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M.Sc. in Civil Engineering

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ANALYSIS OF STEEL FRAMES WITH SOIL STRUCTURE INTERACTION UNDER SEISMIC LOADS

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Analysis of Steel Frames with Soil Structure Interaction under Seismic Loads

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Supervisor

Prof.Dr. Mustafa ÖZAKÇA

by

Cemile ÖZŞAM

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This is to certify that we have read this thesis and that in our majority opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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

Cemile ÖZŞAM

ABSTRAT

ANALYSIS OF STEEL FRAMES WITH SOIL STRUCTURE INTERACTION UNDER SEISMIC LOADS

ÖZŞAM, Cemile
M.Sc. in Civil Engineering
Supervisor: Prof. Dr. Mustafa ÖZAKÇA
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In this research work, response spectrum and linear/nonlinear time history analysis of 2D multi-story steel frames on gravel soil with and with-out soil structure interactions have been done. Frames have different height start from 3 stories and up to 20 stories. A sub-structure method is used to model soil and shallow (footing) foundation system by linear springs and dashpots. Three earthquake records are selected, Kocaeli, Erzincan and Düzce earthquake accelerograms records are selected and scaled to specific response spectrum with PGA 0.35 and soil type B according to Eurocode 8 is considered. Fundamental period and base shears have been investigated with and without soil-structure interactions; the results have been presented and compared.

Key Words: Steel frames, soil structure interaction, response spectrum analysis, time history analysis, seismic loads, fundamental period.

ÖZET

DEPREM YÜKLERI ALTINDA ZEMIN YAPISI ETKILEŞIMLI ÇELIK ÇERÇEVELERIN ANALIZI

ÖZŞAM, Cemile
Yüksek Lisans, İnşaat Müh. Bölümü
Tez Yöneticisi: Prof. Dr. Mustafa ÖZAKÇA
Aralık 2018
73 sayfa

Bu araştırmada, zemin yapısı etkileşimli ve etkileşimsiz, 2 boyutlu çok katlı çelik çerçeveler için doğrusal ve doğrusal olmayan zaman adım analizi yapılmıştır. Çerçeveler 3 kattan 20 katına kadar farklı yüksekliklere sahiptir. Zemin ve tekil temel sistemi doğrusal yaylar ve amartisor ile modellemek için alt yapı analiz metodu kullanılmıştır. Ayrıca tepki-spektrum analizi yapılmıştır. Düzce deprem ivme kaydı seçilerek spesifik tepki spektrumuna ölçeklendirildi. PGA 0.35 ve Eurocode 8'e göre zemin tipi B (katı) seçildi. Temel periyot ve zemin kesmesi, zemin-yapı etkileşimi dahil edilerek ve edilmeyerek araştırılmış ve sonuçlar karşılaştırılmıştır.

Anahtar Kelimeler: Çelik çerçeve, zemin yapı etkileşimi, tepki sepktrum analizi, zaman adım analizi, sismik yük, temel periyot.

To Every One Support Me

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Finally, my special thanks are reserved for my husband for his understanding, patience and constant support. Big huge for my child Meryem which I got my power from her words

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LIST OF ABBRIVATION/SYMBOLS

AISC	American Institute of Steel Construction
FEMA	Federal Emergency Management Agency
IDR	Inter story DRift
MMI	Modified Mercalli Intensity scale
MMS	Moment Magnitude Scale
PGA	Peak Ground Acceleration
SSI	Soil structure interaction
UBC	Uniform Building Code
μ	The rigidity of soil
E	The bulk modulus of elasticity for soil
ρ	Density of soil
v_p	Primary waves velocity
γ	Unit weight of soil
G	Shear modulus of soil
ν	Passion ration of soil
g	Gravitational acceleration
v_s	Shear waves velocity
M	Earthquake magnitude
A	Maximum amplitude of ground motion
D	Distance from epicenter to location of measuring (measured in km)
M_w	Moment Magnitude Scale
u_g	Ground motion
S_d	Maximum displacement relative to the ground of the oscillator

SA	maximum acceleration of the oscillator which is absolute (total value)
SV	Maximum velocity relative to the ground of the oscillator
S_v	Pseudo velocity
S_a	Pseudo acceleration
T	Natural period of the structure
ω	Circular frequency of single degree of freedom model
M	Mass of single degree of freedom model
K	Stiffness of single degree of freedom model
V_e	Base shear force
W	Weight of structure
2D	Two dimension
3D	Three dimenion
Δ	Geometric deformation
P	Verical loads
δ	Curvature deformation
T_{ssi}	Fundamental period with soil structure interaction effect
m^*	Effective modal mass of the fundamental vibration mode of the structure
ζ_0	Damping ratio of structure with soil structure interaction effect
ζ_f	Damping of foundation
h	Effective height of the structure
k_θ	Effective rocking stiffness of the structure
k_x	Effective swaying stiffness of the structure
u_{gm}	Free-field motion, modified for base slab averaging effects for inertial interaction
F_e	Maximum force during the entire excitation of the earthquake based on the linear elastic response of the structure
Δ_e	Maximum displacement during the entire excitation of the earthquake based on the linear elastic response of the structure

F_y	Force at first significant yielding point of structure
Δ_y	Displacement at first significant yielding point of structure
NS	Number of stories
NB	Number of bays
RSN	Record Sequence Number
Rrup	Closet distance from rupture to station
V_{s30}	Average value of propagation velocity of S waves in the upper 30m of the soil profile at shear strain of 10-5
MSE	Mean Squared Error
β	Mass (inertia) Ratio
q	Bearing capacity of the soil
G_s	Secant shear modulus of soil

CHAPTER 1

INTRODUCTION

1.1 Background

The seismic design of structures has advanced steadily by deepening of theoretical foundations. The significant progress in field of seismic design in last four decades is a result of parallel progress in structural analysis and computer hardware. Huge efforts have been done to produce practical methods to simulate SSI. Although some seismic codes produced new methods to take SSI into consideration, still this point needs more researches.

Dynamic properties of the structure are affected by the type of structural system, foundation system and soil. Which in turn effect on the seismic response of the structure, and a different response may happen for the same buildings on a different soil type especially when one soil is weak and the other is rock.

Steel structures were not economical until late in nineteen century, it has become the predominated material for construction of bridges, buildings, towers, and other structure. Steel exhibited desirable physical proprieties that make it one of the most versatile structural materials in use. It is a great strength, uniformly lightweight, easy to use, elasticity which means steel obey Hooks law accurately, ductility and many other properties. One of the steel structures to resist loads is frames, and there are many types of steel frames according to the way of resistive loads.

Structural frames are composed of one-dimensional members connected together in skeletal arrangements which transfer the applied loads to the supports. While most frames are three-dimensional, they may often be considered as a series of parallel two-dimensional frames, or as two perpendicular series of two-dimensional frames. The behavior of a structural frame depends on its arrangement and loading, and on the type of connections used.

When a frame is considered as two-dimensional, then its behavior is similar to that of the beam-columns of which it is composed. With in-plane loading only, it will fail either by in-plane bending or by flexural-torsional buckling out of its plane. However, if the frame or its loading is three-dimensional, then it will fail in a mode in which the individual members are subjected to primary biaxial bending and torsion actions.

Steel frames structure has a system to resist lateral forces. This system is that part of the structural system which designed to resist the seismic forces and steel frames are classified according to this system

1.2 Steel Structure Frame Types

Frames are considered the fundamental structure in engineering. The frame is a 2D series of joined members which are beams and columns. In real structure, the frames are located in two orthogonal directions and connected together to produce 3D frames structure [1]. Steel buildings shall be assigned to one of the following structural types according to the behavior of their primary resisting structure under seismic actions:

a) Moment resisting frames is the most common steel frame type. In this type a rigid connection between columns and beams are used. According to these rigid connections, the frame is able to resist all loads types in vertical and lateral directions, also overturning forces because of moment and shear strength that is developed in its members and connection joints. During seismic action, moment frames resist by stiffness and strength of its member. The main advantage of moment frame is allowing large movements during earthquake [1].

b) Frames with eccentric bracings are those in which the horizontal forces are mainly resisted by axially loaded members, but where the eccentricity of the layout is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear [2].

c) Inverted pendulum structures which have dissipative zones at the bases of columns.

d) Moment resisting frames combined with concentric bracings.

e) Moment resisting frames combined with infill.

The proprieties and other details of the steel frames could be found in AISC [2].

In moment resisting frames, the dissipative zones should be mainly located in plastic hinges in the beams or the beam-column joints so that energy is dissipated by means of cyclic bending. The dissipative zones may also be located at:

1. the base of the frame
2. the top of the columns in the upper story of multi-story buildings
3. the top and bottom of columns in single-story buildings

In frames with concentric bracings, the dissipative zones should be mainly located in the tensile diagonals. It is preferred to locate the plastic hinge in beams to achieve ductile fail, while if the plastic hinges in columns, the failure will be brittle as shown in Figure 1.1 [2].

(a) (b) (c)

Figure 1.1 Plastic hinges locations and failure type (a) frame, (b) weak beams – ductile failure, (c) weak columns – sudden failure [3].

1.3 Earthquake Loads on Steel Frame Structures

Damage experienced during past earthquakes worldwide demonstrates that steel multi-story building structures generally exhibit adequate seismic response. This is due to the favorable mass-to-stiffness ratio of base metal and the enhanced energy absorption of structural ductile systems employed.

Many codes like UBC 97 [4], AISC [2], Euro code 8 [5] and give a calculation of earthquake loads on steel frames and determine the response of steel frames and

detailing to resist loads according to earthquake specification and building lateral system to resist seismic loads. Main methods to evaluate earthquake response are

- *Lateral force method of analysis*: This type of analysis is applied to regular buildings or has limited irregular defined by codes with limited height, and their response depend on the fundamental period of building.
- *Model response spectrum analysis*: This type of analysis is linear dynamic analysis. It is applied to all type of structures which are regular or irregular. All model of vibration is considered in response. A number of modes which the structure needs depends on the sum of model mass. Sum of effective model mass should be more than 90% of building mass. A response spectrum is two horizontal and one vertical.
- *Time history analysis which could be linear or nonlinear analysis*: In this case real records of the earthquake are used and applied to the structure. Each earthquake has three records, two horizontal and one vertical. Earthquake record components are applied in a positive and negative direction, and then the maximum response is used. One of the most important points, when time history analysis is applied, is choosing seismic hazards and scale factor. Choosing earthquake records depends on many factors like Peak Ground Acceleration (PGA) of the earthquake, soil type, and shear waves velocity. All records are scaled to design response spectrum [5].

1.4 Soil Structure Interaction Phenomenon

Modern construction contains a lot of high rise buildings which are built in many seismic zones. It is very important to understand their behavior under seismic loads and model buildings with a more realistic view to keep people in safe. This investigation leads to study SSI because normally bases in seismic analysis of high rise buildings are considered fixed without any take into account foundation system or soil beneath the building as shown in Figure 1.2. But according to the recent earthquake, the effect of soil and foundation system is very clear on the behavior of buildings, so huge efforts are directed to simulate SSI and study their effect on buildings response [6].

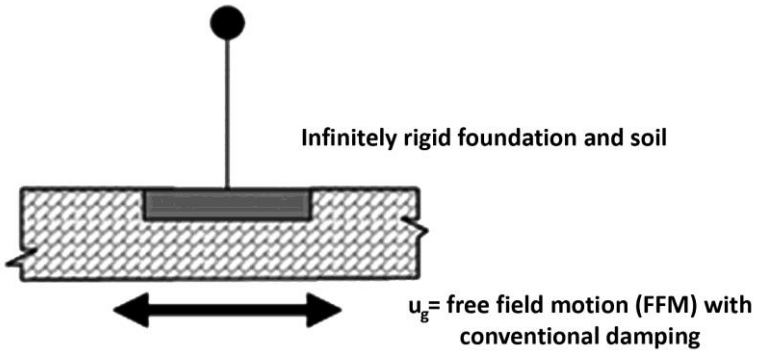


Figure 1.2 Rigid base models [6].

Soil behave as linear material in low stain range, while when large stains happened a behavior is changes to nonlinear. During earthquake, soil has larg stains because earthquake waves effect and large dispalcments and the behavior of soil is changed to nonlinearity [7]

The main soil parameter which is used in seismic analysis is shear modulus of soil. Shear modulus is defined as a ratio of shear stress to shear stains. Soil beahvior during earthquake is nonlinear and tangent shear modulus of soil is used, but in equvilent linear approach a scent shear modulus. Secent shear modulus is shown in Figure 1.3 [7].

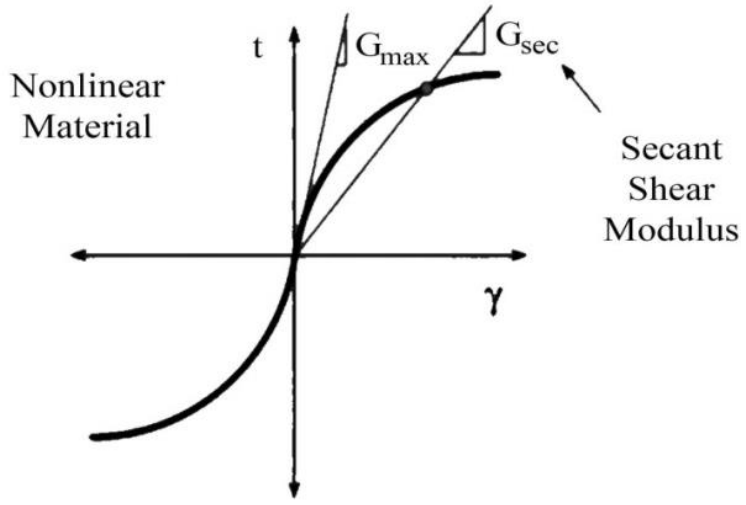


Figure 1.3 Secant shear modulus of soil [7]

Many methods have been developed to model SSI depending on behavior of soil and structure and development in computer hardware and software. Main two methods are substructure method, and direct method

1.4.1 Substructure approach

This approach is considered one of most popular approach to model SSI. In this method the structural system divides into two parts, superstructure and substructure. The part over grade level is called superstructure while the part under grade level is called substructure. Normally the substructure part is modeled by a system of springs and dashpots in many directions. Those springs and dashpots could be linear or nonlinear, connect not connected. This system also may consider the single degree of freedom or multi-degree of freedom as shown in Figure 1.4 [8].

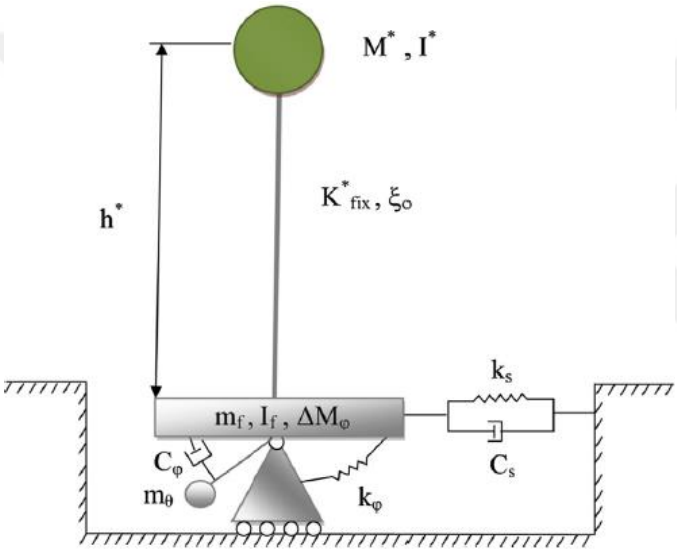


Figure 1.4 Single degree of freedom oscillator of soil and foundation system [9]

1.4.2 Direct approach

In this approach, the whole system of structure and soil are modeled in one step using finite element method as shown in Figure 1.5. Inertial interaction develops in structure due to own vibrations giving base shear and base moment, which in turn causes displacements of the foundation relative to the free field [10].

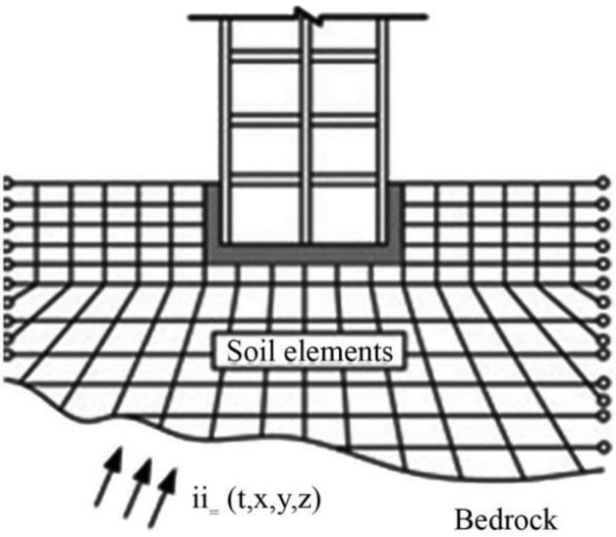


Figure 1.5 Direct approaches for the system [11]

1.5 Objective of Thesis

The objective of this thesis is to study is to study the effect of SSI on the seismic response of steel frames constructed on stiff soil and compare the result with a case of analysis without SSI using response spectrum analysis and time history analysis. Time history analysis will be done as linear and nonlinear analysis. The main parameters investigated in the present study are:

- Fundamental period of the structure
- Base shear of structure
- Size of the earthquake which will effect on the structure
- Inter story DRift (IDR)
- Maximum story displacements
- Story acceleration

1.6 Outline of the Thesis

The content and arrangement of the thesis are expressed as:

Chapter 1 contains background about steel frame structure and earthquake loads. Then a review about the main assumption of SSI and objective of the thesis are mentioned

Chapter 2 is the literature review about the analysis of steel frame under seismic loads with/without SSI effect, type of analysis and the origin of earthquake loads.

Chapter 3 deals with procedures for analysis and main assumptions for earthquakes and SSI.

Chapter 4 deals with analysis of steel frames. The discussion of the result is present. The results of rigid base and SSI are also compared.

Chapter 5 gives conclusion based on the present thesis together with further work that preferred to be done.

CHAPTER 2
SEISMIC, P-DELTA AND SOIL-STRUCTURE INTERACTION ANALYSES

2.1 Introduction

Earthquakes are caused by tectonic plates movements in the Earth's crust. When they stick, relative motion between the plates leads to increasing stress. This continues until the stress rises and breaks, suddenly allowing sliding over the locked portion of the fault, releasing the stored energy. During the earthquake and afterward, the tectonic plates start moving and they continue to move until they get stuck again. Energy spreads far from the source in all direction as seismic waves.

Figure 2.1 illustrate the main characters of the earthquake which are hypocenter, epicenter and focal point. Hypocenter is the point which energy release from, while epicenter is the point on the earth surface directly over hypocenter. When earthquake is scaled at a point far from the source, this point is called focal point **[**12**]**.

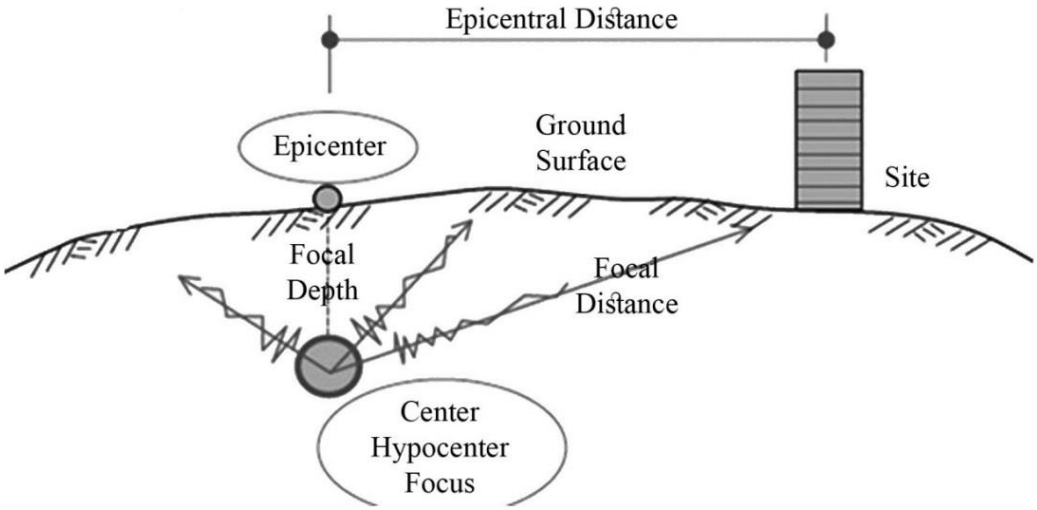


Figure 2.1 Earthquake characteristics [12]

Velocity of seismic waves depends on the specification of the medium and the most important properties of medium which effect on velocity of seismic waves are the following:

- The rigidity of soil (μ).
- The bulk modulus of elasticity for soil (E).
- The density of soil (ρ).

The modeling and analysis of seismic behavior is extremely complex because of the variation in the soil parameters. Many researchers made simplified assumption for modeling complex phenomena. In the present study, it is assumed that the medium is considered a homogenous, isotropic, elastic and infinite in size [12].

2.2 Seismic Waves

There are two types of seismic waves

- Body waves: these waves are the faster and travel through the soil layer.
- Surface waves: these waves are slower and travel along the surface of the earth [12].

2.2.1 Body waves

Body waves divided which are primary waves and secondary waves. Table 2.1 gives a specification for each type [12].

Table 2.1 Primary and secondary body waves

Primary waves (P-waves)	**Secondary waves (S-waves)**
It called also compressional waves (P-waves)	It called also shear waves (S-waves)
Apply longitudinal stress (pull-push) see Figure 2.2a	Apply lateral stress (side by side) see Figure 2.2b
Travel through gas, solid and liquid	Travel through solid
Faster and receive point distance from the epicenter	Slower

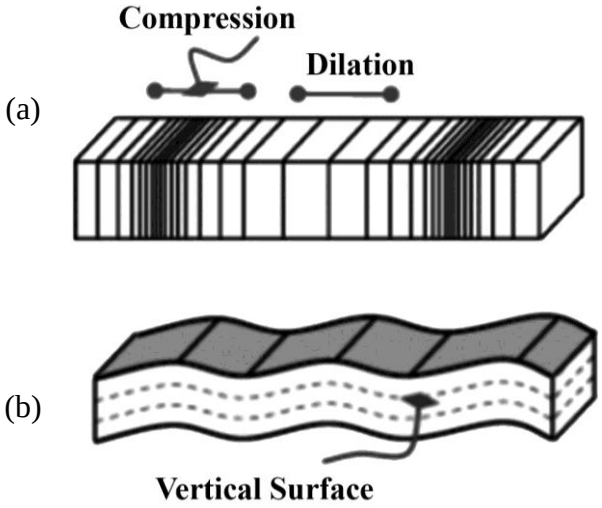


Figure 2.2 Body waves: (a) primary waves, (b) secondary waves [12].

2.2.2 Surface waves:

Surface waves are the slowest but they cause the most damage during an earthquake. Those waves divide into two types:

- **Love waves** as noticed in Figure 2.3a, love waves spread in the horizontal direction of motion. These waves occur when lower speed layer lies on a higher speed layer.
- **Rayleigh waves** are elliptical retrograde motion with respect to the direction of propagation as shown in Figure 2.3b. Their amplitude decreases with depth from the surface and its velocity is lesser than body waves [12].

2.3 Wave Velocity Integration

The velocity of body waves (P-waves and S-waves) are derived from analysis of elastic half space. The amplitude of body waves is proportional with distance (1/r), where r is the distance from rupture point to the defined point, while Rayleigh waves are proportional with distance $r^{0.5}$. Relationship of (P-waves) velocity is given by:

$$v_p = \sqrt{\frac{\lambda+2G}{\rho}} = \sqrt{\frac{(\lambda+2G)g}{\gamma}} \tag{2.1}$$

where γ, ρ and G are the unit weight, density and shear modulus of soil respectively. λ is a parameter and given by the relationship:

$$\lambda = \frac{\mu E}{(1+\mu)(1-2\mu)} \tag{2.2}$$

where μ is Passion ration of soil and E is the modulus of elasticity of soil. Relation among parameters E, G and ν is defined as:

$$G = \frac{E}{2(1+\nu)} \tag{2.3}$$

For shear waves (S-waves), shear velocity is given by

$$v_s = \sqrt{\frac{G}{\rho}} = \sqrt{\frac{Gg}{\gamma}} \tag{2.4}$$

where ρ, γ, G are defined above and g is the gravitational acceleration [12].

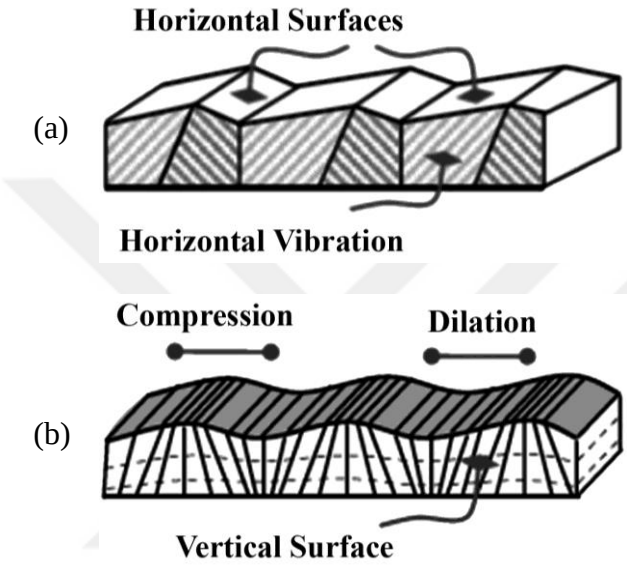


Figure 2.3 Surface waves: (a) love waves, (b) Rayleigh waves [12].

2.4 Measurement of Earthquake

Size and effect of the earth earthquake are considered the effective way to measure seismic action. Size of the earthquake is measured the amount of energy at the source, whereas effect measures the intensity at a specific location. The intensity means the effect of the earthquake at the earth's surface [13].

There are many scales to evaluate the earthquake, like the Modified Mercalli Intensity scale (MMI). MMI scale contains a series of certain key responses such as people awakening, damage to chimneys, movement of furniture, and finally total destruction. It is designed by Roman numerals, instead of scientific basis; it is based on the effects which observed from earthquake [13].

Seismic waves are records by instruments called seismograph and accelerograph. Many types of scales based on earthquake magnitude are developed like a Richter

scale which is introduced by Charles F. Richter 1935. The scale based on defining the distance from seismograph to the earthquake center and observing maximum signal amplitude records, then applying an empirical quantitative of the earthquake. The Richter magnitude is determined by the amplitude logarithm of the waves recorded by seismographs. In other words, Richer scale assigns number M to the maximum amount of ground motion in accordance with the energy amount releases from the earthquake [14]. The earthquake magnitude M is given by the following relationship:

$$M = \log A - 1.73 \log \frac{100}{D} \quad (2.5)$$

Where A is the maximum amplitude of ground motion which measured in micrometer and D is the distance from epicenter to location of measuring (measured in km).

If the earthquake magnitude is 2.00 or lesser, so people don't feel in it. It is generally recorded by local seismograph. The Richter scale is not commonly used anymore, except for small earthquakes recorded locally. For other earthquakes, a more accurate measure of the earthquake size is used like the moment magnitude scale [14].

Moment Magnitude Scale (MMS) which also called M_w developed in the 1970s. M_w is a sensitive scale and measures the accurate amount of energy sent out and depend on fault size and the dislocation. It is also awards an evaluation of the total power released by an earthquake. It can be used to grade earthquakes of all sizes that happen close by or far away [15].

2.5 Ground Acceleration

During the earthquake, an instrument called accelerograph is used to record acceleration versus time. By integrates acceleration records, velocity will be obtained, and by integrating velocity, displacement records will be obtained. Accelerograph is a low magnification seismograph which is designed to record acceleration of ground during seismic events. New accelerograph is used as an electronic transducer which produces an output voltage. This voltage is recorded and converted to acceleration versus time.

For analysis and design purpose engineers interest in peak ground acceleration which is maximum acceleration during the earthquake. Normally it is represented as a friction from gravity **[**16**]**.

2.5.1 Acceleration records

Accelerations are recorded in different horizontal directions and in the vertical direction to pick up maximum effect on the structure. In the present study, Düzce, Kocaeli and Erzincan earthquakes records are used. Record of Düzce earthquake which happened in 1999, Turkey is presented in Figure 2.4 as example of earthquake records. Figure 2.4a represent of acceleration record in horizontal East direction, while in Figure 2.4b represent the acceleration record in horizontal North direction and Figure 2.4c represent the acceleration record in the vertical direction [17].

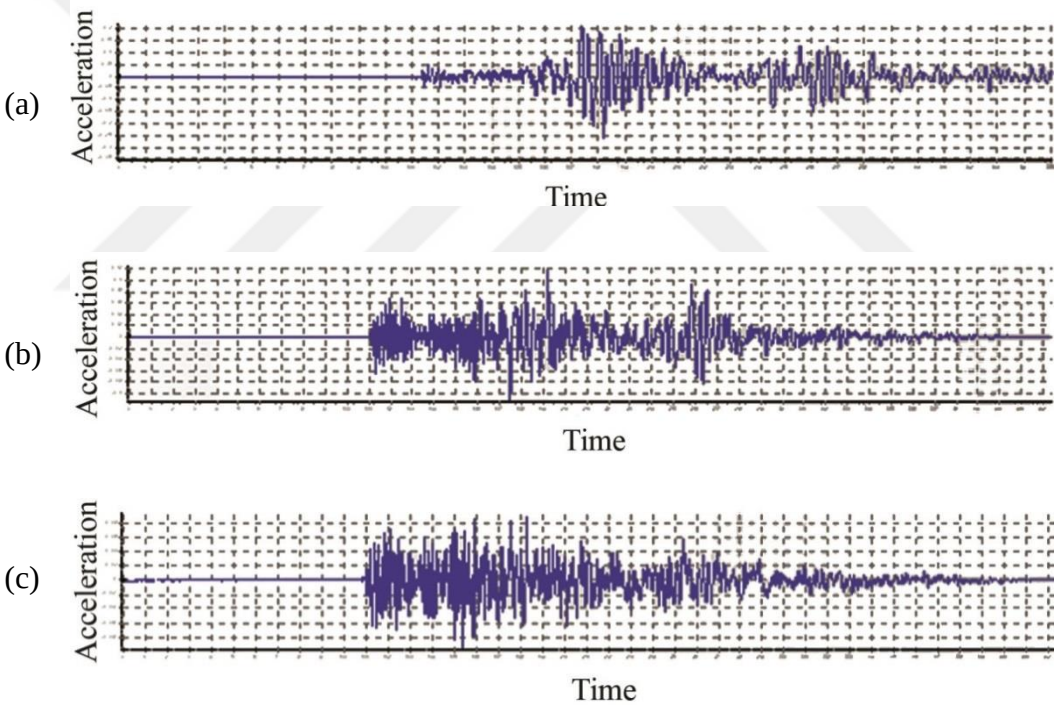


Figure 2.4 Düzce/Turkey 1999 earthquake acceleration versus time records (a) East direction (b) North direction (c) vertical direction **[**17**]**.

2.5.2 Velocity records

Velocity is obtained by integrating acceleration. Figure 2.5 presents the velocity records for East direction in Düzce/Turkey 1999 earthquake**.**

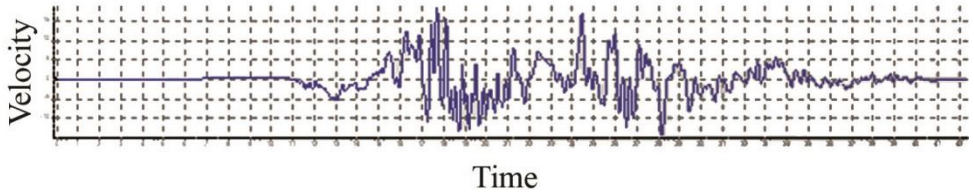


Figure 2.5 Velocity versus time records-East direction-Düzce/Turkey 1999 earthquake [17].

2.5.3 Displacement records

Displacement is obtained by integrating velocity. Figure 2.6 represents displacement records for East direction in Düzce/ Turkey 1999 earthquake**.**

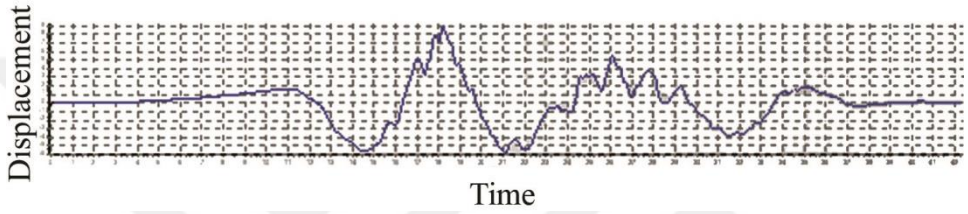


Figure 2.6 Displacement versus time records-East direction-Düzce/Turkey 1999 earthquake [17]

2.6 Response of Multi-Story Buildings

Strong earthquake causes a large destruction like Mexico-earthquake in 1985, Japan 1994, Philippines 1990 and Düzce/Turkey in 1999. Earthquake damage is divided into two types: structural and nonstructural damages:

- **Structural damage**: this type of damage is affected by many factors like shaking strength and duration, building type and soil beneath the structure, in other words by the subsurface condition. This damage happened to the structural element in building like columns, beams, slabs…. etc. Figure 2.7 is an example of structural damage of multi-story building in the Izmit earthquake of 1999 **[**18**]**.

Figure 2.7 Structural damage to a multi-story building, Izmit/Turkey 1999 [19**]**

- **Nonstructural damage**: this kind of damage happened to items which are not part of a structural system like glass panel, cladding, doors, false ceiling, pipeline…..etc. when shaking begin everything shakes and deforms, and at time nonstructural elements will become injured and fatal. Figure 2.8 shows typical damage at nonstructural elements [18].

Figure 2.8 Damage to the nonstructural element in multi-story building [18**]**

2.6.1 Engineering characterization of ground motion

Many characters represent the ground motion of earthquake, like PGA motion; also there is an impulsive character, duration of the earthquake, frequency content. The response of the structure under ground-motion most common modeled as the elastic response spectrum. The elastic response spectrum is the response of damped viscously elastic linear single degree of freedom under single earthquake. This

response is a function of oscillator vibration period single degree of freedom oscillator is represented in Figure 2.9

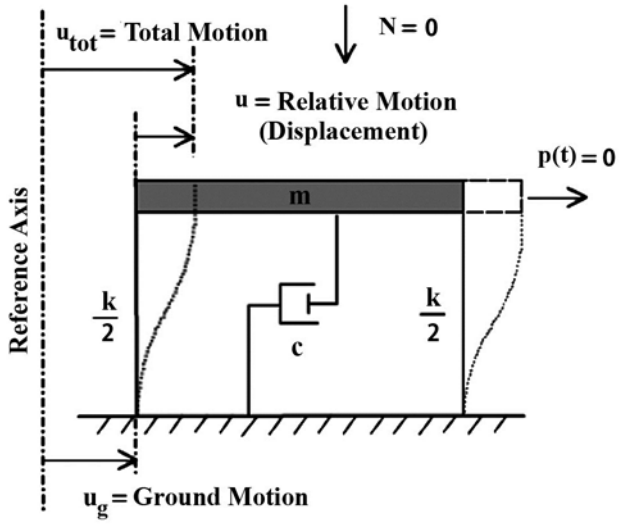


Figure 2.9 Single degree of freedom system oscillator [12].

The response history is calculated for single degree of freedom oscillator which having a specific period and damping, then the maximum responses, which are taken in absolute, are plotted as a function of vibration period. The mentioned process is repeated for all interest values of vibration period and damping to obtain elastic response spectrum. The response will be plotted against the vibration period. The response value can be one of the following:

- The maximum displacement relative to the ground of the oscillator --- S_d.
- The maximum velocity relative to the ground of the oscillator --- SV.
- The maximum acceleration of the oscillator which is absolute (total value) --- SA.
- Pseudo velocity which also called PSV --- S_v.
- Pseudo acceleration which also called PSA --- S_a.

S_v and S_a are given by the following formula and they are used more than SV and SA

$$S_v = \omega S_d \tag{2.6}$$

$$S_a = \omega^2 S_d \tag{2.7}$$

Where ω is the circular frequency of single degree of freedom model which related with period of structure (T) by the following

$$T = \frac{2\pi}{\omega} \tag{2.8}$$

T also given by the relationship

$$T = 2\pi\sqrt{\frac{M}{K}} \tag{2.9}$$

M, K are mass and stiffness of single degree of freedom model respectively.

The earthquake input forces to building is called restoring forces and given by equation (2.10). The maximum restoring force is the base shear force which is used in structure design.

$$V_e = K\, S_d = M\, \omega^2\, S_d = M\, S_a \tag{2.10}$$

For example, Figure 2.10 contains a response spectrum for Kocaeli/Turkey earthquake, this response spectrum is the pseudo acceleration versus period with damping value of 5%. From this spectrum, peak values of response can be determined. For example at $T = 1.3$ second, and $S_a = 0.66g$ So peak base shear [20].

$$V_e = \frac{S_a\, W}{g} \tag{2.11}$$

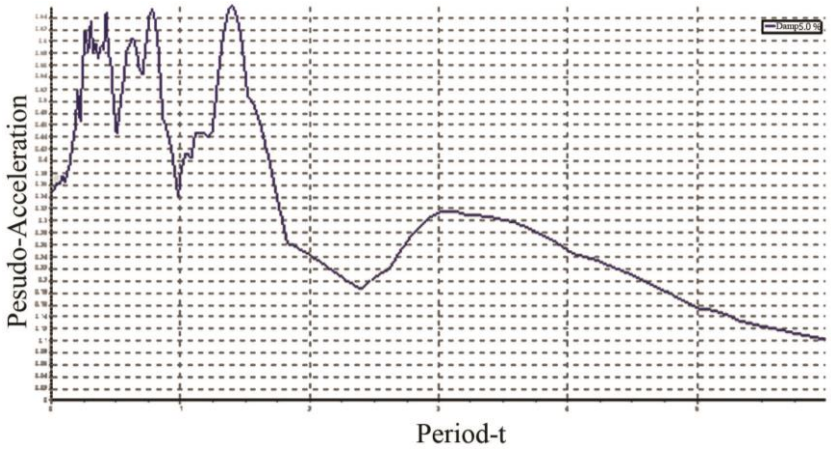


Figure 2.10 Pseudo acceleration records versus period for Kocaeli/Turkey earthquake [21].

2.6.2 Site effects on structural response to earthquake loads.

Sometimes during an earthquake, motions are amplified and stronger seismic loads impact the structure, this happens when the structure is built on soft soil. Codes give

this point very high attention, and all codes contain classification of soil with parameters make the response of structure to seismic loads increase when soil is going to be weaker and highest response is records for structure in soft soil. The classification of ground types is given in Euro code 8 [5]. The ground types are: A- rocks, B- gravel or deposit of very dense sand, C- stiff clay with thickness from several tens to many hundreds of. meters, D - deposit of loose to medium cohesionless soil and E- soil profile consisting of a surface alluvium layer with values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_s 800 m/s

2.6.3 Occupation of structure

Buildings which have vital importance for civil protection like hospital shall not destroy or have big damage during the earthquake, for that, codes amplify design base shear by a parameter called importance factors which classify building into many types and each type has factors to increase designed shear forces.

In Euro code 8 Buildings are classified in four classes, based on the effect of collapse for human being, importance for public safety and urban protection in the period after earthquake, and on the economic and social result of collapse.

First class is the buildings of secondary importance for nation safety like agricultural buildings, second class is ordinary buildings which doesn't not belonging to the other group, third class is the buildings whose seismic durability is of importance in view of the result related to the collapse like schools, fourth class is the buildings whose safety at earthquake duration is having vital value for civil security [5].

2.6.4 Behavior of structure under the seismic hazard

Design of structure under seismic loads with considering the elastic response of structure makes the cost of construction extremely high. While considering inelastic dynamic response in design make the cost much lesser because the design strengthens is much lesser than elastic demand (see Figure 2.11). But inelastic dynamic analysis is complex, needs time and experience, so some simplifications are take into account to simplify procedures. Force reduction factor and seismic properties of structure like nature period are considered a simplification for inelastic dynamic analysis [12**]**.

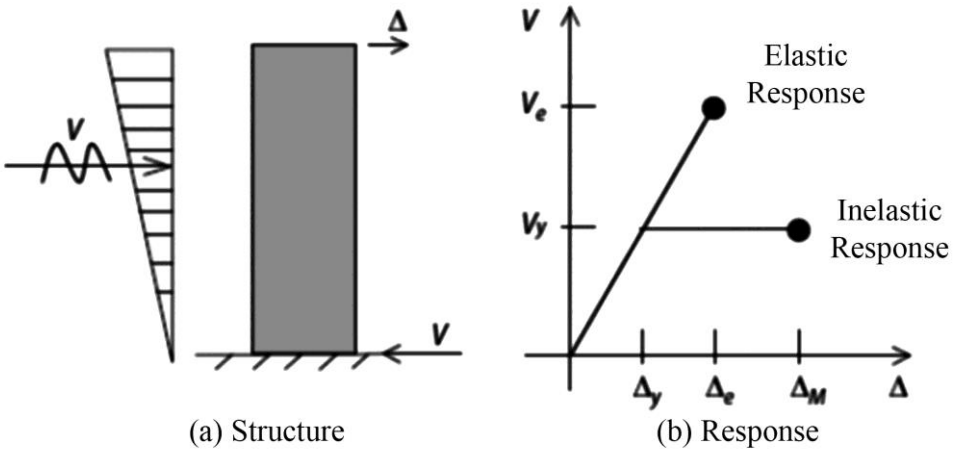


Figure 2.11 Elastic and inelastic response of structure [12**]**

2.6.4.1 Force reduction factor R

As shown in Figure 2.12, force reduction factor is the ratio between elastic strength demand and the real level of yielding of the structure. It is given by the equation:

$$R = \frac{F_e}{F_y} \tag{2.12}$$

Force reduction factor in formulation (2.12) is expressed by a term of forces, but it is also can be expressed by displacement as:

$$R = \frac{\Delta_e}{\Delta_y} \tag{2.13}$$

where F_e and Δ_e is the maximum force and maximum displacement during the entire excitation of the earthquake based on the linear elastic response of the structure. F_y and Δ_y is the force and the displacement at first significant yielding point of structure [12**].**

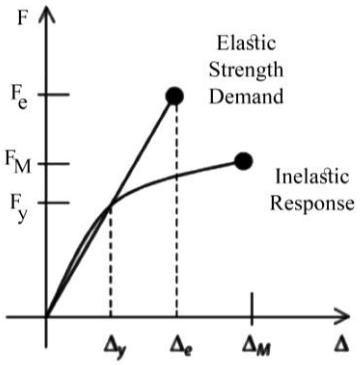
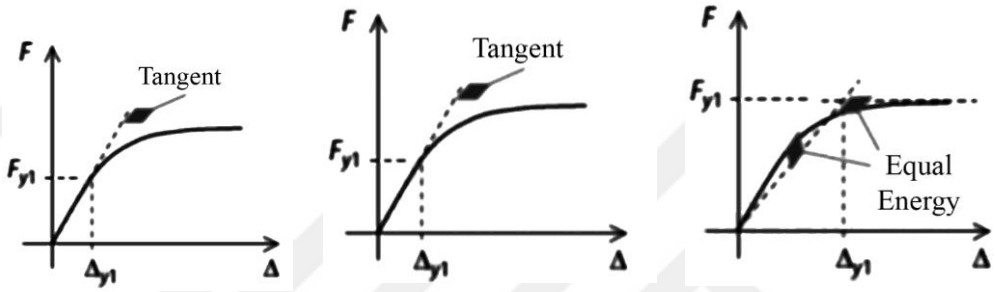


Figure 2.12 Force-reduction factor (elastic and inelastic response) [12**].**

As shown in Figure 2.13, the first significant yielding point of the structure has more than one definition such as first yielding point (see Figure 2.13a), initial tangent and ultimate load (see Figure 2.13b), and equal energy absorption (see Figure 2.13c). So it is not necessary to be the first plastic hinge in the structure.

Value of force reduction factor depends on the ability of the structure to dissipate energy by inelastic deformation and hysteric behavior so the structure dissipates energy without significant reduction in strengthens, and this called ductility of the structure**.**



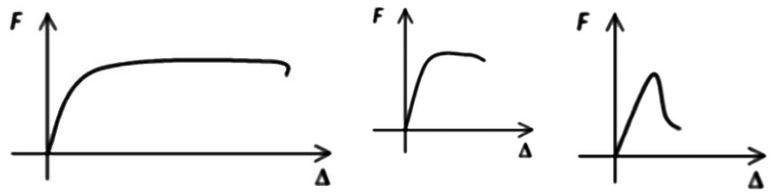
(a) (b) (c)

Figure 2.13 First significant yielding point of the structure, (a) first yielding point, (b) initial tangent and ultimate load and (c) equal energy absorption [12]

In Euro code force reduction factor is called behavior factors. For horizontal actions the evaluated value of behavior factor dependent on the type of the structural system and regulation in elevation. Structural buildings shall be classified into one of the following structural types: frame system, dual system (frame or wall equivalent), ductile wall system (coupled or uncoupled), system of large lightly reinforced walls, and inverted pendulum system, torsional flexible system [5].

2.6.4.2 Ductility of the structure

Ductility of the structure is its capacity to bear large displacement without any significant loss in strengthens. There are many levels of ductility. The level of ductility may be defined as high ductility, low ductility and brittle as shown in Figure 2.14.



(a) (b) (c)

Figure 2.14 Ductility levels; (a) high ductility, (b) low ductility and (c) brittle [12**]**

2.6.5 Seismic hazard

Seismic hazard usually represents by a map for the region. Each country has a seismic map which refers to high and low seismic activity. This map is considered the main point to choose structural seismic resisting system. Figure 2.15 presents the map of seismic hazard of turkey which published in March 2018 by department of earthquake directorate of AFAD. The red areas have the maximum seismic risk while the yellow one has the lowest seismic risk [22].

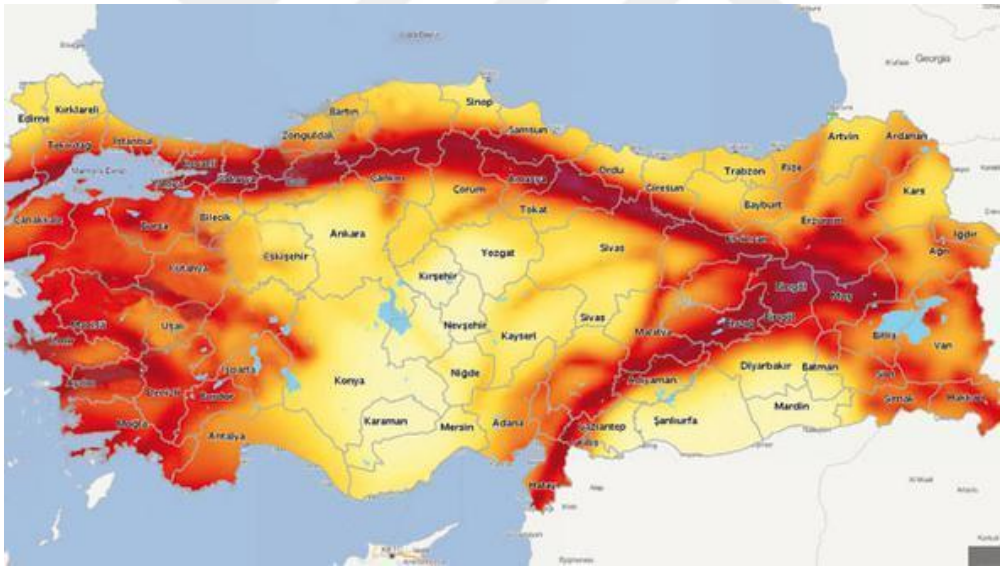


Figure 2.15 Seismic hazard of Turkey [22]

In Turkish code 2007 Seismic zones for Turkey are four, the first, second, third and fourth seismic zones. The maximum seismic risk is at first seismic zone, and the lower seismic risk is at fourth seismic zone [23].

Codes usually raise design seismic loads by factors in high seismic regions. Euro code 8 defines two seismic hazards zone according to the amplitude of design earthquake. So if the amplitude of design earthquake is bigger than 6.1 so we use

response spectrum 1, while if smaller we use response spectrum 2. Also, the vertical design seismic loads will be amplified [5].

2.6.6 Evaluation of earthquake loads on structures

Seismic loads are inertial dynamic loads created by ground acceleration resulted from the earthquake. There are many factors effect on the magnitude of these loads, differ from structure to the site condition and earthquake parameter. Those factors can list as following:

- Structure properties like natural period, mass, and stiffness.
- Site condition: for example, the soft soil is propagating seismic waves and makes loads higher.
- Earthquake parameters, such as density and duration, distance from fault and optional energy.
- Soil structure interaction

Seismic loads, in general, need a complex dynamic analysis to evaluate forces on the structure. Codes present various complicated methods to evaluated seismic actions on the structure using numerical analysis for a more accurate solution. The list below contains methods for define seismic loads on structures [5].

- Equivalent lateral force method which is static linear approach.
- Model response spectrum method which is a linear dynamic method.
- Pushover analysis which is a static nonlinear method.
- Time history analysis: which is linear or nonlinear analysis, in this method, real records of earthquakes are implemented on the structure and scaling factors are applied on records to fit the current situation of the case study.

2.6.6.1 Equivalent lateral force method

The equivalent static method is the simplest one. Considering that lateral forces will be implemented at bottom of the structure, and each floor mass is concentrated at floor level, and then base shear will be distributed on the structure at floor level (see Figure 2.16) [24].

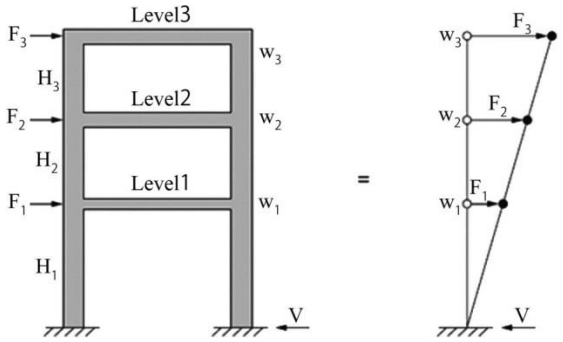


Figure 2.16 Seismic base shear conceptions according to lateral force [24].

Many factors effect on the values of lateral force were determined by codes. These factors in general are:

- Mass of building.
- A structural system which resists seismic loads.
- Ductility of the structural system.
- Ground acceleration and seismic hazard of building location.
- Occupation of the structure.

This method has limitation for implementing due to some structural specification. Main two points of limitation are

- Building height shall not exceed 60meter.
- Regularity in structure: Mass and stiffness should be regular in horizontal plan and vertical elevation [5].

2.6.6.2 Model response spectrum method (linear dynamic analysis)

Response spectrum method is one of the most powerful methods to evaluate the response of the structure under seismic loads. It gives maximum response for a linear system of single degree of freedom model as a function of the period of vibration with specific damping ratio under one earthquake.

The response spectrum of Düzce earthquake is evaluated by using SeismoSignal software. Figure 2.17 shows the response of single degree of freedom model with three different damping ratio of 5%, 10%, 15% and the period from 0 to 4 second under Düzce earthquake in Turkey 1999 [21].

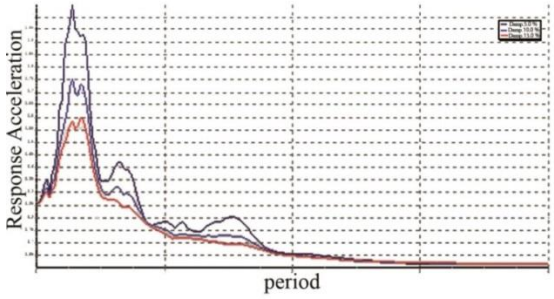


Figure 2.17 Response spectrum of Düzce Turkey 1999 earthquake with different damping ratio [21].

When consider structure consists of multi-degree of freedom so the response will be summing of model vibration of the structure. The number of this mode equal number of freedom in structure, the response of these modes may not happen at the same time (see Figure 2.18).

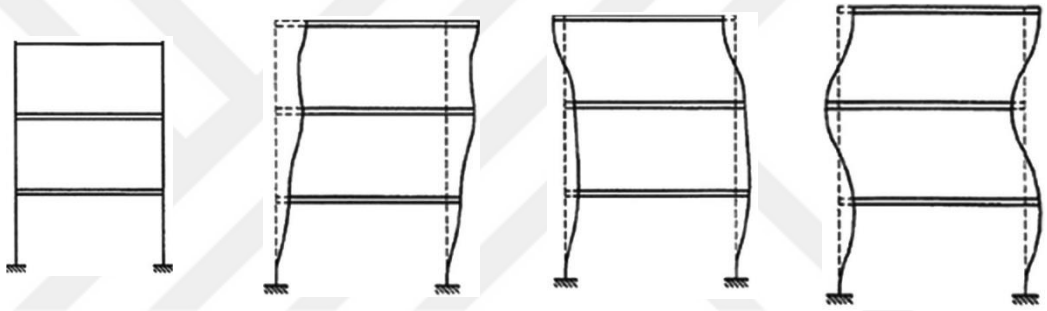


Figure 2.18 Modes of the 2D frame with three degrees of freedom [25]

For design purpose codes developed smooth response spectrum which takes into consideration many earthquakes at specific damping ratio and specified periods. Figure 2.19 is the general shape of smooth response spectrum in code which horizontal axis is period (second) and the vertical axis is spectral acceleration.

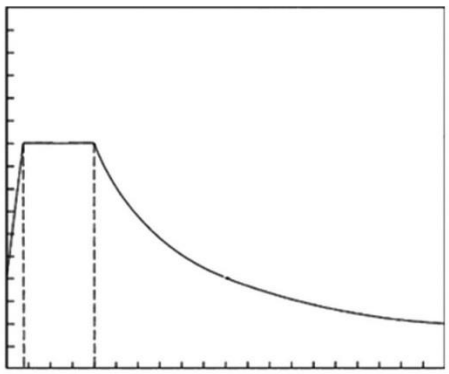


Figure 2.19 General shape of smooth response spectrum [5]

2.6.6.3 Dynamic time history analysis

Linear and nonlinear time history analysis recently are becoming one of the most popular methods in analysis and design for seismic loads. Earthquake time history records contain wealth information about characteristics of the earthquake, like amplitude, duration, energy content, and frequency. By applying these records on structures, all the factors which influence the accelerograms will be taken into consideration like the path of shake, site to source distance, rupture mechanism and source characteristics. Analysis results depend on two points, first is the accuracy of structural model, the second one is proper selection of ground motion which take into account earthquake parameter and site condition like soil type. Records should be scaled to match target response spectrum [5].

Scaling is done by one of the following methods; scaling in the time domain and scaling in the frequency domain. In time domain method, scaling will be done for amplitude while no changing in frequency content will be done, so the difference between target spectrum and scaled ground motion records response spectrums will be minimized within a range of periods. In Euro code 8, the range of period is defined between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of structural in designed direction where the ground motion will be applied [26].

Changing in frequency domain method will be done for frequency content and time. This scaling for wavelets is done to give a harmony between the spectral acceleration of ground motion and target design spectrum. Scaling can be done for a specific natural period and in this case the method is called scaling to single period, the second method is called minimized MSE. Some software is developed to scale records to produce designed response spectrum [26].

In general, time-domain method applied on records in main two horizontal directions and vertical one to get the maximum response of the structure.

2.7 Second Order Effect P-Δ.

In first order effect, the moment is resulted by multiplying lateral force by story height, while second-order effect moment is a gotten from vertical load multiplying by elastic deformation of the story with respect to lower one. Second order effect has two types [27]

- **P-Δ *effect***: this effect due to change in geometry of the structure (see Figure 2.20), this type of analysis will be done in the calculation in this search.

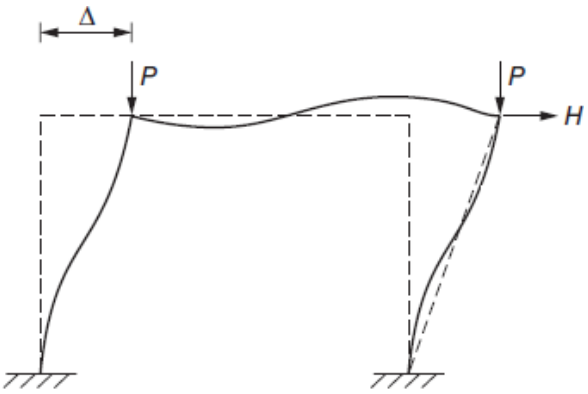


Figure 2.20 P-Δ effect [27]

- **P-δ *effect*:** this effect due to curvature in members and a change in their stiffness under applied loads (see Figure 2.21).

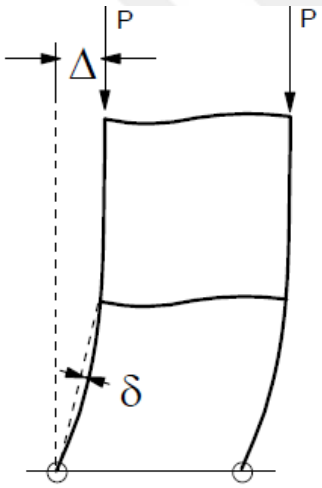


Figure 2.21 P-δ effect [28]

When buildings are designed under seismic loads, two main designed parameters are take into concideration. First of all is base shear of building, second one is the inelastic dispalcement. Inselatic displacment are controlled by interstory drift to be lesser than allowed values presented in codes. Interstory drift is presented in Figure 2.22

P-Delta analysis is nonlinear analysis and normally done in the seismic analysis of structure. Codes define criteria to take this effect in analysis or not. Lateral forces from seismic cause lateral movement in structure, this movement is added to P-Δ and

compared to allowance story drift, and also this value is used to define displacement between buildings [27].

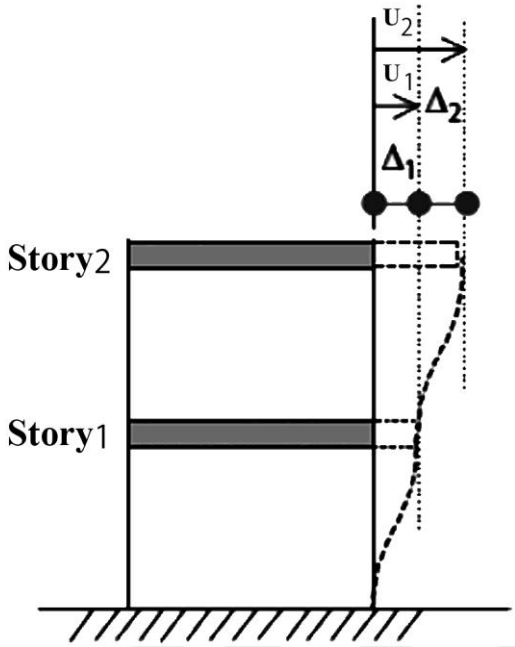


Figure 2.22 Inter story drift [20]

2.8 Soil-Structure Interaction

2.8.1 Analysis approach

In a seismic analysis of structure, there are three different point effects on the response of the structure, first one is the structure, the second one is the foundation of the system and the third one is soil beneath the structure. Normally, in calculation buildings are considered to have rigid base, as shown in Figure 2.23

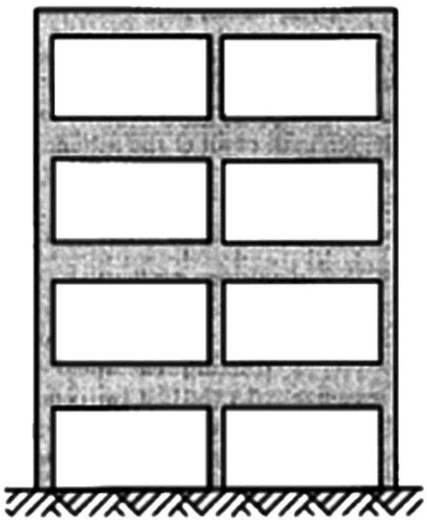


Figure 2.23 Structure with a fixed base [29]

But in the last decades, it has been recognized that SSI effect on the response characteristics of a structural system because of the massive and stiff nature of structure and, soil softness. SSI effect is very complex analysis because of nonlinearity which takes in the calculation; this nonlinearity is categorized in the following fields

- Yielding to lateral resisting system structure
- Yielding in soil
- Yielding in foundation
- The gap between foundation and soil

The numerical methods to evaluated SSI are categorized as direct analysis approach and substructure approach.

2.8.1.1 Direct analysis approach

In this approach the whole system (structure-foundation-soil) is modeled in one step. This type of analysis is very difficult to done by manual calculations and advanced numerical methods like finite element method is used (see Figure 2.24) [10].

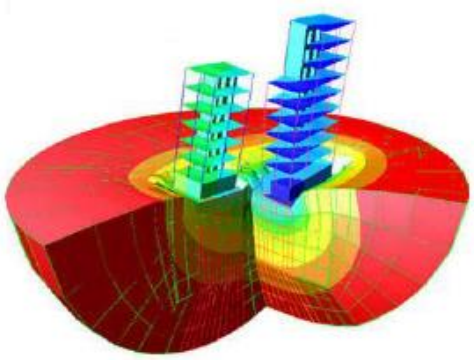


Figure 2.24 Direct approaches (structure, foundation, and soil) [10]

2.8.1.2 Substructure approach

The principle of substructure method has broken the analysis into several steps. This is to enable the foundation to match the free field deformation and the effect of the dynamic response of the structural foundation system on the movement of supporting soil. The method considers building is laying on springs and dashpots. Those springs and dashpots are representing soil around foundation or foundation and soil together; Figure 2.25 is an example of modeling soil as spring and dashpot. Values of springs and dashpot coefficient are considered the main challenge for the engineers [6].

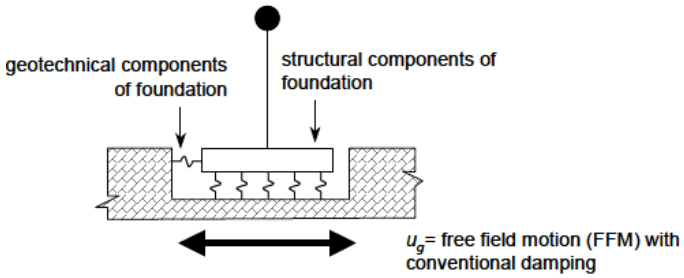


Figure 2.25 Substructure modeling approach (structure, foundation, and spring) [6]

2.8.2 Types of soil structure interaction

2.8.2.1 Inertial interaction

After seismic waves hit the structure, it starts to vibrate and shear forces and moment appear. This shear forces and moment at foundation level transmit from foundation to surrounding soil through surface between foundation and soil in a shape of rotation and displacement and the foundation and soil surround are. This rotation and vibration lead to two main effects:

- The vibration of the structure-foundation-soil system is relative to vibration structure fixed base.
- Damping of the system also is increased due to the hysteretic behavior of soil and radiation damping.

For calculation of structure, these two effects are done by one of the following methods:

- Approximate calculation: it was noticed that inertial interaction can be evaluated by modifying fundamental period and damping ratio of the fixed base system, for example in FEMA 440 [6], the new period and damping ratio of the system are given in the following equation:

$$T_{ssi} = T\sqrt{1 + \frac{k}{k_x}\left(1 + \frac{k_x h^2}{k_\theta}\right)} \tag{2.14}$$

$$k = \frac{4\pi^2}{T}\ m^* \tag{2.15}$$

$$\zeta_0 = \zeta_f + \frac{5\%}{\left({}^{T_{ssi}}/_{T}\right)^3} \tag{2.16}$$

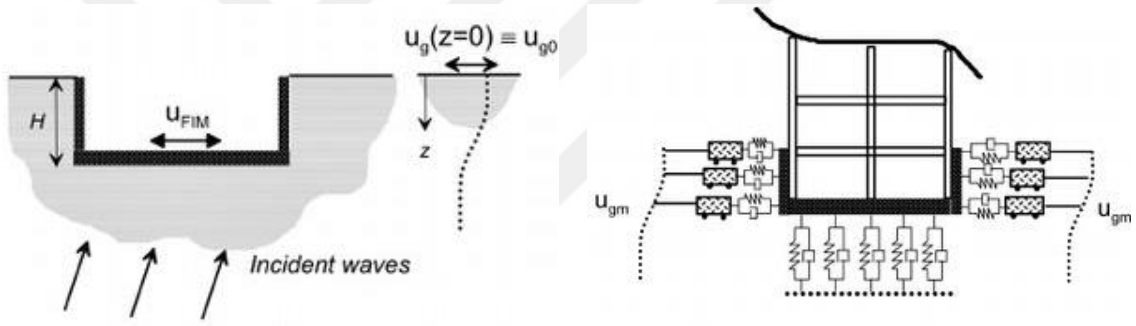
Where T is a fundamental period of the structure, m^* is an effective modal mass of the fundamental vibration mode of the structure, h is effective height of the structure,

k_x, k_θ are effective swaying and rocking stiffness of the foundation in the same order and ζ_f is damping of foundation and soil (radiation and material damping) [6].

2.8.2.2 Kinematic interaction

It is also called embedded effect. Normally buildings have some floors below grade level so the motion at sub-surface is lowered than the motion at the free surface as shown in Figure 2.26. This is because of base-slab averaging primarily affects. Base slab averaging primarily effect is occurs because one ground motion passes through the rigid foundation so the peak ground acceleration is going to be lesser.

Calculation of kinematic interaction develops in direct analysis approach due to the presence of stiff foundation elements on or in the soil causing foundation motion to deviate from free-field motions [30].



(a) (b)

Figure 2.26 Illustration of kinematic: (a) kinematic interaction components of foundation–soil interaction for an embedded foundation; (b) application of free-field motion, modified for base slab averaging effects u_{gm}, for inertial interaction [30]

CHAPTER 3

STRUCTURAL SIESMIC ANALYSIS

3.1 Introduction

In this research, 2D special moment resisting frames have been modeled in two cases. First one, by consider the building is fixed at the bottom, and the second one SSI by considering a flexible base which modeled by linear springs and dashpot. Each spring and dashpot have three component of stiffness; those springs and dashpot represent soil and foundation system. The foundation system for all frames is considered single footing.

Frames are analyzed according to Eurcode3 [32] and Euro code 8 [5]. Dead loads, live loads and earthquake loads are considered. Seismic loads are calculated as dynamic loads. Dynamic seismic analysis has done as linear dynamic spectral analysis and linear/ nonlinear time history analysis. P-Delta effect is also taken in to account for nonlinear analysis. The general procedures of present research are shown in Figure 3.1

3.2 Frames Geometry

Special Moment Resisting Frames (SMRF) which designed before by Karavasilis et al [16] is considered in present study. Design is done according to Euro code 3 and Euro code 8. 12 frames are analyzed with 3bays and 6 bays. The width of each bay is 5 m. 3, 6, 9, 12, 15 and 20 stories are analyzed. The height of each story is 3 meter. The yielding stress of the material is 235MPa (see Figure 3.2). Connections between columns and beams are considered rigid. P-Delta effect which is second-order effects is also considered for nonlinear analysis. Sections for each frames structure are listed in Table 3.1.

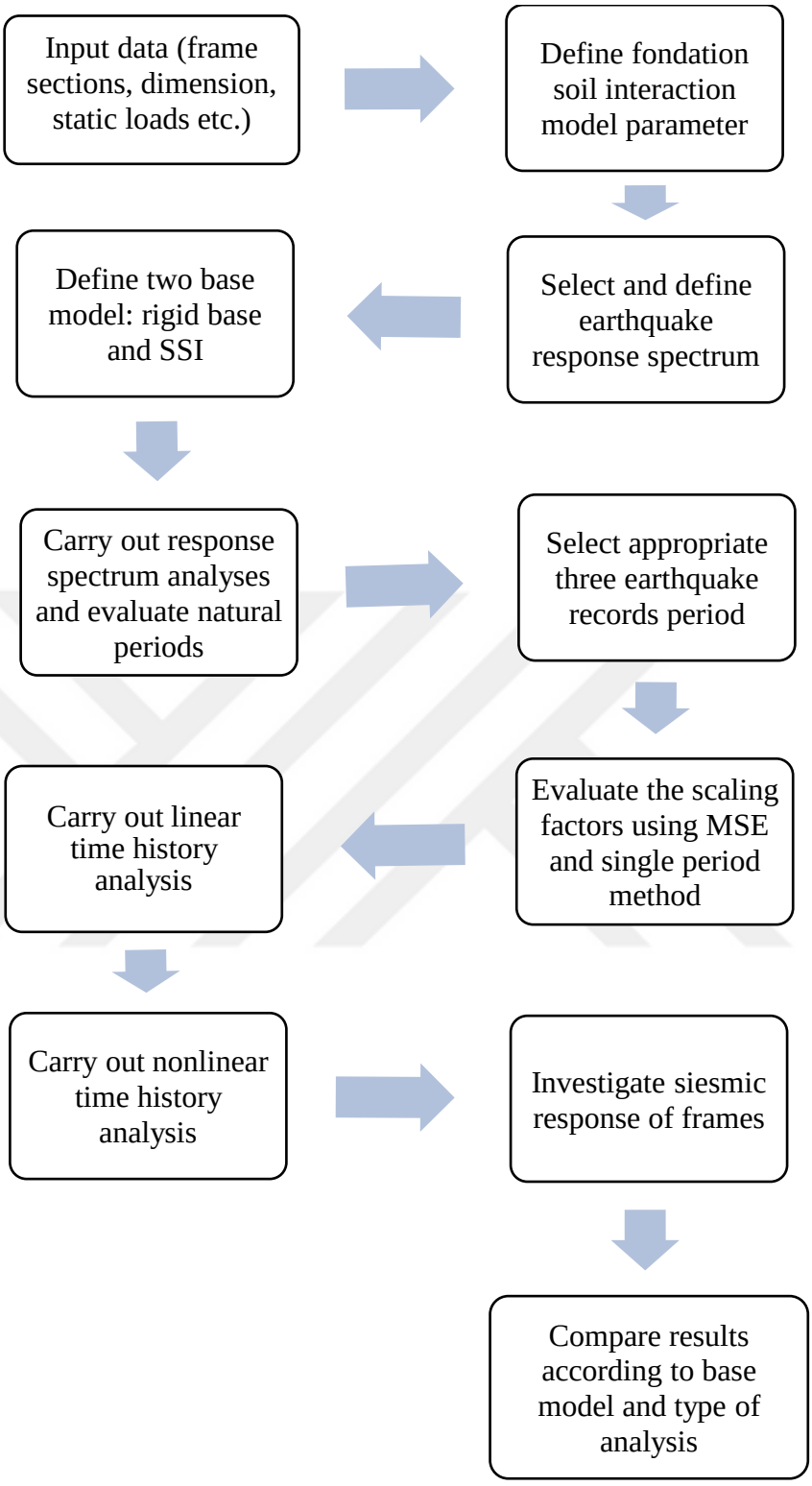


Figure 3.1 Steps of a research methodology

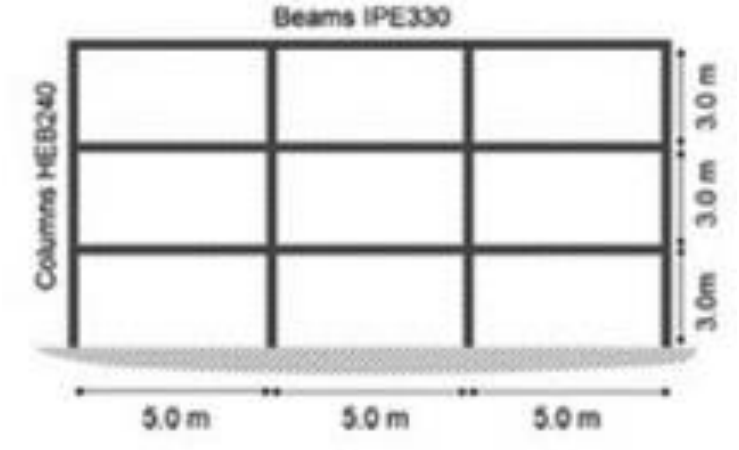


Figure 3.2 Typical moment resisting frames with three bays and three stories [32]

Table 3.1 Designed section for moment resisting frames considered in the study [32]

No.	NS[1]	NB[2]	Sections (Columns HEB[3]-Beams IPE[3])
1	3	3	240–330(1–3)
2	3	6	240–330(1–3)
3	6	3	280–360(1–4), 260–330(5–6)
4	6	6	280–360(1–4), 260–330(5–6)
5	9	3	340–360(1), 340–400(2–5), 320–360(6–7), 300–330(8–9)
6	9	6	340–360(1), 340–400(2–5), 320–360(6–7), 300–330(8–9)
7	12	3	400–360(1), 400–400(2–3), 400–450(4–5), 360–400(6–7), 340–400(8–9), 340–360(10), 340–330(11–12)
8	12	6	400–360(1), 400–400(2–3), 400–450(4–5), 360–400(6–7), 340–400(8–9), 340–360(10), 340–330(11–12)
9	15	3	500–300(1), 500–400(2–3), 500–450(4–5), 450–400(6–7), 400–400(8–12), 400–360–(13–14), 400–330(15)
10	15	6	500–300(1), 500–400(2–3), 500–450(4–5), 450–400(6–7), 400–400(8–12), 400–360–(13–14), 400–330(15)
11	20	3	600–300(1), 600–400(2–3), 600–450(4–5), 550–450(6–10), 500–450(11–13), 500–400(14–16), 450–400(17), 450–360(18–19), 450–330(20)
12	20	6	600–300(1), 600–400(2–3), 600–450(4–5), 550–450(6–10), 500–450(11–13), 500–400(14–16), 450–400(17), 450–360(18–19), 450–330(20)

[1] NS is a number of stories

[2] NB is the number of bays

[3] HEB and IPE are the profile names used for columns and beams respectively.

In the table, the expression of the form (260-360)(1-4) means that the first four (1–4) stories have columns with HEB260 sections and beams with IPE360 sections.

3.3 Specified Load on Frames

Designed loads of frames which are used in the analysis are dead, live and seismic loads. Dead and live loads are considered 27.5 kN/m [35].

3.3.1 Seismic loads

Seismic loads are calculated and analyzed according to Euro code 8. The dead loads are concidered to calculated effective weight of structur. Effective weight of structure is used to calcuated seismic loads on structure. The seismic motion is considered strong with PGA=0.35g. The analysis is done for two types of soil; type B and type C according to Euro code 8. Three methods are used to analyze the seismic response of structure in two cases of bases which are rigid bases and SSI.

- Model response spectrum analysis
- Linear time history analysis of seismic hazard
- Nonlinear time history analysis of seismic hazard

3.3.1.1 Model response spectrum analysis

The analyses are carried out as linear dynamic analysis using two response spectrums which are designed according to Euro code 8 depending on the following parameter:

- Designed PGA=0.35
- Importance class of buildings is II
- Elastic ground acceleration response spectra are type 1
- Damping coefficient 0.03

The soil type for first response spectrum is class B soil. The values of the first response spectrum curve as shown in Figure 3.3 are presented in Table 3.2. The second response spectrum has the same specification, except the soil types which is class C. The values of the second response spectrum curve as shown in Figure 3.3 are presented in Table 3.2.

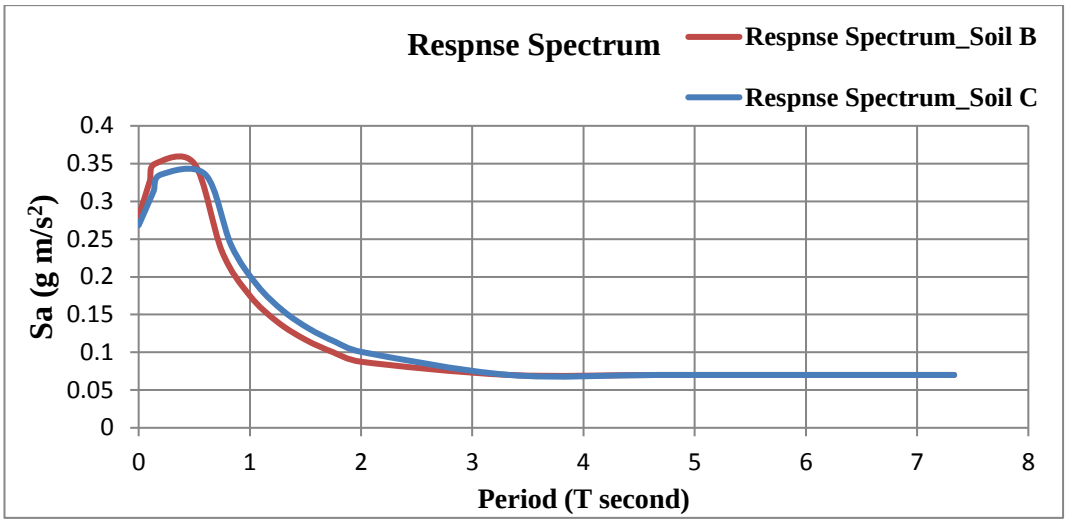


Figure 3.3 The response spectrums based on soil type B and soil type C

Table 3.2 The response spectrum values based on soil type B and soil type C

First response spectrum --- soil type B		Second response spectrum --- soil type C	
Period (T second)	Sa (g m/s^2)	Period (T second)	Sa (g m/s^2)
0.000	0.280	0.000	0.268
0.050	0.303	0.067	0.291
0.100	0.327	0.133	0.313
0.150	0.350	0.200	0.335
0.500	0.350	0.600	0.335
0.750	0.233	0.833	0.242
1.000	0.175	1.067	0.189
1.250	0.140	1.300	0.155
1.500	0.117	1.533	0.131
1.750	0.100	1.767	0.114
2.000	0.087	2.000	0.101
3.333	0.070	3.333	0.070
4.667	0.070	4.667	0.070
6.000	0.070	6.000	0.070
7.333	0.070	7.333	0.070
8.667	0.070	8.667	0.070
10.000	0.070	10.000	0.070

3.3.1.2 Time history analysis of seismic hazard

The linear and nonlinear time history analyses have been done. Real records of acceleration are used. In linear time history analysis, six records of ground acceleration are used which are two records in horizontal directions for each selected earthquake. Vertical acceleration records are not used according to FEMA 440 [6] because the used frames are in 2D. In nonlinear analysis a record with max response in the linear analysis is selected. Components of seismic records are considered. These are two horizontal and one vertical record. In present study, only horizontal directions are used in analysis so two horizontal records of each earthquake are applied on each frame and then the direction of max response is taken for nonlinear analysis. P-Delta effect is considered in nonlinear time history analysis

According to Euro code 8, three earthquake records are applied on the structure and the highest response is used for design purpose, or seven records are applied, an average response is used. In present study, three (Düzce 1999, Kocaeli 1999 and Erzincan1992) real seismic records are applied for linear time history analysis and the maximum response is considered for nonlinear analysis.

Records are obtained from PEER Ground Motion Database [17]. Seismic hazard is selected to be strong motion and magnitude between 6.5 and 7.5. Three records of seismic hazards used in this research are listed in Table 3.3.

Table 3.3 The seismic hazards used in this research [17].

Earthquakes name	RSN[1]	station	year	Magnitude (Mw)	Rrup[2] (km)	V_{s30}[3] (m/s)	PGA (m/s^2)
Kocaeli	1165	Izmit	1999	7.51	7.21	811	0.194
Düzce	1615	Lamont	1999	7.14	9.14	338	0.207
Erzincan	821	Erzincan	1992	6.69	4.38	352.05	0.445

[1] RSN is Record Sequence Number
[2] Rrup is the closet distance from rupture to station
[3] Vs30 is average value of propagation velocity of S waves in the upper 30m of the soil profile at shear strain of 10-5 or less (Euro code8)

In developing the match of the spectrum, a set of time histories are considered. The first step is to scale each record to the level of the design spectrum. There are two methods of scaling; the first scaling is based on a single period and the second method is the Mean Squared Error (MSE) of the differences which is done for a range of period. In MSE method, quantitative measure of the overall fit of the

spectrum of a time history record to a design or target spectrum is MSE of the differences (summed over a discrete set of periods) between the spectral accelerations of the record and the design spectrum, computed using logarithms of spectral acceleration and period. Thus a scaling factor can be determined for each record that minimizes the MSE over a user-defined period range of significance. Records in nonlinear time history analysis are scaled according to MSE methods [26].

In present study, scaling based on a single period method is applied for linear time history analysis. Scaling is done for each frames modeled as rigid base and SSI. These scaled records are used in linear/nonlinear time history analysis.

Figure 3.4 - 3.9 show the records of selected earthquakes which used in present study. These records are in horizontal direction. Each frame, two horizontal records of the earthquake are applied to get the maximum response.

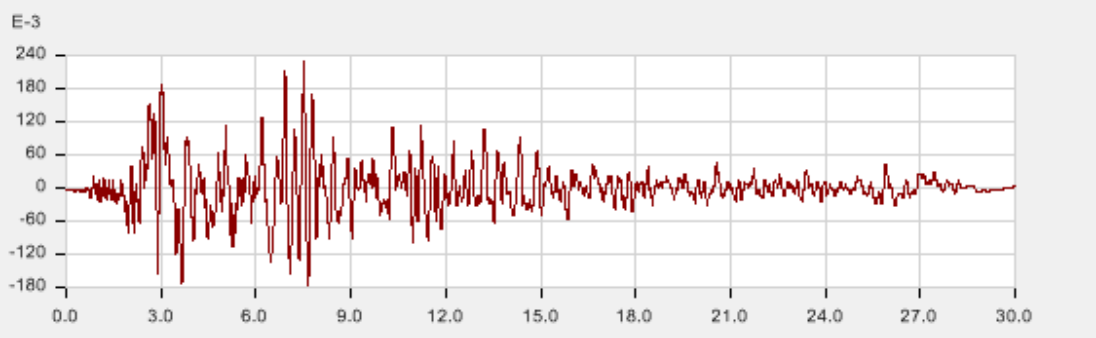


Figure 3.4 Kocaeil earthquake acceleration records versus time in horizontal direction 90 at RSN1165

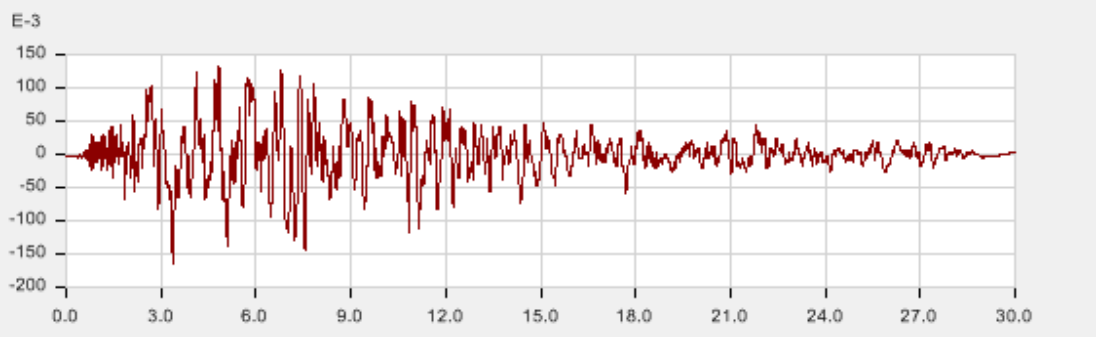


Figure 3.5 Kocaeil earthquake acceleration records versus time in horizontal direction180 at RSN1165

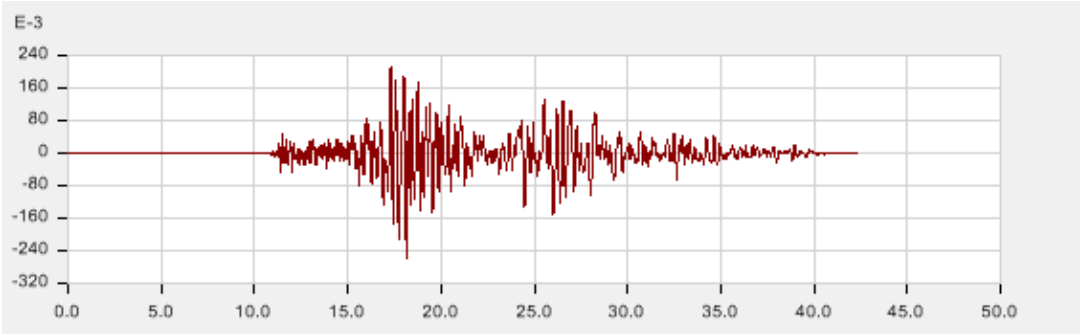


Figure 3.6 Düzce earthquake acceleration records versus time in an East direction at RSN161

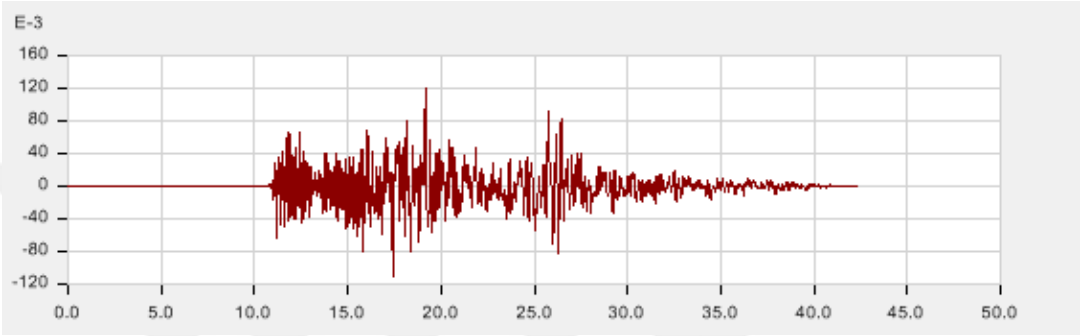


Figure 3.7 Düzce earthquake acceleration records versus time in a North direction at RSN1615

Figure 3.8 Erzincan earthquake acceleration records versus time in East-West direction at RSN821

Figure 3.9 Erzincan earthquake acceleration records versus time in a North-South direction at RSN821

Scaling factors for selected seismic hazard also obtained from PEER ground motion database. Procedure for getting scale factors are:

- Insert target response spectrum
- Insert search limits for seismic hazard
- Search records
- Select scaling method
- Get search result and scale factors according to a single period [26]

3.4 Soil Modeling

The dynamic properties and response of the structure are affected by several factors; one of the most important factors is an interaction between structure and soil. This interaction led to changing dynamic properties of structure especially nature period of the structure. Soil structure interaction will be studied in this search by modeling foundation system and soil around it by linear spring-dashpot system with frequency independent coefficient as shown in Figure 3.10. Foundation structure interaction model parameters are taken from Mulliken and Karabalis [35].Spring and dashpot directions are vertical, horizontal and rocking. Mass, stiffness and damping coefficient for foundation structure interaction system are given in Table 3.4. The soil properties used in foundation structure interaction model are presented in Table 3.5.

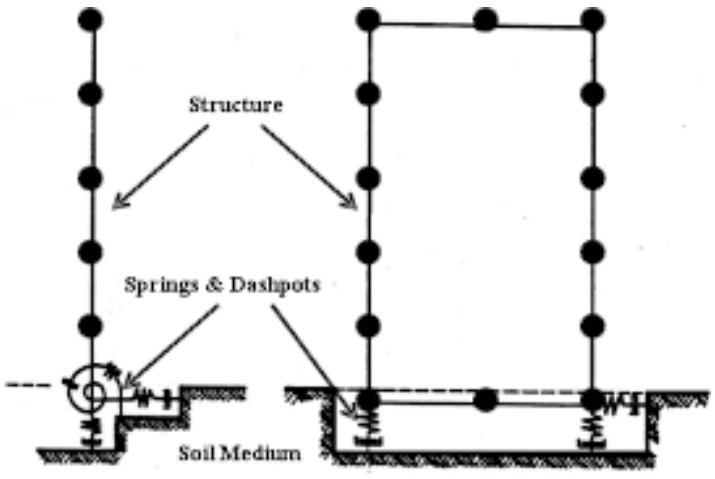


Figure 3.10 Basic 2D dimensional soil foundation model [34]

Foundation system is considered a single footing system. Footing system dimensions for frames are presented in Table 3.6. The used value of foundation structure

interaction model parameters for each frame are listed in Table 3.7 and 3.8 for soil type B and type C respectively

Table 3.4 Mass, stiffness and damping coefficient for foundation structure interaction system [33]

Stiffness	Mass (inertia) Ratio, β	Equivalent Radius, r_0	mass (inertia) m_v	Static stiffness K	Damping C
Vertical	$\frac{(1-\nu)}{4}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{0.27m}{\beta}$	$\frac{4.7Ga}{1-\mu}$	$\frac{0.8a}{V_s}K$
Horizontal	$\frac{(7-8\nu)}{32(1-\nu)}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{0.095m}{\beta}$	$\frac{9.2Ga}{2-\mu}$	$\frac{0.163a}{V_s}K$
Rocking	$\frac{3(1-\nu)}{8}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt[4]{3\pi}}$	$\frac{0.24m}{\beta}$	$\frac{4.0Ga^3}{1-\mu}$	$\frac{0.6a}{V_s}K$

Table 3.5 Soil properties used in foundation structure interaction model [33]

Soil parameter	Description	Assumed value Soil B	
G_0	Shear modulus of soil	260 MPa	
G_s	Secant shear modulus	$0.1G_0$=26 MPa	
ρ	Mass density of soil medium	2000kg/m^3	
V_s	Shear velocity in soil V_s	$\sqrt{\frac{G}{\rho}}$=380	
ν	Passion ratio of the soil medium	0.25	
q	Bearing capacity of the soil	700 kPa	

Table 3.6 Dimension of single footing system used in each frame for soil B/C

Frame name	a×a×H[1] for soil B	a×a×H for soil C
NS3NB3 & NS3NB6	1.50x1.50x0.90	2.1x2.1x1.10
NS6NB3 & NS3NB6	1.80x1.80x1.10	2.45x2.45x1.30
NS9NB3 & NB9NB6	1.95x1.95x1.00	2.60x2.60x1.50
NS12NB3 & NS12NB6	2.20x2.20x1.15	2.80x2.80x1.70
NS15NB3 & NS15NB6	2.75x2.75x1.35	4.50x4.50x1.90
NS20NB3 & NS20NB6	3.00x3.00x1.50	5.00x5.00x2.20

[1] a, a and H are the length, width and depth of foundation respectively

Table 3.7 Horizontal mass, stiffness and damping values for SSI soil type B

Frame name	NS3NB3 NS3NB6	NS6NB3 NS3NB6	NS9NB3 NB9NB6	NS12NB3 NS12NB6	NS15NB3 NS15NB6	NS20NB3 NS20NB6
m_v	12.57	21.73	27.63	39.68	77.50	100.62
c_v	192.92	277.80	326.03	414.98	648.41	771.67
k_v	122200.00	146640.00	158860.00	179226.66	224033.33	244400.00
m_h	3.98	6.88	8.75	12.56	24.54	31.86
c_h	32.97	47.48	55.72	70.93	110.83	131.89
k_h	102514.28	123017.14	133268.57	150354.28	187942.85	205028.57
m_r	22.61	56.27	83.97	153.49	468.42	723.74
c_r	98.51	170.22	216.42	310.79	607.02	788.08
k_r	58500.00	101088.00	128524.50	184565.33	360479.16	468000.00

Table 3.8 Horizontal mass, stiffness and damping coefficient for SSI soil type C

Frame name	NS3NB3 NS3NB6	NS6NB3 NS3NB6	NS9NB3 NB9NB6	NS12NB3 NS12NB6	NS15NB3 NS15NB6	NS20NB3 NS20NB6
m_v	34.51	54.81	65.50	81.81	339.60	465.84
c_v	209.74	285.48	321.51	372.88	963.10	1189.02
k_v	52640.00	61413.33	65173.33	70186.67	112800.00	125333.33
m_h	10.93	17.36	20.74	25.91	107.54	147.52
c_h	35.85	48.80	54.95	63.73	164.62	203.24
k_h	44160.00	51520.00	54674.29	58880.00	94628.57	105142.86
m_r	121.64	262.91	353.87	512.59	5495.92	9307.38
c_r	149.94	238.10	284.57	355.42	1475.39	2023.86
k_r	49392.00	78432.67	93738.67	117077.33	486000.00	666666.67

CHAPTER 4
ANALYSIS AND RESULTS

4.1 Introduction

The linear and nonlinear time history analyses are applied to obtain a more accurate model and simulation of the structure under seismic action, which leads to a more accurate response of the structure. In time history analysis a scaling of real earthquake records to target response spectrum is done. Selection of earthquakes is a very important point. The seismic record station should be close to the construction side and soil typed prefered to be similar. The rules for seleting and scaling ground motions specified at FEMA 440 [6] are used in the present study. This scaling gives an idea about seismic loads which will effect on the structure. It is also good to compare the response of the structure under real earthquakes according to response spectrum defined by codes. In this study, time history analyses are compared with response spectrum (defined according to Euro code 8 analyses. The used response spectrums are evaluated according to Euro code 8 [5].

In literature, it is stated that SSI effects on the dynamic parameter of structure, so the response will change under seismic action. SSI is also one of the important points which give a more accurate simulation of structure to evaluate its response, especially under lateral loads. The frames analyzed with rigid and SSI based foundation in the present study. This comparison gives information about the order of magnitude of SSI effect.

The frames are modeled using ETABS analysis and design program in this study. ETABS program allows users to model the multi-story structure and its loads in easy methods. ETABS offers simple modeling and visualization tools, fast linear and nonlinear analytical power, and comprehensive design capabilities for a wide-range of materials. Also, the program gives analysis results in easy understandable reports and charts. The program gives the ability to apply real records of seismic action in all

direction. It is easy to model response spectrum according to used codes or as user define.

In this research, 12 frames were designed before by Karavasilis et al [14] have been analyzed according to Euro code 8 and Euro code 3. Analyses are done in two cases, one in the rigid base and second with SSI (flexible one) which modeled as linear springs and dampers to represent soil structure interaction effect on structural response.

4.2 Structural Responses Analyses

Seismic performance of the structure is defined by its response to earthquake hazard. The current research aimed to evaluate the most popular seismic response of the structure. This response mainly depends on the way of modeling bases. So the responses are calculated for two cases of the base, first one is rigid base, and the second one is SSI as linear springs and dashpots. Following responses are calculated and compared:

- Naturel period of structure
- The scale factor of earthquake size which hit the buildings
- Base shear
- Inter-story drift
- Story acceleration

4.3 Result and Discussion

The base shear, inter-story drift, maximum story displacement and acceleration are computed using response spectrum analysis, linear and nonlinear time history analyses. The natural period can be obtained by free vibration analysis. The response spectrum analysis module of the ETABS program also includes the free vibration analysis. The presented natural periods are evaluated during response spectrum analysis

4.3.1 Natural period

Seismic performance of the structure mainly depends on the natural period. The natural periods, in turn, depends on the structure stiffness and the support of the structure. In this research, the natural periods are calculated for 12 frames

constructed on soil types B and C. Two cases of bases which are rigid and SSI are also considered. Table 4.1 presents the natural periods for all analyzed frames with difference ratio of period between rigid base and SSI for soil type B and soil type C. It is clearly noticed that the natural period of all frames have been increased when SSI is assumed in the analysis for soil type B and type C. Figure 4.1 shows natural period of analyzed frames on soil B for rigid base and SSI. Figure 4.2 shows the natural period of analyzed frames on soil C for rigid base and SSI.

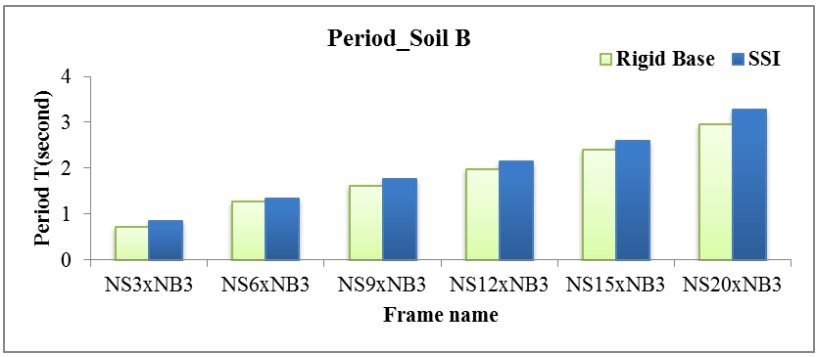
Table 4.1 Natural period of structure for the rigid base and SSI

NB	NS	Natural period T			Ratio	
		Rigid	SSI		$\frac{T_{SSI} - T_{rigid}}{T_{rigid}} \times 100$	
			Soil type B	Soil type C	Soil type B	Soil type C
3	3	0.723	0.851	0.852	17.70	17.84
6	3	0.648	0.727	0.723	12.19	16.00
3	6	1.275	1.346	1.436	5.57	12.63
6	6	1.193	1.263	1.335	5.87	9.00
3	9	1.622	1.760	1.881	8.51	15.97
6	9	1.570	1.680	1.754	7.01	11.72
3	12	1.983	2.152	2.351	8.54	18.56
6	12	1.987	2.113	2.228	6.34	12.13
3	15	2.398	2.596	2.724	8.26	13.59
6	15	2.383	2.517	2.566	5.62	7.68
3	20	2.967	3.285	3.551	10.72	20.00
6	20	2.886	3.063	3.178	6.13	10.00

[1] NS is number of story, [2] NB is number of bay.

Table 4.1 and Figure 4.3 present the increasing ratio of natural period of frames when SSI analyses are considered for both soil type B and C. It is clear that the increasing ratio of natural period is about 10% for all frames constructed on soil type B except frame NS3NB3 which is 17.17%. The increasing ratio of natural period when soil C is considered is between 10% and 20%. The difference between naturel period for rigid base and SSI is decrease when soil is stronger. This is a meaningful result in a way that the behavior of soil is going to be closer to rigid base behavior. The effect of SSI is increased when soil is weaker except for low rise frames with 3 stories height. For this height, the ratio is almost the same.

(a)



(b)

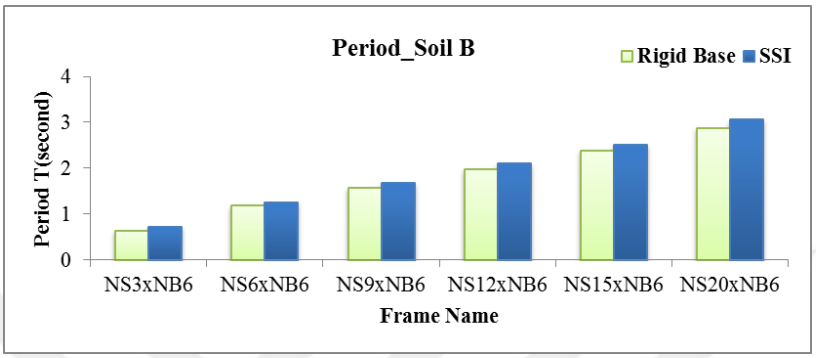
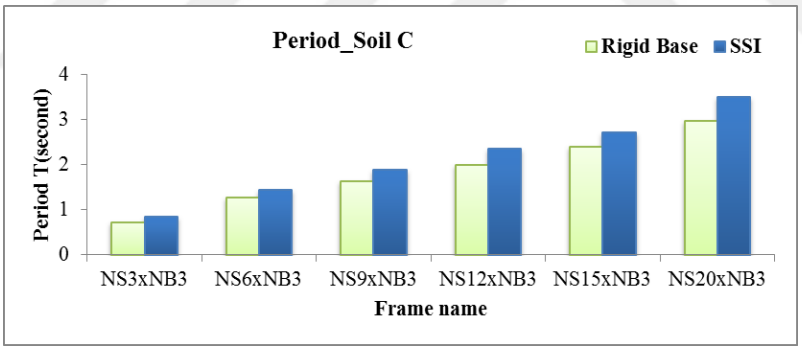


Figure 4.1 Natural period of frames with different height constructed on soil type B (a) 3 bays and (b) 6 bays

(a)



(b)

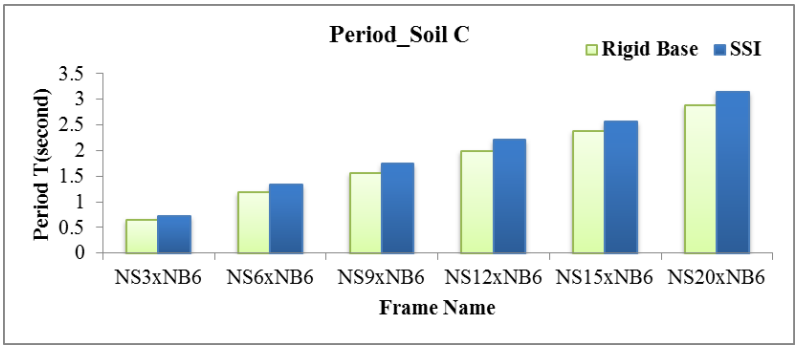
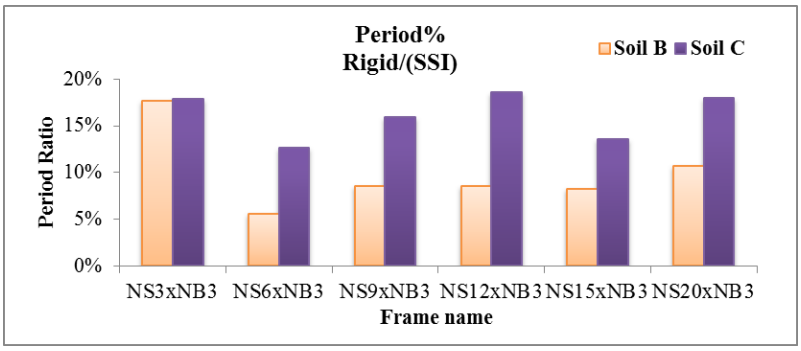


Figure 4.2 Naturel period of frames with different height constructed on soil type C (a) 3 bays and (b) 6 bays

(a)



(b)

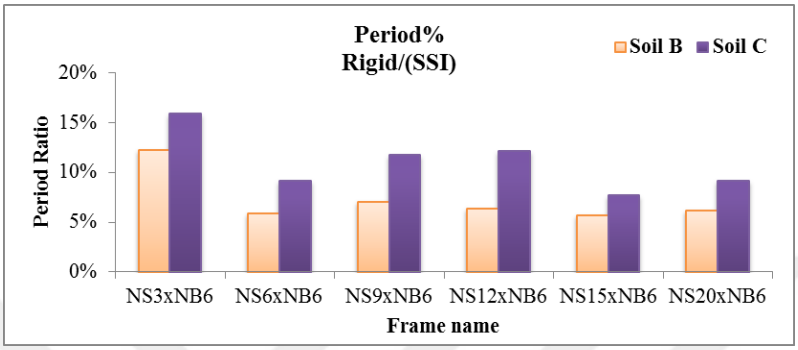


Figure 4.3 Naturel period ratio between rigid and SSI (a) 3 bays (b) 6 bays

4.3.2 Scale factor of earthquakes

There are two methods of scaling as explained in Chapter 3. These are:

- Mean Squared Error (MSE) of the differences between the range of period of structure
- Single period method

Mean Squared Error method is applied for a group of period, and this group was the natural periods of each group of story height for 3 and 6 bays with a rigid base and SSI. Used group of period and corresponding scale factors are listed in Table 4.2. The nonlinear time history analysis is done for soil type B using this scale factors.

In single period method, scale factors are evaluated using the three (Erzincan, Kocaeli, and Düzce) earthquakes records and only Düzce earthquake record for soil B and C respectively. Earthquakes are scaled by using single period method to response spectrum1 which defined in Chapter 3 according to Euro code 8 with PGA0.35 and soil type B. Düzce earthquake is scaled also to response spectrum 2 which is defined in Chapter 3 according to Euro code 8 with PGA0.35 and soil type C. In this method scale factors for rigid base and SSI are different. In single period method, the natural periods of frames are used as period value for scaling, so each frame has a two scaling factors one for rigid base and the other for (SSI). The

computed scalling factors are listed in Table 4.3. In general scale factor and natural period increase when analysis is carried out using SSI.

Table 4.2 Scale factor calculated for soil B by the MSE method

NS[1]	Frame				SF
	NB[2]=3		NB=6		
	Rigid	SSI	Rigid	SSI	
3	0.648	0.723	0.727	0.851	0.6869
6	1.193	1.263	1.275	1.345	0.7981
9	1.570	1.680	1.622	1.760	0.6558
12	1.983	1.987	2.113	2.152	1.3476
15	2.383	2.398	2.596	2.517	1.8577
20	2.886	2.967	3.063	3.285	3.2890

[1] NS is number of story, [2] NB is number of bay.

It is clear from Figure 4.4 and 4.5 that the effects of earthquake on structures are differing according to specification of earthquake and naturel period of structure.The scale factor of Erzincan earthquake is almost same for all natural period. While, in Kocaeli earthquake, it can be seen that the building with low natural periods get higher scale factors from others. It means that the low rise building will be effected by high value of earthquake loads. Düzce earthquake behavior is completely different from the others and signs increasing of scale factors when the natural periods are increase in the studied range of natural period.

According to research results, the natural period is increase when SSI is considered, and the difference between natural period in rigid base and SSI is increeases when soil is going to be weaker. Scale factors based on Düzce earthquake increase when natural periods increase. The scale factors of soil type C are generally slightly higher than soil type B for Düzce earthquake as shown in Figure 4.6.

Düzce earthquake is scaled for two response spectrums which are defined in Chapter 3. The difference between two spectrums is soil types. The scale factors are computed using response spectrum1 for soil type B and response spectrum 2 for soil type C. The results are presented in Table 4.3 and Figure 4.7. The scale factor increases when soil type C is considered. It means that structure will get higher earthquake loads.

Table 4.3 Scales factor of frames according to the single period method

Frame	Scale Factor SF --- Kocaeli/Turkey			Scale Factor SF ---Düzce/Turkey						Scale Factor SF --- Erzincan/Turkey		
				Soil type B			Soil type C					
	Rigid	SSI	% diff[1]	Rigid	SSI	% diff[1]	Rigid	SSI	% diff[1]	Rigid	SSI	% diff[1]
NS3xNB6	0.53	0.63	-10.31	0.62	0.61	59.60	0.70	0.70	51.47	0.20	0.19	-3.98
NS3xNB3	0.63	0.56	20.13	0.61	0.97	-0.70	0.74	1.12	-0.96	0.19	0.18	-4.79
NS6xNB6	0.50	0.47	-5.96	0.87	0.78	-11.63	1.00	0.78	-5.54	0.16	0.16	0.64
NS6xNB3	0.46	0.43	-7.55	0.76	0.67	-10.99	0.71	0.67	-21.58	0.16	0.16	0.58
NS9xNB6	0.36	0.35	1.36	0.56	0.73	61.00	0.63	1.08	115.61	0.15	0.14	-9.69
NS9xNB3	0.35	0.36	-1.34	0.60	0.97	31.11	0.69	1.49	70.58	0.14	0.13	-5.75
NS12xNB3	0.38	0.43	15.23	1.35	1.40	4.26	1.54	1.63	16.69	0.12	0.12	5.41
NS12xNB6	0.38	0.42	11.19	1.35	1.39	3.17	1.54	1.80	5.31	0.12	0.12	3.97
NS15xNB6	0.47	0.48	0.99	1.69	2.03	24.91	1.85	2.25	41.85	0.14	0.14	11.39
NS15xNB3	0.47	0.48	0.87	1.71	2.14	20.26	1.87	2.66	21.70	0.14	0.15	5.01
NS20xNB6	0.50	0.50	4.77	2.99	3.30	-3.03	3.11	3.33	-19.51	0.19	0.21	18.39
NS20xNB3	0.49	0.52	0.69	3.21	3.12	10.16	3.31	2.67	7.12	0.20	0.23	10.69

[1] $[(SF_{SSI} - SF_{rigid})/SF_{rigid}] \times 100$

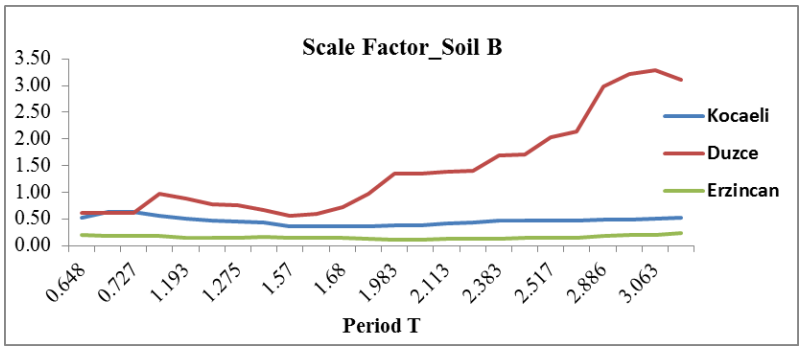
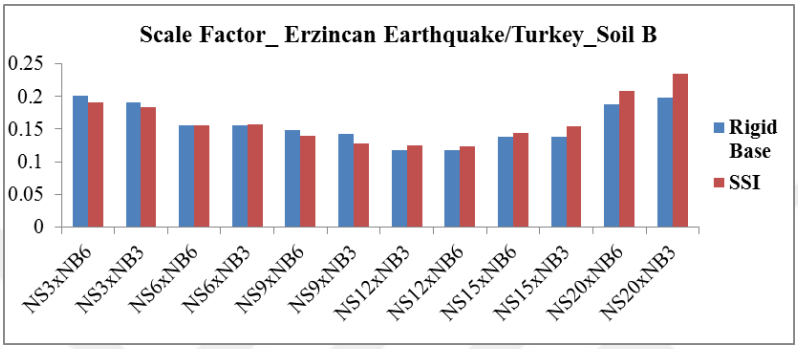
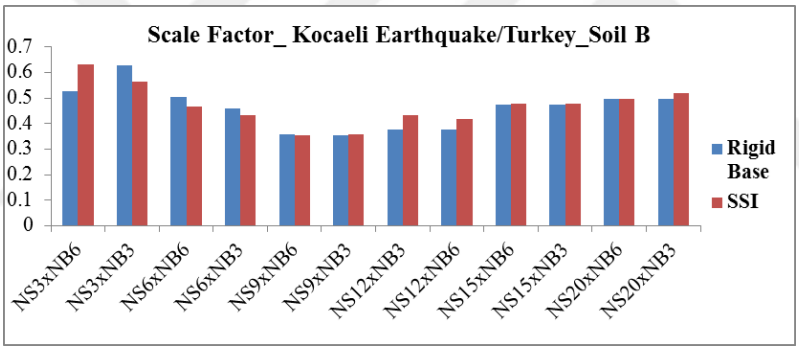


Figure 4.4 Scale factor of studied frames constructed on soil type B

(a)



(b)



(c)

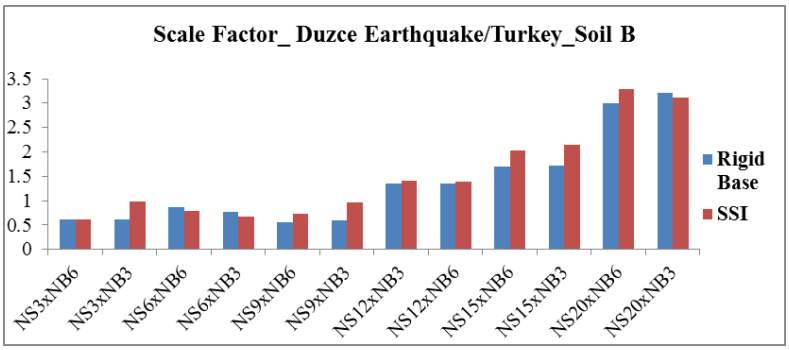


Figure 4.5 Comparison of scale factor for rigid base and SSI constructed soil type B (a) Erzincan earthquake, (b) Kocaeli earthquake and (c) Düzce eartquake

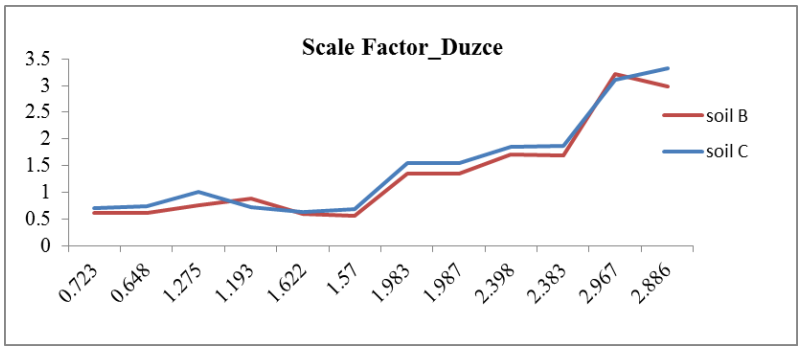


Figure 4.6 Scale factor of frames constructed on soil type B and soil type C for Düzce earthquake Turkey 1999

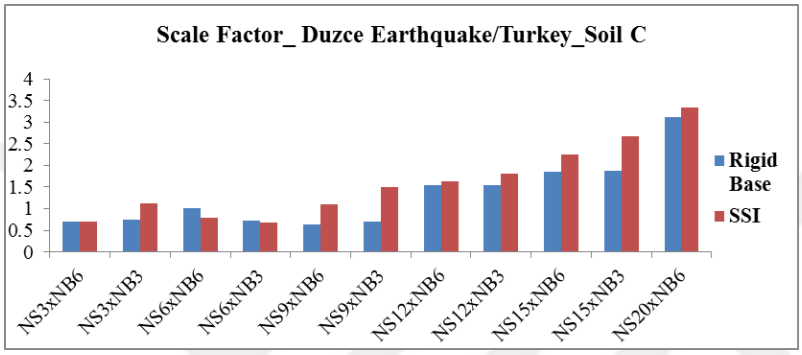


Figure 4.7 Scale factor of frames constructed on soil type C for Düzce earthquake Turkey 1999

Scaling factors which calculated by the single period method are applied on frames which analyzed as linear time history analysis while scaling factors which calculated by MSE method is applied for frames which analyzed as nonlinear time history analysis. In linear and nonlinear time history analysis, the maximum responses of frames are considered. It is noticed that the Düzce earthquake in east direction has the maximum scale factor and maximum response.

4.3.3 Base shear

Base shears are evaluated for frames using linear and nonlinear time history analysis and response spectrum analysis. Response spectrum analysis and linear time history analysis are considered for soil types B and C while nonlinear analysis is carried out only for frame constructed on soil type B.

Base shear of frames constructed on soil type B and C and evaluated using response spectrum analysis are presented in Table 4.4. It is clear that base shears of frames evaluated based on SSI model are lesser than rigid base model for both soil types.

The comparisons of results are illustrated in Figure 4.8 and 4.9 for soil type B and C respectively.

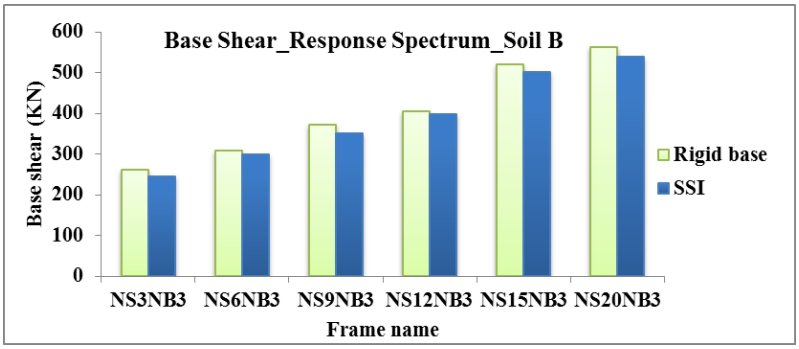
Table 4.4 Base shear of frames constructed on soil type B and soil type C evaluated by response spectrum method for rigid base and SSI

Frame	Soil type B			Soil type C		
	Rigid (kN)	SSI (kN)	% diff[1]	Rigid (kN)	SSI (kN)	% diff[1]
NS3xNB3	260.7	243.8	-6.5	304.2	275.0	-9.6
NS6xNB3	308.7	299.4	-3.0	348.6	320.6	-8.0
NS9xNB3	371.4	349.8	-5.8	419.5	378.3	-9.8
NS12xNB3	405.1	397.1	-2.0	462.4	433.4	-6.3
NS15xNB3	518.7	500.1	-3.6	518.6	485.4	-6.4
NS20xNB3	562.0	538.3	-4.2	594.3	542.6	-8.7
NS3xNB6	624.0	564.1	-9.6	698.2	630.9	-9.6
NS6xNB6	656.6	632.3	-3.7	743.4	696.9	-6.3
NS9xNB6	770.2	736.7	-4.3	868.0	805.2	-7.2
NS12xNB6	804.9	796.9	-1.0	919.4	887.2	-3.5
NS15xNB6	934.2	926.3	-0.8	1036.9	1003.0	-3.3
NS20xNB6	1139.4	1116.2	-2.0	1211.6	1136.8	-6.2

[1] $[(F_{SSI} - F_{rigid})/SF_{rigid}] \times 100$ where F is the base shear.at story level

The percent difference between base shears which calculated for rigid base and SSI are presented in Table 4.4. The difference between the rigid base model and SSI model is slightly higher for soil type C. In response spectrum analysis, the SSI model always gives smaller base shear value compare to the rigid base as shown in Figure 4.10. Anyway the differences are not more than 10% and this lead to consider the effect of SSI on base shear for soil type B and C is small when response spectrum analysis is used to evaluate base shears.

(a)



(b)

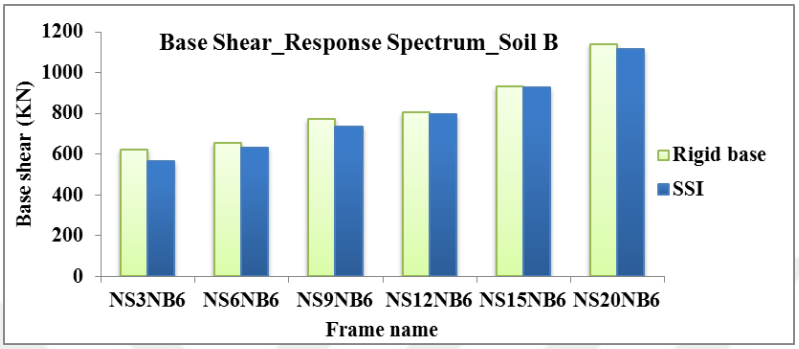
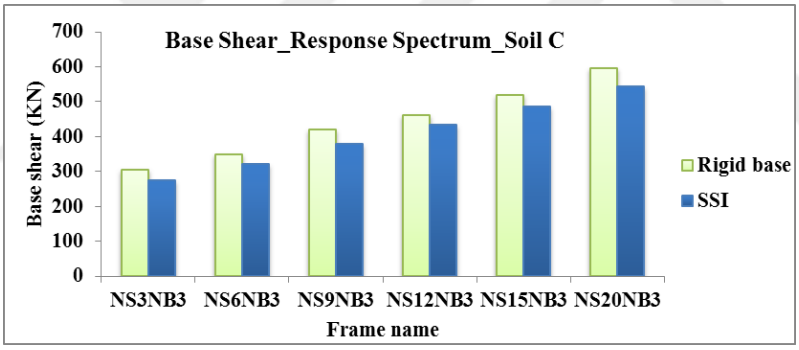


Figure 4.8 Base shear of frames calculated using response spectrum analysis for soil type B (a) 3 bay (b) 6 bay

(a)



(b)

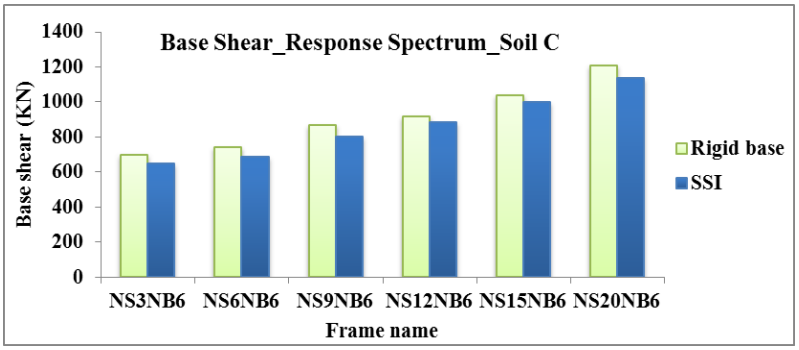
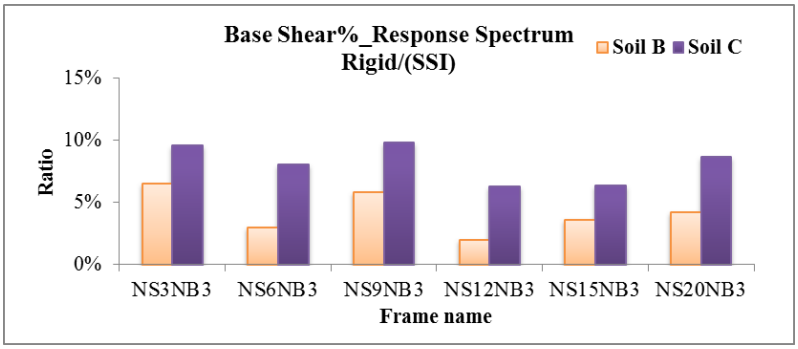


Figure 4.9 Base shear of frames calculated using response spectrum analysis for soil type C (a) 3 bay (b) 6 bay

(a)



(b)

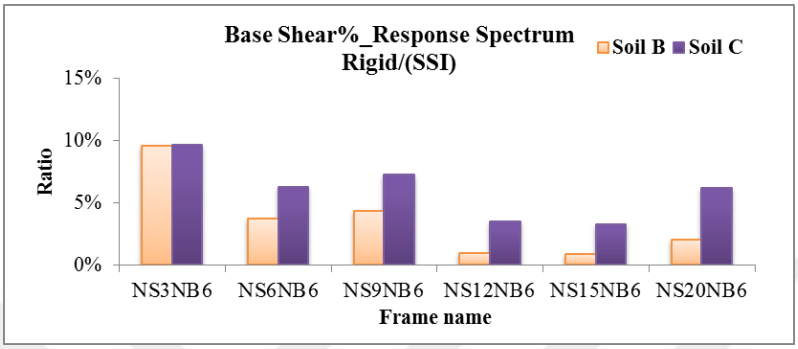
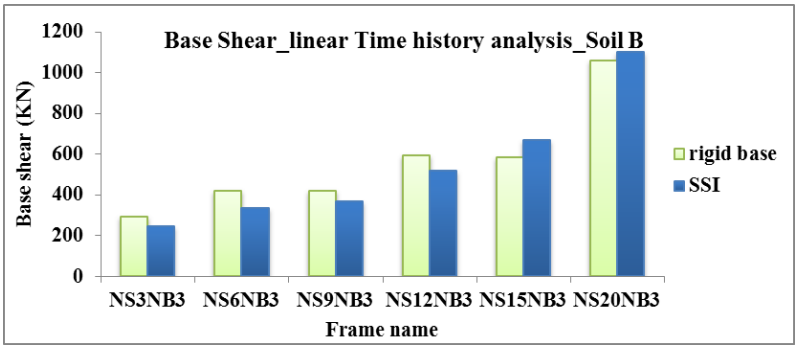


Figure 4.10 Comparision of base shear ratio for rigid and SSI model (a) 3 bays and (b) 6 bays

Base shears are evaluated for frames which construced on two soil types B and C. The results are illustrated in the Figure 4.11 and 4.12 for soil types B and C respectively. Table 4.5 lists maximum base shear of frames analyzed using linear time history analysis. In linear time history analysis, the SSI model generally gives smaller base shear value compare to the rigid base up to 12 stories as shown in Figure 4.11. The one of the reason of changing behaviour is scale factor.

(a)



(b)

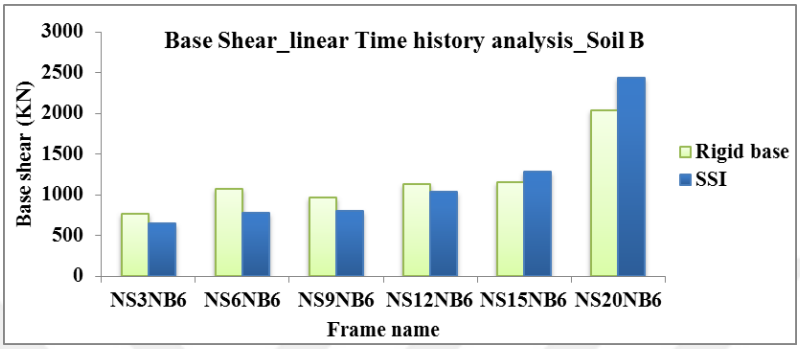
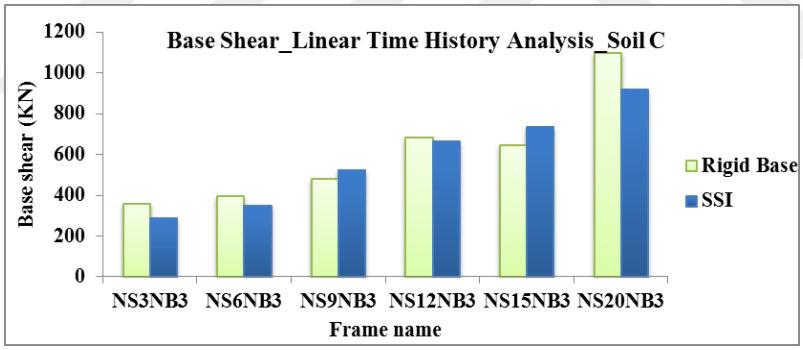


Figure 4.11 Base shear of frames calculated using linear time hisotry analysis for soil B (a) 3 bay (b) 6 bay

(a)



(b)

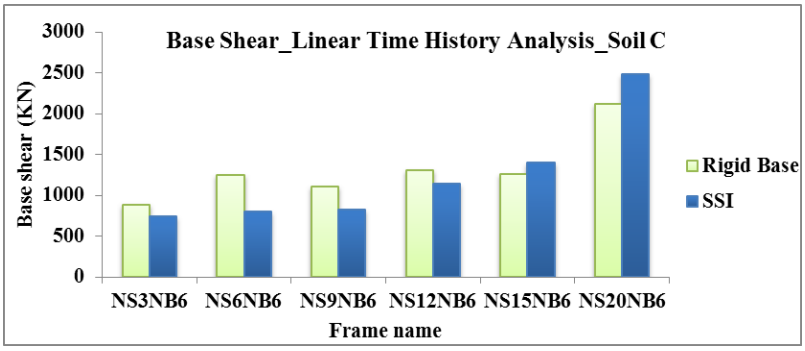


Figure 4.12 Base shear of frames calculated using linear time hisotry analysis for soil type C (a) 3 bay (b) 6 bay

Table 4.5 Base shear of frames constructed on soil types B and C and evaluated by linear time history analysis for rigid base and SSI

Frame	Soil type B			Soil type C		
	Rigid (kN)	SSI (kN)	% diff[1]	Rigid (kN)	SSI (kN)	% diff[1]
NS3xNB3	293.7	247.6	15.70	355.7	284.7	19.97
NS6xNB3	417.4	335.2	19.68	393.6	346.1	12.07
NS9xNB3	418.2	370.1	11.51	480.9	519.1	-7.94
NS12xNB3	592.9	519.5	12.38	682.0	663.6	2.70
NS15xNB3	586.1	669.5	-14.22	641.4	731.2	-13.99
NS20xNB3	1059.1	1101.2	-3.97	1093.4	918.4	16.01
NS3xNB6	763.9	646.0	15.44	878.5	736.8	16.12
NS6xNB6	1074.3	780.9	27.31	1240.7	794.5	35.96
NS9xNB6	962.1	804.2	16.41	1105.9	824.2	25.48
NS12xNB6	1130.6	1029.4	8.95	1299.5	1143.1	12.04
NS15xNB6	1149.0	1279.8	-11.38	1259.5	1392.4	-10.56
NS20xNB6	2029.2	2434.9	-20.00	2110.9	2474.1	-17.21

[1] $[(F_{SSI} - F_{rigid})/SF_{rigid}] \times 100$ where F is the base shear.at story level

Table 4.6 lists maximum base shear of frames analyzed using nonlinear time history analysis. Base shears are evaluated for frames which constructed on soil type B. The results are illustrated in the Figure 4.13. In nonlinear time history analysis, the SSI model generally gives smaller base shear value compare to the rigid base up to 12 stories as shown in Figure 4.13.

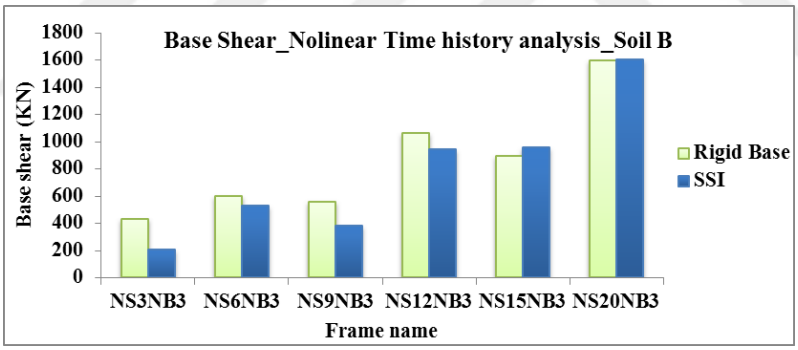
The percent difference between base shears which calculated for rigid base and SSI are presented in Table 4.6 and 4.7 for linear time history analysis and nonlinear time history analysis respectively. The difference between the rigid base model and SSI model is higher when nonlinear analysis is used. One of the reason of changing behaviour is height of structure.

Table 4.6 Base shear calculated by nonlinear time history analysis for the rigid base and (SSI) of frames constructed on soil type B.

Frame	Rigid (kN)	SSI (kN)	% diff[1]
NS3NB3	428.3	202.1	52.82
NS6NB3	597.3	529.2	11.41
NS9NB3	555.0	378.4	31.82
NS12NB3	1057.7	942.7	10.88
NS15NB3	893.3	955.4	-6.95
NS20NB3	1595.5	1602.7	-0.45
NS3NB6	947.3	720.3	23.96
NS6NB6	1368.5	994.2	27.35
NS9NB6	1299.6	934.4	28.10
NS12NB6	2063.1	1416.5	31.34
NS15NB6	1764.3	2143.9	-21.52
NS20NB6	2733.4	2958.1	-8.22

[1] $[(F_{SSI} - F_{rigid})/SF_{rigid}] \times 100$ where F is the base shear.at story level

(a)



(b)

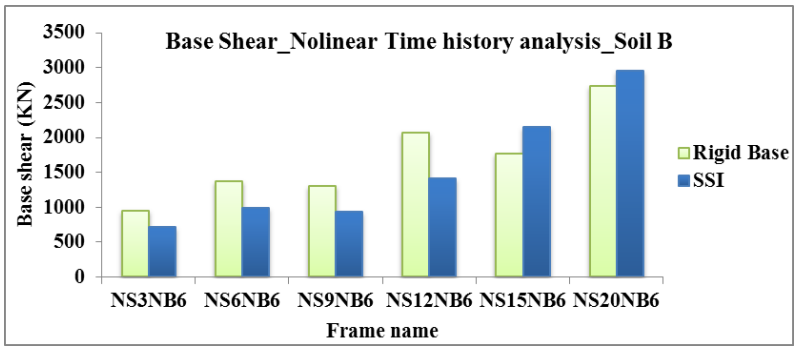


Figure 4.13 Base shear of frames calculated using nonlinear time hisotry analysis for soil B (a) 3 bay (b) 6 bay

4.3.4 Interstory drift

Inter story drift (IDR) evaluated using linear and nonlinear time history analysis and response spectrum analysis. Response spectrum analysis and linear time history analysis are considered for soil types B and C while nonlinear analysis is carried out only for frame constructed on soil type B.

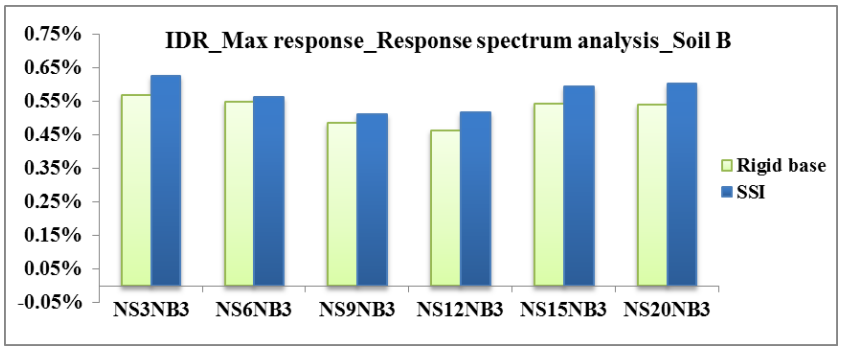
Maximum IDRs of studied frames are evaluated using response spectrum analysis and constructed on soil type B and C are presented in Table 4.7. Generally it is clear that IDR of frames evaluated based on SSI model are higher than rigid base model for both soil types. The comparisons of results are illustrated in Figure 4.14 and 4.15 for soil types B and C respectively. The maximum IDR mostly occur in the second floor, but this location some time changes.

Table 4.7 Maximum IDR based on response spectrum analysis.

Frame	Soil type B				Soil type C			
	Rigid		SSI		Rigid		SSI	
	IDR	SN[1]	IDR	SN[1]	IDR	SN[1]	IDR	SN[1]
NS3xNB3	0.00568	1	0.00625	2	0.00667	2	0.00625	1
NS6xNB3	0.00548	2	0.00563	2	0.00627	2	0.00660	2
NS9xNB3	0.00486	2	0.00511	2	0.00553	2	0.00608	2
NS12xNB3	0.00462	2	0.00517	2	0.00530	2	0.00640	2
NS15xNB3	0.00542	2	0.00594	3	0.00542	3	0.00607	3
NS20xNB3	0.00540	3	0.00601	3	0.00569	3	0.00650	3
NS3xNB6	0.00514	2	0.00493	2	0.00579	2	0.00565	2
NS6xNB6	0.00511	2	0.00521	2	0.00585	2	0.00600	2
NS9xNB6	0.00469	2	0.00489	2	0.00533	2	0.00573	1
NS12xNB6	0.00477	2	0.00524	2	0.00548	2	0.00631	2
NS15xNB6	0.00502	2	0.00562	2	0.00559	2	0.00612	2
NS20xNB6	0.00543	3	0.00586	3	0.00577	3	0.00616	3

[1] SN: story which maximum IDR occurred

(a)



(b)

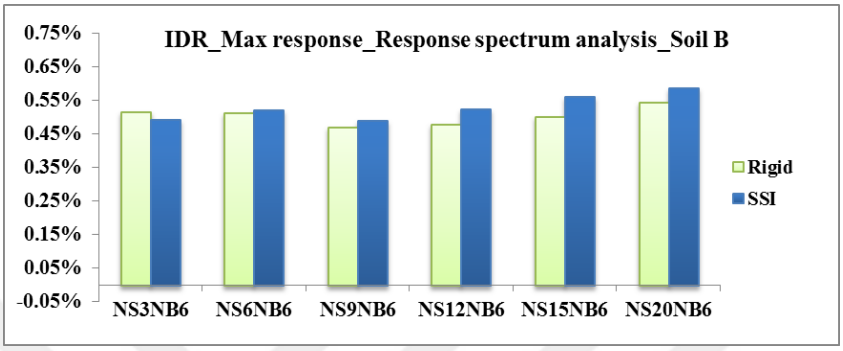
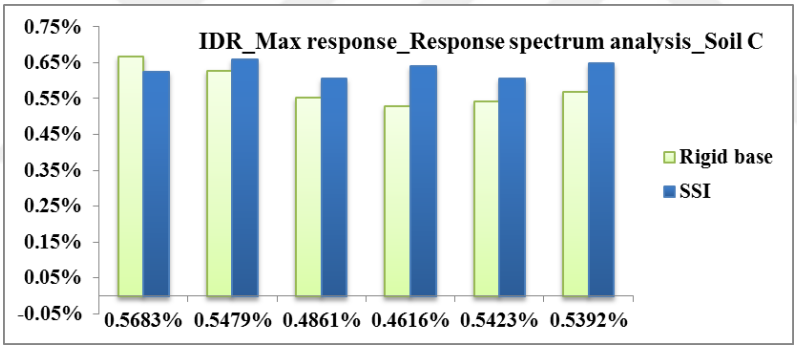


Figure 4.14 Maximum IDR based on response spectrum analysis for soil type B (a) 3 bay (b) 6 bay

(a)



(b)

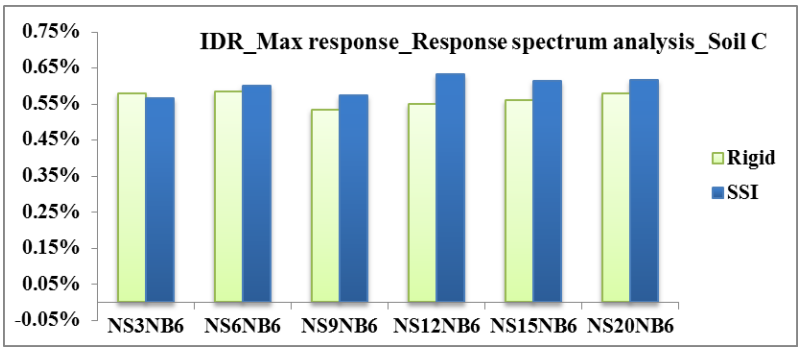


Figure 4.15 Maximum IDR based on response spectrum analysis for soil type C (a) 3 bay (b) 6 bay

Table 4.8 lists maximum IDR of frames analyzed using linear time history analysis. IDR are evaluated for frames which constructed on two soil types B and C. The results are illustrated in the Figure 4.16 and 4.17 for soil type B and C respectively.

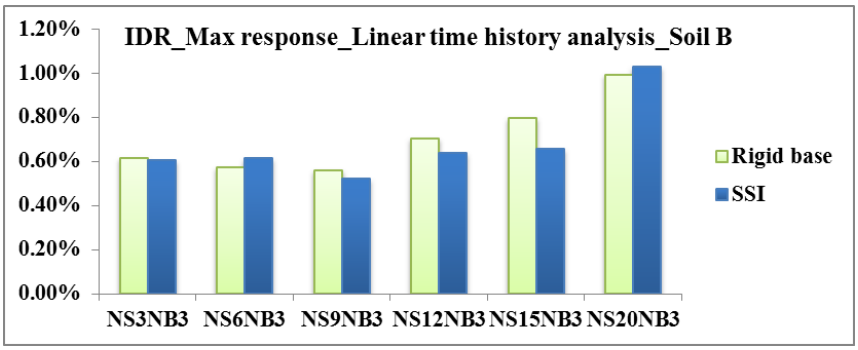
Table 4.8 Maximum IDR based on linear time history analysis

Frame	Soil type B				Soil type C			
	Rigid		SSI		Rigid		SSI	
	IDR	SN[1]	IDR	SN[1]	IDR	SN[1]	IDR	SN[1]
NS3xNB3	0.00614	2	0.00605	1	0.00745	2	0.00697	1
NS6xNB3	0.00574	2	0.00614	2	0.00541	2	0.00701	2
NS9xNB3	0.00557	3	0.00519	4	0.00641	3	0.00770	8
NS12xNB3	0.00705	11	0.00637	8	0.00811	11	0.00837	11
NS15xNB3	0.00794	8	0.00655	2	0.00696	8	0.00850	9
NS20xNB3	0.00992	3	0.01028	15	0.01024	15	0.00891	15
NS3xNB6	0.00577	2	0.00553	1	0.00664	2	0.00609	1
NS6xNB6	0.00739	2	0.00575	2	0.00850	2	0.00578	2
NS9xNB6	0.00591	2	0.00519	2	0.00680	2	0.00558	4
NS12xNB6	0.00655	8	0.00642	8	0.00753	8	0.00703	2
NS15xNB6	0.00637	9	0.00699	9	0.00698	9	0.00734	9
NS20xNB6	0.00892	19	0.01009	15	0.00928	19	0.01045	15

[1] SN: story which maximum IDR occurred

In linear time history analysis, SSI model start to give lower value compare to the rigid base as shown in Figure 4.16 and 4.17. The location of maximum IDR changes from one frame to other. The maximum IDR value also show large variation from one frame to other. The linear time history analysis is not suitable method for IDR evaluation.

(a)



(b)

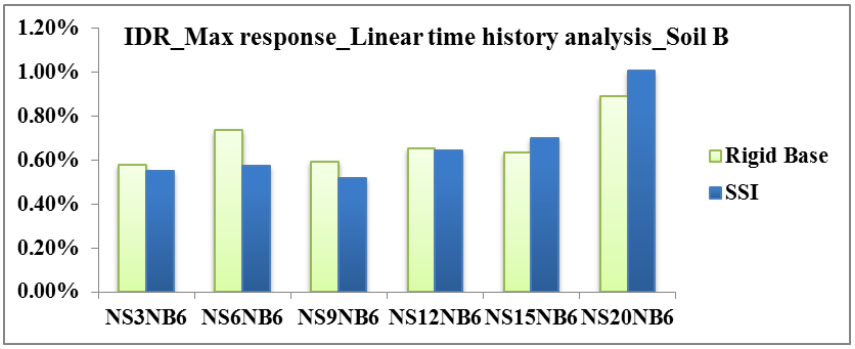
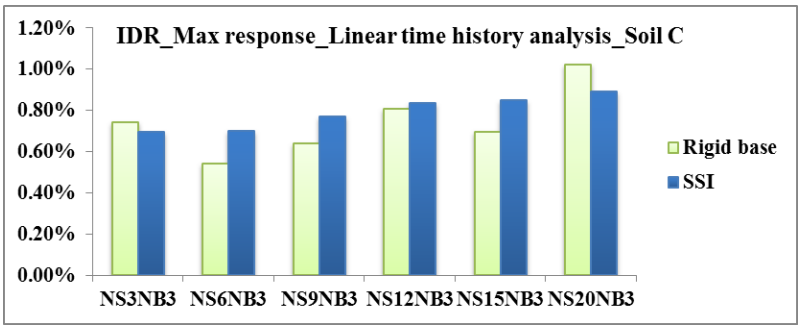


Figure 4.16 Maximum IDR based on linear time history analysis for soil type B (a) 3 bay (b) 6 bay

(a)



(b)

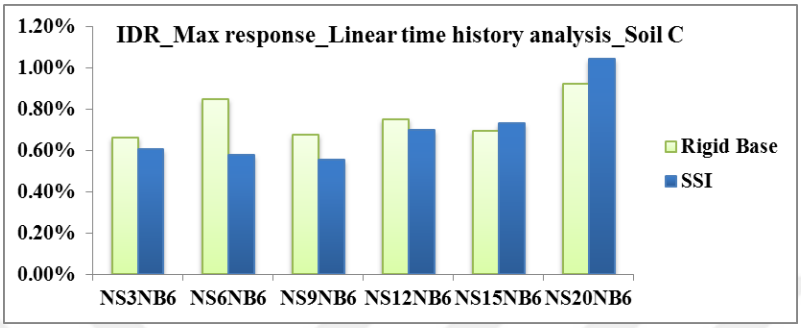


Figure 4.17 Maximum IDR based on linear time history analysis for soil type C (a) 3 bay (b) 6 bay

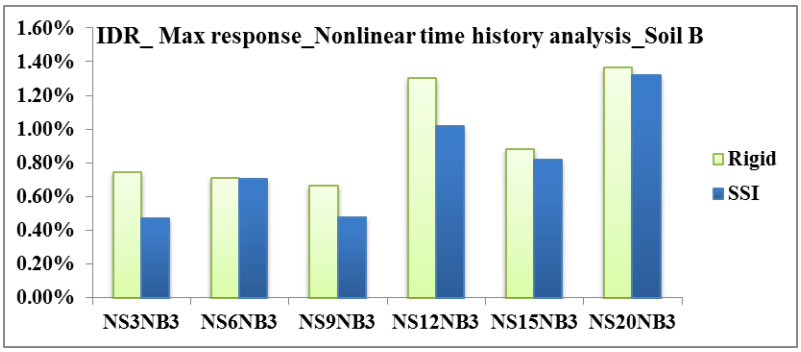
Table 4.9 lists maximum IDR of frames analyzed using nonlinear time history analysis. IDR are evaluated for frames which constructed on soil type B. The results are illustrated in the Figure 4.18. In nonlinear time history analysis, SSI model generally gives smaller IDR value compare to the rigid base.

Table 4.9 Maximum IDR based on nonlinear time history analysis

Frame	Soil type B			
	Rigid		SSI	
	IDR	SN[1]	IDR	SN[1]
NS3xNB3	0.00747	2	0.00471	1
NS6xNB3	0.00713	6	0.00709	4
NS9xNB3	0.00666	2	0.00481	4
NS12xNB3	0.01305	11	0.01018	11
NS15xNB3	0.00881	14	0.00823	9
NS20xNB3	0.01370	19	0.01323	19
NS3xNB6	0.00747	2	0.00471	1
NS6xNB6	0.00713	6	0.00709	4
NS9xNB6	0.00666	2	0.00481	4
NS12xNB6	0.01305	11	0.01018	11
NS15xNB6	0.00881	14	0.00823	9
NS20xNB6	0.01370	19	0.01323	19

[1] SN: story which maximum IDR occurred

(a)



(b)

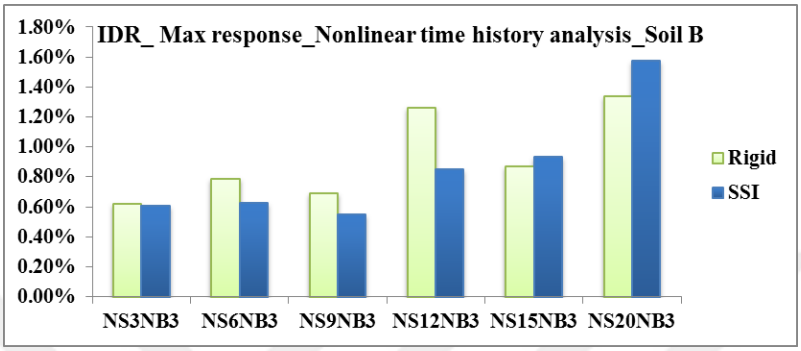


Figure 4.18 Maximum IDR based on nonlinear time history analysis for soil type B (a) 3 bay (b) 6 bay

4.3.5 Story acceleration

Story acceleration is evaluated using linear and nonlinear time history analysis and response spectrum analysis. Response spectrum analysis and linear time history analysis are considered for soil types B and C while nonlinear analysis is carried out only for frame constructed on soil type B.

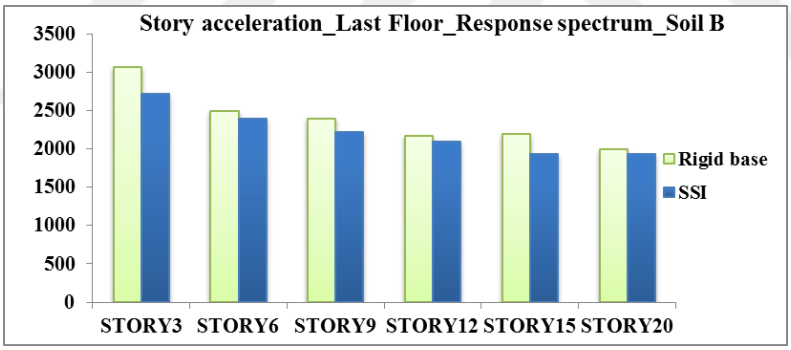
Top story acceleration based on response spectrum analysis for studied frames constructed on soil type B and C are presented in Table 4.10. It is observed that story acceleration of frames evaluated based on SSI model are always smaller than rigid base model for both soil types. The comparisons of results are illustrated in Figure 4.19 and 4.20 for soil type B and C respectively.

Table 4.10 Story acceleration based on response spectrum analysis

Frame	Soil type B			Soil type C		
	a_{rigid} (mm/s^2)	a_{SSI} (mm/s^2)	% diff[1]	a_{rigid} (mm/s^2)	a_{SSI} (mm/s^2)	% diff[1]
NS3xNB3	3072.8	2720.3	-11.47	3522.0	3017.2	-14.33
NS6xNB3	2493.8	2395.1	-3.96	2635.9	2395.4	-9.13
NS9xNB3	2397.6	2229.0	-7.03	2546.8	2395.4	-5.95
NS12xNB3	2175.3	2100.4	-3.44	2358.0	2336.3	-0.92
NS15xNB3	2197.7	1937.4	-11.84	2197.7	2111.0	-3.95
NS20xNB3	2001.5	1937.4	-3.20	2089.9	2021.2	-3.29
NS3xNB6	3685.1	3151.6	-14.48	3685.1	3426.3	-37.92
NS6xNB6	2605.9	2454.8	-5.80	2605.9	2520.6	-9.01
NS9xNB6	2441.1	2250.7	-7.80	2569.9	2328.6	-9.39
NS12xNB6	2131.4	2020.7	-5.20	2131.4	2125.6	-8.04
NS15xNB6	2041.4	1960.0	-3.99	2041.4	2057.1	-4.30
NS20xNB6	1961.0	1897.1	-3.26	1961.0	1970.7	-4.13

[1] $[(a_{SSI} - a_{rigid})/a_{rigid}] \times 100$ where a is the acceleration at top story

(a)



(b)

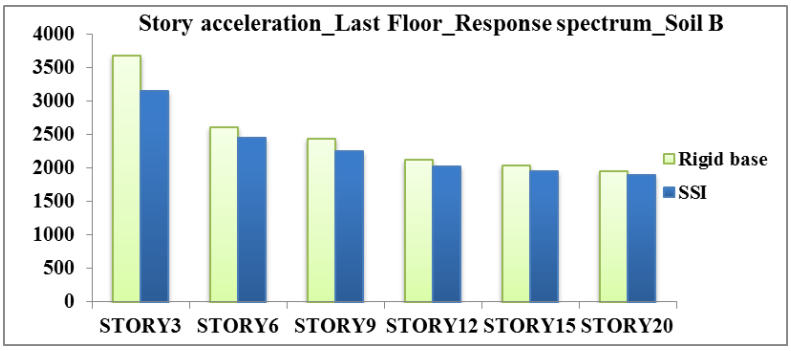
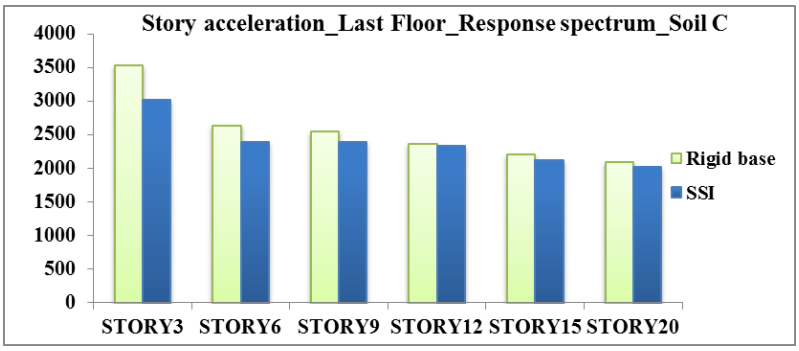


Figure 4.19 Top story acceleration based on response spectrum analysis for soil type B (a) 3 bay (b) 6 bay

(a)



(b)

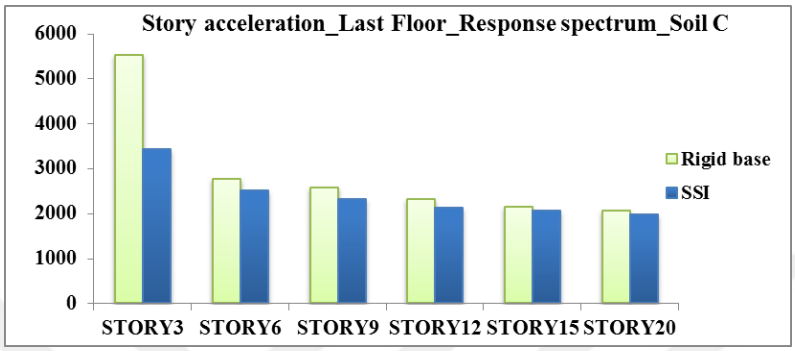


Figure 4.20 Top story acceleration based on response spectrum analysis for soil type C (a) 3 bay (b) 6 bay

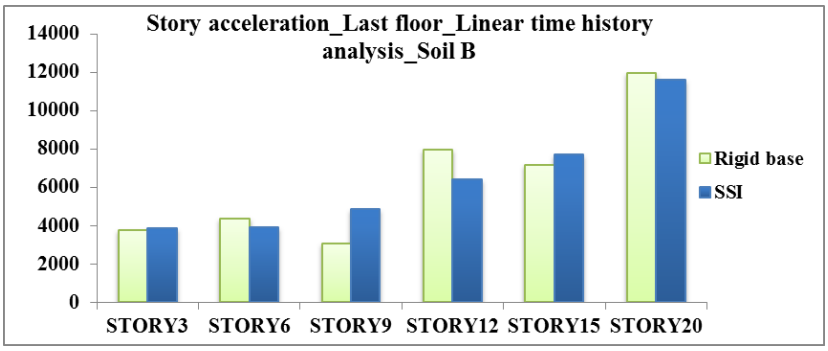
Table 4.11 lists maximum top story acceleration of frames analyzed using linear time history analysis. Story accelerations of last floor are evaluated for frames which constructed on two soil types B and C. The results are illustrated in the Figure 4.21 and 4.22 for soil type B and C respectively. No rule or clear result for increasing or decreasing acceleration of last floor.

Table 4.11 Story acceleration based on linear time history analysis

Frame	Soil type B			Soil type C		
	a_{rigid} (mm/s^2)	a_{SSI} (mm/s^2)	% diff[1]	a_{rigid} (mm/s^2)	a_{SSI} (mm/s^2)	% diff[1]
NS3xNB3	3771.9	3848.3	2.02	4572.8	4375.4	-4.32
NS6xNB3	4337.3	3906.8	-9.93	4090.5	3333.8	-18.50
NS9xNB3	3087.0	4879.2	58.05	3551.0	7618.0	114.53
NS12xNB3	7933.2	6418.7	-19.09	9125.2	6915.0	-24.22
NS15xNB3	7161.4	7690.4	7.39	6274.4	9514.4	51.64
NS20xNB3	11958.2	11570.8	-3.24	12345.6	9993.9	-19.05
NS3xNB6	4799.3	3347.8	-30.24	5519.1	3816.5	-30.85
NS6xNB6	5134.0	4102.5	-20.09	5905.1	4142.1	-29.85
NS9xNB6	2527.5	3294.5	30.34	2906.3	4840.8	66.56
NS12xNB6	6341.2	5199.3	-18.01	7292.7	6180.8	-15.25
NS15xNB6	6359.8	7748.6	21.84	6971.6	8491.7	21.80
NS20xNB6	11358.8	12445.7	9.57	11815.9	12650.4	7.06

[1] $[(a_{SSI} - a_{rigid})/a_{rigid}] \times 100$ where a is the acceleration at top story

(a)



(b)

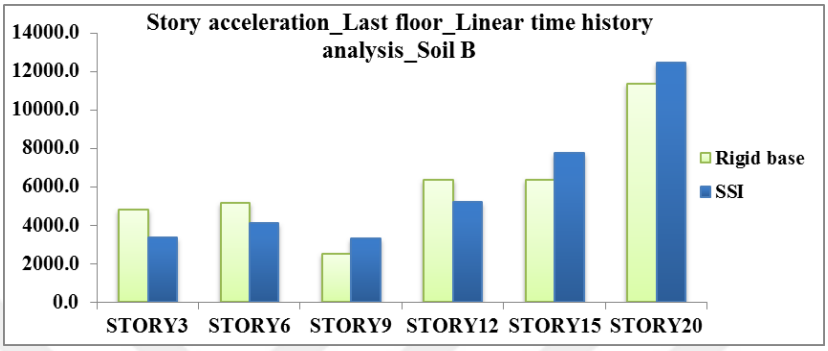
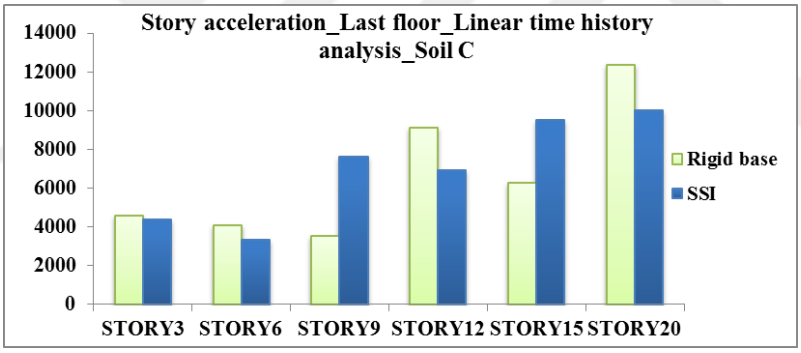


Figure 4.21 Maximum top story acceleration of last floor for frames calculated using linear time history analysis for soil type B (a) 3 bay (b) 6 bay

(a)



(b)

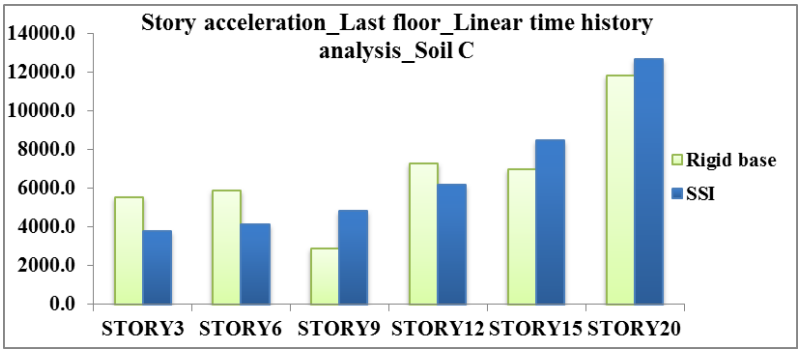


Figure 4.22 Maximum top story acceleration of frames calculated using linear time history analysis for soil type C (a) 3 bay (b) 6 bay

Table 4.12 lists maximum top story acceleration of frames analyzed using nonlinear time history analysis. Story accelerations of last floor areevaluated for frames which constructed on soil type B. The results are illustrated in the Figure 4.23. In nonlinear time history analysis, it is observed that story acceleration of frames evaluated based

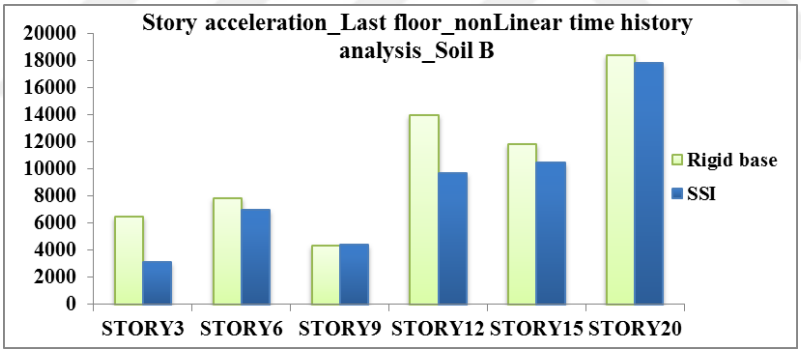
on SSI model are generally smaller than rigid base model for soil type B. The comparisons of results are illustrated in as shown in Figure 4.22.

Table 4.12 Maximum top story acceleration based on nonlinear time history analysis

Frame	a_{rigid} (mm/s^2)	a_{SSI} (mm/s^2)	$\frac{(a_{SSI} - a_{rigid})}{a_{rigid}} \times 100$
NS3NB3	6476.4	3132.8	-51.63%
NS6NB3	7833.1	7014.3	-10.45%
NS9NB3	4347.4	4410.7	1.45%
NS12NB3	13949.2	9685.1	-30.57%
NS15NB3	11871.1	10487.1	-11.66%
NS20NB3	18368.9	17801.9	-3.09%
NS3NB6	6617.7	3510.4	-46.95%
NS6NB6	6417.9	6421.8	0.06%
NS9NB6	4681.6	3758.0	-19.73%
NS12NB6	10303.4	9877.5	-4.13%
NS15NB6	11804.2	13661.1	15.73%
NS20NB6	20412.8	18648.5	-8.64%

[1] $[(a_{SSI} - a_{rigid})/a_{rigid}] \times 100$ where a is the acceleration at top story

(a)



(b)

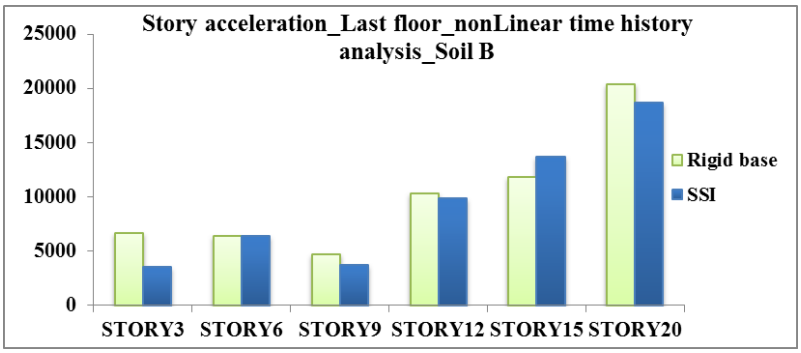


Figure 4.23 Maximum top story acceleration of last floor for frames calculated using nonlinear time history analysis for soil type B (a) 3 bay (b) 6 bay

CHAPTER 5
CONCLUSION AND FURTHER WORK

5.1 General

Over the last years, a significant effort has been done to produce practical methods which model SSI. This is because the response of structure based on flexible soil is significantly different from the structure found on solid rocks. Effect of SSI is neglected in many seismic codes, but many researches confirm the importance of SSI to define the response of structure according to change dynamic characters of it.

In this study, a 2D dynamic linear and nonlinear analysis under seismic loads has been done for 12 frames. A number of stories are changed from 3 to 20, while the number of bays is 3 and 6 bays. The height of stories is 3 m and the width of bays is 5m.

Bases of frames are modeled as and flexible. Foundation soil interaction model is used to determine a parameter of springs and damper for the flexible base. Springs and dampers are considered linear. Foundation system is considered a single footing system.

High strong motion is considered for seismic analysis where PGA 0.35g. Seismic analysis is done in two cases, the first one is response spectrum and the second one is the time history analysis. A response spectrum analysis is carried out according to Euro code 8 with soil types B and C.

Düzce earthquake 1999, Kocaeli 1992 and Erzincan 1999 earthquakes records are used in the time history analysis. These record have a PGA value which close to 0.35g. The analysis is done just in the horizontal direction and two records of each earthquake are applied on frames to determine the highest response.

Natural period of each frame is determined after dynamic response spectrum analysis. This period is used to determine the earthquake scale factor according to a single period method. The scaling factors obtained using single period method are considered in linear time history analysis. The scaling factors obtained from MSE method are used for scaling earthquake in nonlinear analysis. PEER strong ground motion website is used to find records and scaling them.

A linear time history analysis of scaled records has been done. It was found that Düzce earthquake in east direction was the stronger and response of frames was highest. Then a nonlinear analysis with P-Delta effect has been done under Düzce earthquake in the east direction.

A comparison of response parameter has been done for linear and nonlinear time history analysis and response spectrum analysis of frames based on rigid base and SSI model. The parameters are listed below:

- Natural period
- Scaling factor
- Base shear
- Story drift
- Story acceleration

5.2 Conclusions

In this research, SSI effect is investigated for 2D moment resisting frames with different height under seismic loads. Three types of dynamic analysis and two types of soil are considered for comparison. The methods for dynamic analysis are response spectrum analysis, linear and nonlinear time history analysis. Detailed examination of the seismic response of analyzed frames shows that:

1. SSI modeling gives higher natural period value compare to frame on rigid base. Increasing ratio of natural period is about 10% in SSI model for all frames constructed on soil type B except frame NS3NB3 which is 17.17%. The increasing ratio of natural period is between 10% and 20% when soil type C is considered in SSI modeling. The difference between naturel period for rigid base and SSI is decrease when soil is stiffer. The effect of SSI is increased when soil is weaker except for low rise frames with 3 stories height. For this height, the ratio is almost the same.

2. The effects of earthquake on structures are differing according to specification of earthquake and naturel period of structure. The scale factor of Erzincan earthquake is almost same for all natural period. While, in Kocaeli earthquake, it can be seen that the building with low natural periods get higher scale factors from others. It means that the low rise building will be affected by high value of earthquake loads. Düzce earthquake behavior is completely different from the others and signs increasing of scale factors when the natural periods are increase in the studied range of natural period.
3. Response spectrum analysis always led to decrease base shear of frames evaluated based on SSI model for both soil types B and C. The difference between the rigid base model and SSI model is slightly higher for soil type C.
4. In linear and nonlinear time history analysis, the SSI model generally gives smaller base shear value compare to the rigid base up to 12 stories. The one of the reason of changing behavior is scale factor. The difference between the rigid base model and SSI model is higher when nonlinear analysis is used. One of the reasons of changing behavior is height of structure for both soil types B and C.
5. Generally maximum IDR evaluated by response spectrum analysis based on SSI model are higher than rigid base model. In case of nonlinear time history analysis, SSI model generally gives smaller IDR value compare to the rigid base. In linear time history analysis results shows fluctuation.
6. Top story acceleration evaluated based on SSI model for both soil types B and C are always smaller than rigid base model using response spectrum analysis. While in case of linear time history analysis, it is found that no rule or clear result for increasing or decreasing acceleration of last floor. In nonlinear time history analysis, it is observed that story acceleration of frames evaluated based on SSI model are generally smaller than rigid base model for soil types B and C.

5.3 Future Works

Potential developments of the current thesis in the future could be:

1. Since there are many types of soil which differs in specification and behavior. It is worth to study the effect of SSI on the behavior of structures which are constructed on difference types of soil especially weak soil like D and E types

2. Different types of foundation system lead to different SSI parameters, which are the real challenge for engineers. The modeling using another type of foundations system such as piles or raft may give more realist view of SSI effect
3. In this study, the parameters such as stiffness and damping in SSI modeling are considered linear. It is important to observe the effect of nonlinearity of soil in SSI modeling.
4. The substructure approach is used for modeling soil effect. It means that soil is replaced by springs and dashpots. Direct analysis can be used for soil modeling. In this method, structure, foundation and surrounding soil are modeled together.
5. The present study can repeated for different PGA. So various ground motion effect (strong or weak) can be observed.
6. It is important to analyze higher structure which is more than 20 stories and check the effect of SSI
7. Use another system of frames like bracing eccentric and compare response.
8. It is worth to investigate structures in three dimension for observing SSI effect. It is also important to study the effect of SSI on irregular buildings.

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