Seismic Design of Multi-Storey Buildings by Using Stability Coefficient Response Spectra via the FSSDOF System

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ABSTRACT

Earthquake resistant structures must meet the conditions specified in the standard seismic codes of the country. Force-based method is a commonly used method for design and analysis of buildings. The method requires preliminary design of elements to determine the period of the building. Furthermore, iteration can be carried out to satisfy inter-storey drift limits specified by the seismic codes. Recently developed “first storey single degree of freedom”(FSSDOF) approach by Mousavi and Şensoy, is used to determine base shear according to Stability Coefficient (SC) using “Stability Coefficient Response Spectrum” (SCRS). SCRS is formed due to period dependence feature of stability coeffient and spectral acceleration is plotted versus SC for an inverted pendulum with a known height. FSSDOF is used as a single degree of freedom (SDOF) system since the height of first storey rarely changes during the design process and the shear force acting on the first storey is directly equal to the base shear. In order to create response spectra, dynamic response of SDOF systems is analysed by using Newmark’s Method for linear systems. In this thesis, two structures of different heights, five storey and nine storey, were designed and analyzed for different yield strength reduction factors by force based design. Then by calculating the equivalent SC of the fundamental vibration period, base shears were obtained by conventional response spectra and SC response spectra were compared. Higher base shear values were obtained by SCRS since P-delta effect is inherited within the equation of motion.

Keywords: stability coefficient response spectrum, first storey single degree of freedom system, force based design, multi-storey buildings.

ÖZ

Depreme dayanıklı yapılar, ülkenin standart sismik kodlarında belirtilen koşulları karşılamalıdır. Kuvvete dayalı yöntem, yapıların tasarımı ve analizi için en yaygın kullanılan yöntemlerden biridir. Yöntem binanın temel periyodunu belirlemek için elemanların ön tasarımını gerektirir. Ayrıca, sismik kodlar tarafından belirlenen katlar arası ötelelenme sınırlarını karşılamak için iterasyon yapılması gerekebilir. Mousavi ve Şensoy tarafından yeni geliştirilen "Birinci Kat Tek Serbestlik Derecesi"(FSSDOF) yaklaşımı, "Stabilite Katsayısı Davranış Spektrumu" (SCRS) kullanılarak Stabilite Katsayısına (SC) göre taban kesme kuvvetini belirlemek için kullanılır. SCRS, stabilite katsayısının periyot bağımlılığı özelliği sayesinde oluşturulur ve spektral ivme, bilinen bir yüksekliğe sahip ters bir sarkaç için SC değerine karşı çizilir. Birinci katın yüksekliği tasarım sürecinde nadiren değiştiği ve birinci kat üzerinde etkili olan kesme kuvveti doğrudan taban kesim kuvvetine eşit olduğu için FSSDOF, tek serbestlik dereceli (SDOF) sistem olarak kullanılmıştır. Davranış spektrumlarını oluşturmak için, Newmark'ın doğrusal sistemler için geliştirdiği yöntem kullanılarak SDOF sisteminin dinamik davranışı analiz edilmiştir. Bu tezde, biri beş katlı diğeri dokuz katlı olan iki yapı, farklı azaltma faktörleri için kuvvete dayalı tasarım metodu ile tasarlanıp analiz edilmiştir. Daha sonrasında, klasik davranış spektrumu ve temel titreşim periyodu eşdeğer SC değerini hesaplayarak SC davranış spektrumu ile elde edilen taban kesme kuvveti karşılaştırılırmıştır. P-delta etkisi hareket denklemi içine yerleştirilmiş olduğu için, SCRS ile daha yüksek taban kesme değerleri elde edilmiştir.

Anahtar kelimeler: kararlılık-katsayı davranış spektrumu, birinci katlı tek serbestlik derecesi sistemi, kuvvete dayalı tasarım, çok katlı binalar.

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Chapter 1

INTRODUCTION

Ground vibrations occur during an earthquake and these vibrations cause forces and deformations on structures. If the structure can not bear these seismic forces, the building may eventually collapse. As observed from the large earthquakes in the past (the 1995 Kobe, the 1999 Kocaeli, the 1999 Chi-Chi, etc.), in case of building failures big losses occur in terms of human casualties and properties. For this reason, seismic codes are created in order to prevent the structure from collapsing during an earthquake. Although these codes differ from country to country, their use in seismically active regions is mandatory. When designing a structure, priority is given to life safety and to ensure this, excessive damage to the building should be prevented. However, ensuring zero damage to a building is very costly and uneconomical. Therefore, in moderate to severe earthquakes, buildings are allowed to be slightly damaged.

Conventional method used to satisfy seismic building codes is forced-based design due to its simplicity. Base shear can be directly calculated through couple of equations. In this method, preliminary design is carried out to estimate fundamental period of vibration. From the response spectrum, spectral acceleration corresponding to the period of the structure is obtained. The spectral acceleration is multiplied with the seismic weight (W) to calculate total lateral seismic force. Then, this force is distributed along the height of the structure to be applied as equivalent static lateral force. The strength of the structure should be able to cope with the applied equivalent lateral forces to prevent structural collapse. These static lateral forces represent the effects of earthquake. Commonly iteration is required as it does not meet the code requirements at the first trial. This is a major handicap as all calculations need to be repeated all over again. Calvi, Priestley, and Kowalsky (2008) specified three main weakness of force-based design. The method assumes an initial stiffness to obtain the structural period, which is considered as weakness of the method. However, stiffness can not be known until the design is completely finished because stiffness depends on element strength. Another weakness of the system is distributing seismic forces among elements according to their initial stiffness, this assumption is not correct for certain structures as it is not proper to assume that the different elements can be forced to yield at the same time. The last weakness is the assumption that yield-reduction factors are the same for all type of structures and materials which is not correct. Due to these shortcomings, in 2000 Priestley and Kowalsky developed another commonly used design method which is "Direct Displacement Based Design" (DDBD). Although, forced-based design has some deficiencies, the methodology is widely used in seismic codes. Because of this reason, in this thesis this method is used as conventional method for structural design and analysis.

DDBD method determines the design performance of the structures based on their displacement limits. For this reason, during the design process key paramater is displacement. Calvi, Priestley, and Kowalsky (2008) define the main difference of DDBD from force-based design method as DDBD design the structure by peak displacement response of a single degree of freedom (SDOF) system, rather than by its initial stiffness. The methodology can be briefly summarized as follows (Ye, Xiao, & Hu, 2019); an equivalent SDOF system is used to represent the multi storey building by equivalent mass m_e, effective height h_e and target displacement Δ_d are used to represent the equivalent system. When design-based earthquakes are applied, the original MDOF system can behave nonlinearly. To take this into account, SDOF system is linearized by the effective stiffness, k_e. The effective viscous damping ratio, ζ_e can be estimated by use of existing ductility-damping diagrams. Then, effective period $T_{e,}$ which corresponds to target displacement is identified from displacement response spectrum. Effective stiffness is obtained based on effective period and target displacement which is used to calculate the design base shear. Finally, the base shear is distributed to each story as lateral forces.

Nevertheless, both forced-based design and DDBD do not consider the effect created by the action of gravity loads when a structure has a lateral displacement, which is named as P-delta effect (Adam, Ibarra, & Krawinkler, 2004; Rahimi & Estekanchi, 2015). P-delta effect causes a reduction in lateral resistance of a structure. Under severe dynamic excitations, load carrying capacity of a structure may be partially or completely lost due to this effect (Gupta & Krawinkler, 2000; Jäger & Adam, 2013; Adam & Ibarra, 2014). For elastic structures under static forces, P-delta effect elongates the vibration period of the building, as a result stiffness is reduced. Nevertheless, P-delta effect becomes complicated for inelastic response, and under dynamic loading. (Mousavi & Şensoy, 2019). Stability Coefficient (SC) is used to measure P-delta effect. In the past constant value was being used for SC but Aydınoğlu and Fahjan (2003) in their studies showed that SC actually depends on the period. Therefore, giving a constant value to SC causes false results. Moreover, Kalkan and Graizer (2007) stated that when forming a response spectrum for P-delta affected systems if the height of pendulum is not kept constant, period is not a sufficient measure. Instead of plotting spectral information against period of vibration, Mousavi and Şensoy (2019) by using this dependence on period feature of SC they plotted spectral information against SC, and they have named their method "stability coefficient response spectrum" (SCRS). To be able to use the spectrum in MDOF systems, they formed a new system which is called "first-storey-single-degree-of-freedom" (FSSDOF) system. First-storey can be used to represent SDOF system since it has a lot of significant features like:

• The total seismic load of the building is equal to the first-storey axial load,

• The base-shear is equal to the first-storey shear force,

• The height of the first storey seldomly changes during design,

• The first storey dynamic loading is known (Mousavi & Şensoy, 2019).

In FSSDOF system the equation of motion is derived for inverted-pendulum to consider the effect of height as well. By use of SCRS through FSSDOF approach safety against P-delta effect is ensured as it is inherited within the equation of motion.

In this thesis, two different structures, five and nine storey buildings, are designed by the conventional method. For each structure design different yield strength reduction factors (R_y=4 and R_y=8) are used. After the design, base shear obtained from the conventional response spectrum compared with SCRS through FSSDOF approach.

After the introduction given in this chapter, the methodology is briefly explained in the second chapter. Then, in the third chapter, the forming of response spectra is determined. The design of five-storey and nine-storey buildings are given in the fourth chapter. In the fifth chapter, base shears obtained from SCRS and conventional response spectrum are compared. Finally, a general conclusion is made to sum up the main points and results of the whole thesis.

Chapter 2

THE METHODOLOGY

In this thesis, Force-Based Design (FBD) method was used for design and analysis of multi-storey buildings. The new method named “SCRS through the FSSDOF approach” developed by Mousavi and Şensoy (2019) which was used in comparison to see the P-delta effect. Both methods are briefly explained below.

2.1 Conventional Method – Force-Based Design

Force-based design is a static procedure which is used in earthquake resistant design of structures. This method is more commonly used for seismic design codes. The underlying idea of force-based design contains the specification and attainment of a minimum strength, based on assumptions like initial stiffness, design earthquake intensity and ductile capacity of the structure, using force reduction factor (Priestley & Pettinga , 2005). The method, by taking into the applied horizontal loads and vertical forces arising from the weight of the structure itself, examines the stability and the deformation of the structure, and the stress and the strain capacity of each structural member. The procedure is based on Equivalent Static Lateral Force methodology. The applied static loads on a structure are used to represent earthquake induced dynamic loads. Concentrated lateral forces due to dynamic loading act where the concentration of mass at each floor exists. In addition, concentrated lateral forces generally follow the fundamental mode of the structure. In other words, the force is larger as the elevations is higher as seen in Fig. 2.1.

So, the maximum lateral displacements and lateral forces are expected at the top storey of a building (ElAttar, Zaghw, & Elansary, 2014).

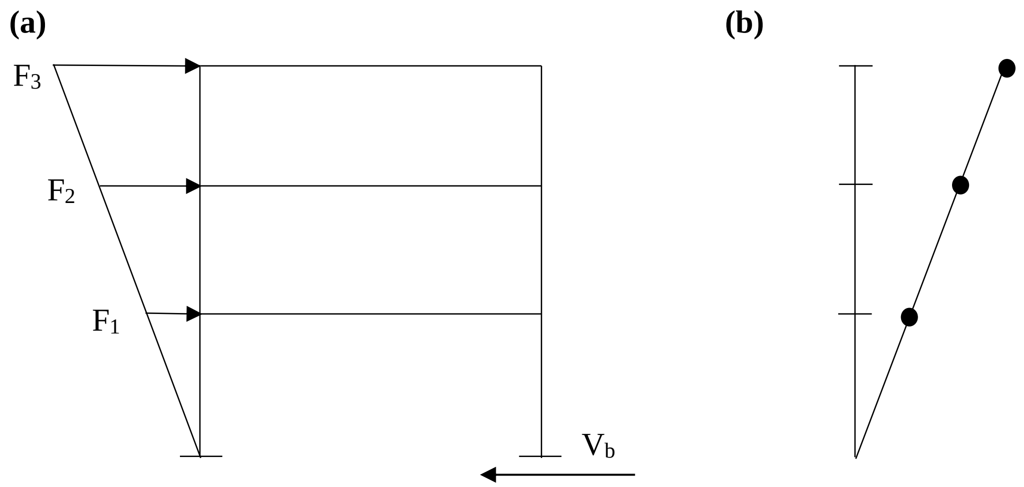

Figure 2.1: (a) Illustration of equivalent static lateral force method, and (b) Illustration of fundamental mode shape

The main objective of this procedure is to determine a base shear corresponding to a response spectrum. Base shear force is estimated considering parameters like importance level of the structure, load carrying system of the structure, and determined effective ground acceleration based on earthquake zone. TS500 determines the base shear force according to the following formula;

$$V_b = \frac{W\,A(T_1)}{R\,T_1} \tag{1}$$

where

V_b = Base shear force, total equivalent earthquake load acting on the building used in Equivalent Earthquake Load Method,

W = The weight of the structure obtained by using the live load participation factor,

A(T) = Spectral acceleration coefficient,

T = The first natural vibration period of the building,

R = Earthquake load reduction coefficient.

Here, yield deformation spectra were used which is explained in more detail in Chapter 3. Therefore, the base shear force is calculated by multiplying the seismic weight of the building directly by the spectral acceleration rather than calculating it by the parameters mentioned above.

$$V_b = S_a(T_1).W \qquad (2)$$

where

$S_a\,(T_1)$ = Spectral acceleration coefficient corresponding to the period of the structure.

After the estimation of the base shear, equivalent lateral forces are spread along the height of the structure. Distribution of base shear force according to Turkish Standard, TS500 is calculated as shown below;

$$F_i = (V_b - \Delta F_N)\frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j} \qquad (3)$$

and

$$\Delta F_N = 0.0075\, N\, V_b \qquad (4)$$

where

F_i = Equivalent lateral load acting on floor i,

H_i = The storey height of the floor i of the building,

w_i = The weight of the floor level i calculated using the live load participation factor,

ΔF_N = Additional equivalent earthquake load affecting the top floor of the structure,

N = The total number of floors from the foundation top of the building.

This method requires preliminary design at the beginning of the design process to obtain initial stiffness of the building. Since a realistic design cannot be made without knowing the equivalent lateral load, iteration is required. The static lateral load is recalculated each time as it changes depending on the period and weight of the building. The assumptions made when using this method are;

• The structure is rigid,

• The first mode dominates the structural response,

• The building is perfectly fixed to the foundation,

• All joints of the building have same acceleration during an earthquake,

• Dominant effect of the earthquake is the horizontal forces of different magnitudes along the height of the building.

2.2 New Method – Stability Coefficient Response Spectra (SCRS) through the First Storey Single Degree of Freedom (FSSDOF) System

Stability Coefficient Response Spectra (SCRS) through the First Storey Single Degree of Freedom (FSSDOF) system is a fairly new method developed by Mousavi and Şensoy (2019). This approach is mainly developed to take P-delta effect into consideration while designing a structure. Seismic forces cause the structure to deform in horizontal direction, while the gravity loads are still acting. On account of this, secondary moments appear in the structure which is equal to the constant vertical load P, times lateral displacement of the structure Δ. In other words, gravity loads reduce the lateral stiffness of buildings. However, the main concern with P-delta effect is not the reduction in lateral stiffness or strength of the building. If the ground acceleration motion is severe enough, the structure might be brought into the negative post-yield stiffness. If the effective stiffness at the largest displacement remains positive, P-delta effect is not very important, but if the stiffness becomes negative then, there is a possibility of collapse of the structure (Gupta & Krawinkler, 2000). Moreover, Williamson (2003) stated in his research that earthquakes may also have vertical acceleration and stability effect is important even if axial load is very low. In this method, inverted-pendulum is used as single degree of freedom (SDOF) system due to the mentioned concerns. By this way, P-delta effect is placed into the equation of motion.

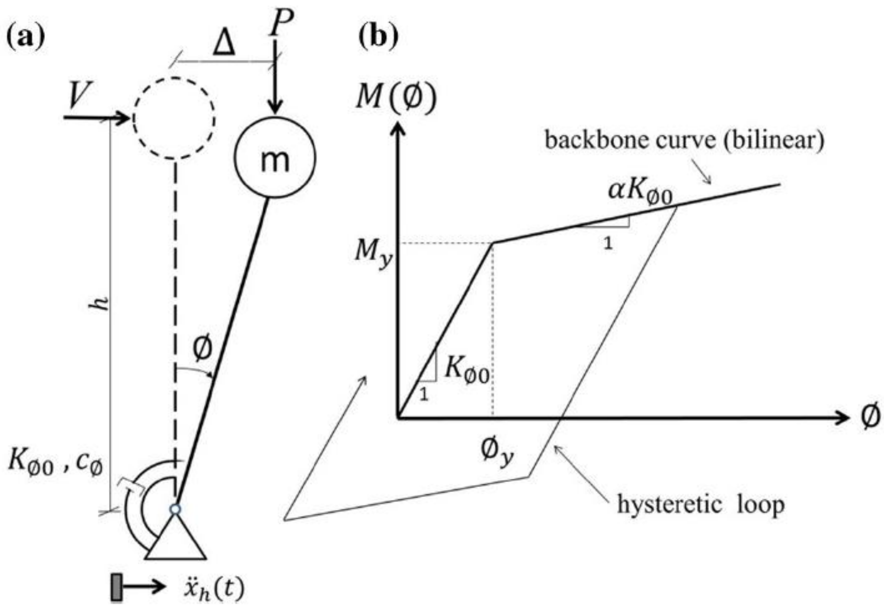

Figure 2.2: (a) The inverted pendulum model, and (b) Bilinear hysteretic behaviour (Mousavi & Şensoy, 2019)

The equation of motion of an inverted pendulum in which P-delta effect is taken into account and base excitation is applied can be expressed as (Hjelmstad & Williamson, 1998; Adam & Jäger, 2012);

$$mh^2\ddot{\phi} + c_\phi\dot{\phi} + M(s) = -mh\ddot{x}_h \qquad (5)$$

and

$$M(s) = M(\phi) - \theta K_{\phi 0}\phi \qquad (6)$$

where

m = lumped mass,

P = vertical load,

h = height of the pendulum,

$M(\phi)$ = inelastic spring resistance,

ϕ = the angular position of the mass,

$\ddot{x}_h$ = horizontal ground acceleration,

$M(\phi)$ = inelastic spring resistance,

$K_{\phi 0}$ = initial rotational stiffness of the spring,

θ = stability coefficient.

The equation of stability coefficient of the elastic system under the vertical load P, is defined as;

$$\theta = \frac{P.\, h}{K_{\phi 0}} \tag{7}$$

Aydınoğlu and Fahjan (2003) proved that SC is time dependent for a pendulum with constant height and using it as a constant value is inaccurate. This idea can be supported by the following equation, which is derived from the Equation (7).

$$\theta = \frac{g/h}{\omega 0^2} = \frac{\omega_G{}^2}{\omega 0^2} \tag{8}$$

where

ω_0 = initial frequency and is equal to $K_{\phi 0}$ / (mh^2).

The dependence of SC on the period of the structure can be used to form stability coefficient response spectra. Putting a limit to the SC at each storey of a multi storey building is equivalent to putting a limit to vibration period of each level (Mousavi & Şensoy, 2019). By this way, necessary lateral stiffness for each level can be obtained. However, to be able to achieve this, input motions for all elevations should be known. In fact, input motions are normally known only at the first level of the building. Mousavi and Sensoy (2019) developed a new method by using this and other features of the first storey such as; axial load on the base level is the total load of the whole building calculated by using live load participation factor, the acting shear force is equal to base shear and the height does not change very often. The new method is called First Storey Single Degree of Freedom System (FSSDOF). In this method, the first storey is used to represent single degree of freedom system so that all features mentioned above can be used. It should be pointed out that the aim of this method is to determine the required lateral stiffness for the first storey. The method cannot be used to represent the dynamic features of the multi storey building. In order to use this method effectively, SCRS should be formed, which is explained in the next chapter.

Chapter 3

RESPONSE SPECTRA

The actual time history record is required for seismic design and analysis of a structure to be built at a certain location. However, it is unlikely to achieve such records for every building field. Moreover, performing a seismic analysis of a structure based on the peak ground acceleration only is not very sufficient, because each earthquake is different and the response of the structure relies upon the frequency of the ground motion and its dynamic characteristics. Therefore, response spectrum is very convenient tool for structural seismic analysis. Response spectrum is actually derived from time history analysis. But in this method, instead of considering the response of the structure at each time instance, maximum response of it is considered. The maximum response is obtained for each period. In other words, the response spectrum is a plot of maximum response of all SDOF systems under the ground motion excitation for given damping ratio.

To be able to form a response spectrum, the dynamic response of SDOF system should be numerically evaluated. Generally, analytical solution of the equation of motion for a SDOF system can not be obtained if the dynamic loading varies arbitrarily (Chopra, 2011). This problem can be overcome by numerical time-stepping methods to solve various types of differential equations. For this purpose, the Newmark's Method (1959) is adopted which depends on the equations given below;

$$\dot{u}_{i+1} = \dot{u}_i + [(1-\gamma)\Delta t]\ \dot{u}_i + (\gamma\Delta t)\ \ddot{u}_{i+1} \qquad (9)$$

$$u_{i+1} = u_i + (\Delta t)\dot{u}_i + [(0.5-\beta)(\Delta t)^2]\ddot{u}_i + [\beta(\Delta t)^2]\ddot{u}_{i+1} \quad (10)$$

β and γ are parameters which are used to define the change of acceleration over a time step and to determine the method's stability and accuracy properties. Usually γ is chosen as $\frac{1}{2}$ and β varies between $\frac{1}{6}$ to $\frac{1}{4}$.

Safkan (2018) selected ground motions by considering hazard deaggregation study results using the parameters like magnitude, distance and the target intensity measure which were obtained from the seismic hazard assessment study of Cagnan and Tanircan (2010). In this thesis, these ground motions were used as input motion to form response spectra especially for Cyprus. List of datasets used as input motion are given in Table 3.1.

Table 3.1: Ground motion dataset used as input motion for the response spectra (Safkan, 2018)

#	Name	Year	Location	Mag.	RJB	VS	Fault
1	PEER531	1986	Puerta La Cruz	6.1	67.5	442	Reverse
2	PEER686	1987	Whitter	5.9	40.9	390	Reverse
3	PEER	1989	Loma Prieta	6.9	79	623	Reverse
4	PEER208	2002	Alaska	6.7	106	341	Strike Slip
5	PEER016	1975	Oroville	5.7	9.8	590	Normal
6	PEER450	2009	L`Aquila Italy	6.3	60.8	535	Normal
7	PEER112	1995	Kozani, Greece	6.4	72.8	650	Normal
8	PEER026	1980	Sahop Casa	6.3	19.0	242	Strike Slip
9	PEER026	1980	Sahop Casa	6.3	39.1	260	Strike Slip
10	PEER031	1981	Corinth, Greece	6.6	10.3	361	Normal
11	PEER046	1981	Taiwan	5.9	26.4	309	Reverse
12	PEER053	1986	San Jacinto	6.1	30.7	331	Reverse
13	PEER054	1986	Chalfant Valley	6.2	21.6	371	Strike Slip
14	PEER071	1987	Imperial Valley	6.2	17.6	179	Strike Slip
15	PEER330	1999	Chi Chi	6.3	27.6	553	Reverse
16	PEER272	1999	Chi Chi	6.2	76.3	247	Strike Slip
17	PEER441	1983	Borah Peak	6.9	80	324	Normal

3.1 Conventional Response Spectra

3.1.1 Numerical Evaluation of Dynamic Response

In the conventional method, mass-spring-damper (MSD) system is used to represent SDOF systems. The illustration of the model is shown in Fig 3.1. The mass is allowed to move in only one direction, therefore, the system has only one degree of freedom.

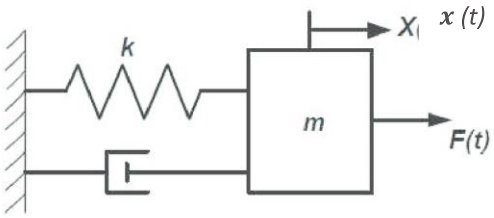

Figure 3.1: Mass-spring-damper model

The equation of motion for the system is;

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = F(t) \tag{11}$$

Under the ground motion excitation F(t) is defined as;

F(t) = $-m\ddot{x}_g(t)$

where

m = mass of the structure,

c = damping coefficient,

k = stiffness,

x = displacement,

$\dot{x}$ = velocity,

$\ddot{x}$ = acceleration,

$\ddot{x}_g(t)$ = horizontal ground acceleration.

For the linear systems Newmark's original formula can be modified as shown below;

$$m\ddot{u}_{i+1} + c\dot{u}_{i+1} + ku_{i+1} = p_{i+1} \tag{12}$$

The summary of time-stepping solution of Newmark's method is given in Table 3.2. In here, linear acceleration method is used and to simplify the calculations, initial conditions ($u(0)$, $\dot{u}(0)$, and $\ddot{u}(0)$) are taken as zero. The implementation of the method is carried out in a MATLAB program, this program is given in **Appendix A**.

Table 3.2: Newmark's method for linear systems

Special Cases	Constant average acceleration method; $\gamma = \frac{1}{2}$ and $\beta = \frac{1}{4}$
	Linear acceleration method; $\gamma = \frac{1}{2}$ and $\beta = \frac{1}{6}$

1. *Initial Calculations*

1.1. $\ddot{u}_0 = \dfrac{p_0 - c\dot{u}_0 - ku_0}{m}$

1.2. Select Δt

1.3. $a_1 = \dfrac{1}{\beta(\Delta t)^2} m + \dfrac{\gamma}{\beta \Delta t} c,$

$a_2 = \dfrac{1}{\beta \Delta t} m + (\dfrac{\gamma}{\beta} - 1)\, c,$

$a_3 = (\dfrac{1}{2\beta} - 1)\, m + \Delta t\, (\dfrac{\gamma}{\beta} - 1)\, c,$

1.4. $\hat{k} = k + a_1$

2. *Calculations for each time step, $i = 0, 1, 2, \ldots$*

2.1. $\hat{p}_{i+1} = p_{i+1} + a_1 u_i + a_2 \dot{u}_i + a_3 \ddot{u}_i$

2.2. $u_{i+1} = \dfrac{\hat{p}_{i+1}}{\hat{k}}$

2.3. $\dot{u}_{i+1} = \dfrac{\gamma}{\beta \Delta t}\,(u_{i+1} - u_i) + (1 - \dfrac{\gamma}{\beta})\dot{u}_i + \Delta t\, (1 - \dfrac{\gamma}{\beta})\ddot{u}_i$

2.4. $\ddot{u}_{i+1} = \dfrac{1}{\beta\,(\Delta t)^2}\,(u_{i+1} - u_i) - \dfrac{1}{\beta \Delta t}\dot{u}_i + (\dfrac{1}{2\beta} - 1)\,\ddot{u}_i$

3. *Repeat the process for the next time step.*

Change i by $i + 1$ and carry out steps 2.1 to 2.4 for the next time step.

3.1.2 Forming Response Spectra

The determination of the yield strength (f_y) and yield deformation (u_y) of the system is important for design purposes for limiting the ductility demand to a particular value. The force-deformation relation for a structure during the initial loading can be idealized by an elastic-perfectly plastic (elastoplastic) relation. In Fig 3.2, typical loading, unloading, and reloading cycle for an elastoplastic system is shown.

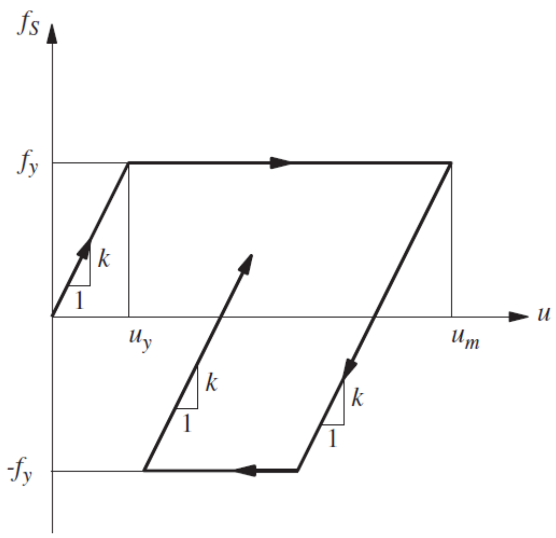

Figure 3.2: Elastoplastic force–deformation relation (Chopra, 2011)

At the initial loading, the system is elastic with stiffness k, until it reaches the yield strength. At this point yielding starts, and corresponding deformation is known as yield deformation. Force is assumed to be constant and stiffness is kept zero during the yielding. The yield strength value is the same in both negative and positive directions. Unloading is parallel to the loading and reloading path. Newmark and Veletsos (1960) developed a response spectrum for elastoplastic systems which can directly determine yield strength or yield deformation. For this method, the yield strength reduction factor R_y and the peak linear deformation u_0 should be known. During the design, yield strength reduction factor R_y is selected by the designer and it is defined by;

$$R_y = \frac{f_0}{f_y} = \frac{u_0}{u_y} \tag{13}$$

where

f_o = peak earthquake induced resisting force in the corresponding linear system ($f_o = ku_0$),

f_y = yield strength ($f_y = ku_y$).

After the evaluation of the elastic system as stated in section 3.1.1, peak deformation in the corresponding linear system u_0 can be determined. Yield deformation can be calculated by dividing u_0 by R_y. In order to form response spectra, yield deformation should be calculated for each period. MATLAB code created for the conventional response spectrum is included in the **Appendix B.** Response Spectra are plotted against fundamental vibration period for;

$$D_y = u_y \qquad V_y = \omega_n u_y \qquad A_y = \omega_n^2 u_y \tag{14}$$

where

ω_n = natural frequency of the SDOF system ($\omega_n = \frac{2\pi}{T}$),

D_y = Spectral displacement,

V_y = Spectral velocity,

A_y = Spectral acceleration.

In order to form the conventional response spectrum, elastic response spectrum should be obtained so that peak deformation can be determined for each period. By the yield strength response spectra, yield strength can be obtained directly by;

$$f_y = \frac{A_y}{g} w \tag{15}$$

where

w = weight of the structure.

Conventional response spectra created for Cyprus with 5% damping ratio is shown in Fig 3.3 to 3.6 for R_y = 1,2,4, and 8.

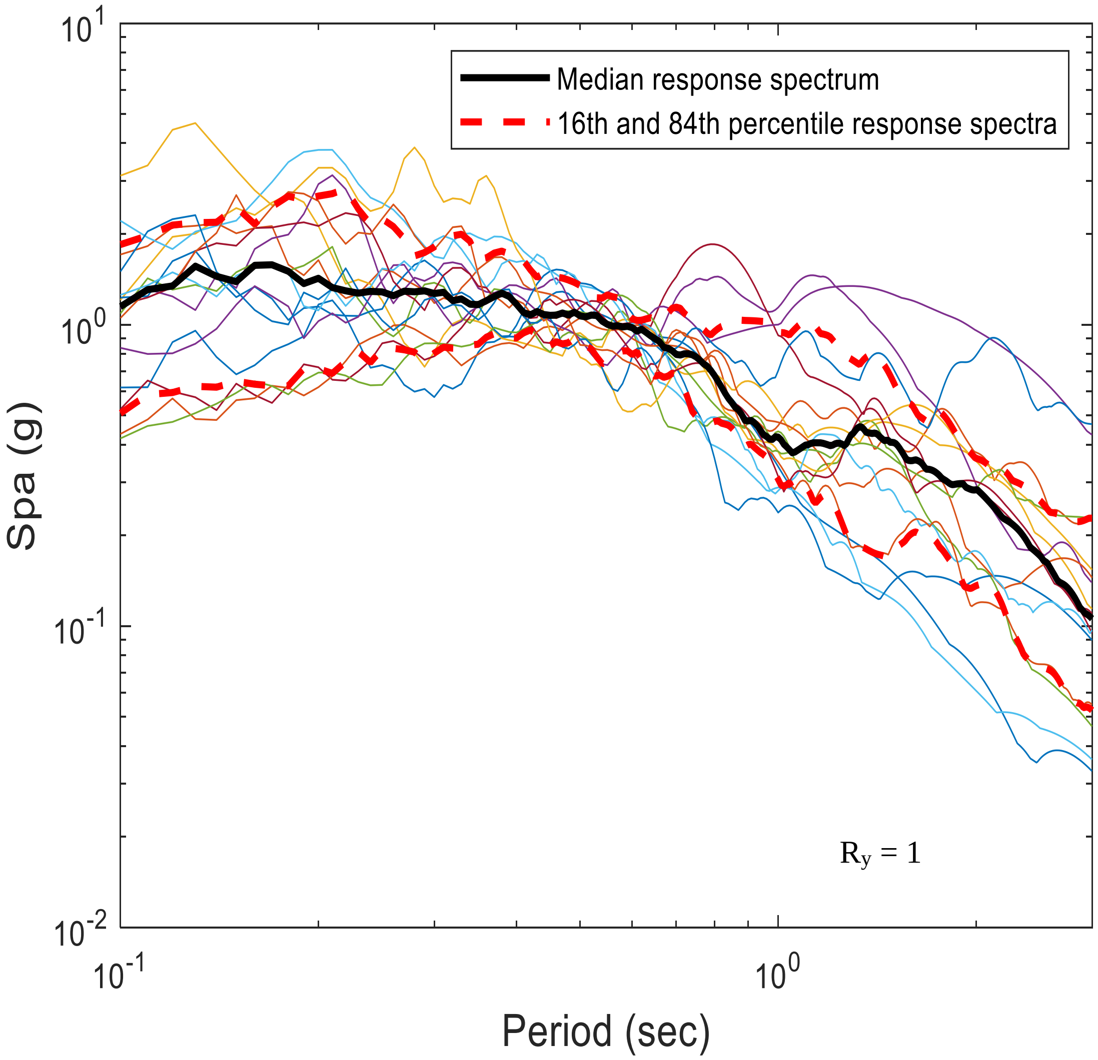

Figure 3.3: Conventional response spectra ($\zeta = 5\%$) for selected ground motions for R_y=1

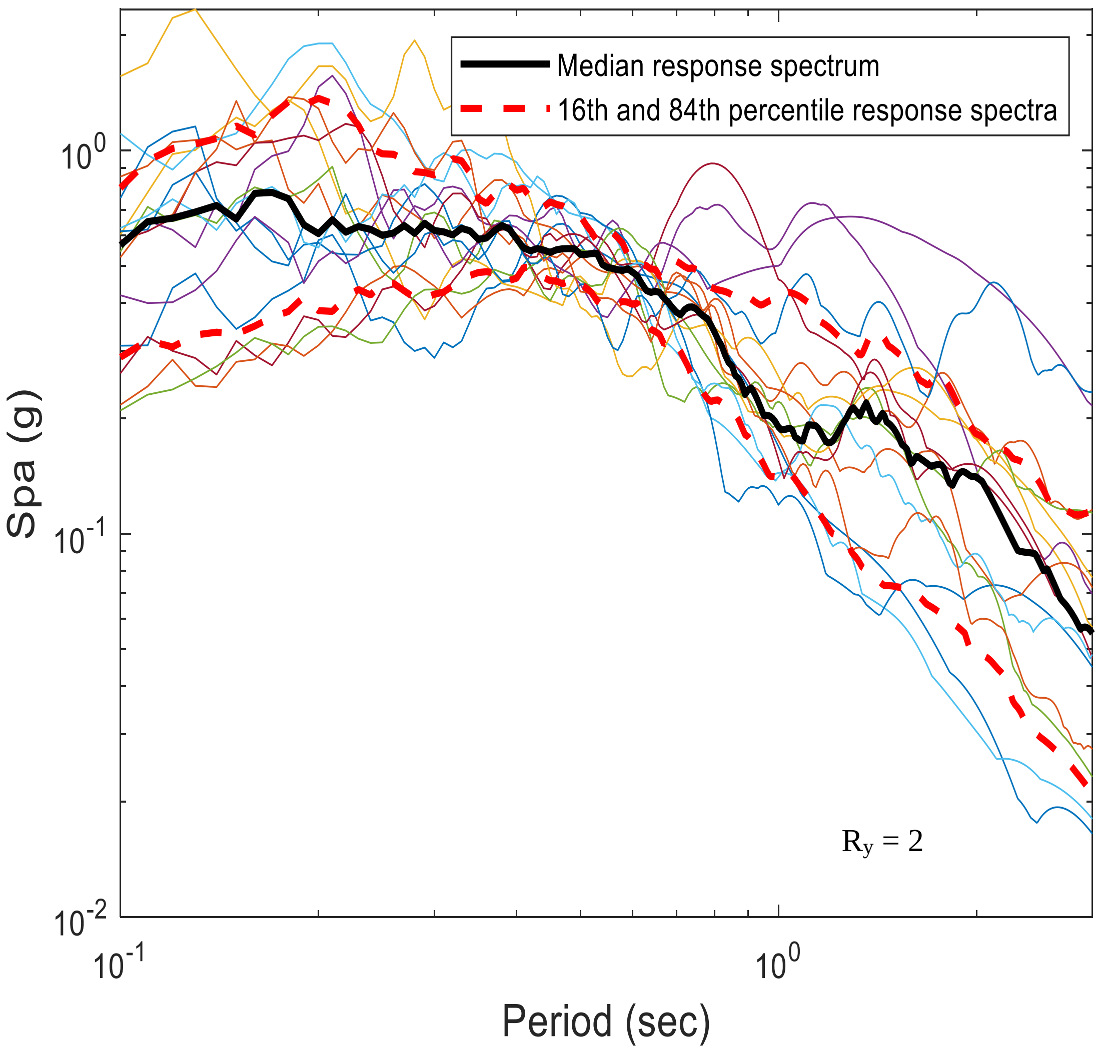

Figure 3.4: Conventional response spectra ($\zeta = 5\%$) for selected ground motions for R_y=2

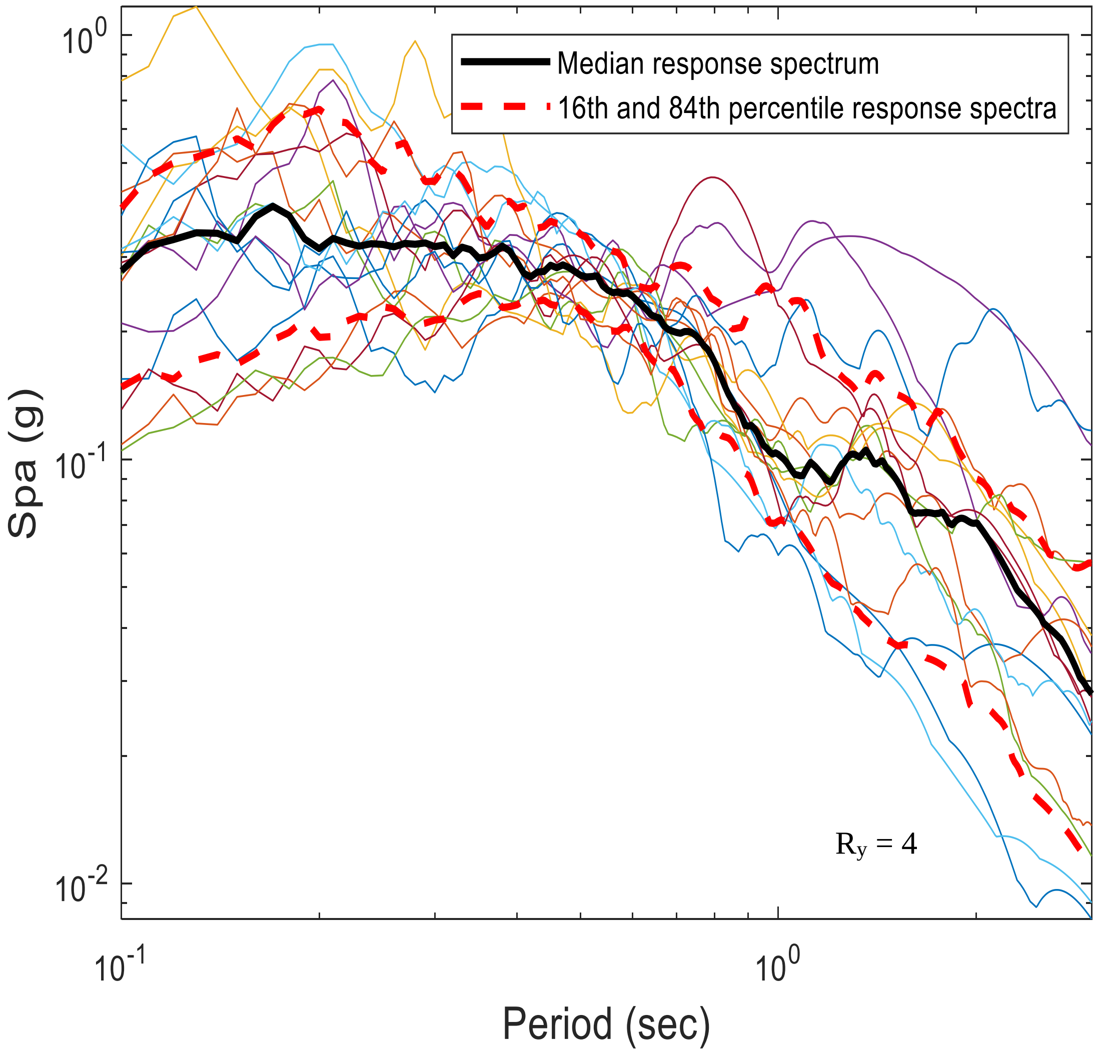

Figure 3.5: Conventional response spectra ($\zeta = 5\%$) for selected ground motions for R_y=4

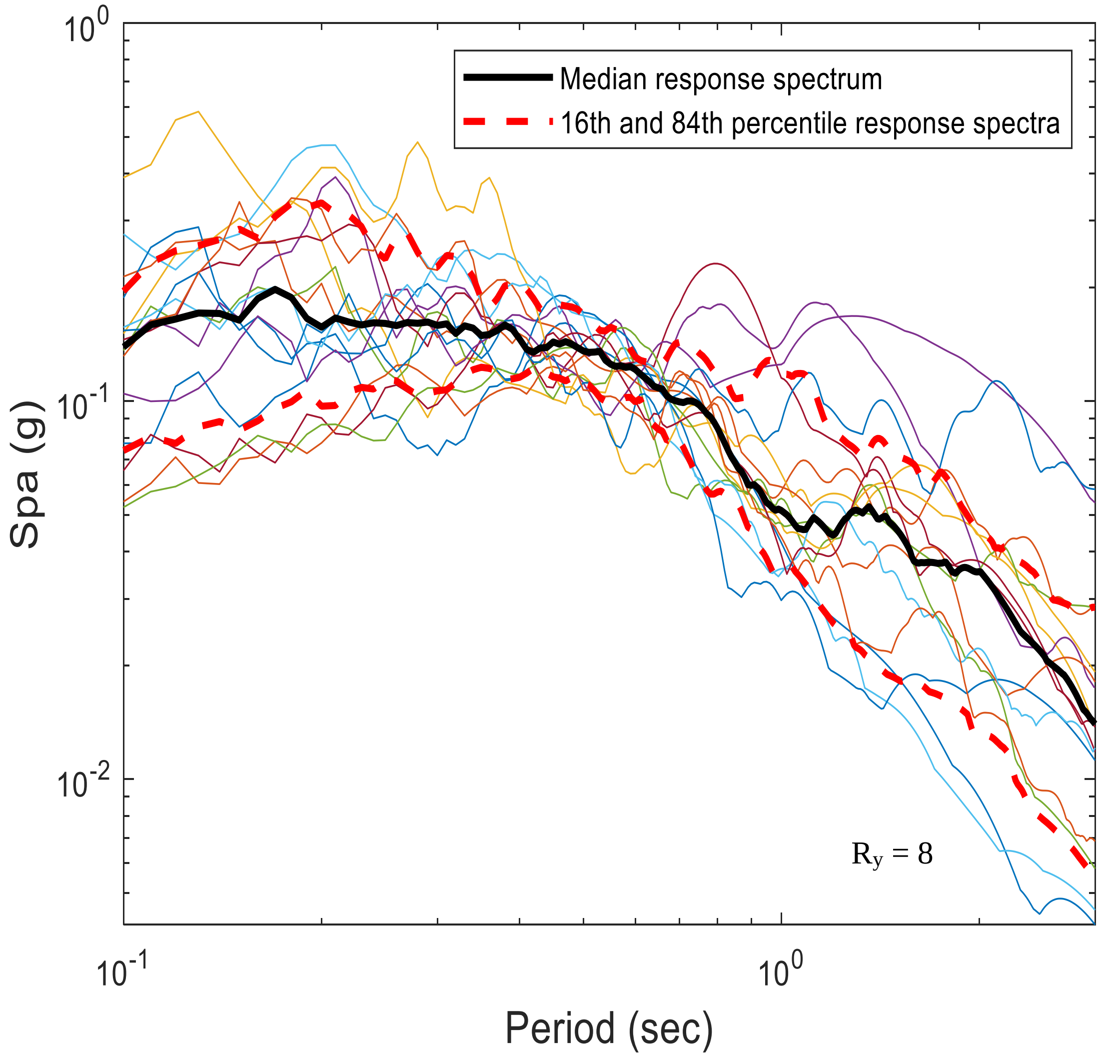

Figure 3.6: Conventional response spectra ($\zeta = 5\%$) for selected ground motions for R_y=8

3.2 New method – Stability Coefficient Response Spectra

3.2.1 Numerical Evaluation of Dynamic Response

Newmark's original formula was required to adapt for the inverted pendulum model to be able to create SCRS. Adaption steps used for Newmark's method are placed in the **Appendix C** and the summary of steps are shown in Table 3.3. It is essential to use height as a constant value in inverted pendulum-based response spectra. Because the period dependent feature of SC can be adapted by use of constant height.

Table 3.3: Modified Newmark's method for inverted pendulum model

Special Cases	Constant average acceleration method; $\gamma = \frac{1}{2}$ and $\beta = \frac{1}{4}$
	Linear acceleration method; $\gamma = \frac{1}{2}$ and $\beta = \frac{1}{6}$
1.	*Initial Calculations*
	1.1. Select Δt and $\Delta\emptyset$
	1.2. $a_1 = \frac{1}{\beta(\Delta t)^2} mh^2 + \frac{\gamma}{\beta\Delta t} c_\emptyset,$
	1.3. $a_2 = \frac{1}{\beta\Delta t} mh^2 + (\frac{\gamma}{\beta} - 1)\, c_\emptyset$,
	1.4. $a_3 = (\frac{1}{2\beta} - 1)\, mh^2 + \Delta t\, (\frac{\gamma}{\beta} - 1)\, c_\emptyset$
	1.5. $\hat{k} = K_{\emptyset 0} + a_1$
2.	*Calculations for each time step, i = 0, 1, 2,…*
	2.1. $\hat{p}_{i+1} = p_{i+1} + a_1\emptyset_i + a_2\dot{\emptyset}_i + a_3\ddot{\emptyset}_i$
	2.2. $\emptyset_{i+1} = \frac{\hat{p}_{i+1}}{\hat{k}}$
	2.3. $\dot{\emptyset}_{i+1} = \frac{\gamma}{\beta\Delta t}\, (\emptyset_{i+1} - \emptyset_i) + (1 - \frac{\gamma}{\beta})\dot{\emptyset}_i + \Delta t\, (1 - \frac{\gamma}{\beta})\ddot{\emptyset}_i$
	2.4. $\ddot{\emptyset}_{i+1} = \frac{1}{\beta\, (\Delta t)^2}\, (\emptyset_{i+1} - \emptyset_i)) - \frac{1}{\beta\Delta t}\dot{\emptyset}_i + (\frac{1}{2\beta} - 1)\, \ddot{\emptyset}_i$
3.	*Repeat for the next time step.*
	3.1. Change i by $i + 1$ and carry out steps 2.1 to 2.4 for the next time step.

3.2.2 Forming Response Spectra

Yielding displacements are calculated using the maximum displacements as in the conventional method. The period-dependent feature of SC is used to plot response spectra against SC with a constant height of 3 m which is the first storey height of the designed buildings. Stability coefficient response spectra is illustrated in Fig 3.7 to 3.10 for R_y = 1,2,4, and 8.

When SCRS and conventional response spectra are compared, spectral information is slightly higher in SCRS since it includes P-delta effect as well. The vibration period can not represent P-delta effect unless the height of the pendulum is kept constant (Kalkan & Graizer, 2007).

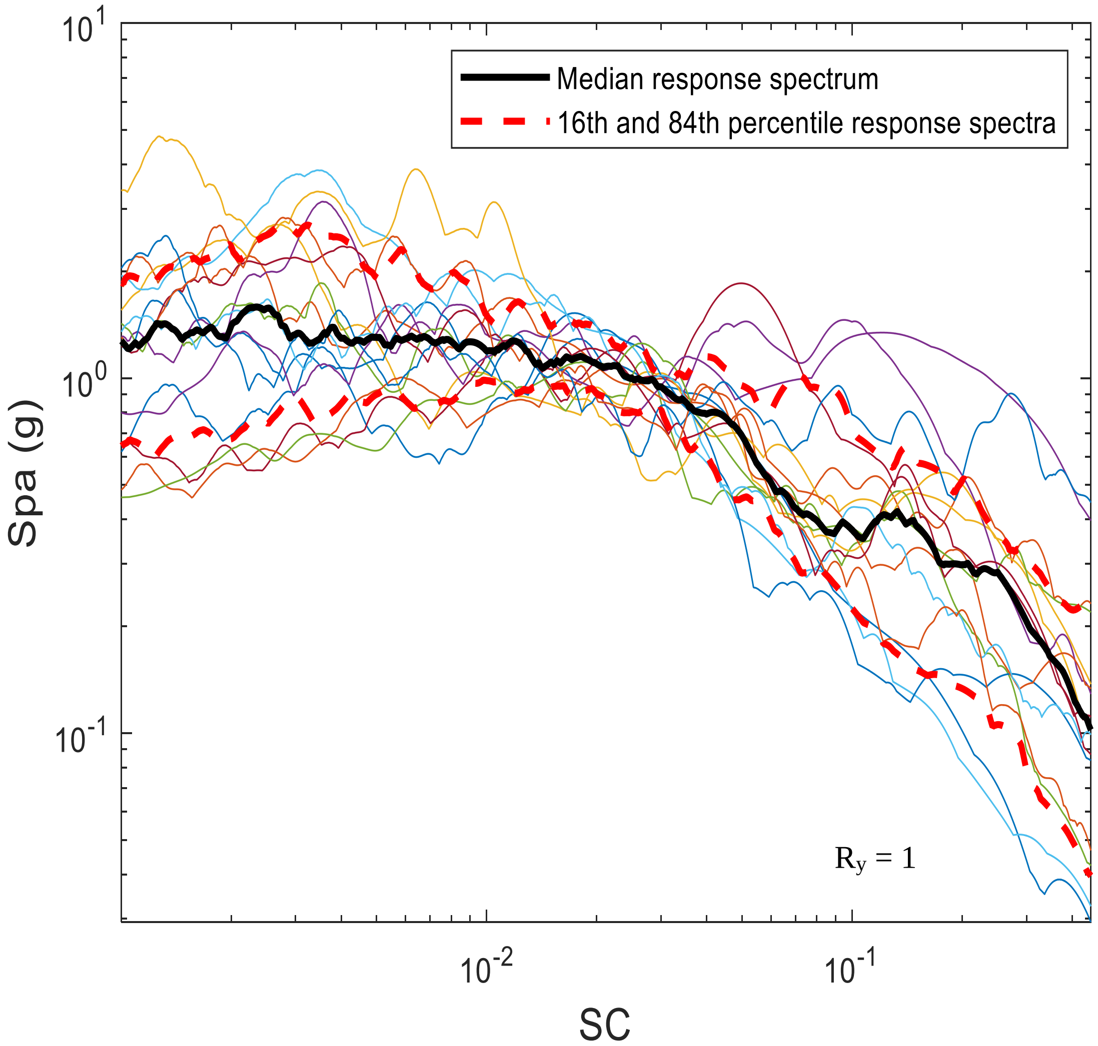

Figure 3.7: Stability coefficient response spectra (ζ = 5%) for selected ground motions for R_y=1

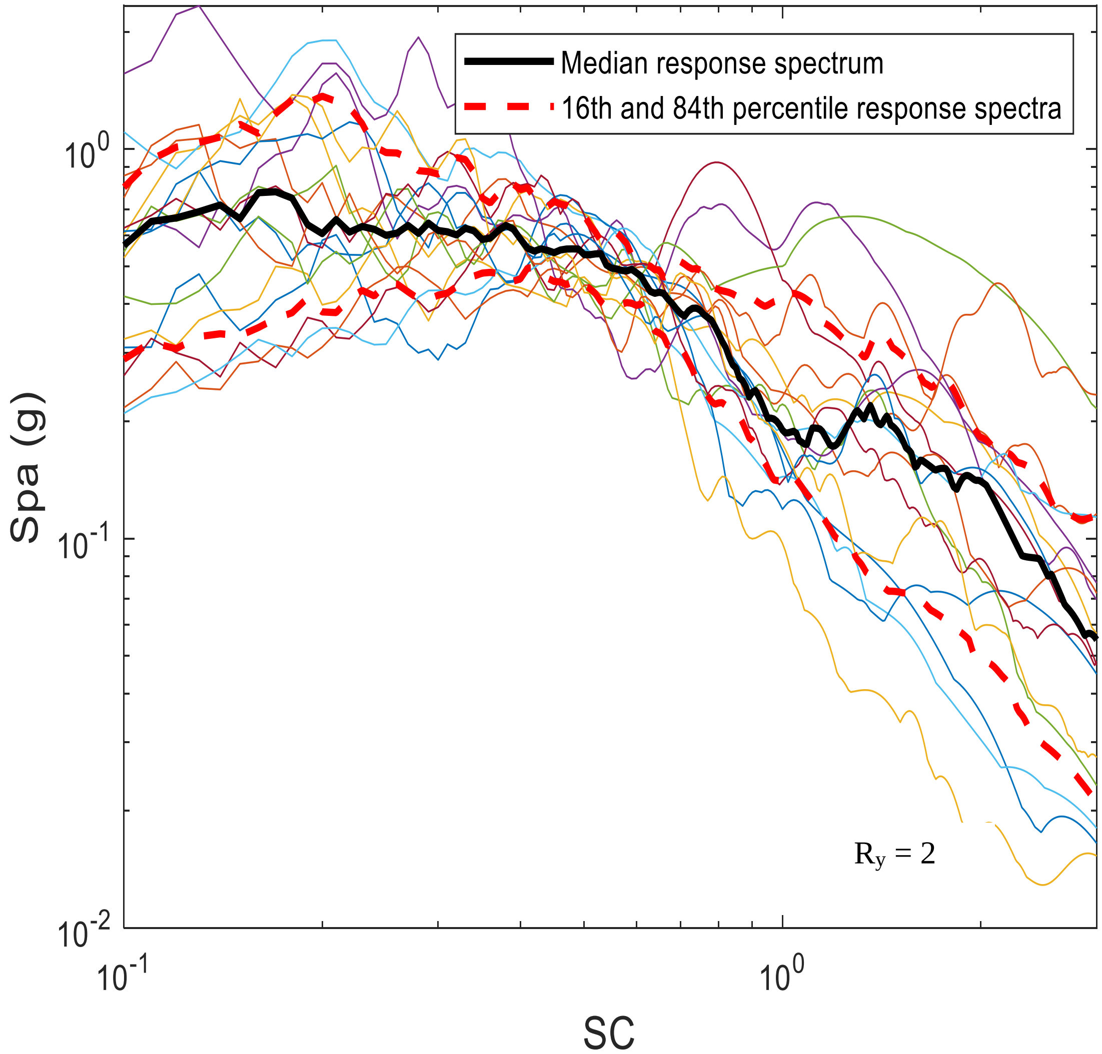

Figure 3.8: Stability coefficient response spectra (ζ = 5%) for selected ground motions for R_y=2

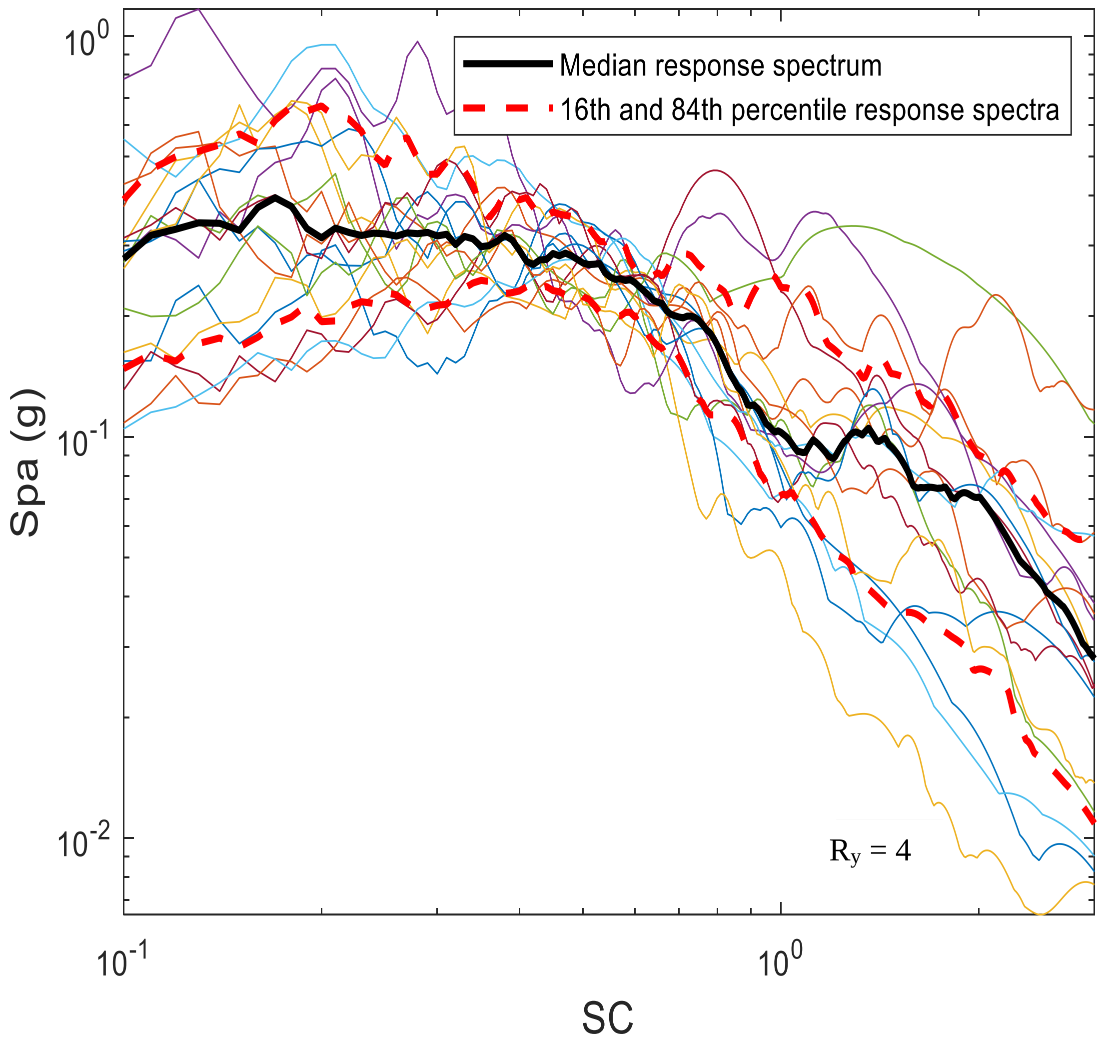

Figure 3.9: Stability coefficient response spectra ($\zeta = 5\%$) for selected ground motions for R_y=4

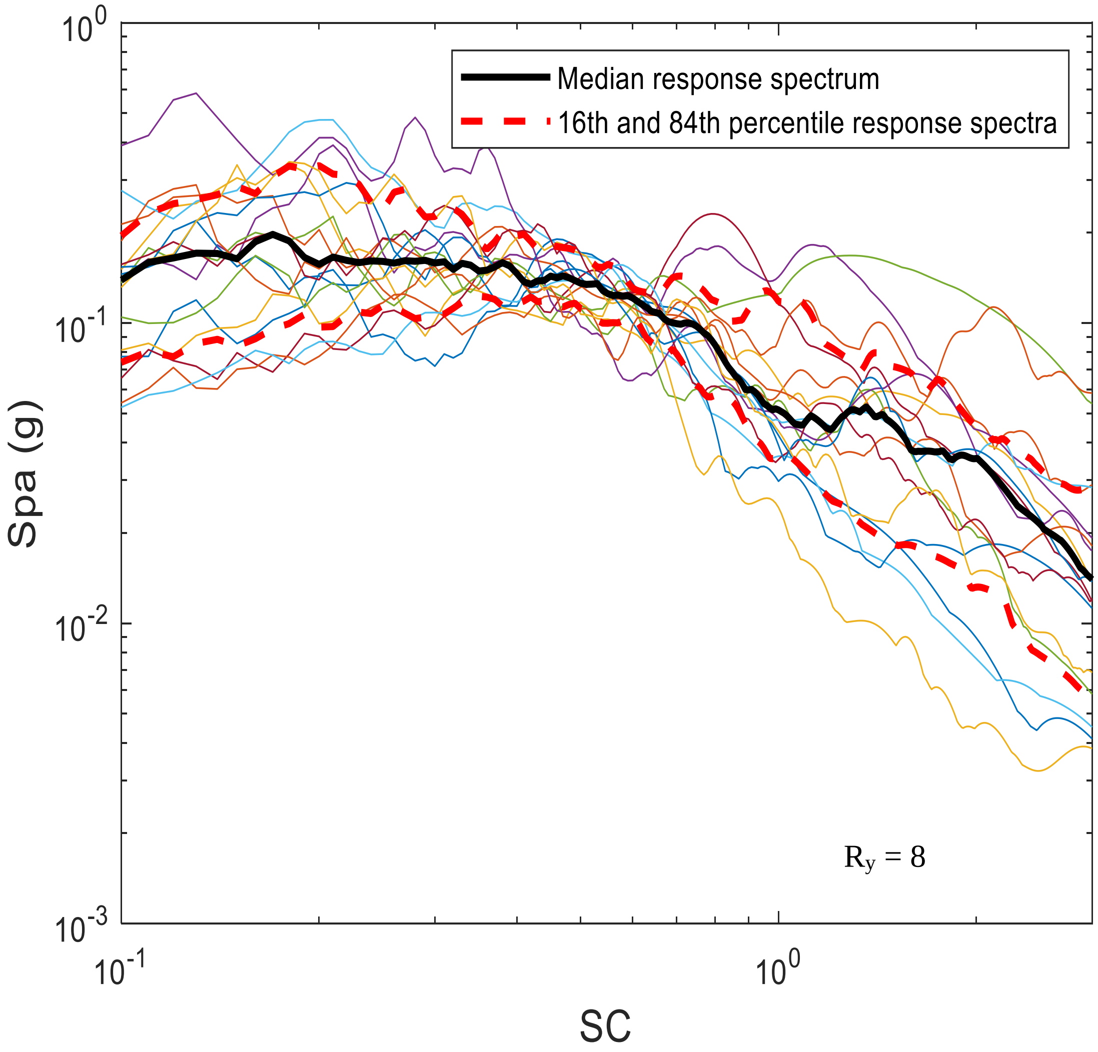

Figure 3.10: Stability coefficient response spectra ($\zeta = 5\%$) for selected ground motions for R_y=8

3.3 Ductility Demand Curves

Ductility Demand can simply be expressed as the ratio of the peak non-linear displacement u_m to the yield displacement u_y. These curves vary considerably according to the ground motion. For this reason, forming the ductility curve by taking the mean of many ground motions is a more reasonable approach. With this approach in Figures 3.11 and 3.12, ductility demand curves were plotted by taking the mean of 17 ground motions. A MATLAB code was written to obtain peak nonlinear displacement and to plot the ductility demand curve is added to the **Appendix D**.

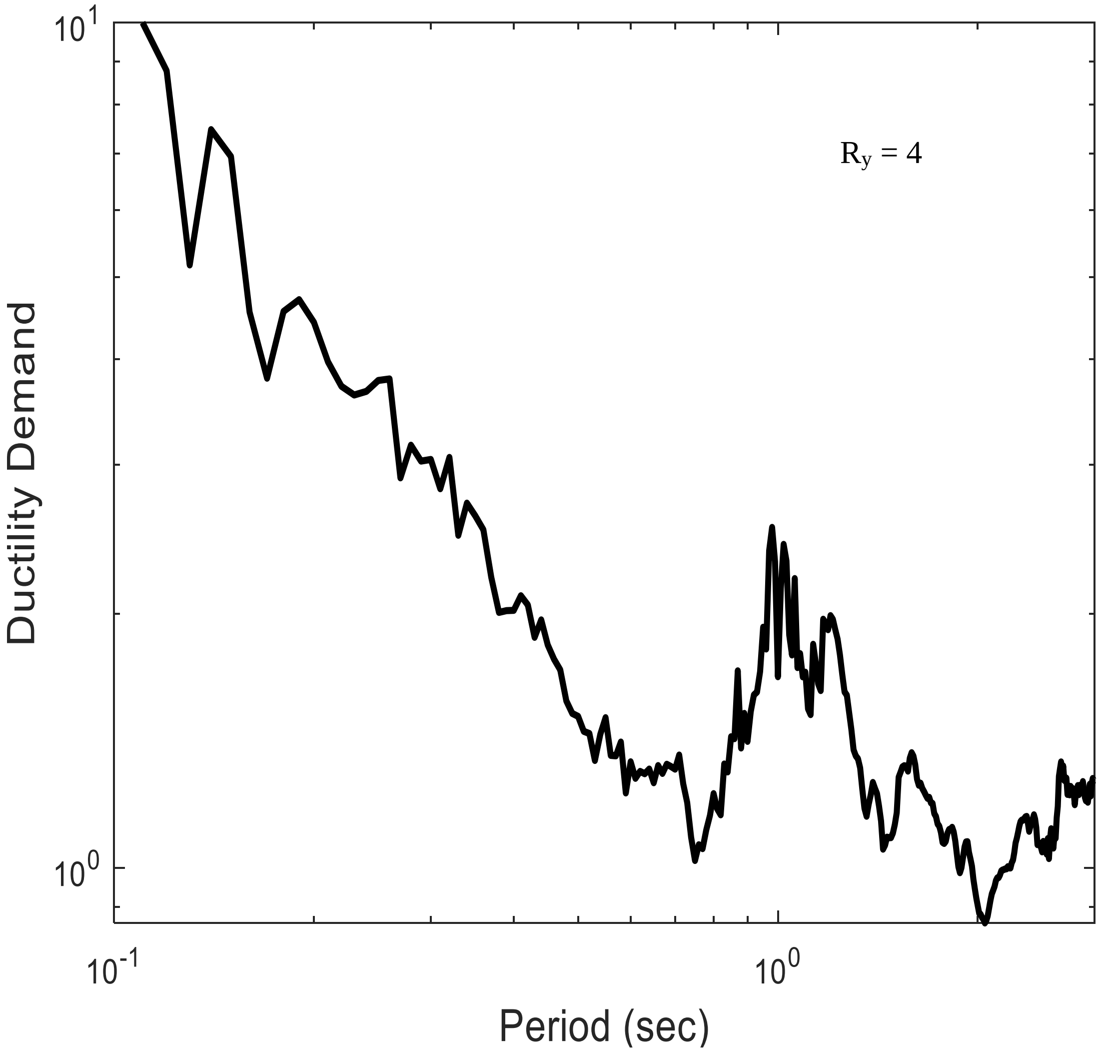

Figure 3.11: Ductility Demand Curve for yield reduction factor four

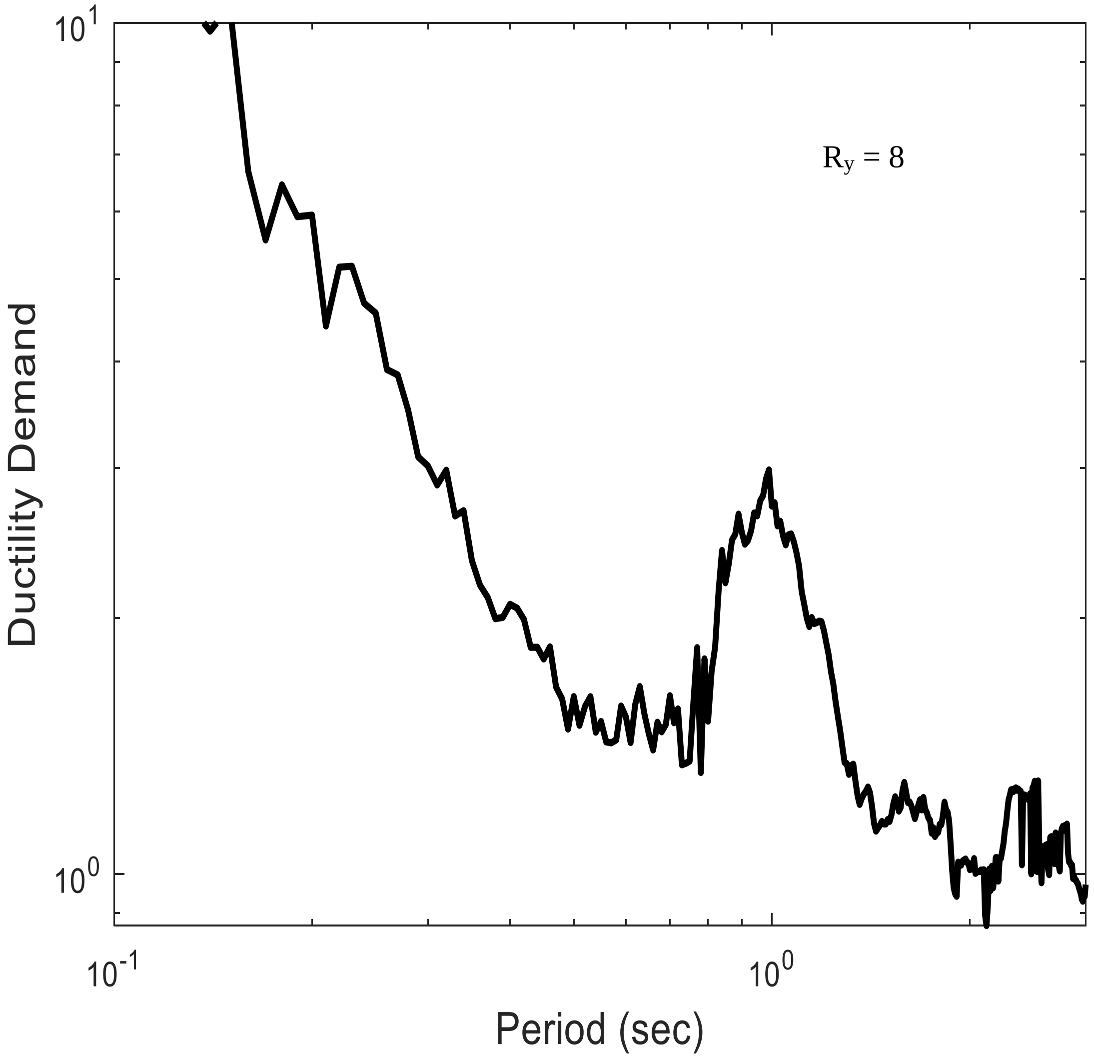

Figure 3.12: Ductility Demand Curve for yield reduction factor eight

Chapter 4

BUILDING DESIGN

A five-storey building and a nine-storey building were designed according to Turkish Standard (TS 500) and Turkish Earthquake Code (2007). These structures were designed and analysed in order to see and compare the P-delta effect in long and short buildings. Concrete grade C25 was used for the two buildings and S420 was used for the reinforcement. In Fig 4.1 and Fig 4.2, five storey and nine storey buildings designed in SAP2000 are illustrated respectively. A typical storey plan is given in Fig 4.3. The middle part was kept empty as it represents the staircase and elevator space. The reason for placing this space in the middle part is to form a regular building. In regular buildings, when lateral force is applied, the structure moves equally so the energy dissipation is even at both side of the structure. Nonetheless, in irregular structures since the energy dissipation is not equal, there is more force on one side. For this reason, there will be more damage in one side and this may even cause collapse of the whole building.

Here, only the design steps and results of a five-storey building with a yield reduction factor of four are included. The load transferred from the stairs, slabs, walls, and beams to the columns is shown in the Table 4.1.

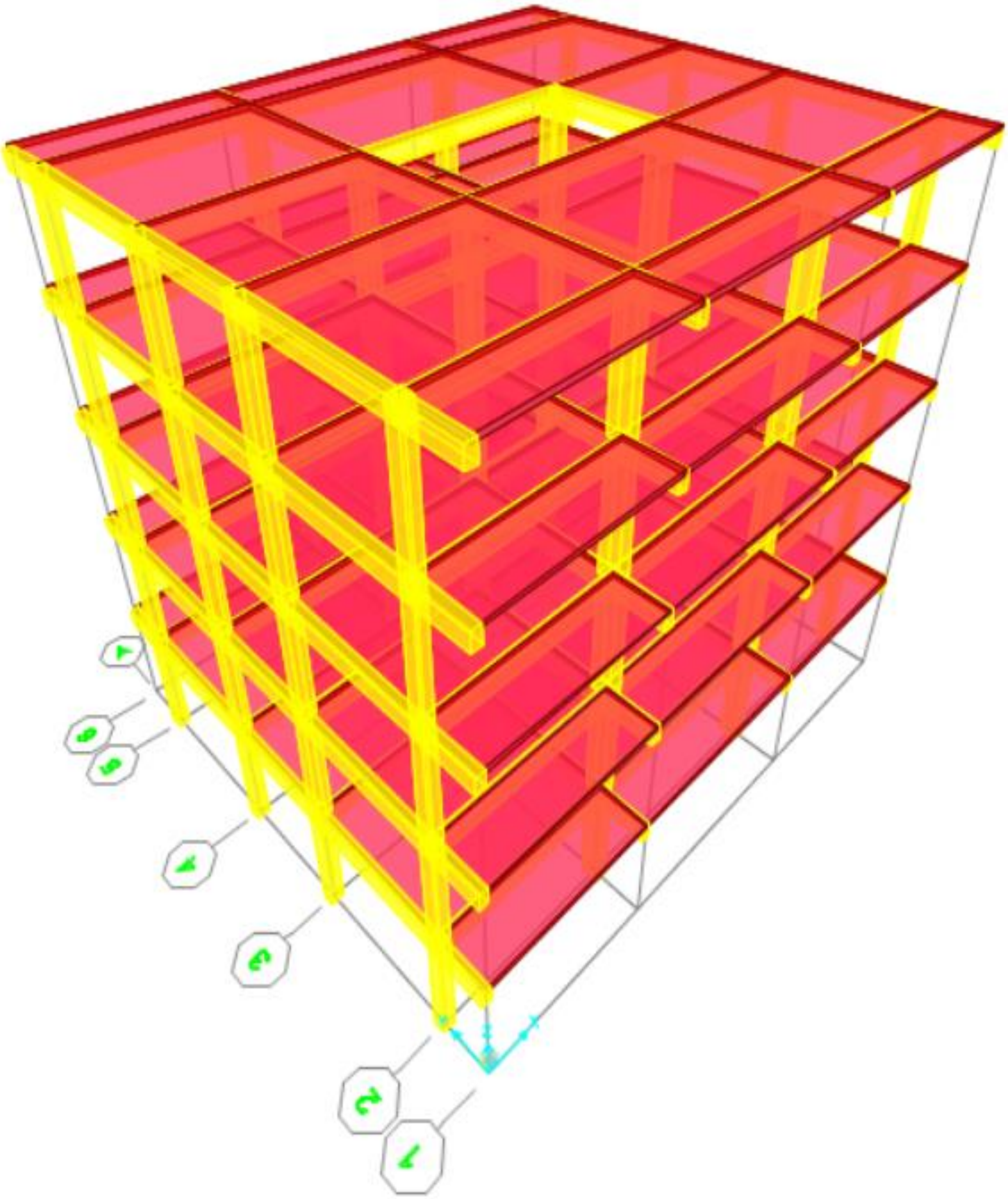

Figure 4.1: Five-storey building modelled in SAP2000

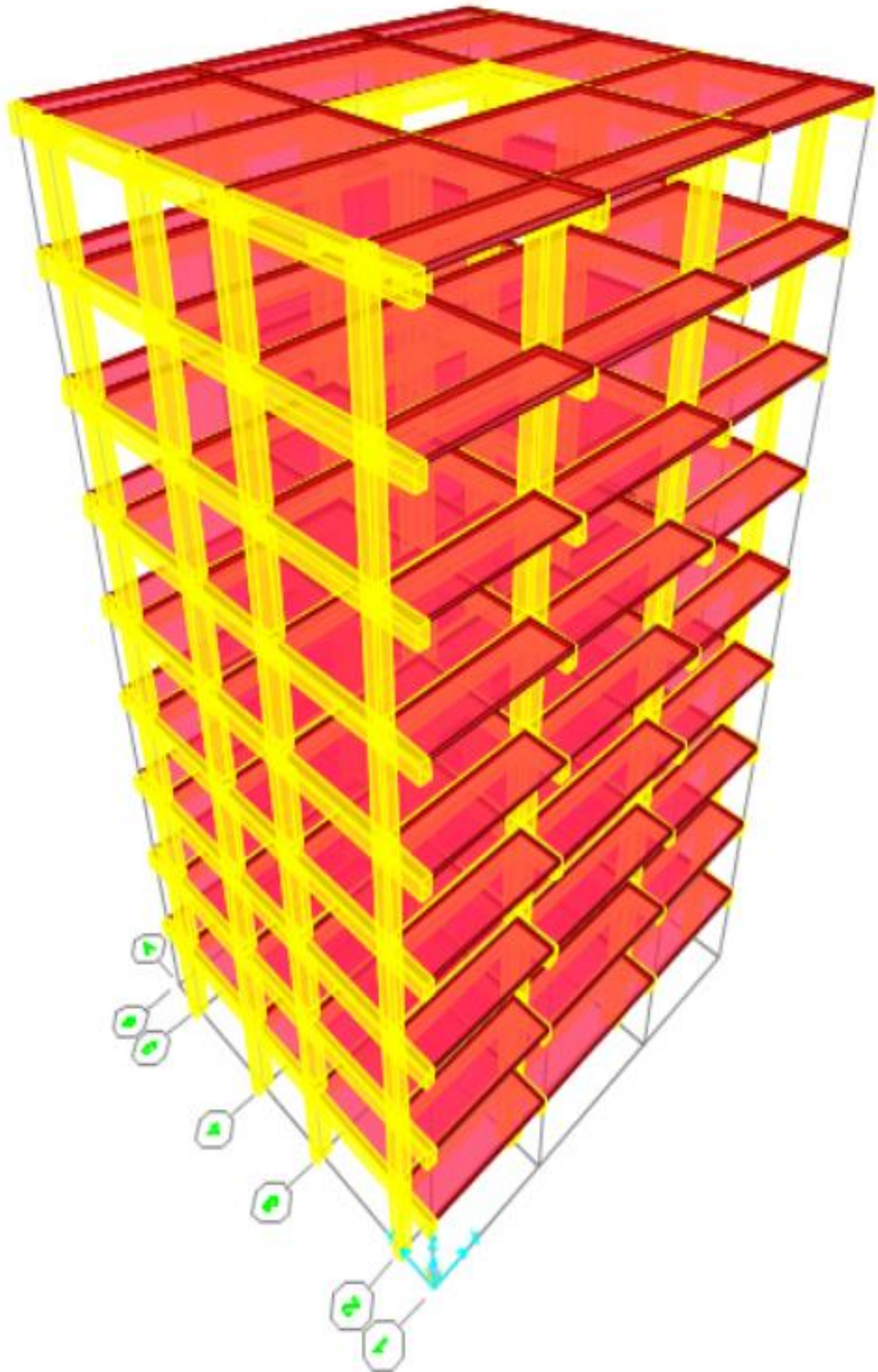

Figure 4.2: Nine-storey building modelled in SAP2000

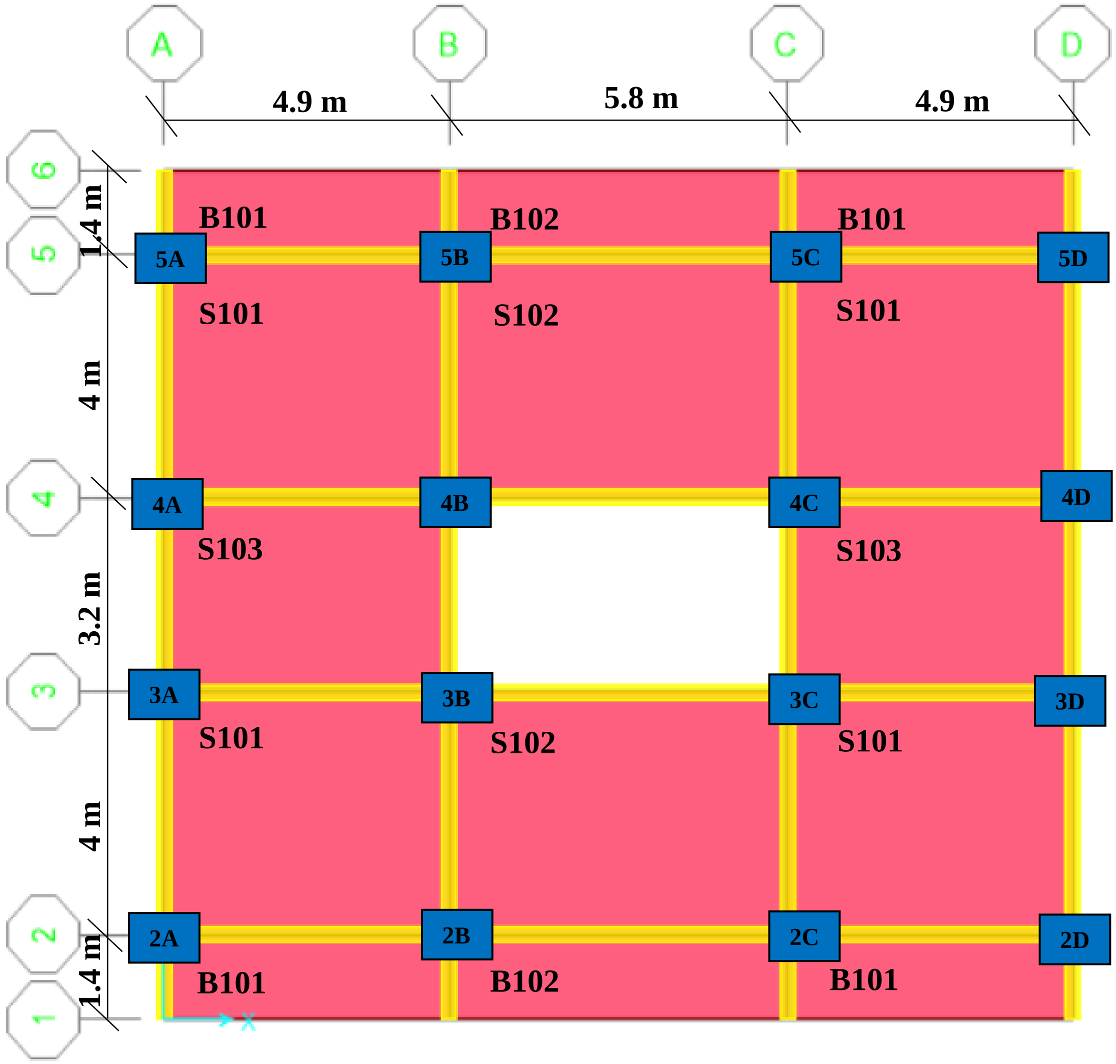

Figure 4.3: Typical storey plan

Table 4.1: Summary of loads applied on columns

Staircase Loads				
	Landing		**Stair**	
Dead Load	Plaster + Cladding	1.5 kN/m^2	Plaster + Cladding	1.5 kN/m^2
	Self-weight of slab	4 kN/m^2	Self-weight of slab	4 kN/m^2
			Wight of stair	1.875 kN/m^2
Live Load	From TS 498	3.5 kN/m^2	From TS 498	3.5 kN/m^2
Slab Loads				
	S101, S102, and S103		**B101 and B102**	
Dead Load	Plaster +	1 kN/m^2	Plaster +	1 kN/m^2
	Self-weight of	3 kN/m^2	Self-weight of	3 kN/m^2
Live Load	From TS 498	2 kN/m^2	From TS 498	5 kN/m^2
Wall Loads				
Dead Load	Wall		4.809 kN/m	
	Plaster		1.019 kN/m	
Beam Loads				
Dead Load	Self-weight of Beam		2.5 kN/m	

4.1 Preliminary Design

In conventional method, preliminary design is required to obtain equivalent lateral load which includes determination of slab thickness, beam dimensions and column dimensions.

4.1.1 Determination of Slab Thickness

According to TS500, thickness of two-way slabs can not be less than the value obtained from Equation 16;

$$h \geq \frac{l_{sn}}{15 + \frac{20}{m}} \left(1 - \frac{\alpha_s}{4} \right) \text{ and } h \geq 80 \text{ mm} \tag{16}$$

where

h = thickness of slab,

l_{sn} = free span of slab in short direction,

m = the ratio of slab long edge length to short edge length,

α_s = the ratio of slab continuous edge lengths to total edge lengths.

In addition, the ratio of slab thickness of a one-way slab to free span cannot be less than the values given below;

For simple support, single span slab	1/25,
For continuous slab	1/30,
For cantilever slab	1/12.

Considering these restrictions, the minimum slab thickness required for each slab is determined below;

Table 4.2: Calculation of slab thickness

Slab thickness					
Slab ID	**m**	**Load**	l_{sn}	α_s	**h_{min} (cm)**
S101	1.23	Two-way	3.75	0.7753	10
S102	1.45	Two-way	3.75	0.7041	11
S103	1.53	Two-way	2.95	0.6049	9
B101	-	One-way	1.275	-	11
B102	-	One-way	1.275	-	11

In order to provide continuity in each slab, slab thickness is selected as 11 cm.

4.1.2 Determination of Beam Dimensions

Beam dimensions should be determined to satisfy the TS 500 regulations which are illustrated in the following figure.

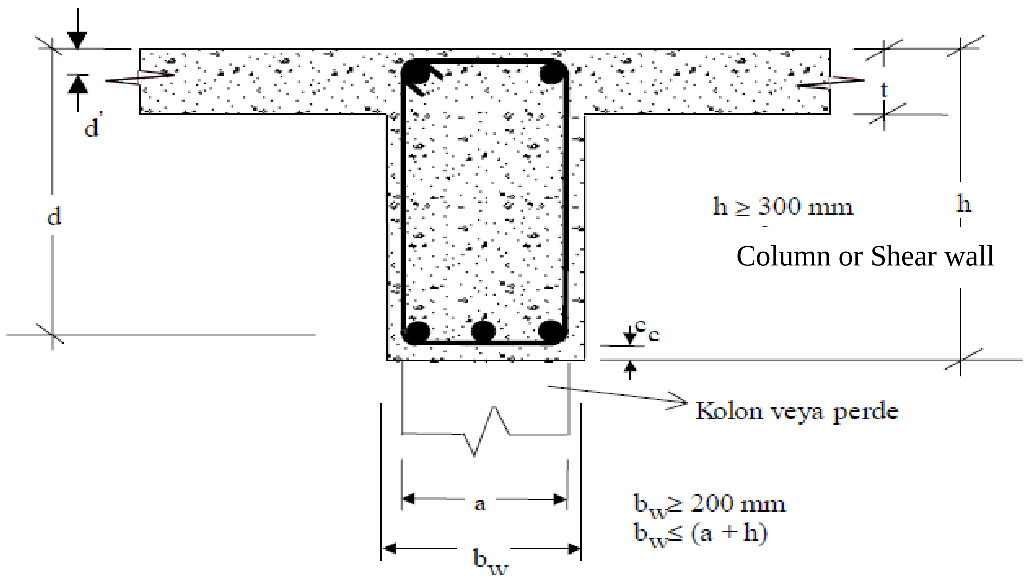

Figure 4.4: Required beam cross section dimensions according to TS 500

Considering the Turkish Earthquake Code (2007), beam width can not be less than 250 mm. The determined beam cross section is;

Table 4.3: Determination of beam cross-section

			Selected
b_w (mm)	≥ 250	≤ (a+h)	250
h (mm)	≥ 300	≥ 360 (3t)	400

As a result, 250x400 is selected.

4.1.3 Determination of Column Dimensions

In conventional buildings, load transfer is from slabs to beams and beams to columns. In preliminary design, column dimensions are determined according to the vertical load generated from the impact areas of the columns. The domain of the column consists of half of the slabs and beams around the column. Since the structure is symmetrical, there are actually four columns to be designed. The domain of each column is shown in Table 4.4.

Table 4.4: The domain of each column

Column ID	Beam ID	Beam ID	Beam ID	Beam ID	Slab ID	Slab ID	Slab ID	Slab ID
2A	A-1-2	A-2-3	2-A-B	-	S101	B101	-	-
2B	B-1-2	B-2-3	2-A-B	2-B-C	S101	S102	B101	B102
3A	A-2-3	A-3-4	3-A-B	-	S101	S103	-	-
3B	B-2-3	B-3-4	3-A-B	3-B-C	S101	S102	S103	-

After calculating the axial loads on each column, column dimensions are determined according to the TS 500 minimum column area restriction.

$$A_c \geq \frac{N_d}{0.9 f_{cd}} \tag{17}$$

According to TS500, the smallest dimension of rectangular columns cannot be smaller than 250 mm. In TEC 2018 it is stated that the smallest column section is 300 mm. Although the design was made according to TEC 2007, in order to stay on the safe side, 300 mm was chosen as the smallest column section. The axial loads were increased by 10% for taking into account horizontal loads and to reduce the number of iterations.

Table 4.5: Determination of column cross section of five storey building

Column ID	N_d	min A_c	Selected
2A	66411.32	221.37	300x300
2B	126210.22	420.70	300x400
3A	61762.77	205.88	300x300
3B	154690.81	441.97	350x450

Table 4.6: Determination of column cross section of nine storey building

Column ID	N_d	min A_c	Selected Dimensions
2A	155142.33	443.26	350x450
2B	294919.26	589.84	500x600
3A	144543.71	481.81	500x300
3B	361407.53	721.37	500x750

4.2 Iteration for Determining the Horizontal Loads

After the preliminary design, the spectral acceleration corresponding to the designed building was obtained, which was used to calculate the equivalent earthquake forces. In general, the column cross sections determined in preliminary design are not sufficient to bear the applied load, therefore, the column sizes are increased. The corresponding spectral acceleration value changes when the column dimensions are increased. Determining the final horizontal loads and column cross sections requires several iterations like this. The axial loads on the columns were increased by 10% in order to decrease the number of iterations. Three iterations were required for a five-story structure with a yield reduction factor of four, one iteration for the same structure with a yield reduction factor of eight, two iterations for a nine-story structure with a yield reduction factor of four, and one iteration for the same structure with a yield reduction factor of eight.

Then, the vibration period of the structure and its corresponding spectral acceleration from the conventional response spectrum were determined. Equivalent earthquake loads were calculated from the spectral acceleration as given in Equation 3 and 4. Then, they were imposed on the structure and it was checked whether the structural elements can bear these loads. In the tables below, the checks of each iteration and the new structural element sizes determined for the next iteration, are given. After the lateral loading, if an element can still service letter S is typed to indicate that element size is sufficient. If not, letter I is typed to indicate that the structural element is insufficient and its size should be increased.

Table 4.7: Results of first iteration of a five storey building with reduction factor four

Iteration 1								
Tx(sec)	1.02							
Ty(sec)	1.01							
Sa(g)	0.093							
Sa(g)	0.093							
Selected Beam	25x40							
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Selected Column	30x30	30x40	30x40	30x30	30x30	35x45	35x45	30x30
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Selected Column	30x30	35x45	35x45	30x30	30x30	30x40	30x40	30x30
Beam ID	2-A-B	2-B-C	2-C-D	3-A-B	3-B-C	3-C-D	4-A-B	4-B-C
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	4-C-D	5-A-B	5-B-C	5-C-D	A-1-2	A-2-3	A-3-4	A-4-5
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	A-5-6	B-1-2	B-2-3	B-3-4	B-4-5	B-5-6	C-1-2	C-2-3
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	C-3-4	C-4-5	C-5-6	D-1-2	D-2-3	D-3-4	D-5-6	D-6-7
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Z=3	S	I	I	S	I	S	I	I
Z=6	S	S	S	S	I	S	S	I
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Z=3	I	S	I	S	S	I	I	S
Z=6	I	S	S	I	S	S	S	S
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S

Table 4.8: Results of second iteration of a five storey building with reduction factor four

Iteration 2								
Tx(sec)	0.97							
Ty(sec)	0.97							
Sa(g)	0.101							
Sa(g)	0.101							
Selected Beam	25x40							
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Selected Column	30x30	30x45	30x45	30x30	30x35	35x45	35x50	30x35
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Selected Column	30x35	35x45	35x50	30x35	30x30	30x45	30x45	30x30
Beam ID	2-A-B	2-B-C	2-C-D	3-A-B	3-B-C	3-C-D	4-A-B	4-B-C
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	4-C-D	5-A-B	5-B-C	5-C-D	A-1-2	A-2-3	A-3-4	A-4-5
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	A-5-6	B-1-2	B-2-3	B-3-4	B-4-5	B-5-6	C-1-2	C-2-3
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	C-3-4	C-4-5	C-5-6	D-1-2	D-2-3	D-3-4	D-5-6	D-6-7
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Z=3	S	I	I	S	S	S	S	S
Z=6	S	S	S	S	S	S	S	S
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Z=3	S	S	S	S	S	I	I	S
Z=6	S	S	S	S	S	S	S	S
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S

Table 4.9: Results of third iteration of a five storey building with reduction factor four

Iteration 3								
Tx(sec)	0.96							
Ty(sec)	0.95							
Sa(g)	0.101							
Sa(g)	0.101							
Selected Beam	25x40							
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Selected Column	30x30	30x50	30x50	30x30	30x35	35x45	35x50	30x35
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Selected Column	30x35	35x45	35x50	30x35	30x30	30x50	30x50	30x30
Beam ID	2-A-B	2-B-C	2-C-D	3-A-B	3-B-C	3-C-D	4-A-B	4-B-C
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	4-C-D	5-A-B	5-B-C	5-C-D	A-1-2	A-2-3	A-3-4	A-4-5
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	A-5-6	B-1-2	B-2-3	B-3-4	B-4-5	B-5-6	C-1-2	C-2-3
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	C-3-4	C-4-5	C-5-6	D-1-2	D-2-3	D-3-4	D-5-6	D-6-7
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Z=3	S	S	S	S	S	S	S	S
Z=6	S	S	S	S	S	S	S	S
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Z=3	S	S	S	S	S	S	S	S
Z=6	S	S	S	S	S	S	S	S
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S

4.3 Storey Drift and Stability Coefficient Check According to TEC2007

After the third iteration since all structural elements were sufficient, the sections of the elements were determined. However, according to TEC 2007, it is necessary to check the relative storey drifts and stability coefficient values. If these conditions are not fulfilled, it is necessary to increase the dimensions of the elements and check them again.

4.3.1 Relative Storey Drift Check

TEC 2007 determines that relative storey drift can not be greater than 0.02. This is checked as shown in the following equations;

$$\Delta_i = d_i - d_{i-1} \tag{18}$$

$$\delta_i = R\,\Delta_i \tag{19}$$

$$\frac{(\delta i)_{max}}{h_i} \tag{20}$$

where

d_i = displacement of the storey i of the structure due to reduced earthquake loads,

Δ_i = reduced relative storey drift of the storey i of the structure,

δ_i = effective relative storey displacement of the storey i of the building.

The storey drift checks of the building in x and y direction are illustrated in Table 4.10. The condition of drift less than 0.02 at each floor was provided in all designed buildings.

Table 4.10: Storey drift check of a five storey building with a reduction factor four

	X - Direction					**Y – Direction**				
Z	**d_i**	Δ_i	$\boldsymbol{\delta_i}$	$\boldsymbol{\delta_i/h_i}$	**≤0.02**	**d_i**	Δ_i	$\boldsymbol{\delta_i}$	$\boldsymbol{\delta_i/h_i}$	**≤0.02**
15	0.04	0.003	0.01	0.004	OK	0.04	0.004	0.01	0.004	OK
12	0.03	0.007	0.02	0.008	OK	0.03	0.006	0.02	0.008	OK
9	0.03	0.009	0.03	0.011	OK	0.03	0.009	0.03	0.011	OK
6	0.02	0.010	0.04	0.013	OK	0.02	0.010	0.04	0.013	OK
3	0.01	0.008	0.03	0.010	OK	0.01	0.008	0.03	0.010	OK

4.3.2 Stability Coefficient Check

According to TEC 2007 stability coefficient can not be greater than 0.2. The check of this condition is as follows;

$$\theta = \frac{(\Delta i)avg \sum_{j=1}^{N} wj}{V_i h_i} \tag{21}$$

where

$(\Delta i)_{avg}$ = average reduced relative storey drift of the storey i of the structure,

w_j = storey weight of the storey i, calculated using live load participation coefficient,

V_i = shear force acting on the storey i of the structure in the direction of the considered earthquake,

h_i = storey height of the storey i.

In order to do this check, the seismic weight and shear force of each storey must be calculated. Base shear force is distributed to each storey as shown in Equation 3 and 4. In table 4.13, check results are given and all stability coefficient values were less than the limit 0.2. This code specified limits were satisfied in other designed buildings as well.

Table 4.11: Calculation of weight of each storey of a five storey building

Elevation (m)	Slab		Staircase		Beam	Wall	Column	Total		Seismic Weight (kN)
	Dead	Live	Dead	Live	Dead	Dead	Dead	Dead	Live	
3	749.4	530.7	214.0	129.9	295.9	624.8	153.3	2037.4	660.6	2235.6
6	749.4	530.7	214.0	129.9	295.9	624.8	153.3	2037.4	660.6	2235.6
9	749.4	530.7	214.0	129.9	295.9	624.8	153.3	2037.4	660.6	2235.6
12	749.4	530.7	214.0	129.9	295.9	624.8	153.3	2037.4	660.6	2235.6
15	749.4	530.7	214.0	129.9	295.9	-	153.3	1412.6	660.6	1610.8
									ΣW	10553.1

Table 4.12: Calculation of shear force acting on each storey of a five storey building

ΔF_N (kN)	40.25			
Elevation (m)	**W_j (kN)**	**w_iH_i (kN.m)**	**$W_iH_i/\Sigma W_iH_i$**	**V_i (kN)**
3	2235.58	6706.74	0.07	75.95
6	2235.58	13413.49	0.15	151.90
9	2235.58	20120.23	0.22	227.85
12	2235.58	26826.98	0.29	303.80
15	1610.83	24162.46	0.26	313.87

Table 4.13: Storey drift check of a five storey building with a reduction factor four

	X - Direction					**Y – Direction**				
Z	**$(d_i)_{avg}$**	**$(\Delta_i)_{avg}$**	**V_i**	**θ_i**	**≤0.12**	**$(d_i)_{avg}$**	**$(\Delta_i)_{avg}$**	**V_i**	**θ_i**	**≤0.12**
15	0.036	0.003	313.8	0.013	OK	0.035	0.003	314.3	0.013	OK
12	0.032	0.006	303.8	0.013	OK	0.032	0.006	304.2	0.013	OK
9	0.026	0.008	227.8	0.022	OK	0.025	0.008	228.1	0.021	OK
6	0.017	0.010	152	0.028	OK	0.017	0.009	152.1	0.027	OK
3	0.007	0.007	75.9	0.026	OK	0.007	0.007	76.05	0.025	OK

Chapter 5

INCLUSION OF P-DELTA EFFECT

Buildings were designed using conventional response spectrum. However, P-delta effect is not included in these spectra. In order to see the P-delta effect, after the design of the building, the equivalent SC corresponding to the fundamental vibration period of the building was calculated by using Equation 22 and the corresponding spectral acceleration from SCRS was calculated and compared.

$$T_0 = 2\pi\sqrt{\frac{\theta . h}{g}} \tag{22}$$

The comparisons are in the table below.

Table 5.1: Comparison of base shears obtained by conventional response spectrum and SCRS

	Five-Storey Building		Nine-Storey Building	
	R=4	**R=8**	**R=4**	**R=8**
Conventional				
T_x (sec)	0.96	1.02	1.19	1.37
S_a (g)	0.10185	0.0467	0.0865	0.0534
V_b (kN)	1074.84	489.01	1922.31	1137.43
SCRS				
θ_x	0.0748	0.0862	0.1173	0.1555
S_a (g)	0.1035	0.0473	0.0889	0.0547
V_b (kN)	1092.57	495.71	1974.96	1164.27
Conventional				
Ty (sec)	0.95	1.01	1.18	1.36
S_a (g)	0.10171	0.0467	0.087	0.0549
V_b (kN)	1073.36	488.65	1932.75	1169.38
SCRS				

θ_y	0.0763	0.0845	0.1153	0.1532
S_a (g)	0.1036	0.0478	0.0887	0.0563
V_b (kN)	1093.31	500.59	1971.56	1199.20

When the earthquake load obtained using SCRS was applied to a five-story building with a reduction factor of four, it was observed that some columns of the building failed. This shows that, when the P-delta effect is not taken into account, although the limitations in the earthquake code is satisfied, it can be a cause of failure of the structural elements or even collapse of the whole building. As shown in Tables 5.3 and 5.4, as the height of the building increases, the effect of the P-delta effect increases as it is directly related to the weight of the building. It is a known fact that the P-delta effect increases in long-term buildings, and the results of this study are compatible with this judgement.

Table 5.2: Checking the adequacy of structural elements under earthquake load obtained from SCRS

Tx(sec)	0.96							
Ty(sec)	0.95							
Sa(g)	0.103							
Sa(g)	0.103							
Selected Beam	25x40							
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Selected Column	30x30	30x50	30x50	30x30	30x35	35x45	35x50	30x35
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Selected Column	30x35	35x45	35x50	30x35	30x30	30x50	30x50	30x30
Beam ID	2-A-B	2-B-C	2-C-D	3-A-B	3-B-C	3-C-D	4-A-B	4-B-C
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	4-C-D	5-A-B	5-B-C	5-C-D	A-1-2	A-2-3	A-3-4	A-4-5
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	A-5-6	B-1-2	B-2-3	B-3-4	B-4-5	B-5-6	C-1-2	C-2-3
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Beam ID	C-3-4	C-4-5	C-5-6	D-1-2	D-2-3	D-3-4	D-5-6	D-6-7
Z = 3 m	S	S	S	S	S	S	S	S
Z = 6 m	S	S	S	S	S	S	S	S
Z = 9 m	S	S	S	S	S	S	S	S
Z = 12 m	S	S	S	S	S	S	S	S
Z = 15 m	S	S	S	S	S	S	S	S
Column ID	2A	2B	2C	2D	3A	3B	3C	3D
Z=3	I	S	S	I	S	S	S	S
Z=6	S	S	S	S	S	S	S	S
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S
Column ID	4A	4B	4C	4D	5A	5B	5C	5D
Z=3	S	S	S	S	I	S	S	I
Z=6	S	S	S	S	S	S	S	S
Z=9	S	S	S	S	S	S	S	S
Z=12	S	S	S	S	S	S	S	S
Z=15	S	S	S	S	S	S	S	S

Table 5.3: Increase in base shear (%) of a five storey building due to P-delta effect

	Five Storey Building			
	R=4		**R=8**	
	X-Direction	**Y-Direction**	**X-Direction**	**Y-Direction**
Increase in V_b (%)	1.65	1.86	1.37	2.44

Table 5.4: Increase in base shear (%) of a nine storey building due to P-delta effect

	Nine Storey Building			
	R=4		**R=8**	
	X-Direction	**Y-Direction**	**X-Direction**	**Y-Direction**
Increase in V_b (%)	2.74	2.01	2.36	2.55

Chapter 6

CONCLUSION

Ground vibrations created by earthquakes cause damage to buildings. Therefore, if a building is located in a seismically active region, the design according to earthquake codes are mandatory. Earthquake codes have a guiding role in seismic building design. After the design of the building, second order effect and storey drift of the structure should be checked according to the codes. If the structure does not fulfill the specified code limits, iteration should be continued until it does.

P-delta has a significant effect if the structure is not a short period system. Because in long-period structures it causes prolongation of the period and decrease in lateral stiffness. If neglected, it may even cause collapse of the whole structure. For this reason, it is very convenient to create a response spectrum that includes the P-delta effect. Thus, the P-delta effect will be included when obtaining the spectral acceleration. Stability coefficient response spectrum was formed in this aspect. FSSDOF system with a known height was used as a SDOF system so that P-delta effect is placed into the equation of motion. In order to emphasize the importance of the P-delta effect, base shears obtained from the conventional response spectrum, and from stability coefficient response spectrum were compared. Considering the base shear values, a slight increase of 1-3% was observed. However, when the spectral acceleration obtained from SCRS was imposed on the building, failure of the four columns were observed. Although the building was designed according to the earthquake code and fulfilled the all specified limits, it could not bear the load containing the P-delta effect. So, P-delta effect cannot be ignored in real life, as its negligence can cause serious problems.

There is only one limitation for the formed SCRS which is they can only be used for the buildings with the first storey height of 3 m. If the first storey height is not 3 m, the spectra would give wrong spectral information.

Recommendations:

- In this study, spectral acceleration, including the P-delta effect, is imposed on the building and it is observed that there are elements that cannot bear the load. By taking this into consideration, the building can be designed using the spectral acceleration obtained from SCRS and the building designed with the conventional response spectrum can be compared in terms of cross section of structural elements, structure weight, and the base shear and equivalent lateral loads on the structure.
- Earthquake codes have a limit for stability coefficient but they do not consider the vibration period of the buildings. This limit can be used as a starting point to reduce the number of iterations. For this purpose, SCRS through FSSDOF approach can be used. However, in the equations developed by Mousavi and Şensoy, rotational stiffness is used since inverted pendulum is used as SDOF system and further research is required to determine the equivalent lateral horizontal stiffness of the first floor.
- In this thesis SCRS are formed for first storey height of 3 m. This can be increased (e.g 4 m, 5 m and 6 m) in further studies.

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APPENDICES

Appendix A: The Implementation of Newmark's Method in a MATLAB Program

```
load gm1.txt
g=9.810;
dt=0.005;
zet=0.05;
endp=3;
m=1;
Ag=-g*gm1(:,2);
T(1)=0.01;
for j=1:round(endp/dt)
   if dt/T(j)<=0.551
      gamma=0.5;
      beta=1/6;
   else
      gamma=0.5;
      beta=0.25;
   end
   omega(j)=2*pi/T(j);
   k=(omega(j))^2*m;
   c=2*m*omega(j)*zet;
   a1=m/(beta*dt^2)+gamma*c/(beta*dt);
   a2=m/(beta*dt)+c*(gamma/beta-1);
   a3=m*(1/(2*beta)-1)+dt*c*(gamma/(2*beta)-1);
   K=k+a1;
   for i=1:length(Ag)-1
      u(1)=0;
      v(1)=0;
      ac(1)=0;
      Ag_(i+1)=Ag(i+1)+a1*u(i)+a2*v(i)+a3*ac(i);
      u(i+1)=Ag_(i+1)/K;
      v(i+1)=gamma/(beta*dt)*(u(i+1)-u(i))+(1-gamma/beta)*v(i)+(1-
gamma/(2*beta))*dt*ac(i);
      ac(i+1)=1/(beta*(dt^2))*(u(i+1)-u(i))-1/(beta*dt)*v(i)-(1/(2*beta)-
1)*ac(i);
   end
   Sd(j,1)=max(abs(u));
   Spv(j,1)=Sd(j)*omega(j);
   Spa(j,1)=Sd(j)*(omega(j))^2/g;
   T(j+1,1)=T(j)+dt;
end
```

Appendix B: MATLAB Code for Yield Deformation Response Spectrum

```
load gm1u0.txt
T(1)=0.01;
g=9.810;
endp=3;
dt=0.01;
R=4;
uy=gm1u0(:,1)/R;

for j=1:round(endp/dt)
    omega(j)=2*pi/T(j);
    Sd=uy;
    Spv(j,1)=Sd(j)*omega(j);
    Spa(j,1)=Sd(j)*(omega(j))^2/g;
    T(j+1,1)=T(j)+dt;
end

%Ag(end)=[];
T(end)=[];
%Sd(2,1)=0; Spv(1:2,1)=0;Spa(1:2,1)=max(abs(Ag))/g;
subplot(2,1,1)
 %figure('Name','Spectral Displacement','NumberTitle','off')
 plot(T,Sd,'LineWidth',2.)
 grid on
xlabel('Period (sec)','FontSize',13);
ylabel('Sd (m)','FontSize',13);
title('Displacement Spectrum','FontSize',13)

subplot(2,1,2)
 %figure('Name','Pseudo Acceleration Spectrum','NumberTitle','off')
 plot(T,Spa,'LineWidth',2.)
 grid on
xlabel('Period (sec)','FontSize',13);
ylabel('Spa (g)','FontSize',13);
title('Pseudo Acceleration Spectrum','FontSize',13)


loglog(x,gm1,x,gm2,x,gm3,x,gm4,x,gm6,x,gm7,x,gm8,x,gm9,x,gm10,x,gm11
,x,gm12,x,gm14,x,gm15,x,gm16,x,gm18,x,gm19,x,gm20,'HandleVisibility','of
f')
hold on
xlim([0.1 3])
loglog(x,M,'k','linewidth',2,'DisplayName','Median response spectrum')
```

```
loglog(x,percentile84th,'--r','linewidth',2,'HandleVisibility','off')
loglog(x,percentile16th,'--r','linewidth',2,'DisplayName','16th and 84th
percentile response spectra')

hold off

xlabel('Period','FontSize',13);
ylabel('Spa (g)','FontSize',13);
%title('Response Spectra of Selected Ground Motions','FontSize',13)
legend
annotation('textbox', [0.7, 0.1, 0.1, 0.1], 'String', "R=8")
```

Appendix C: Adaptation of Newmark's Method for Inverted Pendulum Model

Equation of Motion for Inverted Pendulum: $mh^2 \ddot{\emptyset}_{i+1} + c_{\emptyset} \dot{\emptyset}_{i+1} + K_{\emptyset 0} \emptyset_{i+1} = p_{i+1}$

Two main equation of Newmark's Method is adapted to inverted pendulum model as;

$$\dot{\emptyset}_{i+1} = \emptyset_i + [(1 - \gamma)\Delta t]\dot{\emptyset}_i + (\gamma\Delta t)\ddot{\emptyset}_{i+1} \qquad (1)$$

$$\emptyset_{i+1} = \emptyset_i + \Delta t\dot{\emptyset}_i + [(0.5 - \beta)(\Delta t)^2]\ddot{\emptyset}_i + [\beta(\Delta t)^2]\ddot{\emptyset}_{i+1} \qquad (2)$$

In the second equation $\ddot{\emptyset}_{i+1}$ is brought to left side of the equation.

$$\ddot{\emptyset}_{i+1} = \frac{1}{\beta\Delta^2}(\emptyset_{i+1} - \emptyset_i) - \frac{1}{\beta\Delta t}\dot{\emptyset}_i + (1 - \frac{1}{2\beta})\ddot{\emptyset}_i \qquad (3)$$

Substitute third equation in to the first one.

$$\dot{\emptyset}_{i+1} = \frac{\gamma}{\beta\Delta t}(\emptyset_{i+1} - \emptyset_i) + (1 - \frac{\gamma}{\beta})\dot{\emptyset}_i + \Delta t(1 - \frac{\gamma}{2\beta})\ddot{\emptyset}_i \qquad (4)$$

Substitute equations 1, 3, and 4 into the Equation of Motion;

$$mh^2\left[\frac{1}{\beta\Delta^2}(\emptyset_{i+1} - \emptyset_i) - \frac{1}{\beta\Delta t}\dot{\emptyset}_i + (1 - \frac{1}{2\beta})\ddot{\emptyset}_i\right] + c_{\emptyset}\left[\frac{\gamma}{\beta\Delta t}(\emptyset_{i+1} - \emptyset_i) + (1 - \frac{\gamma}{\beta})\dot{\emptyset}_i + \Delta t(1 - \frac{\gamma}{2\beta})\ddot{\emptyset}_i\right] + K_{\emptyset 0}\left[\emptyset_i + \Delta t\dot{\emptyset}_i + [(0.5 - \beta)(\Delta t)^2]\ddot{\emptyset}_i + [\beta(\Delta t)^2]\ddot{\emptyset}_{i+1}\right]$$

$$\left[K_{\emptyset 0} + (\frac{\gamma}{\beta\Delta t}c_{\emptyset} + \frac{1}{\beta\Delta t^2}mh^2)\right]\emptyset_{i+1} = p_{i+1} + \left[\frac{1}{\beta\Delta t}mh^2 + \frac{\gamma}{\beta\Delta t}c_{\emptyset}\right]\emptyset_i + \left[\frac{1}{\beta\Delta t}mh^2 + (\frac{\gamma}{\beta} - 1)c_{\emptyset}\right]\dot{\emptyset}_i + \left[(\frac{1}{2\beta} - 1)mh^2 + (\frac{\gamma}{2\beta} - 1)\Delta t\, c_{\emptyset}\right]\ddot{\emptyset}_i$$

$$\hat{k} = K_{\emptyset 0} + (\frac{\gamma}{\beta\Delta t}c_{\emptyset} + \frac{1}{\beta\Delta t^2}mh^2)$$

$$a_1 = \frac{1}{\beta\Delta t}mh^2 + \frac{\gamma}{\beta\Delta t}c_{\emptyset}$$

$$a_2 = \frac{1}{\beta\Delta t}mh^2 + (\frac{\gamma}{\beta} - 1)c_{\emptyset}$$

$$a_3 = (\frac{1}{2\beta} - 1)mh^2 + (\frac{\gamma}{2\beta} - 1)\Delta t\, c_{\emptyset}$$

$\hat{k}\ \emptyset_{i+1} = \hat{p}_{i+1}$

$\hat{p}_{i+1} = p_{i+1} + [\ \frac{1}{\beta\Delta t}\ mh^2 + \frac{\gamma}{\beta\Delta t}\ c_\emptyset\ ]\ \emptyset_i + [\ \frac{1}{\beta\Delta t}\ mh^2 + (\ \frac{\gamma}{\beta} - 1\ )\ c_\emptyset\ ]\ \dot{\emptyset}_i + [\ (\ \frac{1}{2\beta} - 1)mh^2 + (\frac{\gamma}{2\beta} - 1)\Delta t\ c_\emptyset\ ]\ \ddot{\emptyset}_i$

$$\hat{p}_{i+1} = p_{i+1} + a_1\ \emptyset_i + a_2\ \dot{\emptyset}_i + a_3\ \ddot{\emptyset}_i$$

Appendix D: The Implementation of Newmark's Method for Non-Linear Systems in a MATLAB Program

```
load gm1.txt
load gm1u0.txt
g=9.810;
dt=0.01;
zet=0.05;
endp=3;
m=1;
Ag=-g*gm20(:,2);
n=length(Ag)-1;
TOL=10e-11;
R=8;
u0=gm1u0;
T(1)=0.01;
 for j=1:round(endp/dt)
    if dt/T(j)<=0.551
       gamma=0.5;
       beta=1/6;
    else
       gamma=0.5;
       beta=0.25;
    end
    omega(j)=2*pi/T(j);
    k=(omega(j))^2*m;
    c=2*m*omega(j)*zet;
    f0=k*u0(j);
    fy=1/R*f0;
    uy=fy/k;
    a1=m/(beta*dt^2)+gamma*c/(beta*dt);
    a2=m/(beta*dt)+c*(gamma/beta-1);
    a3=m*(1/(2*beta)-1)+dt*c*(gamma/(2*beta)-1);
    k_=k+a1;
    u=zeros(length(Ag),1);
    v=zeros(length(Ag),1);
    ac=zeros(length(Ag),1);
    fs=zeros(length(Ag),1);
    Ag_(1)=0;
    Ag_(2)=Ag(2)+a1*u(1)+a2*v(1)+a3*ac(1);
    u(2)=Ag_(2)/k_;
    v(2)=gamma*(u(2)-u(1))/(beta*dt)+v(1)*(1-gamma/beta)+dt*ac(1)*(1-
gamma/(2*beta));
    ac(2)=(u(2)-u(1))/(beta*dt.^2)-v(1)/(beta*dt)-ac(1)*(1/(2*beta)-1);
    fs(2)=fs(1)+(u(2)-u(1))*k;
```

```
for i=2:n
   Ag_(i+1)=Ag(i+1)+a1*u(i)+a2*v(i)+a3*ac(i);
   if (v(i)*v(i-1)<0) || (abs(fs(i))<fy)
      u(i+1)=Ag_(i+1)/k_;
      fs(i+1)=fs(i)+(u(i+1)-u(i))*k;
      if abs(fs(i+1))>fy
         if fs(i+1)>0
            fs(i+1)=fy;
         else
            fs(i+1)=-fy;
         end
      R_=Ag_(i+1)-fs(i)-a1*u(i);
      U=u(i);
      if fs(i)>fy
            kT=0;
      else
            kT=k;
      end
      while abs(R_)>TOL
         kT_=kT+a1;
         du=R_/kT_;
         U=U+du;
         f=fs(i)+(U-u(i))*k;
         if abs(f)>=fy && f>0
            f=fy;
            kT=0;
         elseif abs(f)>=fy && f<0
            f=-fy;
            kT=0;
         else
            kT=k;
         end
      R_=Ag_(i+1)-f-a1*U;
      if abs(R_)<TOL
         u(i+1)=U;
         fs(i+1)=f;
      end
      end
      end
   else
      if fs(i)>0
         fs(i+1)=fy;
      else
         fs(i+1)=-fy;
      end
      R_=Ag_(i+1)-fs(i)-a1*u(i);
```

```
            U=u(i);
            if fs(i)>fy
                kT=0;
            else
                kT=k;
            end
            while abs(R_)>TOL
                kT_=kT+a1;
                du=R_/kT_;
                U=U+du;
                f=fs(i)+(U-u(i))*k;
                if abs(f)>=fy && f>0
                    f=fy;
                    kT=0;
                elseif abs(f)>=fy && f<0
                    f=-fy;
                    kT=0;
                else
                    kT=k;
                end
            R_=Ag_(i+1)-f-a1*U;
            if abs(R_)<TOL
                u(i+1)=U;
                fs(i+1)=f;
            end
            end
        end
        v(i+1)=gamma*(u(i+1)-u(i))/(beta*dt)+v(i)*(1-
gamma/beta)+dt*ac(i)*(1-gamma/(2*beta));
        ac(i+1)=(u(i+1)-u(i))/(beta*dt.^2)-v(i)/(beta*dt)-ac(i)*(1/(2*beta)-1);
    end
    Sd(j,1)=max(abs((u(:,1))));
    Spv(j,1)=Sd(j)*omega(j);
    Spa(j,1)=Sd(j)*(omega(j))^2/g;
    T(j+1,1)=T(j)+dt;
 end
Ag(end)=[];
T(end)=[];
Sd(2,1)=0; Spv(1:2,1)=0;Spa(1:2,1)=max(abs(Ag))/g;
subplot(2,1,1)
 %figure('Name','Spectral Displacement','NumberTitle','off')
 plot(T,Sd,'LineWidth',2.)
 grid on
xlabel('Period (sec)','FontSize',13);
ylabel('Sd (mm)','FontSize',13);
title('Displacement Spectrum','FontSize',13)
```

```
subplot(2,1,2)
 %figure('Name','Pseudo Acceleration Spectrum','NumberTitle','off')
 plot(T,Spa,'LineWidth',2.)
 grid on
xlabel('Period (sec)','FontSize',13);
ylabel('Spa (g)','FontSize',13);
title('Pseudo Acceleration Spectrum','FontSize',13)
```

Ductility Demand Curve is plotted by;

```
loglog(x,Mu,'k','linewidth',2)
hold on
xlim([0.1 3])
ylim([0 10])


hold off

xlabel('Period','FontSize',13);
ylabel('Ductility Demand','FontSize',13);
%title('Response Spectra of Selected Ground Motions','FontSize',13)
annotation('textbox', [0.7, 0.72, 0.1, 0.1], 'String', "R=8")
```