility so that the overall structural period turns out to be in the region where the earthquake caused accelerations are low. However the horizontal displacement rises up to around 40cm. which can be reduced to 5cm-10cm with high level of damping.

The common part of isolator devices is rubber bearings which mainly supplies horizontal flexibility and restoring force. If bearings were used just composed of rubber, there would be bulging problem. To overcome this problem in other words to enhance vertical stiffness, thin steel plates are placed in between rubber.

The average damping for a natural rubber bearing can be considered as 5% which is quite small in base isolated structure. To supply additional damping high damping rubber bearings can be used in which the damping characteristic is viscous type. This kind of damping can also be achieved by using external dampers. Another way is to consider yielding of metals like steel or lead. In average they yield after 2-3 cm of deformation. So the force-deformation characteristic will be bilinear supplying hysteretic damping.

One of the most popular devices is lead rubber bearings because using only one device, three functions can be achieved namely; vertical support, horizontal flexibility and hysteretic damping by the plastic deformation of lead. Before yielding of lead core stiffness of the isolator is due to lead core and rubber, after the yielding stiffness of the isolator is only due to rubber.

3.5 Characteristics of the System Used in the Program

In this chapter the program used for the numerical analyses explained briefly. Afterwards two main issues in numerical modeling and soil structure interaction namely; the effect of mesh size and the boundaries of the system were discussed.

3.5.1 Overview of the Program

FLAC is a two-dimensional explicit finite difference program for engineering mechanics computation. This program simulates the behavior of materials that may undergo plastic flow when their yield limits are reached. Materials are represented by elements, or zones, which form a grid that is adjusted by the user to fit the shape of the model. Each element behaves according to a prescribed linear or nonlinear stress/strain law in response to the applied forces or boundary restraints. The material can yield and flow and the grid can deform (in large-strain mode) and move with the material that is represented.

If FLAC and more-common method of finite elements for numerical modeling are to be compared; both methods translate a set of differential equations into matrix equations for each element, relating forces at nodes to displacements at nodes.

The finite difference method is perhaps the oldest numerical technique used for the solution of sets of differential equations, given initial values and/or boundary values. In the finite difference method, every derivative in the set of governing equations is replaced directly by an algebraic expression written in terms of the field variables (e.g., stress or displacement) at discrete points in space; these variables are undefined within elements. In contrast, the finite element method has a central requirement that the field quantities (stress, displacement) vary throughout each element in a prescribed fashion, using specific functions controlled by parameters.

Even though we want FLAC to find a static solution to a problem, the dynamic equations of motion are included in the formulation. One reason for doing this is to ensure that the numerical scheme is stable when the physical system being modeled is unstable. With nonlinear materials, there is always the possibility of physical instability. FLAC models this process directly, because inertial terms are included. In contrast, schemes that do not include inertial terms must use some numerical procedure to treat physical instabilities. In an implicit method (which is commonly used in finite element programs), every element communicates with every other element during one solution step: several cycles of iteration are necessary before compatibility and equilibrium are obtained.

Since a global stiffness matrix is not necessary to form, it is a trivial matter to update coordinates at each time step in large-strain mode which is used during plastic flow. The incremental displacements are added to the coordinates so that the grid moves and deforms with the material it represents. This is termed a “Lagrangian” formulation, in contrast to an “Eulerian” formulation, in which the material moves and deforms relative to a fixed grid.

Finite Element Model, FEM codes usually represent steady plastic flow by a series of static equilibrium solutions. The quality of the solution for increasing applied displacements depends on the nature of the algorithm used to return stresses to the yield surface, following an initial estimate using linear stiffness matrices. The best FEM codes will give a limit load (for a perfectly plastic material) that remains constant with increasing applied displacement. The solution provided by these codes will be similar to that provided by FLAC. However, FLAC’s formulation is simpler because no algorithm is necessary to bring the stress of each element to the yield surface: the plasticity equations are solved exactly in one step. Therefore, FLAC may be more robust and more efficient than some FEM codes for modeling steady plastic flow. FLAC is also robust in the sense that it can handle any constitutive model with no adjustment to the solution algorithm; many FEM codes need different solution techniques for different constitutive models. (FLAC 4.0 User’s Guide, 1998)

3.5.2 System Boundaries and Effects of Mesh Size

The calculation methodology used in FLAC is based on the explicit finite difference scheme as discussed above to solve the full equations of motion, using lumped grid point masses derived from the real density of surrounding. This formulation can be coupled to the structural element model, thus permitting analyses of soil-structure interaction.

However there are two main issues to pay attention. These are:

1. Boundary conditions;

2. Wave transmission through the model.

3.5.2.1 Boundary Conditions

FLAC models a region of material subjected to external or internal dynamic loading by applying a dynamic input boundary condition at either the model boundary or at internal grid points. Wave reflections at model boundaries are minimized by specifying either quiet (viscous), free-field or three-dimensional radiation-damping boundary conditions.

The modeling of geo-mechanics problems involves media which are better represented as unbounded. Surface and near-surface structures are assumed to lie on a half-space. However, because of the limitations some artificial boundaries are to be used. Numerical methods relying on the discretization of a finite region of space require that appropriate conditions be enforced at the artificial numerical boundaries. In static analyses, fixed or elastic boundaries can be realistically placed at some distance from the region of interest.

In dynamic problems, however, such boundary conditions cause the reflection of outward propagating waves back into the model and do not allow the necessary energy radiation. The use of a larger model can minimize the problem, since material damping will absorb most of the energy in the waves reflected from distant boundaries. However, this solution leads to a large computational burden. The alternative is to use quiet (or absorbing) boundaries. Several formulations have been proposed. The viscous boundary developed by Lysmer and Kuhlemeyer (1969) is used in FLAC. It is based on the use of independent dashpots in the normal and shear directions at the model boundaries. The scheme has the advantage that it operates in the time domain.

The quiet-boundary scheme proposed by Lysmer and Kuhlemeyer (1969) involves dashpots attached independently to the boundary in the normal and shear directions. The dashpots provide viscous normal and shear tractions given by

$$t_n = -r.C_p.v_n \tag{3.9}$$

$$t_s = -r.C_s.v_s \tag{3.10}$$

where v_n and v_s are the normal and shear components of the velocity at the boundary;
r is the mass density; and
C_p and C_s are the p and s wave velocities.

These viscous terms can be introduced directly into the equations of motion of the grid points lying on the boundary. A different approach, however, was implemented in FLAC, whereby the tractions t_n and t_s are calculated and applied at every time step in the same way that boundary loads are applied. However it would give better results to use free field boundaries rather than quiet boundaries when the dynamic source is applied as a boundary condition at the top or base, because the wave energy will “leak out” of the sides. (FLAC 4.0 User’s Guide, 1998)

3.5.2.1.1 Free-Field Boundaries

Numerical analyses of the seismic response of surface structures require the discretization of a region of the material adjacent to the foundation. The seismic input is normally represented by plane waves propagating upward through the underlying material. The boundary conditions at the sides of the model must account for the free-field motion which would exist in the absence of the structure. In some cases, elementary lateral boundaries may be sufficient. These boundaries should be placed at sufficient distances to minimize wave reflections and achieve free-field conditions. For soils with high material damping, this condition can be obtained with a relatively small distance (Seed et al. 1975). However, when the material damping is low, the required distance may lead to an impractical model. An alternative procedure is to “enforce” the free-field motion in such a way that boundaries retain their non-reflecting properties i.e., outward waves originating from the structure are properly absorbed. A technique of this type was developed for FLAC, involving the execution of a one-dimensional free-field calculation in parallel with the main-grid analyses.

The lateral boundaries of the main grid are coupled to the free-field grid by viscous dashpots to simulate a quiet boundary and the unbalanced forces from the free-field grid are applied to the main-grid boundary. Both conditions are expressed in Equation 3.11 and Equation 3.12.

$$F_x = -[r.C_p.(v_x^m - v_x^{ff}) - s_{xx}^{ff}]\Delta S_y \tag{3.11}$$

$$F_y = -[r.C_s.(v_y^m - v_y^{ff}) - s_{xy}^{ff}]\Delta S_y \tag{3.12}$$

where r : density of material along vertical model boundary;

C_p : p wave speed at the left-hand boundary;

C_s : s wave speed at the left-hand boundary;

ΔS_y : mean vertical zone size at boundary grid point;

v_x^m : x velocity of grid point in main grid at left boundary;

v_y^m : y velocity of grid point in main grid at left boundary;

v_x^{ff} : x velocity of grid point in left free field;

v_y^{ff} : y velocity of grid point in left free field;

s_{xx}^{ff} : mean horizontal free-field stress at grid point;

s_{xy}^{ff} : mean free-field shear stress at grid point.

In this way, plane waves propagating upward suffer no distortion at the boundary because the free-field grid supplies conditions that are identical to those in an infinite model. If the main grid is uniform, and there is no surface structure, the lateral dashpots are not exercised because the free-field grid executes the same motion as the main grid. However, if the main-grid motion differs from that of the free field (due to a surface structure that radiates secondary waves), then the dashpots act to absorb energy in a manner similar to the action of quiet boundaries. The free-field model consists of a one-dimensional “column” of unit width, simulating the behavior of the extended medium.

This characteristic was examined by several analyses, firstly the lateral boundaries were taken quite far away from the structure so that the energy would have been absorbed by the mechanical damping and the effect of the surface structure would have been diminished; afterwards the lateral boundaries were taken rather close to the surface structure, it was observed that the results obtained in these two ways were quite close to each other proving the effectiveness of the free field boundaries. So in the final analyses the lateral boundaries were chosen to be close to the structure to decrease the run time.

3.5.2.1.2 Three-Dimensional Radiation Damping

A vibrating structure located on the surface of the modeled region creates a disturbance both in the plane of analyses and in the out-of-plane direction. The energy radiated in-plane is reasonably absorbed by the quiet boundary condition; however, in a three-dimensional system, energy would be radiated in the out-of-plane direction. To represent this effect approximately, dashpots are connected from all grid points in the main grid to corresponding grid points in the free field (although the force is not applied to the free-field grid). This mechanism is termed three-dimensional radiation damping. The 3D damper acts on the difference between the actual particle velocity under the structure and the free field velocity around the model region. The scheme is identical to that described by Lysmer et al. (1975). The dashpot constant, c, has the value:

$$c = \frac{2.r.C_s^{ff}}{W} \qquad (3.13)$$

where c: coefficient of 3D damping;

C_s^{ff} : free-field shear wave velocity;

W: out-of-plane width of structure.

3.5.2.2 Sizes of the Meshes

Numerical distortion of the propagating wave can occur in dynamic analyses as a function of the modeling conditions. Both the frequency content of the input wave and the wave-speed characteristics of the system will affect the numerical accuracy of wave transmission. Kuhlemeyer and Lysmer (1973) showed that for accurate representation of wave transmission through a model, the spatial element size, Δl, must be smaller than approximately one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave i.e.,

$$\Delta l \leq \frac{l}{10} \qquad (3.14)$$

where l is the wave length associated with the highest frequency component that contains appreciable energy.

Expressing l in the form of shear wave velocity, V_s and the highest frequency introduced to the system f_{max} Equation 3.14 can be written as:

$$\Delta l \leq \frac{V_s}{10.f_{\max}} \qquad (3.15)$$

This requirement may necessitate a very fine spatial mesh and a corresponding small time step. The consequence is that reasonable analyses may be time and memory consuming. In such cases, it may be possible to adjust the input by recognizing that most of the power for the input history is contained in lower frequency components. By filtering the history and removing high frequency components, a coarser mesh may be used without significantly affecting the results.

However in this study no filtering was performed for the input motions. In the Figure 3.8 the power spectra of the input motions were presented. As can be seen from Figure 3.8 the highest frequency is 20 Hz. The shear wave velocities were also given in the previous sections, taking the lowest V_s as 130 m/sec and inserting these values to the Equation 3.15; the maximum mesh size that is found to be 0.65 m. In this study the mesh sizes were chosen to be 0.5 m.

As summarized in Figure 3.9, the sizes of the soil medium were taken to be 15m x 20m, and this medium was discretized by using 0.5m x 0.5m grids which were proven to be small enough to transmit all the frequency components of the input motions. The sizes and the characteristics of the structure were also presented in the Figure 3.9. Additionally, the free field boundaries and the 3D damping were shown schematically.

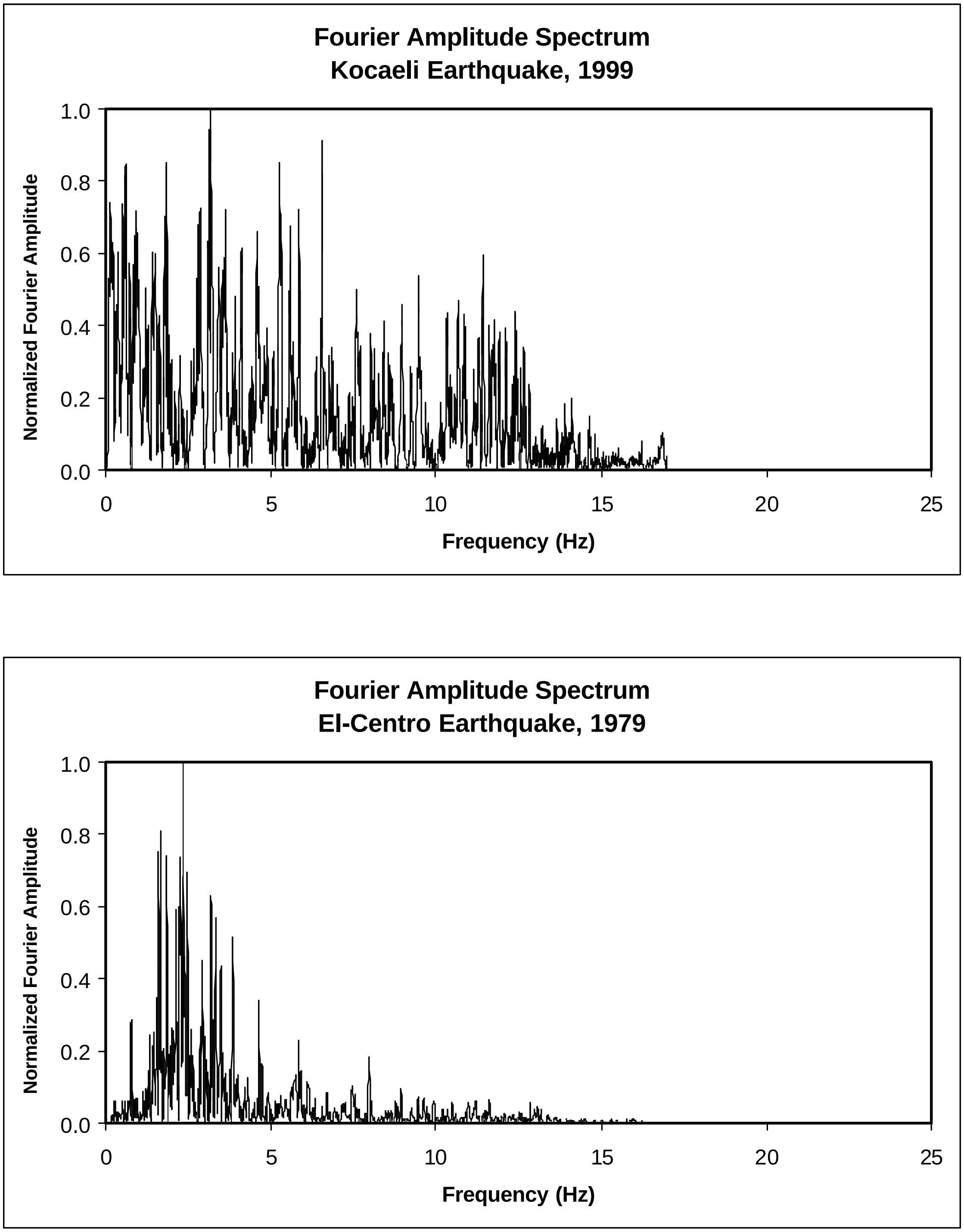


Figure 3.8 Fourier Amplitude Spectra for the Input Motions Used in the Analyses

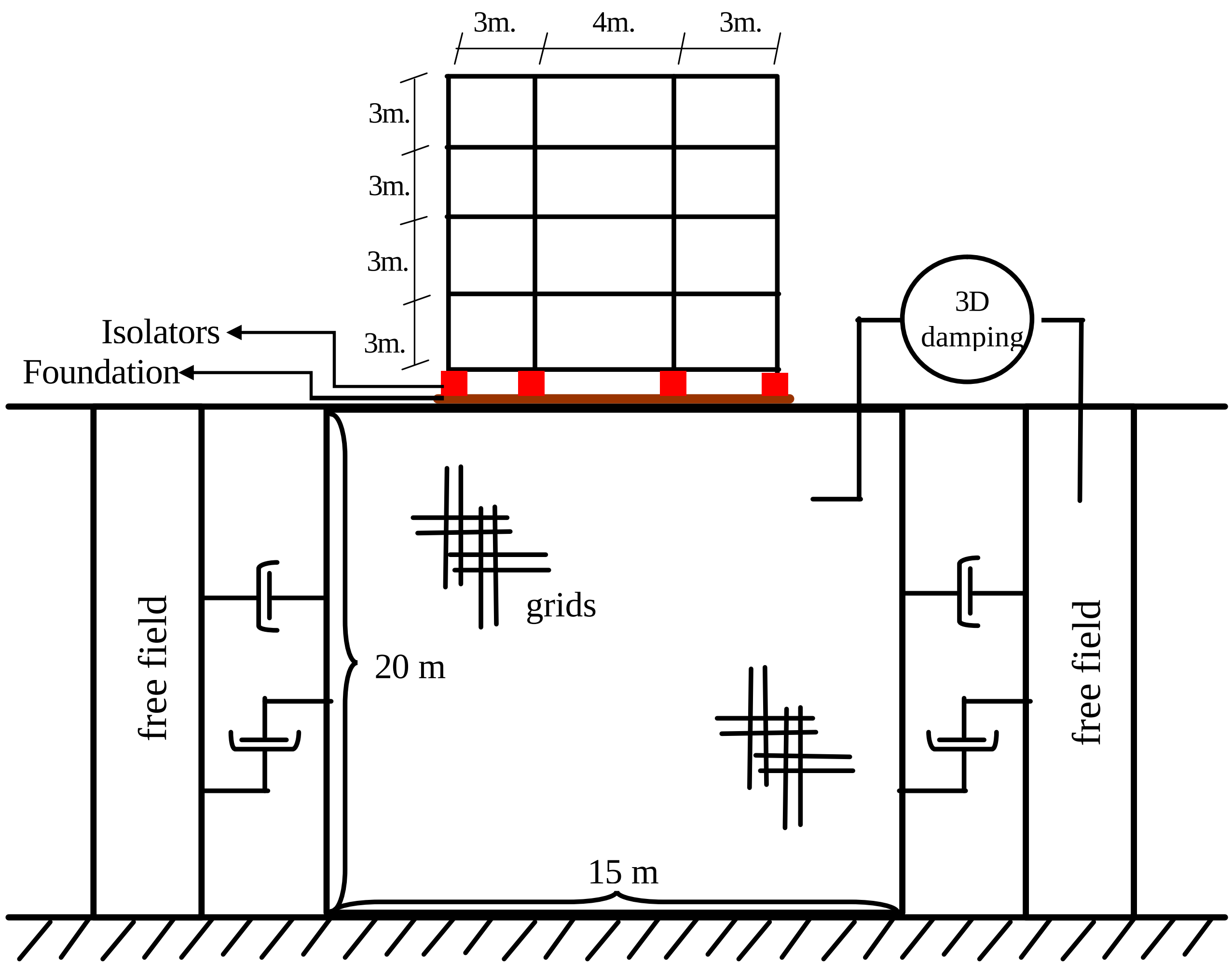


Figure 3.9 Schematic View of the System Used in the Analyses

CHAPTER IV

BEHAVIOUR OF THE WHOLE SYSTEM BOTH FROM GEOTECHNICAL AND STRUCTURAL ASPECTS

4.1 Introduction

In this chapter the results obtained from the numerical analyses by FLAC were given and discussed. In the previous chapters the characteristics and details were examined regarding the input motions, the sites and the structures, separately. In Figure 4.1 all these components were combined to give an overall sense about the whole system analyzed. As discussed before the results were obtained for each different combination of the input motions, sites and structures; also the free field analyses were performed for all the sites. The structures were analyzed both as non-isolated and isolated. The third type of structure has the same natural period and a 50% additional weight to the second type of structure.

The characteristics of the sites and the structures can be found in Table 4.1 and Table 4.2. The figures given in these tables were obtained from the SHAKE analyses for the soil strata and from the SAP analyses for the structures. As stated in the previous chapters these are just the figures to have a rough idea about the whole system.

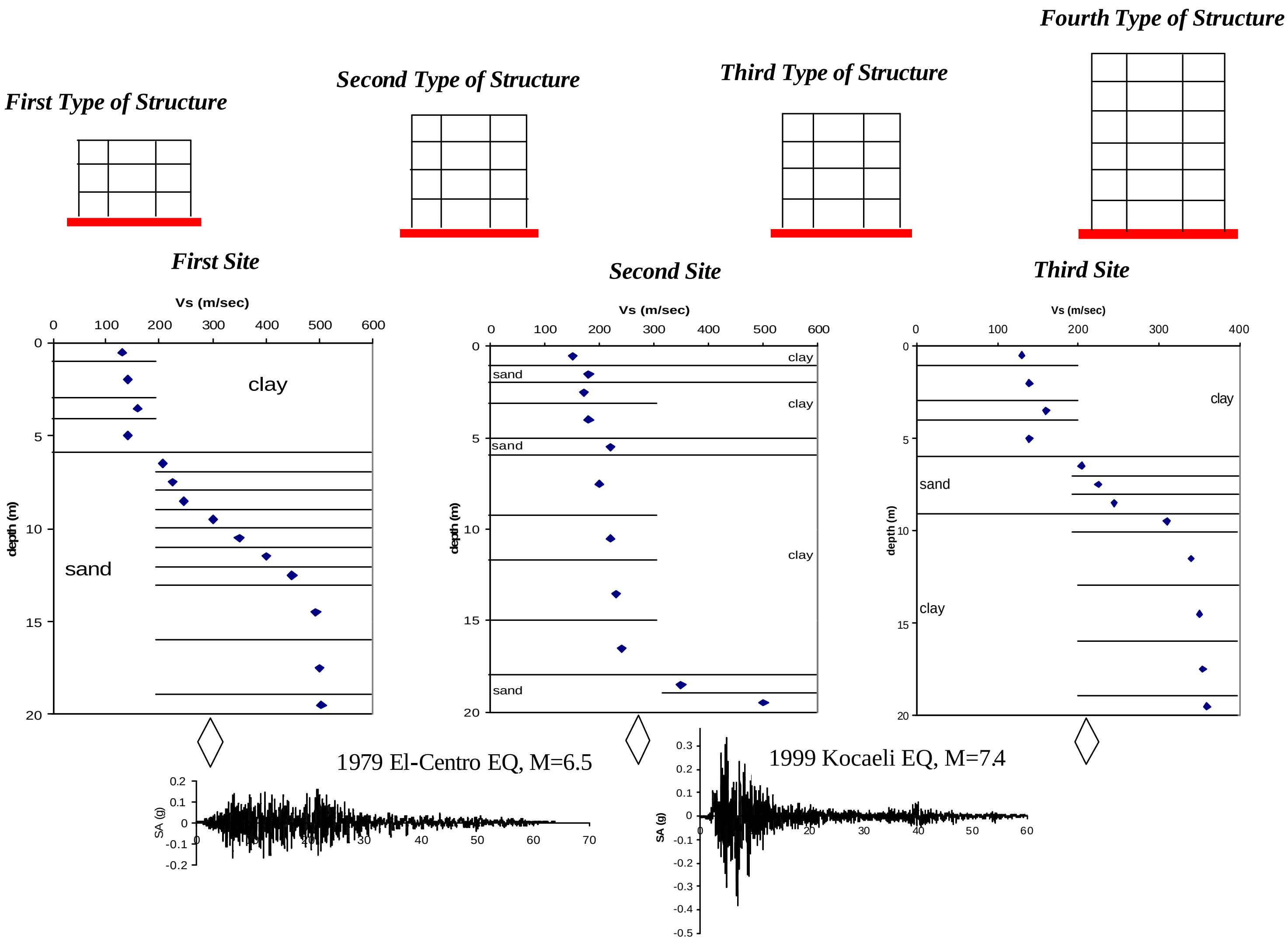


Figure 4.1 Overall System Analyzed

Table 4.1 Natural Periods of the Structures

	Natural Period of the Non-Isolated Structure	Natural Period of the Isolated Structure
First Type of Structure	0.32	2.1
Second Type of Structure	0.43	2.6
Third Type of Structure	0.43	2.6
Fourth Type of Structure	0.65	2.9

Table 4.2 Natural periods of the Sites

	Natural Periods of the Sites
First Site	0.29
Second Site	0.42
Third Site	0.7

4.2 The Geotechnical Aspects

In this section firstly a comparison between the SHAKE and FLAC analyses for free field were made; secondly the liquefaction assessments for the sites including the effects of the structures were done by using the results obtained from the FLAC analyses.

4.2.1 Site Response by FLAC and Equivalent Linear Analysis

The “equivalent-linear” method is a common practice in earthquake engineering for modeling wave transmission in layered sites and dynamic soil-structure interaction.

Since this method is widely used, and the fully nonlinear method embodied in FLAC is not, it is worth pointing out some of the differences between the two methods.

In the equivalent-linear method (Seed and Idriss 1969), a linear analysis is performed, with some initial values assumed for damping ratio and shear modulus in the various regions of the model. The maximum cyclic shear strain is recorded for each element and used to determine new values for damping and modulus, by using the curves that relate damping ratio and secant modulus to amplitude of cycling shear strain. The new values of damping ratio and shear modulus are then used in a new numerical analysis of the model. The whole process is repeated several times, until there is no further change in properties. At this point, it is said that "strain-compatible" values of damping and modulus have been found, and the analysis using these values is representative of the response of the real site.

In contrast, only one run is done with a fully nonlinear method, because nonlinearity in the stress-strain law is followed directly by each element. Provided that an appropriate nonlinear law is used, the dependence of damping and apparent modulus on strain level is automatically modeled.

Both methods have their strengths and weaknesses. The equivalent-linear method makes many assumptions on the real case but is user-friendly. The fully nonlinear method correctly represents the physics but demands more user involvement.

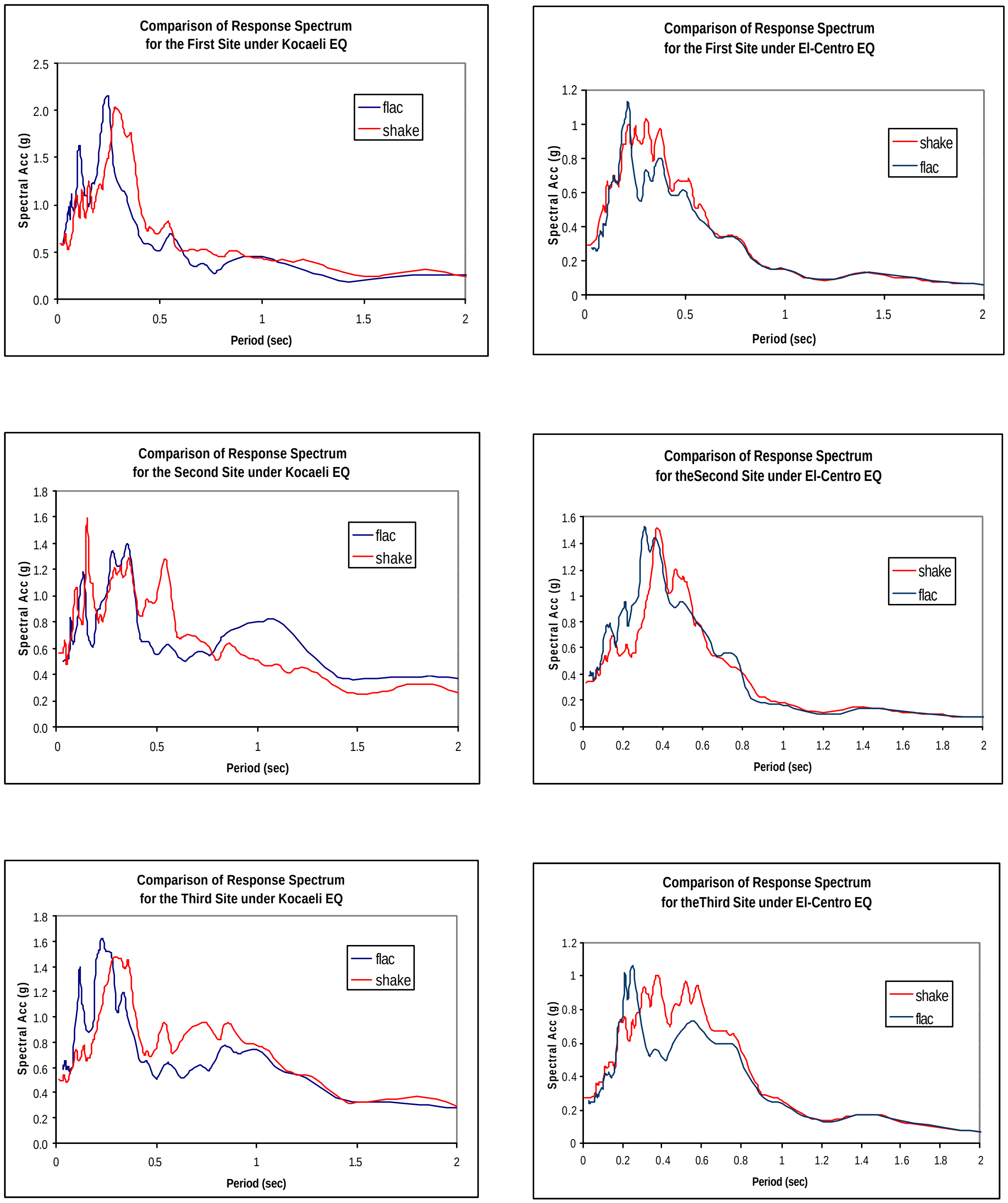


Figure 4.2 Comparison of Response Spectra Obtained from SHAKE & FLAC
(free field motions) under two Different Input Motions

As can be seen in Figure 4.2, the match between the results of two fundamentally different approaches is satisfactory. This agreement was observed to be better if an elastic model was chosen in FLAC runs.

Another observation from the estimated response spectra is that the match in the El-Centro case is much better compared to the ones from Kocaeli record which could be mainly due to the low levels of shaking in El-Centro input motion. It can be observed in Figure 4.2 that the response values are higher for the Kocaeli Earthquake record.

4.2.2 Assessment of Liquefaction Potential

As stated in the previous sections, liquefaction assessments have turned out to be a very controversial aspect of the geotechnical earthquake engineering after the big urban earthquakes occurred in the last several decades causing liquefaction related problems. There are several methods to assess the potential of liquefaction at a site as the cyclic stress approach or probabilistic approach. In this study the liquefaction potential was examined on the basis of simplified procedure (Seed and Idriss, 1971).

4.2.2.1 Liquefaction Assessment by Means of Cyclic Stress Ratio

In the last four decades many advances in the state of practice related with the liquefaction phenomena have been achieved mainly by the pioneering researches of Prof. H.B. Seed.

A typical liquefaction analysis for level ground conditions in the free field involves three steps:

1) The normalized shear stresses required to trigger soil liquefaction are determined;

2) The normalized shear stresses induced by the earthquake are calculated;

3) The induced stress is compared with the stress causing liquefaction to determine if liquefaction will trigger.

The normalized shear stresses required to trigger soil liquefaction may be determined either by laboratory cyclic shear testing on frozen samples or by correlations with in-situ test results. In common practice the SPT correlations are widely used to determine the liquefaction resistance.

There are correlation charts between liquefaction resistance and SPT blow count proposed by Seed et al. (1984). Using these charts the cyclic stress ratio $\frac{t_{ave}}{s'}$ required for the liquefaction can be determined for different values of SPT blow counts. t_{ave} is the equivalent uniform cyclic shear stress induced by the earthquake and s' is the vertical effective stress.

The cyclic stress ratio induced by the earthquake can be determined by using the simplified equation developed by Seed and Idriss (1971):

$$\frac{t_{ave}}{s'} = 0.65.\frac{a_{\max}}{g}.\frac{s}{s'}.r_d \qquad (4.1)$$

where a_{max} is the peak horizontal acceleration at the ground surface, g is the acceleration of gravity, s and s' are total and effective vertical stresses respectively, and r_d is a factor to take into account the deformability of the soil and within a depth of 6-9 m below the ground surface its value is around 0.8-1.0.

Finally to find out if liquefaction in the free field is possible or not, the shear stress induced by the earthquake is compared with the shear stress required to cause liquefaction. However; there are at least two factors which should be expected to change the cyclic stress ratio induced by earthquake under the structure from that one which would be expected in the free field. These are:

1) The change in the vertical effective stress induced by the structure,
2) The influence of the structure on shear stress introduced to the soil.

4.2.2.2 Structural Induced Cyclic Stress Ratios

In this section mainly the effect of structure on the liquefaction potential was discussed and the numerical results were given.

Because of the two main reasons stated above, it turns out to be important how much the shear stress induced by the structure in the ratio $\frac{t_{ave}}{s'}$ will increase. According to the ratio of the increases in these two values, it is possible to compare the cyclic stress ratio under structure and in the free field. The effect of the change in the vertical stress is obvious because of the additional weight of structure. Considering the simple equation $\frac{t_{ave}}{s'}$, the increase in the vertical stress will always decrease the cyclic stress ratio. The main question is if this decrease will be compensated by the increase in the shear stress because of the structure.

Rollins and Seed (1990) proposed a simplified procedure for the effect of building. According to this study, the shear stress induced by structure to soil is correlated by the weight of the structure, spectral acceleration of the structure and its flexibility. So a simple equation for the base shear force is written as:

$$V_{\max} \approx 0.8.(\frac{S_a}{g}).W \tag{4.2}$$

where S_a is spectral acceleration of the structure, W is the weight of the structure.

It is considered to be $t_{ave} \approx 0.65.t_{\max}$ and after some simplifications, they states that the cyclic stress ratio can be found as:

$$\left(\frac{t_{ave}}{s'}\right)_s \approx 0.52.\left(\frac{S_a}{g}\right) \tag{4.3}$$

Taking the water level near to the ground surface, the cyclic stress ratio developed beneath the building and in the free field will be the same if

$$\left(\frac{S_a}{a_{\max}}\right) \approx 2.75 \tag{4.4}$$

Rollins and Seed (1990) states that if the spectral acceleration ratio, $\left(\frac{S_a}{a_{\max}}\right)$, corresponding to the building period is higher than 2.75, the induced cyclic stress ratio would be higher beneath the building than in the free field. If the ratio $\left(\frac{S_a}{a_{\max}}\right)$ is less than about 2.75, then the induced cyclic stress ratio will be lower beneath the building and the potential for liquefaction will tend to decrease.

Also according to their findings, for the buildings with periods greater than about 0.75 sec the liquefaction potential will be lower beneath the building than in the free field. However; for the buildings with a period between 0.1 and 0.5 sec the liquefaction potential might be worse than in the free field.

After explaining this simplified procedure, the numerical results were given in the Figure 4.3 obtained from the analyses. These results are just for one site and under Kocaeli input motion. Rest of the results can be seen in Appendix A. As can be seen in Figure 4.3, for each case the cyclic stress ratios $\frac{t_{ave}}{s'}$ were plotted at four different points under the structure up to a depth of 10 m.

It can be seen that after a depth of nearly 10 m the effect of the structure on the cyclic stress turns out to be invisible and the value of the cyclic stress ratio in the free field and under the structure comes out to be same so an upper limit was put for the figures.

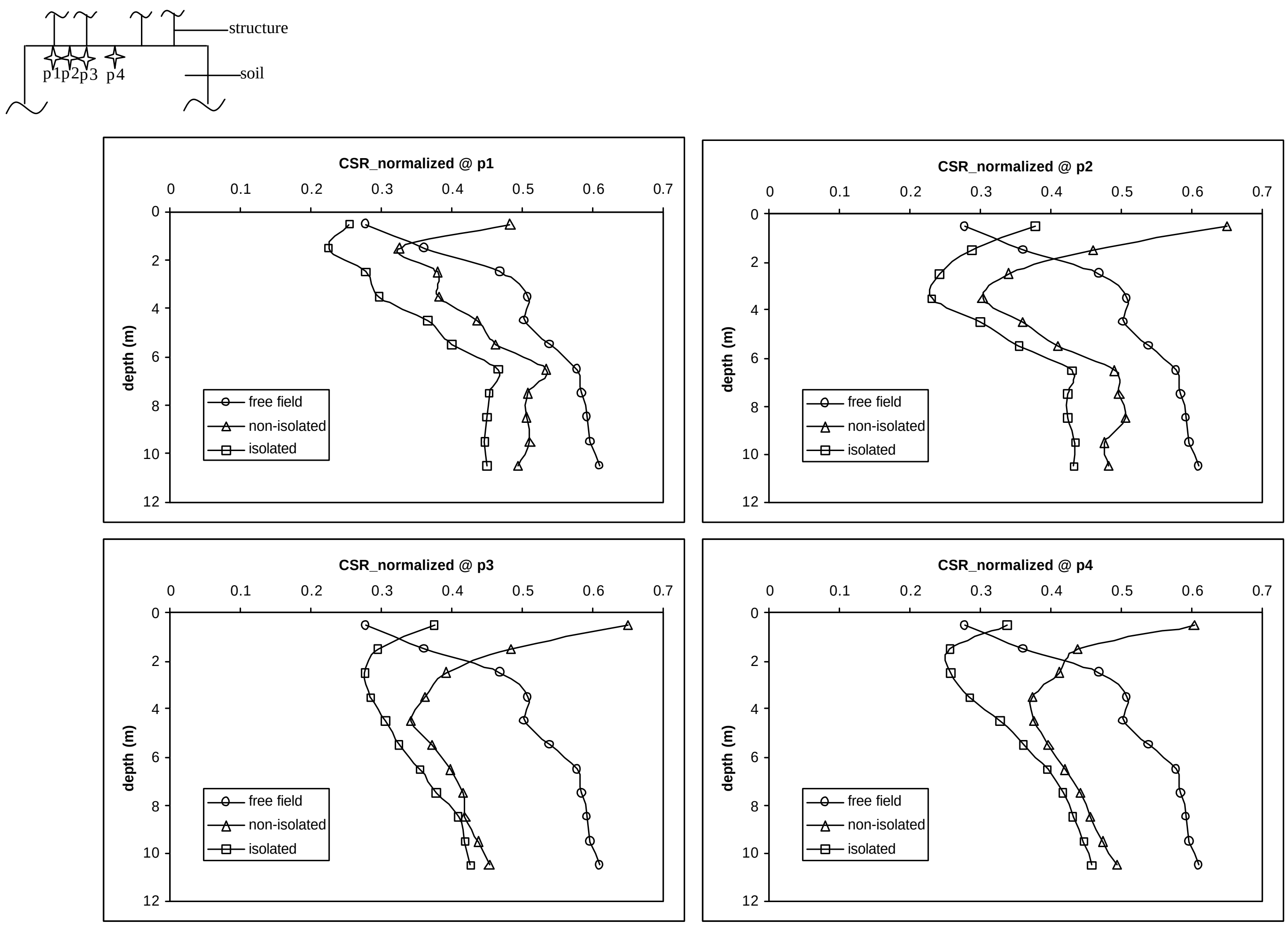


Figure 4.3.a First Type of Structure with second site under Kocaeli Earthquake

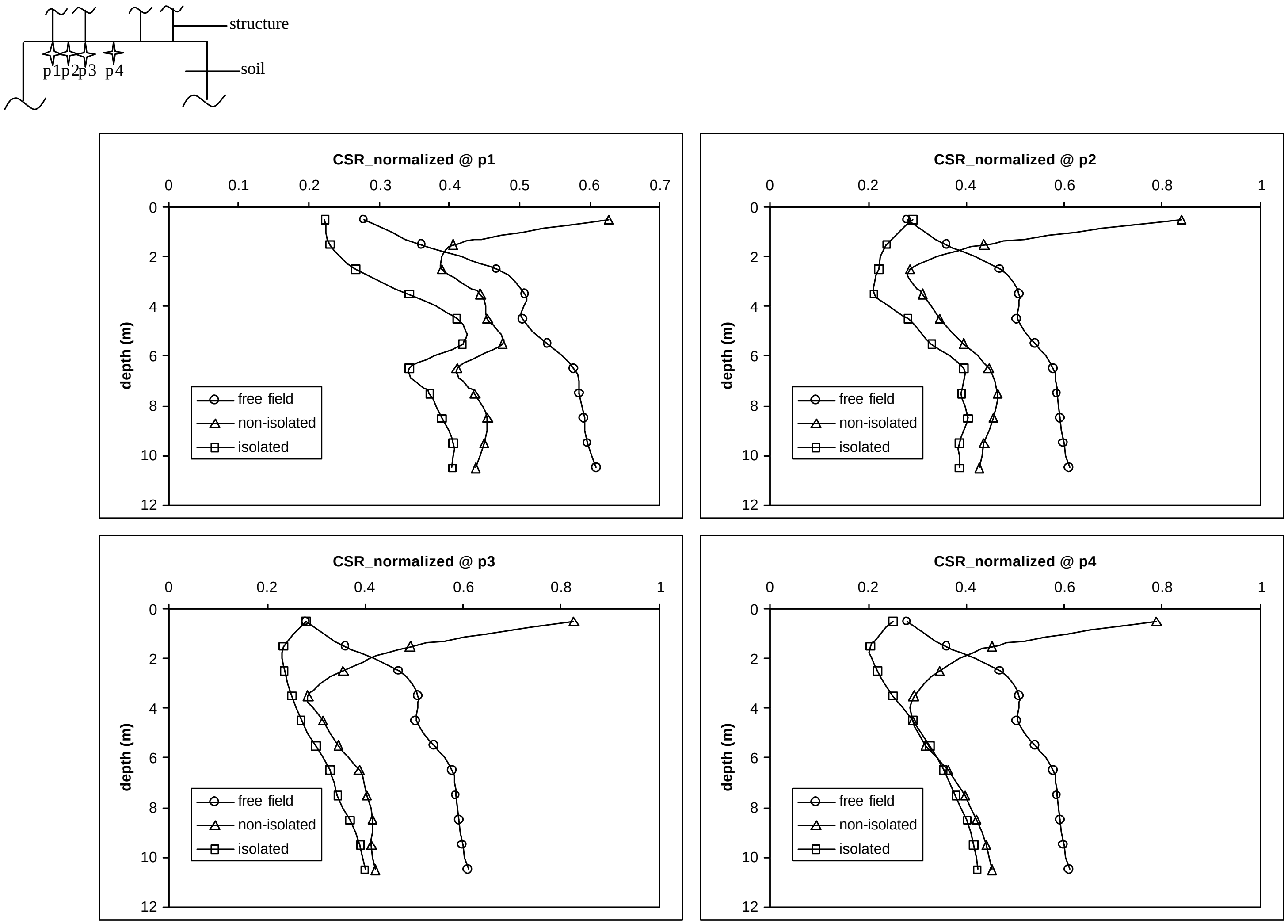
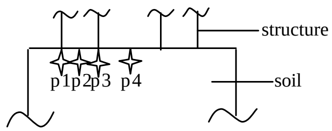


Figure 4.3.b Second Type of Structure with second site under Kocaeli Earthquake



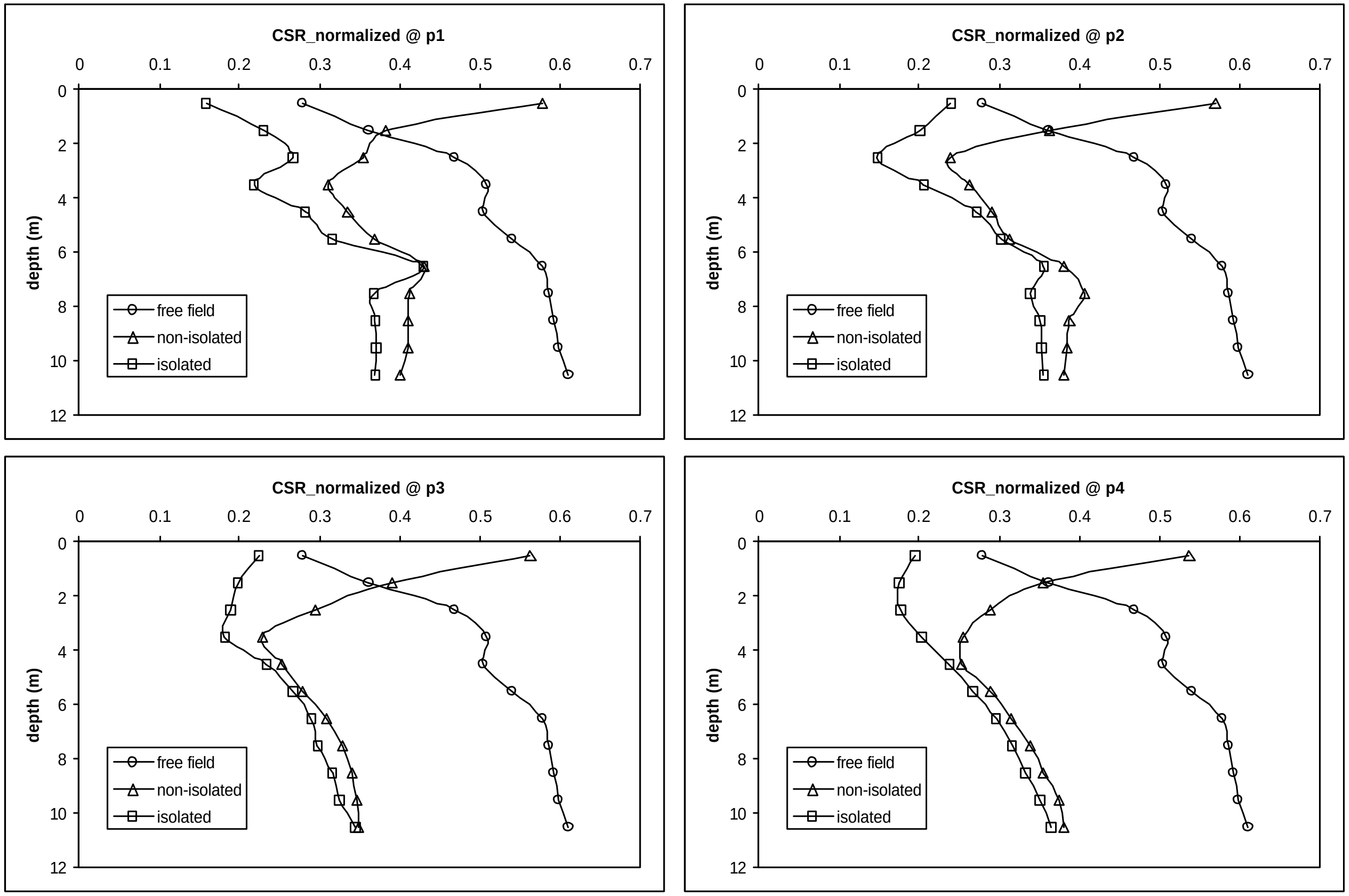


Figure 4.3.c Third Type of Structure with second site under Kocaeli Earthquake

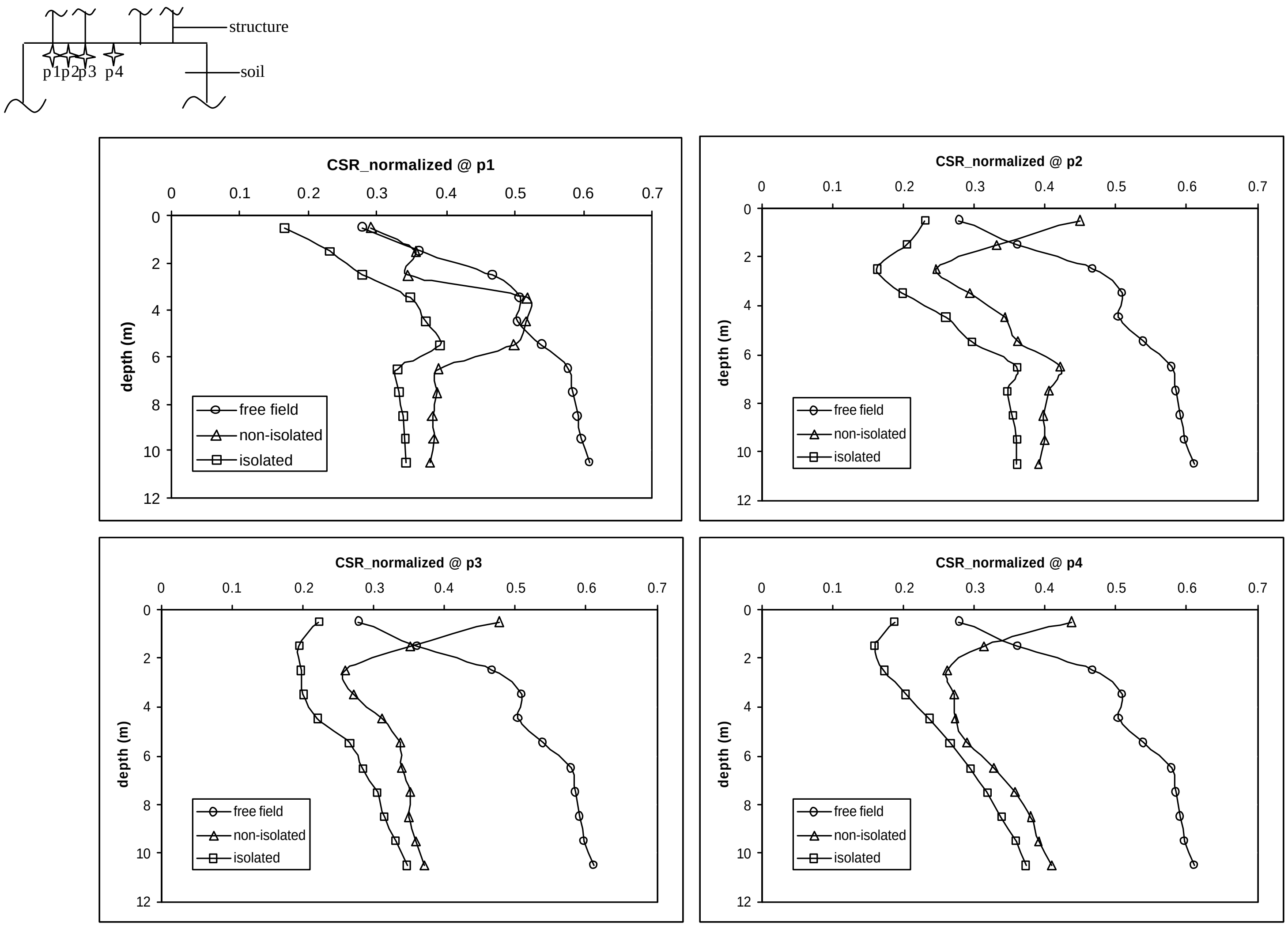


Figure 4.3.d Fourth Type of Structure with second site under Kocaeli Earthquake

Four different points were chosen so that different static shear stresses existing before the earthquake shaking could be examined. For the soil element in the free field there is no shear stress on the horizontal plane; however for soil elements under the edge of the structure static shear stress exists because of the loading of the structure. An a value can be obtained as $\frac{t_{static}}{s'}$, in this study relationships between a and K_a, correction factor, proposed by Seed (1983) was used. These correlations depend on the relative density of the soil. The corrected liquefaction resistance for a given a value and relative density can be determined by multiplying the resistance obtained from the free field analyses by the correction factor, K_a. This correction will result in an increase in liquefaction resistance near the corner of the structure for dense sands and a decrease in resistance for loose sands.

In the Figure 4.4 it can be seen that the a values close to the edge of the structure are higher which in turn results in a significant correction factor of K_a. Figure 4.4 was plotted for up to a depth of 10 m after which the effect of the structure was observed to be invisible. Also the a values changes up to type of the structure.

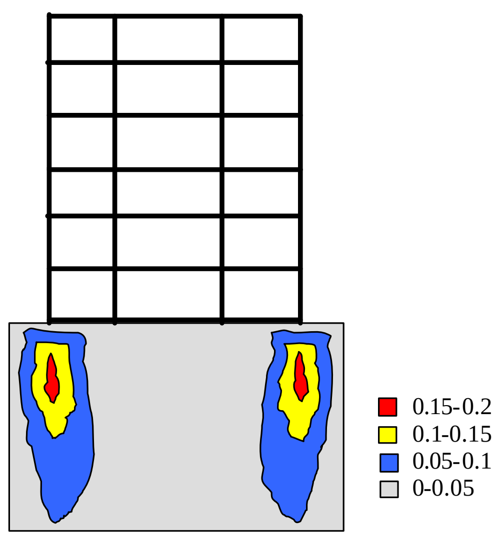


Figure 4.4 Plot of a values under the structure

From the figures some observations can be made. First of all, for all the cases the CSR value under the structure is always greater than the CSR value obtained in the free field this is because the increase in $t_{\max}$ is greater than the increase in s' induced by the structure giving a higher ratio of $\frac{t_{ave}}{s'}$ than that one occurred in the free field. However the effect of increase in the $t_{\max}$ decreases faster than the increase in s', so after the first 2 m the CSR value turns out to be smaller than the CSR value in the free field. The ratio of D, depth up to which the CSR values are higher under the structure, and B, the width of the structure, then can be found to be $\frac{D}{B}$=0.2. This higher CSR values in the upper levels might be one of the reasons for the greater liquefaction potential under the structure than in the free field.

Secondly it can be concluded from the graphs that for the fourth type structure which stands for the six story structure, the CSR values come out to be greater than the CSR values in the free field not only in the upper layers but also in the mid portions of the soil strata. This is because of the K_a correction. It was observed that the a values for six story structure were much higher than the a values obtained for the other type of the structures. At a first glance it would have been concluded for the six story structures that there was a very big increase in s' and not that much increase in $t_{\max}$ because of its spectral characteristics compared to the other type of structures analyzed in this study, so the CSR values would have been expected to be lower than the ones obtained for other types of structures.

Another observation is that all these higher CSR values under the structure, which will potentially give rise to a higher probability of liquefaction occurrence, can be overcome by the base isolated structures. What achieved by isolation effect is the structural period is elongated to a higher value and the spectral acceleration that the building will experience is decreased. So simply $t_{\max}$ value on the soil elements induced by the structure is decreased and also s' value is increased because of the additional isolation layer compared to non isolated structures. As a result the ratio, $\frac{t_{ave}}{s'}$, under a base isolated structure is decreased considerably compared to the $\frac{t_{ave}}{s'}$ value obtained under the non isolated structures.

All these observations agree with the one proposed by Rollins and Seed (1990); the liquefaction potential under the structure with a period range of 0.3-0.6 seconds are greater; however for the structures having a natural period greater than 2 seconds which means for this study the base isolated structures, the liquefaction potential under the structure is lower than that one in the free field. However in their simplified model they didn't take into account the effect of K_a correction which was examined to be an important factor for some of the cases in this study.

4.3 Structural Response Values

In this section mainly the results obtained from the analyses for non-isolated and isolated structures were compared also the effect of flexible base, which means analyzing the structure including the soil beneath it, on the response values of the structure was examined.

4.3.1 Comparison of Non-Isolated and Isolated Systems

Structural performances were evaluated for each case on the basis of base shear introduced to the structure and the relative displacements of the floors. All the structural responses can be found in Appendix B. In Table 4.3 only the response values for one site under Kocaeli Earthquake were given. It is clear from the values that the relative displacements and base shears can be decreased significantly by use of isolation systems. Also the base displacements were given; these values can be decreased further by additional dampers. It can be observed that the reduction in the base shear forces is very close to the reduction in the relative displacements. The small discrepancies might possibly be a result of the base rocking effect. If the base rocking introduces displacements to the system this will be a rigid body motion so the relative displacements will increase however there will be no change in the base shear force. It was shown in the study performed by Stewart, 1997 that base rocking motion can result in 0% to 100% of the total displacements depending upon the site and input motion characteristics.

Table 4.3 Structural Response Values of Four Different Structures on the Third Site for Kocaeli Earthquake

3 story building (Base displacement: 32 cm.)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	584	84
Base shear obtained by SAP (kN)	465	60
Rel. Disp. on the first floor	1.31	0.22
Rel. Disp. on the second floor	1.12	0.15
Rel. Disp. on the third floor	0.55	0.09

4 story building (Base displacement: 38 cm.)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	632	100
Base shear obtained by SAP (kN)	670	68
Rel. Disp. on the first floor	1.52	0.23
Rel. Disp. on the second floor	1.21	0.21
Rel. Disp. on the third floor	0.92	0.16
Rel. Disp. on the fourth floor	0.53	0.11

4 story building with %50 additional weight (Base displacement: 36 cm.)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	924	132
Base shear obtained by SAP (kN)	840	84
Rel. Disp. on the first floor	1.25	0.35
Rel. Disp. on the second floor	1.03	0.35
Rel. Disp. on the third floor	0.82	0.25
Rel. Disp. on the fourth floor	0.45	0.25

6 story building (Base displacement: 48 cm.)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	488	142
Base shear obtained by SAP (kN)	600	124
Rel. Disp. on the first floor	1.24	0.32
Rel. Disp. on the second floor	1.22	0.28
Rel. Disp. on the third floor	1.11	0.27
Rel. Disp. on the fourth floor	0.74	0.23
Rel. Disp. on the fifth floor	0.65	0.19
Rel. Disp. on the sixth floor	0.35	0.11

The reduction factors for each case were given in the Table 4.4. Depending upon the characteristics of the structure, site and the input motion the reduction in the structural response values can be decreased by a factor of 7-10 by use of isolation systems.

Table 4.4.a Reduction Factors of the Isolated Structures for Kocaeli Earthquake

	First Type of Structure	Second Type of Structure	Third Type of Structure	Fourth Type of Structure
First Site	7.1	7.2	4.8	2.7
Second Site	4	6.8	5.2	4.8
Third Site	7	6.3	7	3.4

Table 4.4.b Reduction Factors of the Isolated Structures for El-Centro Earthquake

	First Type of Structure	Second Type of Structure	Third Type of Structure	Fourth Type of Structure
First Site	9.5	12.8	11.4	11.2
Second Site	6.8	12	14.6	10
Third Site	17	11.8	10.6	12

However it is more worth to focus on the effect of different structural type, soil site and input motion to these reduction factors. For Kocaeli input motion the effect of isolation is clearer for the first three types of structures. In the fourth type of structure already the spectral acceleration experienced by the structure is low so the reduction achieved by base isolation in the response values is not that effective. On the other hand, this observation turns out to be wrong for the El-Centro input motion. This is because of the characteristics of the input motion; it is clear from the response spectra given in the previous chapters that El-Centro Earthquake has quite high spectral values for three different sites for the periods around 0.6 seconds which matches with the natural period of the fourth type of structure.

Even if the reduction factors are examined for each different input motion separately, again it is difficult to make a conclusion that if a site period matches with the structural period it will be the worst combination regarding the structural response values. If it was the case the diagonal elements would have been higher than the other values on the same row, i.e. first type of structure on the first site, second and third type of structure on the second site and lastly the fourth type of structure on the third site would have had higher reduction factors because of the higher non-isolated response values. However it can be seen that this is not the case mainly because of the soil structure interaction including the characteristics of the input motion. In other words, because of the soil structure interaction effect the structural period elongates and spectral acceleration turns out to be a different value than it was expected to be.

4.3.2 Effect of Flexible Base

The research performed by Stewart (1997) was explained in detail in Chapter 2. Two main findings up to this study were explained as: there was no significant change in the response spectra regarding the spectral values because of the soil structure interaction. This kind of effects will be discussed in the next section on the numerical results of this thesis. The second finding of the research by Stewart (1997) was that soil structure interaction would result in elongation of the period of the structure due to flexible base.

This effect was examined in this thesis by comparing the structural response values obtained by FLAC analyses which stands for the flexible base system, with the values obtained by SAP analyses which stands for the fixed base system of the same superstructure. For SAP analyses the surface outcrop motions were obtained by SHAKE analyses. In the analyses for the fixed base systems, it was known that the soil structure interaction could not have taken into account. However it was aimed to examine just the effect of flexible base considering soil structure interaction effect had no effect on the response spectra.

All the structural response values were given in Appendix B as stated before. In these results it can be seen that as a general trend the base shear forces for the same structures were found to be lower obtained by FLAC analyses than those ones obtained by SAP analyses. It means that the results of the flexible base analyses give lower values than the ones obtained from fixed base analyses as a general trend for the cases analyzed in this thesis. The structural periods for the fixed base systems can be obtained easily; however the lengthening in the structural periods due to flexible base effect, is not that easy to determine so it is impossible at a first glance to obtain the spectral values if the system is being analyzed as flexible base.

In this study Base Shear Coefficients, BSC, were plotted for fixed base and flexible base systems for the same superstructure. So each case has a pair of values; one for fixed base and one for flexible base. In Figure 4.5 the dots stand for one of the flexible base systems for which a polynomial was fitted. The Base Shear Coefficient can be defined as:

$$BSC = \frac{V_s}{W} = \frac{S_A}{g} \tag{4.5}$$

where V_s is base shear force, W is total weight of structure and S_A is the spectral acceleration.

In Figure 4.5 it can be observed that up to a value on the x axis the values obtained for the flexible base systems turn out to be lower than the values obtained for the fixed base systems. Considering the Equation 4.5, this observation means that the S_A experienced by the structure has lower values if it is analyzed as flexible base systems. This is because the system is moved to the portions with lower acceleration values on the response spectra due to the elongation of the structural period.

However there is a cut of value for which the flexible base has no effect on the structural response. These values can be seen to be 0.5 for Kocaeli input motion and 0.2 for El-Centro input motion in Figure 4.5. These values match with the spectral acceleration values for Kocaeli and El-Centro input motion for the period of 0.6 seconds, respectively. So for the structures with a period greater than 0.6 seconds, the soil structure interaction, flexible base, has no effect in period lengthening.

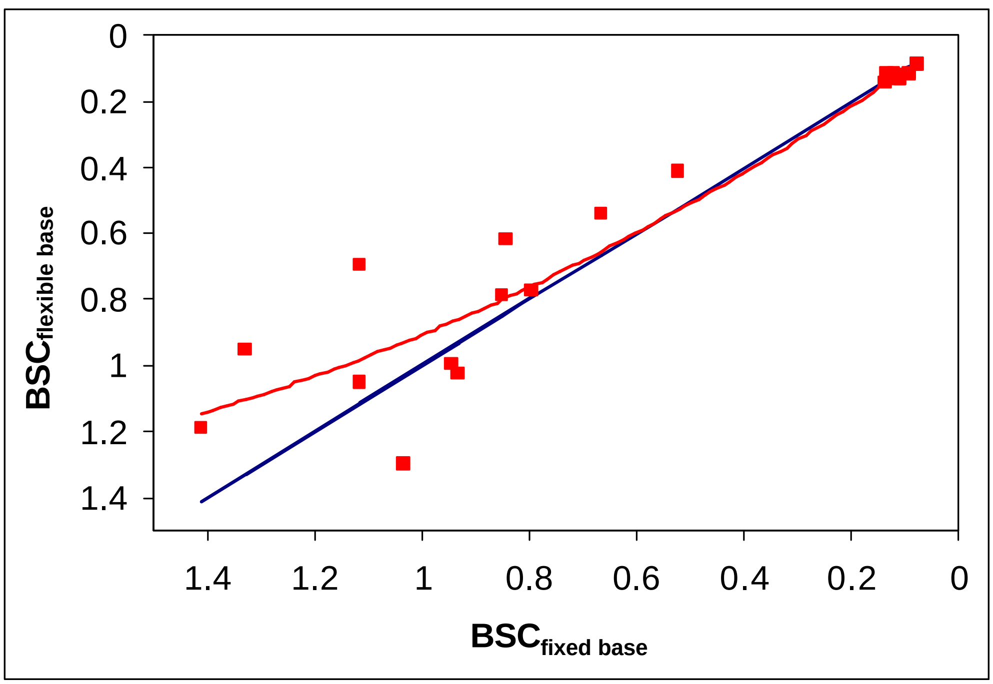


Figure 4.5.a Effect of Flexible Base on Structural Response under Kocaeli Earthquake

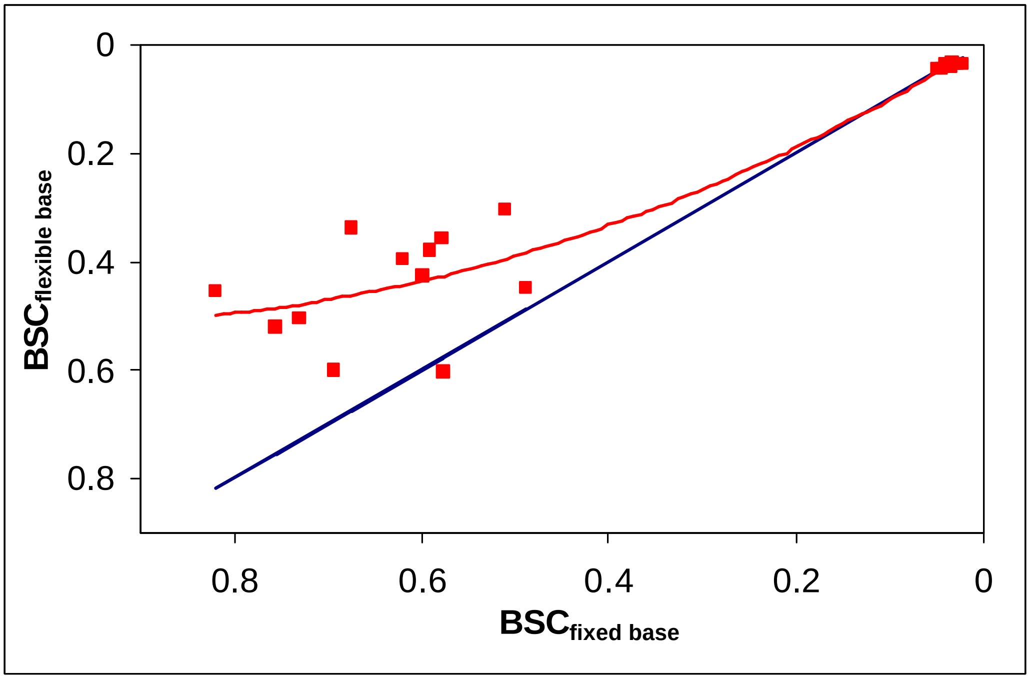


Figure 4.5.b Effect of Flexible Base on Structural Response under El-Centro Earthquake

4.4 Soil Structure Effects on Response Spectrum

It was a common practice to consider that the soil structure interaction effect would decrease the response spectral values. As a consequence it was believed that it would be enough to find out the peak ground acceleration by taking the soil structure interaction into effect and scale the whole response spectrum to obtain the spectral values. However in the research performed by Stewart (1997) it was examined the effect of soil structure interaction on response spectra and it was concluded that as an overall behavior there was no change regarding the response spectrum.

In this thesis for each input motion and site condition, 5 different response spectra were plotted. Firstly, the free field motions was used and to compare the effect of soil structure interaction four different input motions under four different type of structures were taken into consideration. These four different types of structures were chosen to stand for different natural periods; namely, first type of structure for 0.32 seconds, second type of structure for 0.43 seconds, the fourth type structure for 0.65 and the base isolated one for 2.5 seconds. To feed the discussions on response spectra the Peak Ground Acceleration, PGA, values were given in Table 4.5.

Table 4.5.a PGA values in free field and under four different types of structures for Kocaeli Earthquake

	Free field	Under first type of structure	Under second type of structure	Under third type of structure	Under fourth type of structure
First Site	0.60	0.63	0.61	0.65	0.71
Second Site	0.50	0.53	0.55	0.46	0.52
Third Site	0.61	0.54	0.52	0.58	0.61

Table 4.5.b PGA values in free field and under four different types of structures for El-Centro Earthquake

	Free field	Under first type of structure	Under second type of structure	Under third type of structure	Under fourth type of structure
First Site	0.28	0.2	0.22	0.26	0.24
Second Site	0.26	0.25	0.24	0.26	0.25
Third Site	0.25	0.24	0.23	0.20	0.24

In Figure 4.6 the plots were given for the Kocaeli input motion. In the Figure 4.6.a the free field response spectrum has higher spectral values than the ones obtained from the motions under the structure. There is a very significant change for the first type of structure especially around period 0.3 which is the natural period of the fixed base structure. Another observation for this first case is that from the Table 4.5.a it can be seen that the PGA value for the free filed case is lower than the PGA values obtained under the structures, so if a scaling had been done to find out the spectral values of the motions under the structure all would have had higher spectral values than the ones obtained for free field motion.

In the Figure 4.6.b the behavior is different; the spectral values are nearly same for each different motion. This behavior matches quite well with the PGA values in this case, i.e. the PGA values are quite close to each other so if a scaling for the spectral values had been done depending upon the PGA's, the spectral values obtained would have been close to the real ones. In the Figure 4.6.c the spectral values for the free field case is lower for some of the motions and higher for some of them. Again a scaling depending upon the PGA values would have led a wrong conclusion.

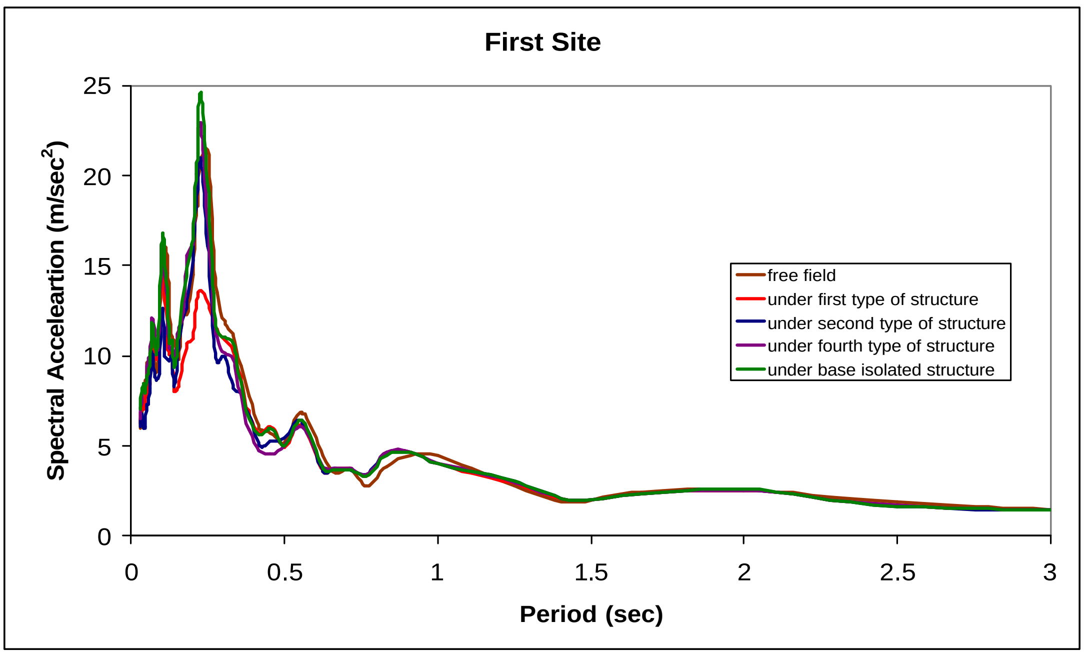


Figure 4.6.a Response Spectrum for free field and base motions under the structures for ß=5 % damping under Kocaeli Earthquake with the first site

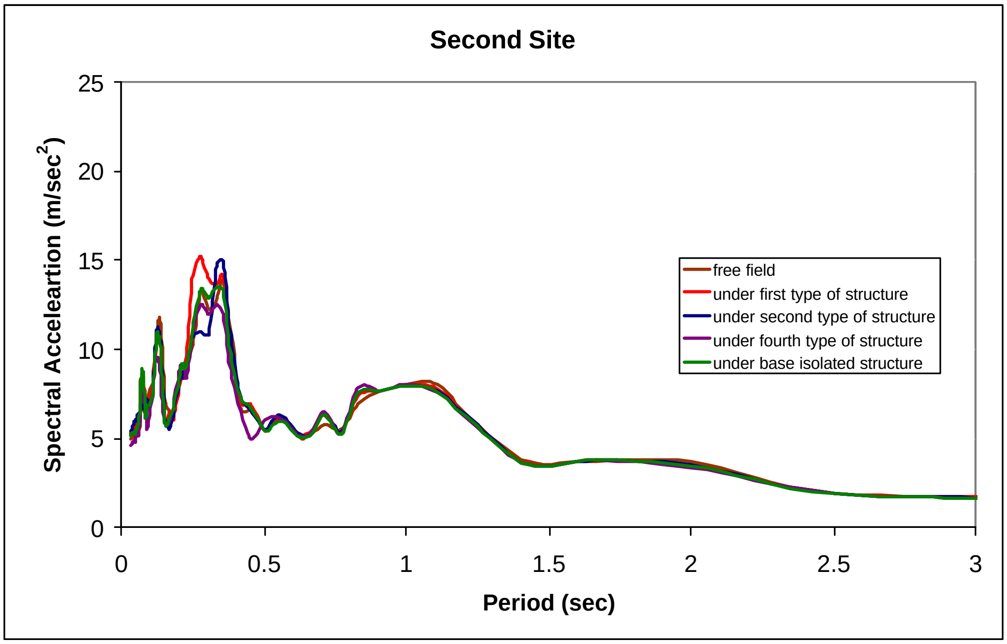


Figure 4.6.b Response Spectrum for free field and base motions under the structures for ß=5 % damping under Kocaeli Earthquake with the second site

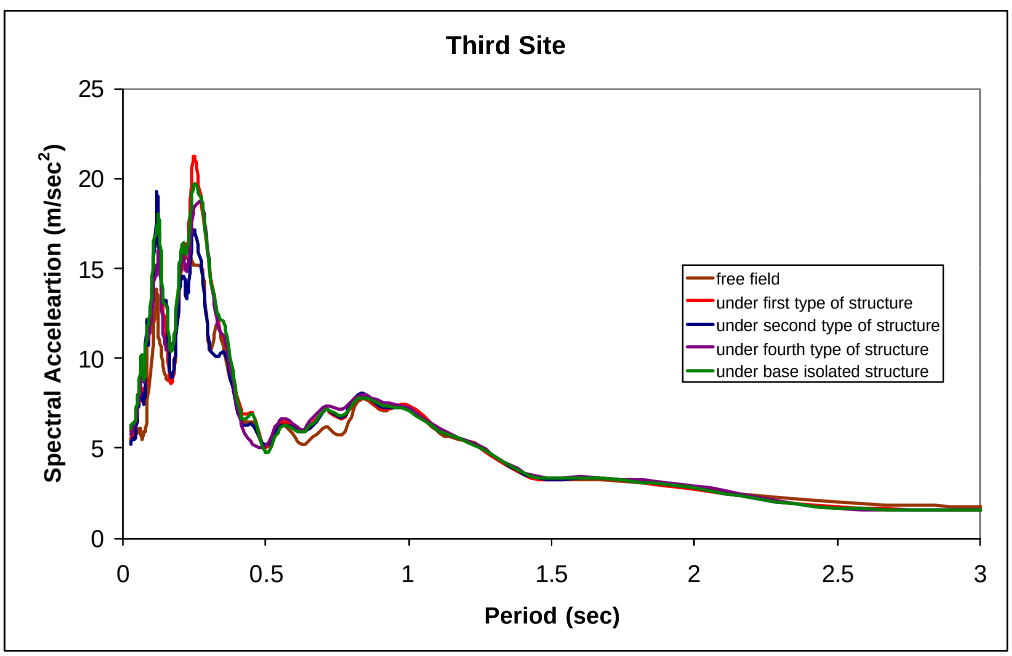


Figure 4.6.c Response Spectrum for free field and base motions under the structures for ß=5 % damping under Kocaeli Earthquake with the third site

In the Figure 4.7.a the spectral values obtained under the El-Centro input motion from free field motion is higher than the ones obtained from the motions under the structure. This agrees in this case with the PGA values, i.e. the PGA value for the free field motion is higher than the PGA values obtained for the motions under different type of structures. However the decrease in the spectral values is not proportional for all the periods. While in some ranges there is small decrease, for some periods there is a significant change in the spectral values. This is the case for the first type of structure; there is a 0.4 g decrease around the period of 0.25 seconds which is again very close to the natural period of the fixed base structure.

In the Figure 4.7.b there is a different behavior from the previous case; the spectral values are nearly same for each different motion. Also this behavior matches quite well with the PGA values, i.e. the PGA values are quite close to each other so if a scaling had been done depending upon the PGA's the spectral values obtained would have been close to the real ones.

In the Figure 4.7.c except than the spectral values obtained for the motion under the base isolated structure, spectral values obtained for the motions under the three different types of structure are lower than ones obtained for the free field motion. There is a small decrease in the PGA values of these motions compared with the PGA value of the free field motion; however the decrease in the spectral values are not proportional with the decrease in PGA's.

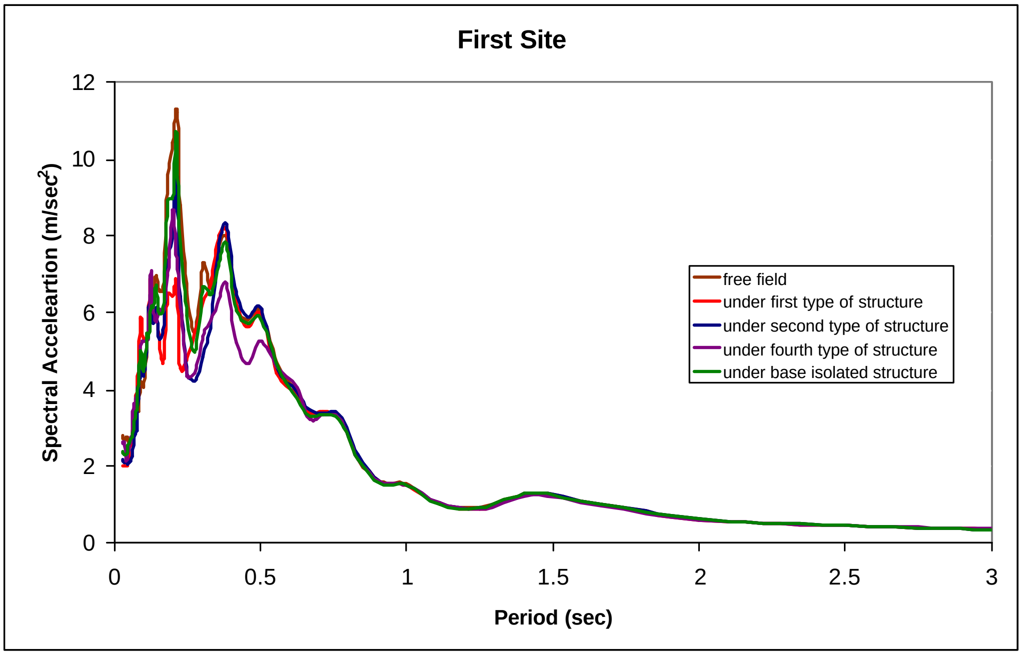


Figure 4.7.a Response Spectrum for free field and base motions under the structures for ß=5 % damping under El-Centro Earthquake with the first site

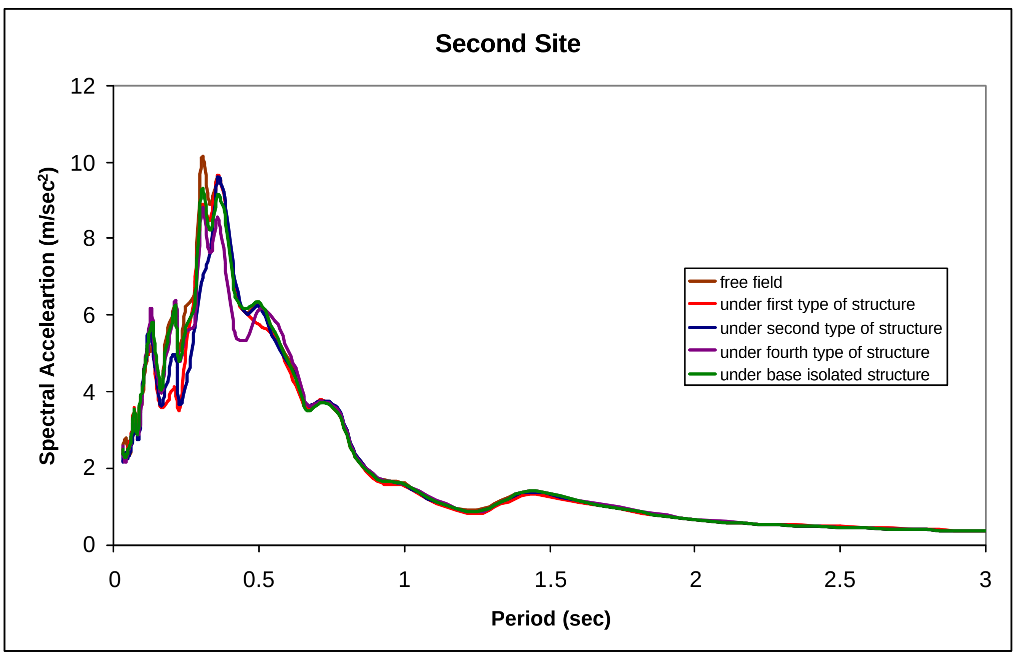


Figure 4.7.b Response Spectrum for free field and base motions under the structures for ß=5 % damping under El-Centro Earthquake with the second site

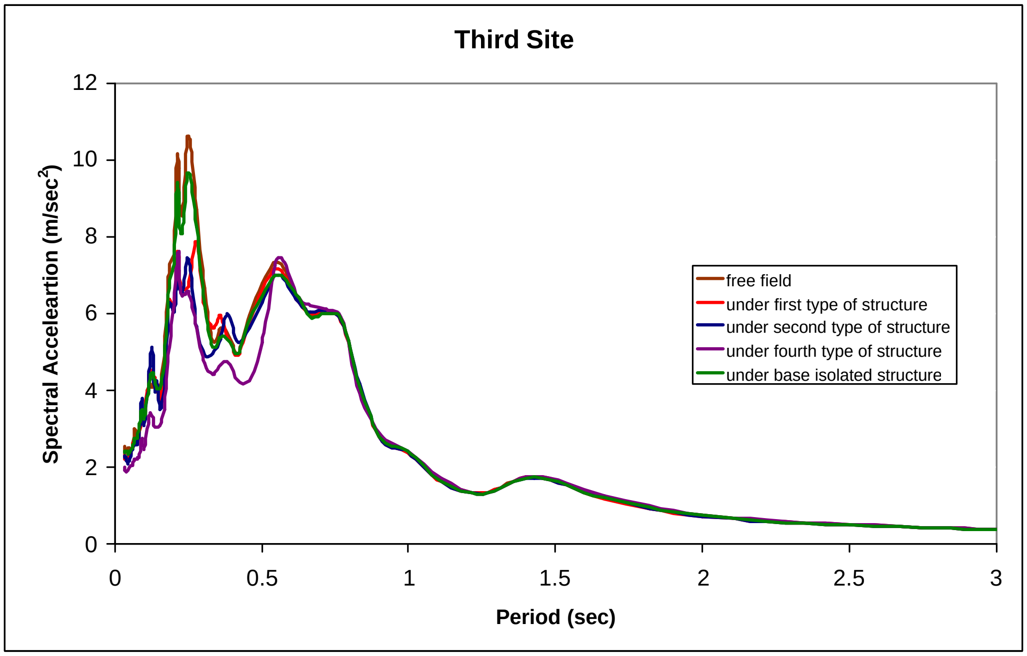


Figure 4.7.c Response Spectrum for free field and base motions under the structures for ß=5 % damping under El-Centro Earthquake with the third site

After these detailed observations on the response spectra for different motions. It is worth to make some statements for all the cases. Beyond a value of the period which is around 1 second, there is no difference in the response spectra obtained for five different cases. Another observation for all the figures is that if the period of the structure is considerable high, in this thesis this is the case for base isolated structures, the spectral values for all the periods are almost the same for both free field motions and for the motions under the structure.

It can be observed that for some cases the spectral values are higher for the free field case and for some other cases the spectral values obtained for the motion under the structure are higher. This observation does not depend on the site characteristics or structural type. If the same type of structure and site are taken but only the input motion is changed, the spectral values will be different; it might have lower or higher values than the ones obtained from free field motions. This conclusion will hold also for different sites, i.e. if the input motion and the structural type are kept same but only the site is changed, the response behavior might be different.

As a conclusion from all these discussions, the interaction is not only between soil and structure but also between input motions. Also it is not possible to make a conclusion on spectral values depending upon PGA values. However in average, it is possible to say in the framework of this thesis that there will not be significant changes regarding the spectral values obtained from the free filed motions and from the motions under the structure except than some cases for which the spectral values can differ up to 1g that can be observed in Figure 4.6.a for the first type of structure. So even only this last observation is enough to point out the need to consider the effect of soil structure interaction although it was concluded that the differences were not considerable as an average.

CHAPTER V

SUMMARY, CONCLUSIONS
AND POSSIBLE FUTURE EXTENSIONS

5.1 Summary

The liquefaction phenomenon has been examined extensively in the last several decades mainly after the big urban earthquakes. During these events many liquefaction related structural damage was observed. However many research focused on the assessment of liquefaction potential in the free field. This was mainly because the common belief that the structure plays a role in favor of liquefaction occurrence. However several big earthquakes proved that structural induced liquefaction problems could be densely distributed under or around foundation soils.

Passive structural control, i.e. use of base isolation systems, turned out to be an effective method to decrease the structural damage. There were some debates on the effectiveness of the structural systems; however these systems all over the world performed well during the earthquakes as they were expected to be. So the use of base isolation systems is now getting more and more popular. The main idea of these systems is using very flexible isolators to elongate the structural period and as a consequence make the structure experience less accelerations and shear forces. Also because the modal behavior is changed completely by use of isolator; the superstructure undergoes rigid body motion so the inter-story drifts decreases considerably.

The common practice in engineering field is to analyze the structure or soil excluding the interaction between each other except than some special projects. Also the researchers have mainly focused on the numerical analyses in soil structure interaction field. Recently based on recorded response data, some empirical conclusions and relations were presented. After which the numerical findings can be calibrated more efficiently.

On the basis of above discussions, what was done in this study is to model the structure and soil together so that the soil structure interaction could be taken into account. The effect of structure on the liquefaction potential could be observed by comparing the liquefaction potential in the free field and under the structure. Also base isolated structures were analyzed to examine the effect of isolation systems both on structural performance and on liquefaction potential.

To model the whole system, the program FLAC which implements the finite difference method, was used. Two input motions were selected as Kocaeli (1999) and El-Centro (1979) Earthquakes. The first one was selected mainly to investigate possible causes of damage observed after Kocaeli Earthquake (1999). El-Centro input motion was chosen mainly because it has a long duration and it does not have high frequency content like Kocaeli input motion has so El-Centro input motion has its peak values in the acceleration response spectrum in longer periods than the Kocaeli Earthquake.

Three different types of soil strata were modeled. For this purpose mainly the bore-hole data from the Sakarya City was used. The sites were chosen so that each one had different periods. The input motions used in the FLAC analyses were obtained by deconvolving the original surface outcrop motions stated above.

Four different types of structures were chosen with three different types of structural periods. The structures were modeled as three, four and six story structures. Also one more four story structure was modeled having the same structural period with the original four story structure with an additional 50% weight. All types of structures were analyzed also by adding an isolation layer.

An introductory section was given for the liquefaction assessment in the free field and another simplified procedure related with the structural induced liquefaction potential was examined in detail. Afterwards the numerical results were used to assess the probability of liquefaction. For each different combination of input motion, site and structure; cyclic stress ratio, CSR, plots were obtained in the free field, under the non isolated and isolated structure. The effect of the static shear stresses was also taken into account in these plots.

The effectiveness of the base isolation systems on the structural response values were examined on single degree of freedom systems as a preliminary study. Also some real data obtained for base isolated structures was given. Afterwards to examine the effectiveness of the isolation systems, the relative displacements of the floors and the total base shear force introduced to the system for non isolated and isolated systems were compared. Also analyzing the structure including the soil beneath it was examined. For this purpose all the structures were also analyzed as if they were fixed base. The results from these analyses were compared with the ones obtained from the FLAC analyses which stand for flexible base analyses.

Lastly, the effect of soil structure interaction on the response spectra was examined. Response spectra obtained from the free field motions and from the motions under the structures were given for each site and under different input motions.

5.2 Conclusions

Following conclusions based on and limited to the results of our analyses were listed as follows:

- Dynamic response of a structure is defined by the interaction of underlying soils, earthquake shaking and superstructure itself.

- The liquefaction potential under the structures was found to be greater than the one in the free field for the D/B ratio equal to 0.2, where D is depth into the soil and B is width of the structure. However after this level the liquefaction potential in the free field turns out to be greater.

- The liquefaction potential under the structures was observed to be affected by the structural type: For the six story structure, it was observed that the liquefaction potential of foundation soils was nearly equal to the one of free field soils. This was mainly because the spectral acceleration level corresponding to the period of the six story structure is lower. Similarly for the six story structure, higher weight of the structure leads to a higher a value which makes the CSR values under the structure more critical than the CSR values in the free field, not close to the surface but in the mid portions of the site.

- The effectiveness of the base isolation systems on liquefaction can be seen clearly. The higher CSR values under the structure which can potentially give rise to liquefaction were reduced even to a lower value than the one obtained in the free field. This was a result of the lower acceleration level experienced by the isolated structures which leads to a lower level of shear forces on the soil elements induced by the structure.

- It can be concluded that the liquefaction potential under the structure will be higher if the structural period is lower than 0.6-0.7 seconds. For the structures having a longer period, in this study this is the case for base isolated structures, the liquefaction potential will be lower under the structure than the one in the free field.

- The structural response values were observed to be decreased by a ratio of 7-10 by use of isolation systems. The isolation systems are thought to be less effective as the structural period increases. However in this study it was observed for the El-Centro input motion that spectral acceleration might be higher even in the longer periods. So the isolation systems turn out to be effective for the structures with higher structural periods depending on the input motion characteristics.

- The natural period of the structure is usually longer when it is analyzed as flexible base systems, i.e. analyzing the structure including the soil beneath it, as opposed to fixed base systems, i.e. analyzing the structure alone. Even though some exceptions do exist, flexible base system will generally experience a lower level of acceleration causing a lower base shear force.

- It was observed that for some cases there might be a 1 g spectral acceleration difference for specific periods between the response spectra obtained from the free field motions and the motions under the structure.

- The difference between the structural response values obtained for fixed base systems and flexible base systems is mainly due to the period lengthening not the change in the response spectrum because of soil structure interaction.

- Because the real natural period of the structure is different from the one obtained from fixed base analyses, it is impossible to state that most critical combination for structural response would be when the site period and the fixed base structural period coincides. This is a common engineering practice; however in this study it was observed that to obtain the real structural response values the whole system, soil and structure, is to be analyzed together.

5.3 Possible Future Extensions

This study can possibly be extended in the future as follows:

- Although the three dimensional, 3D, damping was used to make the analyses more realistically. It would be better to model the system in 3D. However it must be known that using 3D models will increase the run time considerably.

- In this study Mohr-Coulomb failure criterion was used for the soil elements by which soil could be modeled as elastic-perfectly plastic material. The Finn model might be a better solution to examine the liquefaction phenomenon; however again it must be kept in mind that using Finn model will increase the run time.

- The structures were tried to be modeled realistically; however number of the bays might be increased which was three in this study. This will result in a wider structure which will require using bigger systems.

- On the basis of the above statements it can be concluded that the system used in this study was quite realistic; however only two input motions were used. As can be seen from the conclusions the characteristics of the input motions play a significant role in the results so more input motions can be used in the future studies to draw more reliable conclusions.

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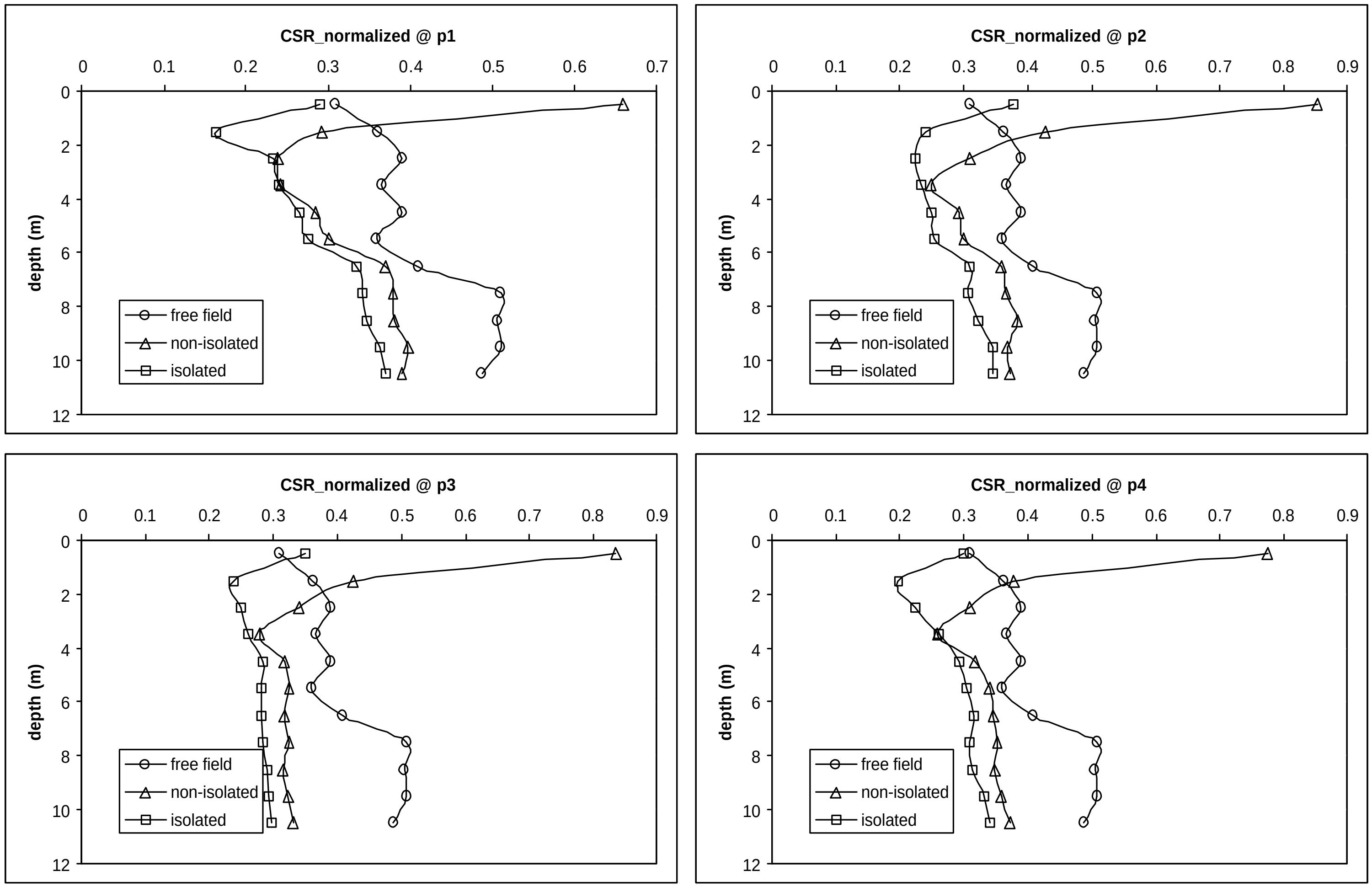
APPENDIX A

CYCLIC STRESS RATIO PLOTS

In Appendix A the plots of Cyclic Stress Ratio vs. Depth were given. In each plot the CSR values for free filed, under the non isolated structure and under the isolated structure can be found. For each combination of different input motion, site and structure four plots were given for different points under the structure.



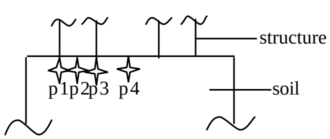
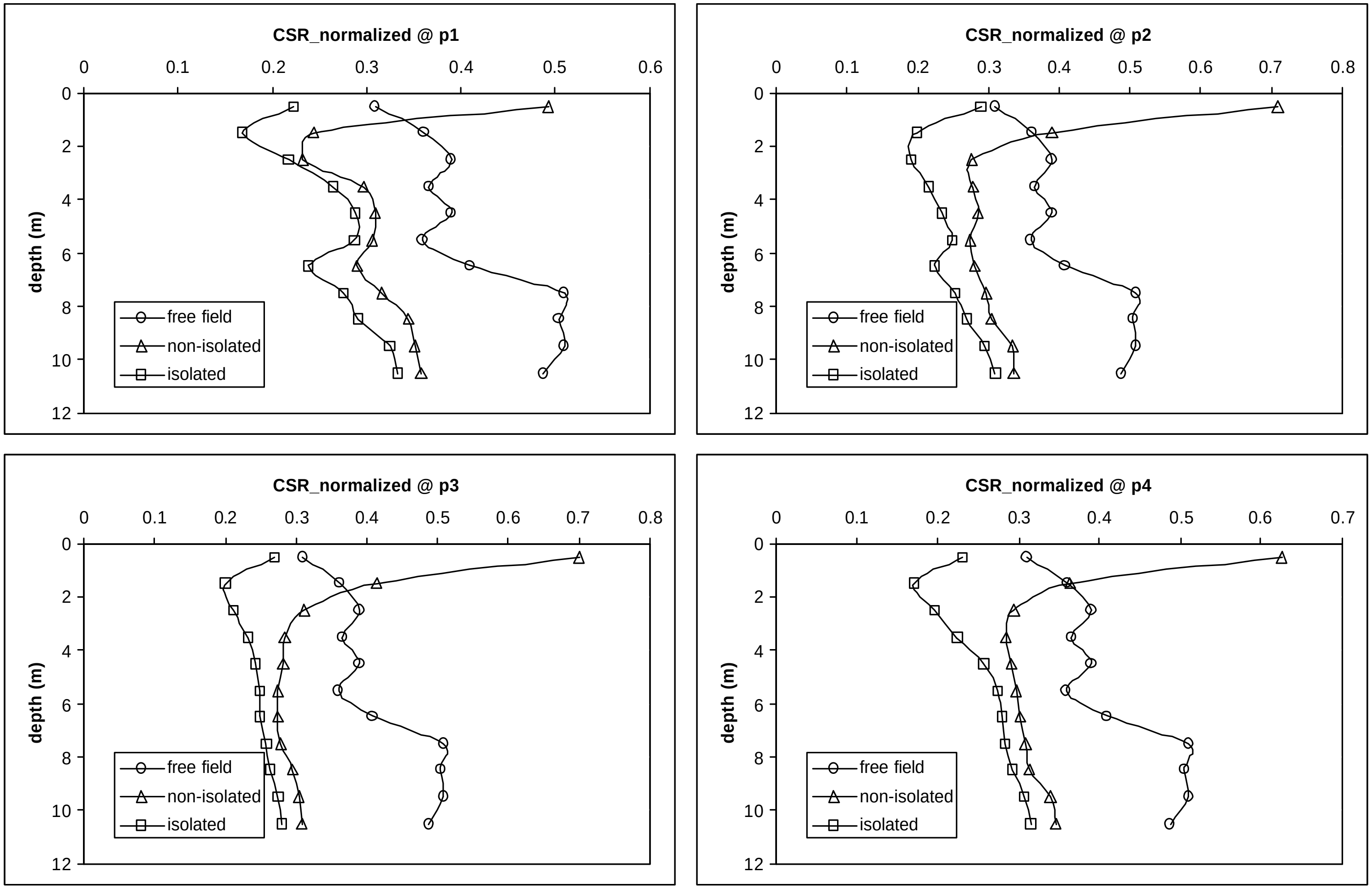


Figure A.1 First Type of Structure with first site under Kocaeli Earthquake



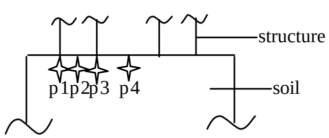


Figure A.2 Second Type of Structure with first site
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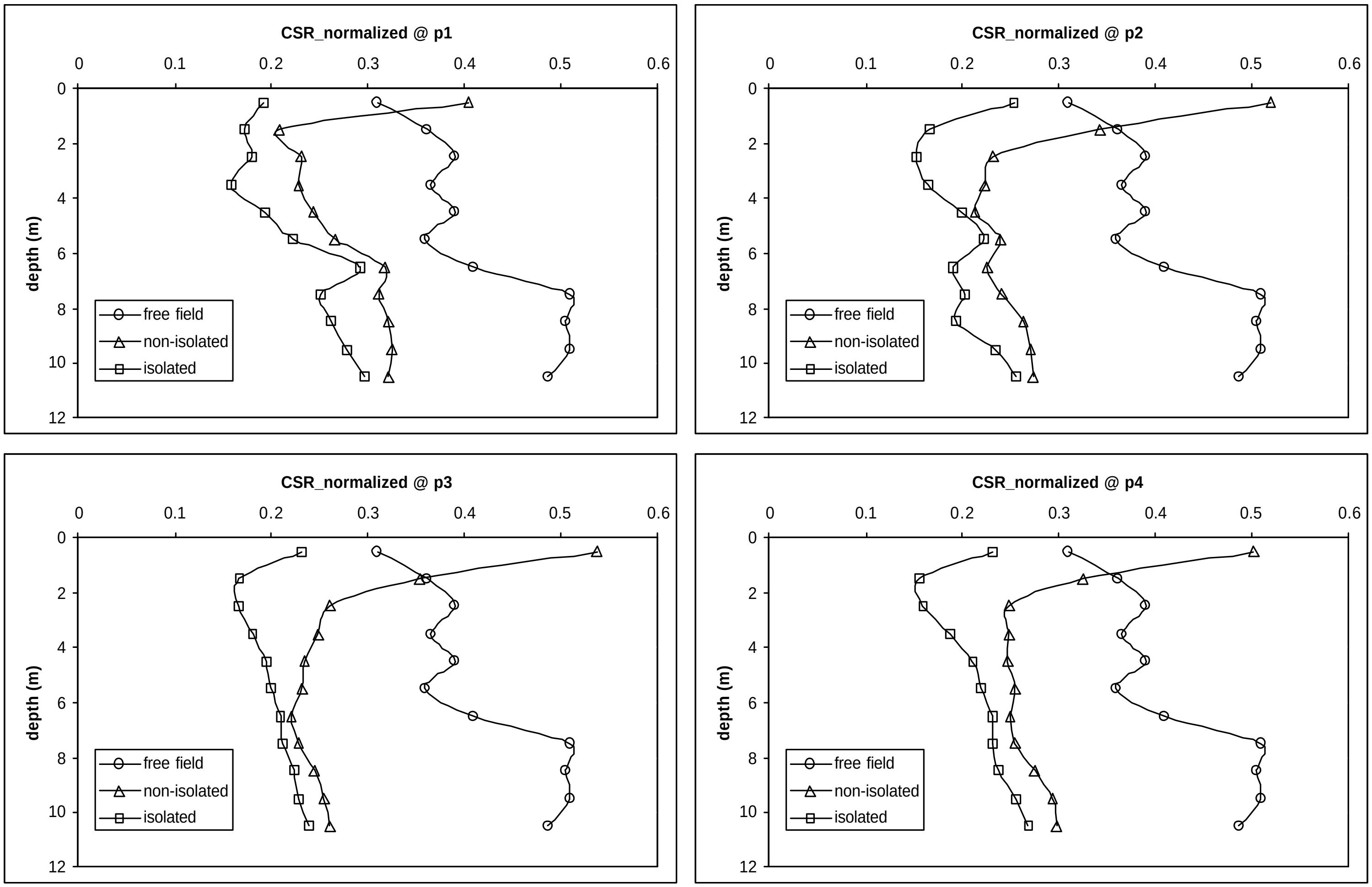

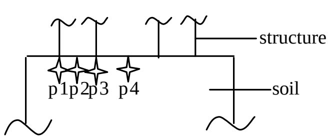

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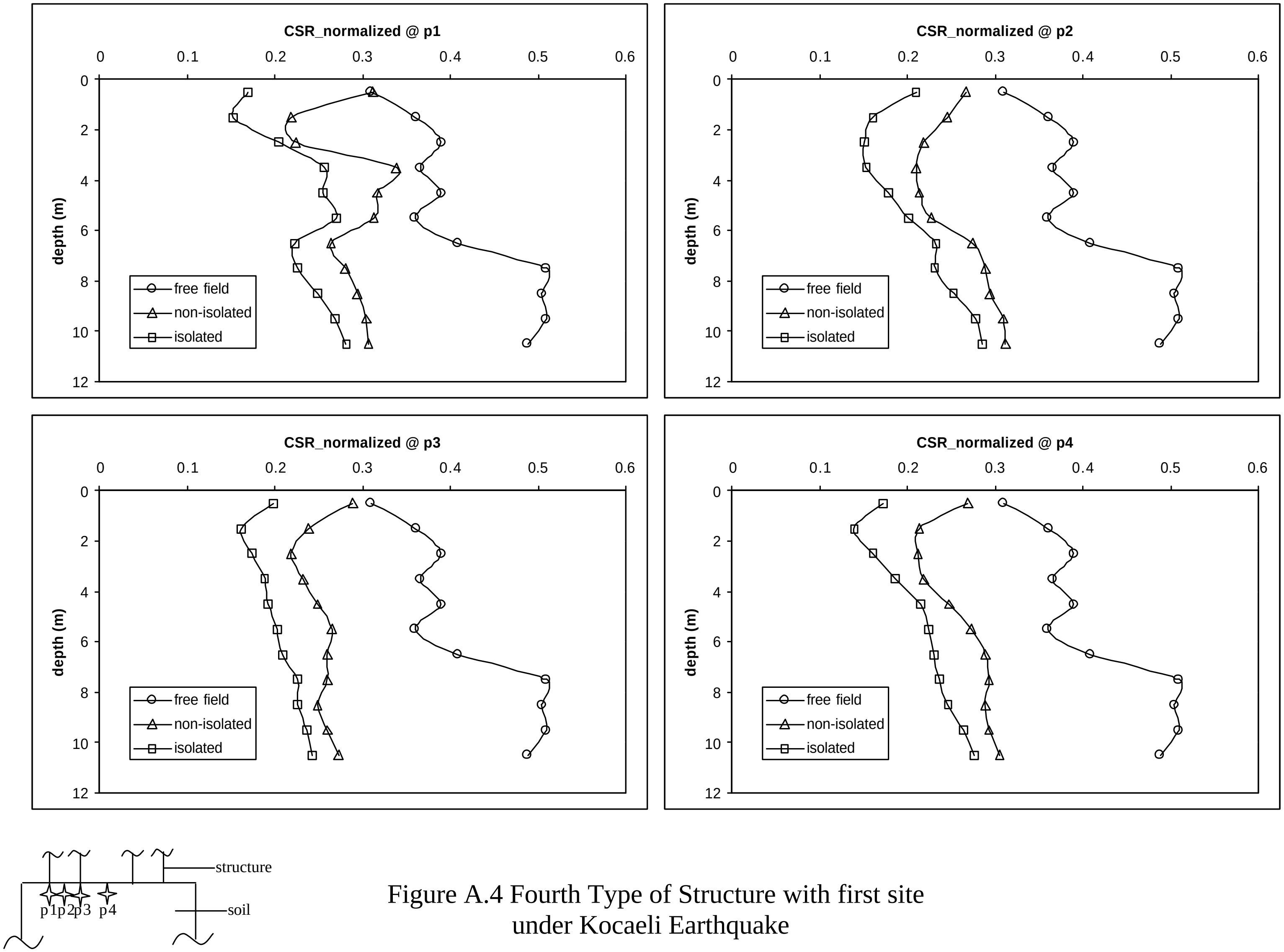

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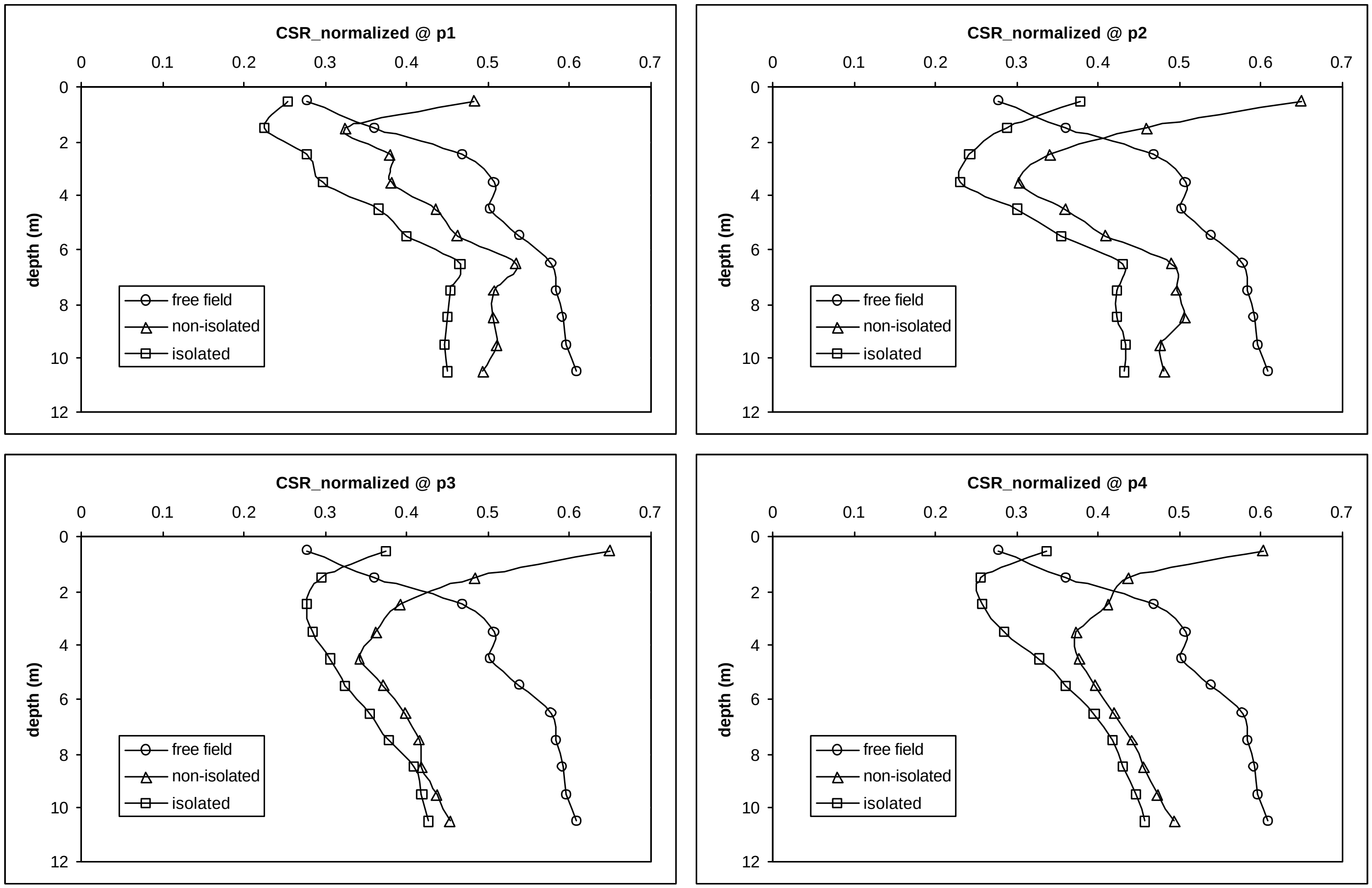

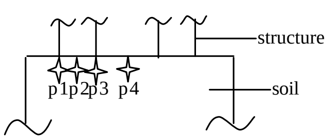

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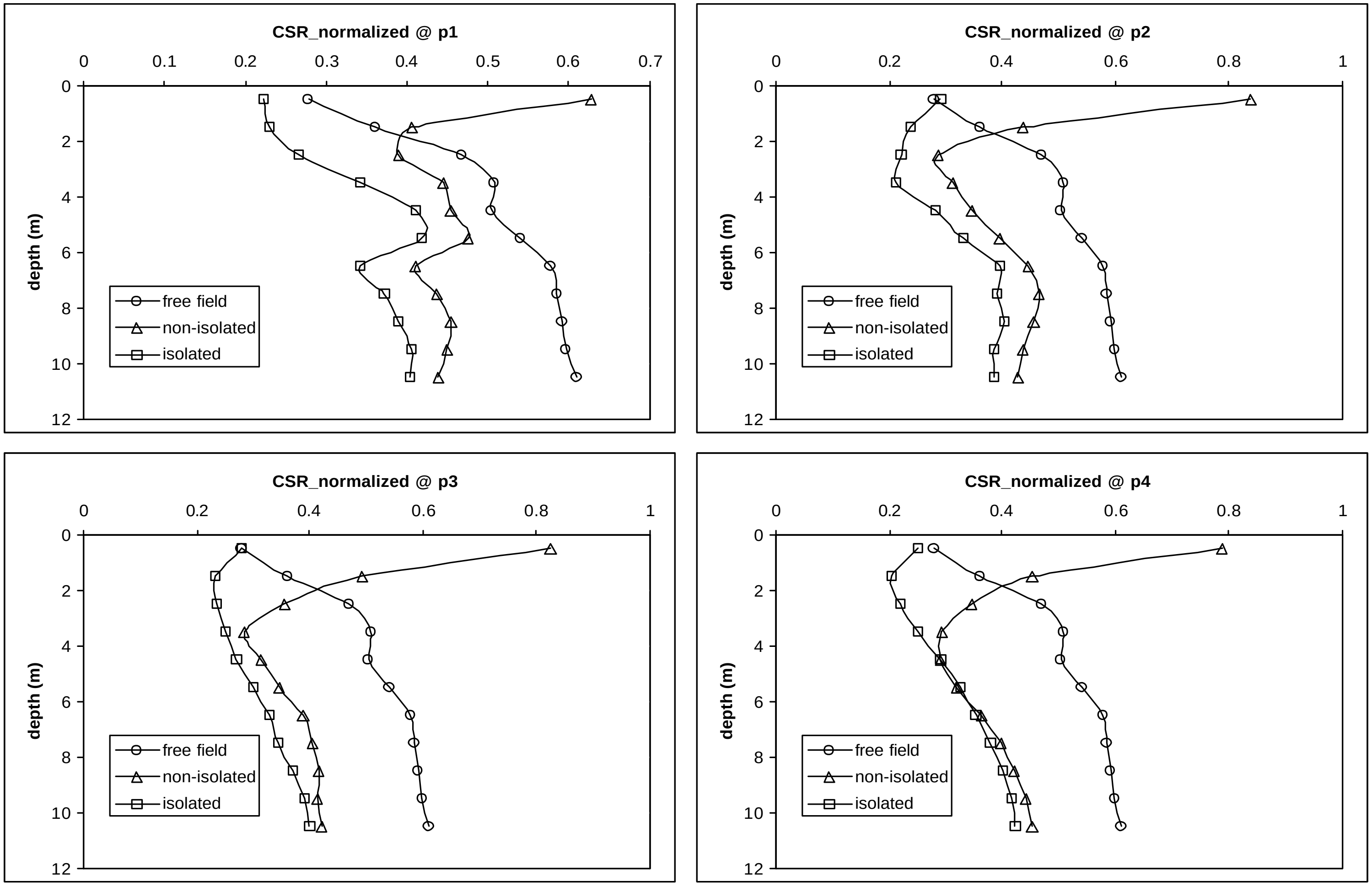

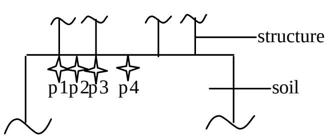

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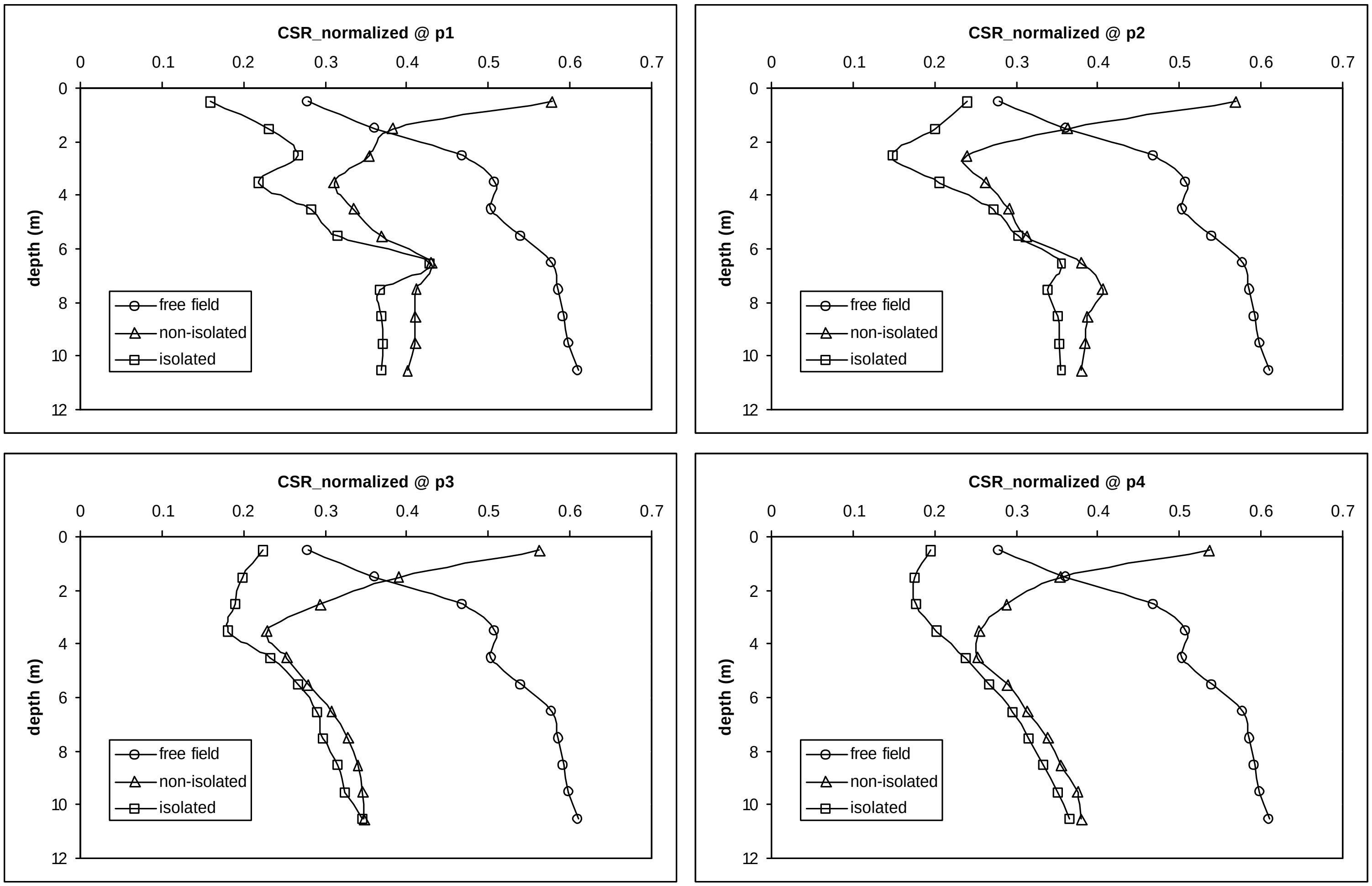

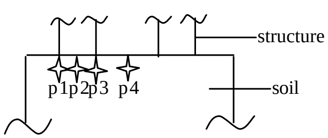

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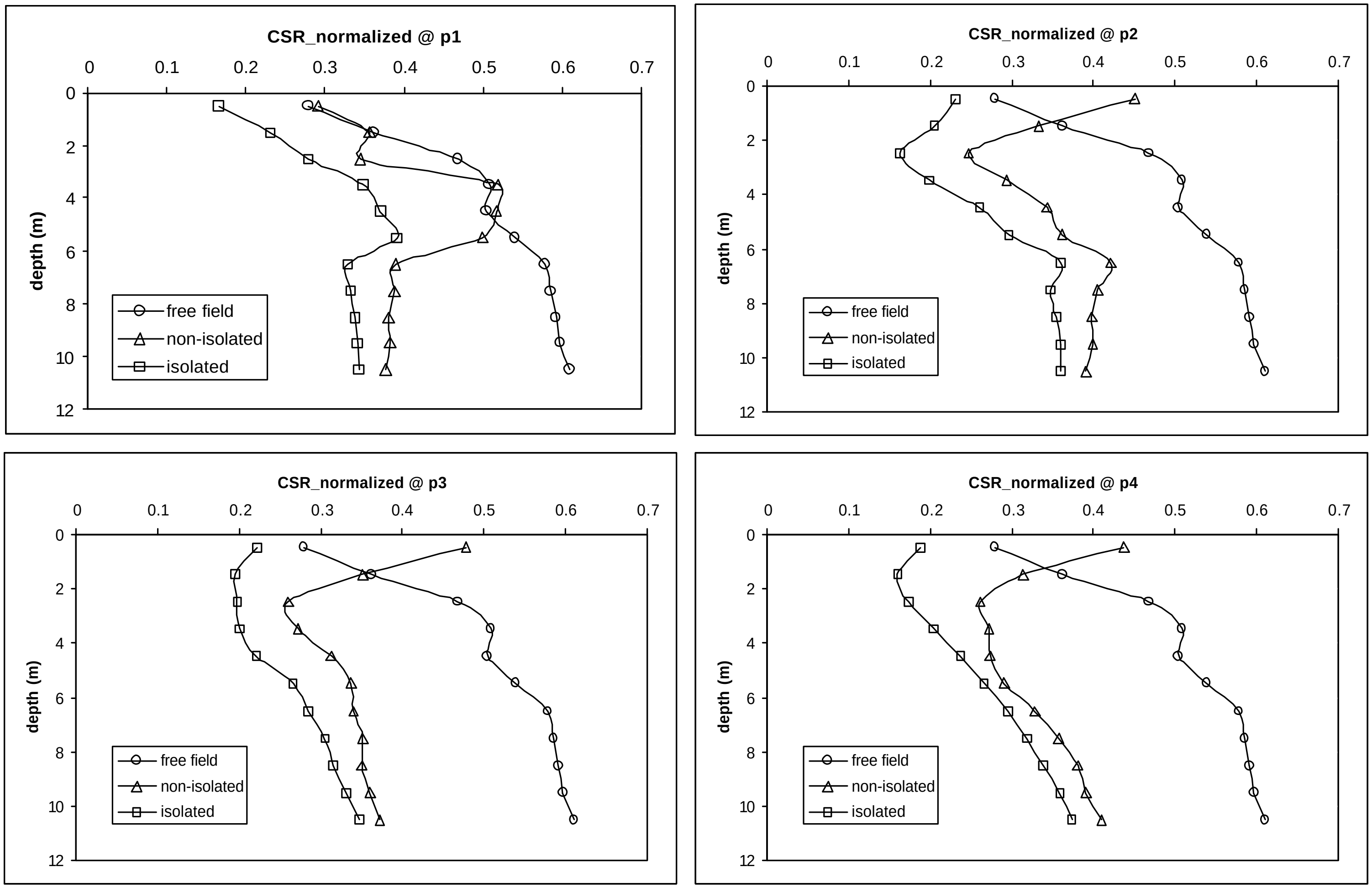

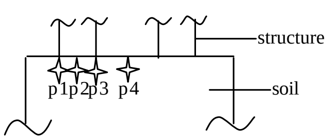

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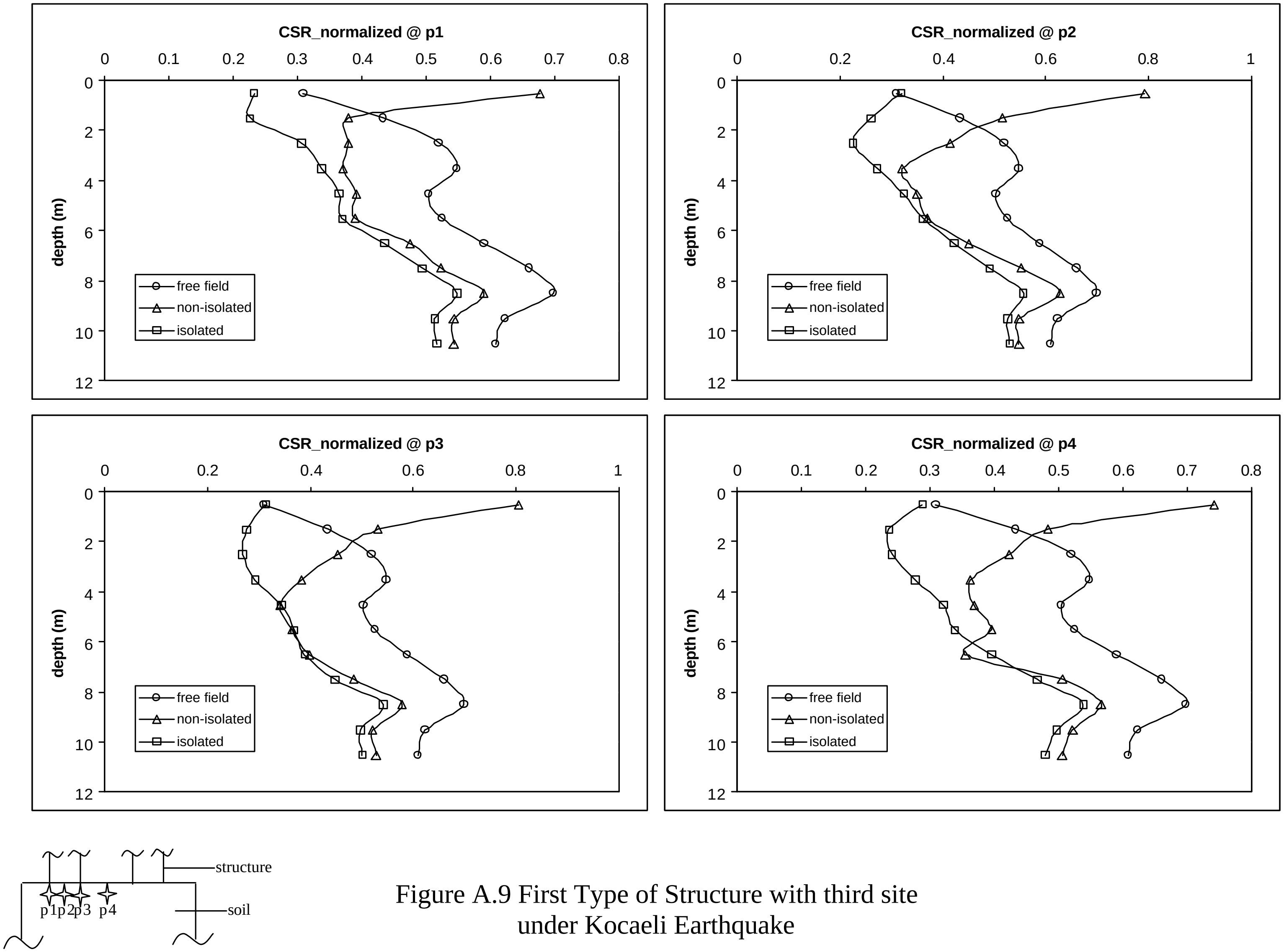

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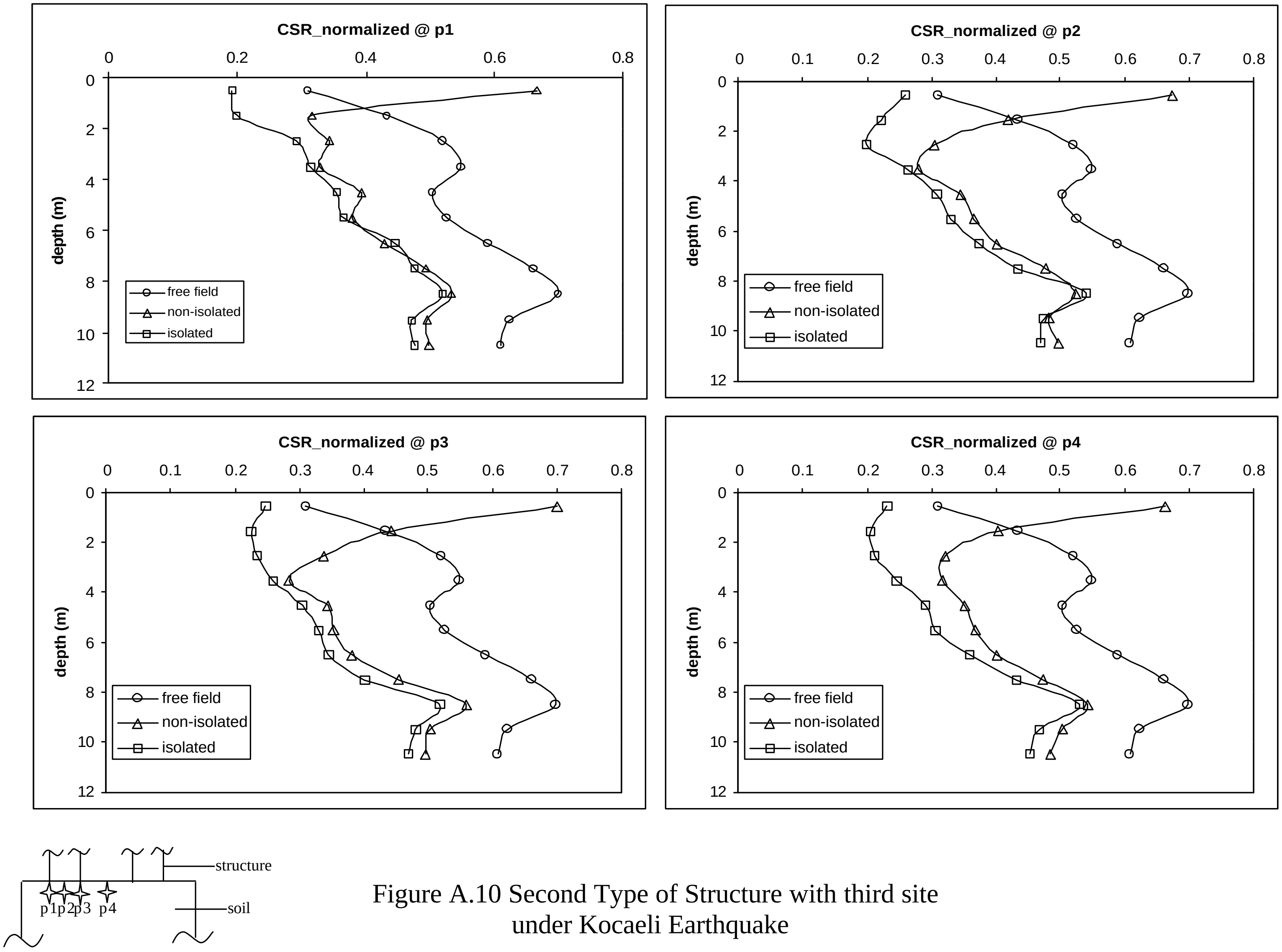

Figure A.10 Second Type of Structure with third site
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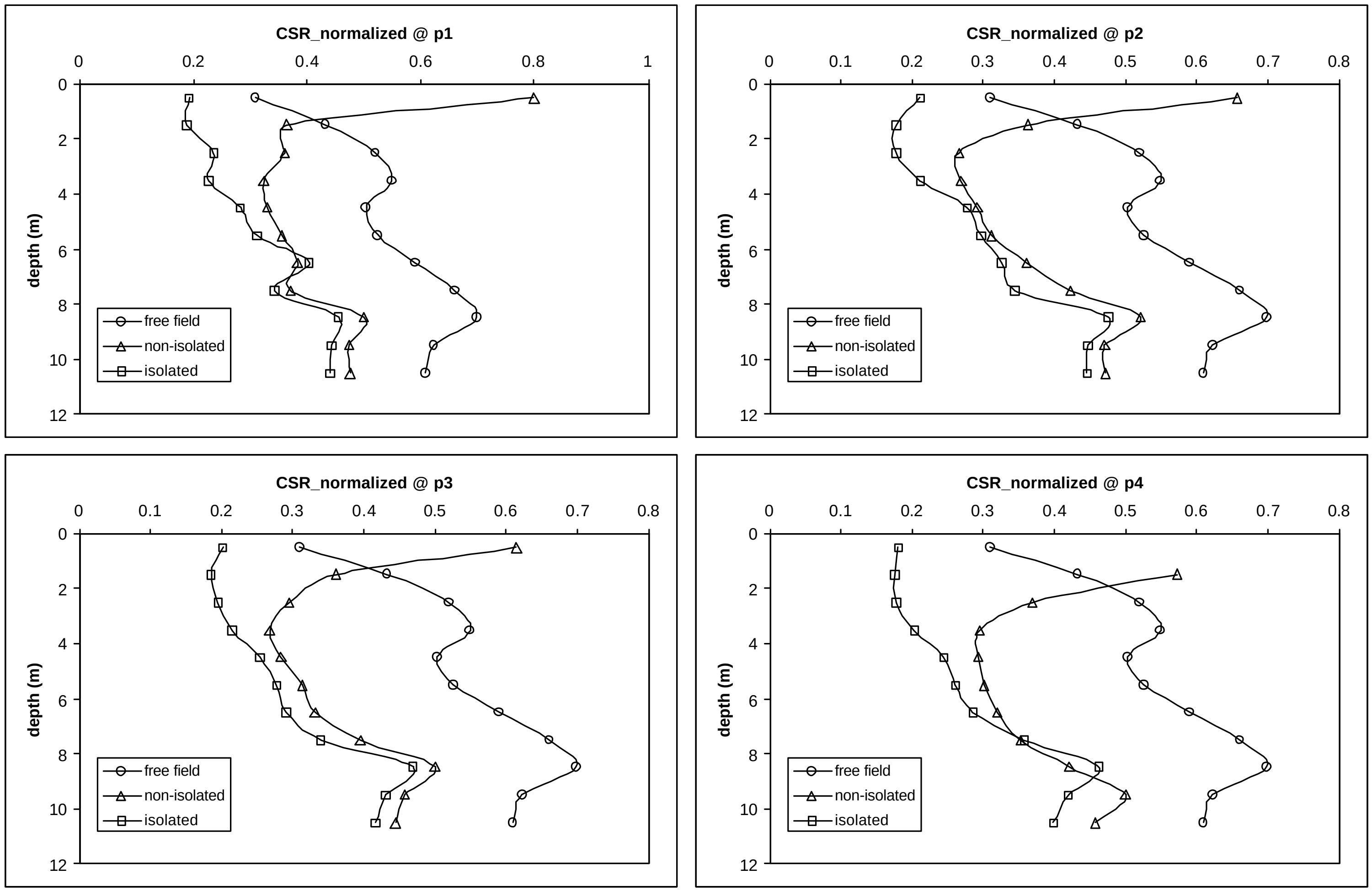

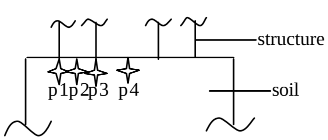

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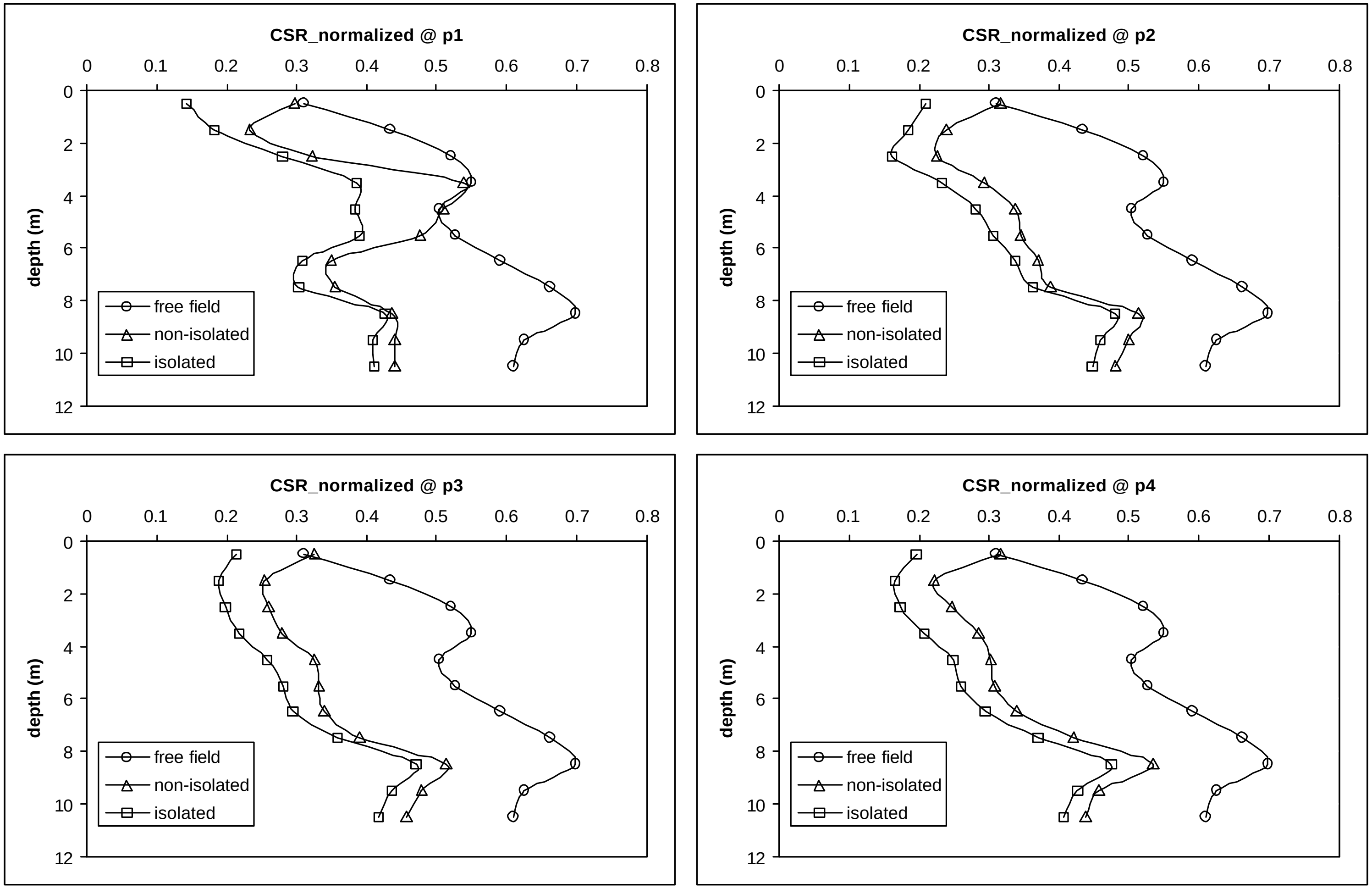

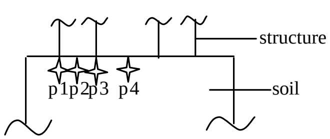

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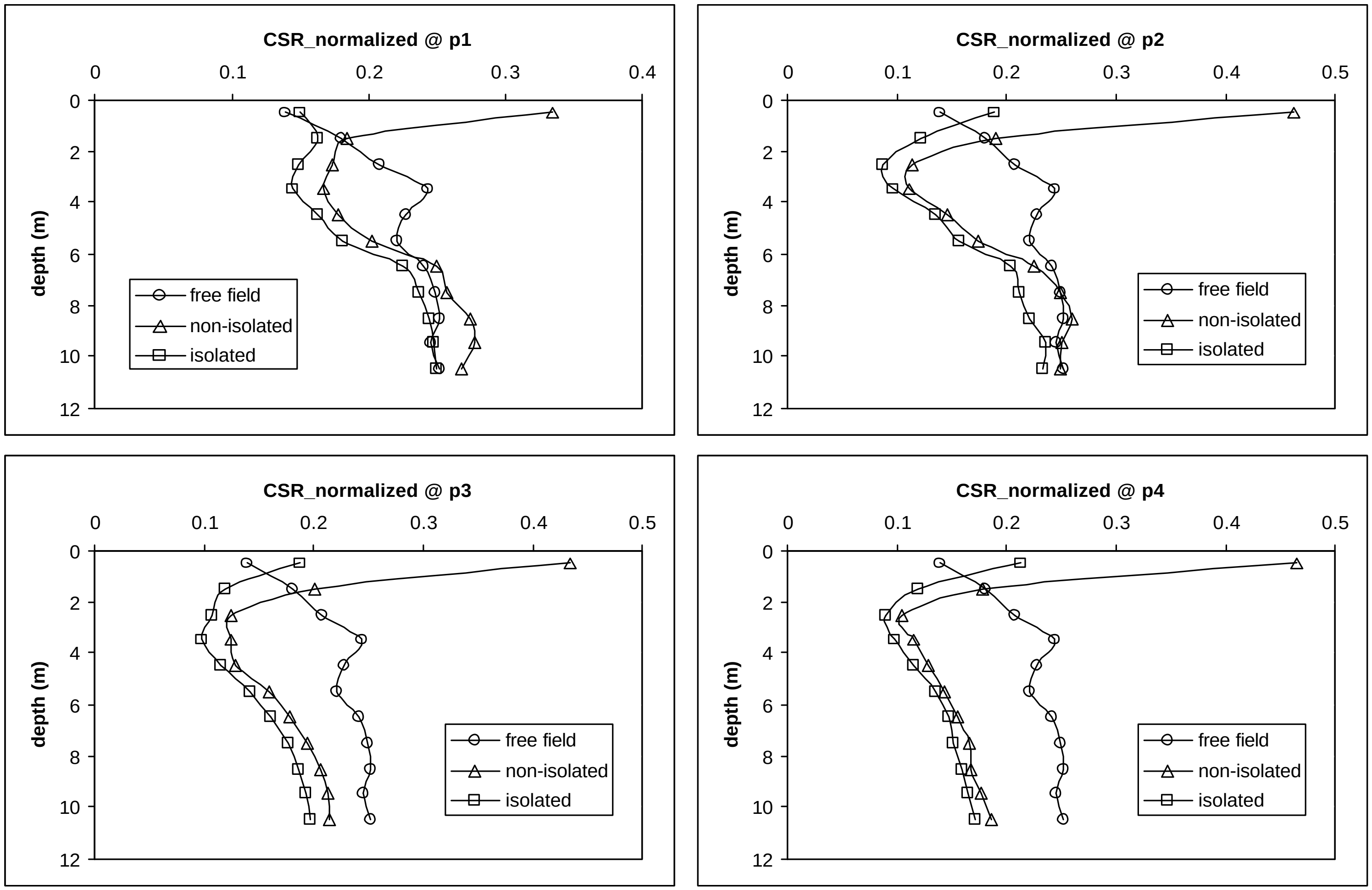

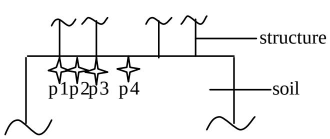

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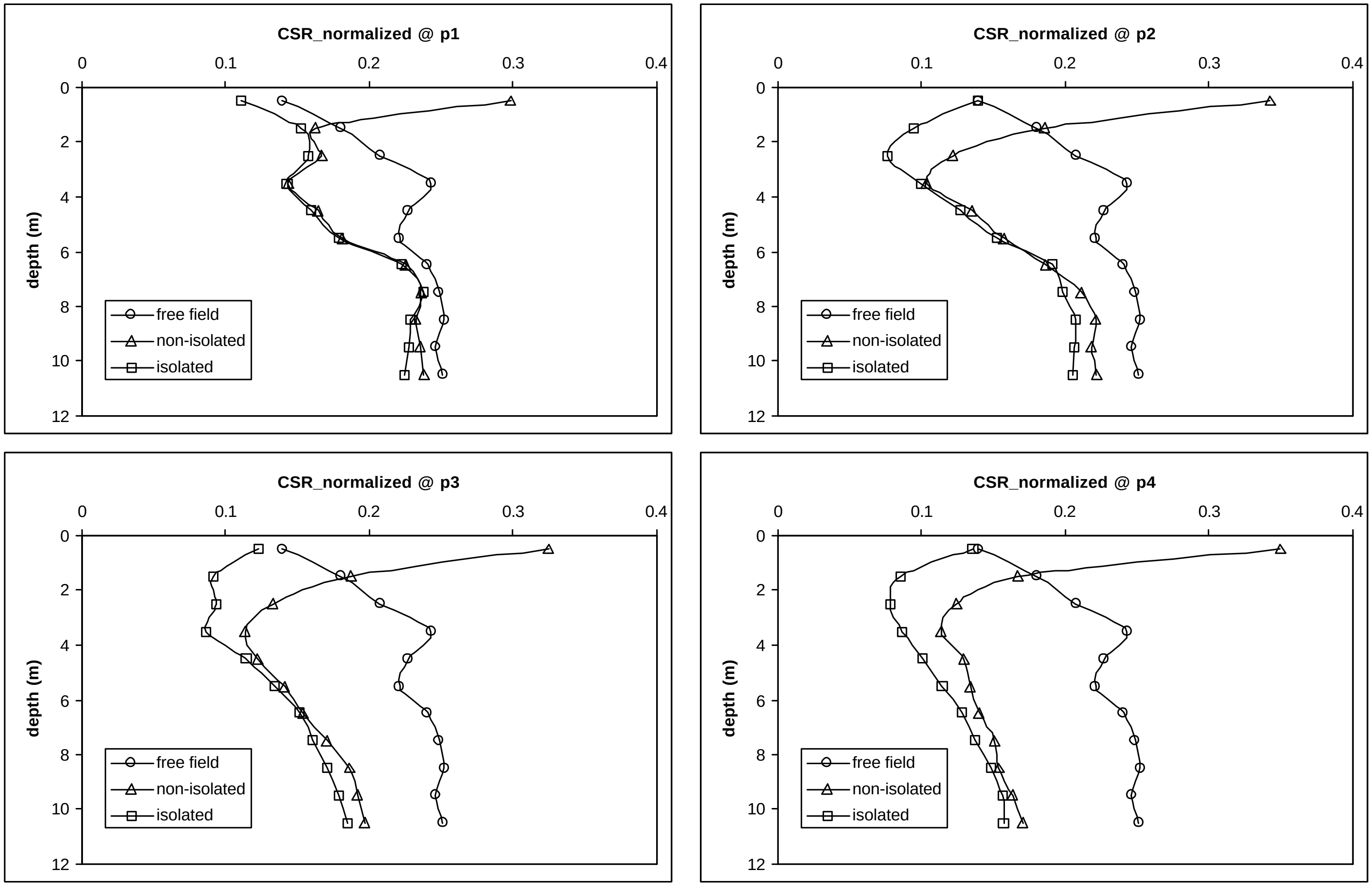

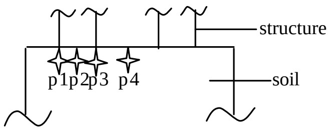

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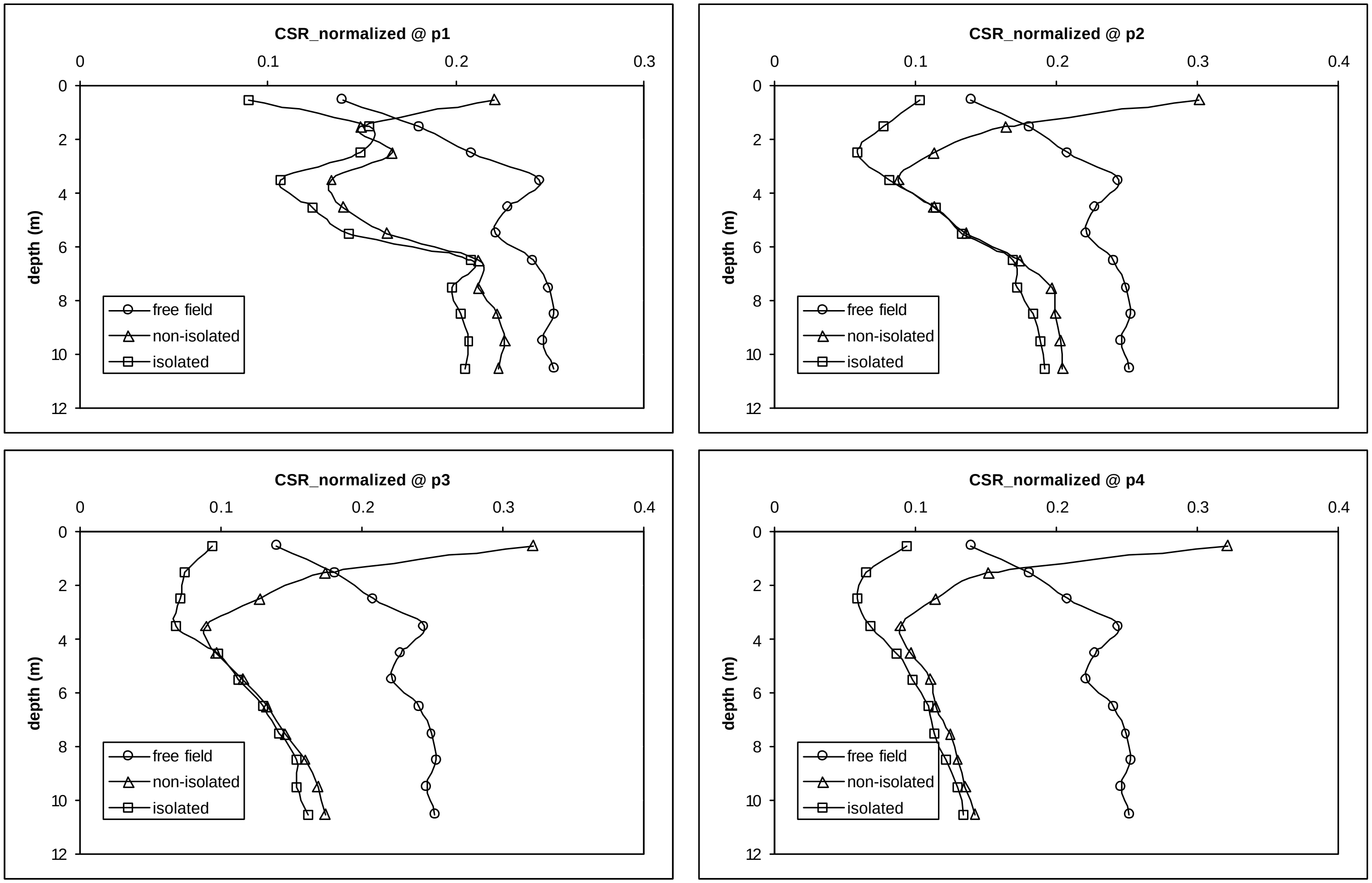


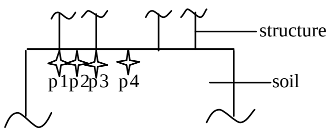
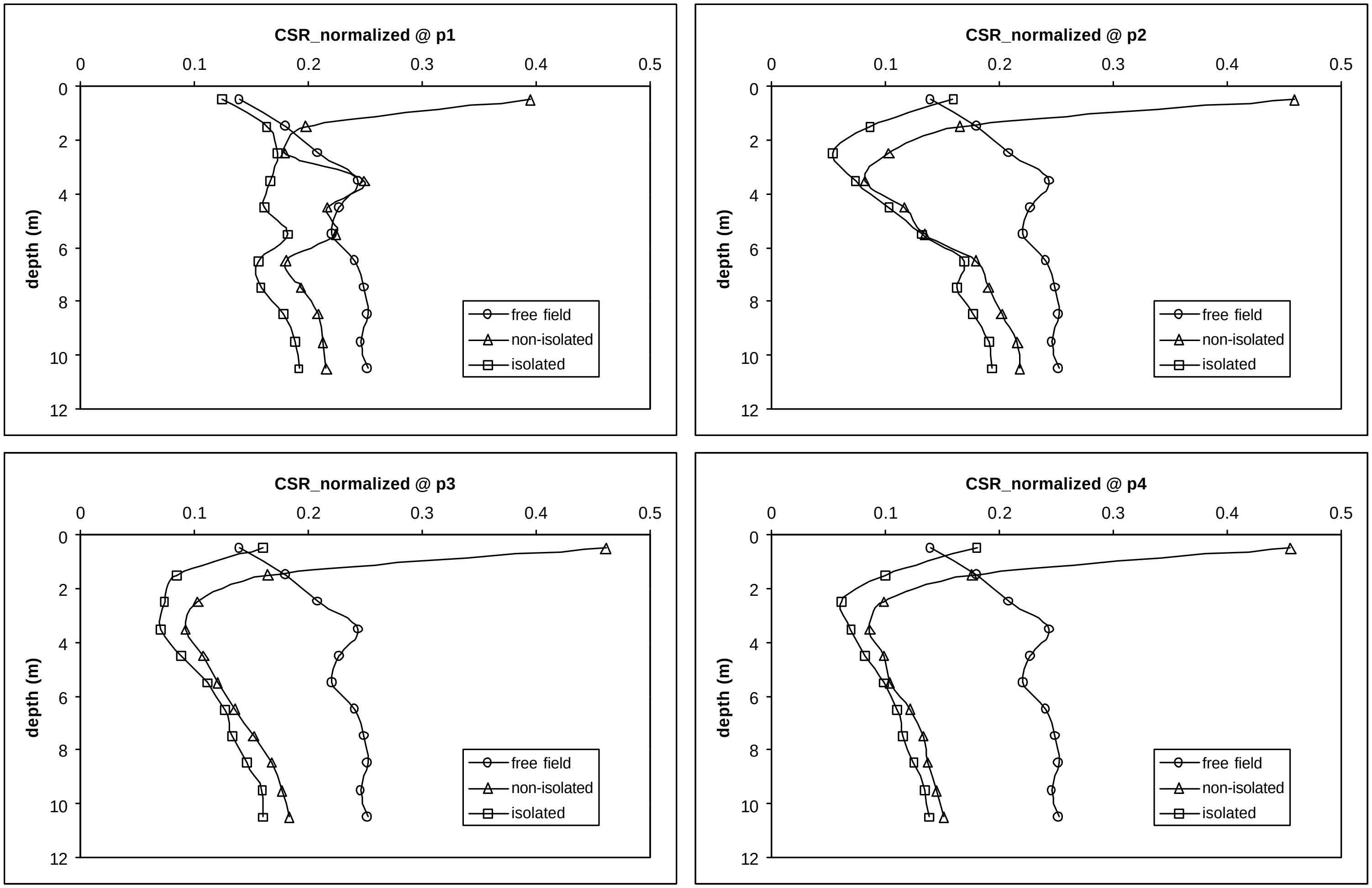


Figure A.14 Third Type of Structure with first site under El-Centro Earthquake



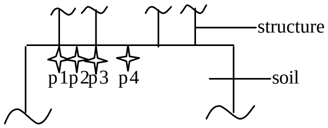


Figure A.15 Fourth Type of Structure with first site
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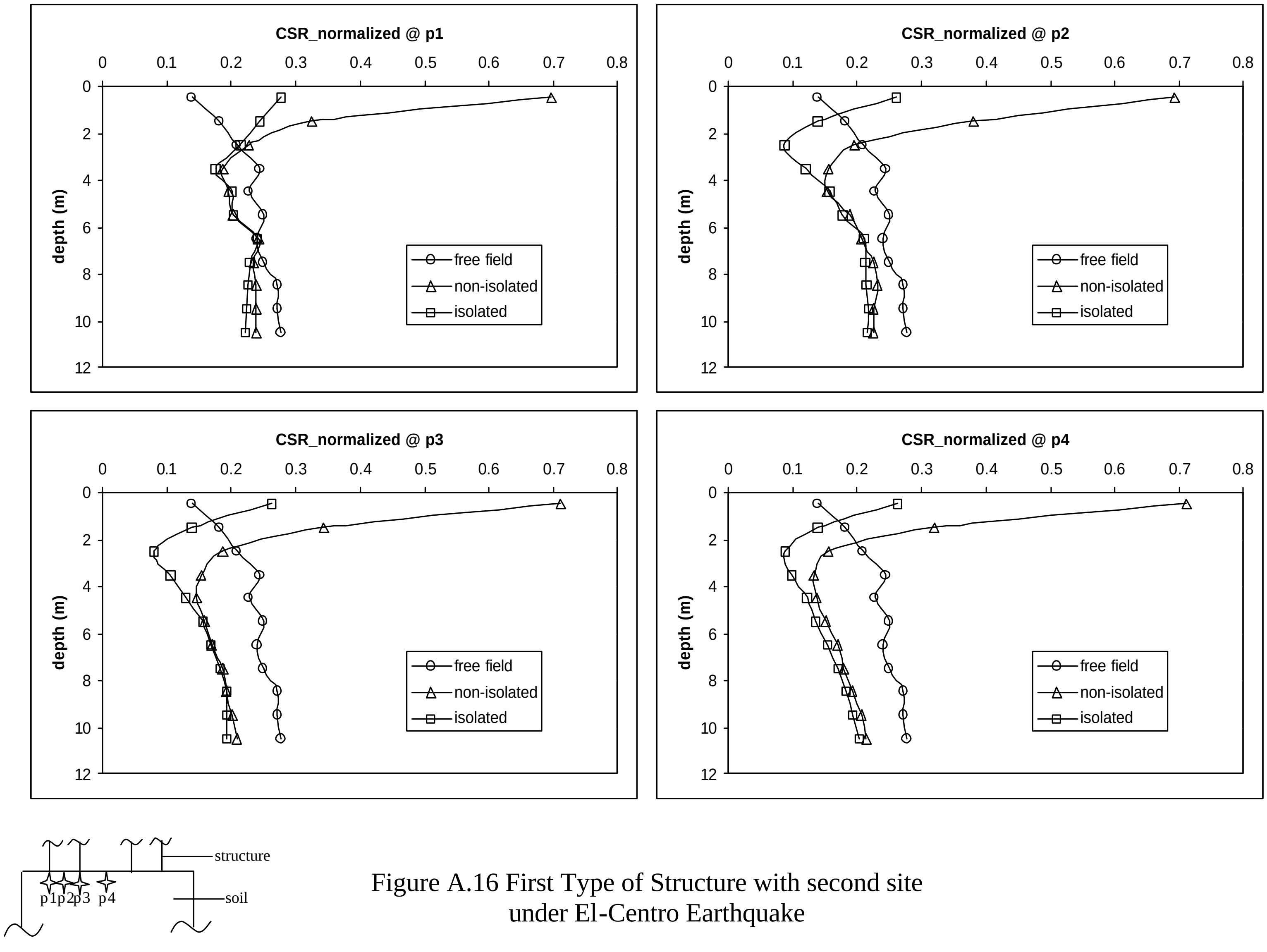

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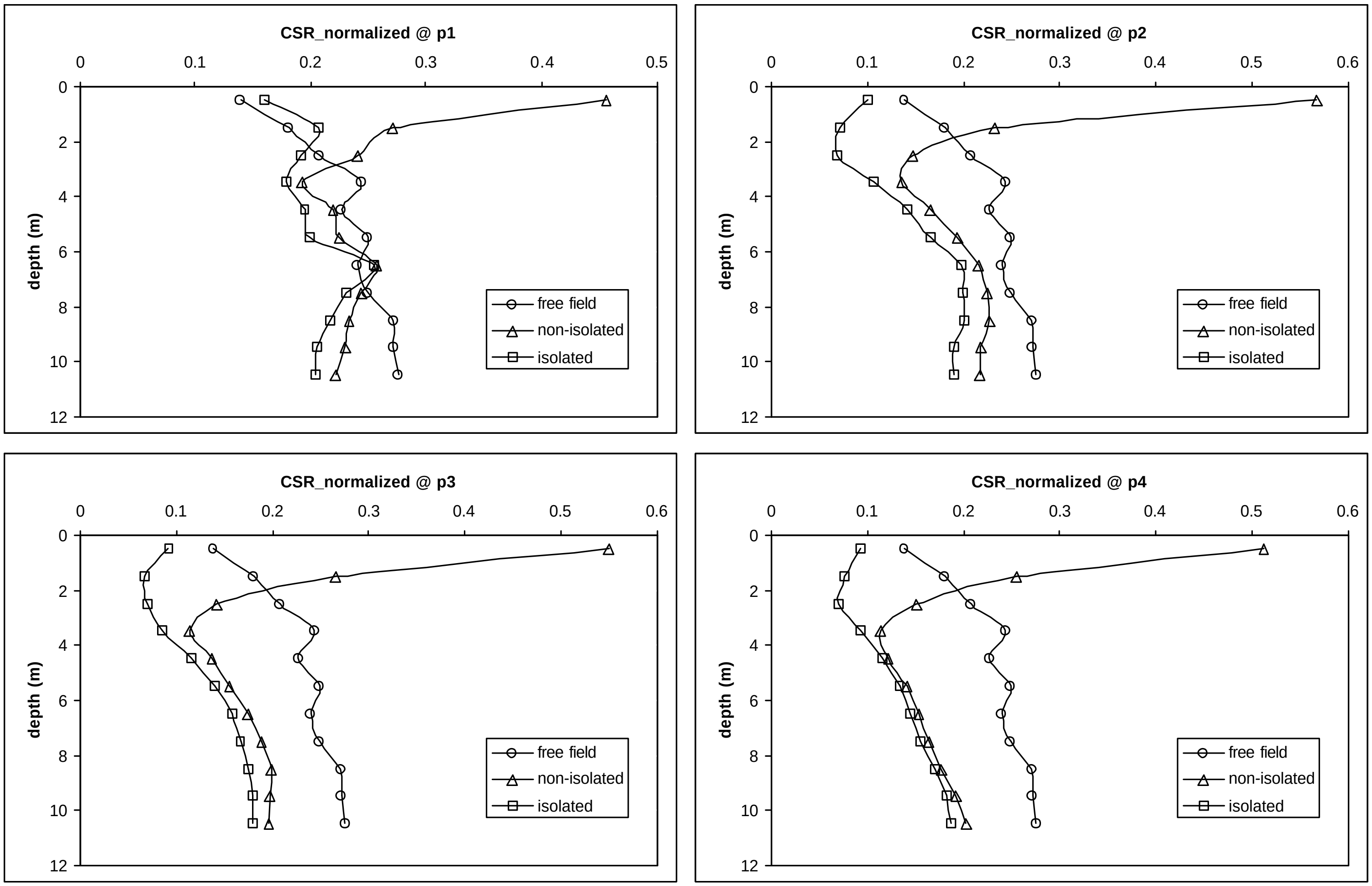

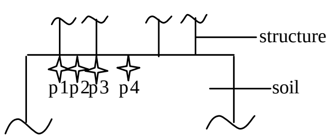

Figure A.17 Second Type of Structure with second site
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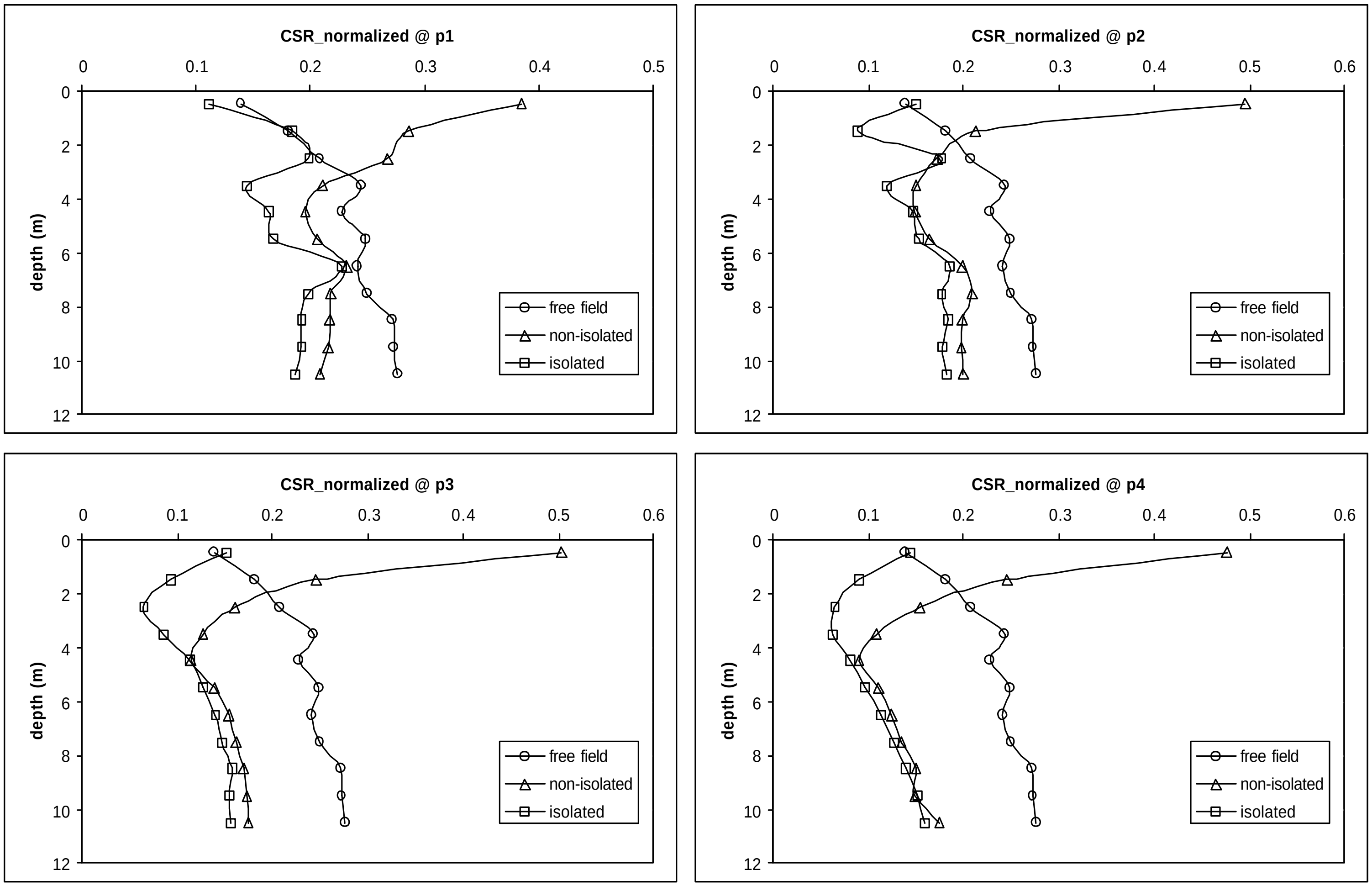

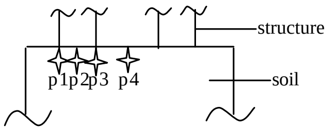

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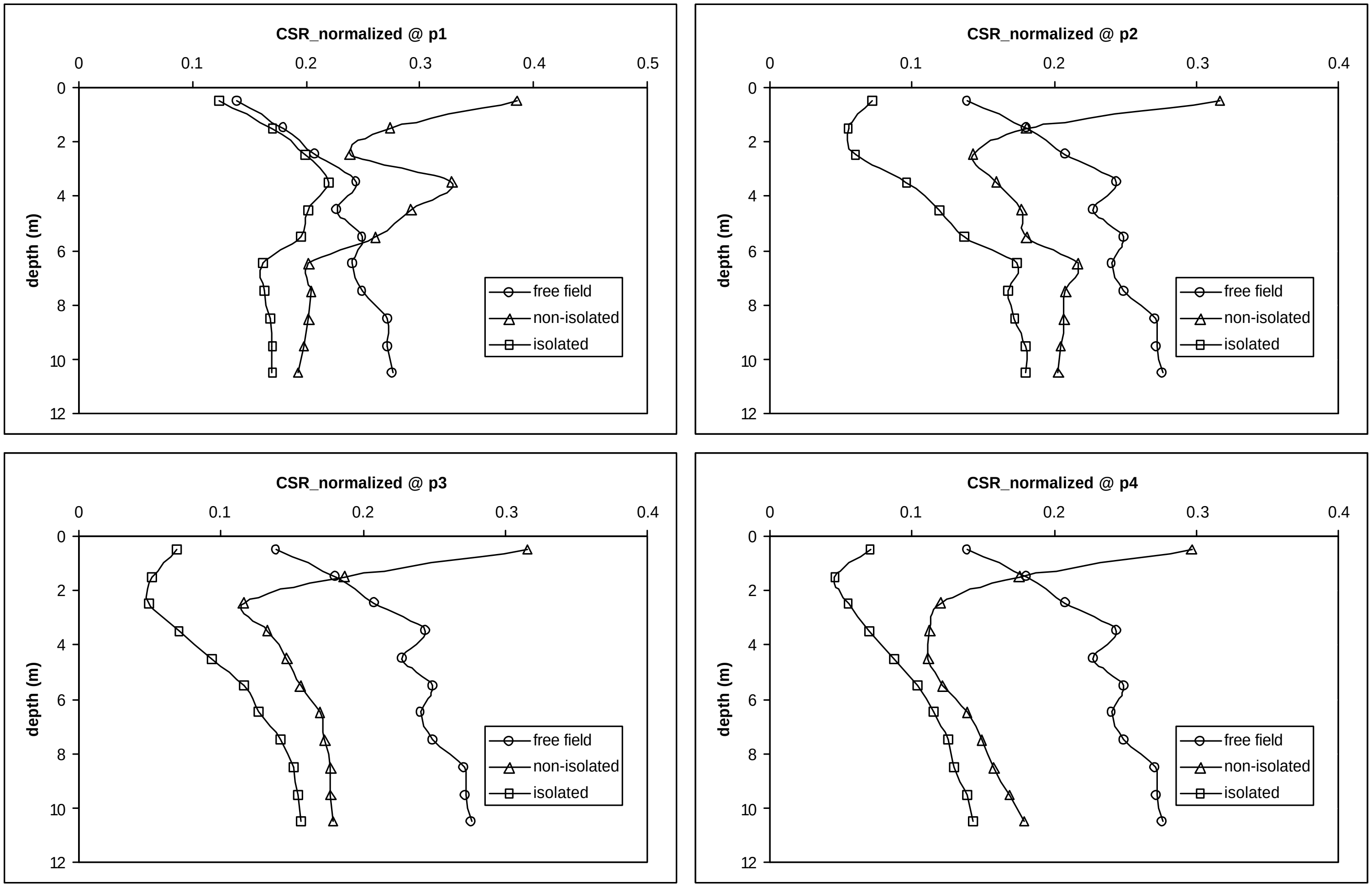

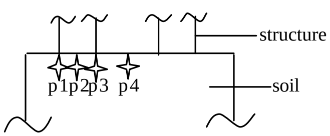

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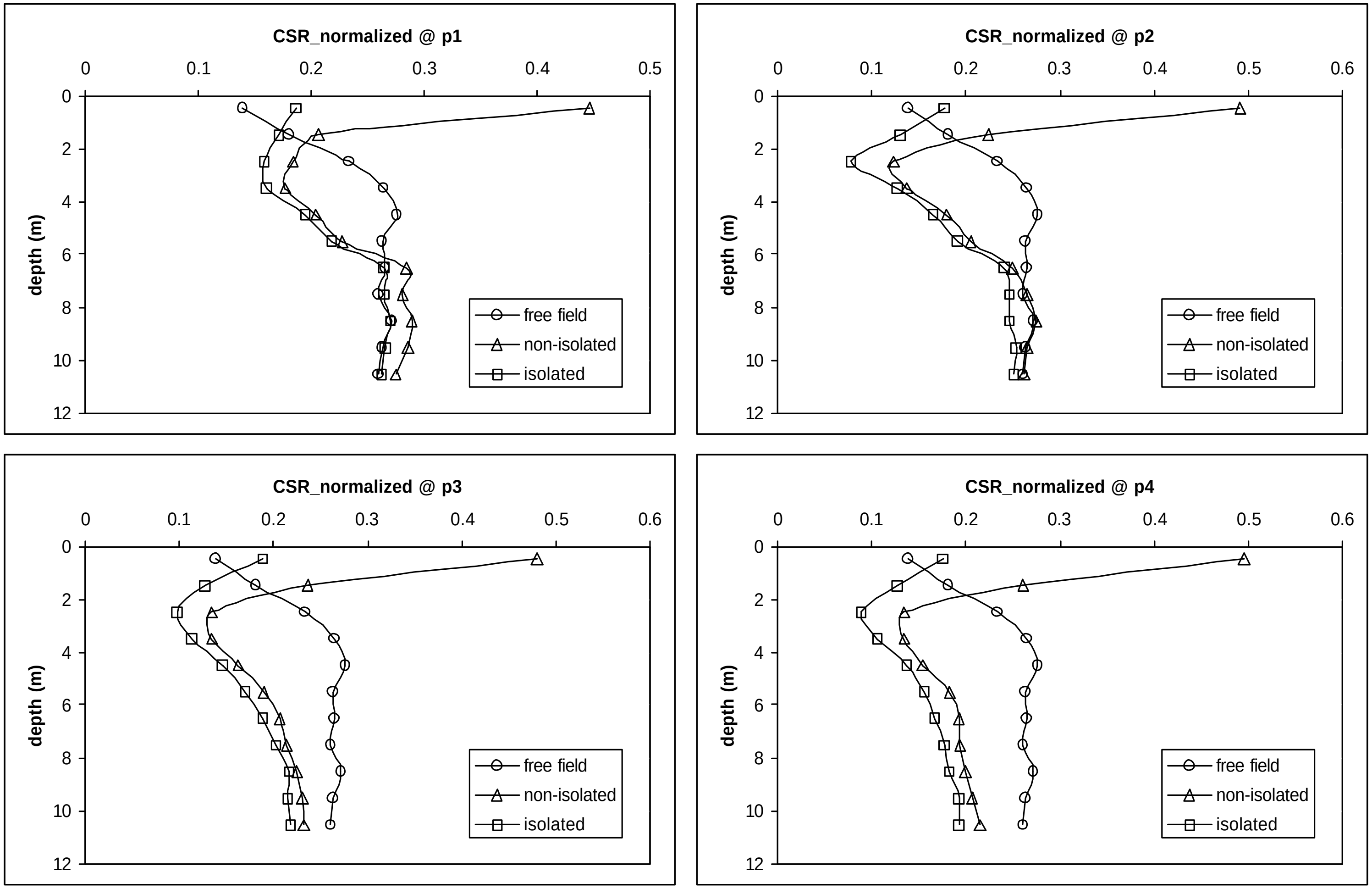

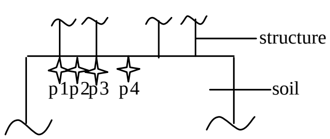

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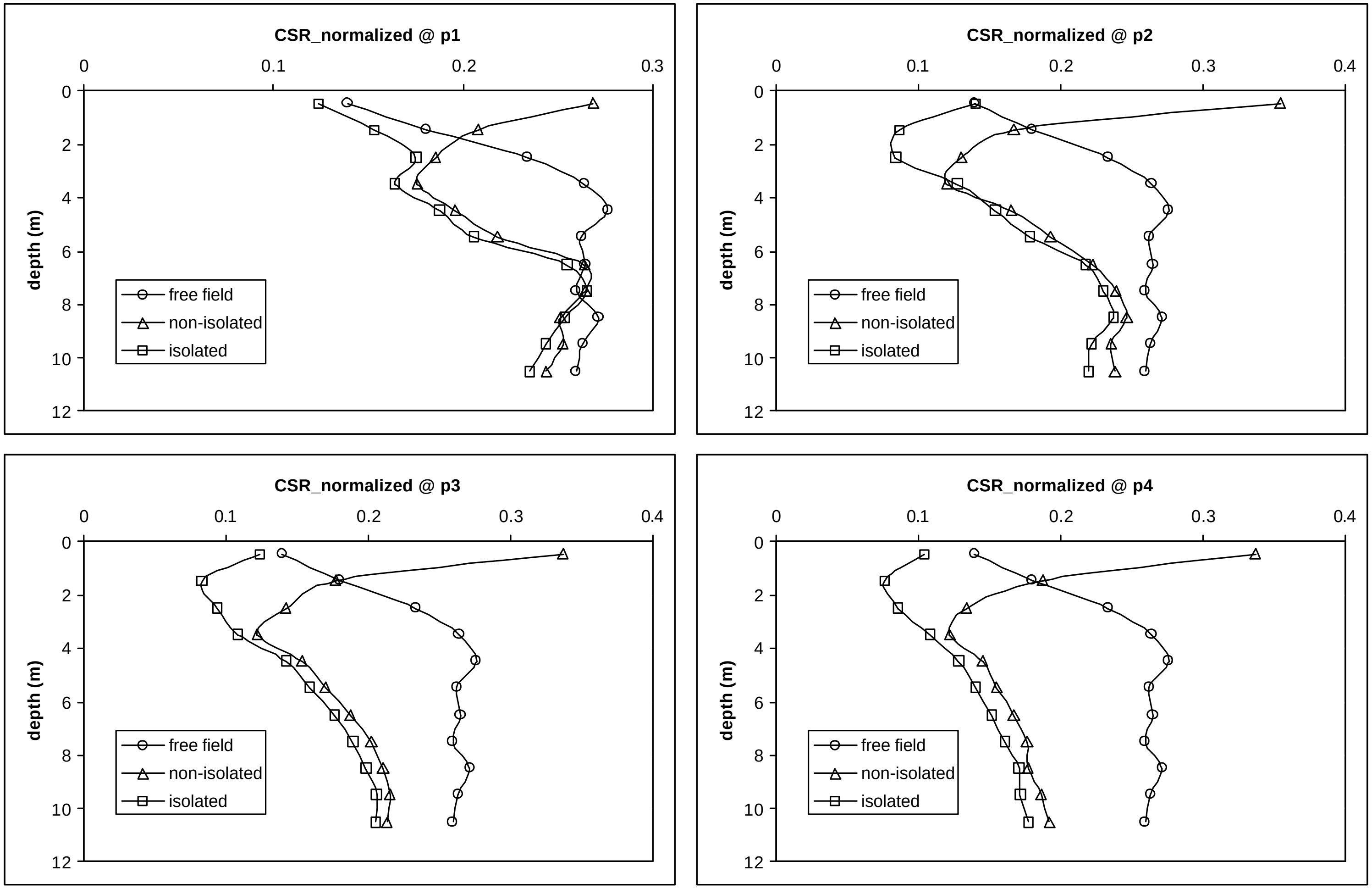

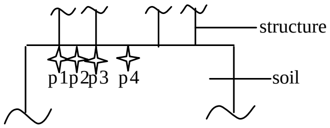

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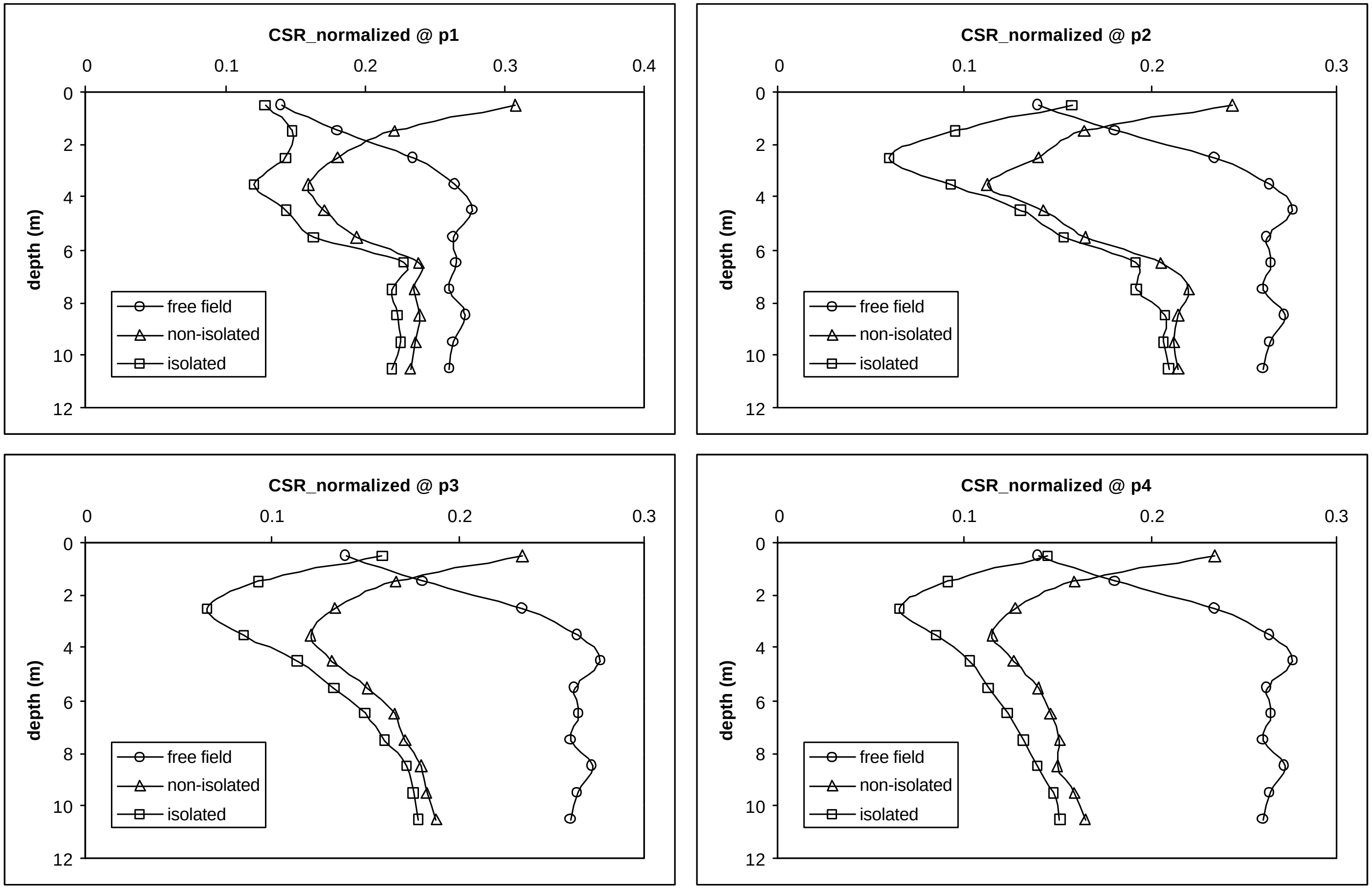


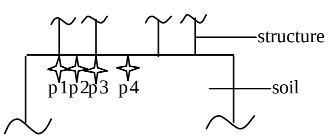
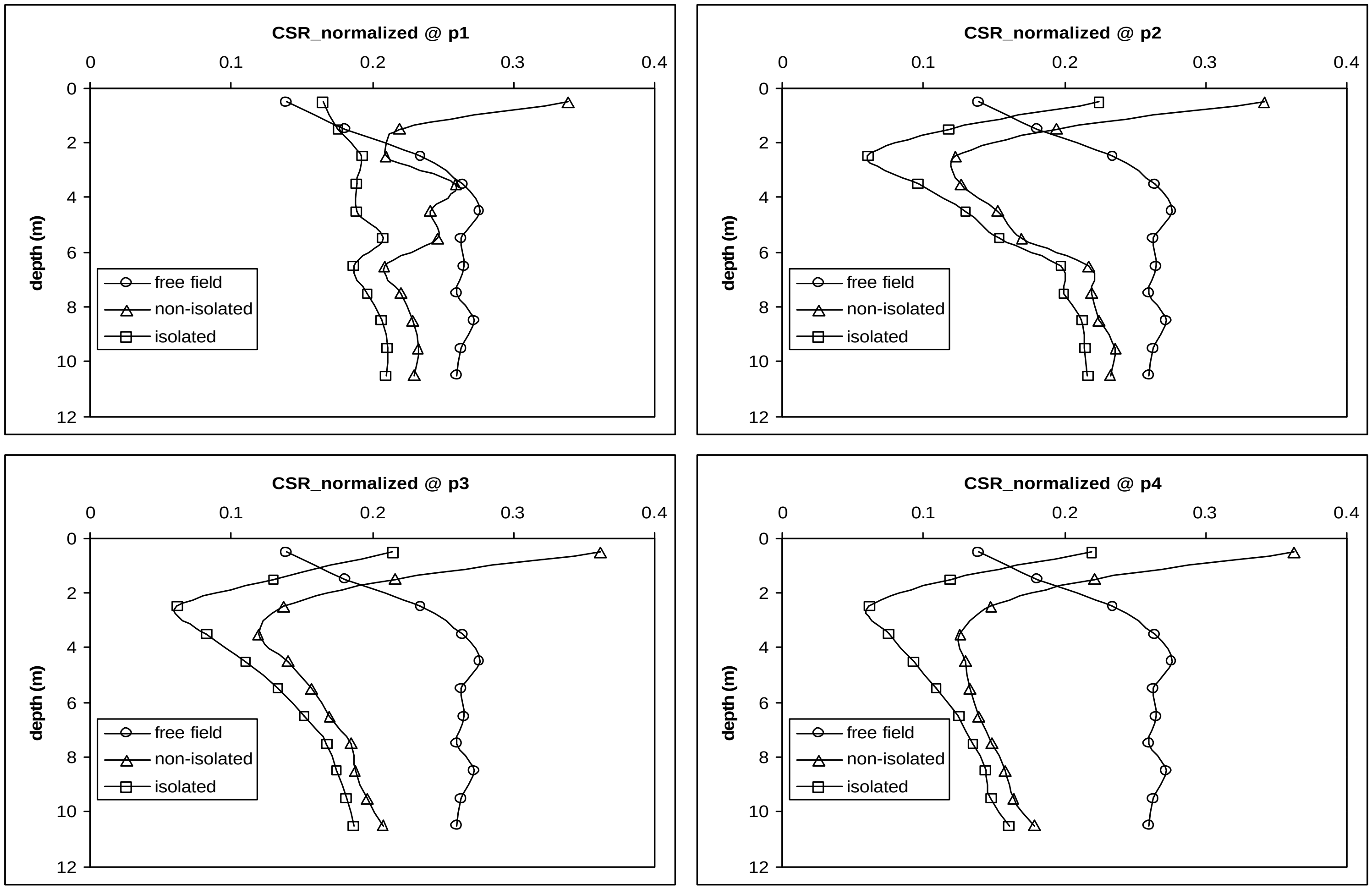


Figure A.22 Third Type of Structure with third site
under El-Centro Earthquake



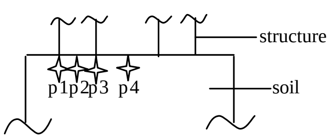


Figure A.24 Fourth Type of Structure with third site
under El-Centro Earthquake

APPENDIX B

STRUCTURAL RESPONSE VALUES

In Appendix B the structural response values can be found. For each combination of different input motion, site and structure; base shear forces obtained from SAP analysis to stand for fixed base case, and shear forces obtained from FLAC analysis to stand for flexible base case, were given. Also the relative displacements obtained from the FLAC analysis were shown. All the results were given both for non isolated and isolated structures.

Table B.1 Structural Response Values under Kocaeli Earthquake on the first site

3 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	536	76
Base shear obtained by SAP (kN)	635	48
Rel. Disp. on the first floor (cm)	1.25	0.21
Rel. Disp. on the second floor (cm)	0.95	0.15
Rel. Disp. on the third floor (cm)	0.55	0.08

*Base displacement: 28 cm.

4 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	572	80
Base shear obtained by SAP (kN)	798	56
Rel. Disp. on the first floor	1.32	0.23
Rel. Disp. on the second floor	1.25	0.20
Rel. Disp. on the third floor	0.92	0.18
Rel. Disp. on the fourth floor	0.51	0.08

*Base displacement: 36 cm.

4 story building (%50 additional weight)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	624	130
Base shear obtained by SAP (kN)	1004	68
Rel. Disp. on the first floor	0.92	0.21
Rel. Disp. on the second floor	0.75	0.16
Rel. Disp. on the third floor	0.53	0.11
Rel. Disp. on the fourth floor	0.32	0.07

*Base displacement: 36 cm.

6 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	372	136
Base shear obtained by SAP (kN)	471	104
Rel. Disp. on the first floor	0.90	0.32
Rel. Disp. on the second floor	0.82	0.28
Rel. Disp. on the third floor	0.78	0.25
Rel. Disp. on the fourth floor	0.61	0.24
Rel. Disp. on the fifth floor	0.52	0.19
Rel. Disp. on the sixth floor	0.33	0.14

*Base displacement: 45 cm.

Table B.2 Structural Response Values under Kocaeli Earthquake on the second site

3 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	356	90
Base shear obtained by SAP (kN)	383	48
Rel. Disp. on the first floor	0.82	0.21
Rel. Disp. on the second floor	0.64	0.18
Rel. Disp. on the third floor	0.35	0.12

*Base displacement: 40 cm.

4 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	596	88
Base shear obtained by SAP (kN)	567	60
Rel. Disp. on the first floor	1.41	0.20
Rel. Disp. on the second floor	1.23	0.17
Rel. Disp. on the third floor	0.96	0.14
Rel. Disp. on the fourth floor	0.55	0.08

*Base displacement: 42 cm.

4 story building (%50 additional weight)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	696	135
Base shear obtained by SAP (kN)	716	72
Rel. Disp. on the first floor	1.39	0.30
Rel. Disp. on the second floor	1.15	0.25
Rel. Disp. on the third floor	0.86	0.19
Rel. Disp. on the fourth floor	0.49	0.11

*Base displacement: 38 cm.

6 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	556	116
Base shear obtained by SAP (kN)	758	108
Rel. Disp. on the first floor	1.32	0.31
Rel. Disp. on the second floor	1.21	0.25
Rel. Disp. on the third floor	1.11	0.22
Rel. Disp. on the fourth floor	0.82	0.18
Rel. Disp. on the fifth floor	0.64	0.13
Rel. Disp. on the sixth floor	0.35	0.09

*Base displacement: 52 cm.

Table B.3 Structural Response Values under Kocaeli Earthquake on the third site

3 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	584	84
Base shear obtained by SAP (kN)	465	60
Rel. Disp. on the first floor	1.31	0.22
Rel. Disp. on the second floor	1.12	0.15
Rel. Disp. on the third floor	0.55	0.09

*Base displacement: 32 cm.

4 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	632	100
Base shear obtained by SAP (kN)	670	68
Rel. Disp. on the first floor	1.52	0.23
Rel. Disp. on the second floor	1.21	0.21
Rel. Disp. on the third floor	0.92	0.16
Rel. Disp. on the fourth floor	0.53	0.11

*Base displacement: 38 cm.

4 story building (%50 additional weight)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	924	132
Base shear obtained by SAP (kN)	840	84
Rel. Disp. on the first floor	1.25	0.35
Rel. Disp. on the second floor	1.03	0.35
Rel. Disp. on the third floor	0.82	0.25
Rel. Disp. on the fourth floor	0.45	0.25

*Base displacement: 36 cm.

6 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	488	142
Base shear obtained by SAP (kN)	600	124
Rel. Disp. on the first floor	1.24	0.32
Rel. Disp. on the second floor	1.22	0.28
Rel. Disp. on the third floor	1.11	0.27
Rel. Disp. on the fourth floor	0.74	0.23
Rel. Disp. on the fifth floor	0.65	0.19
Rel. Disp. on the sixth floor	0.35	0.11

*Base displacement: 48 cm.

Table B.4 Structural Response Values under El-Centro Earthquake on the first site

3 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	152	16
Base shear obtained by SAP (kN)	304	16
Rel. Disp. on the first floor	0.35	0.04
Rel. Disp. on the second floor	0.25	0.03
Rel. Disp. on the third floor	0.15	0.02

*Base displacement: 8 cm.

4 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	256	20
Base shear obtained by SAP (kN)	359	20
Rel. Disp. on the first floor	0.55	0.04
Rel. Disp. on the second floor	0.47	0.03
Rel. Disp. on the third floor	0.35	0.03
Rel. Disp. on the fourth floor	0.22	0.02

*Base displacement: 10 cm.

4 story building (%50 additional weight)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	320	28
Base shear obtained by SAP (kN)	520	30
Rel. Disp. on the first floor	0.42	0.04
Rel. Disp. on the second floor	0.35	0.04
Rel. Disp. on the third floor	0.28	0.03
Rel. Disp. on the fourth floor	0.17	0.02

*Base displacement: 9 cm.

6 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	404	36
Base shear obtained by SAP (kN)	439	32
Rel. Disp. on the first floor	1.13	0.09
Rel. Disp. on the second floor	0.81	0.08
Rel. Disp. on the third floor	0.71	0.08
Rel. Disp. on the fourth floor	0.55	0.06
Rel. Disp. on the fifth floor	0.35	0.04
Rel. Disp. on the sixth floor	0.18	0.03

*Base displacement: 17 cm.

Table B.5 Structural Response Values under El-Centro Earthquake on the second site

3 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	136	20
Base shear obtained by SAP (kN)	230	20
Rel. Disp. on the first floor	0.32	0.05
Rel. Disp. on the second floor	0.25	0.03
Rel. Disp. on the third floor	0.12	0.02

*Base displacement: 8 cm.

4 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	360	20
Base shear obtained by SAP (kN)	416	20
Rel. Disp. on the first floor	0.82	0.09
Rel. Disp. on the second floor	0.73	0.08
Rel. Disp. on the third floor	0.54	0.05
Rel. Disp. on the fourth floor	0.25	0.03

*Base displacement: 9 cm.

4 story building (%50 additional weight)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	468	32
Base shear obtained by SAP (kN)	680	32
Rel. Disp. on the first floor	0.63	0.05
Rel. Disp. on the second floor	0.51	0.04
Rel. Disp. on the third floor	0.42	0.04
Rel. Disp. on the fourth floor	0.25	0.03

*Base displacement: 10 cm.

6 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	408	40
Base shear obtained by SAP (kN)	737	44
Rel. Disp. on the first floor	0.84	0.09
Rel. Disp. on the second floor	0.84	0.09
Rel. Disp. on the third floor	0.75	0.08
Rel. Disp. on the fourth floor	0.65	0.07
Rel. Disp. on the fifth floor	0.53	0.04
Rel. Disp. on the sixth floor	0.35	0.04

*Base displacement: 15 cm.

Table B.6 Structural Response Values under El-Centro Earthquake on the third site

3 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	272	16
Base shear obtained by SAP (kN)	259	16
Rel. Disp. on the first floor	0.72	0.05
Rel. Disp. on the second floor	0.51	0.04
Rel. Disp. on the third floor	0.23	0.02

*Base displacement: 8 cm.

4 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	236	20
Base shear obtained by SAP (kN)	372	24
Rel. Disp. on the first floor	0.55	0.05
Rel. Disp. on the second floor	0.54	0.05
Rel. Disp. on the third floor	0.35	0.04
Rel. Disp. on the fourth floor	0.25	0.03

*Base displacement: 10 cm.

4 story building (%50 additional weight)

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	340	32
Base shear obtained by SAP (kN)	532	28
Rel. Disp. on the first floor	0.58	0.06
Rel. Disp. on the second floor	0.53	0.05
Rel. Disp. on the third floor	0.36	0.05
Rel. Disp. on the fourth floor	0.21	0.04

*Base displacement: 10 cm.

6 story building

	Non isolated	Isolated
Base shear obtained by FLAC (kN)	456	38
Base shear obtained by SAP (kN)	657	32
Rel. Disp. on the first floor	1.11	0.09
Rel. Disp. on the second floor	1.22	0.09
Rel. Disp. on the third floor	1.23	0.08
Rel. Disp. on the fourth floor	0.75	0.06
Rel. Disp. on the fifth floor	0.52	0.04
Rel. Disp. on the sixth floor	0.27	0.03

*Base displacement: 17 cm.