Seismic Performance Evaluation of 2-Dimensional Reinforced Concrete, Steel and Mixed Frames

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ABSTRACT

It is well known that, Turkey and Cyprus are in high seismic activity zone. Therefore designers should consider the seismic effects according to described earthquake. Reinforced concrete and structural steel are the two main materials that are used in construction industry. However, these two materials possess different characteristics in their behavior. In recent years structural steel has become more popular due to some of its characteristics such as being light, ability for being prefabricated, fast erection and ductility levels.

Nowadays, with lack of space for the development of cities multi-story structures are more in favor by the building developers and contractors. Demolishing old structures and constructing new high-rise buildings are not always the optimized solution from economic point of view. In recent years using light materials to build extra stories on existing structures is believed to be a good option for the solution of load problem. Under static loads there may not be any serious concern. However the analysis and assessment of seismic performance of these mixed-material frames would be different when compared to conventional structures. Designing such structures would be a challenging task since current design codes do not support the analysis and design solutions for the structures having frames with different damping ratios.

The scope of this work is to evaluate the seismic performance of mixed and regular structures. For this, the mixed structural models considered had two parts; a lower part constructed from reinforced concrete and the upper part made constructed of structural steel.

In order to investigate and evaluate the seismic performance of mixed structures and compare the results with those of normal structures, the nonlinear time history analysis method was used including geometric and material nonlinearities. In order to achieve a reliable comparison the response of mixed structures under dynamic loads were investigated together with normal structural steel and reinforced concrete structures. Comparison of the results showed that changing the story numbers or structural materials will cause contrasting results.

Plain frame dynamic analyses were performed on three different frame structural models. Two regular framed models; fully reinforced concrete and fully structural steel were designed according to codes and the third one was created by combination of the other two models.

Keywords: seismic performance, mixed structure, composite framing, time history, dynamic analysis.

ÖZ

Bilindiği gibi, Türkiye ve Kıbrıs yüksek deprem riski bulunan coğrafi bir konumda bulunmaktadır. Bununla birlikte deprem etkilerinin mühendisler tarafından tasarımında gözönüne alınması yönetmeliklerde zorunlu hale getirilmiştir. Betonarme ve yapısal çelik inşai yapıların birçoğunun ana malzemesi olarak kullanılmaktadır. Bu iki malzeme karakteristik özellikleri açısından farklılık göstermektedir. Yapısal çelik, fabrikada hızlı üretimi, sahada kolay uygulamaları ve yapılardaki hafifliği nedeni ile son yıllarda daha popüler olmaya başlamıştır.

Şehirlerdeki büyük gelişimler ve inşai alanların azalması ile birlikte mütahitlerin yüksek binalara ihtiyaçları artmaya başlamıştır. Bu yükseliş çerçevesinde eski binaların yıkılıp yerine daha yükseklerinin yapılması ekonomik açıdan her zaman verim sağlamamaktadır. Bu nedenle mevcut yapıları üzerlerine daha hafif yapısal malzemeler kullanarak düşey yükler altında yüksek binalara sahip olma fikri oluşmuştur. Ancak, bu tipte karmaşık yapısal sisteme sahip yapıların deprem yükleri altındaki analizleri diğer geleneksel yapılarınkine göre daha farklılık göstermektedir. Farklı sönüm oranlarına sahip bu tip karmaşık yapısal elemanlardan oluşan binaların tasarımları ile ilgili olarak şu anda kullanılan mevcut yönetmelikler bilgi ve öneri sağlamamaktadır.

Yapılan bu çalışmanın amacı, yapısal elemanlar açısından karmaşık ve düzenli yapıların deprem performanslarını değerlendirmek ve karşılaştırmaktır. Karmaşık yapıdaki modeller iki parçadan oluşmaktadır; alt katların yapısal elemanları geleneksel betonarme modeli, üst kat elemanları ise yapısal çelik kesitlerin kullanıldığı modellerdir.

Yapsal elemanları açısından karmaşık ve düzenli yapıların deprem performanslarını değerlendirmek ve karşılaştırmak amacı ile doğrusal olmayan zaman tanım alanında hesap yöntemi, malzeme kesitlerindeki doğrusal olmayan davranış ve ikinci mertebe etkilerinin dahil edilmesi ile birlikte kullanılmıştır. Gerçekçi yapısal davranış değerlerinin elde edilebilmesi amacı ile karmaşık yapılar ve düzenli yapılar aynı dinamik yükler altında incelenmiştir. Yapılan incelemelerde yapısal elemanların tipi ve kat sayısının sonuçlarda farklılıklar yarattığı ve etkili rol aldığı gözlenmiştir.

İki boyutlu analizler, yapısal elemanları açısından değişik üç farklı model üzerinde yapılmıştır. Kullanılan üç modelden ikisi yapısal elemanları açısından tamamen betonarme ve tamamen yapısal çelik mollerin yönetmelikler gereği tasarımları ile diğeri ise tasarımı yapılan iki modelin kombine edilmesi ile oluşturulmuştur.

Anahtar Kelimeler: deprem performansı, karmaşık yapılar, kompozit çerçeve, zaman tanım alanından hesap, dinamik analiz.

To My Family

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LIST OF ABBREVIATIONS AND SYMBOLS

OP	-	Operational Performance
IO	-	Immediate Occupancy
LS	-	Life Safety
CP	-	Collapse Prevention
ASCE	-	American Society of Civil Engineers
FEMA	-	Federal Emergency Management Agency
TEC	-	Turkish Earthquake Code
ATC	-	Applied Technology Council (Seismic evaluation and retrofit of concrete buildings)
EC8	-	Eurocode 8: Design of Structures for Earthquake Resistance
RC	-	Reinforced Concrete
SS	-	Structural Steel
fy	-	Minimum yield stress for steel.
fu	-	Maximum tensile strength of steel
Es	-	Modulus of elasticity of steel
$\mathcal{E}_{su}$	-	rapture strain of steel
$\varepsilon_{c'}$	-	Peak strain
fc'	-	Compressive strength of concrete

fctk	-	Tensile strength of concrete
Ec	-	Modulus of elasticity of concrete
Ø	-	Diameter of reinforcement
Δ	-	Delta (displacement function in nonlinear method)
δ	-	Sigma (deformation function in nonlinear method)

Chapter 1

INTRODUCTION

1.1 General

Earthquake has a special place among natural hazards considering that it happens without warning, predicting the exact time and place of earthquake is not possible up till now. It is well known that, Turkey and Cyprus are in the zone of high seismic activity. The seismic effects should be considered by the Engineers in their structural design according to design codes. Specified earthquake codes make engineers to design safe structures. Codes are classified according to the earthquakes in approximate magnitudes of them. Using this approach; there will be no structural or nonstructural damage in minor earthquake, repairable damage on the structural or nonstructural elements in medium scale earthquake, at least the life safety limitation in accordance to structural elements in major earthquake (TEC 2007).

Concrete and steel are different types of material according to their characteristic behavior. Concrete and steel are combined and used in the buildings as structural members. Steel itself is rolled in factory and used as structural member as well.

Traditionally reinforced concrete and steel framing are two common types of building framing systems used for many different types of structures in the region. Over the years, reinforced concrete framing has been used extensively all around the world.

During the second half of the 20th century, there has been an increase in the production of structural steel and nowadays it is more widely available in countries where it is produced and can easily be exported and imported by countries in need. Furthermore, the new methods of production, fabrication, transportation, erection, recyclability and the many advantages of using structural steel for structures are the main reasons of why it is becoming a more popular construction material and competitive in price when used as framing material for structures.

1.2 Problem Statement

Most people in Turkey and North Cyprus construct their own houses on their lands. But sometimes their financial condition forces them to construct their building at stages, for example, ground floor followed by first floor, etc. hoping to complete them later on (Figure 1.1).

Figure 1.1: A building appears to be incomplete, due to the starter bars left on the roof

The world's construction industry is expanding day by day and the building plots are becoming more precious. Therefore, the building permission system is changing accordingly. For instance, the building permission relating to story height limitation may change to allow construct the owners build higher buildings.

Here are possible approaches for constructing more stories on existing buildings without demolishing them; The first approach is strengthening the concrete building so that it can tolerate additional loads for new stories. This method consumes a lot of time and money and simultaneously make considerable disturbance for residents of building. On the other hand, there is an other possible method where materials that are light in weight can be used and therefore the existing building would be able to carry the new floors.

It is well known that, steel structures are generally lighter in weight than reinforced concrete ones which is an advantage in earthquake susceptible regions. The speed, quality and weight of construction are important parameters that require careful consideration. Furthermore, sometimes there is a need to build additional floors on existing reinforced concrete buildings. Since steel frame provides a lighter additional floor over existing buildings, this method is becoming popular in North Cyprus, in particular for investors (Figure 1.2).

Figure 1.2: One of the application; first four stories made of reinforced concrete and upper two stories made of structural steel.

Researchers are given a variety of names to these types of buildings as composite structures, mixed structures, complex structures, irregular in height structures and etc. In this research mixed frames and mixed structures are used.

1.3 Objectives and Scope

The scope of this work is to evaluate the seismic performance of mixed and regular structures. Mixed structures consist of two parts; The lower part is called primary or substructure and the upper part is called secondary or superstructure. The primary structures are made of reinforced concrete and the secondary structures are made of structural steel.

The regular structures are composed of one type of structural framing material; either reinforced concrete or structural steel. The abbreviations of the structures according to their structural framing material is (RC) for reinforced concrete, (SS) for the structural steel and (RC-SS) for the mixed structures.

Current earthquake codes (EC8 and TEC 2007) give provisions to engineers for the reinforced concrete framed structures, structural steel framed structures and masonry structures separately. In seismic design of mixed structures, current design codes do not provide clear guidelines to designers. However, there are specific recommendations in some codes, such as UBC (Uniform Building Code) and NEHRP (National Earthquake Hazard Reduction Program).

In order to evaluate mixed structures, several types of structural models are prepared, analyzed and designed. For the investigation of the seismic performance of the mixed structures and their comparison with the normal structures, the nonlinear time history analysis method is used. In time history analysis (also known as dynamic analysis) geometric and material nonlinearities are considered separately as one variable. Three measured real earthquake data is applied to the six analytical models. Three types of framing systems are used, which are reinforced concrete, structural steel and mixed structures (combination of concrete and structural steel). For dynamic analysis, FEMA 356 procedures are used extensively.

1.4 Outline of Thesis

This thesis is composed of six chapters and three appendices. Chapter 1 gives general information about study and problem statement. Chapter 2 includes the previous research and analysis methods. In chapter 3 some information is given about methodology of this research and creation of models in the computer program. Chapter 4 contains description of analytic models. Analysis and results of the structural models are given in chapter 5. Finally, the conclusion and recommendations for future work are given in chapter 6.

Chapter 2

LITERATURE REVIEW

2.1 General

As discussed in Chapter 1, current codes do not give specific provisions for seismic design of mixed structures. In this chapter the current background of seismic performance assessment procedures and development of the seismic procedure for mixed structures are covered. In order to investigate the seismic performance of structures FEMA 356 and TEC 2007 have been considered and some specific details are given. Finally, the methods that are previously developed for seismic assessment of mixed structures are briefly explained.

2.2 Earthquake Hazard Levels

Earthquake hazard levels are stated as mean return period in terms of probability of exceedance. In this approach, the probability of occurrence of earthquake types, depending on their magnitude, has been characterized in terms of earthquake hazard levels. FEMA 356 presents four different hazard levels, which are 50%, 20%, 10% and 2% in accordance to probability of exceedance in 50 years (Table 2.1). For instance, a hazard level with a 50% exceedance in 50 years has a return period of 72 years.

Table 2.1: Earthquake probability of exceedance and mean return periods(ASCE 2000).

Earthquake Having Probability of exceedance	Mean Return Period (years)
50%/50 years	72
20%/50 years	225
10%/50 years	474
2%/50 years	2475

TEC 2007 has specified three different earthquake hazard levels, with probabilities of exceedance are 50%, 10% and 2% in 50 years. The definitions of these levels are given below;

D1 Earthquake hazard level: it has the highest probability of occurrence, but is low in magnitude. The ordinate of this response spectrum would be half of the main design spectrum. The exceedance probability of the main design spectrum is %10 in 50 years.

D2 Earthquake Hazard Level: this type has moderate probability of occurrence and specifies quite strong ground motions. This level of earthquake spectrum ordinates is also used as design spectrum.

D3 Earthquake Hazard Level: this level is the most severe seismic motion that the building could face. These types of earthquakes happen very rarely and the probability of exceedance is 2% in 50 years. The acceleration spectrum is about 1.5 times bigger than the D2 design level. These earthquake levels and parameters are tabulated by Celep (2008) in Table 2.2.

Table 2.2: Earthquake effect parameters (Celep 2008).

Earthquake Type	Earthquake Affect Ratio	Probability of Exeedance in 50 years	Mean Return Period
Ready for Usage Level	≈0.50	50%	72 years
Design Earthquake	1.00	10%	474 years
Highest Level Earthquake	≈1.50	2%	2475 years

2.3 Definitions of Performance Level

Limitations on the maximum damage sustained during a ground motion are described as performance levels. Wide range of structural performances can be preferred by different building owners. The FEMA 356 presents, four main structural performance levels; Operational Level (OP) Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP).

Operational Level (OP): Very light structural damage may occur, structures substantially retains original strength and stiffness. Overall damage range is “Very light”. In this level building may be used after the earthquake.

Immediate Occupancy (IO): Very limited structural damage can occur on the structural or nonstructural elements. Overall damage range is “light”. The basic vertical and lateral force resisting systems (structural members) of the building should behave close to their pre-earthquake strength and stiffness. The structural elements may pass the range of linear elastic limit a bit. The building structure may need minor repair, but it can be used after a short time of earthquake happening. In this range the global system cannot have permanent drift.

Life Safety (LS): significant damage can occur on the structure. Overall damage range is "moderate". System may behave as in the plastic range, but strength and stiffness on all stories should be left same as before. Some permanent drift may be permitted. Some parts of structure may have partial or total structural collapse. Repairing of structure may not be economical when compared to its rebuilding.

Collapse Prevention (CP): heavy damages may occur on the structural elements. Overall damage range is "severe". In this performance level of the structure is at the edge of the collapse limit. Building structure is very close to collapse. Large permanent drifts occur at different levels of structure. However, all significant structural components must continue to carry the gravity load demands of buildings. These performance levels are summarized in FEMA 356 (APPENDIX C).

2.4 Target Building Performance Levels

Most of the buildings are designed according to their purposes of usage. This purpose can change over time. For instance, the apartment that was once designed for normal residence can be changed and used as a hospital. According to different hazard and performance levels that have been discussed above, assessment of the related buildings can be done. In TEC 2007, these performance and hazard levels are tabulated in accordance to purpose of usage the buildings (Table 2.3). The performance level aimed for the moderate hazard should be at least life safety (LS).

Table 2.3: Minimum performance levels of buildings for different earthquake hazard levels (TEC 2007).

The usage purpose and the Type of the Building	*Probability for the Earthquake to be exceeded*		
	50 % in 50 years	*10 % in 50 years*	*2 % in 50 years*
The buildings that should be used after earthquakes: Hospitals, heath facilities, fire stations, communications and energy facilities, transportation stations, provincial or district administrative bodies, disaster management centers etc.	–	**RU**	**LS**
The buildings that people stay in for a long time period: Schools, accommodations, dormitories, pensions, military posts, prisons, museums, etc.	–	**RU**	**LS**
The buildings that people visit densely and stay in for a short time period: cinema, theatre and concert halls, culture centers, sports facilities	**RU**	**LS**	–
Buildings containing hazardous materials: The buildings containing toxic, flammable and explosive materials and the buildings in which the mentioned materials are stored.	–	**RU**	**PC**
Other buildings: The buildings that does not fit the definitions given above (houses, offices, hotel, tourist facilities, industrial buildings, etc.)	–	**LS**	–

RU: Ready for Usage; **LS**: Life Safety; **PC**: Pre-Collapse (See **7.7**)

2.5 Global Level Evaluation

Assessment of structural performance covers both global level limits and member level limits which are known as, drift and plastic rotation respectively (Hueste and Bai 2007).

In order to have global assessment of buildings, FEMA 356 provides some permanent and transient drift limit values for various types of structures. In this research only steel and reinforced concrete limiting values will be considered. Transient drift limit values for the reinforced concrete framed structures are 1%, 2%, and 4% for immediate occupancy (IO), life safety (LS), and collapse prevention (CP) performance levels, respectively. For the steel moment frames transient drift values are 0.7%, 2.5% and 5% for (IO), (LS), and (CP) performance levels. The performance levels and damages are given in APPENDIX C.

2.6 Member Level Evaluation

Column and beam ends are the places that take most of the stresses during the earthquake excitation (Celep 2008). FEMA 356 provides generalized load-deformation relations and performance levels for members. IO, LS, and CP levels are defined for primary (P) and secondary (S) members on Figure 2.1.

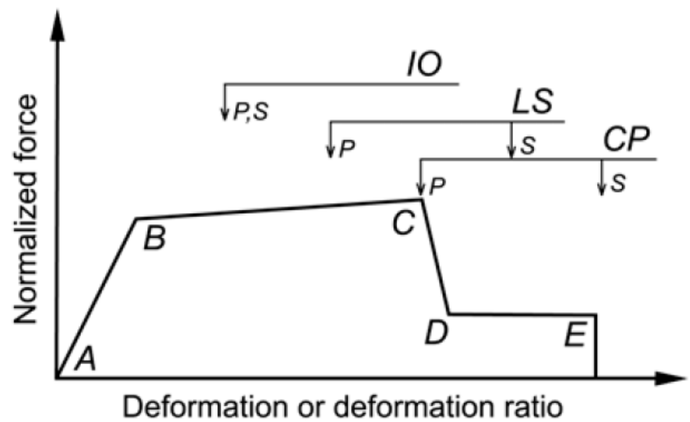


Figure 2.1: Component or element deformation acceptance criteria (ASCE 2000)

In this figure, the slope between point A and B represents the system in elastic range. After reaching point B the system behavior is in inelastic range until the point C. Point C represents the ultimate strength of material. FEMA 356 has generalized the slope between point B and C as 0-10% of elastic range. Then the strength is reduced with a sudden slope and drops to point D, and the remaining resistance continues to point E (ASCE 2000).

2.7 Earthquake Performance of Structural System

The behavior of the structural system under the earthquake excitation can be assessed through the curve developed by top displacement of the structure and base shear force. This curve appears like the behavior of a member, but this time curve is created for the whole structure's behavior (Celep 2008). The structure's performance levels (IO, LS, and CP) can be investigated on this curve.

According to the damage occurred on the sections the member can be evaluated and in the same way the evaluation of structure can be done according to the member. The evaluation should be carried out in both directions of the structure and for all the stories (Celep 2008) (Fig 2.2).

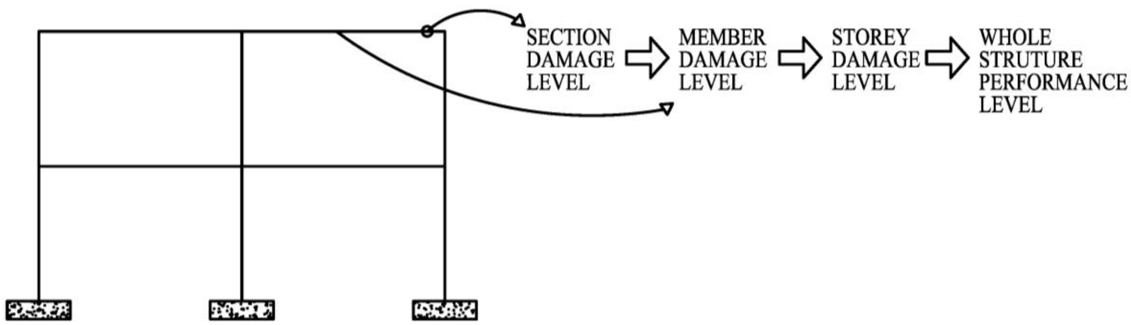


Figure 2.2: From section damage levels to structure performance level (Celep 2008).

In TEC 2007, structural performance levels have been defined as following:

IO: in each story, maximum 10% of the beam sections can be in between LS and IO limit. But the other structural members should be under the level of IO. If, so there is any brittle member under the condition making them as ductile, this building can be in assumed in IO level.

LS: in each story, at least 30% of the beams can be on the limit between LS and CP, except the secondary members. For the columns, in the level between LS and CP of members for each story, the effect of shear forces should not stay any lower than 20% of the whole shear forces. For members between LS and CP levels, the total shear forces of columns on top story can be maximum 40% of the related story shear forces of all columns. The other structural members should be in the level below IO or between IO and LS limits.

In addition, reaching to the damage level of two ends of columns is assumed meaningfully dangerous. This damage can create the “story mechanism” on structure. For the condition of brittle members, the member can be assumed to be in the LS limit by updating it as a ductile member.

CP: in each story, at least 20% of the beams can be beyond the CP limit, except the secondary members. All other structural members are below the IO limit, between IO and LS and LS and CP. However, if any column passes the limit of IO then the shear force of this column should not exceed 30% shear capacity of all the columns of the related story. The level of CP building is problematic from LS aspect.

2.8 Collecting information from Buildings

For the assessment of the existing structures, collecting information from the buildings is the basic stage. Collected information certainty will lead to more realistic results in the assessment. Most of the well-known design codes and procedures, including FEMA 356, have instructions about collecting information from the existing structures. This is defined as knowledge level under the codes.

In performance evaluation of existing structures, data collection is the first and one of the most important step for the evaluation of structures capacity. These data will generally include geometrical information of the structures, foundation details, ground properties, damages occurred to the structural elements if available and also material characteristic properties (TEC 2007).

The knowledge level is characterized as chief factor and is described as minimum, usual and comprehensive levels. These data collection requirements and conditions are shown in APPENDIX C. The knowledge factor is used to calculate the structural section capacities.

2.9 Performance Analysis Methods

Analysis approach can be defined most broadly as linear or nonlinear, depending upon how structure responses to the loading (CSI 2009).

2.9.1 Linear Elastic Systems

Foundation of nonlinear analysis is set on linear elastic analysis method, and most of the recent seismic codes and specifications are based on linear elastic analysis theory (Lee et al. 2004). In the linear system, the relationship between the lateral force and deformation is linear and structure involves the solution of the system of linear equations (Equation 2.1):

$$fs = ku \tag{2.1}$$

Where; k is the stiffness of system, u is the displacement of system and fs is the external force. The linear force displacement relation is shown in Figure 2.3.

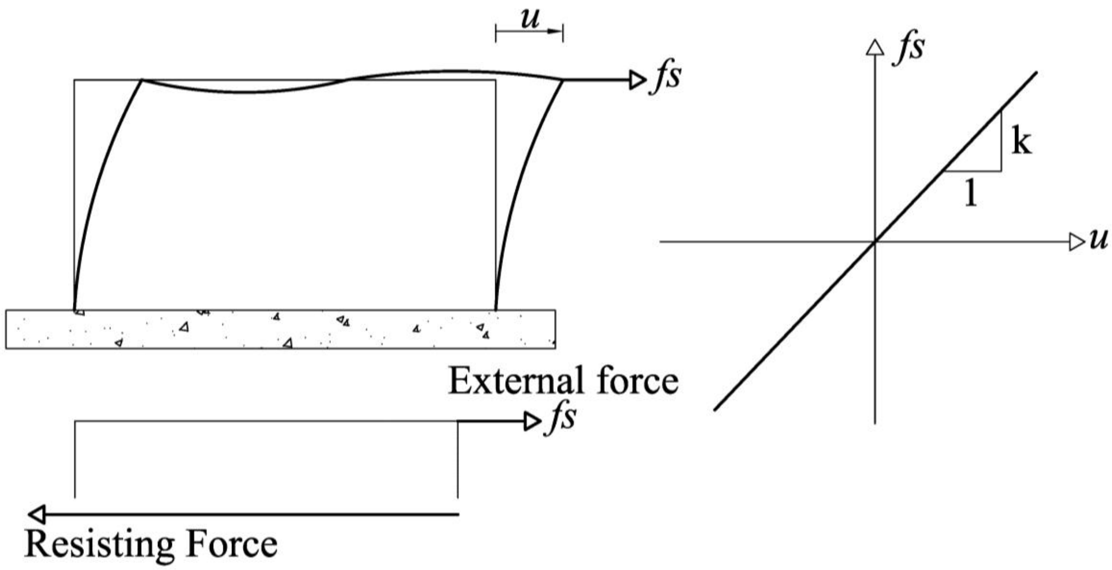


Figure 2.3: Force-deformation relation in linear elastic system (Chopra 2007)

TEC 2007 presents two linear elastic methods: Equivalent Force method and Modal Analysis (Response Spectrum) method.

1. Equivalent Force Method: This method is useful in low rise structures while only one mode effect is in consideration (Celep 2008). It is the only method that can be handled by hand calculation (Lee et al. 2004). This method depends on the calculation of the base shear force and its distribution to the stories.
2. Modal Analysis (Response Spectrum) Method: In this method the internal forces and displacements are calculated separately for each mode. This method depends on the superposition of the mode shapes. These modes help engineers to understand the realistic behavior of the structure under the earthquake excitation.

2.9.2 Nonlinear Inelastic System

Almost all materials have nonlinear characteristic properties (Celep 2001). The force-displacement relations behave linear at small deformations, but it would become nonlinear at large deformations. Accordingly first loading curve is nonlinear at large deformations and the unloading and reloading curves differ from the initial loading, such systems are said to be inelastic. In this approach force corresponding to deformation is not valued individually and depends on the increase or decrease of history of deformations (Chopra 2007).

It is well known that many buildings are designed with the expectation of inelastic behavior. In this type of analysis method, a more realistic structural behavior can be developed. The irregularity in the structures is completely affected by the analysis results when compared with the linear methods (Celep 2008).

There are two analysis methods available in literature for nonlinear analysis, one is nonlinear static (known as Pushover) analysis and the other one is nonlinear dynamic analysis.

2.9.2.1 Nonlinear Static Analysis (Pushover)

Nonlinear static analysis is the most used method to get the seismic performance of structures. This method is based on meeting the lateral force carrying capacity with the earthquake demand and to find the performance point of the related structure (Celep 2008).

In this analysis method material and geometric nonlinearities can be used to perform the nonlinear response of structures (CSI 2009).

In the mentioned method, increased force function is expressed either in terms of horizontal forces or displacements which are applied to the lateral action-resisting system. In order to simulate the inertia forces and the effect of them, static forces or displacements are distributed along the height of the structure (Elnashai and Di Sarno 2008).

The increased force functions are applied to the structure until the structure capacity fails. The capacity (pushover) curve is obtained from control node displacement and the base shear force function together (Figure 2.4) (Elnashai and Di Sarno 2008). This displacement control node shall be located at the center of mass at the roof of building (ASCE 2000).

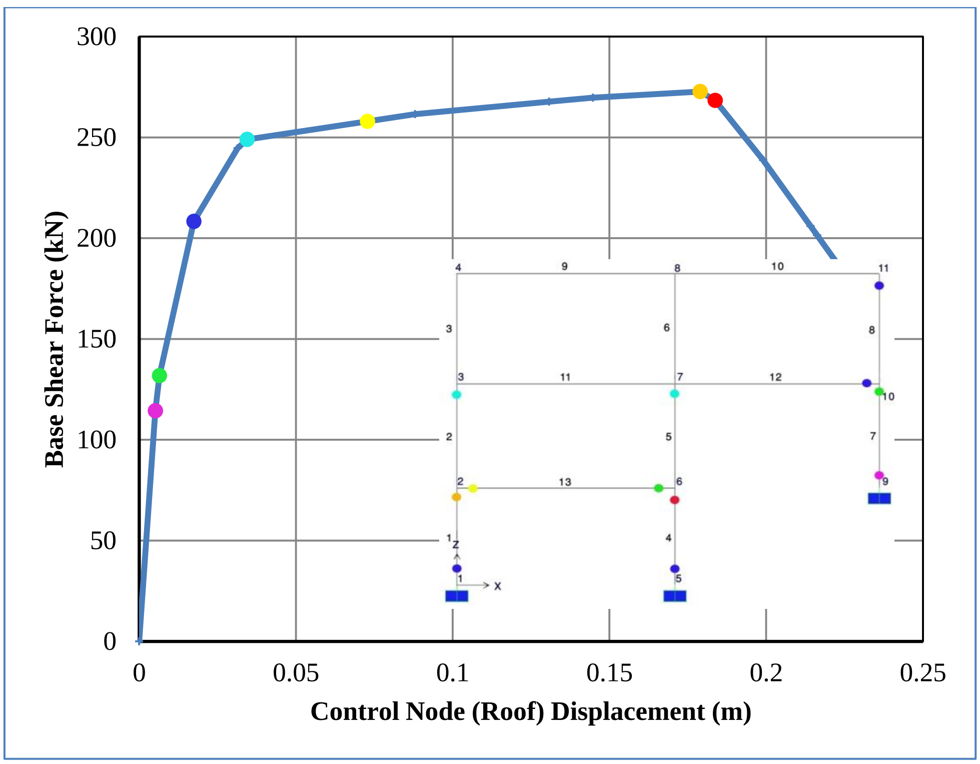


Figure 2.4: Example of pushover curve

The pushover analysis can be used either for one or multiple modes. Pushover analysis has developed two main types of methods, conventional and adaptive pushover. The main difference between these methods is that the conventional method uses only one mode shape and keeps displacement or load pattern constant, but in the conventional method the load patterns are changing in order to adapt to the structures mode shapes (Elnashai and Di Sarno 2008).

In order to have the performance relation under the pushover analysis some methods are developed; namely ATC40 capacity spectrum method (CSM) and the FEMA 356 displacement coefficient method (DCM).

2.9.2.2 Nonlinear Dynamic Analysis

The design codes based on equivalent elastic force approach are proved unable to prevent the damages of strong earthquakes. After some major earthquakes like Kocaeli 1999 and Northridge1994, there was a need for developing more accurate methods in order to investigate geometrical nonlinearities and material inelasticity on seismic demand on structures. Therefore, the dynamic time history analysis method was developed to investigate the response of the structures within the real ground motions (Pecker 2007).

Nonlinear dynamic analysis, also known as time history analysis, requires a step by step process to find the dynamic response of a structure to specified acceleration algorithm (CSI 2009). The step sizes are important parameter to have more accurate results.

By providing proper approximations and modeling, the nonlinear time history analysis can be a very powerful tool to find the performance of existing structures. Nonlinear Time History analysis is widely known as an accurate way for simulating the response of structures under earthquake excitations.

This analysis method is the most complex and probably the most time-consuming method according to the choice of integration time steps and geometry of the structures (Pecker 2007).

In addition, Elnashai and Di Sarno (2008) mention that the most natural approach toward the assessment of earthquake response is nonlinear time history analysis. On the other hand, it can be more challenging than static analysis since it needs more computational effort and interpretations for results.

In this analysis method, real ground motions, accelerations are applied to the structure in terms of time. Number of variables and parameters that are considered in time history method requires careful engineering knowledge. The selected ground motions shall be similar to the design earthquake spectrum that is given in the earthquake codes. In order to have more realistic approach, the number of used ground motions shall be kept as high as possible (Celep 2008).

2.10 Selecting ground motions

TEC 2007 and FEMA 356 provide some recommendations for selecting the ground motion records. Both of these standards state that, time history analysis shouldn't be performed with less than three data sets. According to codes selected, ground motions shall be scaled according to desired earthquake spectrum level. If there are three data sets, the maximum of the results can be used to determine the design acceptability, in case of seven ground motions, the average of the results shall be used.

The pacific earthquake engineering research center (PEER) of university of Berkley, California, has been providing major earthquake records on their website. In this website the ground motions can be downloaded as original data set or it can be scaled according to desired target spectrum level (P.E.E.R 2010).

2.11 Damping in Structures

Damping is one of the processes of steadily diminishing in amplitude of vibration. In damping, the kinetic and strain energy of vibration system is dissipating by various mechanisms. In real structures, mechanisms can have more than one variable. The friction at steel connections, opening and closing of micro-cracks in concrete, and friction between structures, such as partition walls effects can be included as mechanism. Therefore, it is impossible to identify or describe mathematically, the types of energy dissipation mechanism in real structures (Chopra 2007).

Consequently, damping in real structures is usually represented in a highly idealized way. In Chopra's book "Dynamics of Structures", the linear viscous damper is subjected to a force f_D along the DOF u (Figure 2.5).

$$f_D = c\dot{u} \tag{2.2}$$

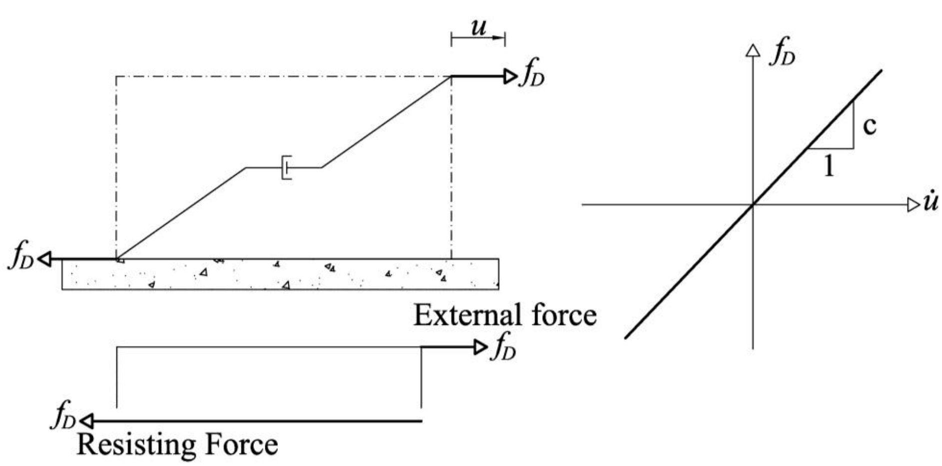


Figure 2.5: Linear damping in structure

The classical damping in linear system is generally specified by numerical values for the modal damping ratios. The experimental data gives the recommended value for the modal damping ratios (Table 2.4). For linear analysis with non-classical damping and for the nonlinear analysis of structures, the damping matrix is needed (Chopra 2007).

Table 2.4: Recommended damping values (Chopra 2007).

Stress Level	Type and Condition of Structure	Damping Ratio (%)
Working Stress, no more than about 1/2 yield point	Welded steel, pre-stressed concrete, well- reinforced concrete (only slight cracking)	2-3
	Reinforced Concrete with considerable cracking	3-5
	Bolted and/or riveted steel, wood structures with nailed or bolted joints	5-7
At or just below yield point	Welded steel, pre-stressed concrete (without complete loss in pre-stress)	5-7
	Pre-stressed concrete with no pre-stress left	7-10
	Reinforced concrete	7-10
	Bolted and/or riveted steel, wood structure with bolted joints	10-15
	Wood structure with nailed joints	15-20

2.12 Development of Analysis Methods for Mixed Structures

Many researchers have been trying to develop new methods for seismic analysis of mixed structures (which are attached to top of the existing buildings) during the last few decades.

The starting point on the theory of secondary structures is dependent on nonstructural component behavior and damages. After occurrence of many major earthquakes, the need to avoid the nonstructural component failure was understood. These failures considerably affect the total cost of damage. To prevent these damages, it is important to have a proper understanding of the seismic behavior of secondary systems (Lin and Mahin 1985).

Most general approach for the purpose of analysis and design of secondary or complex structures can be included along with the supporting structure in the analytical model to allow evaluation of the time history response to ground motions (Lin and Mahin 1985).

Most of the codes (TEC 2007, IBC, and EC8) do not give provision for seismic analysis of those kinds of structures, which have different framing systems according to their material type.

As it is mentioned before in Table 2.4, the recommended damping ratios in elastic systems are used in analysis of structure. Typically 5% damping ratio is being used in reinforced concrete structural systems and 2% for the steel structures. Many design engineers use overall conventional damping ratio of 2% for mixed structures in order to be on safe side (Papageorgiou and Gantes 2010b).

(Villaverde 1997) has presented different existing analysis methods and their related code provisions. These methods can be grouped in two main categories, decouple and couple approach. In decouple approach two sub systems are modeled separately and with their different damping ratios. This approach neglects the interaction between two parts. On the other hand, in the coupled approach, the whole building is modeled together and the non-uniform damping ratios are reflected in the structure model as it is shown in Figure 2.6 (Papageorgiou and Gantes 2010b).

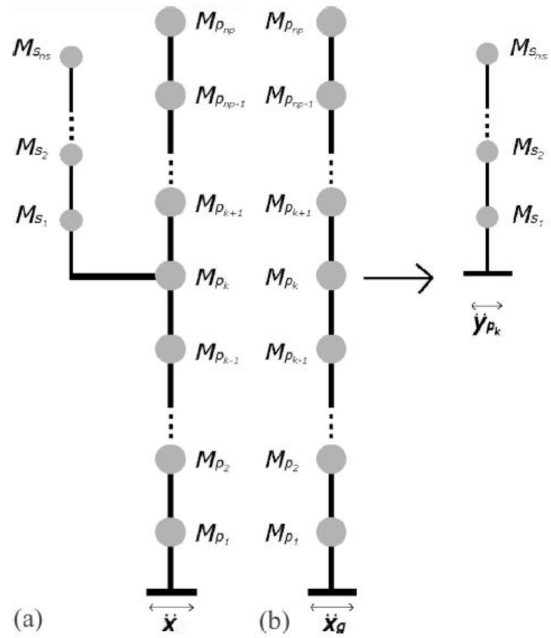


Figure 2.6: (a) coupled, (b) decoupled analysis procedures (Papageorgiou and Gantes 2010a).

The coupled approach may avoid the decoupling errors, but it has some difficulties since the formulation of the irregular damping matrix is a procedure not supported by computer programs and results in the complexity of eigenvalues (Papageorgiou and Gantes 2010b).

Papageorgiou and Gantes (2010) have studied the performance of structural response with equivalent modal damping ratios to have approach in irregularly damped structures. In this approach the irregular multi degree of freedom (MDOF) system is converted to the 2DOF system as shown in Figure 2.7.

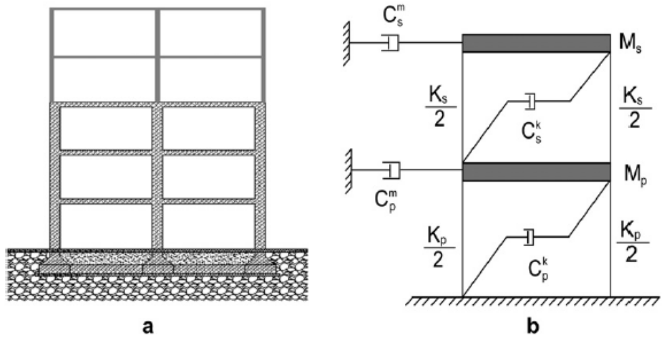


Figure 2.7: a) MDOF irregular structure b) and equivalent 2-DOF structure (Papageorgiou and Gantes 2010b).

(Chen and Soong 1994) have studied the energy based dynamic analysis of secondary systems. This approach provides simple and consistent response analysis of secondary systems. In this method, the practical coupled analysis in the modal space is presented for MDOF primary secondary systems in which dynamic response of the secondary system is calculated from modal properties of primary secondary systems.

(Lee et al. 2004) has worked on the assessment of the comparable damping ratios of structures with added supplemental damping devices to assess the vibration effect quantitatively.

(Lai and Soong 1991) have studied the seismic design consideration for secondary structural systems. In this research the design procedure is developed by examining the behavior of the relative displacement and absolute accelerations of case study building as the functions of parameters stiffness and damping ratios.

Various methods are being developed in order to evaluate the seismic response of these types of structures (mixed or complex or secondary) in a simpler, more effective way. Until now these studies are not reflected in design codes, however, some of the essential codes have given brief recommendations for seismic design of mixed structures. Therefore, more research, study and data collection is needed in this area. Furthermore, nonlinear dynamic time history analysis method is still the best method to perform realistic behavior of mixed structures when compared to other methods. In this research nonlinear dynamic time history analysis method will be used to evaluate the behavior of six building frames having three types of structural framing elements.

Chapter 3

NONLINEAR TIME HISTORY ANALYSIS METHOD

3.1 General

In order to investigate the seismic performance of structures, most rigorous and realistic method is the fully nonlinear time history analysis (Papageorgiou and Gantes 2010a). In this chapter the application of this method in SAP2000 and the concept of nonlinearity are explained as well as evaluation procedure.

3.2 Nonlinearity Concept

Nonlinear structural behavior can be investigated under geometric or material nonlinearities. Geometric nonlinearities directly depend on the global structural deformation. Geometric nonlinearities generally are defined with two forms; these are P-δ (member curvature) and the P-Δ (chord rotation) effect (Figure 3.1).

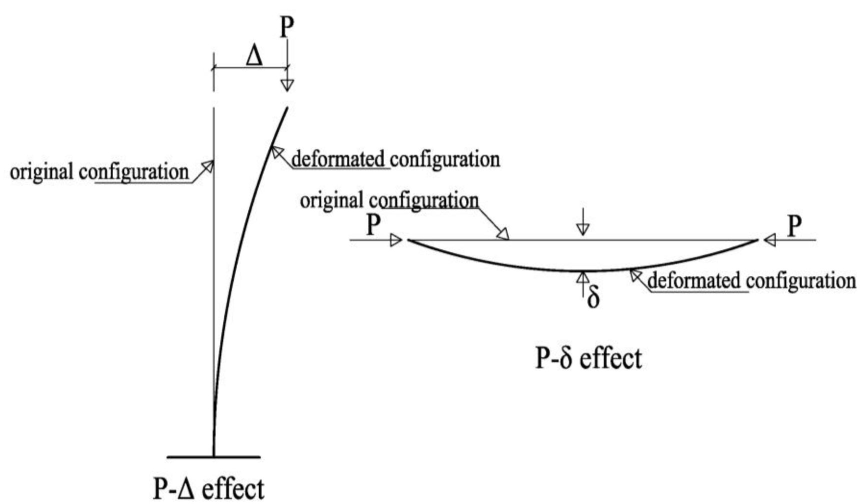


Figure 3.1: P-Δ and P-δ effects (Li 1996).

P-Δ effect is directly related to the flexural or lateral stiffness of the structure. This effect is caused by side sway of system. P-Δ effect creates the additional overturning moments to the structure and this effect reduces the flexural stiffness of elements and system. The P-δ effect can be caused by the side sway and non-side sway in element (Li 1996). P-Δ effect is mostly related to the compression member and it has a great role in overall stability of structures. This effect should be considered in analysis. However, in this report the P-Δ effect will be one of the variables in analysis option.

It is well known that, material's stress-strain relations generally have nonlinear behavior. Material nonlinearities are subjected to the nonlinear behavior of members, according to materials stress-strain relation (Figure 3.2). This behavior can be investigated by single degree of freedom or multiple degree of freedom consideration. However, in this study only one dimensional or one degree of freedom (Flexural-M3) inelastic behavior of material is used. The inelastic behavior of members should be investigated under loading and unloading paths (Celep 2008).

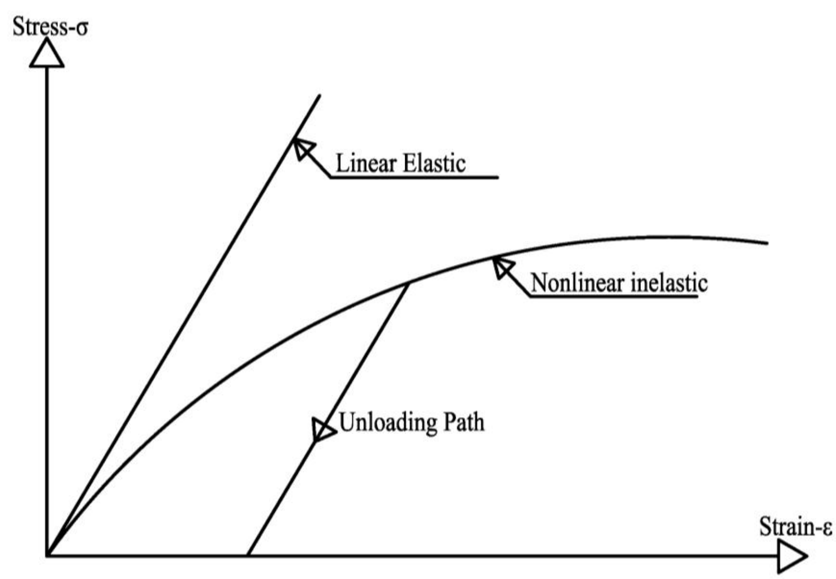


Figure 3.2: Elastic and inelastic material behavior (Li 1996).

The nonlinear structural behavior depends on nonlinear responses of the elements and it is greatly complicated. Plastic hinge is the term that refers to nonlinear response of the structural member. Plastic hinge location and its state affects the structural behavior correspondingly. Before starting the nonlinear analysis, nonlinear behavior of structural elements should be investigated and described with loading and unloading paths.

3.3 Nonlinear Time History Analysis

Nonlinear time history analysis is the method to have nonlinear behavior of building structures depending on the real ground motions. This analysis method is quite different from the other approximate analysis methods. The internal forces, plastic rotations and displacements of the building structure are directly determined from the ground motions. All responses of the building, deformations and forces are developed as a function of time, considering the nonlinear properties of the building structure.

The general dynamic equilibrium equation can be written as:

$$Ku(t) + C\dot{u}(t) + M\ddot{u}(t) = r(t) \tag{3.1}$$

Where, K is the stiffness matrix; C is the damping matrix; M is the diagonal mass matrix; u, $\dot{u}$, and $\ddot{u}$ are the displacements, velocities and accelerations of the structure; and r is the applied dynamic load.

In nonlinear dynamic analysis, the stiffness, damping and load may all depend upon the displacements, velocities and time. This type of relation requires iterative approximation to the equations of motion (CSI 2009).

There are different options to calculate the dynamic response of structures. The solution method can be modal or direct integration.

3.3.1 Modal Time History Analysis

Time history analysis by modal superposition is a method using combination of modal responses to eliminate the difficulties in dynamic calculation. Seismic responses of structures can be characterized by some important lateral deformation modes. Most of the time these lateral deformations are reflecting first fundamental modes as shown in Figure 3.3 (Li 1996).

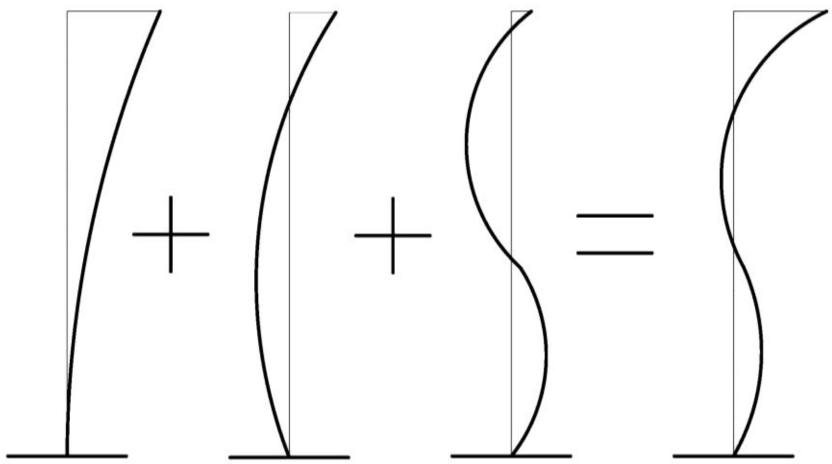


Figure 3.3: Superposition of mode shapes (Li 1996).

3.3.2 Time History Analysis by Direct Integration

Nonlinear time history analysis by direct integration is the most accurate available analysis method. In the direct integration method the system solves the equations for the entire structure at each divided step. The divided time step sizes are extremely sensitive in results of analysis. SAP2000 manual states that users should run their analysis with decreasing time step sizes until the results are not affected any more. However, in this study time history analysis by direct integration method is used in the analysis of analytical models. Modeling procedure and parameters are summarized with step numbers in the following sections.

3.4 Creation of Structural Models

Various computer programs with nonlinear analysis capabilities can be used to perform dynamic time history analysis. It is well known that, SAP2000 is the most frequently used structural analysis software. In this thesis, SAP2000 v14 program is used to calculate the dynamic responses of structures. For this case several structural models have been developed and subjected to specified ground motions.

The frame joints and members investigated are numbered and illustrated in Chapter 4 (Analytical Models) under the name of "Structural Model". The dimensions of the beams and columns for all models and members have been tabulated in Chapter 4 as well. The following steps are included in the nonlinear time history by direct integration analysis application in SAP2000 software.

Step 1: Creation of Computer Model

The basic computer model (without the nonlinear data) is created in the usual manner.

The material characteristic properties, geometries, loads, constraints to joints and mass sources of the structure are defined.

Step 2: Moment-Curvature Relationship

In the nonlinear time history analysis sections nonlinear behavior (hinge properties) should be defined as proper step sizes. Idealized moment-curvature relation should be integrated to the structural model (Figure 3.4).

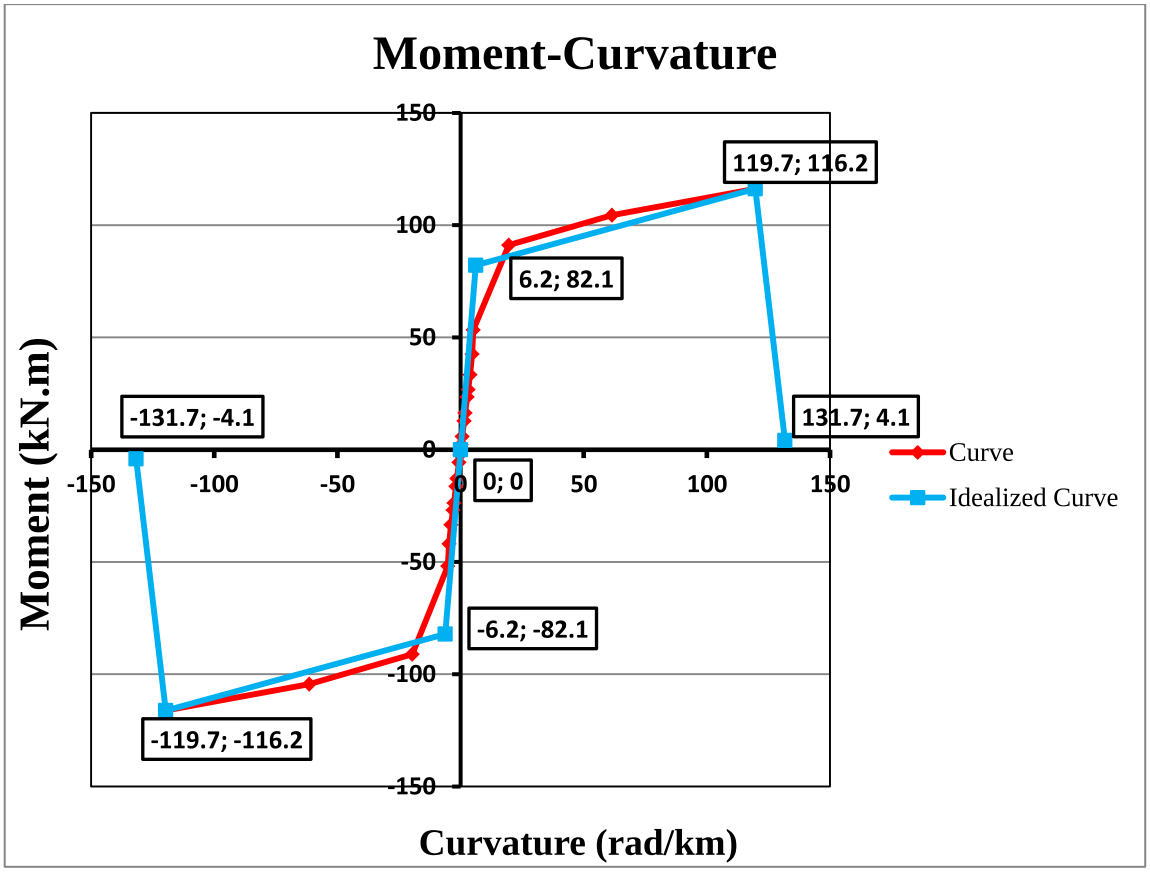


Figure 3.4: Idealize moment-curvature relation for RC beams.

SAP2000 allows only four points to define the hinge properties. The developed moment-curvature relations should be idealized to have adaptation to the structural program. In this RC beam section, used moment-curvature relation is symmetric for loading and unloading path, because of the selected section reinforcement details. However, the idealized moment-curvature relations have to be scaled according to its yield moment and curvature. This means, when the corresponding moment (M) reaches to yield moment (M_y) the behavior scale factor is equal to 1 at point "B" Figure 3.5.

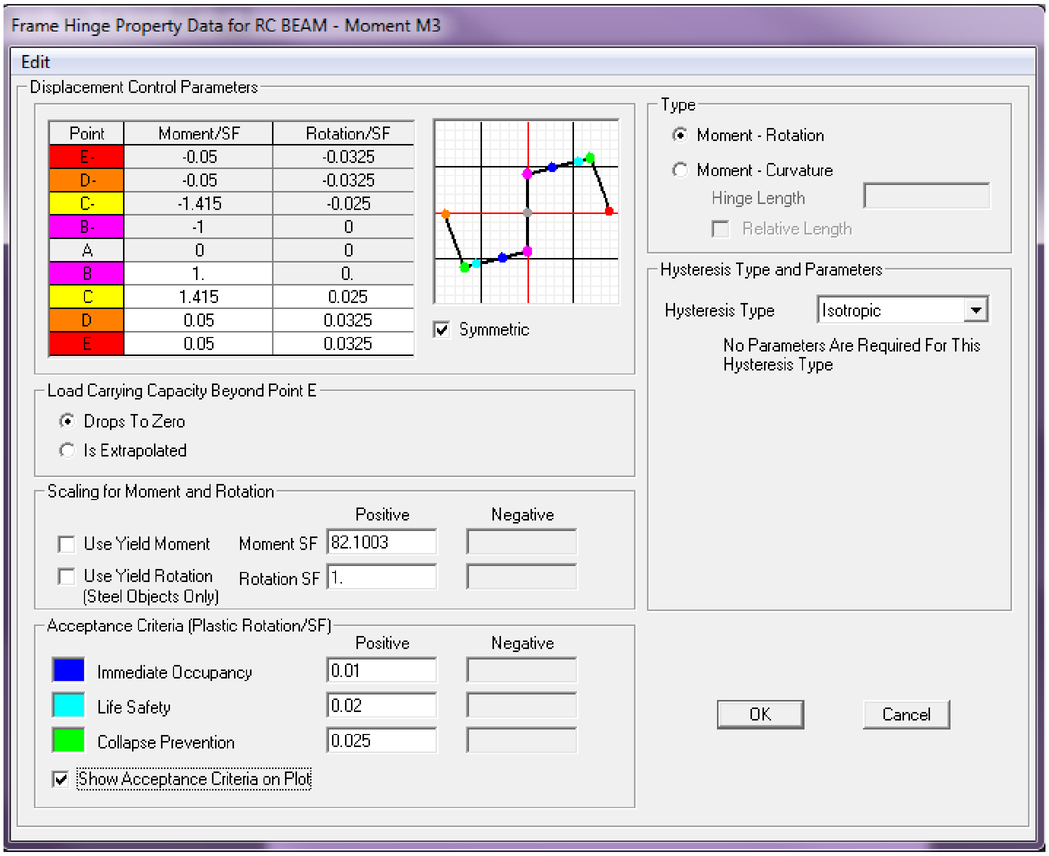


Figure 3.5: Defining hinge properties.

SAP2000 has adopted most of the effective codes. The program includes several built-in default hinge properties that are based on average values from FEMA 356 for concrete and steel members. However, for assignment of hinge properties of concrete sections the moment-curvature relations developed were used and for the steel sections FEMA 356 modeling and performance parameters were used (APPENDIX C).

In determination of moment-curvature relation for the RC column members, the static axial forces were considered. FEMA 356 refers to load combination for the component gravity loads to have an approximate approach. These combinations are specified with two conditions. First one; when the effect of gravity and seismic load is additive use Equation (3.1). Second one; when the effect of gravity and seismic load is counteracting use Equation (3.2).

$$Q_G = 1.1(Q_D + Q_L + Q_S) \quad (3.2)$$

$$Q_G = 0.9 Q_D \quad (3.3)$$

Where:

Q_G = Component gravity load

Q_D = Dead load (action)

Q_L = Effective live load (action)

Q_S = Effective snow load (action)

According to these combinations, the axial loads for each member are tabulated in Chapter 5. These axial loads will be used to calculate the moment-curvature relation for the column members.

All sections moment-curvature relations are developed and illustrated under the conditions given in APPENDIX A.

Step 3: Defining the Time History Function from File

The selected ground motions data should be defined to the program which is subjected to ground acceleration versus time (Figure 3.6).

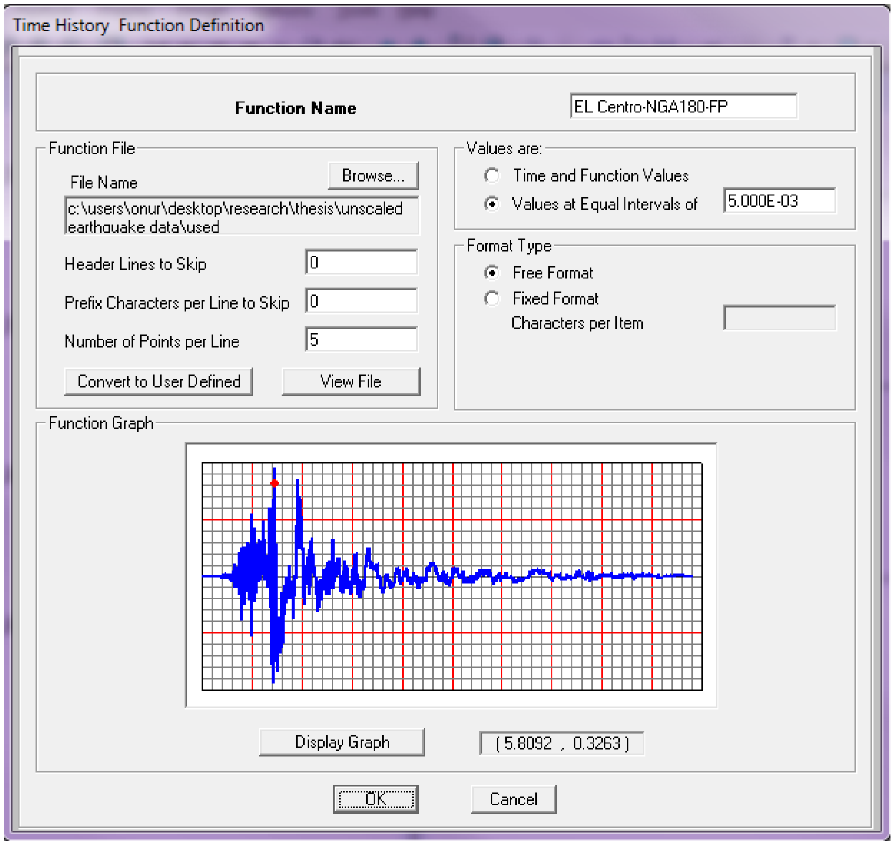


Figure 3.6: Defining time history function.

Step 4: Defining the Time History Load Case

The load case and the related parameters should be described in order to perform nonlinear time history analysis (Figure 3.7).

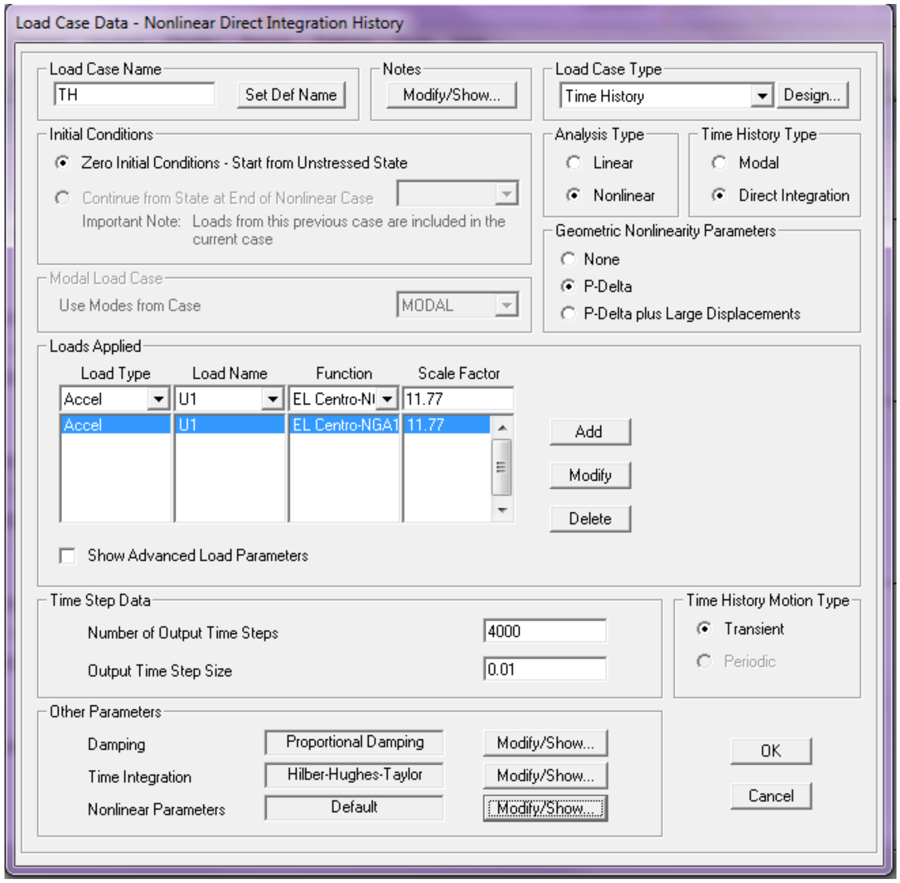


Figure 3.7: Time history load case in SAP2000.

In this analysis, time is one of the variables and step sizes are one of the critical parts to have more accurate analysis results. Total time of the analysis is multiplication of the output time step size and number of step sizes. In these analysis output time step sizes have been kept as 0.01 for all models. The scale factors for selected ground motions should be multiply with 9.81 m/s^2 as the g unit. P-Δ effect is one of the variables in this analysis and it will be included in the analysis part.

3.5 Evaluation of Analysis Results

FEMA 356 criteria were used in evaluation of seismic performance of case study models. FEMA 356 provides the analytical procedures and criteria for seismic evaluation and rehabilitation of buildings. As mentioned before in Chapter 2, structural performance levels in FEMA 356 include; immediate occupancy (IO), life safety (LS), and collapse preventions (CP). According to these performance levels, both global level limits (drift) and member level (plastic rotations) limits, building structures can be evaluated and assessed for structural performance (Hueste and Bai 2007).

3.5.1 Global Level Evaluation

Limiting drift values are given by FEMA 356 to evaluate the seismic performance of building structures as approximate values. The specified inter-story drift ratios were given in Chapter 2 and the limiting values are changing according to structural type. However, our structural types will be including mixed concrete and steel structural type and the drift ratios will be used separately according to structural members.

Inter-story drift ratios are defined by FEMA 273 as “The relative horizontal displacement of two adjacent floors in a building. Inter-story drift can be expressed as a percentage of the story height separating the two adjacent floors”. According to this approach each story level relative displacement values will be evaluated in terms of story height and performance limit values.

3.5.2 Member Level Evaluation

FEMA 356 presents member level criteria for three different performance levels, which are IO, LS, and CP. To have member level evaluation, FEMA 356 has characterized the plastic rotations and moment capacities ratios by combining various test results. According to test results, specific tables are created and presented to the user. SAP2000 has adopted itself with these tables by members of steel and concrete. The evaluation will be done considering the limitations.

3.5.3 Expected Seismic Behavior of Structures

It is almost impossible to resist all earthquake forces without any deformation or damage and even if this behavior is desired, the structural sections will be extremely huge and it will not be economical. Most of the current design codes provide the engineers with tools to design ductile structures with reduced earthquake forces by deformation and elastic or inelastic rotations. To have desired ductility performance, the places of plastic formations should be selected carefully. According to this approach, place of the expected plastic formations is desirable to be in beam's end sections, but not in the columns ends (Figure 3.8).

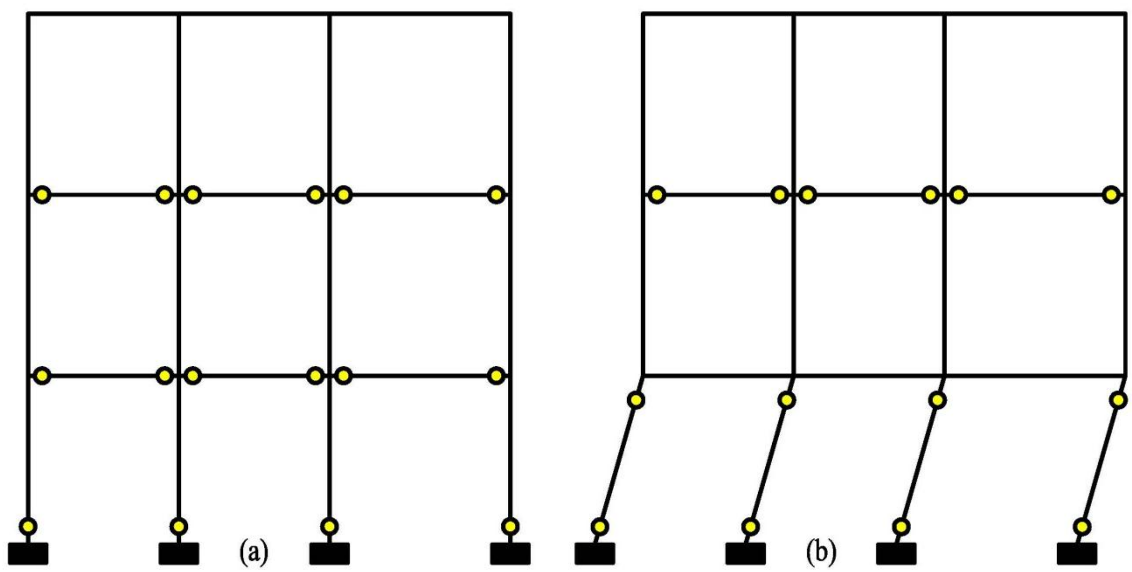


Figure 3.8: Positions of plastic hinges in different frame systems. (a) Desired hinges formation, (b) not desired hinges formation, first story mechanism (Aydınoğlu et al. 2009).

There are two reasons for this state. First is that the beam section normally do not carry much axial forces therefore it has more ductile behavior. For the columns the axial forces are directly affecting the ductility of the sections. Consequently, desired plastic formations will be in the beam's sections. It is not desired to have hinges on the column's top and bottom parts, because it creates one of the most dangerous problems for structure called "story mechanism". However, forming hinges on the bottom face of columns in contact point with foundation cannot be fully prevented. These formations do not create any stability problem for structure (Aydınoğlu et al. 2009).

In order to prevent "story mechanism" most of the codes have made the strong column-weak beam rule compulsory for the structure designs. Nevertheless, these conditions will be more investigated in the evaluation part of analysis and results chapter in this study.

Chapter 4

ANALYTICAL MODELS

4.1 General

The nonlinear time history analysis method is applied to different types of structural models. The main difference between the structural models is that they are made of structural steel and reinforced concrete. In this chapter, analytical models of structures are described in detail. The first two models were designed according to Turkish and Eurocodes. The rest of the models were formed as if they are the combination of the first two models.

Each model in this study is named according to the structural element type and its number of stories. For example model name “RC3” refers to the model that has reinforced concrete frame and 3 stories. Another example is for the mixed structure, with a model name “RC1SS2”. This means that the first floor is built by using reinforced concrete and the two floors above are built by using structural steel. While discussing the results of the study these model names would help to better understand the model type. The properties of the models investigated are shown in Table 4.1.

4.2 Description of the Frames Designed

Two types of structural frame models were analyzed and designed. The first one is 3-story high fully reinforced concrete (RC) and the second one is 3-story fully structural steel (SS) moment framed buildings. The combination of these frames will also be included in the following section.

The case study buildings were designed according to Turkish and Eurocodes. The 3 story reinforced concrete (RC) and structural steel (SS) buildings have moment frame system, specially designed and detailed for ductile behavior. For simplicity the floor system is assumed to be either pre-cast or in-situ solid slab has a thickness of 150 mm. Figures 4.1 and 4.2 show the structural models and their details.

Geometries of structures have been kept as same. The spans of the longitudinal frames are equal to 5 m, while the story heights are same, 3 m. The RC frames sections have 250x500mm dimensions. Reinforcement details of the sections are tabulated in Table 4.2. For the SS type, the columns and beams were selected as HEB180 and IPE240, respectively. The beam to column connections were designed as fully restrained moment connections.

The materials that are used in the structures were selected as C20 for concrete and S420 for reinforcement and for structural steel S275. These types of materials are the most commonly used materials in Turkey and North Cyprus. More detailed materials characteristic properties are given in Table 4.3.

In order to achieve more realistic results two types of frames have been designed under the same load conditions. In the design part only dead, live and earthquake loads were considered. The dead load assumed without considering the self-load of the solid slab is

$2kN/m^2$ and the live load is $2kN/m^2$. Normal weight concrete has been selected and the density is assumed $24kN/m^3$ for all types of concrete. For the earthquake loads the Response Spectrum method has been used. The structures assumed as in the first degree earthquake zone and ground class is Z1 (TEC 2007).

Table 4.1: Structural details of models.

Model Name	First Story		Second Story		Third Story		Forth Story	
	Sections		Sections		Sections		Sections	
	Columns (mm)	Beams (mm)	Columns (mm)	Beams (mm)	Columns (mm)	Beams (mm)	Columns (mm)	Beams (mm)
RC3	250x500	250x500	250x500	250x500	250x500	250x500	N/A	N/A
SS3	HEB180	IPE240	HEB180	IPE240	HEB180	IPE240	N/A	N/A
RC1-SS2	250x500	250x500	HEB180	IPE240	HEB180	IPE240	N/A	N/A
RC4	250x500	250x500	250x500	250x500	250x500	250x500	250x500	250x500
SS4	HEB180	IPE240	HEB180	IPE240	HEB180	IPE240	HEB180	IPE240
RC1-SS3	250x500	250x500	HEB180	IPE240	HEB180	IPE240	HEB180	IPE240

Note: N/A refers Not Assigned

Table 4.2: Reinforcements details for concrete members.

Reinforcement Details				
Beams			Columns	
Straight	Top	Stirrup	Longitudinal	Stirrup
3Ø14	3Ø14	Ø8/10	8Ø18	Ø8/10

Table 4.3: Material characteristic properties.

Material Characteristic Properties					
St37 Structural Steel		C20 Concrete		S420 Reinforcement	
fy (MPa)	240	*fc'* (MPa)	20	*fy*	420
fu (MPa)	370	*fctk* (MPa)	1.5	*fu*	630
Es (GPa)	200	Ec(GPa)	28.5	Es(GPa)	200
ε_{su}	0.2	$\varepsilon_{c'}$	0.002	ε_{su}	0.1

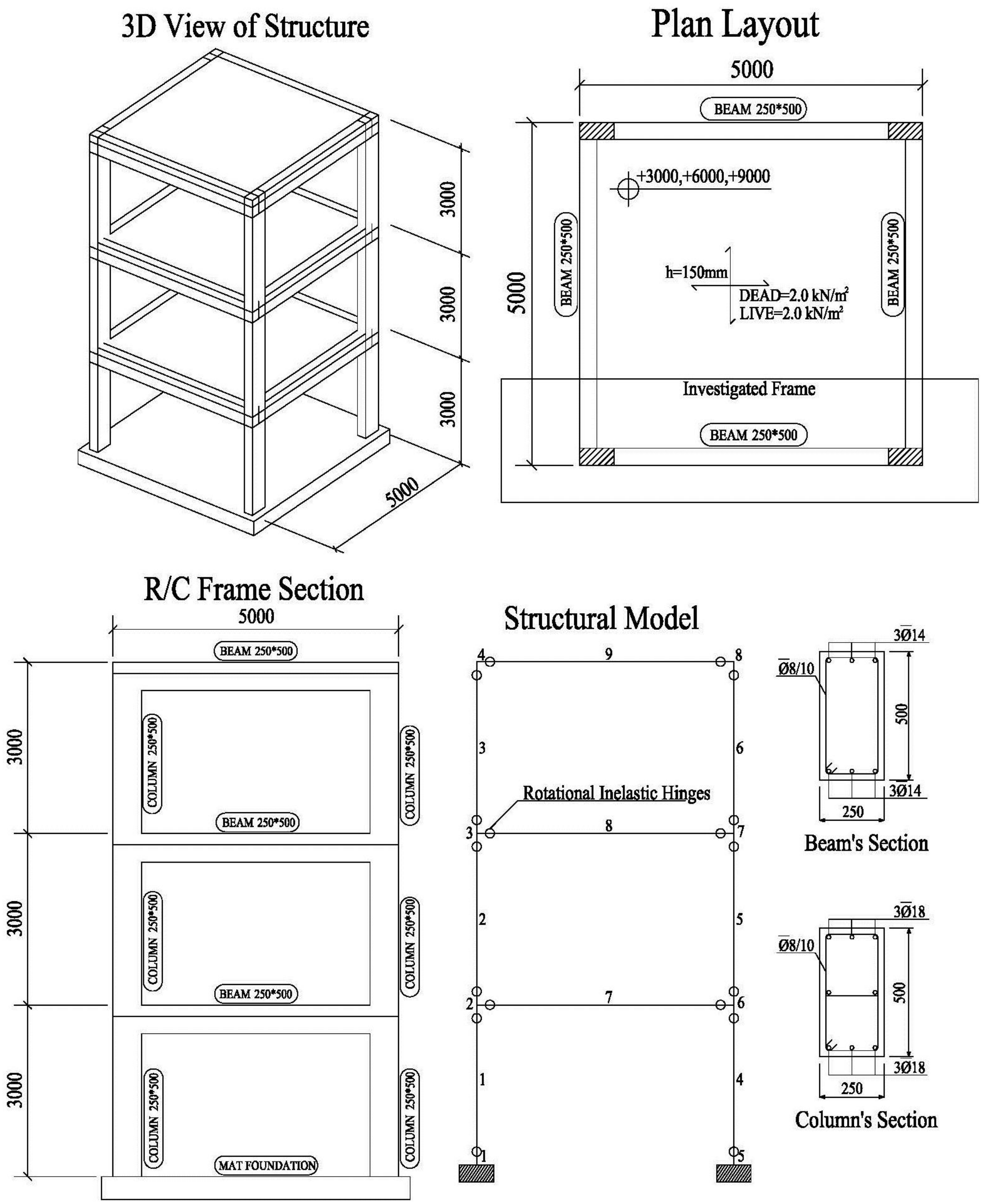


Figure 4.1: Model RC3 details, designed as RC structural frame.

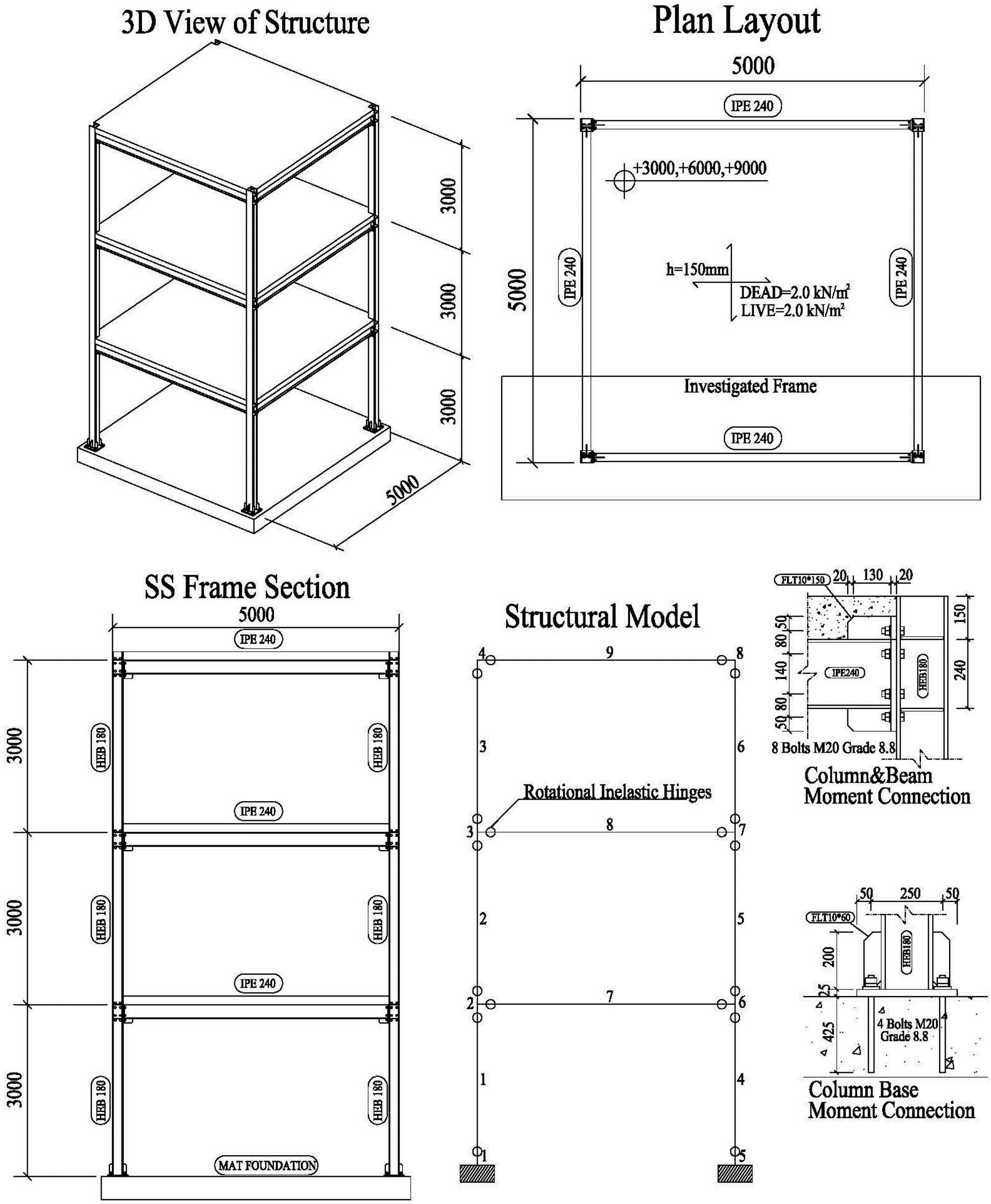


Figure 4.2: Model SS3 details, designed as SS structural framing

4.3 Description of Investigated Buildings

In this study three types of framing materials were used for the three structural frame models. These are Fully Reinforced Concrete (RC), Fully Structural Steel (SS) and Mixed Concrete Steel (RC+SS) structures.

In total six structural models were investigated under earthquake excitation. These six models include two of the models described above plus the combination of them. In this part the rest of the four models are described in detail (Figures 4.3 to 4.6).

Model RC3 and Model SS3 were designed and described above and these models were also used as part of the investigation carried out into understanding the mixed frame behavior.

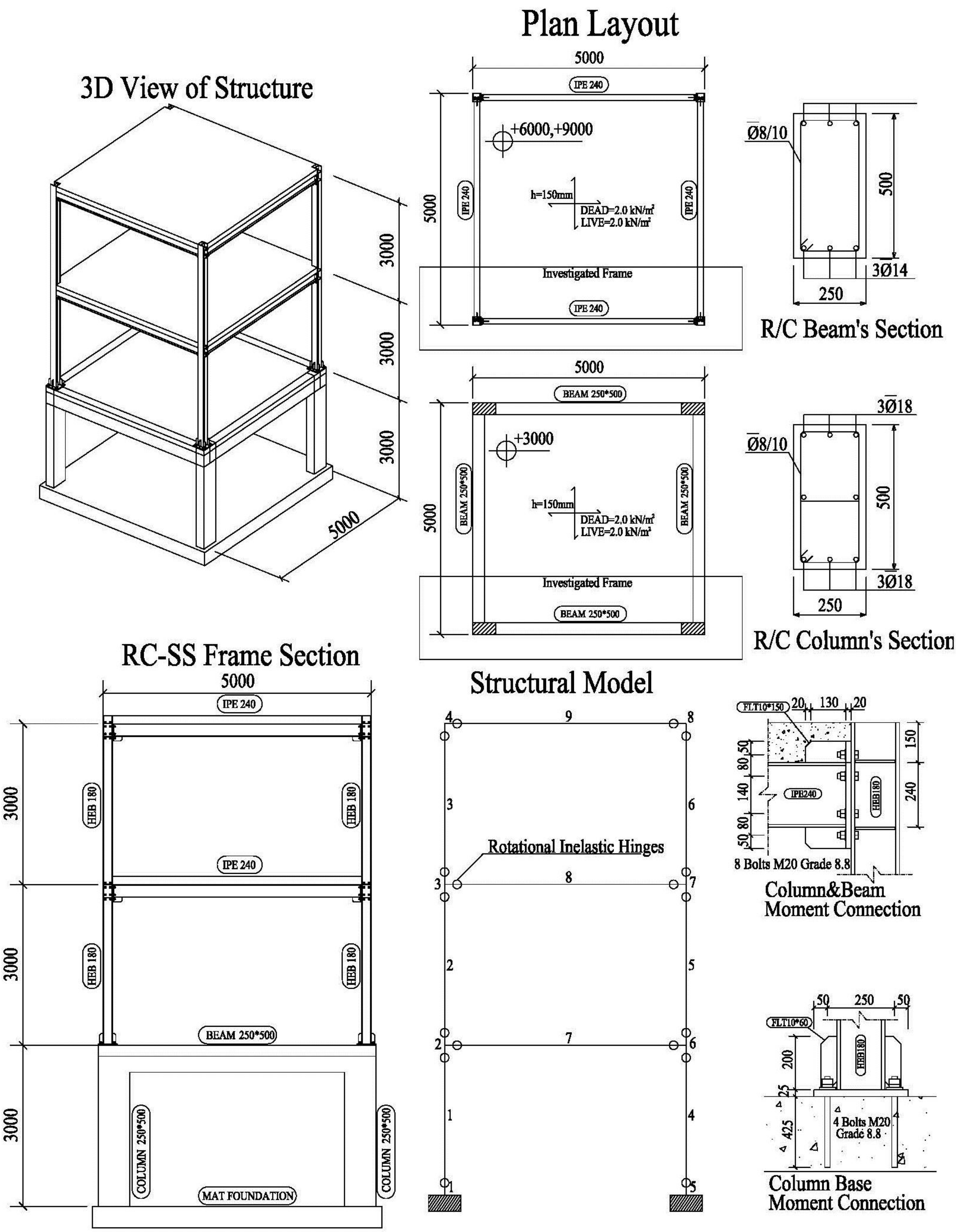


Figure 4.3: Model RC1SS2 details, mixed structural frame.

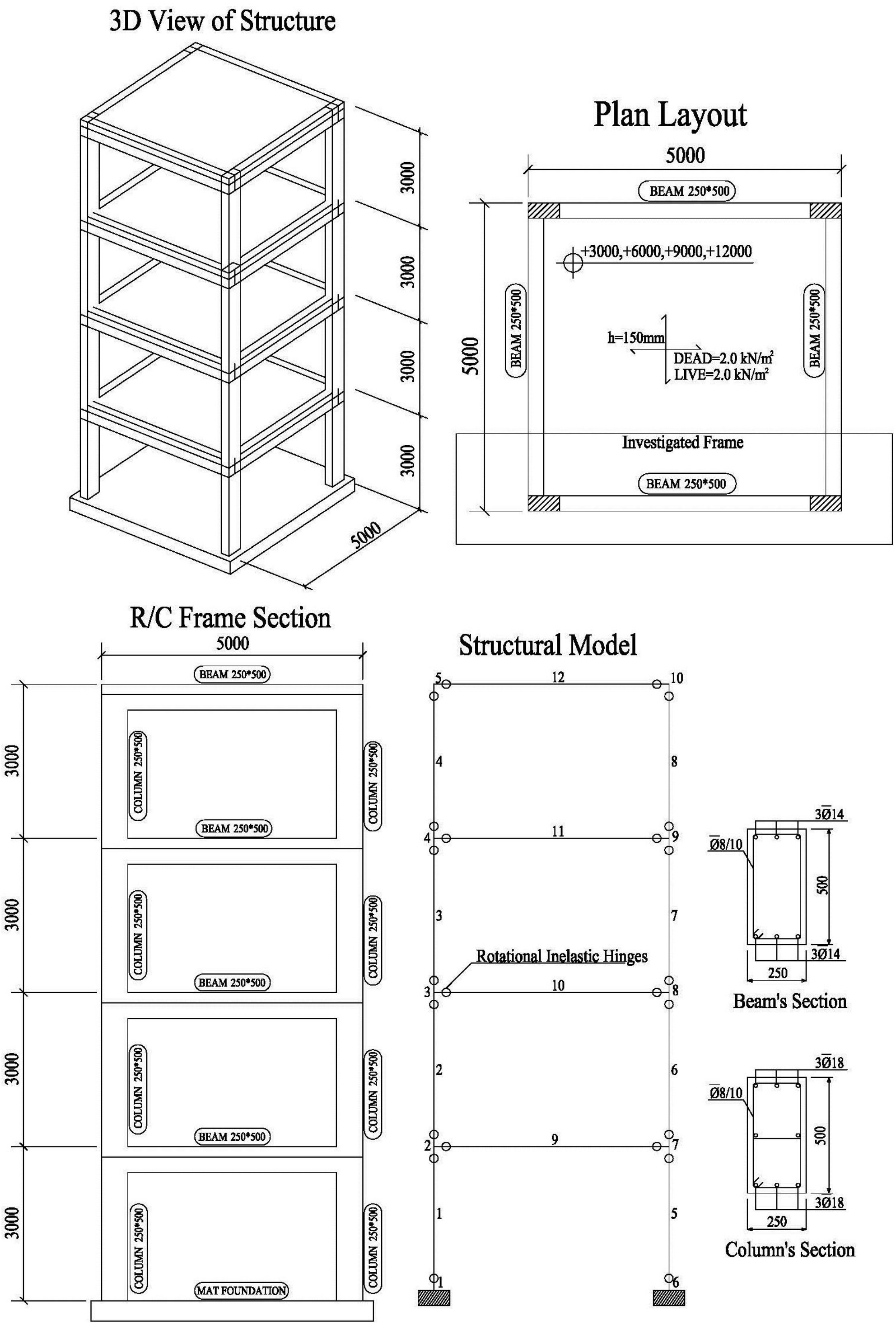


Figure 4.4: Model RC4 details, RC structural frame.

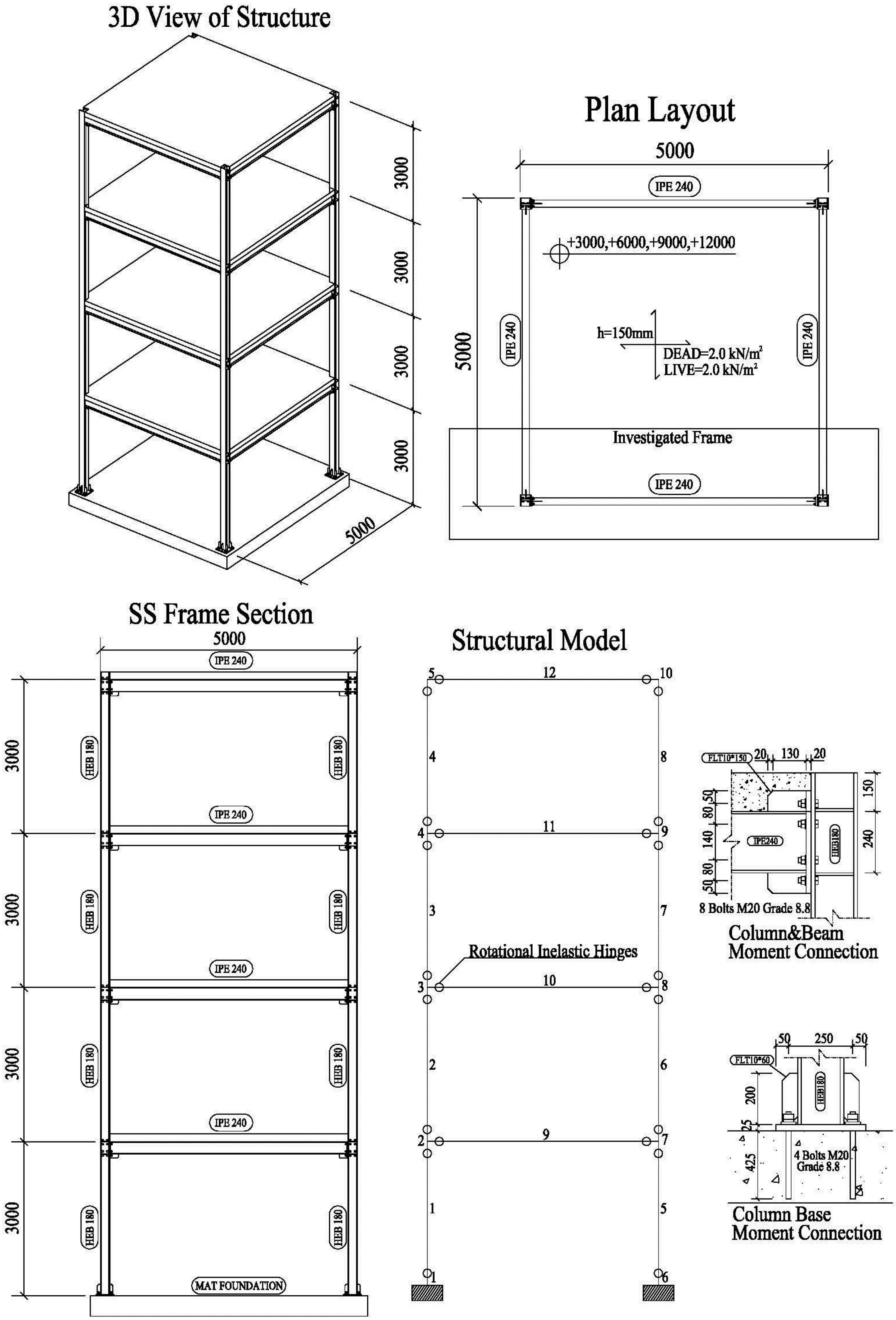


Figure 4.5: Model SS4 details, SS structural frame.

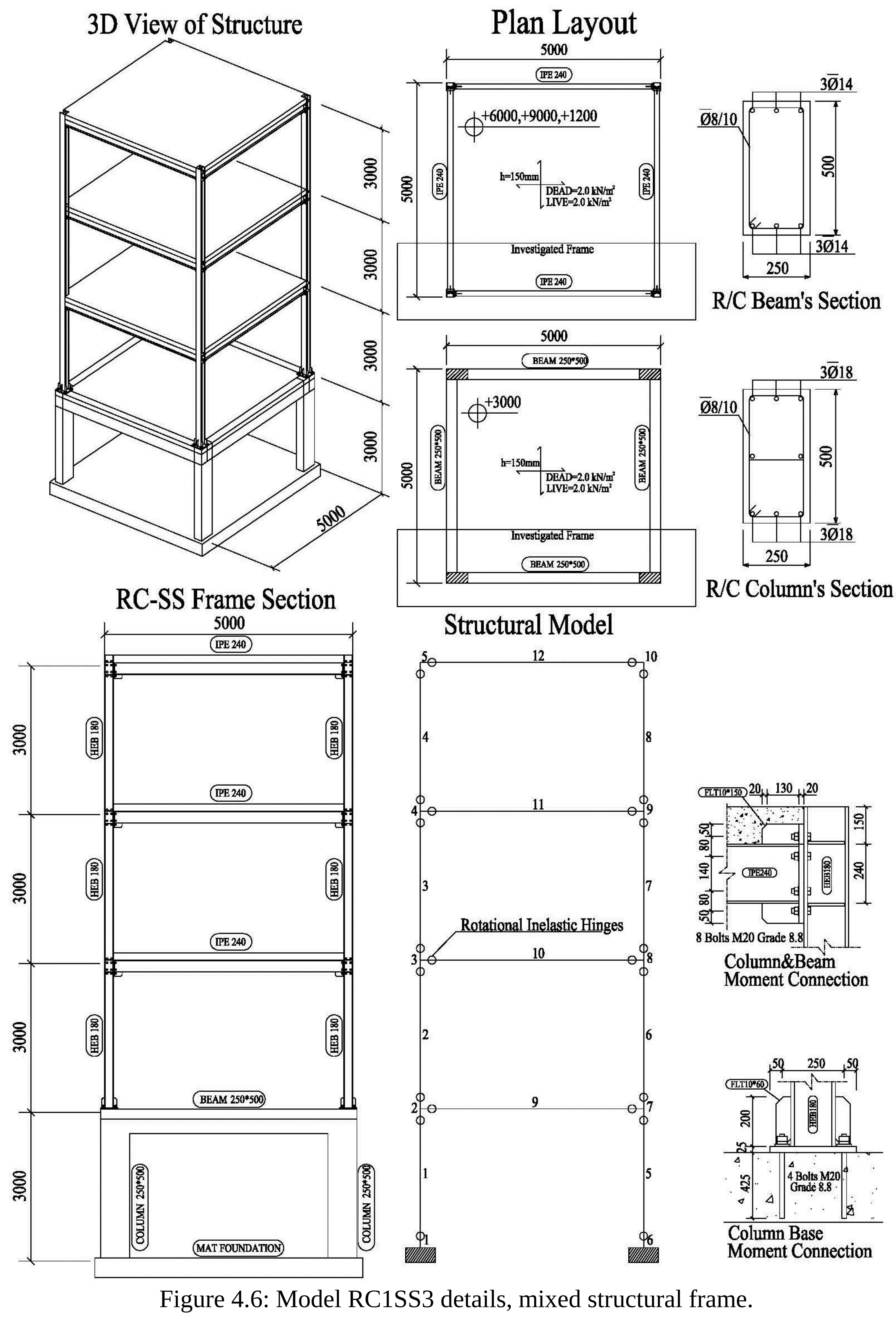


Figure 4.6: Model RC1SS3 details, mixed structural frame.

Chapter 5

ANALYSIS AND RESULTS

5.1 Analysis

In this part the results of the nonlinear time-history analysis are presented for the created models. As it was mentioned in earlier chapter, model names are abbreviated as RC or SS standing for reinforced concrete and structural steel respectively, each material is followed with a number which shows the number of the stories of the relevant material, for example RC1SS2 means Reinforced Concrete at the first story and Structural Steel in the two following stories.

5.1.1 Creating the Design Acceleration Spectrum

The design spectrum defined by TEC 2007, is created according to the ground and earthquake zone parameters. In order to have a design acceleration spectrum, the elastic spectrum should be characterized according to the given parameters. TEC 2007 specifies spectral acceleration coefficient and design acceleration spectrum for 5% dumped elastic acceleration spectrum. Therefore, our building is assumed to be in the first degree earthquake zone, building importance factor is equal to one and ground class is Z1. According to these parameters elastic design acceleration spectrum is created (Figure 5.1) and it will be used for the evaluation of selected case study models.

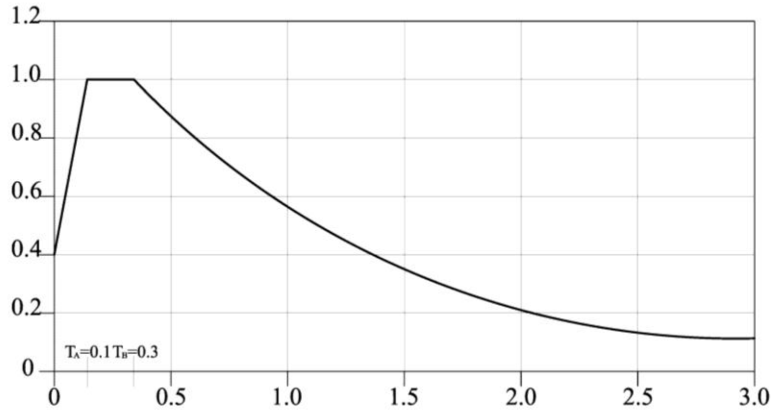


Figure 5.1: Elastic design spectrum.

5.1.2 Selecting Earthquake Hazard Level

As mentioned in chapter two, TEC 2007 specifies 3 different earthquake hazard levels, D1, D2 and D3. According to this approach, D2 level defines as the design level for residential buildings, specified 10% exceedance in 50 years, which is the earthquake occurrence expected to happen throughout the life of these buildings. D3 specifies the highest level of earthquake, this type of hazard level can be selected for the structures that are more important than the residential buildings (eg. hospital). The last D1 level is specified as ready for usage. According to TEC 2007, elastic design spectrum reflects the D2 level earthquake spectrum, which covers 10% probability of exceedance in 50 years. D1 level can be taken as approximately half of the D2 level and D3 level can be taken as 1.5 times bigger than the D2 level. However, our target level is in the D2 level and the design acceleration spectrum created will be the target spectrum for the evaluation. This spectrum specifies 10% probability of exceedance in 50 years. According to target spectrum three earthquake data will be scaled to make records to fit to the spectrum.

5.1.3 Scaling Earthquake Records

In order to have a dynamic time history analysis, three major earthquake records are selected; Düzce 1999, El Centro 1979 and Northridge 1994. These are the mostly preferred records by the researcher (Figures 5.2 to 5.4).

Pacific earthquake research center (PEER) provides most of the major earthquake records on their web site. In this web site the desired earthquake records can be downloaded as original data without scaling, or with scale factor. In this application, selected ground motions are scaled according to mean square error (MSE) approach to get the finest match with target spectrum as shown in Figure 5.5 (P.E.E.R 2010). In Table 5.1, selected three ground motion's scale factors and details are given.

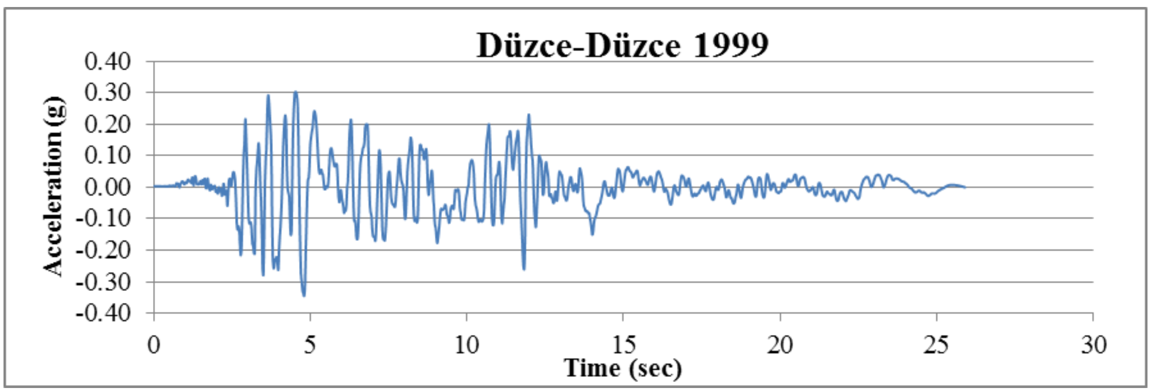


Figure 5.2: Düzce earthquake 1999 ground motion record.

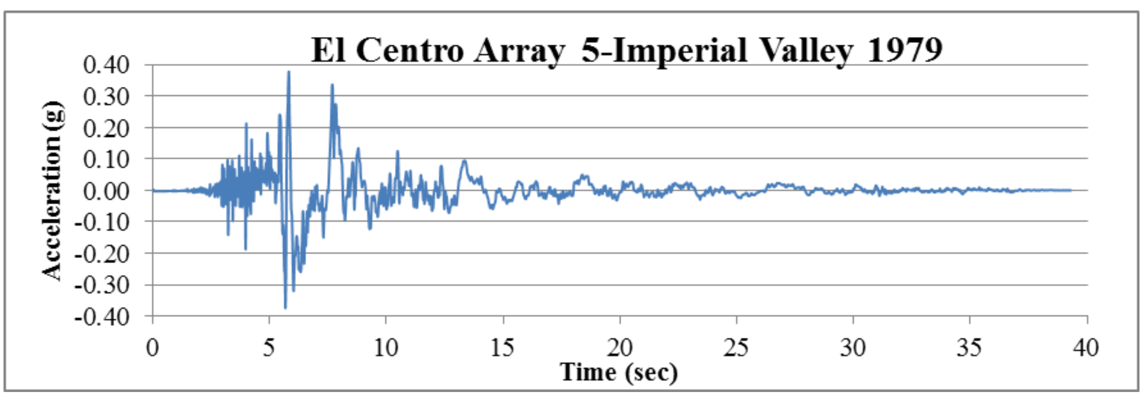


Figure 5.3: El Centro earthquake 1979 ground motion record.

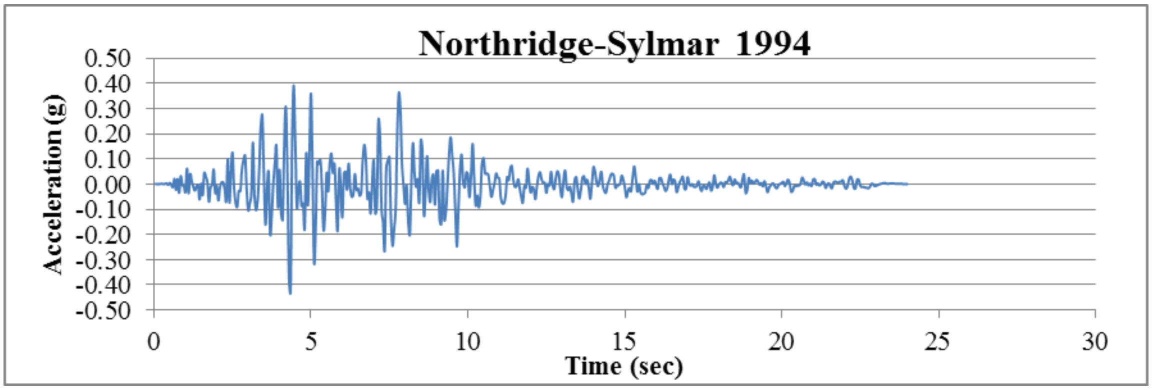


Figure 5.4: Northridge earthquake 1999 ground motion record.

Table 5.1: Details of selected ground motions.

Details of the Ground Motions									
Event Name	Station	NGA#	MSE	S.F.	Year	Magnitude	Mechanism	Component	PGA
Düzce	Düzce	1605	0.062	1.08	1999	7.14	Strike-Slip	Fault Normal	0.348
Imperial Valley-06	El Centro Array #5	180	0.069	1.20	1979	6.53	Strike-Slip	Fault Paralel	0.376
Northridge-01	Sylmar	1084	0.225	0.94	1994	6.69	Reverse	Fault Paralel	0.442

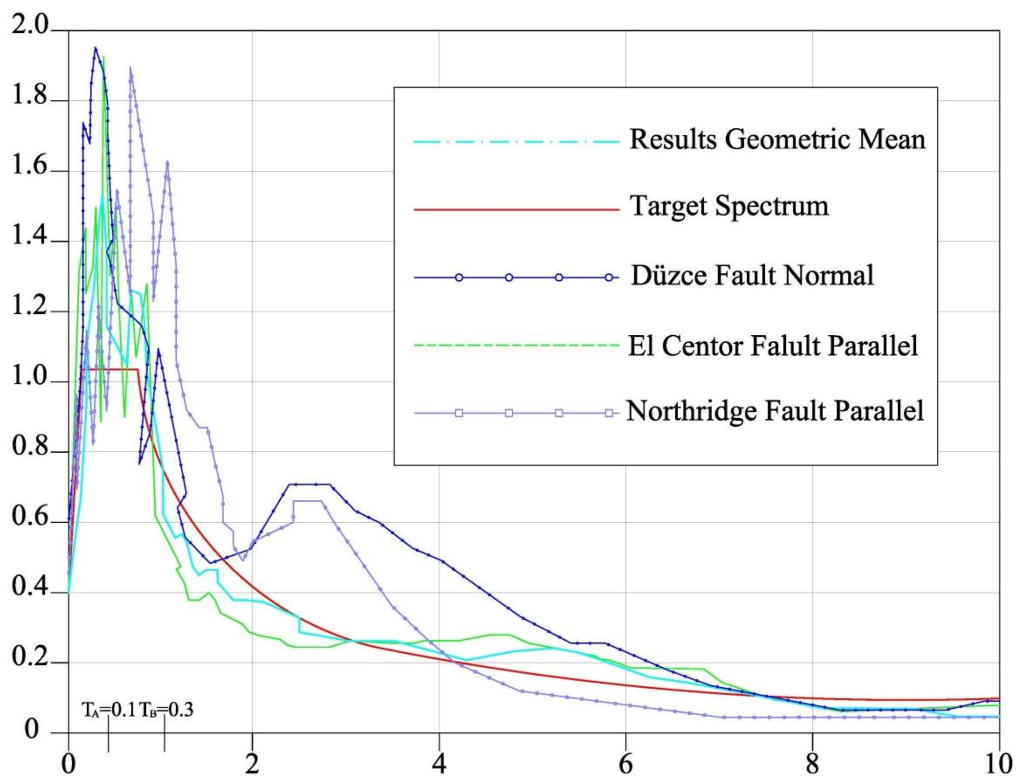


Figure 5.5: Elastic target spectrum and scaled earthquake spectrums.

5.1.4 Knowledge Level

In order to investigate the seismic performance of existing structures, knowledge level should be defined according to the specifications of the design codes. However, the imaginary analytical models are created and the material properties are assumed to be same as those in the design part. Due to this, knowledge factor is assumed as comprehensive and the corresponding value is 1.

5.2 Creation of Analysis Model

In order to, investigate the three types of buildings according to their framing material (reinforced concrete, structural steel, and mixed reinforced concrete and structural steel), six different models, and three applied earthquakes 36 nonlinear direct integration time-history analysis is done. For the first 18 models only material nonlinearities are considered. For the rest of the 18 models material and geometric nonlinearities (P-Δ) were also considered. Details of analytical models are given in previous chapter. Analyses of the analytical models are done in two dimensional plane frames. Each story on the models is assumed to have rigid body diaphragm effect. Each structural component's nonlinear behavior is specified in the structural analysis program SAP2000 with limitation of FEMA 356 acceptance criteria.

5.3 Hinge Properties of Sections

In order to evaluate seismic performance of structures, nonlinear material properties should be investigated according to the expected deformation shape (number of degree of freedom).

5.3.1 Columns

For the column member, the differences in the axial forces under earthquake excitation should be considered. Moreover the axial forces directly affect the moment capacity and ductility of the sections. Therefore, the components gravity loads (axial forces) are taken from static analysis that are given load combinations from FEMA 356 to develop moment-curvature relation of sections. In this relation, actions are controlled by flexure limitations for columns and only one degree of freedom (M3) is considered in modeling. The inelastic behavior of sections has strain hardening curve. For columns section, moment-curvature relations are considered for loading and unloading path as well. The moment-curvature relations of sections are given with graphical order in APPENDIX A. Only one of them is illustrated below as understanding of the terminology in Figure 5.6.

The axial loads, on the columns with combination of live and dead loads (1.1G+1.1Q) are taken from static analysis. The axial forces for each column are given in Table 5.2.

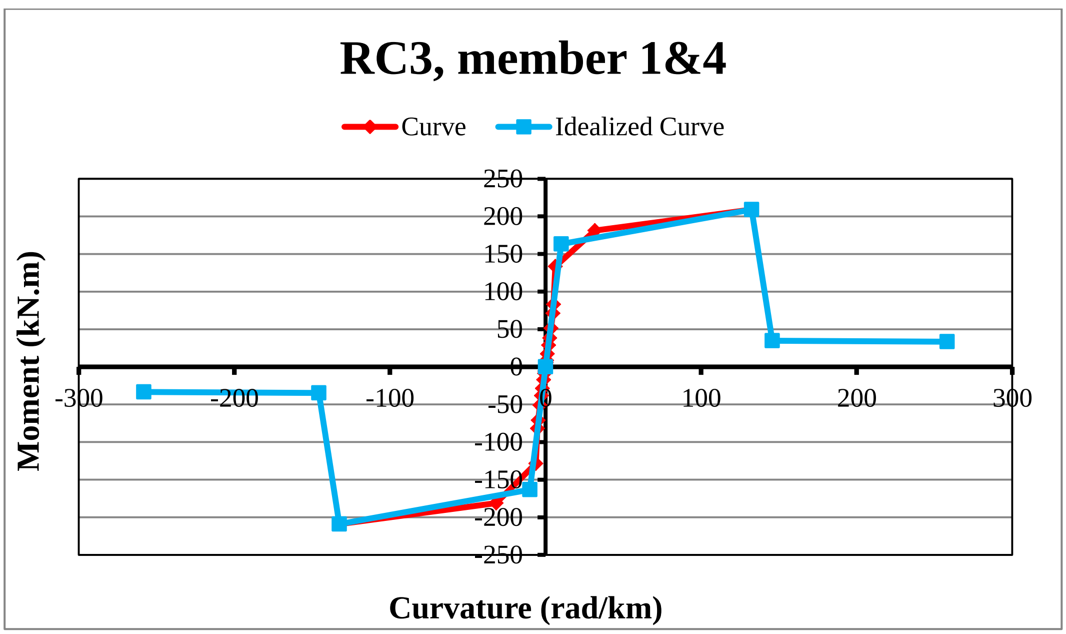


Figure 5.6: RC1SS2 Moment-Curvature relation of column 1&4 (N= -150.32 kN).

Table 5.2: Factored (1.1G+1.1Q) static axial loads on columns.

Model Name	Member No	Joint No	Axial Load (kN)
RC3	1	1,2	-150.32
	2	2,3	-100.21
	3	3,4	-50.11
	4	5,6	-150.32
	5	6,7	-100.21
	6	7,8	-50.11
SS3	1	1,2	-102.62
	2	2,3	-68.41
	3	3,4	-34.21
	4	5,6	-102.62
	5	6,7	-68.41
	6	7,8	-34.21
RC1SS2	1	1,2	-110.8
	2	2,3	-68.41
	3	3,4	-34.21
	4	5,6	-110.8
	5	6,7	-68.41
	6	7,8	-34.21

Table 5.2: continued.

Model Name	Column No	Joint No	Axial Load (kN)
RC4	1	1,2	-200.42
	2	2,3	-150.32
	3	3,4	-100.21
	4	4,5	-50.11
	5	6,7	-200.42
	6	7,8	-150.32
	7	8,9	-100.21
	8	9,10	-50.11
SS4	1	1,2	-136.82
	2	2,3	-102.62
	3	3,4	-68.41
	4	4,5	-34.21
	5	6,7	-136.82
	6	7,8	-102.62
	7	8,9	-68.41
	8	9,10	-34.21
RC1SS3	1	1,2	-145.02
	2	2,3	-102.62
	3	3,4	-68.41
	4	4,5	-34.21
	5	6,7	-145.02
	6	7,8	-102.62
	7	8,9	-68.41
	8	9,10	-34.21

Note: The member number and joint numbers are specified in the chapter 4 (analytical models).

5.3.2 Beams

For beams only 1 degree of freedom (M3) has considered. The inelastic behavior of sections has strain hardening curve. For beam sections, moment-curvature relations were considered for loading and unloading path as well.

5.4 Nonlinear Time History Analysis

Before starting the nonlinear time history analysis, the linear static analysis has been done according to the loads that were considered. Nonlinear time history analysis is done under load combination that is given by TEC 2007:

$$G + nQ = G + 0.3Q \tag{5.1}$$

Where:

G specifies the dead weight of structure, n is the participation factor of live load (0.3 for residential type buildings, TEC 2007) and Q is the live load on the structure.

5.5 Results

In order to investigate seismic performance of structures, global and member level evaluation will be considered in this part. For the global level evaluation, story displacements are developed for each model and each earthquake record. In the evaluation, maximum inter-story drift ratios are going to be investigated with limitations given by FEMA 356.

In member level evaluation FEMA 356 acceptance criteria is used to recognize performance level of the sections ends. For the acceptance levels OP, IO, LS and CP limits will be considered. Each model will be investigated separately, global and member level stage and the total results will be discussed in Chapter 5.

5.5.1 General Information about the given Figures and Tables

In this part, results of analytical models will be given in terms of tables and figures. In "story displacement versus time" figures, the relative story displacements are given in function of time for each story level with different line styles.

In “Plastic formations and performance levels” table the maximum plastic rotations were given in radian and specifying the performance level of hinges with respect to plastic rotation limits given by FEMA 356.At the left side of the table, plastic sections and their locations on the structural model for three different earthquake records are given as schematic pictures.

In addition, the inter-story drift ratios are figured for each model with selected three ground motions result including transient and permanent drift limits (maximum transient and permanent inter-story drift ratios). The limit values for steel and reinforced concrete are illustrated separately in figures with different line styles.

5.5.2 Model RC3 Results

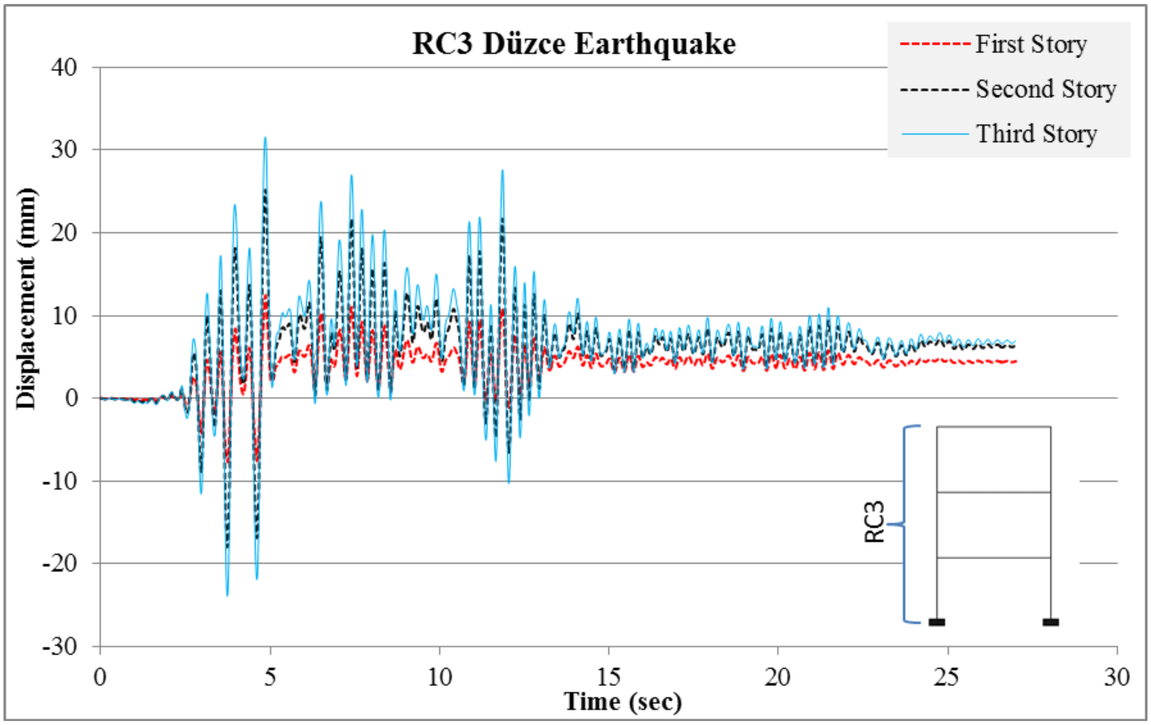


Figure 5.7: RC3 Düzce earthquake story displacements versus time function.

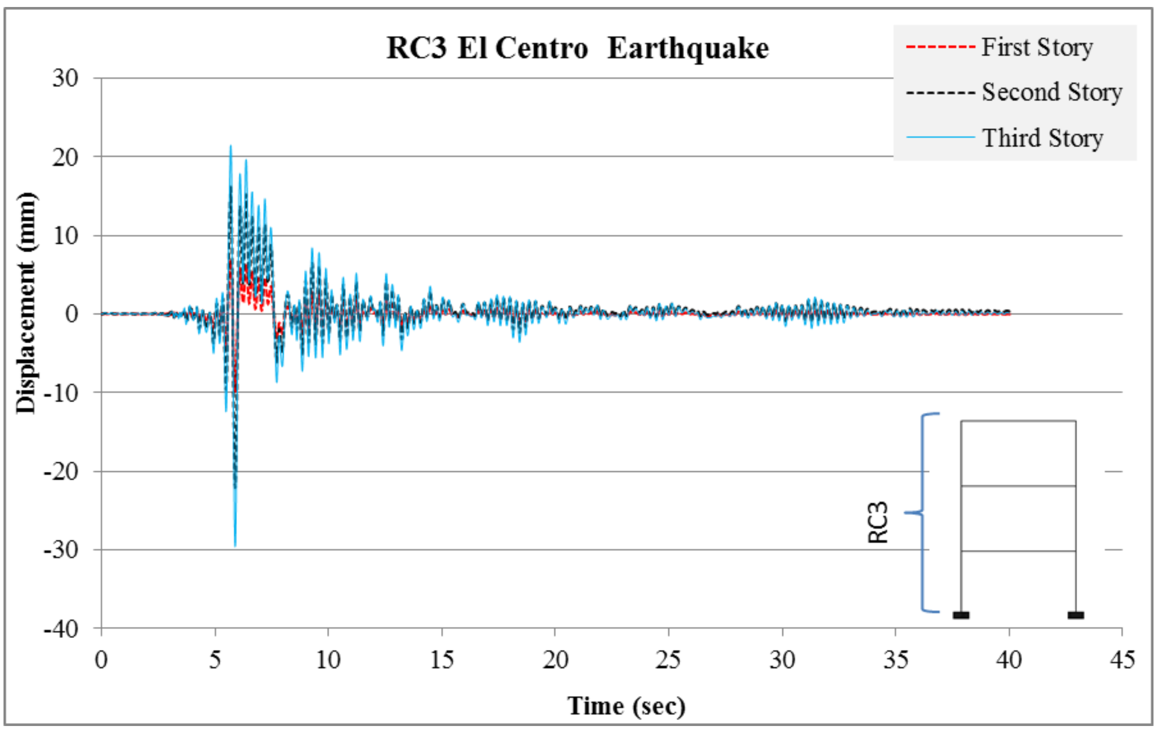


Figure 5.8: RC3 El Centro earthquake story displacements versus time function.

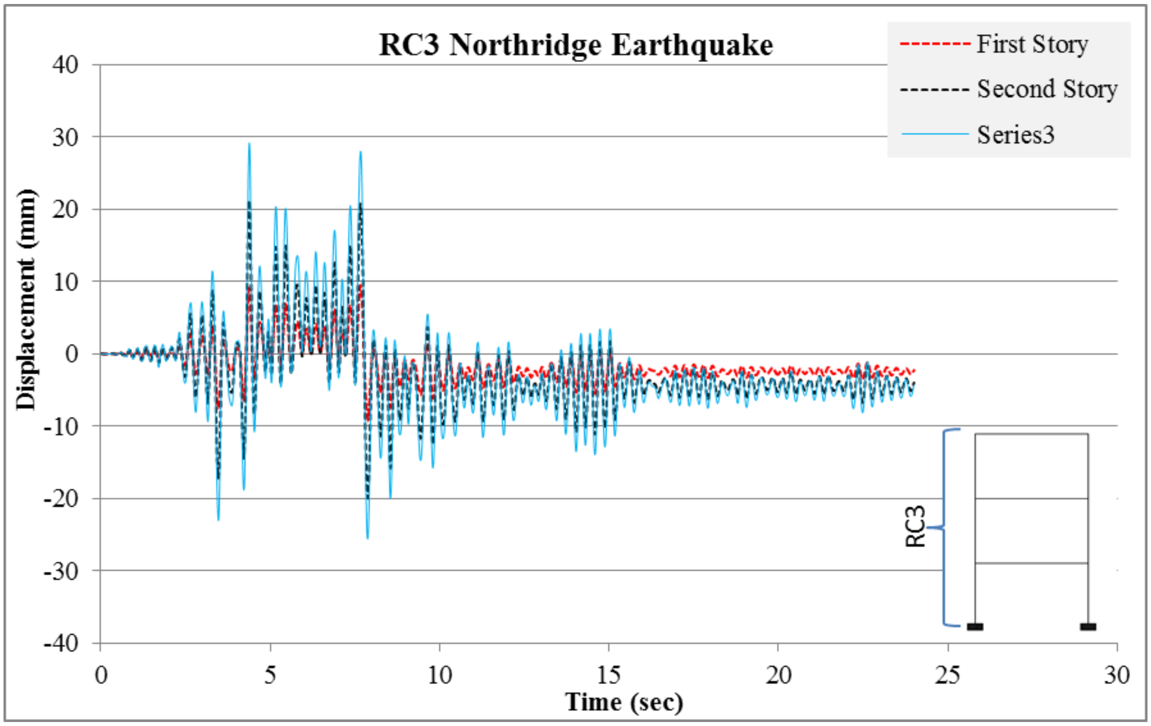


Figure 5.9: RC3 Northridge earthquake story displacements versus time function.

According to these results; peak roof displacements values are found to be 32, 30 and 29 mm for Düzce, El Centro and Northridge earthquakes respectively (Figures 5.7, 5.8 and 5.9). For Düzce, El Centro and Northridge earthquake, hinge formations are investigated. Totally eight section ends have exceeded their elastic limits for each model. Six hinges occurred at first, second and third story beam sections. Two hinges were formed at the bottom face of column sections in first story.

None of the earthquake records could create stability or mechanism problem for RC3 structural model. Global structure and member level evaluations are required to have decision of building performance level. Firstly Member level evaluation is carried out.

5.5.2.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotation of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.3. In this table FEMA 356 rotation limitations were compared with maximum plastic rotations in terms of radian.

5.5.2.2 Global Level Evaluation

In order to evaluate global performance of structure, transient and permanent inter-story drift ratios are investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Fig 5.10.

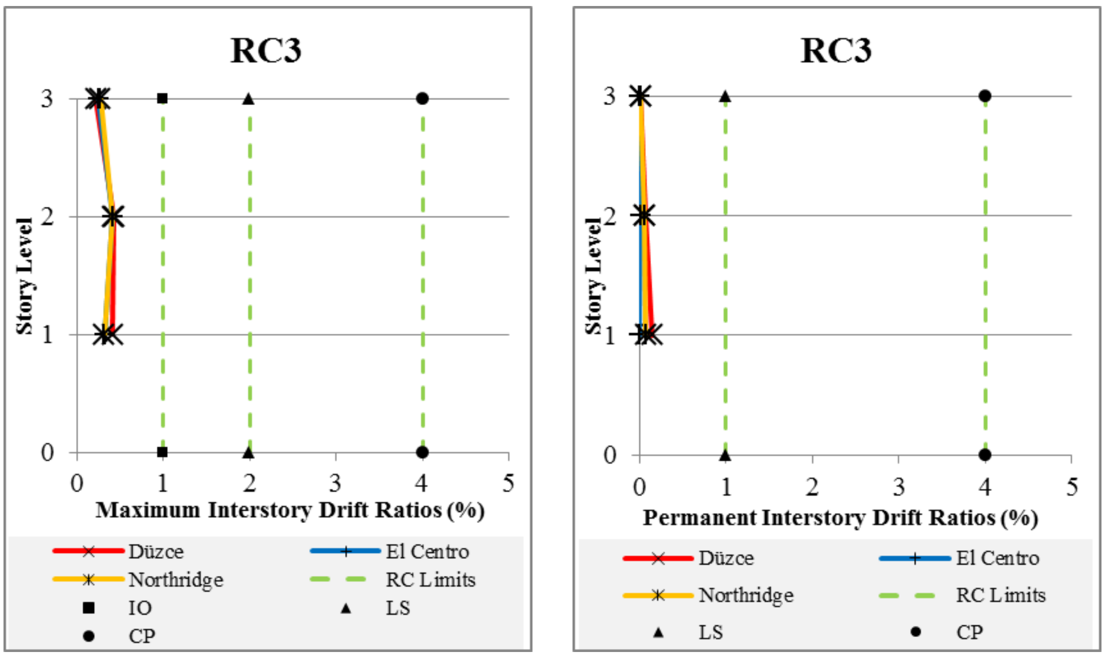
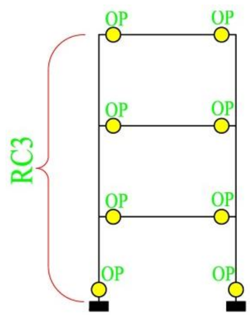
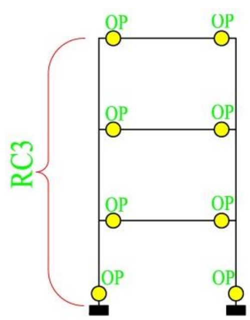
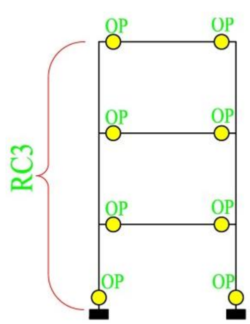


Figure 5.10: RC3 maximum transient and permanent interstory drift ratios.

Table 5.3: RC3 Plastic formations and performance levels.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce 	1	BEAM	RC	7	2&6	88.98	**0.005**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	8	3&7	87.35	**0.0038**	0.01 0.02 0.025	**OP**
	3	COLUMN	RC	1&4	1&5	168.9	**0.0024**	0.005 0.015 0.02	**OP**
	4	BEAM	RC	9	4&8	84.16	**0.0015**	0.01 0.02 0.025	**OP**
El Centro 	1	BEAM	RC	7	2&6	87.86	**0.0042**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	8	3&7	87.03	**0.0035**	0.01 0.02 0.025	**OP**
	3	COLUMN	RC	1&4	1&5	166.42	**0.0014**	0.005 0.015 0.02	**OP**
	4	BEAM	RC	9	4&8	83.7	**0.0013**	0.005 0.015 0.02	**OP**
Northridge 	1	BEAM	RC	7	2&6	87.65	**0.004**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	8	3&7	86.93	**0.0035**	0.01 0.02 0.025	**OP**
	3	COLUMN	RC	1&4	1&5	167.73	**0.0019**	0.005 0.015 0.02	**OP**
	4	BEAM	RC	9	4&8	83.8	**0.0012**	0.005 0.015 0.02	**OP**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel.

According to results, transient inter-story drift ratios for Düzce earthquake were 0.41%, 0.43% and 0.21%, for El Centro Earthquake 0.32%, 0.41%, and 0.25%, for Northridge Earthquake 0.31%, 0.41%, and 0.27% for first, second and third story levels respectively for each earthquake. As a result none of the stories exceeded IO drift limits.

However, structure reached to the plastic limits, and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore, this structure can be assumed in LS performance level.

5.5.3 Model RC1SS2 Results

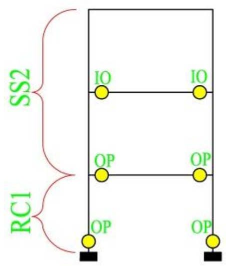
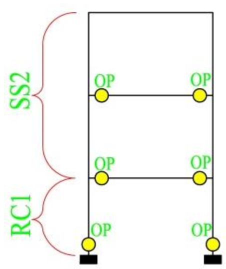
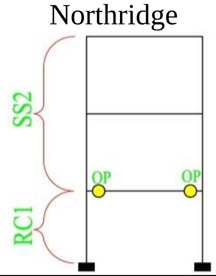
According to figures given in APPENDIX B; peak roof displacements values are investigated to be 90, 73 and 52 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce and El Centro earthquakes, hinge formations are investigated. Totally six section ends were exceeded their elastic limits for each model. Four hinges occurred at first and second story beam sections. Two hinges are formed at first story bottom face of the column sections. For the Northridge earthquake, two hinges are formed at the first story beam ends.

None of the earthquake records created stability and mechanism problem for RC1SS2 structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.3.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to the results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.4. In this table FEMA 356 rotation limitations were compared with maximum plastic rotation in terms of radian.

Table 5.4: RC1SS2 Plastic formations and performance levels.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce 	1	BEAM	RC	7	2&6	90.29	**0.006**	0.01 0.02 0.025	**OP**
	2	COLUMN	RC	1&4	1&5	169.05	**0.0025**	0.005 0.015 0.02	**OP**
	3	BEAM	SS	8	3&7	102.04	**0.0032**	0.00216 0.0173 0.0259	**IO**
El Centro 	1	BEAM	RC	7	2&6	90	**0.0058**	0.01 0.02 0.025	**OP**
	2	COLUMN	RC	1&4	1&5	169.6	**0.0028**	0.005 0.015 0.02	**OP**
	3	BEAM	SS	8	3&7	100.95	**0.00015**	0.00216 0.0173 0.0259	**OP**
Northridge 	1	BEAM	RC	7	2&6	83.29	**0.00087**	0.01 0.02 0.025	**OP**
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-

Notes:
1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel.

5.5.3.2 Global Level Evaluation

In order to evaluate global performance of structure transient and permanent inter-story drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.11.

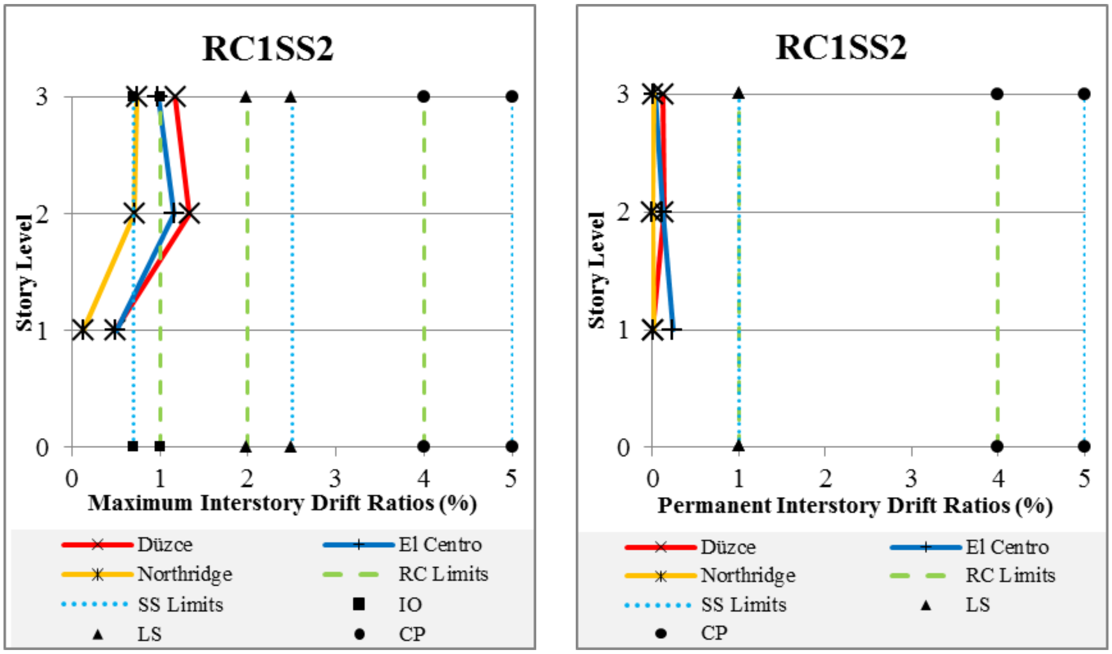


Figure 5.11: RC1SS2 maximum transient and permanent interstory drift ratios.

According to results, transient inter-story drift ratios for Düzce earthquake were 0.49%, 1.33% and 1.17%, for El Centro Earthquake 0.49%, 1.16%, and 0.97%, for Northridge Earthquake 0.13%, 0.71% and 0.74% for first, second and third story levels respectively for each earthquake. As a result, first story's drift ratio values did not exceed the limitation of IO, while second and third stories have exceeded the limitation of IO remained in LS limit.

Although, structure reached to the plastic limits and permanent drifts were occurred, permanent drift ratios did not exceed the LS limit. Therefore, this structure can be assumed in LS performance level in terms of drift performance.

5.5.4 Model SS3 Results

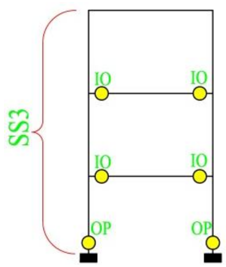
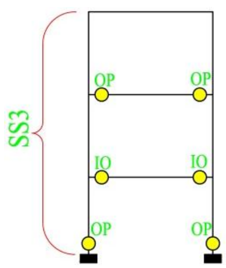
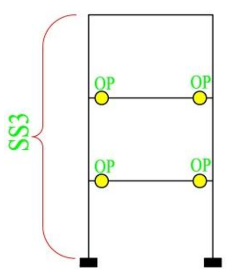
According to figures given in APPENDIX B; peak roof displacement values are found to be 120, 110 and 90 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce and El Centro earthquakes, hinge formations are investigated. Only six section ends exceeded their elastic limits for each model. Four hinges occurred at first and second story beam sections. Two hinges were formed at the bottom face of the column sections in first story. For the Northridge earthquake, totally four hinges occurred at first and second story beam sections.

None of the earthquake records could create stability and mechanism problem for SS3 structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.4.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.5. In this table FEMA 356 rotation limitations were compared with maximum plastic rotations in terms of radian.

Table 5.5: SS3 Plastic formations and performance levels.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce 	1	BEAM	SS	7	2&6	103.37	**0.0062**	0.00216 0.0173 0.0259	**IO**
	2	BEAM	SS	8	3&7	102.19	**0.0036**	0.00216 0.0173 0.0259	**IO**
	3	COLUMN	SS	1&4	1&5	132.26	**0.00016**	0.00179 0.0143 0.0215	**OP**
El Centro 	1	BEAM	SS	7	2&6	103.24	**0.0066**	0.00216 0.0173 0.0259	**IO**
	2	BEAM	SS	8	3&7	101.46	**0.00153**	0.00216 0.0173 0.0259	**OP**
	3	COLUMN	SS	1&4	1&5	132.3	**0.00015**	0.00179 0.0143 0.0215	**OP**
Northridge 	1	BEAM	SS	7	2&6	101.41	**0.00139**	0.00216 0.0173 0.0259	**OP**
	2	BEAM	SS	8	3&7	101.47	**0.00177**	0.00216 0.0173 0.0259	**OP**
	-	-	-	-	-	-	-	-	-

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.4.2 Global Level Evaluation

In order to evaluate global performance of a structure, transient and permanent inter-story drift ratios are investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.12.

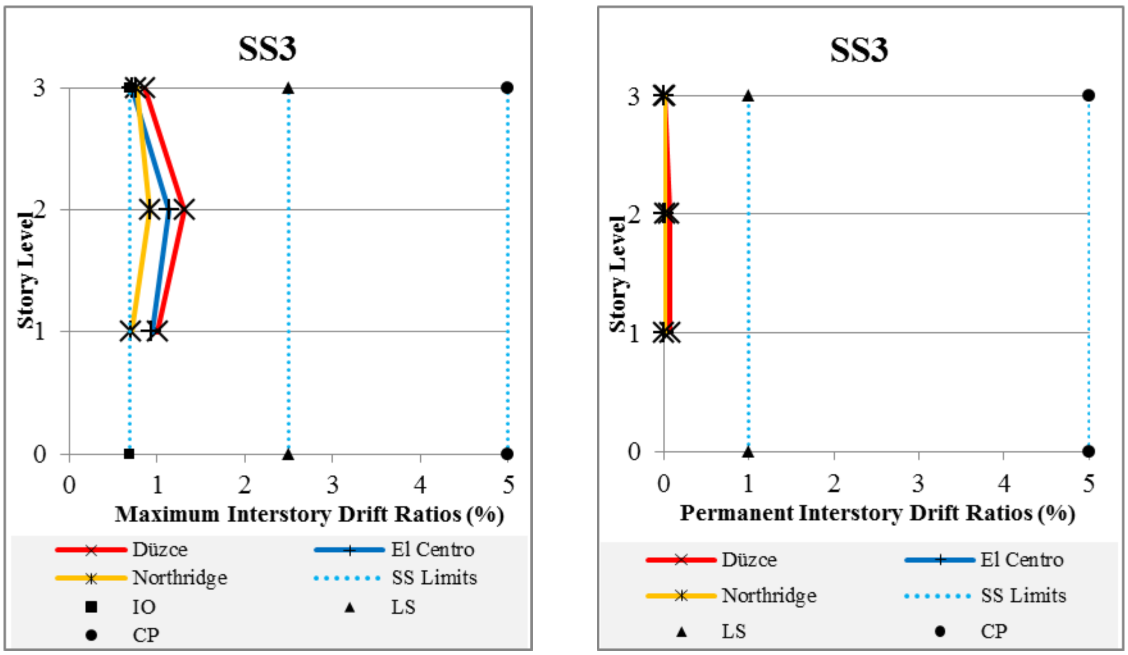


Figure 5.12: SS3 maximum transient and permanent interstory drift ratios.

According to results, transient inter-story drift ratios for Düzce earthquake were 1.01%, 1.32% and 0.86%, for El Centro Earthquake 0.94%, 1.41% and 0.72%, for Northridge Earthquake 0.71%, 0.92% and 0.77% for first, second and third story levels respectively for each earthquake. As a result all stories were exceeded the limitations of IO and the structure is in the limitation of LS.

However, global structure has lost its elastic properties and structure reached to the plastic limits and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore, this structure can be assumed in LS performance level.

5.5.5 Model RC3 P-Delta Results

According to figures given in APPENDIX B; peak roof displacement values are investigated to be 33, 30 and 30 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce, El Centro and Northridge earthquakes, hinge formations are investigated. Totally eight section ends have exceeded their elastic limits for each model. Six hinges occurred at first, second and third story beam sections. Two hinges were formed at bottom face of columns sections in first story.

None of the investigated earthquake records did create stability and mechanism problem for RC3 P-Delta structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.5.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members were investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.6. In this table FEMA 356 rotation limitations were compared with maximum plastic rotations in terms of radian.

Table 5.6: RC3 P-Delta Plastic formations and performance levels.

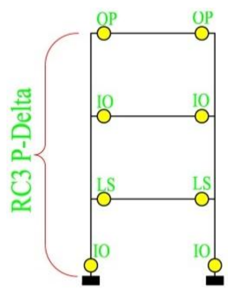
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce	1	BEAM	RC	7	2	106.44	**0.021**	0.01 0.02 0.025	**LS**
	2	BEAM	RC	7	6	108.7	**0.025**	0.01 0.02 0.025	**LS**
	3	BEAM	RC	8	3	96.27	**0.012**	0.01 0.02 0.025	**IO**
	4	BEAM	RC	9	7	98.24	**0.015**	0.01 0.02 0.025	**IO**
	5	COLUMN	RC	1	1	169.28	**0.0043**	0.005 0.015 0.02	**IO**
	6	COLUMN	RC	4	5	174.33	**0.008**	0.005 0.015 0.02	**IO**
	7	BEAM	RC	9	4	84.26	**0.002**	0.01 0.02 0.025	**OP**
	8	BEAM	RC	9	8	82.9	**0.0011**	0.01 0.02 0.025	**OP**

Table 5.6: continued.

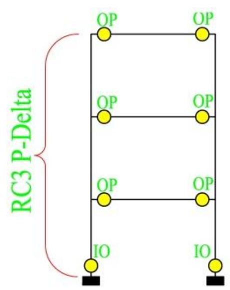
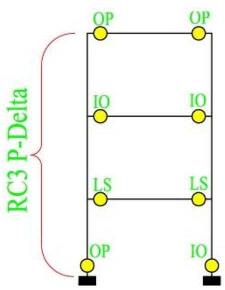
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
El Centro 	1	BEAM	RC	7	2	91.34	**0.0081**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	7	6	91.91	**0.0086**	0.01 0.02 0.025	**OP**
	3	BEAM	RC	8	7	88.55	**0.0057**	0.01 0.02 0.025	**OP**
	4	BEAM	RC	8	3	89.26	**0.0063**	0.01 0.02 0.025	**OP**
	5	COLUMN	RC	1	1	178.17	**0.011**	0.005 0.015 0.02	**IO**
	6	COLUMN	RC	4	5	168.04	**0.0058**	0.005 0.015 0.02	**IO**
	7	BEAM	RC	9	4&8	83.7	**0.0014**	0.01 0.02 0.025	**OP**
	8	BEAM	RC	9	4&8	84.26	**0.0019**	0.01 0.02 0.025	**OP**

Table 5.6: continued.

<table>
<tr><th>Damage with Probability of exceedance 10% in 50 years</th><th>Sequence</th><th>Section</th><th>Material</th><th>Member</th><th>Joint</th><th>Moments (kNm)</th><th>Maximum Plastic Rotation (rad)</th><th>FEMA 356 limits (rad)</th><th>Performance Level</th></tr>
<tr><td rowspan="8">Northridge

</td><td>1</td><td>BEAM</td><td>RC</td><td>7</td><td>2</td><td>106.44</td><td>0.021</td><td>0.01
0.02
0.025</td><td>LS</td></tr>
<tr><td>2</td><td>BEAM</td><td>RC</td><td>7</td><td>6</td><td>108.67</td><td>0.025</td><td>0.01
0.02
0.025</td><td>LS</td></tr>
<tr><td>3</td><td>BEAM</td><td>RC</td><td>8</td><td>3</td><td>96.27</td><td>0.012</td><td>0.01
0.02
0.025</td><td>IO</td></tr>
<tr><td>4</td><td>BEAM</td><td>RC</td><td>8</td><td>7</td><td>98.24</td><td>0.015</td><td>0.01
0.02
0.025</td><td>IO</td></tr>
<tr><td>5</td><td>COLUMN</td><td>RC</td><td>1</td><td>1</td><td>169.22</td><td>0.0043</td><td>0.005
0.015
0.02</td><td>OP</td></tr>
<tr><td>6</td><td>COLUMN</td><td>RC</td><td>4</td><td>5</td><td>174.33</td><td>0.0080</td><td>0.005
0.015
0.02</td><td>IO</td></tr>
<tr><td>7</td><td>BEAM</td><td>RC</td><td>9</td><td>4</td><td>84.25</td><td>0.0019</td><td>0.01
0.02
0.025</td><td>OP</td></tr>
<tr><td>8</td><td>BEAM</td><td>RC</td><td>9</td><td>8</td><td>83.93</td><td>0.0016</td><td>0.01
0.02
0.025</td><td>OP</td></tr>
</table>

Notes:
1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.5.2 Global Level Evaluation

In order to evaluate global performance of structure, transient and permanent interstory drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.13.

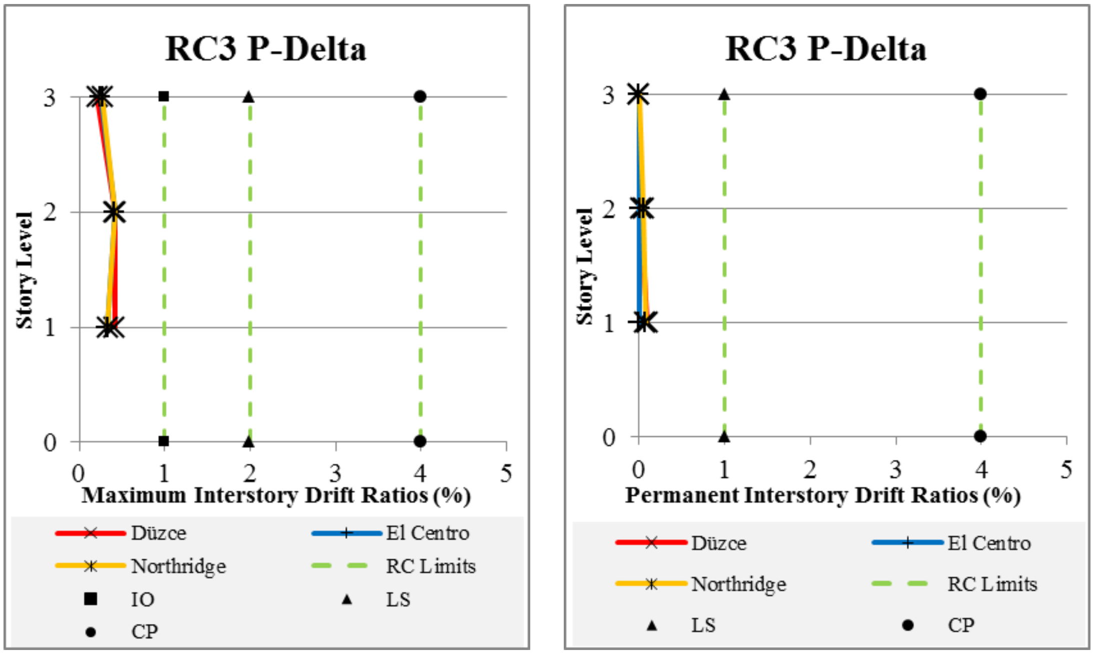


Figure 5.13: RC3 P-Delta maximum transient and permanent interstory drift ratios.

According to the results, transient inter-story drift ratios for Düzce earthquake were 0.41%, 0.42%, and 0.21%, for El Centro Earthquake 0.32%, 0.41% and 0.25%, for Northridge Earthquake 0.32%, 0.41% and 0.25% for first, second and third story levels respectively for each earthquake. As a result none of the stories could exceed IO drift limits.

However, structure reached to the plastic limits and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore this structure can be assumed in LS performance level.

5.5.6 Model RC1SS2 P-Delta Results

According to figures given in APPENDIX B; peak roof displacement values are investigated to be 89, 74 and 46 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce and El Centro earthquakes, hinge formation is investigated. Six section ends were exceeded their elastic limits for each model. Four hinges occurred at first and second story beam sections. Two hinges were formed at first story, bottom face of columns. For the Northridge earthquake, totally two hinges were formed at the first story beams ends.

None of the earthquake records could create stability and mechanism problem for RC1SS2 P-Delta structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.6.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members were investigated according to results of rotations of the sections ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.7. In this table FEMA 356 rotation limitations were compared with maximum plastic rotations in terms of radian.

5.5.6.2 Global Level Evaluation

In order to evaluate global performance of structure, transient and permanent inter-story drift ratios are investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.14.

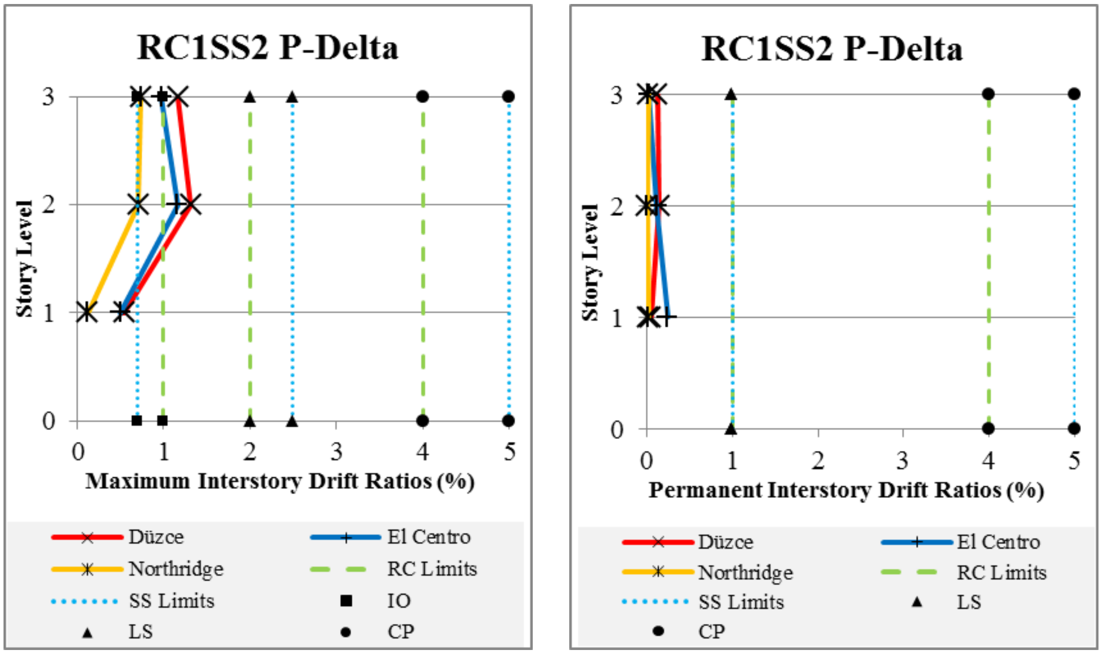


Figure 5.14: RC1SS2 P-Delta maximum transient and permanent inter-story drift ratios.

Table 5.7: RC1SS2 P-Delta Plastic formations and performance levels

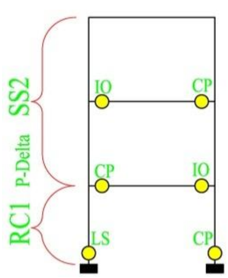
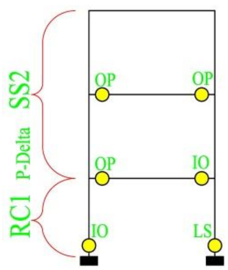
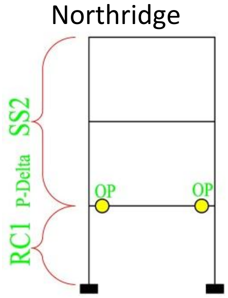
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce	1	BEAM	RC	7	2	110.61	**0.025**	0.01 0.02 0.025	**CP**
	2	BEAM	RC	7	6	104.29	**0.019**	0.01 0.02 0.025	**IO**
	3	COLUMN	RC	1	1	181.58	**0.0176**	0.005 0.015 0.02	**LS**
	4	COLUMN	RC	4	5	192.45	**0.0245**	0.005 0.015 0.02	**CP**
	5	BEAM	SS	8	3	101.85	**0.0051**	0.00216 0.0173 0.0259	**IO**
	6	BEAM	SS	8	7	102.04	**0.034**	0.00216 0.0173 0.0259	**CP**

Table 5.7: continued.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
El Centro 	1	BEAM	RC	7	2	93.43	**0.0099**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	7	6	93.59	**0.01**	0.01 0.02 0.025	**IO**
	3	COLUMN	RC	1	1	172.08	**0.0071**	0.005 0.015 0.02	**IO**
	4	COLUMN	RC	4	5	181.7	**0.015**	0.005 0.015 0.02	**LS**
	5	BEAM	SS	8	3	101.28	**0.001**	0.00216 0.0173 0.0259	**OP**
	6	BEAM	SS	8	7	101.27	**0.0009**	0.00216 0.0173 0.0259	**OP**
Northridge 	1	BEAM	RC	7	2	95.59	**0.0084**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	7	6	88.04	**0.0044**	0.01 0.02 0.025	**OP**
	-	-	-	-	-	-	**-**	-	**-**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.

2. RC: Reinforced Concrete, SS: Structural Steel

According to the results, transient inter-story drift ratios for Düzce earthquake were 0.54%, 1.31%, and 1.17%, for El Centro Earthquake 0.50%, 1.15%, and 0.97%, for Northridge Earthquake 0.12%, 0.71%, and 0.73% for first, second and third story levels respectively for each earthquake. As a result, first story the drift ratio values did not exceed the limitation of IO, while second and third stories have exceeded the limitation of IO remained in LS limit.

However, global structure has lost its elastic properties and structure reached the plastic limits and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore, this structure can be assumed in LS performance level.

5.5.7 Model SS3 P-Delta Results

According to figures given in APPENDIX B; peak roof displacements values were found to be 89, 82 and 66 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce and Northridge earthquakes, hinge formations are investigated. Totally four section ends have exceeded their elastic limits. All hinges occurred at first and second story beams sections. For the El Centro earthquake, totally six hinges occurred, four hinges at the first and second story beam ends, and two hinges are at the first story column's bottom face.

None of the earthquake records created stability or mechanism problem for SS3 P-Delta structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.7.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members were investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.8. In this table FEMA 356 rotation limitations were compared with maximum plastic rotations in terms of radian.

5.5.7.2 Global Level Evaluation

In order to evaluate global performance of structure, transient and permanent interstory drift ratios were investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.15.

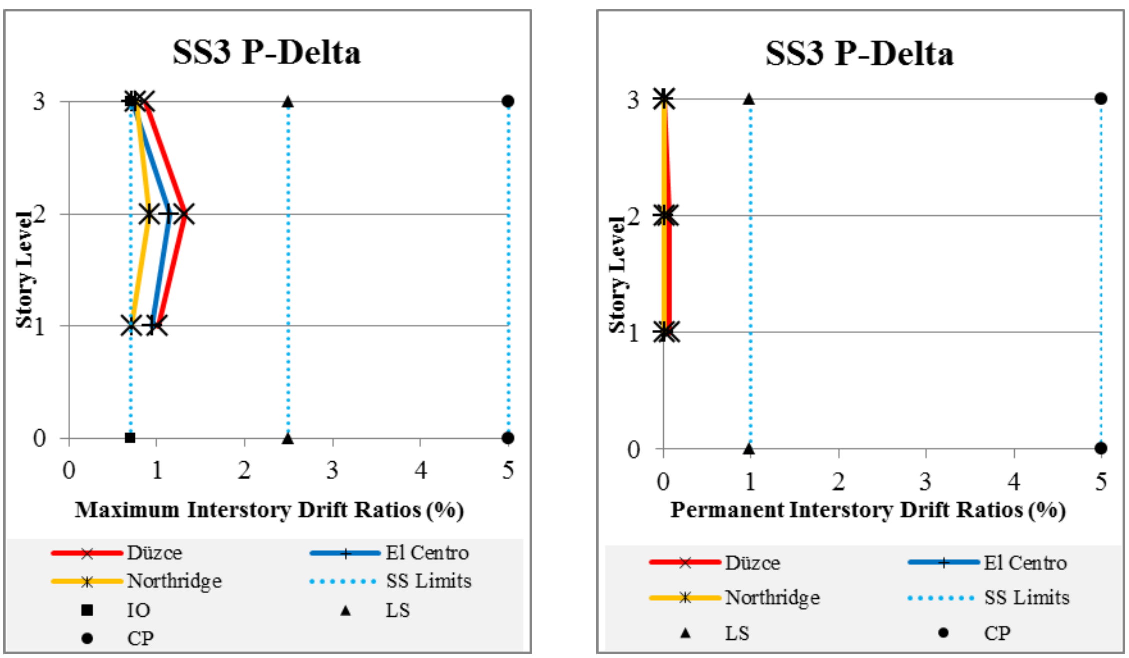


Figure 5.15: SS3 P-Delta maximum transient and permanent interstory drift ratios.

Table 5.8 SS3 P-Delta Plastic formations and performance levels.

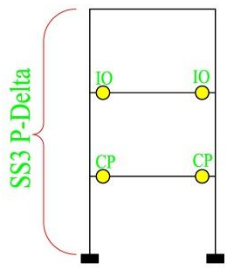
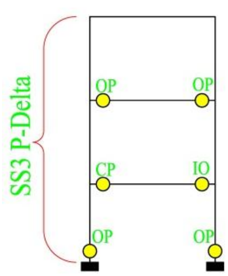
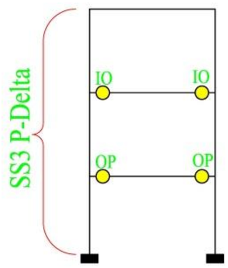
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce	1	BEAM	SS	7	2	137.99	**0.0411**	0.01 0.02 0.025	**CP**
	2	BEAM	SS	7	6	136.3	**0.0380**	0.00216 0.0173 0.0259	**CP**
	3	BEAM	SS	8	3	119.52	**0.0085**	0.00216 0.0173 0.0259	**IO**
	4	BEAM	SS	8	7	119.52	**0.0101**	0.00216 0.0173 0.0259	**IO**
El Centro	1	BEAM	SS	7	2	112.44	**0.0329**	0.00216 0.0173 0.0259	**CP**
	2	BEAM	SS	7	6	105.51	**0.0131**	0.00216 0.0173 0.0259	**IO**
	3	BEAM	SS	8	3	101.47	**0.0016**	0.00216 0.0173 0.0259	**OP**
	4	BEAM	SS	8	7	101.46	**0.0016**	0.00216 0.0173 0.0259	**OP**
	5	COLUMN	SS	1	1	132.31	**0.0007**	0.00179 0.0143 0.0215	**OP**
	6	COLUMN	SS	4	5	132.36	**0.0002**	0.00179 0.0143 0.0215	**OP**

Table 5.8 continued.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Northridge 	1	BEAM	RC	7	2	101.53	**0.0018**	0.00216 0.0173 0.0259	**OP**
	2	BEAM	RC	7	6	136.3	**0.0014**	0.00216 0.0173 0.0259	**OP**
	3	BEAM	SS	8	3	106.55	**0.0167**	0.00216 0.0173 0.0259	**IO**
	4	BEAM	SS	8	7	103.66	**0.0078**	0.00216 0.0173 0.0259	**IO**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel.

According to results, transient interstory drift ratios for Düzce earthquake were 1.01%, 1.32%, and 0.87%, for El Centro Earthquake 0.94%, 1.14%, and 0.72%, for Northridge Earthquake 0.71%, 0.92%, and 0.77% for first, second and third story levels respectively for each earthquake. As a result all stories have exceeded the limitations of IO and the structure is in the limitation of LS.

However, global structure has lost its elastic properties and structure totally reached to the plastic limits, and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore, this structure can be assumed in LS performance level.

5.5.8 Model RC4 Results

According to figures given in APPENDIX B; peak roof displacements values are investigated to be 59, 48 and 42 mm for Düzce, El Centro and Northridge earthquakes respectively. In Düzce, El Centro and Northridge earthquakes, hinge formations are investigated. Totally ten section ends have exceeded their elastic limits for each earthquake. Eight hinges occurred at first, second, third and fourth story beam ends. Two hinges are formed at first story bottom face of the column sections.

None of the earthquake records created stability and mechanism problem for RC4 structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.8.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.9. In this table FEMA 356 rotation limitations were compared with maximum plastic rotation in terms of radian.

Table 5.9: RC4 Plastic formations and performance levels.

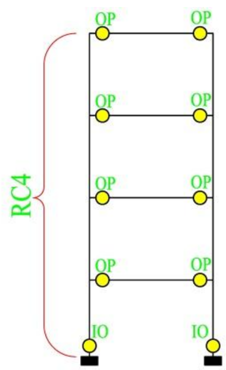
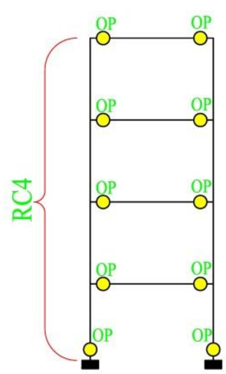
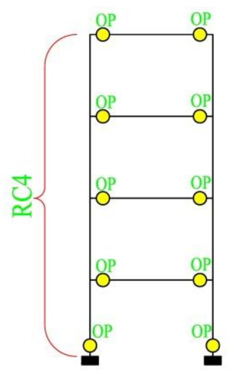
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce 	1	BEAM	RC	9	2&7	92.42	**0.0091**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	10	3&8	92.21	**0.0089**	0.01 0.02 0.025	**OP**
	3	BEAM	RC	11	4&9	89.31	**0.0063**	0.01 0.02 0.025	**OP**
	4	COLUMN	RC	1&4	1&6	178	**0.0051**	0.005 0.015 0.02	**IO**
	5	BEAM	RC	9	4&8	86.13	**0.0035**	0.01 0.02 0.025	**OP**
El Centro 	1	BEAM	RC	9	2&7	90.8	**0.0076**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	10	3&8	89.45	**0.0065**	0.01 0.02 0.025	**OP**
	3	BEAM	RC	11	4&9	86.33	**0.0037**	0.01 0.02 0.025	**OP**
	4	COLUMN	RC	1&4	1&6	177.17	**0.0044**	0.005 0.015 0.02	**OP**
	5	BEAM	RC	9	4&8	83.24	**0.0011**	0.01 0.02 0.025	**OP**

Table 5.9: continued.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Northridge 	1	BEAM	RC	9	2&7	87.32	**0.0045**	0.01 0.02 0.025	**OP**
	2	BEAM	RC	10	3&8	87.95	**0.0051**	0.01 0.02 0.025	**OP**
	3	BEAM	RC	11	4&9	85.79	**0.0033**	0.01 0.02 0.025	**OP**
	4	COLUMN	RC	1&4	1&6	173.15	**0.0012**	0.005 0.015 0.02	**OP**
	5	BEAM	RC	9	4&8	82.64	**0.0004**	0.01 0.02 0.025	**OP**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.8.2 Global Level Evaluation

In order to evaluate global performance of structure transient and permanent interstory drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.16.

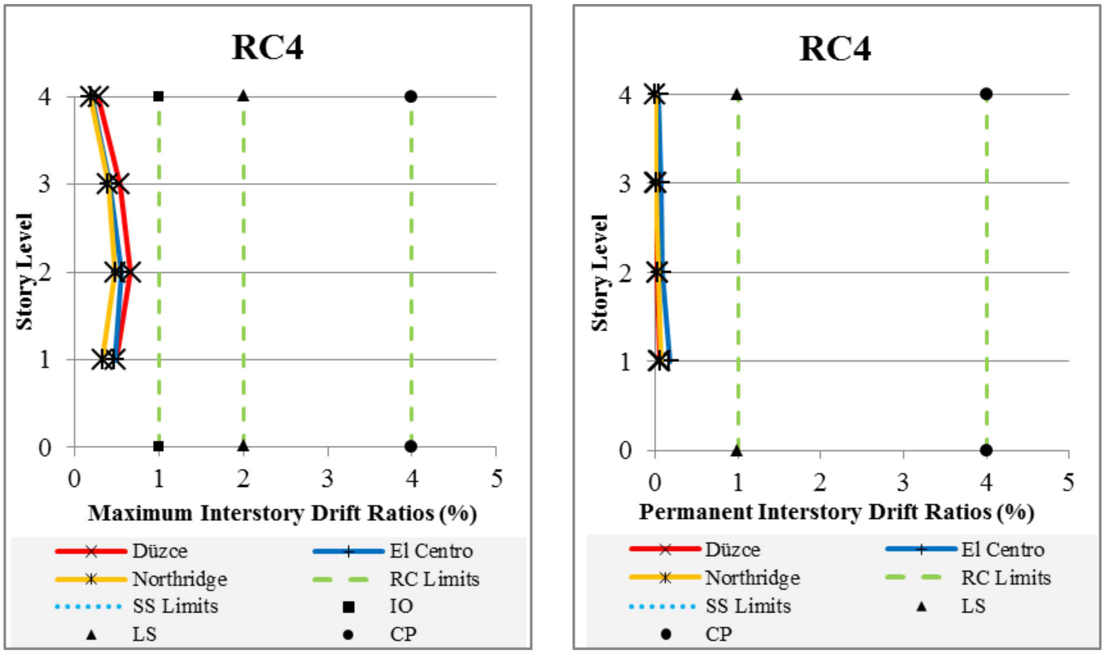


Figure 5.16: RC4 maximum transient and permanent interstory drift ratios.

According to results, transient interstory drift ratios for Düzce earthquake were 0.49%, 0.66%, 0.54% and 0.29%, for El Centro Earthquake 0.48%, 0.56%, 0.42% and 0.20%, for Northridge Earthquake 0.33%, 0.49%, 0.40% and 0.19% for first, second, third and fourth story levels respectively for each earthquake. As a result none of the stories could exceed IO drift limits.

However, global structure has lost its elastic properties and structure totally reached to the plastic limits, and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore, this structure can be assumed in LS performance level.

5.5.9 Model RC1SS3 Results

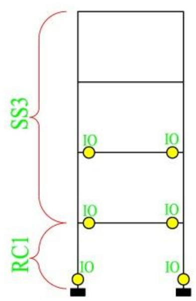
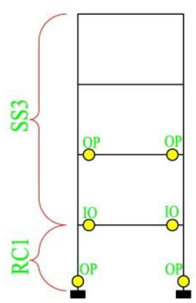
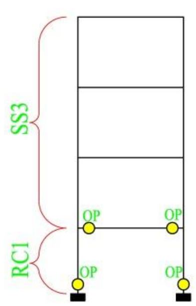
According to figures given in APPENDIX B; peak roof displacements values are investigated to be 112, 81 and 72 mm for Düzce, El Centro and Northridge earthquakes respectively. In Düzce and El Centro earthquakes, hinge formations are investigated. Totally six section ends have exceeded their elastic limits. Four hinges occurred at first and second story beam sections. Two hinges are formed at first story bottom face of the column sections. In Northridge earthquake, totally four hinges are formed, two of them at third story beam ends and the others are at the first story columns bottom face.

None of the earthquake records created stability and mechanism problem for RC1SS3 structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.9.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.10. In this table FEMA 356 rotation limitations were compared with maximum plastic rotation in terms of radian.

Table 5.10: RC1SS3 Plastic formations and performance levels.

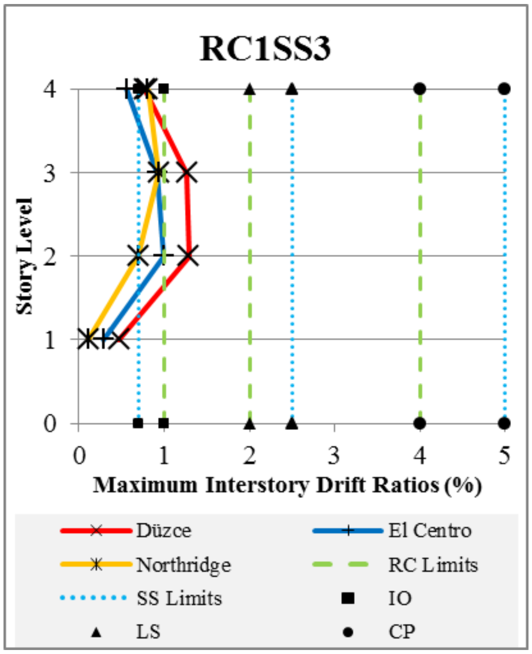
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce 	1	BEAM	RC	9	2&7	99.08	**0.0149**	0.01 0.02 0.025	**IO**
	2	COLUMN	RC	1&5	1&6	176.69	**0.0098**	0.005 0.015 0.02	**IO**
	3	BEAM	SS	10	3&8	102.94	**0.0059**	0.00216 0.0173 0.0259	**IO**
El Centro 	1	BEAM	RC	9	2&7	90.55	**0.0062**	0.01 0.02 0.025	**IO**
	2	COLUMN	RC	1&5	1&6	170.49	**0.00319**	0.005 0.015 0.02	**OP**
	3	BEAM	SS	10	3&8	94.74	**0.00006**	0.00216 0.0173 0.0259	**OP**
Northridge 	1	BEAM	RC	9	4&9	84.86	**0.002**	0.01 0.02 0.025	**OP**
	2	COLUMN	RC	1&5	1&6	152.03	**0.000045**	0.005 0.015 0.02	**OP**
	-	-	-	-	-	-	-	-	-

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.9.2 Global Level Evaluation

In order to evaluate global performance of structure transient and permanent interstory drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.17.



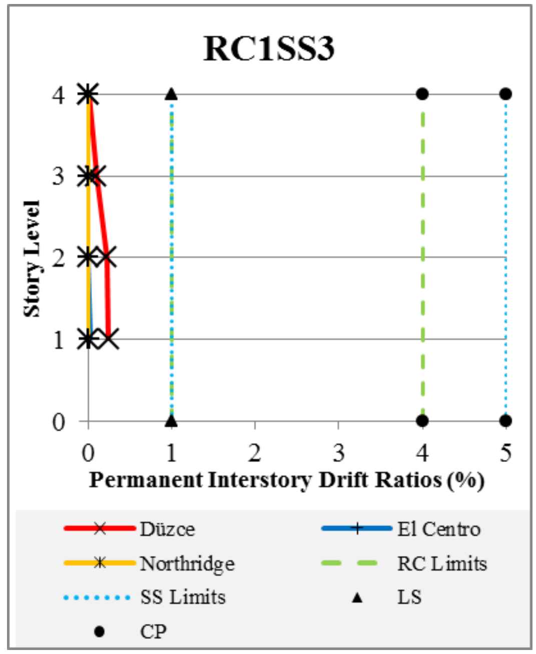


Figure 5.17: RC1SS3 maximum transient and permanent interstory drift ratios.

According to results, transient interstory drift ratios for Düzce earthquake were 0.47%, 1.29%, 1.26% and 0.77%, for El Centro Earthquake 0.29%, 1.00%, 0.92% and 0.56%, for Northridge Earthquake 0.11%, 0.69%, 0.93% and 0.81% for first, second, third and fourth story levels respectively for each earthquake. As a result, first story's drift ratio values did not exceed the limitation of IO, while second third and fourth stories have exceeded the limitation of IO remained in LS limit.

However, structure totally reached to the plastic limits and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore this structure can be assumed in LS performance level.

5.5.10 Model SS4 Results

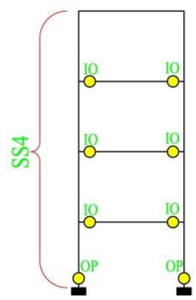
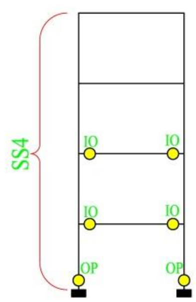
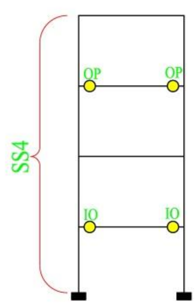
According to figures given in APPENDIX B; peak roof displacements values are investigated to be 172, 134 and 79 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce earthquake, hinge formations are investigated. Totally eight section ends have exceeded their elastic limits. Six hinges occurred at first, second and third story beam ends. Two hinges are formed at first story bottom face of the column sections. For the El Centro earthquake, totally six hinges occurred, four hinges at the first and second story beam ends, and two hinges were at the first story column's bottom faces. For the Northridge earthquake, totally four hinges are formed, all at them has occurred at first and third story beam ends.

None of the earthquake records created stability and mechanism problem for SS4 structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.10.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.11 In this table FEMA 356 rotation limitations were compared with maximum plastic rotation in terms of radian.

Table 5.11: SS4 plastic formations and performance levels.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce 	1	BEAM	SS	9	2&7	104.32	**0.0097**	0.00216 0.0173 0.0259	**IO**
	2	BEAM	SS	10	3&8	92.21	**0.0148**	0.00216 0.0173 0.0259	**IO**
	3	BEAM	SS	11	4&9	102.18	**0.0037**	0.00216 0.0173 0.0259	**IO**
	4	COLUMN	SS	1&4	1&6	132.5	**0.00056**	0.00179 0.0143 0.0215	**OP**
El Centro 	1	BEAM	SS	9	2&7	104.2	**0.0095**	0.00216 0.0173 0.0259	**IO**
	2	BEAM	SS	10	3&8	103.68	**0.0078**	0.00216 0.0173 0.0259	**IO**
	3	COLUMN	SS	1&4	1&6	132.64	**0.00074**	0.00179 0.0143 0.0215	**OP**
Northridge 	1	BEAM	SS	9	2&7	101.93	**0.0028**	0.00216 0.0173 0.0259	**IO**
	2	BEAM	SS	11	4&9	101.52	**0.0017**	0.00216 0.0173 0.0259	**OP**
	-	-	-	-	-	-	**-**	-	**-**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.9.2 Global Level Evaluation

In order to evaluate global performance of structure transient and permanent interstory drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.18.

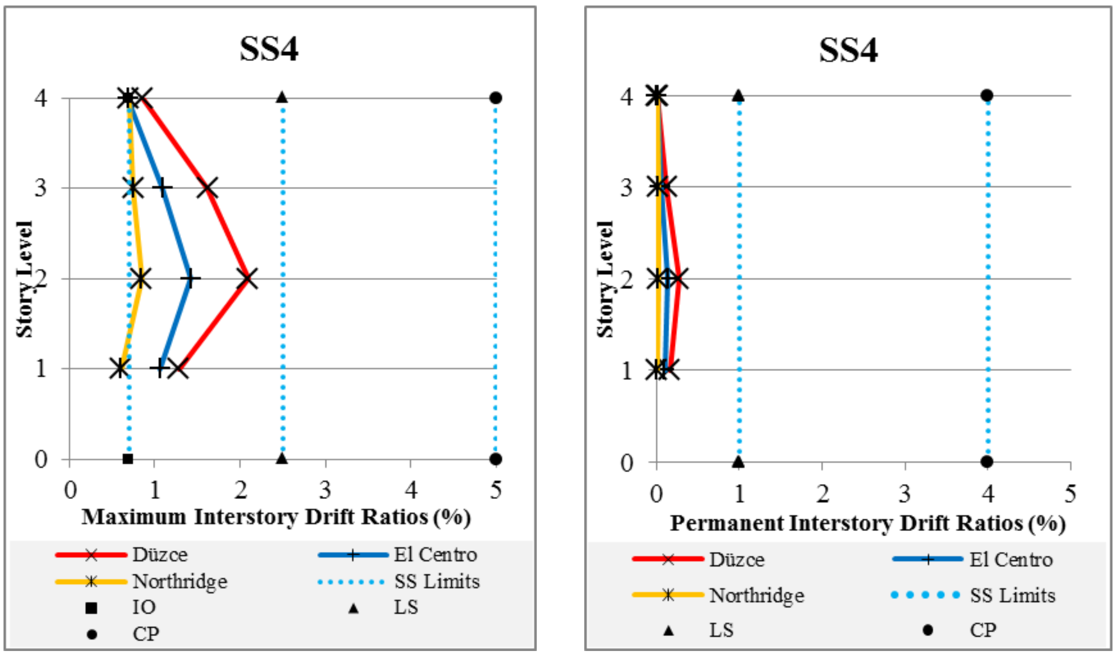


Figure 5.18: SS4 maximum transient and permanent interstory drift ratios.

According to results, transient interstory drift ratios for Düzce earthquake were 1.28%, 2.06%, 1.62% and 0.86%, for El Centro Earthquake 1.06%, 1.43%, 1.09% and 0.69%, for Northridge Earthquake 0.60%, 0.85%, 0.75% and 0.69% for first, second, third and fourth story levels respectively for each earthquake. As a result, in Düzce and El Centro earthquakes all stories drift ratios exceeded the IO limit. For Northridge, first story results are below the IO limit but second, third and fourth stories have exceeded this limit.

Structure totally reached to the plastic limits and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore this structure can be assumed in LS performance level, but close to the LS level.

5.5.11 Model RC4 P-Delta Results

According to figures given in APPENDIX B; peak roof displacements values are investigated to be 67, 49 and 46 mm for Düzce, El Centro and Northridge earthquakes respectively. In Düzce, El Centro and Northridge earthquakes, hinge formations are investigated. Totally ten section ends have exceeded their elastic limits. Eight hinges occurred at first, second, third and fourth story beam ends. Two hinges are formed at first story bottom face of the column sections.

None of the earthquake records created stability and mechanism problem for RC4 P-Delta structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.11.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.12. In this table FEMA 356 rotation limitations were compared with maximum plastic rotation in terms of radian.

Table 5.12: RC4 P-Delta, Plastic formations and performance levels.

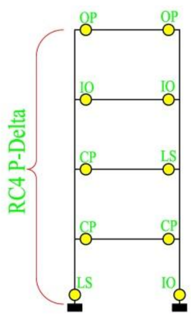
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce	1	BEAM	RC	9	2	112.73	**0.0269**	0.01 0.02 0.025	**CP**
	2	BEAM	RC	9	7	113.76	**0.0278**	0.01 0.02 0.025	**CP**
	3	BEAM	RC	10	3	115.09	**0.029**	0.01 0.02 0.025	**CP**
	4	BEAM	RC	10	8	107.44	**0.0232**	0.01 0.02 0.025	**LS**
	5	BEAM	RC	11	4	95.07	**0.0114**	0.01 0.02 0.025	**IO**
	6	BEAM	RC	11	9	94.3	**0.0107**	0.01 0.02 0.025	**IO**
	7	COLUMN	RC	1	1	182.3	**0.019**	0.005 0.015 0.02	**LS**
	8	COLUMN	RC	4	6	182.3	**0.011**	0.005 0.015 0.02	**IO**
	9	BEAM	RC	12	5	85.81	**0.00327**	0.01 0.02 0.025	**OP**
	10	BEAM	RC	12	10	85.71	**0.00319**	0.01 0.02 0.025	**OP**

Table 5.12: continued.

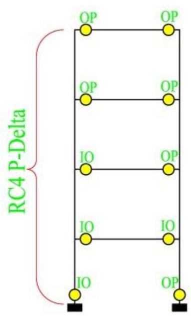
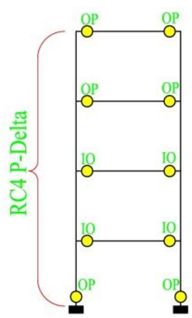
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
El Centro 	1	BEAM	RC	9	2	96.4	**0.0126**	0.01 0.02 0.025	**IO**
	2	BEAM	RC	9	7	94.52	**0.0113**	0.01 0.02 0.025	**IO**
	3	BEAM	RC	10	3	94.74	**0.0112**	0.01 0.02 0.025	**IO**
	4	BEAM	RC	10	8	93.21	**0.0097**	0.01 0.02 0.025	**OP**
	5	BEAM	RC	11	4	89.35	**0.0064**	0.01 0.02 0.025	**OP**
	6	BEAM	RC	11	9	88.49	**0.0058**	0.01 0.02 0.025	**OP**
	7	COLUMN	RC	1	1	178.11	**0.0053**	0.005 0.015 0.02	**IO**
	8	COLUMN	RC	5	6	176.75	**0.0041**	0.005 0.015 0.02	**OP**
	9	BEAM	RC	12	5	83.24	**0.0011**	0.01 0.02 0.025	**OP**
	10	BEAM	RC	12	10	83.16	**0.0009**	0.01 0.02 0.025	**OP**

Table 5.12: continued.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Northridge	1	BEAM	RC	9	2	99.02	**0.0149**	0.01 0.02 0.025	**IO**
	2	BEAM	RC	9	7	99.98	**0.0157**	0.01 0.02 0.025	**IO**
	3	BEAM	RC	10	3	98.44	**0.0143**	0.01 0.02 0.025	**IO**
	4	BEAM	RC	10	8	97.48	**0.0135**	0.01 0.02 0.025	**IO**
	5	BEAM	RC	11	4	91.71	**0.0089**	0.01 0.02 0.025	**OP**
	6	BEAM	RC	11	9	90.24	**0.0086**	0.01 0.02 0.025	**OP**
	7	COLUMN	RC	1	1	173.16	**0.0014**	0.005 0.015 0.02	**OP**
	8	COLUMN	RC	5	6	173.32	**0.0014**	0.005 0.015 0.02	**OP**
	9	BEAM	RC	12	5&10	82.3	**0.0002**	0.01 0.02 0.025	**OP**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.11.2 Global Level Evaluation

In order to evaluate global performance of structure transient and permanent interstory drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.19.

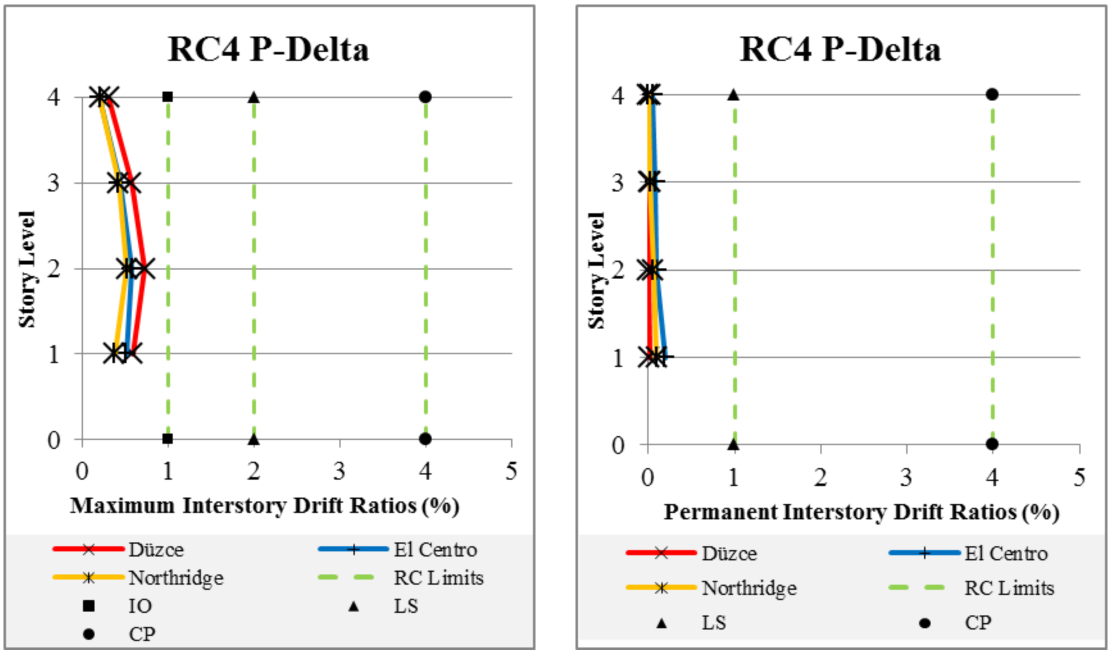


Figure 5.19: RC4 P-Delta maximum transient and permanent interstory drift ratios.

According to results, transient interstory drift ratios for Düzce earthquake were 0.58%, 0.72%, 0.57% and 0.31%, for El Centro Earthquake 0.50%, 0.57%, 0.43% and 0.20%, for Northridge Earthquake 0.37%, 0.52%, 0.41% and 0.19% for first, second, third and fourth story levels respectively for each earthquake. As a result none of the stories could exceed IO drift limits.

However, global structure has lost its elastic properties and structure totally reached to the plastic limits, and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore, this structure can be assumed in LS performance level.

5.5.12 Model RC1SS3 P-Delta Results

According to figures given in APPENDIX B; peak roof displacements values are investigated to be 108, 82 and 70 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce earthquake, hinge formations are investigated. Totally six section ends have exceeded their elastic limits. Four hinges occurred at first and second story beam sections. Two hinges are formed at first story bottom face of the column sections. For the El Centro earthquake, totally five hinges occurred, three hinges at the first and second story beam ends, and two hinges were at the first story column's bottom faces. For the Northridge earthquake, totally four hinges are formed, two of them at first story beam ends and the others are at the first story columns bottom face.

None of the earthquake records created stability and mechanism problem for RC1SS3 P-Delta structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.12.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.13. In this table FEMA 356 rotation limitations were compared with maximum plastic rotation in terms of radian.

Table 5.13: RC1SS3 P-Delta Plastic formations and performance levels.

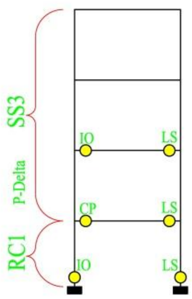
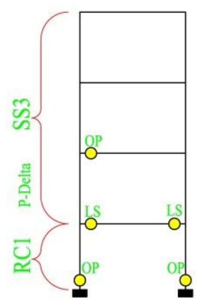
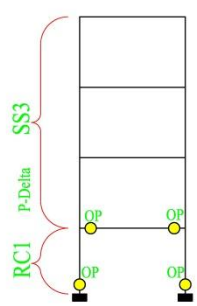
Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce	1	BEAM	RC	9	2	110.62	**0.0251**	0.01 0.02 0.025	**CP**
	2	BEAM	RC	9	7	108.5	**0.0236**	0.01 0.02 0.025	**LS**
	3	BEAM	SS	10	3	105.69	**0.0138**	0.00216 0.0173 0.0259	**IO**
	4	BEAM	SS	10	8	108.39	**0.021**	0.00216 0.0173 0.0259	**LS**
	5	COLUMN	RC	1	1	180.67	**0.0133**	0.005 0.015 0.02	**IO**
	6	COLUMN	RC	5	6	184.26	**0.015**	0.005 0.015 0.02	**LS**

Table 5.13: continue.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
El Centro 	1	BEAM	RC	9	2	95.82	**0.012**	0.01 0.02 0.025	**IO**
	2	BEAM	RC	9	7	95.54	**0.0118**	0.01 0.02 0.025	**IO**
	3	COLUMN	RC	1&5	1&6	168.38	**0.0037**	0.005 0.015 0.02	**OP**
	4	COLUMN	RC	1&5	1&6	169.49	**0.0045**	0.005 0.015 0.02	**OP**
	5	BEAM	SS	10	3	100.93	**0.000032**	0.00216 0.0173 0.0259	**OP**
Northridge 	1	BEAM	RC	11	4&9	84.88	**0.002**	0.01 0.02 0.025	**OP**
	2	COLUMN	RC	1&4	1&6	152.03	**0.000045**	0.005 0.015 0.02	**OP**
	-	-	-	-	-	-	**-**	-	**-**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.12.2 Global Level Evaluation

In order to evaluate global performance of structure transient and permanent interstory drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.20.

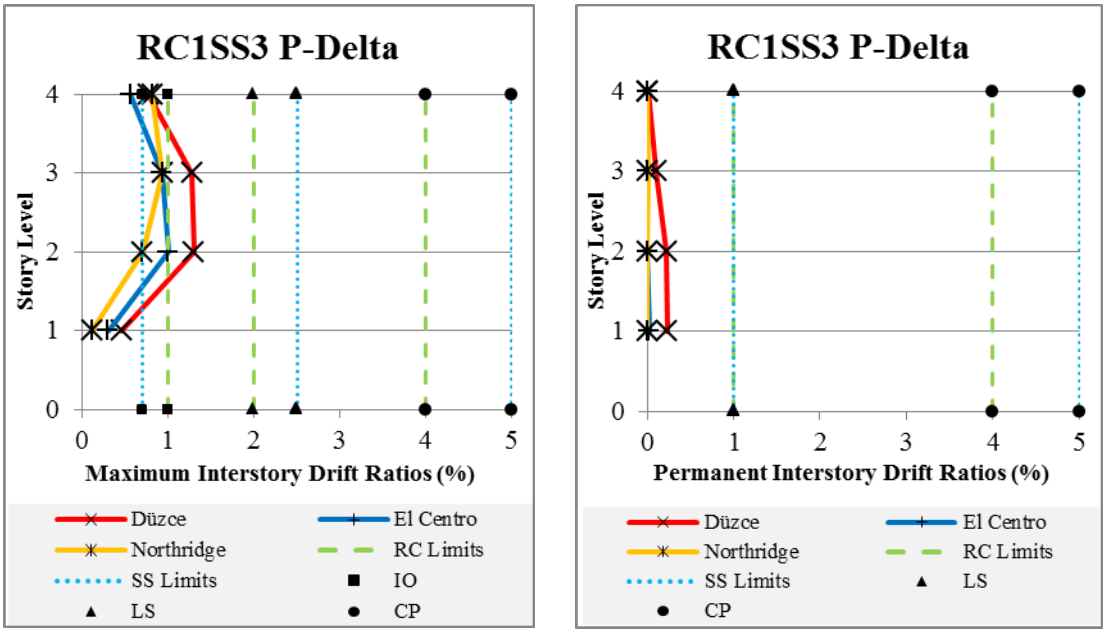


Figure 5.20: RC1SS3 P-Delta maximum transient and permanent interstory drift ratios.

According to results, transient interstory drift ratios for Düzce earthquake were 0.45%, 1.29%, 1.27% and 0.77%, for El Centro Earthquake 0.29%, 1.00%, 0.92% and 0.56%, for Northridge Earthquake 0.11%, 0.70%, 0.93% and 0.81% for first, second, third and fourth story levels respectively for each earthquake. As a result, first story's drift ratio values did not exceed the limitation of IO. Düzce and Northridge results of second third and fourth stories have exceeded the limitation of IO. But in El Centro results fourth story was below the IO limit.

However, structure totally reached to the plastic limits and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore this structure can be assumed in LS performance level.

5.5.13 Model SS4 P-Delta Results

According to figures given in APPENDIX B; peak roof displacements values are investigated to be 174, 105 and 76 mm for Düzce, El Centro and Northridge earthquakes respectively. For the Düzce earthquake, hinge formations are investigated. Totally eight section ends have exceeded their elastic limits. Six hinges occurred at first, second and third story beam ends. Two hinges are formed at first story bottom face of the column sections. For the El Centro earthquake, totally six hinges occurred, four hinges at the first and second story beam ends, and two hinges were at the first story column's bottom faces. For the Northridge earthquake, totally four hinges are formed, all at them has occurred at first and third story beam ends.

None of the earthquake records created stability and mechanism problem for SS4 P-Delta structural model. Global structure and member level evaluations are required to have decision of building structures performance level. Firstly Member level evaluation is carried out.

5.5.13.1 Member Level Evaluation

In order to evaluate the seismic performance assessment, structural members are investigated according to results of rotations of the section ends. Therefore, maximum plastic rotations that were collected from dynamic analysis are summarized with formation sequences of the hinges in Table 5.14. In this table FEMA 356 rotation limitations were compared with maximum plastic rotation in terms of radian.

Table 5.14: SS4 P-Delta Plastic formations and performance levels.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
Düzce	1	BEAM	SS	9	2	109.71	**0.0252**	0.00216 0.0173 0.0259	**LS**
	2	BEAM	SS	9	7	110.49	**0.0272**	0.00216 0.0173 0.0259	**CP**
	3	COLUMN	SS	1	1	134.78	**0.0046**	0.00179 0.0143 0.0215	**IO**
	4	COLUMN	SS	5	6	134.99	**0.0049**	0.00179 0.0143 0.0215	**IO**
	5	BEAM	SS	10	3	107.72	**0.0194**	0.00216 0.0173 0.0259	**IO**
	6	BEAM	SS	10	8	105.03	**0.0117**	0.00216 0.0173 0.0259	**IO**
	7	BEAM	SS	11	4	103.39	**0.007**	0.00216 0.0173 0.0259	**IO**
	8	BEAM	SS	11	9	102.89	**0.0054**	0.00216 0.0173 0.0259	**IO**

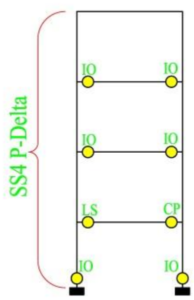
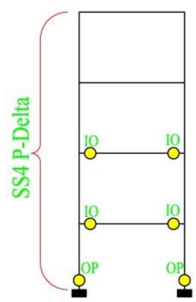
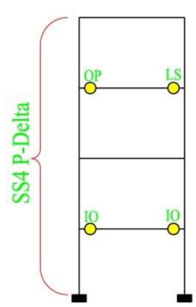


Table 5.14: continued.

Damage with Probability of exceedance 10% in 50 years	Sequence	Section	Material	Member	Joint	Moments (kNm)	Maximum Plastic Rotation (rad)	FEMA 356 limits (rad)	Performance Level
El Centro 	1	BEAM	SS	9	2	103.12	**0.0064**	0.00216 0.0173 0.0259	**IO**
	2	BEAM	SS	9	7	104.72	**0.0108**	0.00216 0.0173 0.0259	**IO**
	3	BEAM	SS	10	3	103.65	**0.0079**	0.00216 0.0173 0.0259	**IO**
	4	BEAM	SS	10	8	103.63	**0.0078**	0.00216 0.0173 0.0259	**IO**
	5	COLUMN	SS	1	1	132.61	**0.0007**	0.00179 0.0143 0.0215	**OP**
	6	COLUMN	SS	5	6	132.77	**0.0009**	0.00179 0.0143 0.0215	**OP**
Northridge 	1	BEAM	SS	9	2	102.51	**0.0111**	0.00216 0.0173 0.0259	**IO**
	2	BEAM	SS	9	7	103.17	**0.0082**	0.00216 0.0173 0.0259	**IO**
	3	BEAM	SS	11	4	101.44	**0.0013**	0.00216 0.0173 0.0259	**OP**
	4	BEAM	SS	11	9	101.62	**0.0221**	0.00216 0.0173 0.0259	**LS**

Notes:

1. OP: Operational Performance, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse prevention.
2. RC: Reinforced Concrete, SS: Structural Steel

5.5.13.2 Global Level Evaluation

In order to evaluate global performance of structure transient and permanent interstory drift ratios have been investigated according to limitations given by FEMA 356. Peak values are illustrated for three different earthquake records with different line types in Figure 5.21.

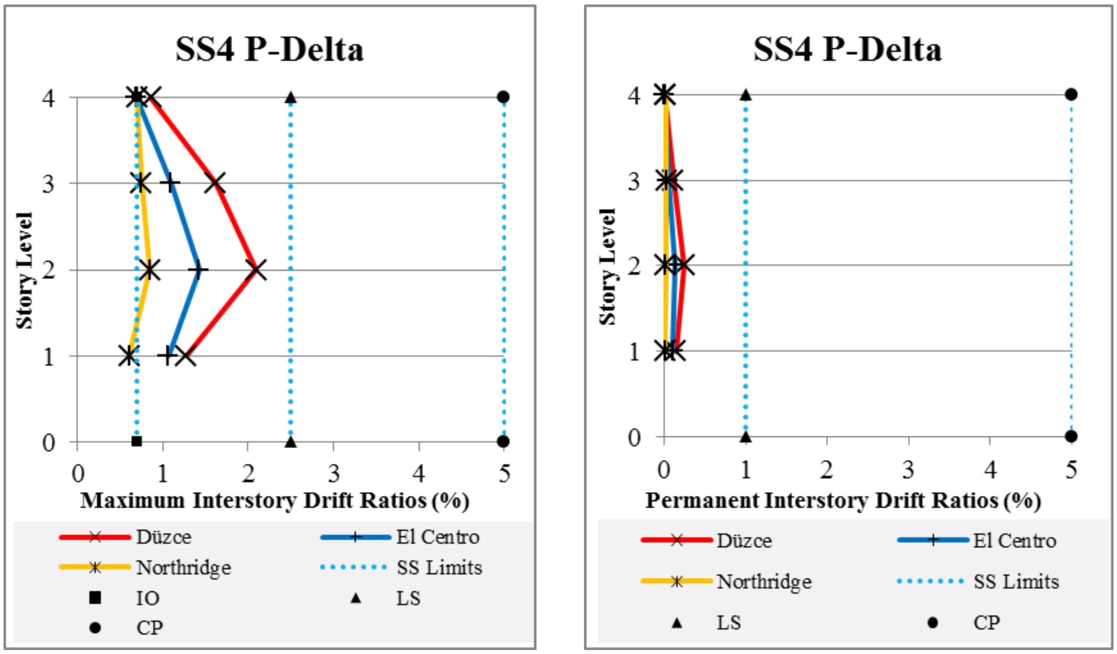


Figure 5.21: SS4 P-Delta maximum transient and permanent interstory drift ratios.

According to results, transient interstory drift ratios for Düzce earthquake were 1.28%, 2.09%, 1.62% and 0.86%, for El Centro Earthquake 1.06%, 1.43%, 1.09% and 0.69%, for Northridge Earthquake 0.60%, 0.85%, 0.75% and 0.69% for first, second, third and fourth story levels respectively for each earthquake. As a result, in Düzce and El Centro earthquakes all stories drift ratios exceeded the IO limit. For Northridge, first story results are below the IO limit but second, third and fourth stories have exceeded this limit.

Structure totally reached to the plastic limits and permanent drifts were occurred. Permanent drift ratios did not exceed the LS limit. Therefore this structure can be assumed in LS performance level.

Chapter 6

DISCUSSION OF RESULTS, CONCLUSION AND RECOMMENDATIONS

6.1 General

In this chapter the results of the presented analysis are discussed to get an idea about the seismic performance of mixed structures compared to concrete and steel structures. Collected results are compared to each other and graphically presented and discussed. As a conclusion, some remarks are given for the results compared. Finally some recommendations are given for the future research works.

6.2 Comparison of Results

In order to compare the results for each type of model, maximum peak story drifts and member hinge performance levels were evaluated. Therefore, the results shall be compared in the same manner with generalized peak values.

In story drift evaluations maximum values were used for each model and each earthquake. The comparison is carried out between the three story and four story models. In Figure 6.1 Peak inter-story drift ratios of different three story models are compared in a bar chart. The same comparison is carried out for four story models, shown in Figure 6.2.

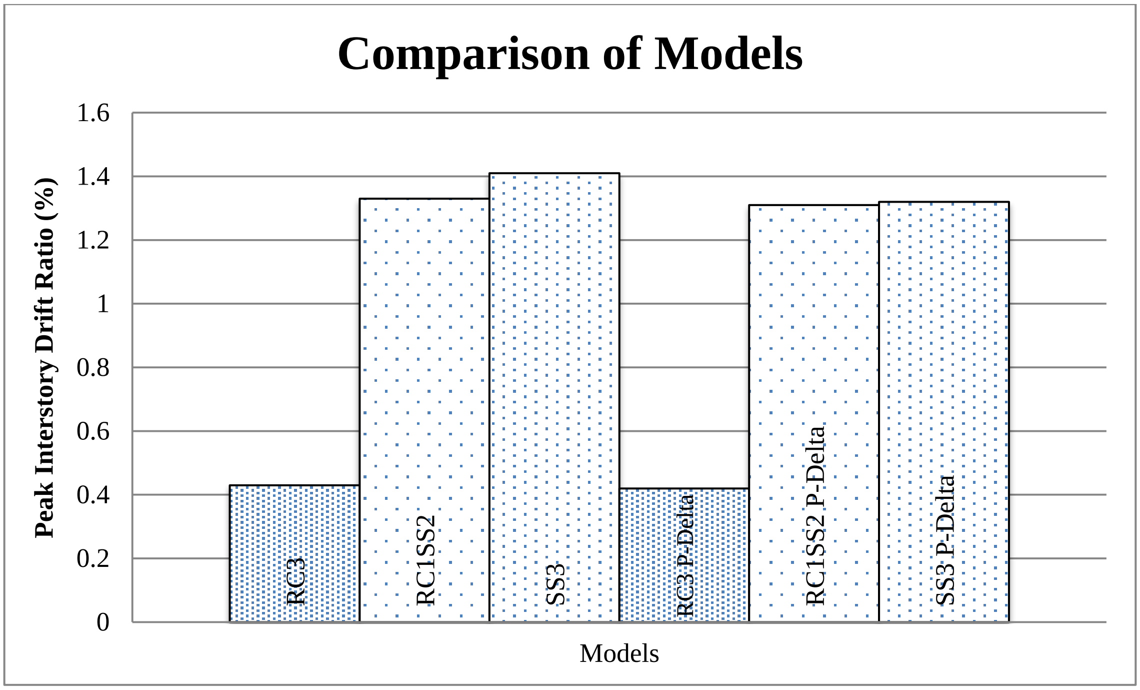


Figure 6.1: Three story models peak interstory drift ratio evaluations.

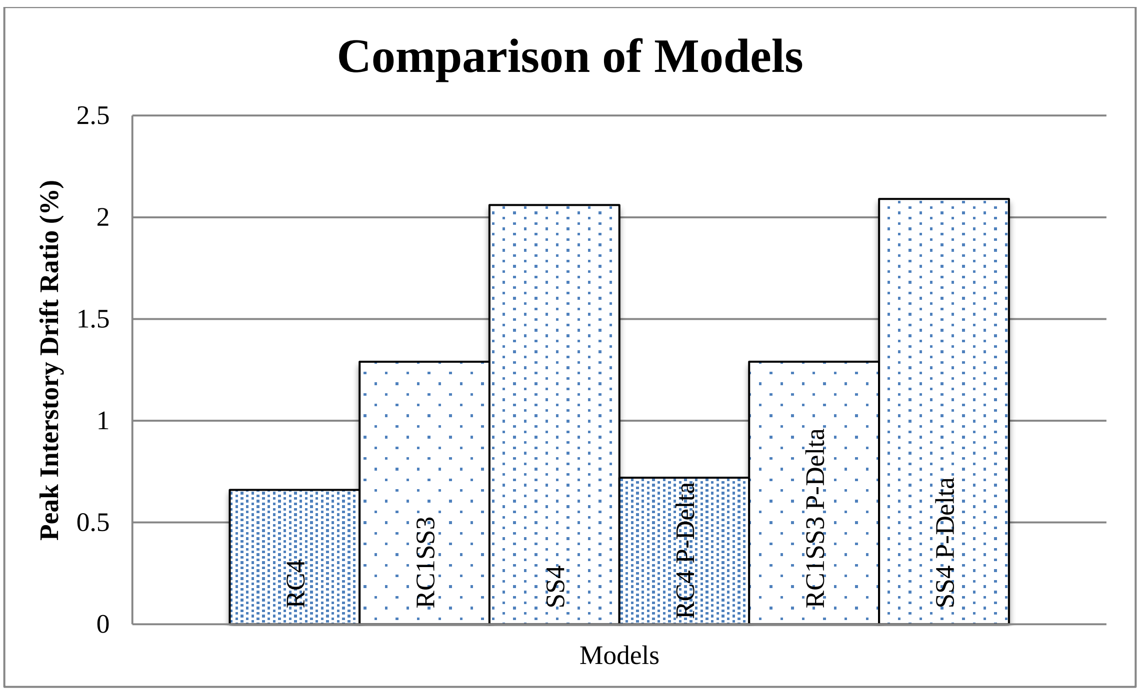


Figure 6.2: Four story models peak interstory drift ratio evaluations.

Considering the above figures, as expected the steel moment frame is more flexible than reinforced concrete frames under dynamic earthquake forces. As a result, most of the steel frame drift results are almost twice as big as reinforced concrete frames.

For the three story models, maximum drift ratios are very close in value, 1.3%, both for mixed frame structure and steel framed models including P-Delta models Reinforced concrete frame results are all about 0.4%. Consequently, reinforced concrete moment frame models are given better results than steel framed and mixed framed structures. Mixed structures drift performances are close to each other and P-Delta effects did not play a significant role in terms of drift ratios for structural models.

The results of four story models differ from those of the three story models when drift ratios are considered. It is observed that the mixed steel concrete models have showed better results than steel framed models. On the other hand consideration of P-Delta had ignorable effects on drift ratio of different models. For the structural models with reinforced concrete frame and also P-delta effect, the maximum values are observed to be 0.66% and 0.72% respectively and these values are less than 1% which is immediate occupancy limit. The displacement differences between steel and concrete mainly originated from the steel moment frames being more flexible than the others. In order to have a better comparison with different parameters, the member damages on the building is required to be investigated thoroughly.

As it is generally known, each earthquake has different characteristic properties and the structural behavior should not be expected to be same in different earthquakes. Therefore, the response of structures may show different results with different earthquakes. According to experimental research results each earthquake has created different damages on the structural models. In order to compare the results, maximum damages on different structural models are used in comparison for columns and beams separately in graphical order with different percentage of damage.

According to these results, the heaviest damage that has occurred on models was in Düzce earthquake with probability of exceedance 10% in 50 years when compare to other earthquakes. Consequently, most Düzce results were included in the comparison of structural member damage. The comparison is carried out using the graphics for beams shown in Figures 6.3 and 6.4, and for columns in Figures 6.5 and 6.6 for three and four story models, respectively.

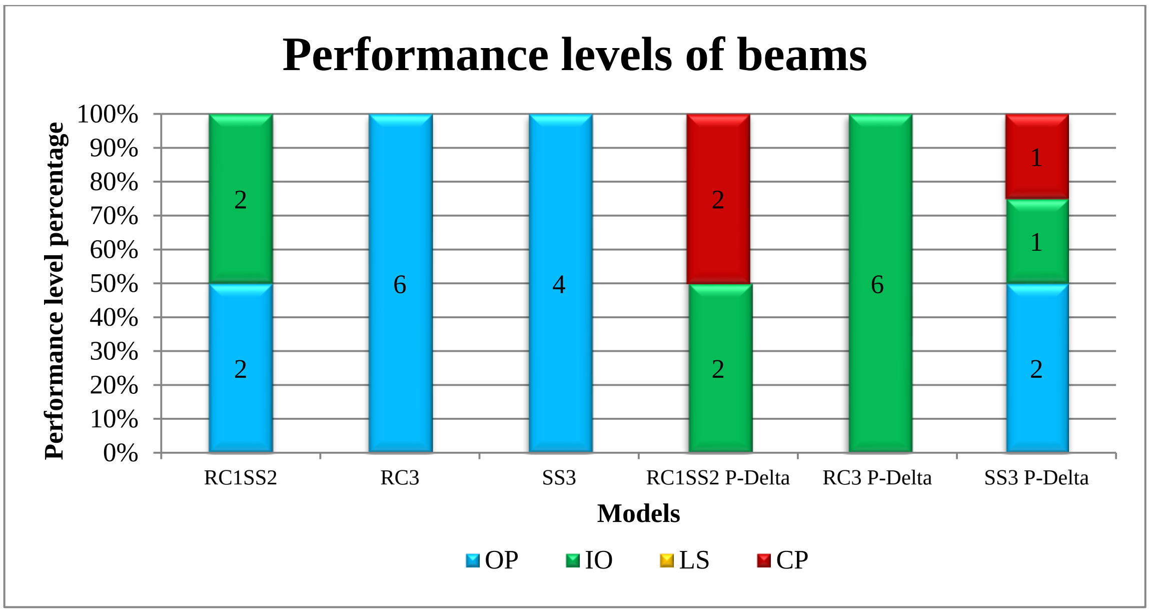


Figure 6.3: Three story models, comparison of beam member performances.

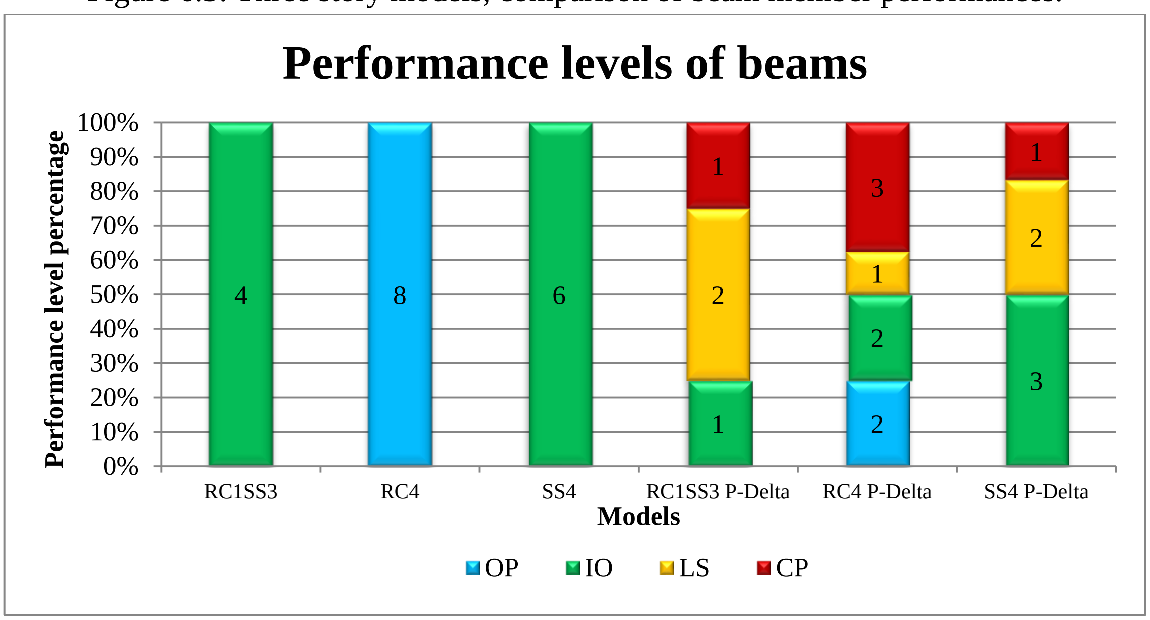


Figure 6.4: Four story models, comparison of beam member performances

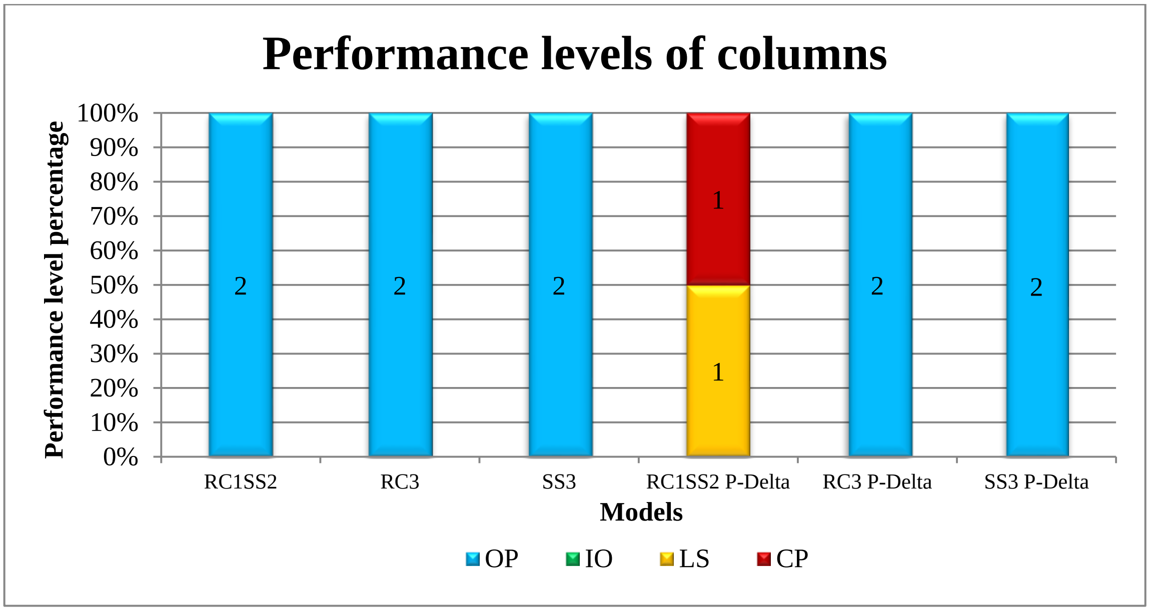


Figure 6.5: Three story models, comparison of column member performances

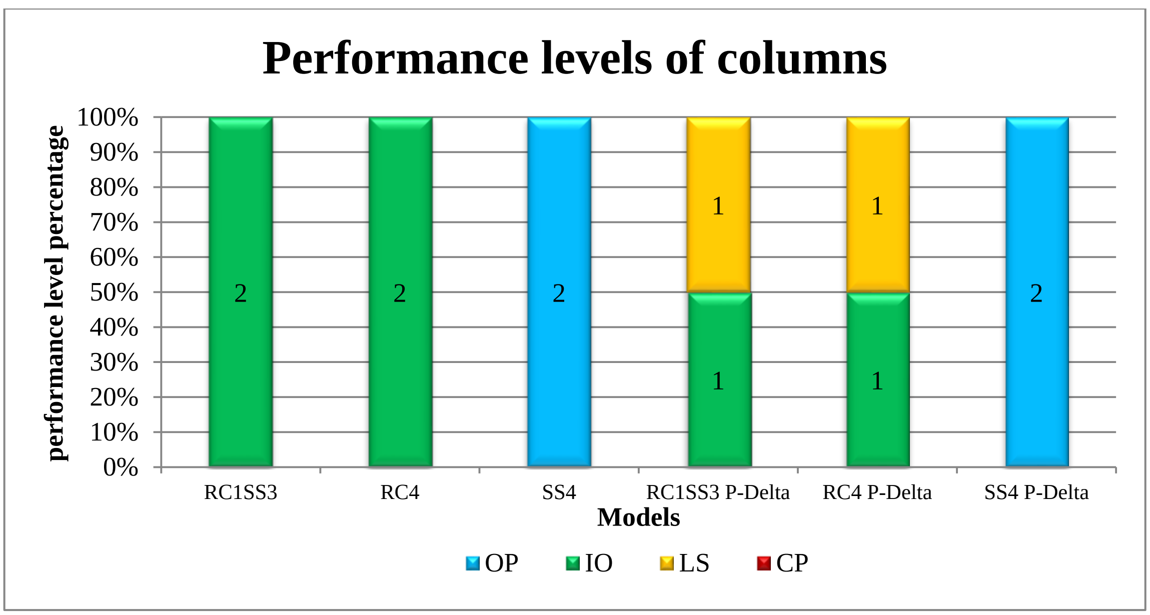


Figure 6.6: Four story models, comparison of column member performances

In member hinge performance level evaluations, the results show a contrast in performance of members when compared with drift performance of structures. The change of story number had also a significant effect on member level performances. Additionally, the geometrical nonlinearity (P-Δ) has extremely affected the hinge performances.

The beam evaluations of the three story models have shown that the highest risk damage has occurred on RC1SS2 P-Delta and SS3 P-Delta models. For the columns of three story models RC1SS2 P-Delta model also had significant damage. Furthermore, the evaluation of columns for the three story models, all of the column's bottom faces reached the plastic limit.

In four story models the results are quite diverse when compared to three story models, in these models more damage has occurred on RC4 P-Delta model. Nevertheless, RC1 SS3 P-Delta and SS4 P-Delta models illustrate high destructions as well. The other models have a number of joints, though they are not so diverse in limits. For instance, a high number of joints are seen in RC4 model (8 joints) but none of them exceeded the limit of Operational Performance. Furthermore, the evaluation of column's for the four story models, all of the column's bottom faces have reached to plastic limit, however, this behavior was expected. Finally, the highest damage was observed in RC1SS3 P-Delta and RC4 P-Delta.

Investigating the hinge locations of the models has shown that the system behaved as designed and the strong column and weak beam variation was completed.

It is interesting to see that RC1SS3 P-Delta model demonstrate a better performance than the RC1SS2 P-Delta model while it has more stories and therefore more loads. Hence, it can be stated that seismic performance cannot be summarized with respect to story numbers or static loads alone, in dynamic analysis.

6.3 Conclusion

This research was aimed to investigate the seismic performance of six analytical models with different framing systems. To have a dynamic response of these six models three different earthquake records were selected and scaled according to code specifications calculated with direct integration method. The considered models were studied mainly according to global level (drift ratios) and member level (moment rotation) evaluations. Finally, the collected results were compared to each other as demonstrated in tables and figures.

It is widely accepted that weight of structural steel frames being lighter than other frames would make it a good option for buildings where there is a need to put additional floors with static vertical loads. In this research it was observed that adding dynamic horizontal load may lead to disprove this well-known theory. If only static methods were used in the analysis of seismic loads, this theory could be fully responding. However, the results show that the structural behavior under dynamic earthquake excitation is far away from expected static actions.

Applied real ground motion records in the dynamic analysis shows that the difference of variables could directly affect the structural behavior. In the evaluation of the member hinge performance levels, the results show a contrast in performance of members when compared with drift performance of structures. The change of story number had also a significant effect on member level performances. Additionally, the geometrical nonlinearity (P-Δ) has extremely affected the hinge performances, but it did not have a significant effect on drift performance of global structures.

Simplest models were selected and used for investigation of seismic behavior of mixed structures. Complex models may not be a good option for the assessment because of the lack of information and code specifications from past studies. In real life, the structural models are not as simple and study on more realistic structural models is required.

6.4 Recommendations for future studies

This research also revealed that dynamic analysis is the most accurate and correct method for the seismic analysis. Nevertheless, extra studies will be needed in these types of framed structural models. The future work can be recommended as;

a) Various types of analytical models including more story numbers, more bay numbers, three dimensional analyses, braced systems, shear walled systems and different combinations of them.
b) The comparison of seismic analysis methods for same models.
c) Real application of steel parts to the roof of existing reinforced concrete structures and optimization of connection and anchoring.
d) Analysis of dynamic behavior of anchored steel parts based on test results from laboratory according to standards.
e) Economic investigation according to daily prices comparing with fees of concrete strengthening and production of steel. For example, instead of constructing more steel stories on an existing reinforced concrete structure, it

may be cheaper to strengthen the existing part and continue with the same material (reinforced concrete).

f) Investigation for using the base isolation system on top of the existing building before adding more stories and aim to save the new stories from additional seismic forces.

g) Developing and creating more statistical information to provide technical support for current codes.

h) In order to have extra stories on existing structure it is expected to have better results when reinforced concrete shear walls with combination of structural steel is extended to the floors above. This combination is expected to lead to a better performance with carrying lateral loads by shear walls and less static vertical load for global structures.

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APPENDICES

Appendix A: Moment-Curvature Relation of Sections.

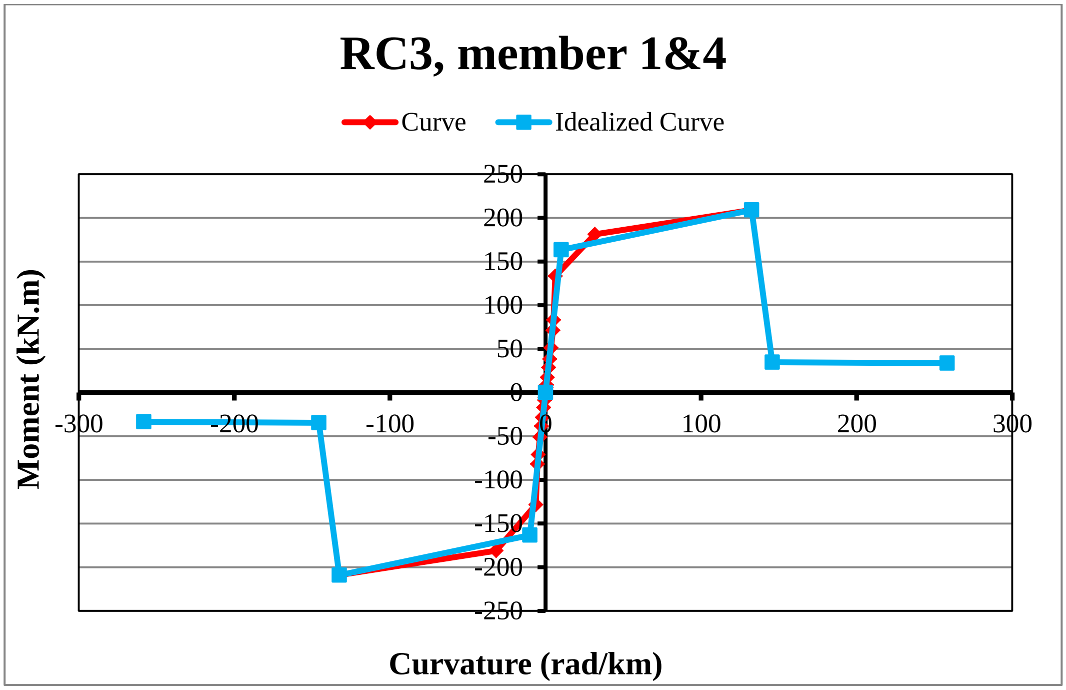

Figure A.1: RC3 Moment-curvature relation of column 1&4 (N= -150.32 kN)

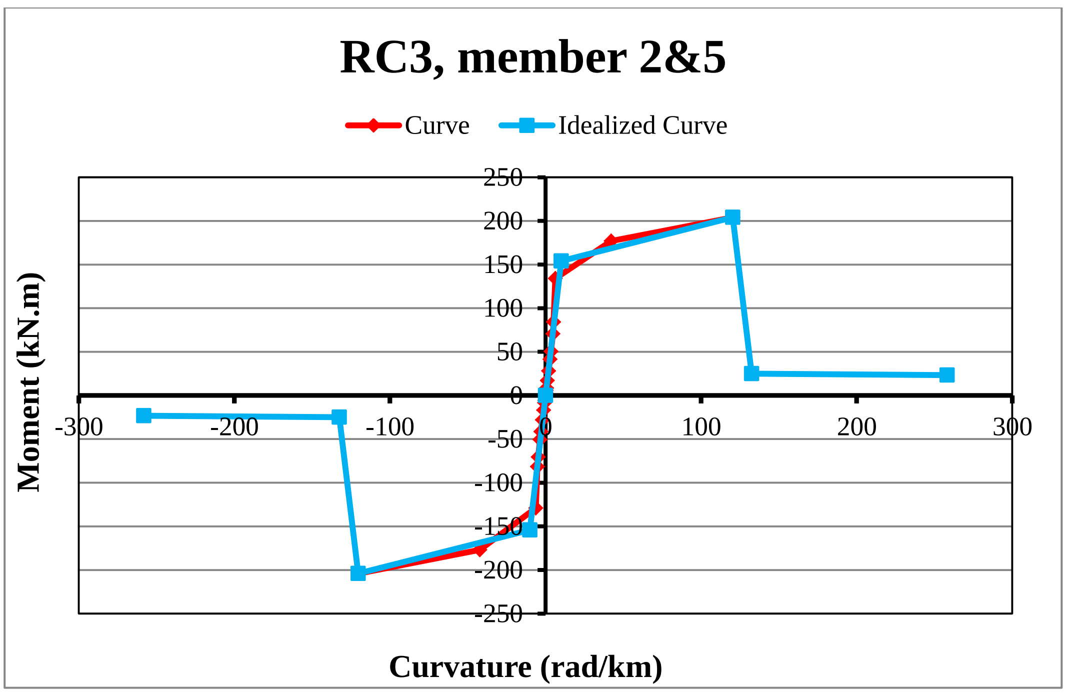

Figure A.2: RC3 Moment-curvature relation of column 2&5 (N= -100.21 kN)

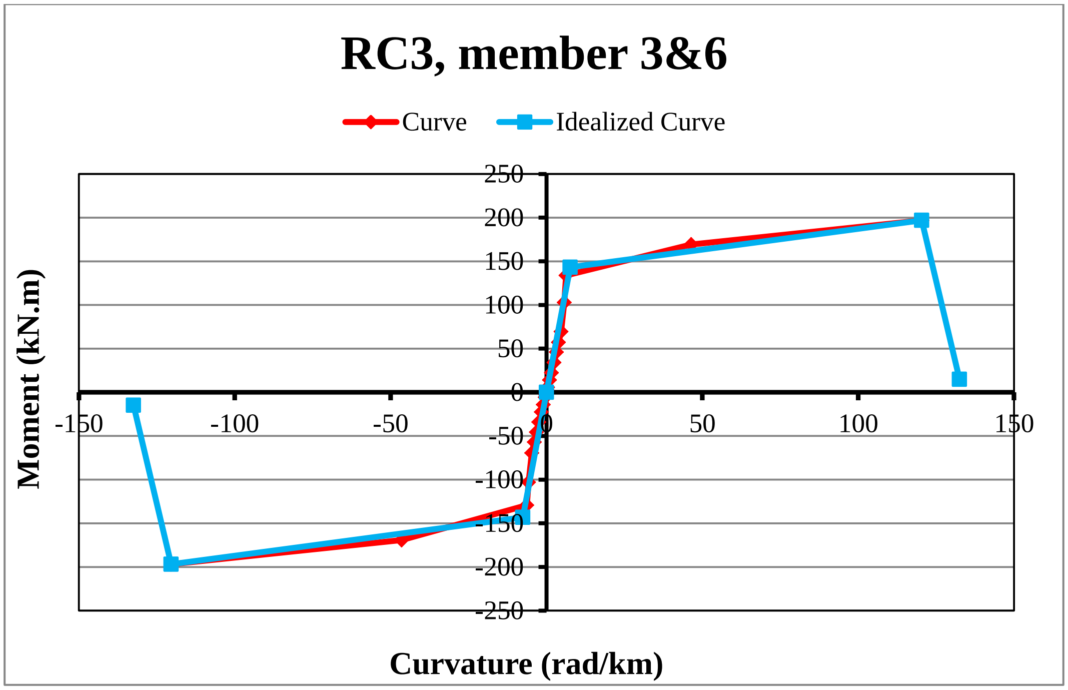

Figure A.3: RC3 Moment-curvature relation of column 3&6 (N= -50.11 kN)

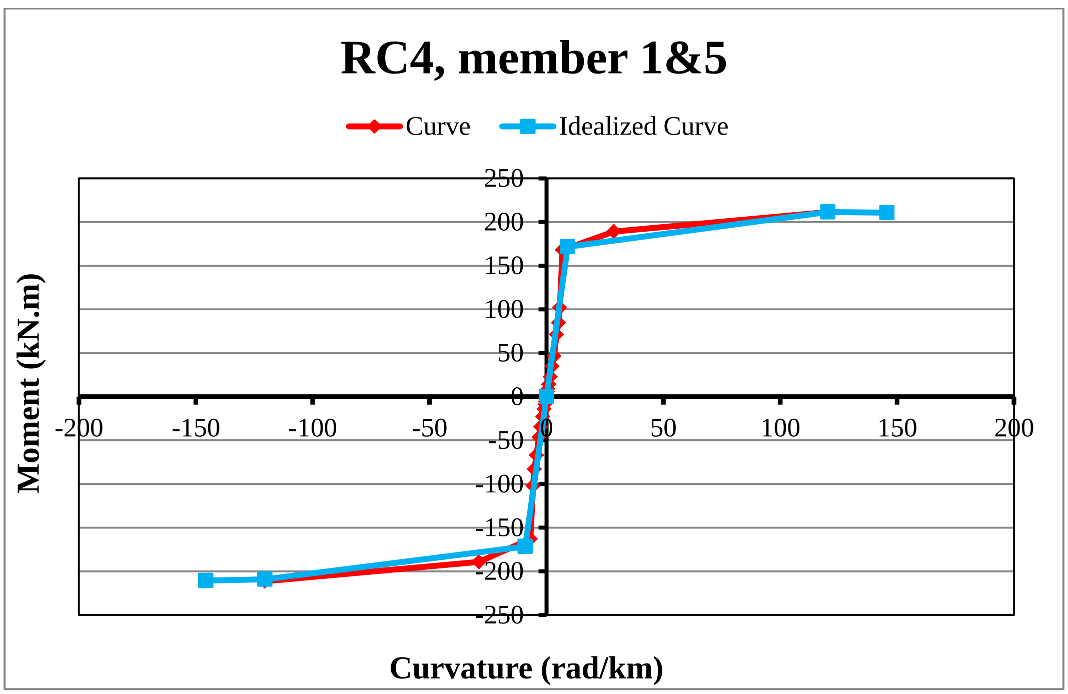

Figure A.4: RC4 Moment-curvature relation of column 1&5 (N= -200.4 kN)

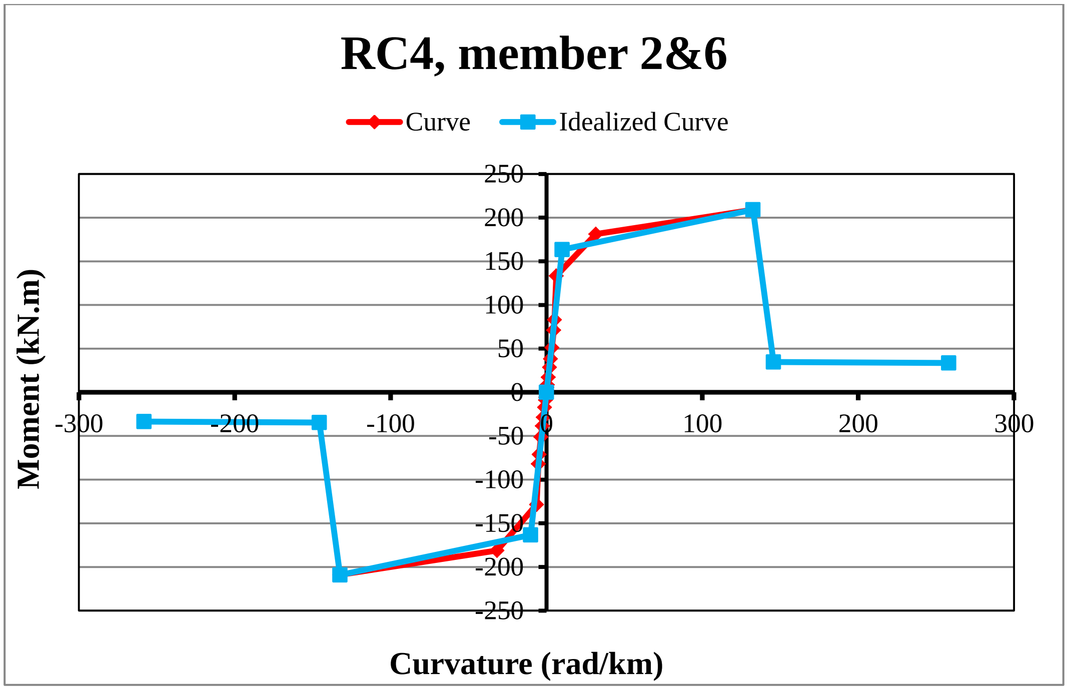

Figure A.5: RC4 Moment-curvature relation of column 2&6 (N= -150.32 kN)

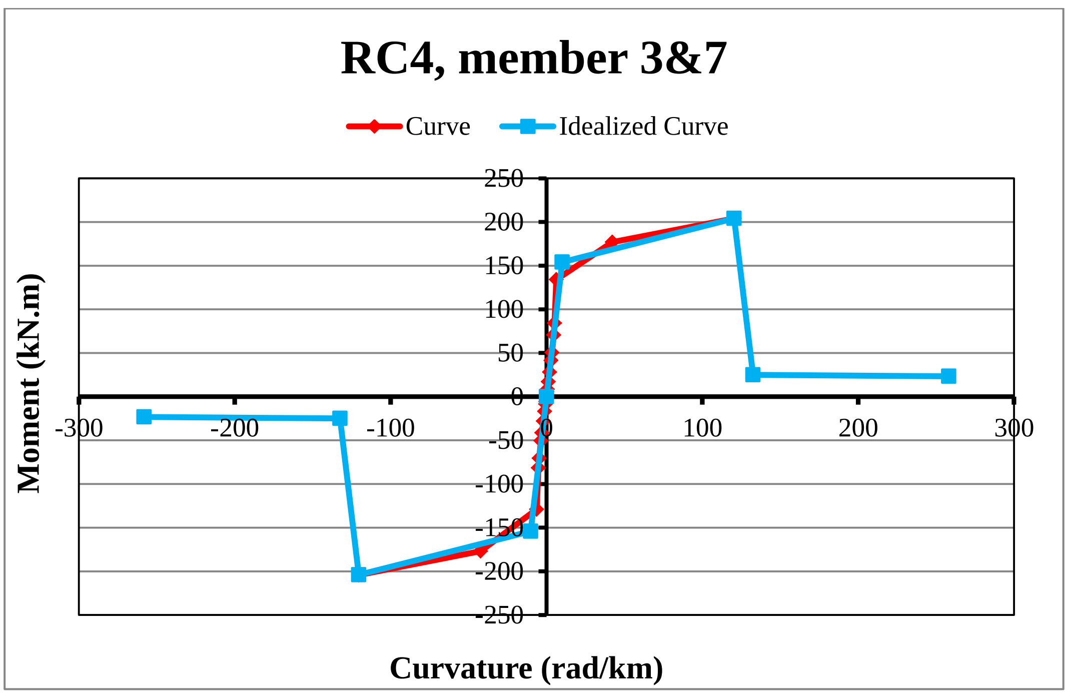

Figure A.6: RC4 Moment-curvature relation of column 3&7 (N= -100.2 kN)

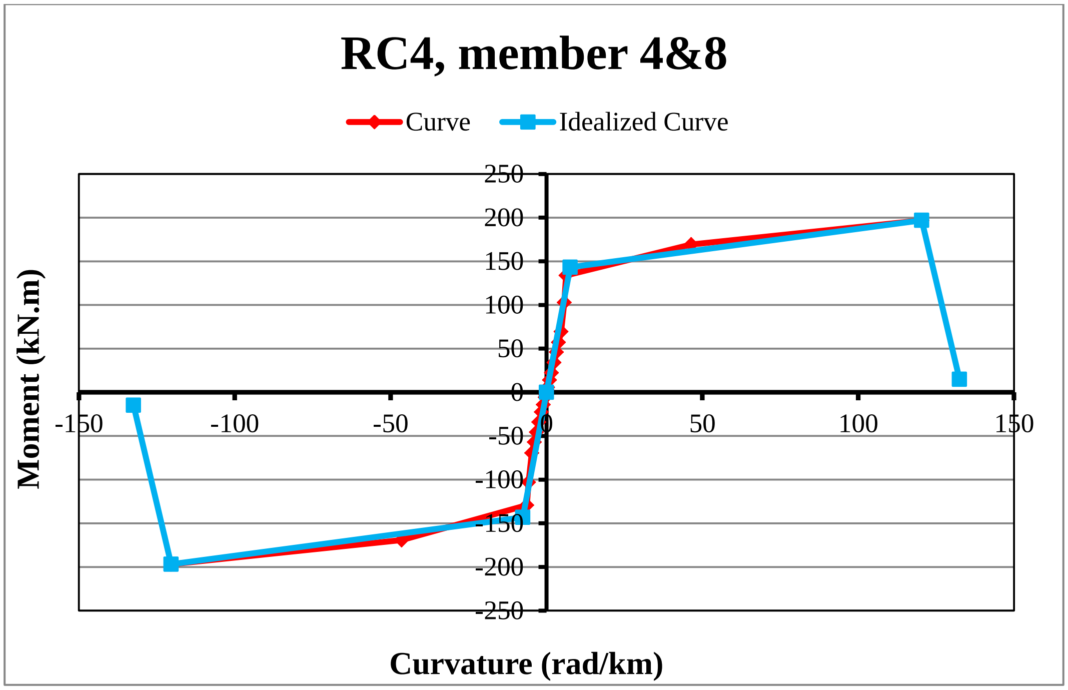

Figure A.7: RC4 Moment-curvature relation of column 4&8 (N= -50.1 kN)

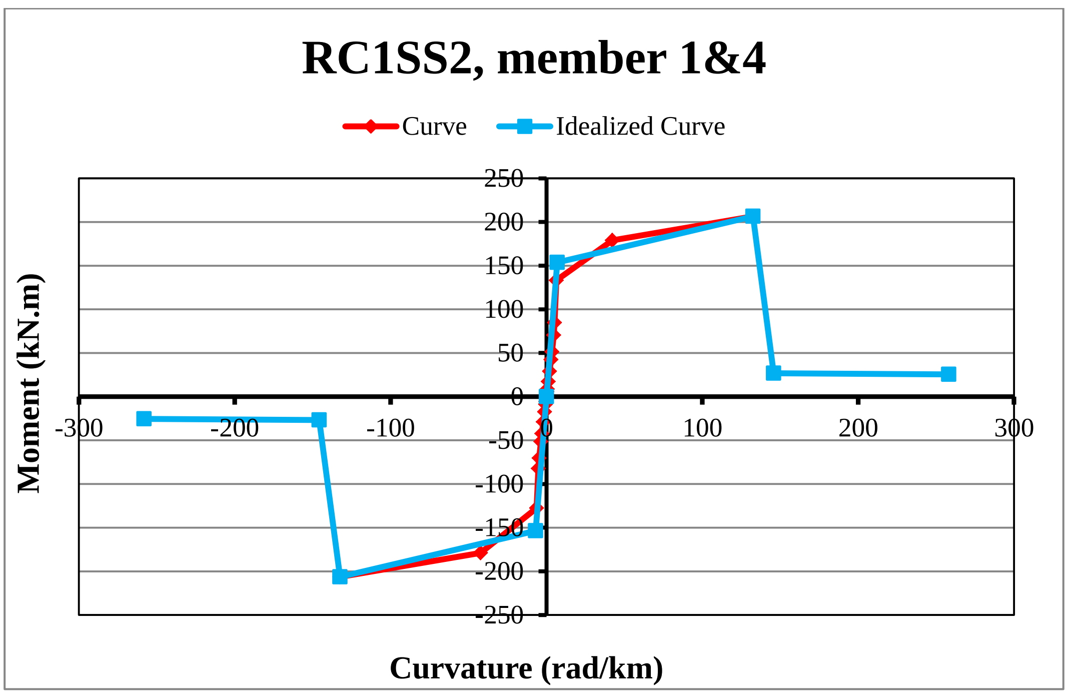

Figure A.8: RC1SS2 Moment-curvature relation of column 1&4 (N= -110. 8 kN)

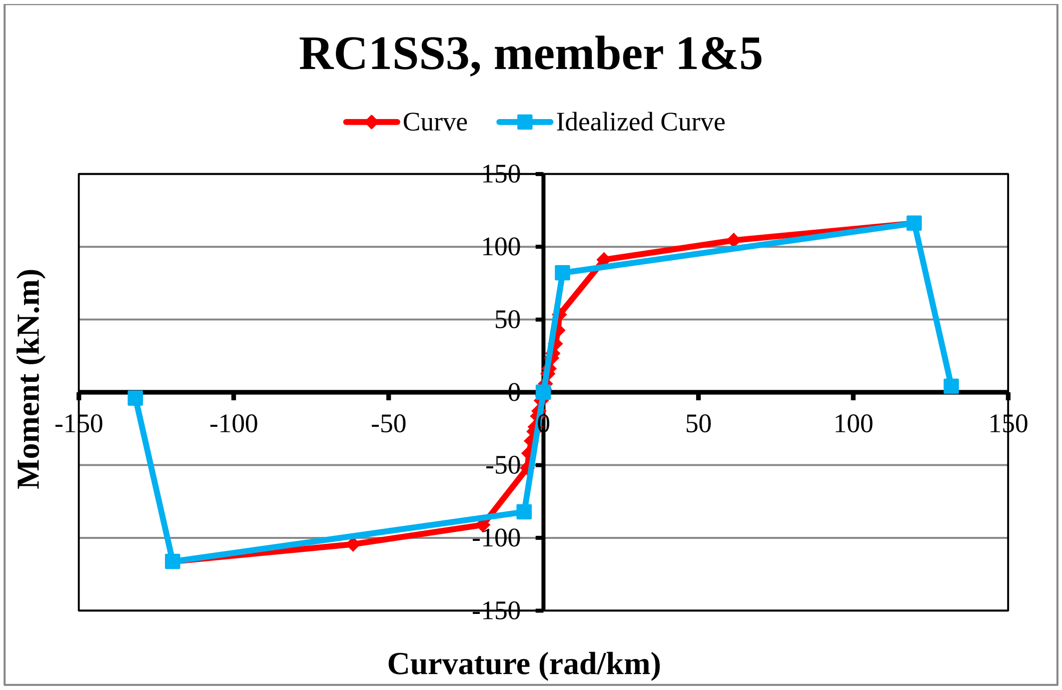

Figure A.9: RC1SS3 Moment-curvature relation of column 1&5 (N= -145.02 kN)

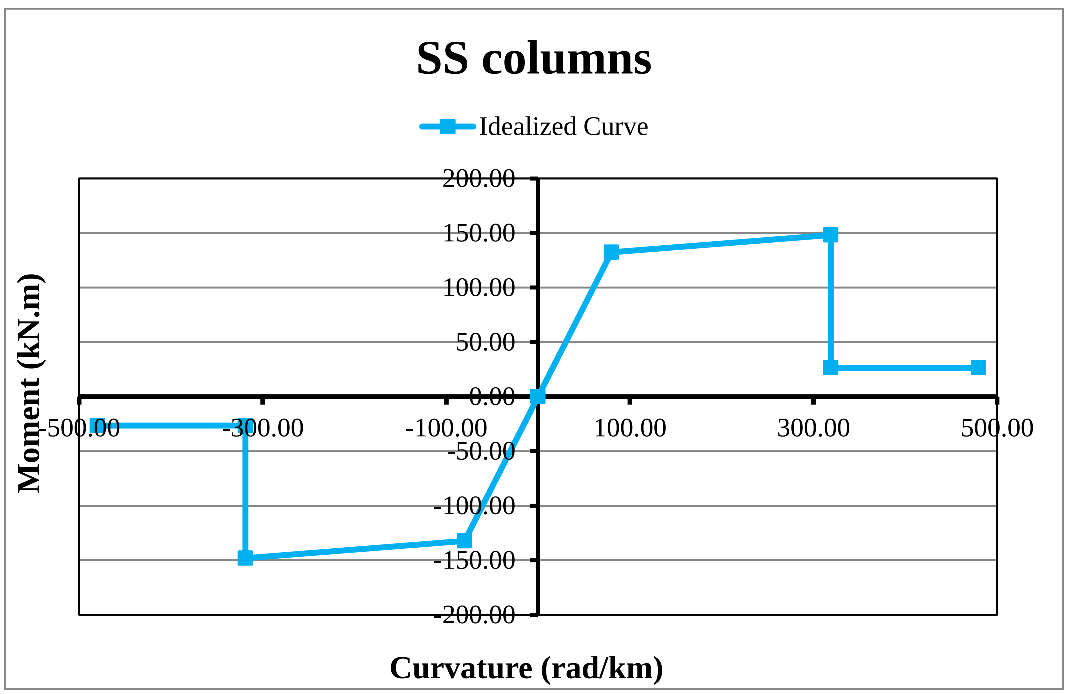

Figure A.10: SS Moment-curvature relation of columns

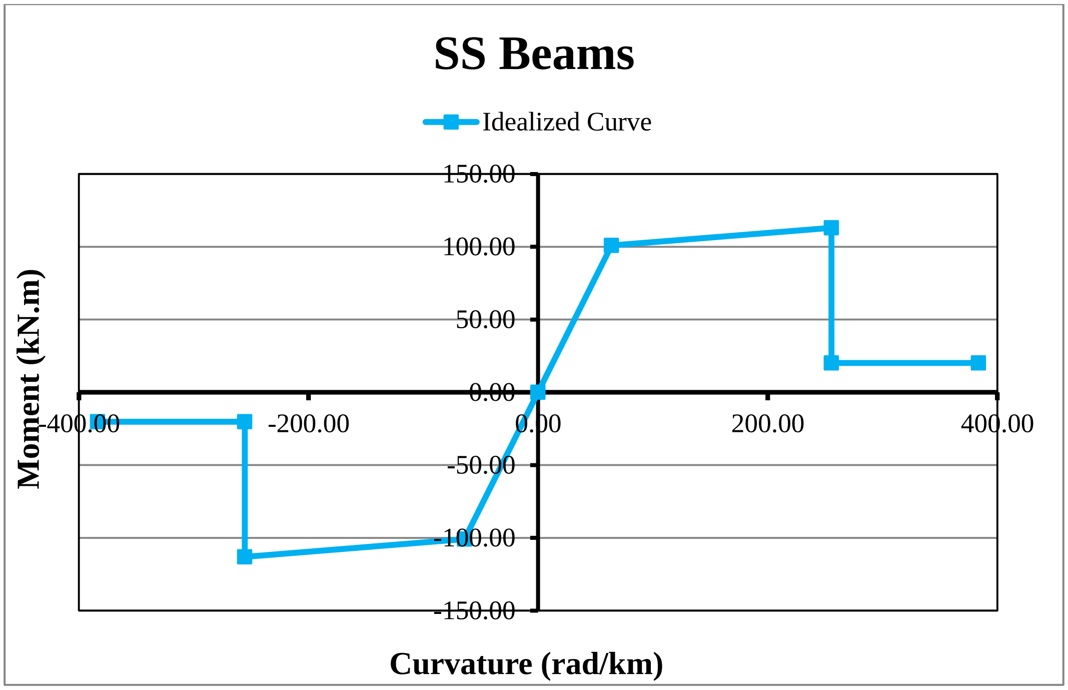

Figure A.11: SS Moment-curvature relation of beams

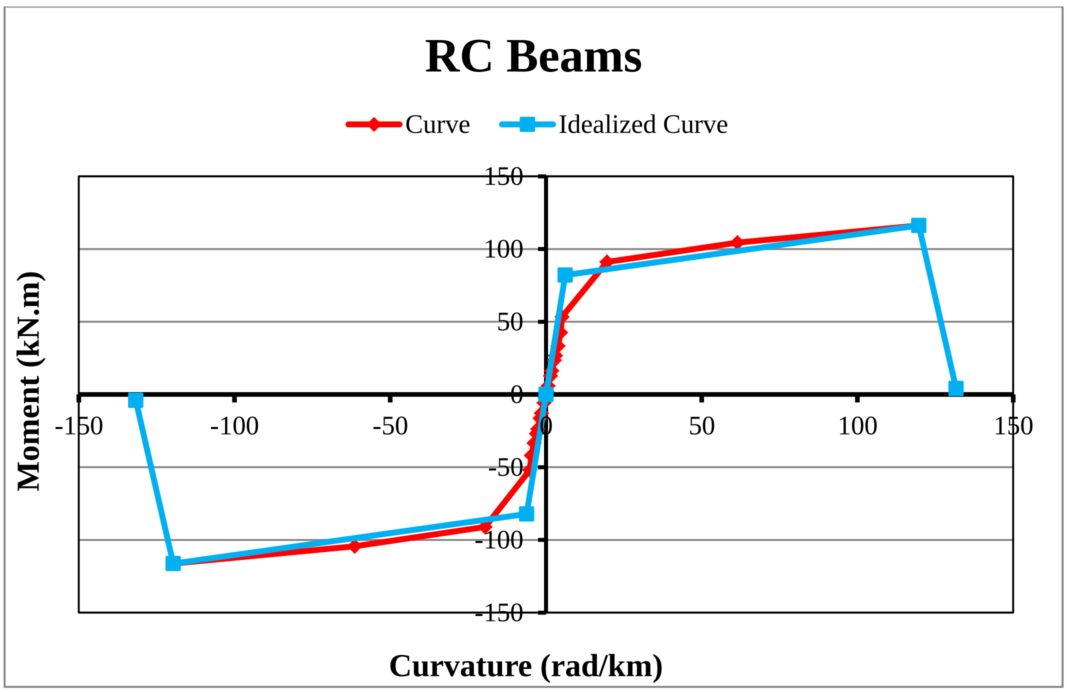

Figure A.12: RC Moment-curvature relation of beams

Appendix B: Displacement-Time Relation

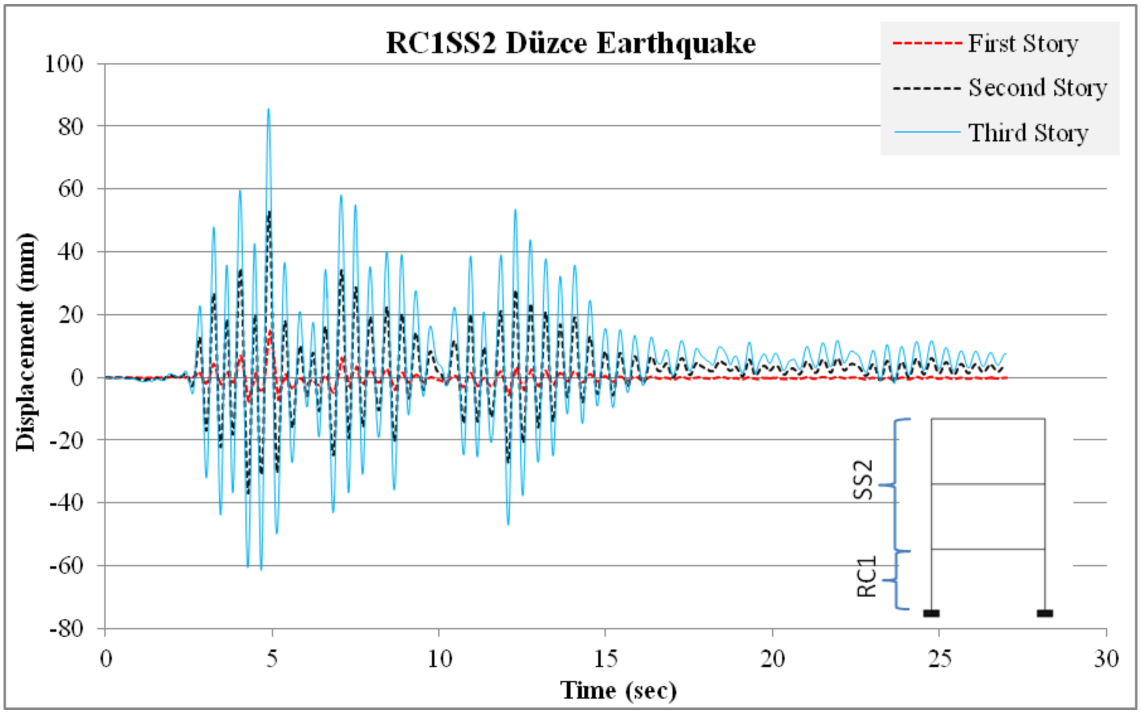

Figure B.1: RC1SS2 Düzce earthquake story displacements versus time.

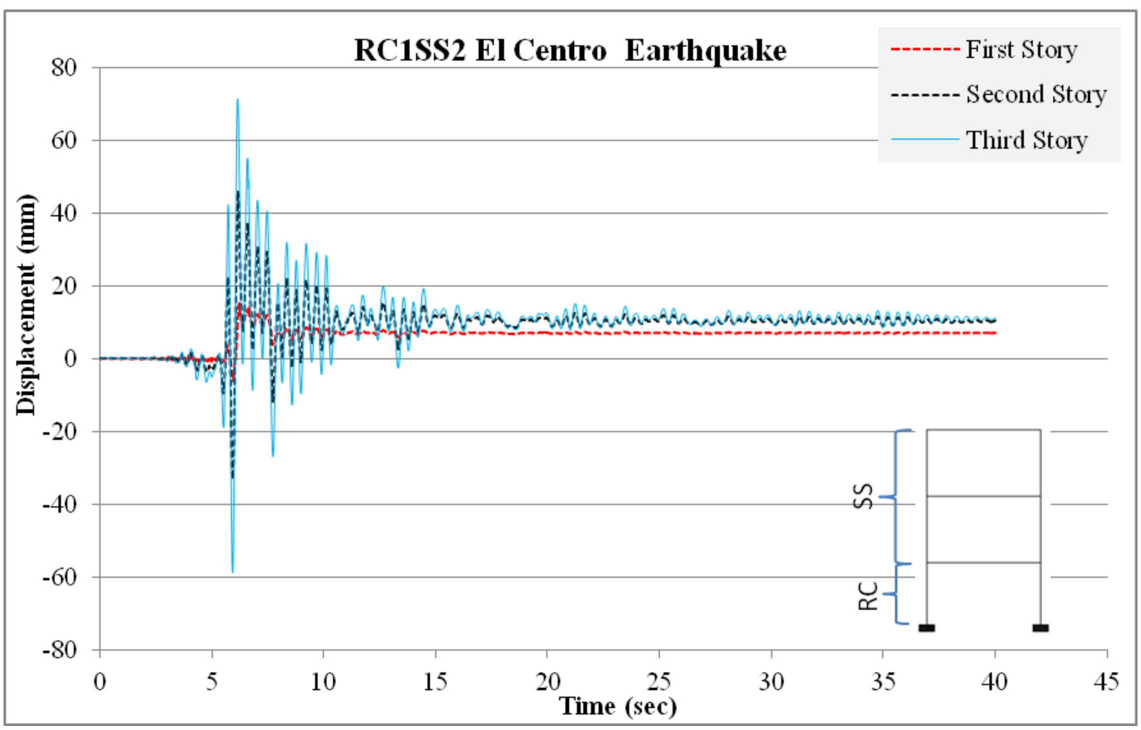

Figure B.2: RC1SS2 El Centro earthquake story displacements versus time.

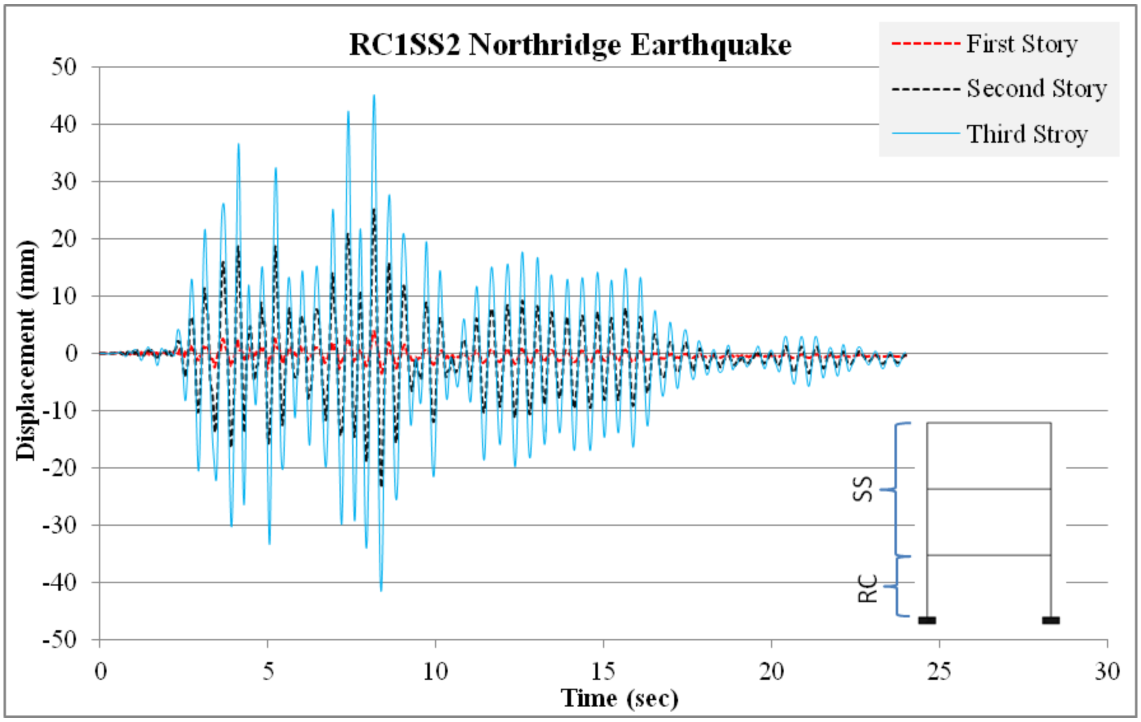

Figure B.3: RC1SS2 Northridge earthquake story displacements versus time.

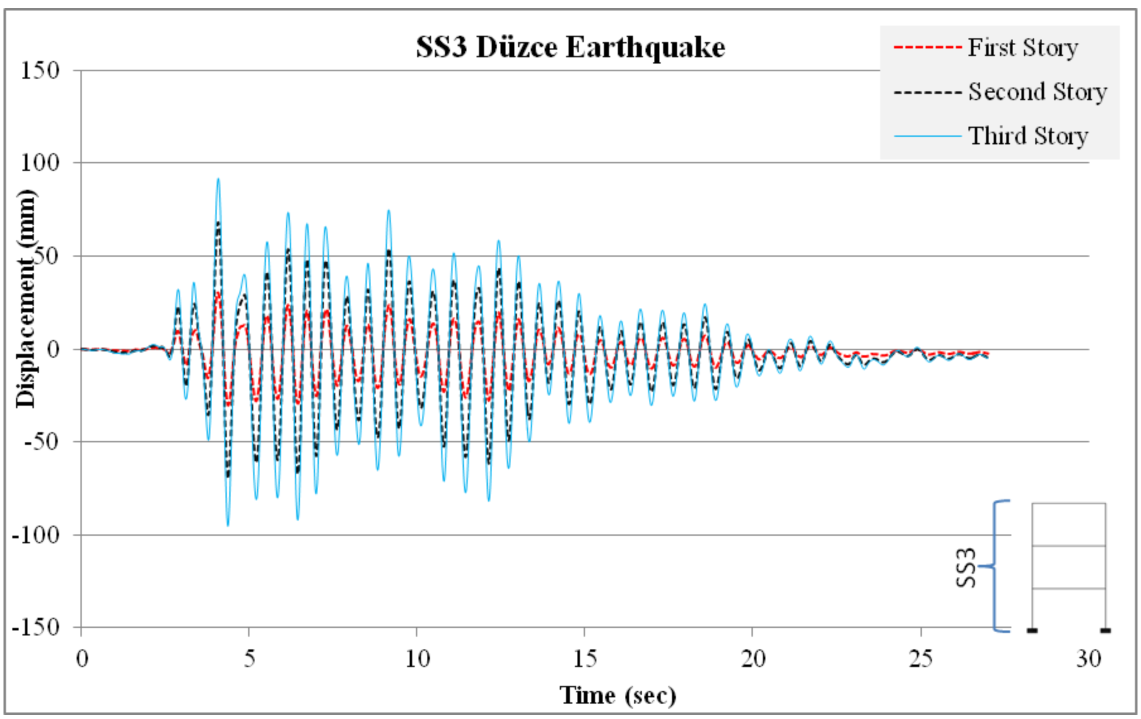

Figure B.4: SS3 Düzce earthquake story displacements versus time.

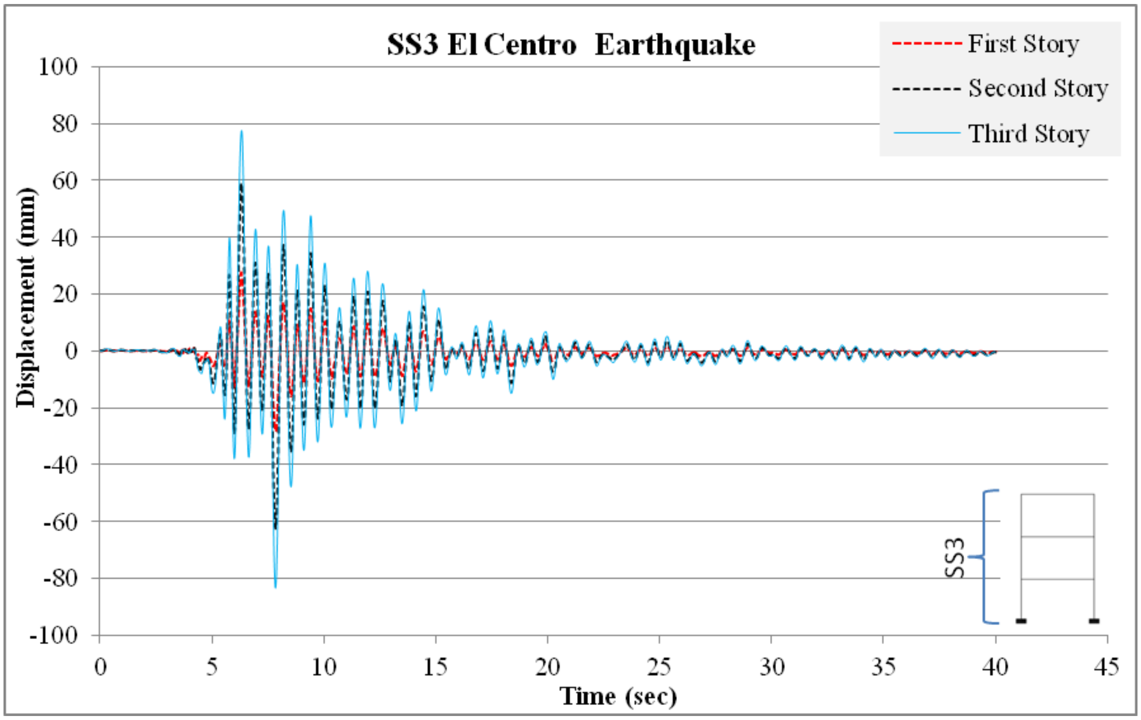

Figure B.5: SS3 El Centro earthquake story displacements versus time.

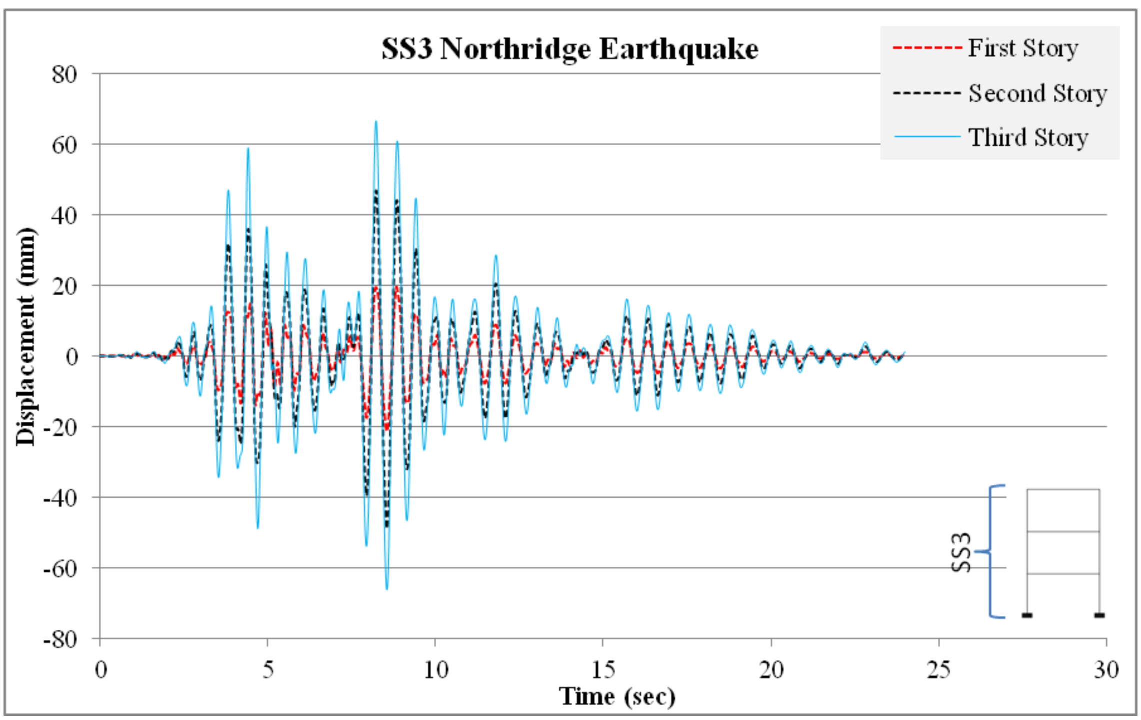

Figure B.6: SS3 Northridge earthquake story displacements versus time.

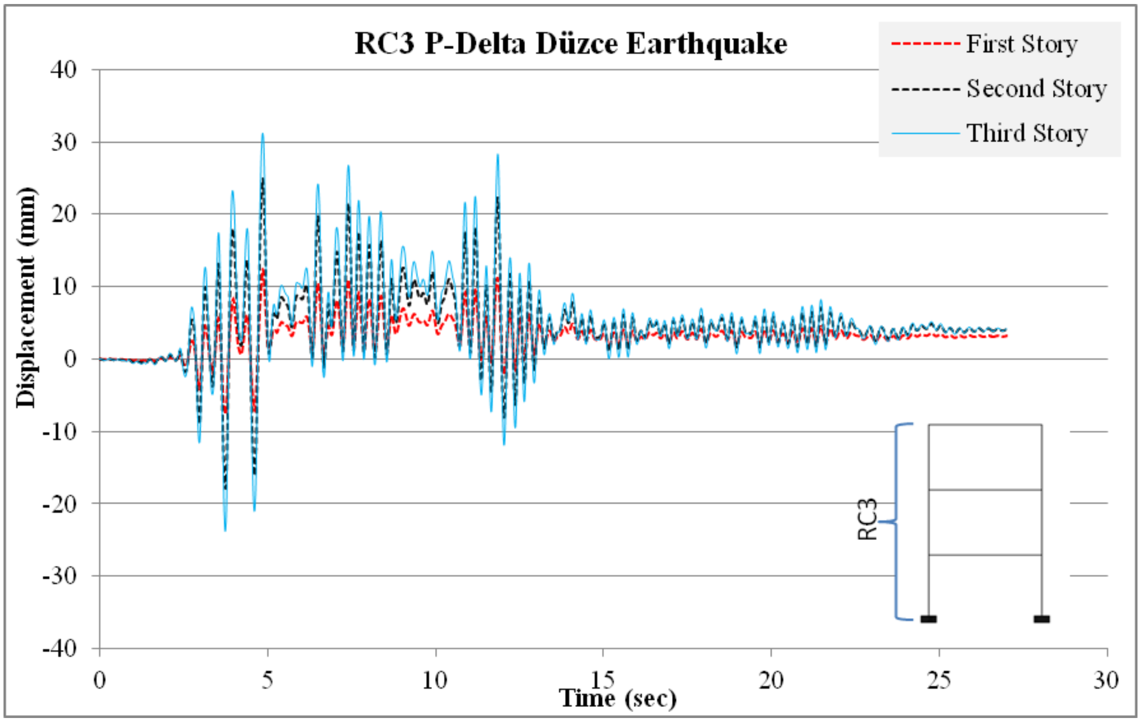

Figure B.7: RC3 P-Delta Düzce earthquake story displacements versus time.

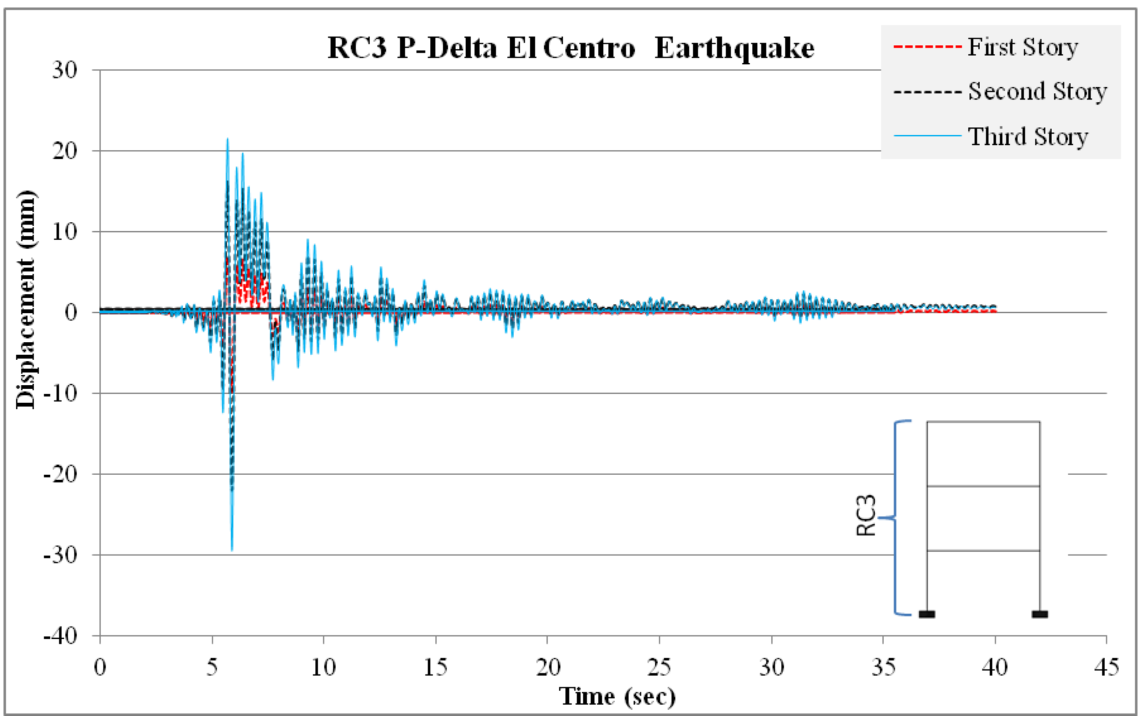

Figure B.8: RC3 P-Delta El Centro earthquake story displacements versus time.

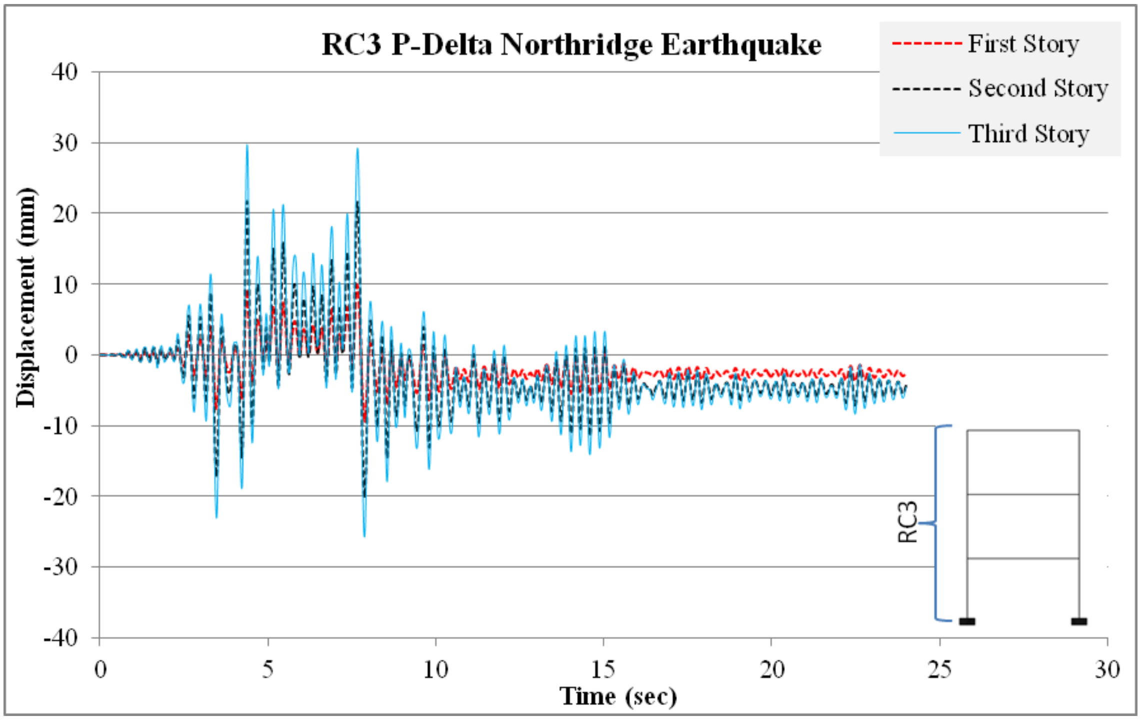

Figure B.9: RC3 P-Delta Northridge earthquake story displacements versus time.

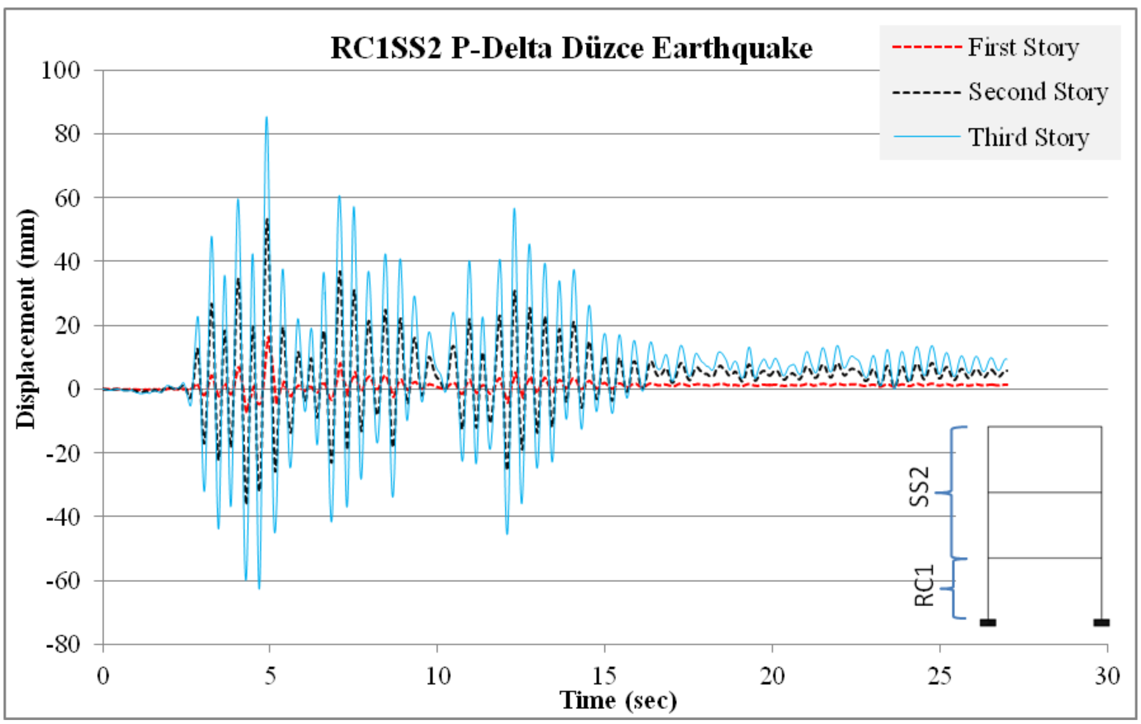

Figure B.10: RC1SS2 P-Delta Düzce earthquake story displacements versus time.

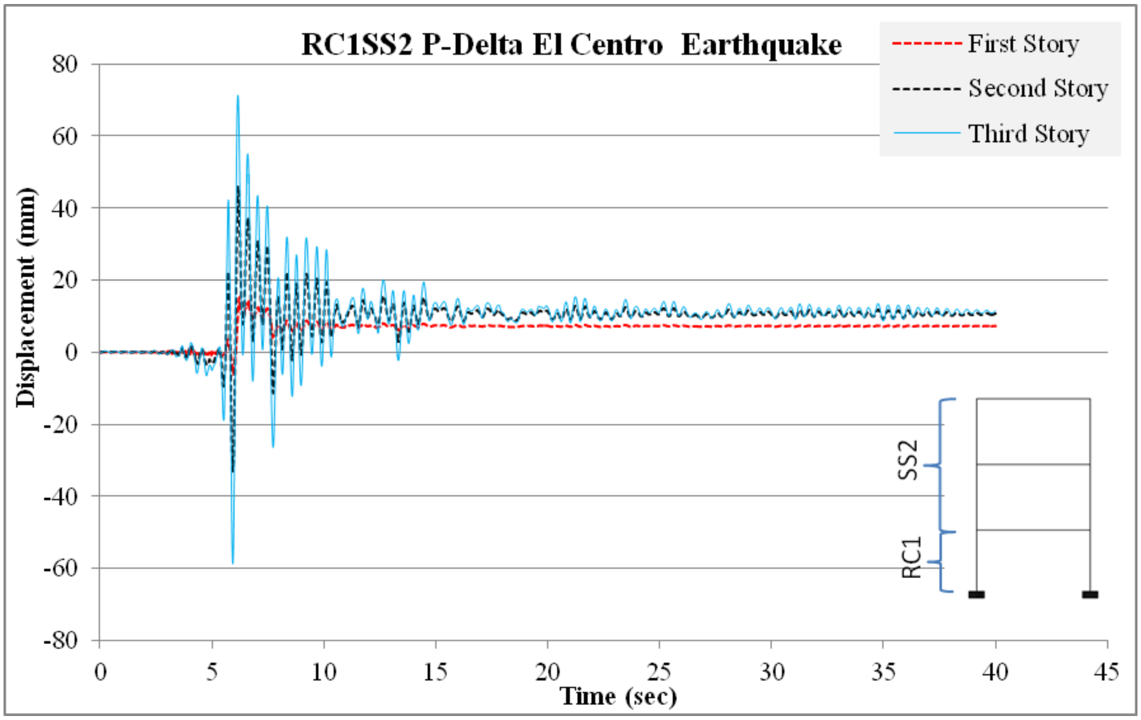

Figure B.11: RC1SS2 P-Delta El Centro earthquake story displacements versus time.

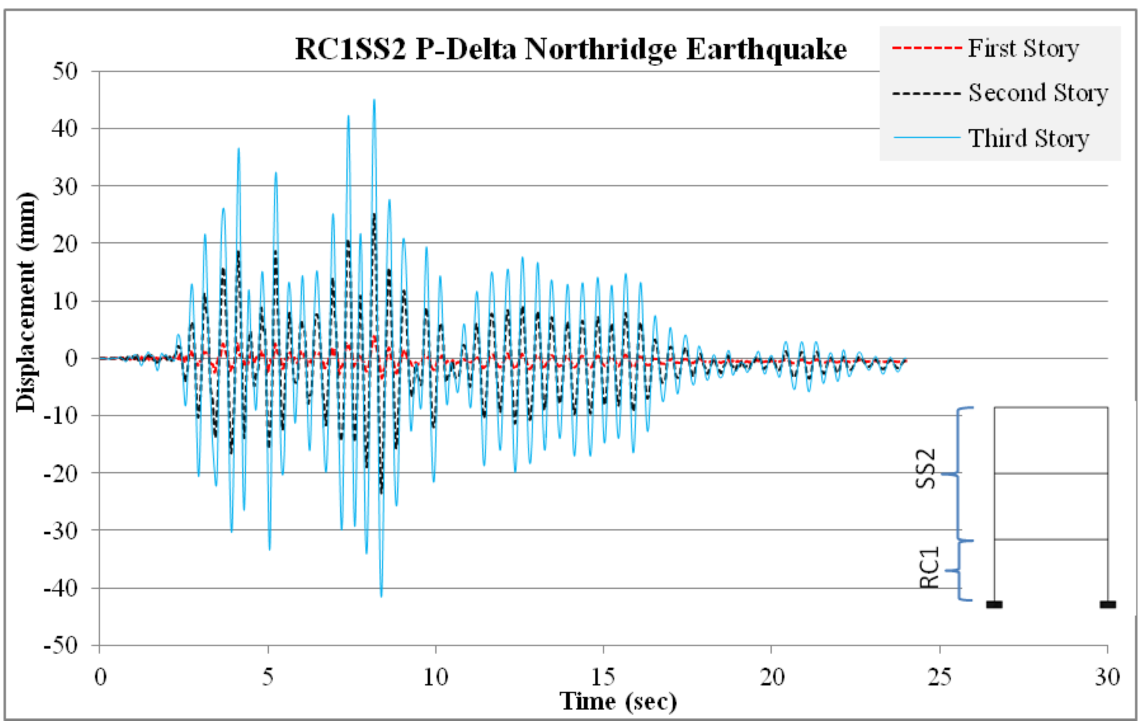

Figure B.12: RC1SS2 P-Delta Northridge earthquake story displacements versus time.

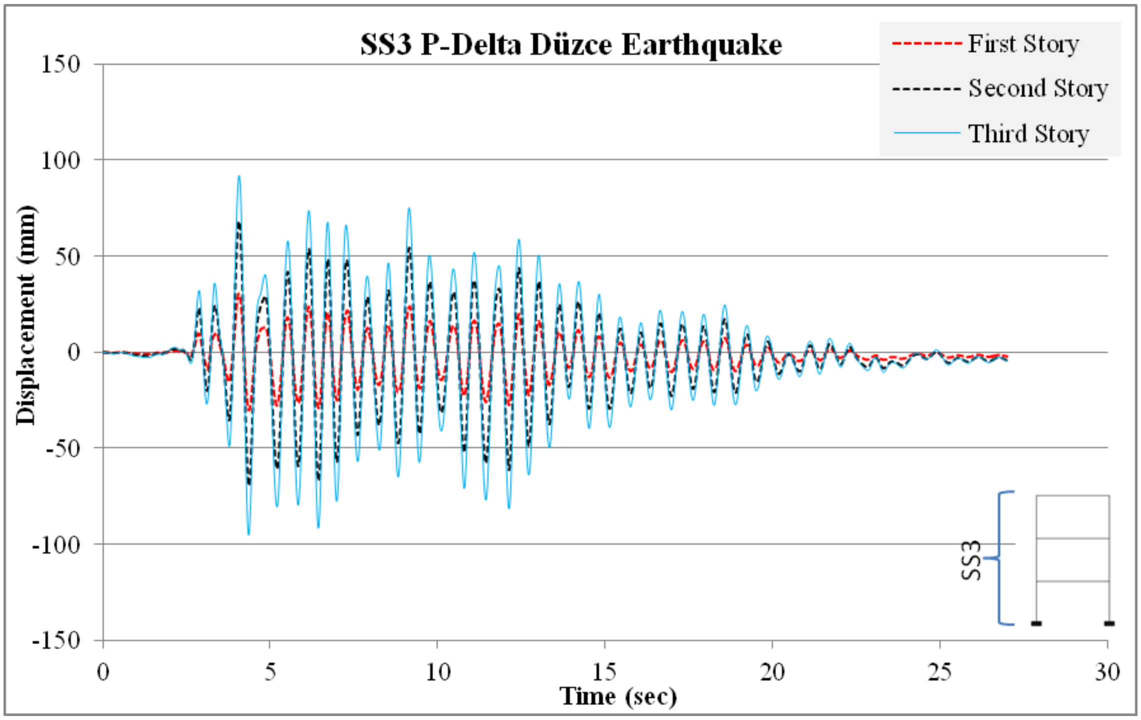

Figure B.13: SS3 P-Delta Düzce earthquake story displacements versus time.

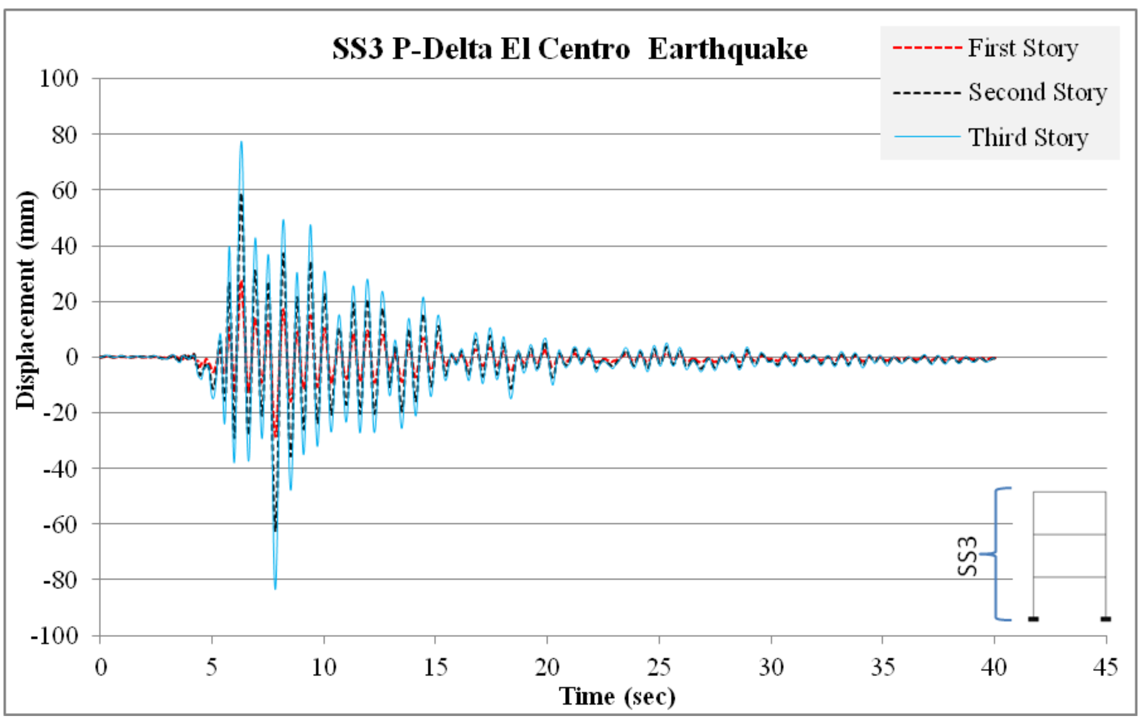

Figure B.14: SS3 P-Delta El Centro earthquake story displacements versus time.

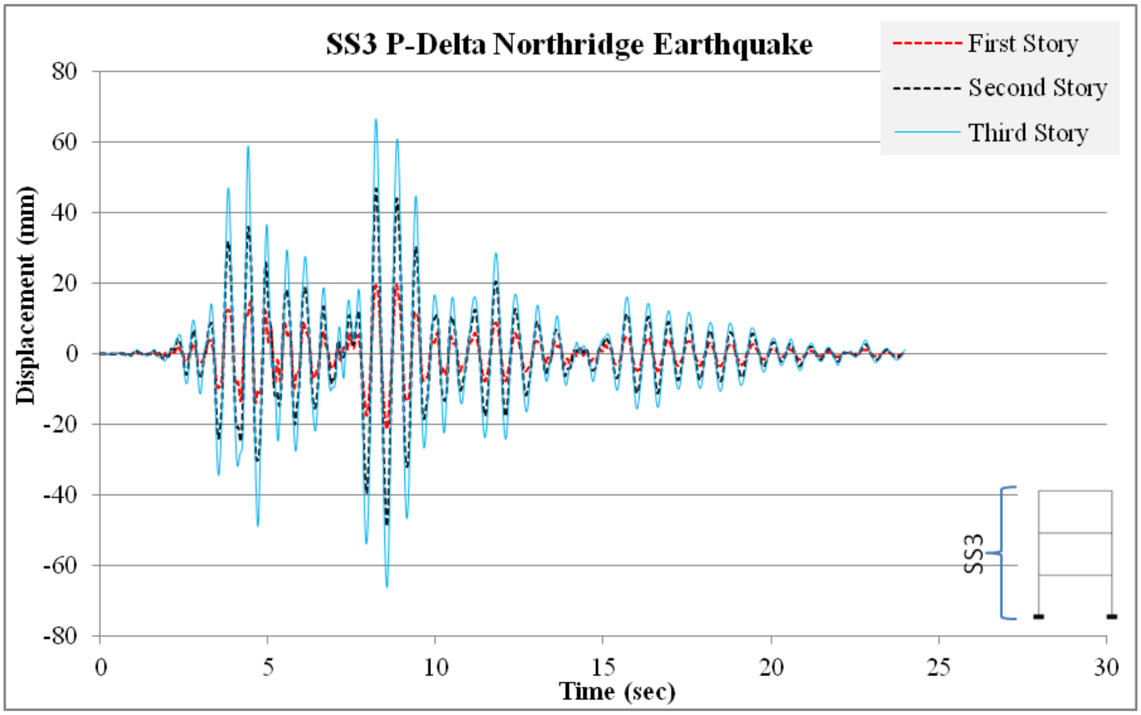

Figure B.15: SS3 P-Delta Northridge earthquake story displacements versus time.

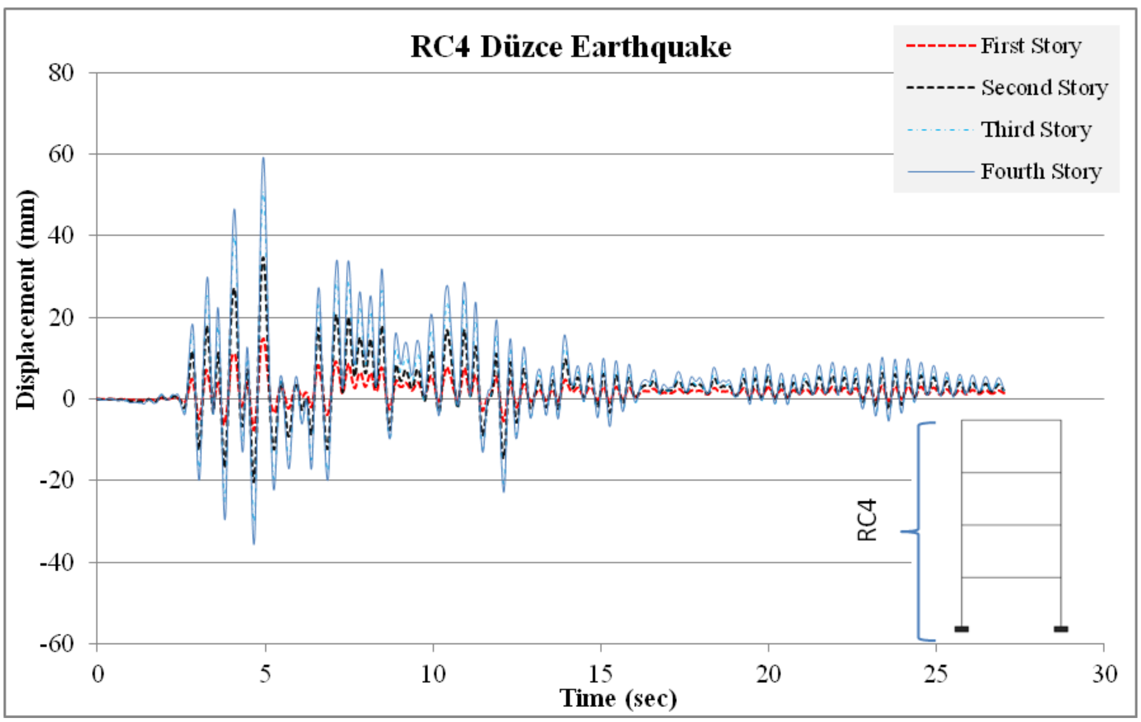

Figure B.16: RC4 Düzce earthquake story displacements versus time.

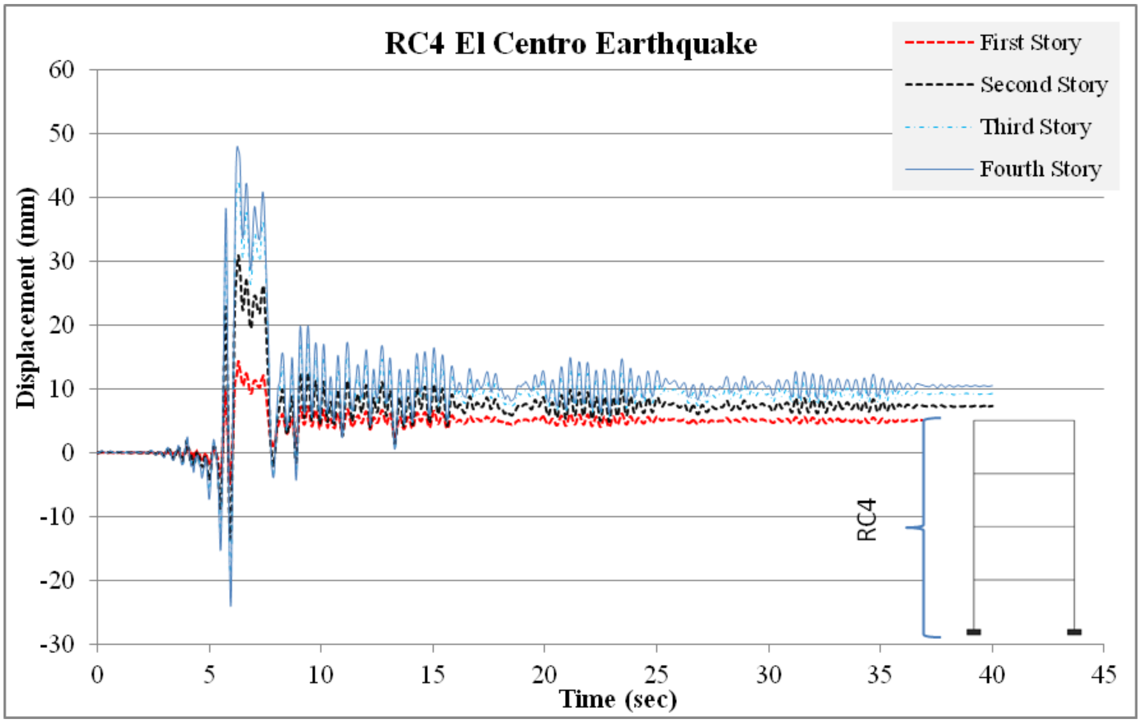

Figure B.17: RC4 El Centro earthquake story displacements versus time.

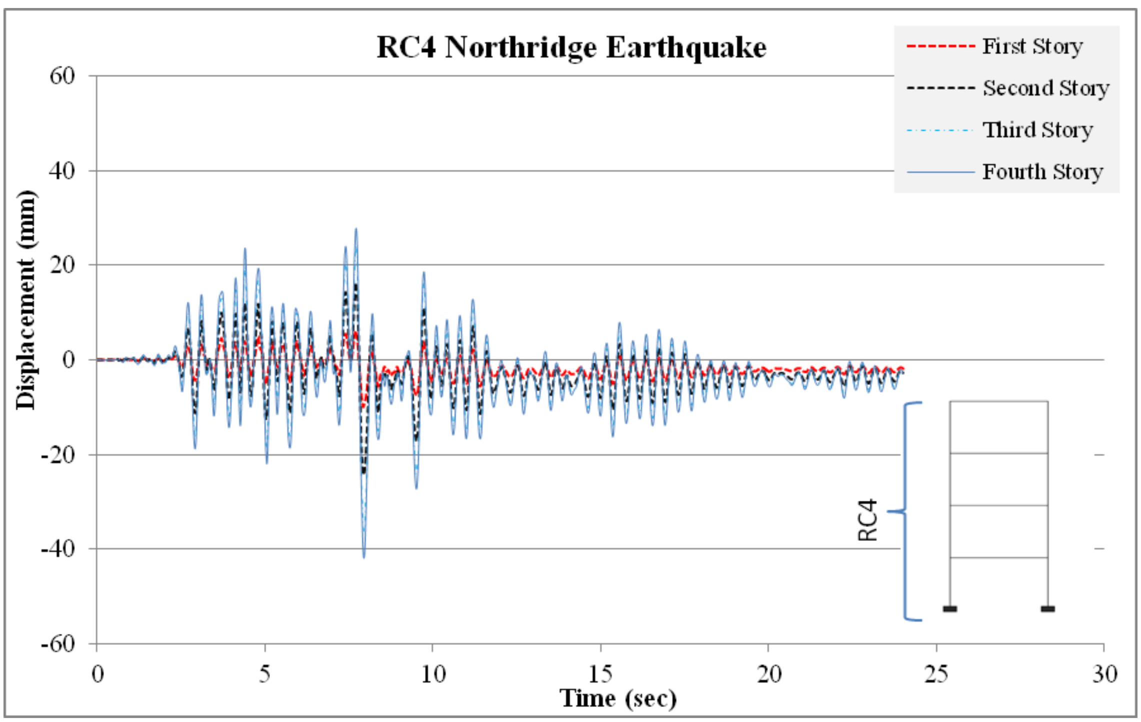

Figure B.18: RC4 Northridge earthquake story displacements versus time.

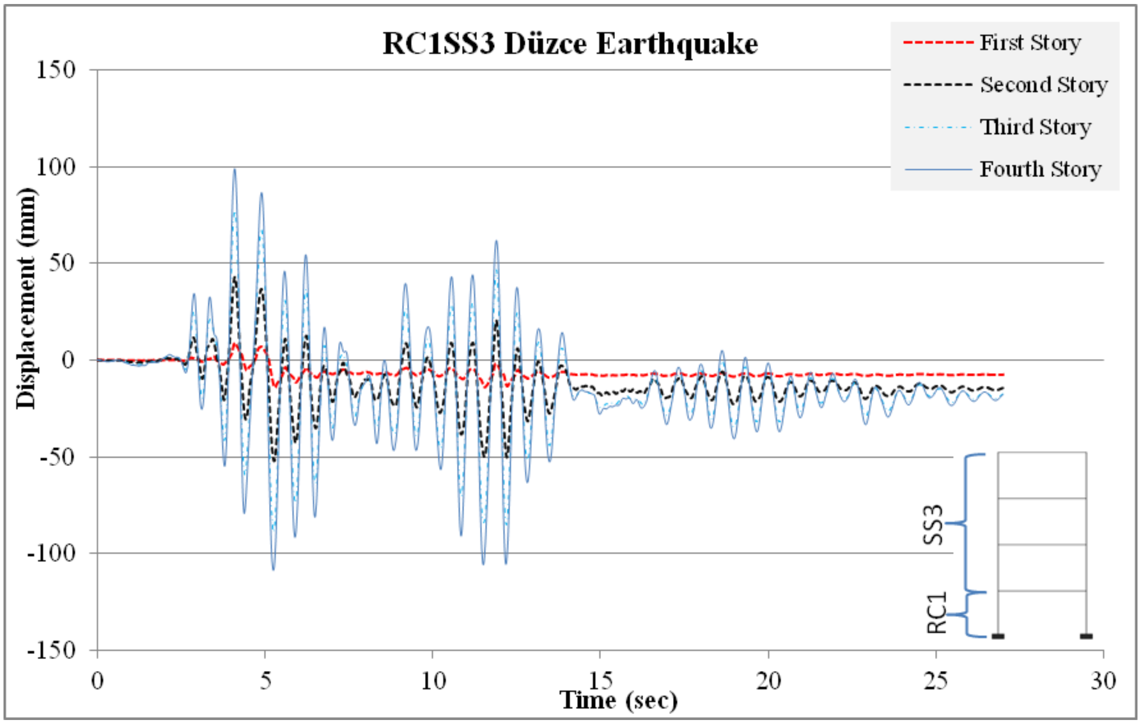

Figure B.19: RC1SS3 Düzce earthquake story displacements versus time.

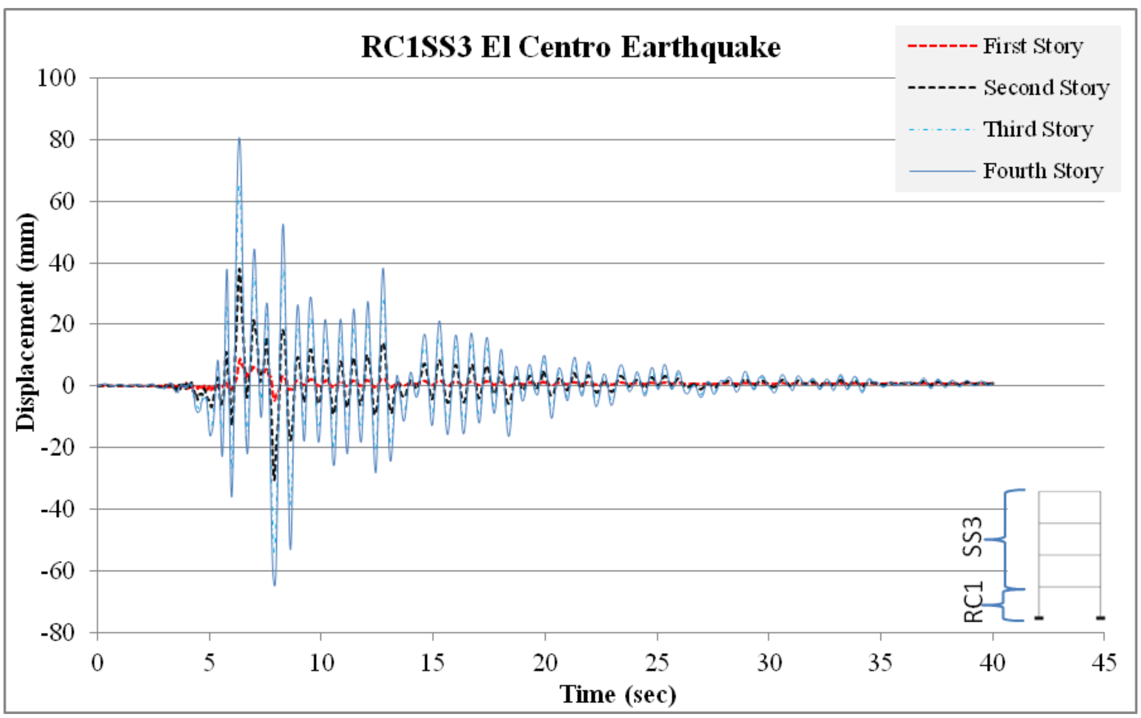

Figure B.20: RC1SS3 El Centro earthquake story displacements versus time.

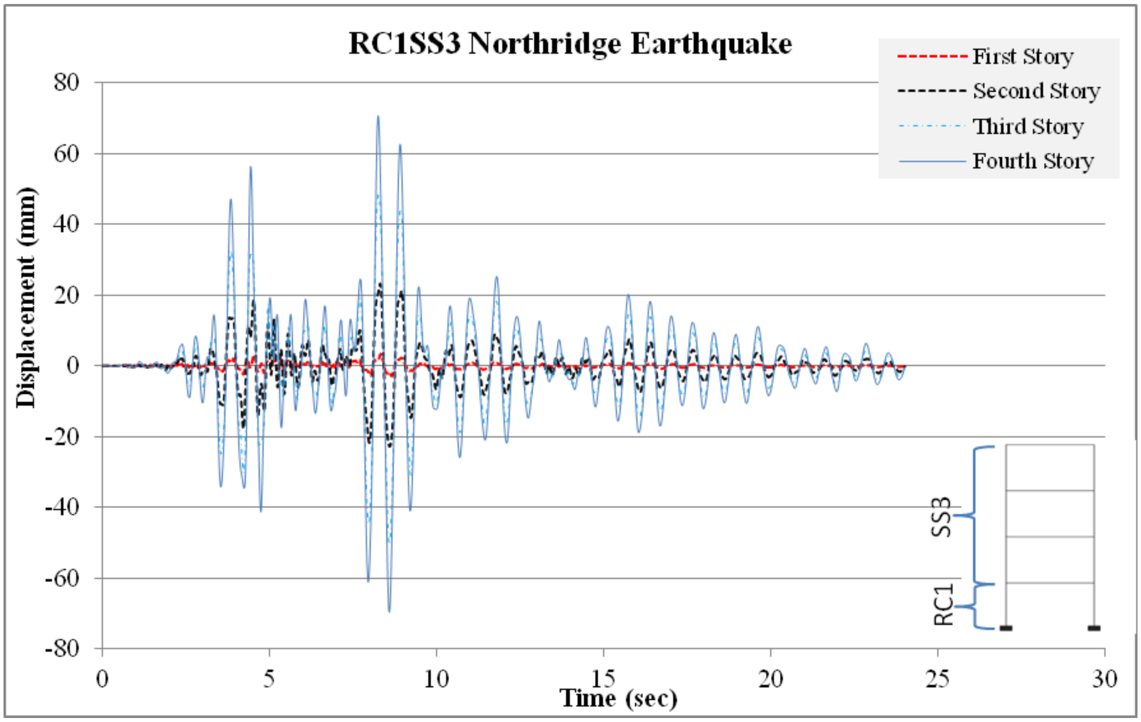

Figure B.21: RC1SS3 Northridge earthquake story displacements versus time.

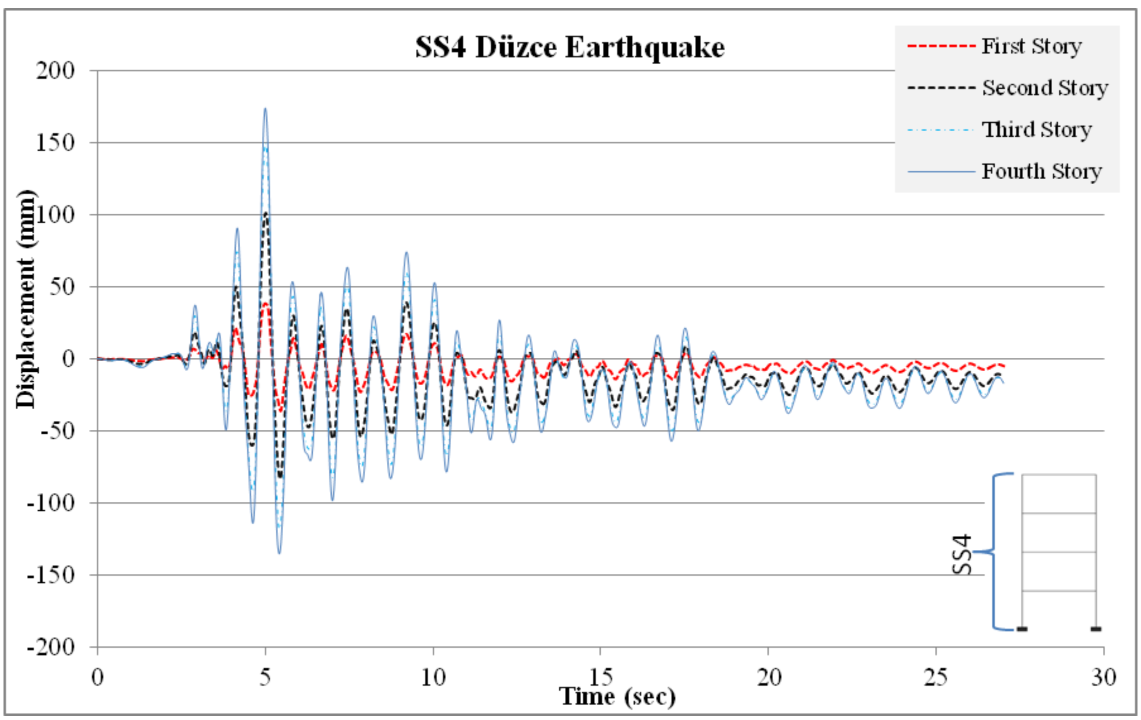

Figure B.22: SS4 Düzce earthquake story displacements versus time.

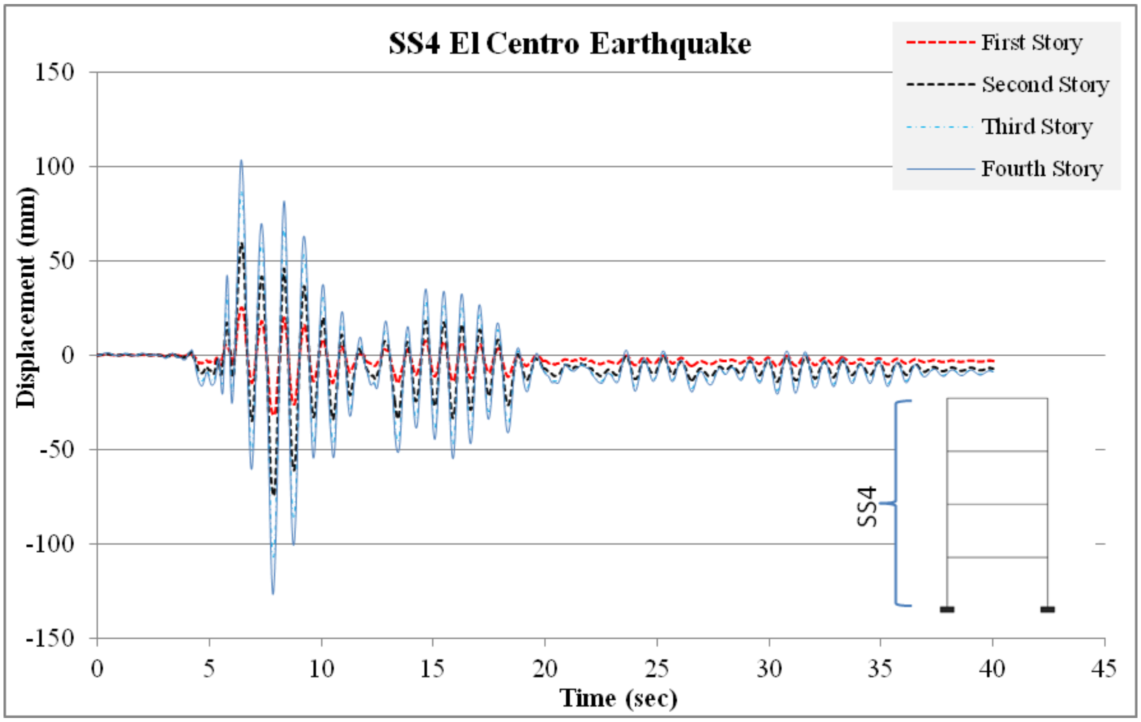

Figure B.23: SS4 El Centro earthquake story displacements versus time.

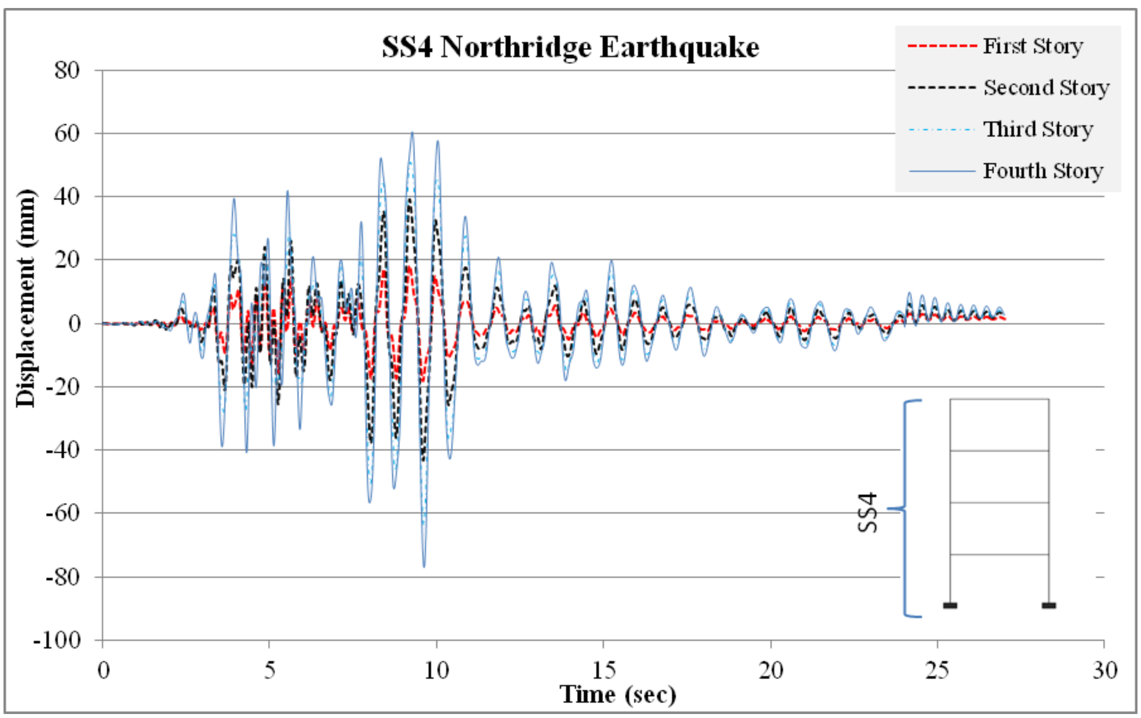

Figure B.24: SS4 Northridge earthquake story displacements versus time.

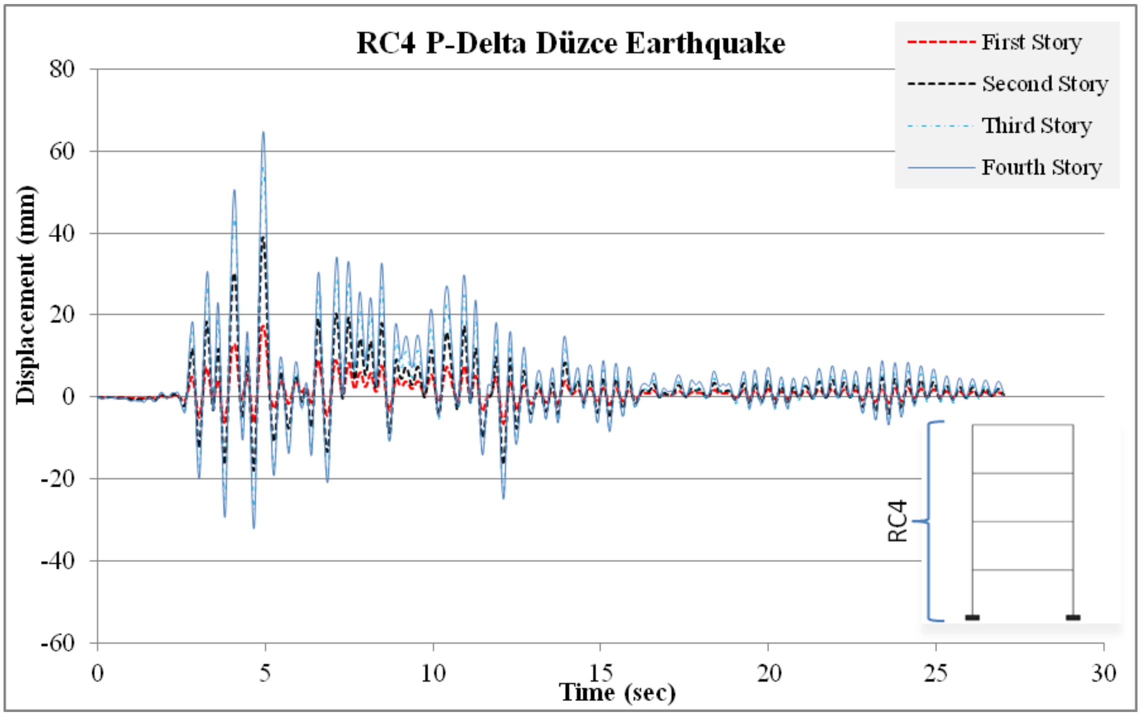

Figure B.25: RC4 P-Delta Düzce earthquake story displacements versus time.

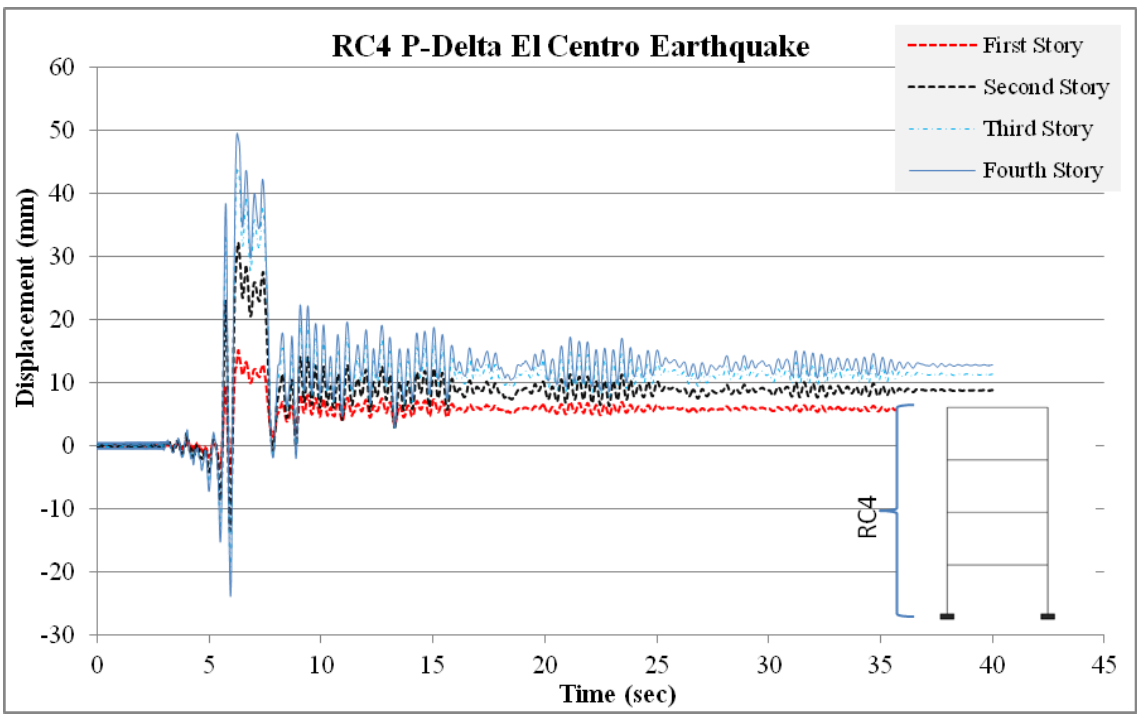

Figure B.26: RC4 P-Delta El Centro earthquake story displacements versus time.

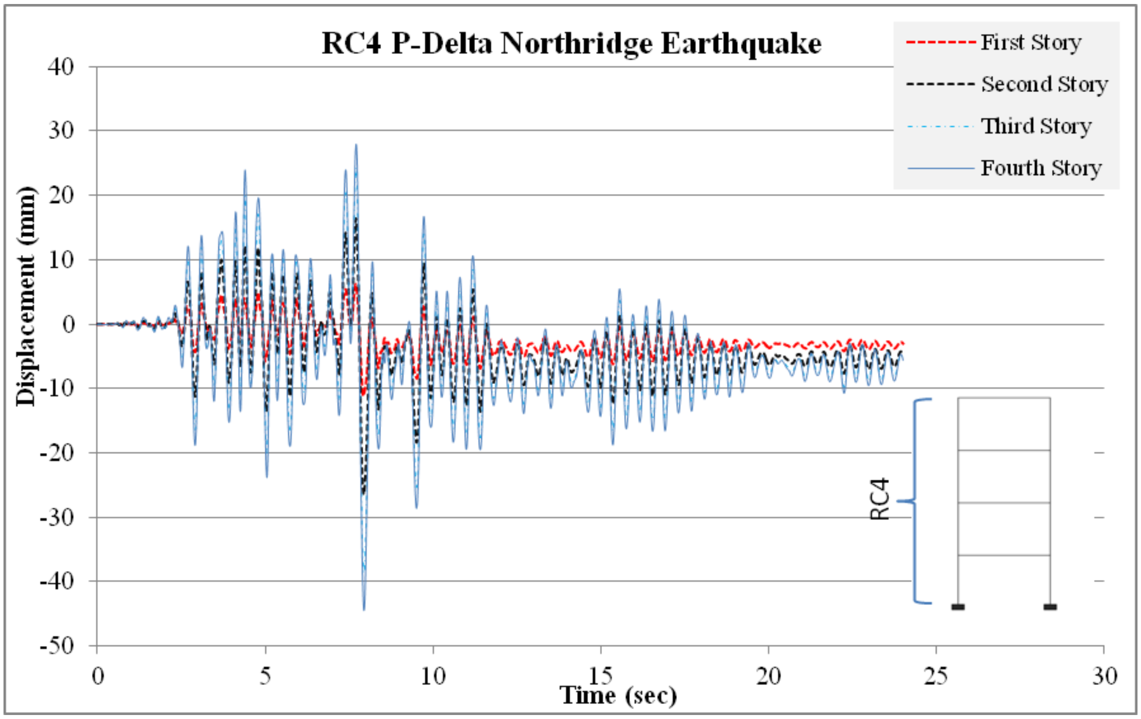

Figure B.27: RC4 P-Delta Northridge earthquake story displacements versus time.

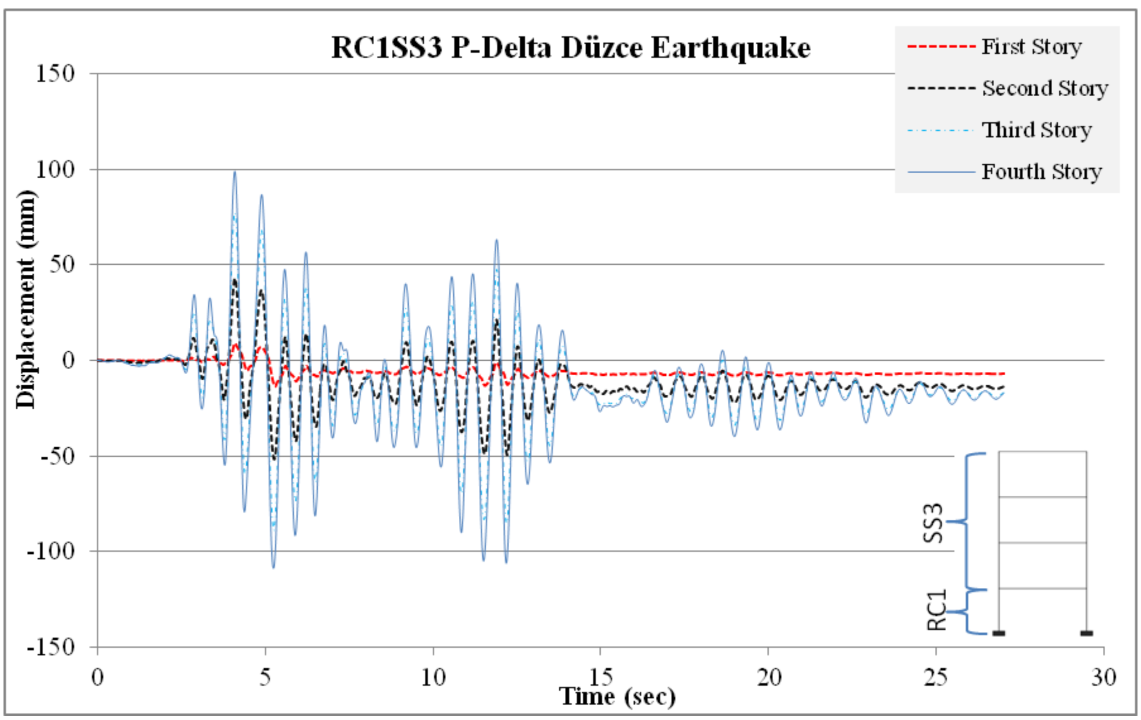

Figure B.28: RC1SS3 P-Delta Düzce earthquake story displacements versus time.

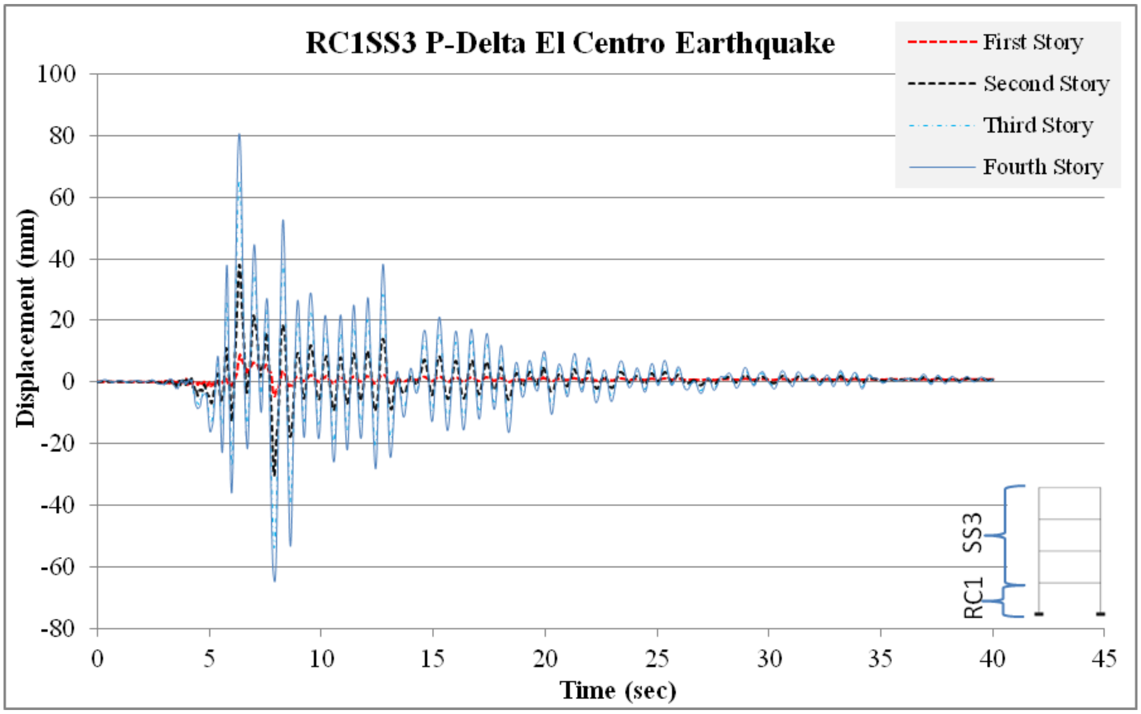

Figure B.29: RC1SS3 P-Delta El Centro earthquake story displacements versus time.

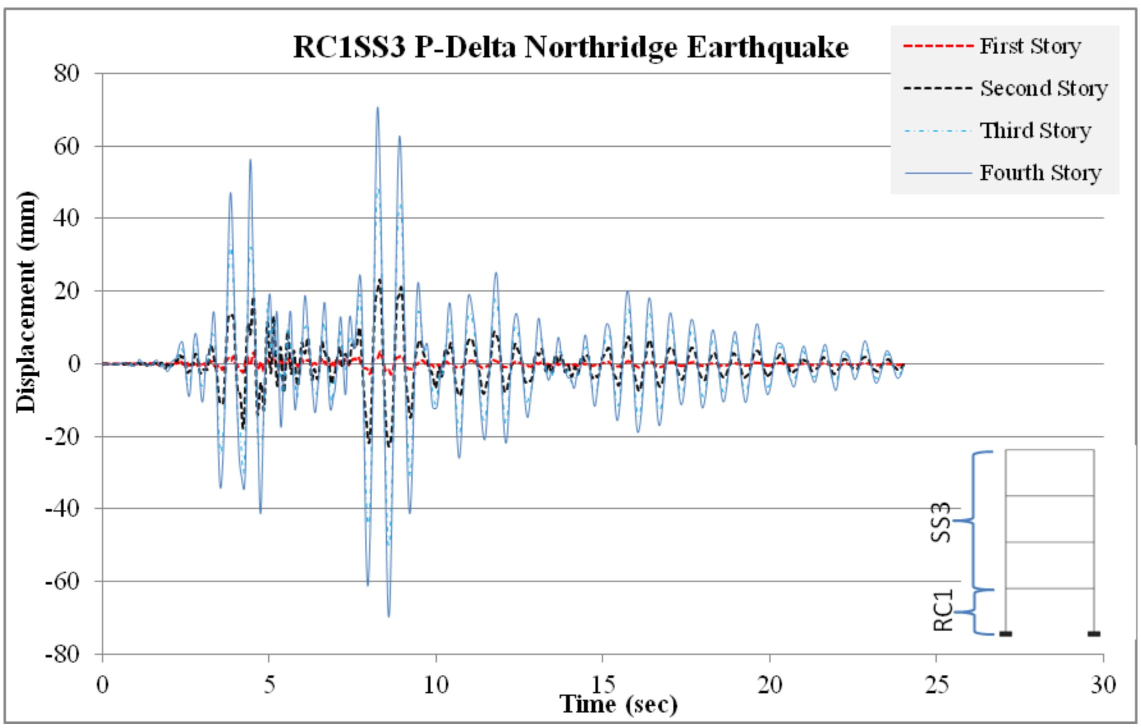

Figure B.30: RC1SS3 P-Delta Northridge earthquake story displacements versus time.

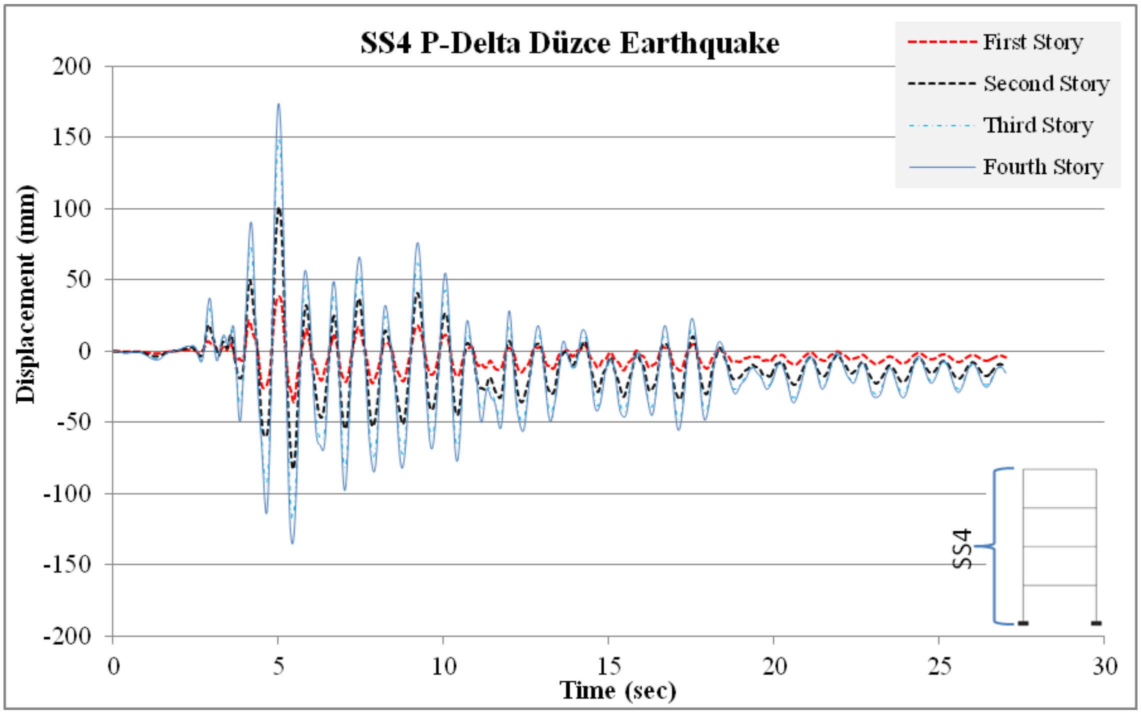

Figure B.31: SS4 P-Delta Düzce earthquake story displacements versus time.

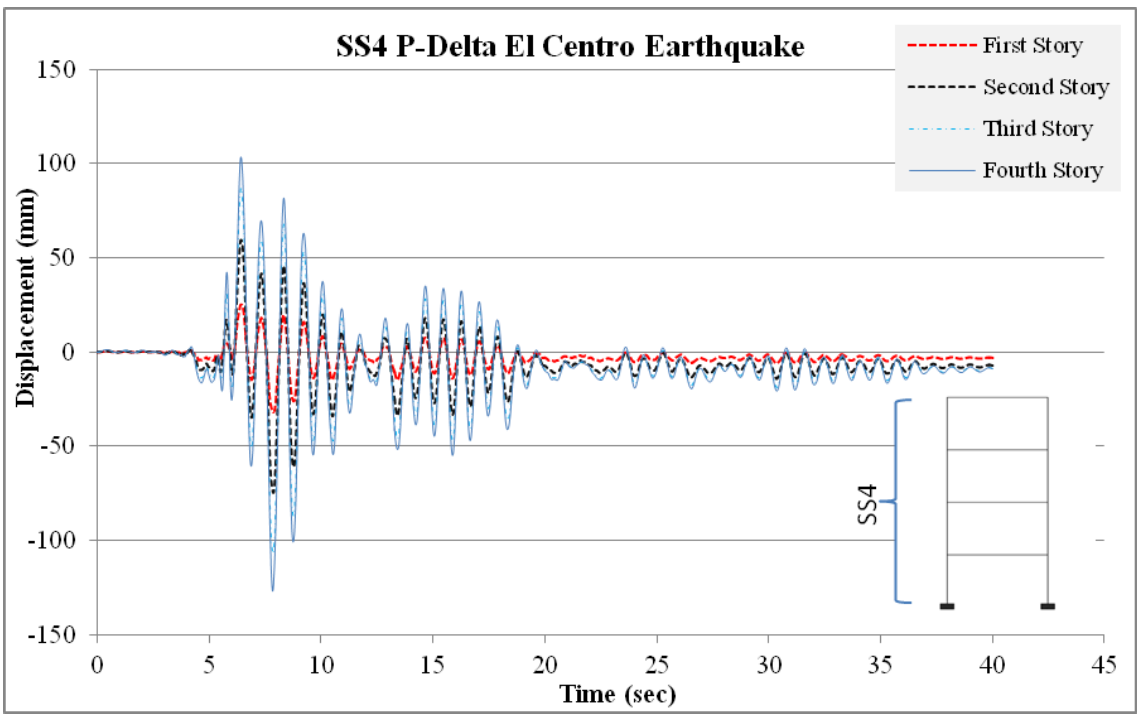

Figure B.32: SS4 P-Delta El Centro earthquake story displacements versus time.

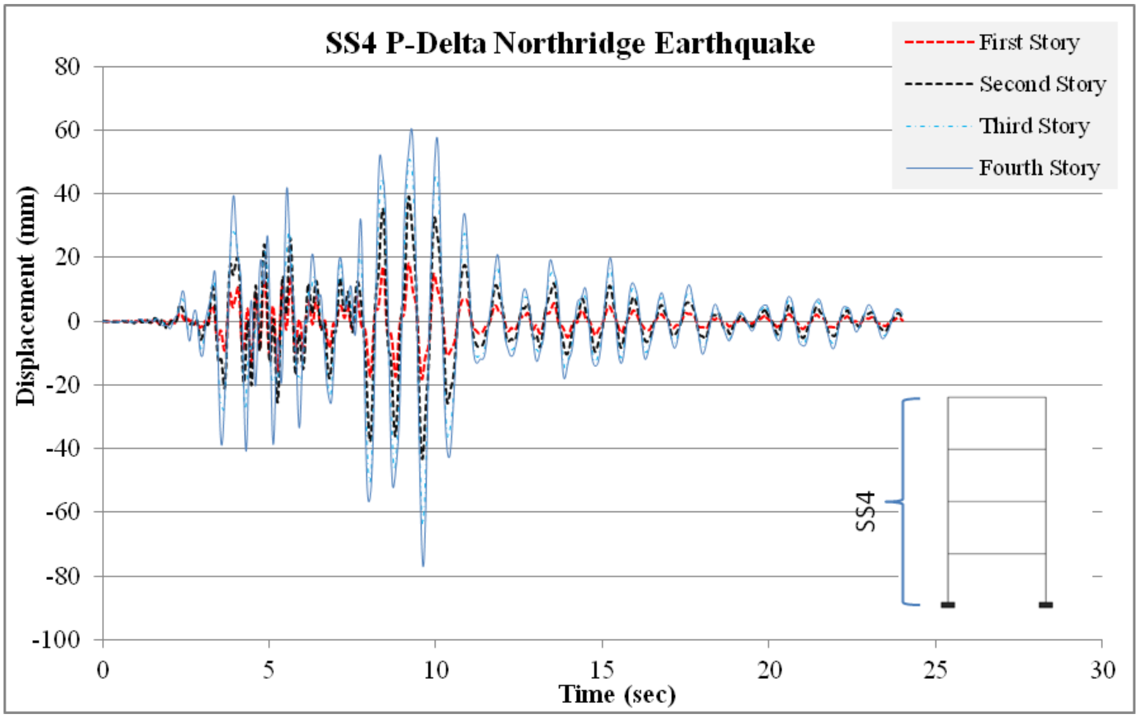

Figure B.33: SS4 P-Delta Northridge earthquake story displacements versus time.

Appendix C: FEMA 356 Parameters

Table C.1: Damage Control and Building Performance Levels (ASCE 2000).

	Target Building Performance Levels			
	Collapse Prevention Level (5-E)	**Life Safety Level (3-C)**	**Immediate Occupancy Level (1-B)**	**Operational Level (1-A)**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Comparison with performance intended for buildings designed under the *NEHRP Provisions*, for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Less damage and lower risk.	Much less damage and lower risk.

Table C.2: Data Collection Requirements (ASCE 2000).

	Level of Knowledge							
Data	**Minimum**		**Usual**				**Comprehensive**	
Rehabilitation Objective	BSO or Lower		BSO or Lower		Enhanced		Enhanced	
Analysis Procedures	LSP, LDP		All		All		All	
Testing	No Tests		Usual Testing		Usual Testing		Comprehensive Testing	
Drawings	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Construction Documents	Or Equivalent
Condition Assessment	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive
Material Properties	From Drawings or Default Values	From Default Values	From Drawings and Tests	From Usual Tests	From Drawings and Tests	From Usual Tests	From Documents and Tests	From Compre-hensive Tests
Knowledge Factor (κ)	0.75	0.75	1.00	1.00	0.75	0.75	1.00	1.00

Table C.3: Structural Performance Levels and Damages-Vertical Members (ASCE 2000).

Elements	Type	Structural Performance Levels: Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Many braces yield or buckle but do not totally fail. Many connections may fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary.	Same as primary.	Same as primary.
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent

Table C.4: Modeling parameters and acceptance criteria for nonlinear procedures-structural steel components (ASCE 2000).

Component/Action	Modeling Parameters: Plastic Rotation Angle, Radians a	b	Residual Strength Ratio c	Acceptance Criteria: Plastic Rotation Angle, Radians IO	Primary LS	Primary CP	Secondary LS	Secondary CP
Beams—flexure								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							
Columns—flexure [2, 7]								
For $P/P_{CL} < 0.20$								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $d\ \frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{460}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							

Table C.5: Modeling parameters and numerical acceptance criteria for nonlinear procedures-reinforced concrete beams (ASCE 2000).

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table C.6: Modeling parameters and numerical acceptance criteria for nonlinear procedures-reinforced concrete columns (ASCE 2000).

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases [5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ $d/2$			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > $d/2$			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.
5. For columns controlled by shear, see Section 6.5.2.4.2 for acceptance criteria.