INVESTIGATION OF THE EFFECT OF STRUCTURAL GRID DISCONTINUITY ON THE EARTHQUAKE BEHAVIOR OF MIDRISE RC MOMENT FRAMES

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ABSTRACT

INVESTIGATION OF THE EFFECT OF STRUCTURAL GRID DISCONTINUITY ON THE EARTHQUAKE BEHAVIOR OF MIDRISE RC MOMENT FRAMES

Reinforced concrete (RC) moment frames are the most common form among the building type structures in Turkey. The contemporary seismic design of building type structures evolves around the definition of deviations from an ideal structure that has a square-like floor plan, symmetric and uniform framing and mass distribution. These deviations are called the irregularities which are grouped in two as horizontal and vertical irregularities. There exists a horizontal irregularity that is not addressed in the current approach but it is needed to be investigated due to its possible impact on the lateral stiffness of the structure. It is the discontinuity of the horizontal grid in the structural frame.

The purpose of this study is to investigate the earthquake response of the RC buildings that have horizontal grid discontinuities. It is intended to observe the level of additional vulnerability on the seismic response of RC moment frames due to this irregularity.

Two 5-story reinforced concrete buildings are modeled in order to investigate the effects of the grid discontinuity phenomenon using nonlinear time-history analysis. The first building has discontinuous beams and framing that demonstrates the irregularity in the plan while the second one is the control case with uniform structural framing.

The results are evaluated based on the member damage level. It is observed that the buildings with grid discontinuities are more vulnerable than those without irregularities to seismic excitation. Further study is needed to define a procedure to mitigate the vulnerability created by the horizontal grid discontinuity.

ÖZET

ORTA YÜKSEKLİKTEKİ BETONARME MOMENT ÇERÇEVELERDE YAPISAL IZGARA SÜREKSİZLİKLERİNİN DEPREM DAVRANIŞI ÜZERİNDEKİ ETKİLERİNİN DEĞERLENDİRİLMESİ

Betonarme çerçeveler Türkiye'de bulunan bina tipi yapılarda en sık görülen yapısal çerçeve tipidir. Bina tipi yapıların sismik tasarımı, düzgün kalıp planı, simetrik ve düzgün çerçeve ve kütle dağılımından farklılaşma tanımı çevresinde tanımlanan bir takım sınırlama ve tasarım şartları ile yapılmaktadır. Bu sapmalar yatay ve düşey düzensizlikler olmak üzere iki grup içerisinde tanımlanır. Modern şartnamelerde yer almamakla birlikte yatay düzensizlik kategorisinde değerlendirilebilecek bir düzensizlik türü daha mevcuttur. Izgara süreksizlik düzensizliği olarak adlandırılabilecek bu düzensizliğin yapının yatay rijitliği üzerindeki muhtemel etkisi sebebiyle deprem taleplerini değiştirme potansiyelinden dolayı araştırılması faydalı olacaktır.

Bu çalışmanın amacı grid süreksizlik düzensizliği bulunan betonarme binaların bu sebeple oluşan davranış değişikliğinin araştırılmasıdır. Söz konusu süreksizlik sebebiyle yapılarda oluşması muhtemel risk artışı değerlendirilecektir.

Grid süreksizlik düzensizliğinin etkilerini araştırmak amacıyla 5 katlı, üç boyutlu, 2 bina modellenerek zaman tanım alanında doğrusal olmayan analizler yapılmıştır. İlk bina plan düzensizliği gösteren süreksiz kiriş ve çerçeveye sahip, ikinci bina ise kontrol amaçlı düzenli binadır.

Yapılan çalışma sonucunda eleman hasarı bazında yapılan sistem performans değerlendirmeleri neticesinde grid süreksizlik düzensizliği olan binaların düzenli binalara kıyasla deprem talepleri altında daha savunmasız bir performans seviyesine ilerlediği görülmektedir. Yatay ızgara süreksizlik düzensizliğinin etkilerini daha iyi anlayabilmek ve etkileri dizginleyebilme yönünde yöntemler geliştirebilmek amacıyla daha ileri çalışmalara ihtiyaç vardır.

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CHAPTER 1

INTRODUCTION

1.1. General

Reinforced concrete (RC) frames are the most common form of the structural frames among the building type structures in Turkey. The seismic design of building type structures revolves around the definition of deviations from an ideal structure that has a square-like floor plan, symmetric and uniform framing and mass distribution. Limitations and requirements are defined for the deviations from this ideal structure. These deviations are called the irregularities which are grouped in two groups as horizontal and vertical irregularities. There exists a horizontal irregularity that is not addresses in the modern codes but needed to be investigated due to its possible impact on the lateral stiffness of the structure. It is the discontinuity of the horizontal grid in the structural frame. It is a known fact that decreases in the lateral stiffness could cause an increase in the period of the structure which results an increase on the drift demand of the structure. Therefore, there is a risk that the displacement demand of the structure related to the earthquake excitation could reach to limits that could danger the integrity of the structure.

The grid irregularities in plan are a common defect for the structures in the Turkish building stock. It is partly due to architectural concerns but also due to lack of seismic knowledge both in the architects and the engineers. One of the main features in these buildings is the discontinuity of the beams lines. Since the framing action relies on the resistance of the beams and columns against rotations at the joints together, beam discontinuity affects the seismic performance adversely. Although there are observations that some of the structures having discontinuous beams perform insufficiently in the seismic actions, there are no existing limitations or recommendations about it in the both 2007 and 2018 Turkish Earthquake Regulations (TER 2007, TER 2018). Therefore, it is up to designers to decide about the extent of beam discontinuity.

Mid-rise reinforced concrete moment resisting frames constitutes the large portion of present building stock in Turkey and they are susceptible to earthquake excitation due to many reasons, such as improper structural framing, poor concrete quality, poor steel grade, poor workmanship etc. The grid discontinuity exacerbates the existing vulnerabilities further.

1.2. Purpose of the Study

The purpose of this study is to investigate the earthquake response of the reinforced concrete (RC) buildings having grid discontinuities in floor plan. It is intended to observe the level of additional vulnerability it is causing on the seismic response of RC moment frames. Due to the starting time of the study and the time limitations, 2007 version of the Turkish Earthquake Regulation (TER 2007) is considered as the reference regulation for the study. Also, the requirements of TS 500, Requirements for design and construction of reinforced concrete structures (TS 500) are followed.

1.3. Literature Review

Irregularities such as frame discontinuity and improper structural plans are widely seen in buildings in Turkey and it has a considerable effect on the seismic behavior of the structures. However the limited amount of research is available with respect to this type of frame discontinuities in plan.

Bal and Ozdemir (2006) investigated the behavior of reinforced concrete frames which have frame discontinuities along the perimeter frames. This perimeter frame discontinuity is caused by architectural concerns and constitutes slab bands instead of beams (Figure 1.1). Structural damages related to frame discontinuities are seen in Figure 1.2. For this study, 12 buildings which were designed in accordance with 1975 and 1997 Turkish Earthquake Regulation were chosen and seismic analyses were conducted for 5 different conditions. It is reported that nearly 40% reduction on ultimate strength of existing structures has been observed due to the perimeter frame discontinuity. The authors stated that beam discontinuity issue and column-slab connections control has to be considered in the earthquake specifications.

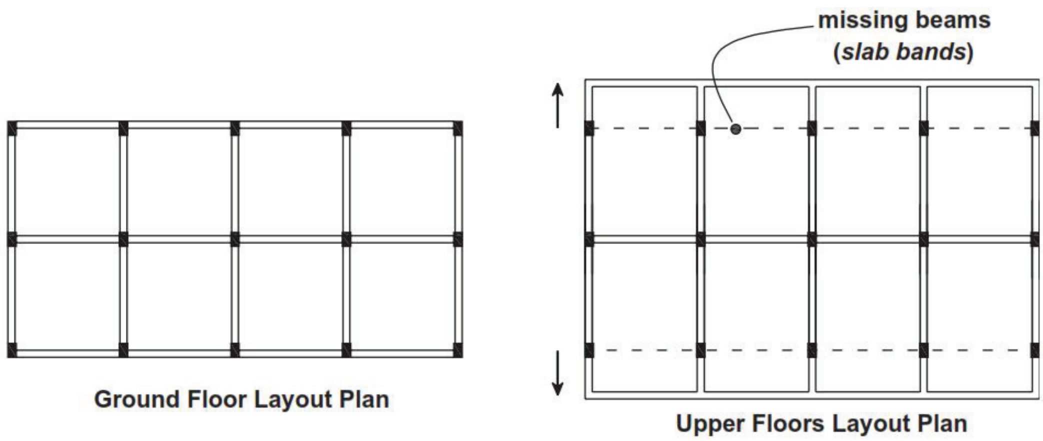

Figure 1.1. Slab bands between perimeter columns
(Source: Bal & Ozdemir, 2006)

Figure 1.2. Damages related to frame discontinuity
(Source: Bal & Ozdemir, 2006)

Erkan (2003) carried out a statistical study in relation with earthquake damages in Düzce region and investigated the soil properties, damage states, soil-structure interaction and design parameters. The buildings examined were classified in terms of beam discontinuities and a discontinuity coefficient, α was determined as shown in Equation 1.1. The damage states according to average α values of the examined buildings are demonstrated in Figure 1.3 and the buildings having α value larger than 70% are in the undamaged condition as seen in the Figure.

$$\alpha = \frac{The\ number\ of\ continous\ beam\ frames}{The\ number\ of\ all\ frames} \qquad \text{(Eq 1.1)}$$

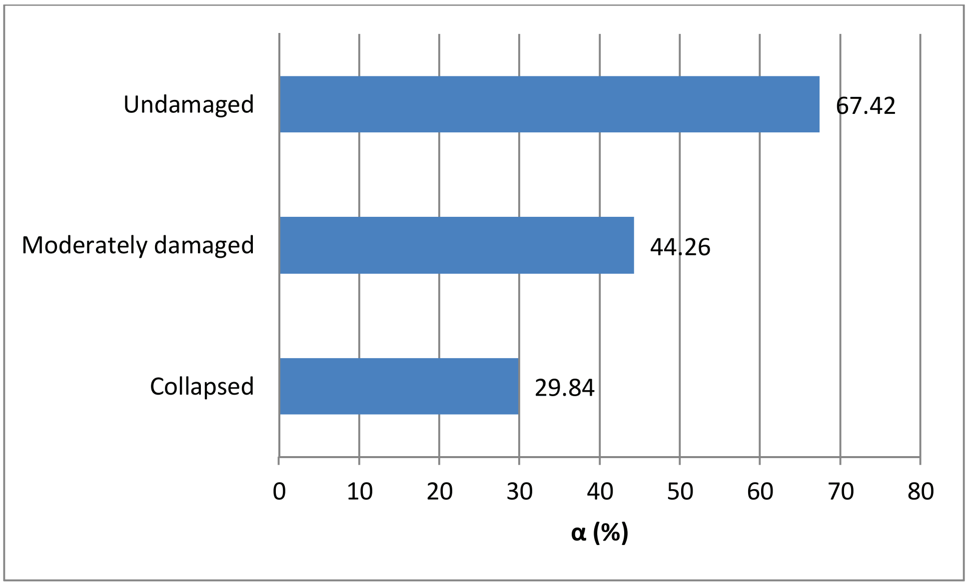

Figure 1.3. Damage states according to α values
(Source: Erkan, 2003)

Ozmen (2012) evaluated the horizontal discontinuities in the buildings by using the reassessment of the previous studies about the irregular frames. The author mentioned that it is not possible to find foreign studies related to beam discontinuities because this irregularity especially in the perimeter frames are specific to Turkish architecture. Due to the fact that the beam discontinuities can be only in one direction of the buildings, the author proposed a criterion which is an improved version of Equation 1.1. As it could be seen in Equation 1.2, the buildings are assessed in each directions separately. The author applied this criterion for the buildings which were studied by Bal and Ozdemir (2006) and concluded that the criterion is successful in determining the beam discontinuity.

$$\alpha_{x(y)} = \frac{\textit{The number of columns with beams}}{\textit{The number of all columns}} \geq 0.70 \qquad \text{(Eq 1.2)}$$

Ozmen (2012) also tried the proposed criterion for three floor plans (Figure 1.4) and calculated α_x and α_y values in order to compare with the limiting value, 0.70. In conclusion of the assessments of the buildings, the author stated that the restriction of the amount of beam discontinuity considering the limiting criterion is essential in the earthquake-prone regions. In addition, the author mentioned that new remarks related to beam discontinuities have to be added into earthquake codes.

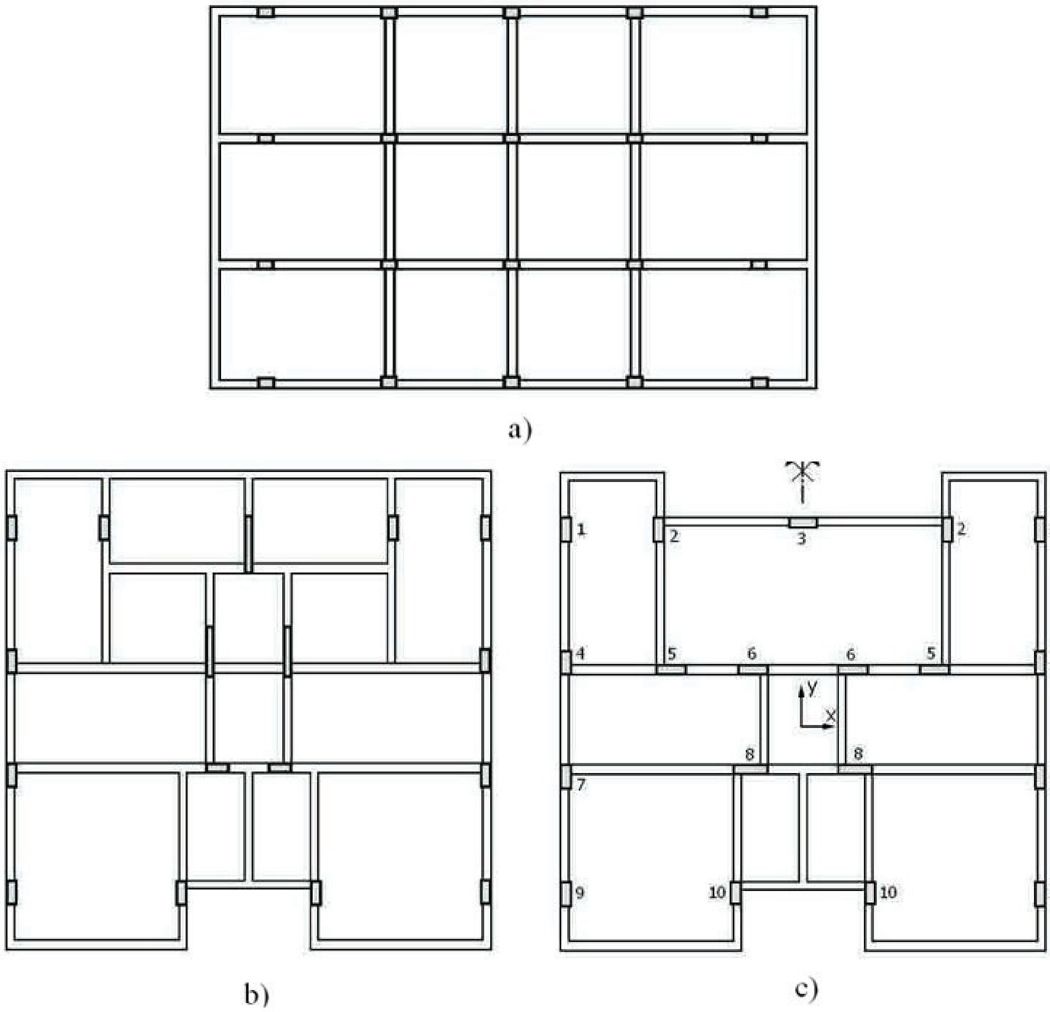

Figure 1.4. Schematical Floor Plans a) Type1 b) Type 2 c) Type 3
(Source: Ozmen, 2012)

Arslan et al. (2018) investigated reinforced concrete buildings having torsional irregularities such as beam discontinuities and asymmetric plans. In this study, 8 different five and seven storey buildings with and without beam discontinuities and with different torsional irregularities as shown in Figure 1.5 were taken into consideration by using linear elastic, pushover and time history analysis. Life Safety target performance was attained according to the time history analysis results; however more conservative results were obtained in the linear elastic analysis. The authors also stated that further study is needed to examine different types of buildings in terms of performance assessment and the results should be reflected in the earthquake code requirements.

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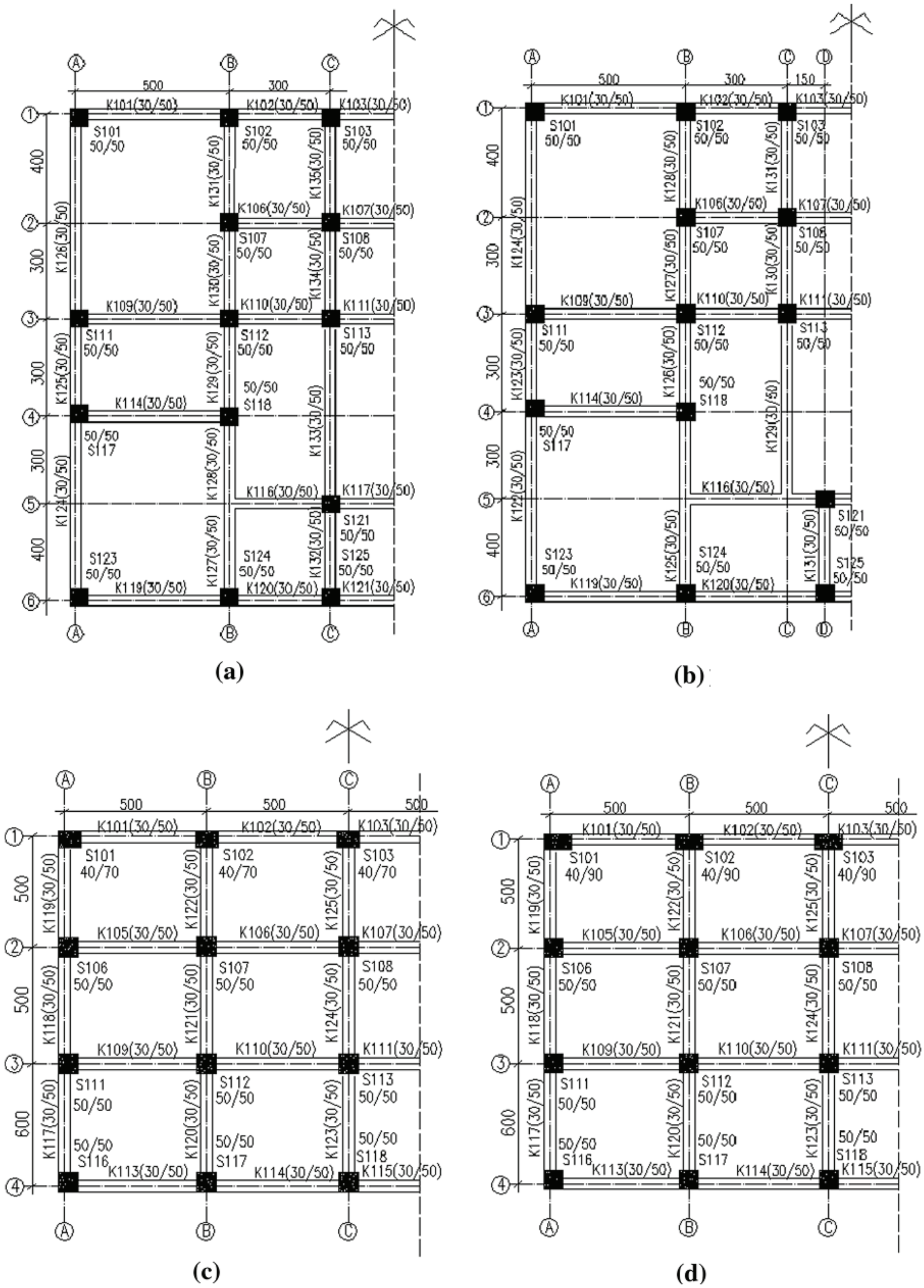

Figure 1.5. Typical floor plans of considered buildings a) building 1 b) building 2 c) building 3 d) building 4 (Source: Arslan et al., 2018)

1.4. Thesis Organization

The first chapter of the thesis describes the purpose of the study and presents the literature review about the grid discontinuities on plan. The last section of the chapter presents the organization of the thesis.

In the second chapter, the methodology used for the thesis is presented. The methods for seismic performance evaluation and the proposed assessment technique are described in a detailed way in this chapter as well.

Case studies regarding the performance evaluation of the selected buildings are given in the third chapter of the thesis.

The final chapter includes the discussion of results and concluding remarks of the thesis.

CHAPTER 2

METHODOLOGY

2.1. Introduction

In this chapter, the method used to categorize and evaluate the performance of the RC moment frames with horizontal grid discontinuity is presented. In order to create a measure to indicate the level of horizontal grid discontinuity, a grading scheme is proposed. The performance evaluation is performed through application of the method defined in the Turkish Earthquake Regulation 2007 version (TER 2007). The relevant parts of the definitions in TER 2007 will be summarized in this chapter for convenience. TER 2007 performance evaluation procedures have an increasing level of complexity for the increased level of irregularities. Considering that some of the cases that are going to be analyzed have high level of irregularities, the most sophisticated form of the methods is utilized. The analysis is performed by three-dimensional nonlinear time-history procedures.

2.2. An Attempt to Categorize the Grid Discontinuities

TER 2007 defines the irregularities in buildings in plan and elevation. However these irregularity definitions do not cover the issue regarding the horizontal grid discontinuity. Especially, due to the architectural concerns, the grid discontinuity is developed in the framing decisions. Unfortunately, the common architectural and engineering knowledge is not sensitive the possible effects of such a preference on the seismic response of the structures. The drift limitations in the seismic code regulations provide an indirect control on the extent of the softening it creates. Unfortunately, the drift definitions in the 2007 regulations for new structures are not always sufficient to provide the necessary level of seismic safety (Donmez, 2013). Therefore, it is attempted to develop a measure to quantify the level of horizontal grid discontinuity.

The proposed method includes a discontinuity factor with the following definition, Equation 2.1.

$$Discontinuity\ Factor = 1 - \frac{connectivity\ points}{all\ points} \qquad \text{(Eq 2.1)}$$

The discontinuity factor is calculated in two orthogonal directions separately. The procedure for this factor is established according to the connectivity of columns to the beams and the conditions for achieving connectivity are demonstrated as follows:

- If the column is connected to two beams for the considered direction and the beams are spanning from column to columns, this column will take 2 points for the inner and 1 for the outer columns.
- If cantilever beams exist, the length of the cantilever has to be smaller than one third of the adjacent beam span. Otherwise the cantilever span is accepted as a regular span and graded accordingly.
- The score of the shear wall is doubled to imply its importance on the lateral resistance.
- The total grade is calculated as 2 points for internal columns and 1 point for external columns.
- The calculated factor has a value between 0 and 1. Zero means no discontinuity and 1 means full discontinuity.

2.2.1. Examples for Calculation of Discontinuity Factor

Some examples related to reinforced concrete buildings with and without grid discontinuities are given in this section in order to present some outputs with the defined procedure

- Building A

$$Discontinuity\ Factor\ X = 1 - \frac{22}{33} = 0.33$$

$$Discontinuity\ Factor\ Y = 1 - \frac{21}{34} = 0.38$$

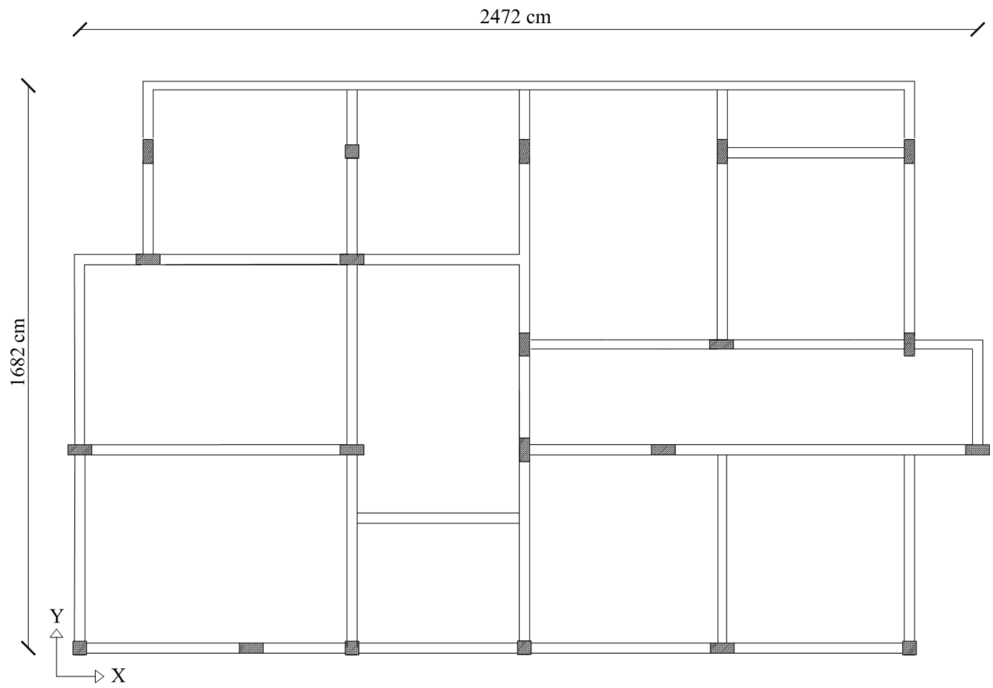

Figure 2.1. Floor plan of building A

- Building B

$$Discontinuity\ Factor\ X = 1 - \frac{34}{34} = 0$$

$$Discontinuity\ Factor\ Y = 1 - \frac{35}{35} = 0$$

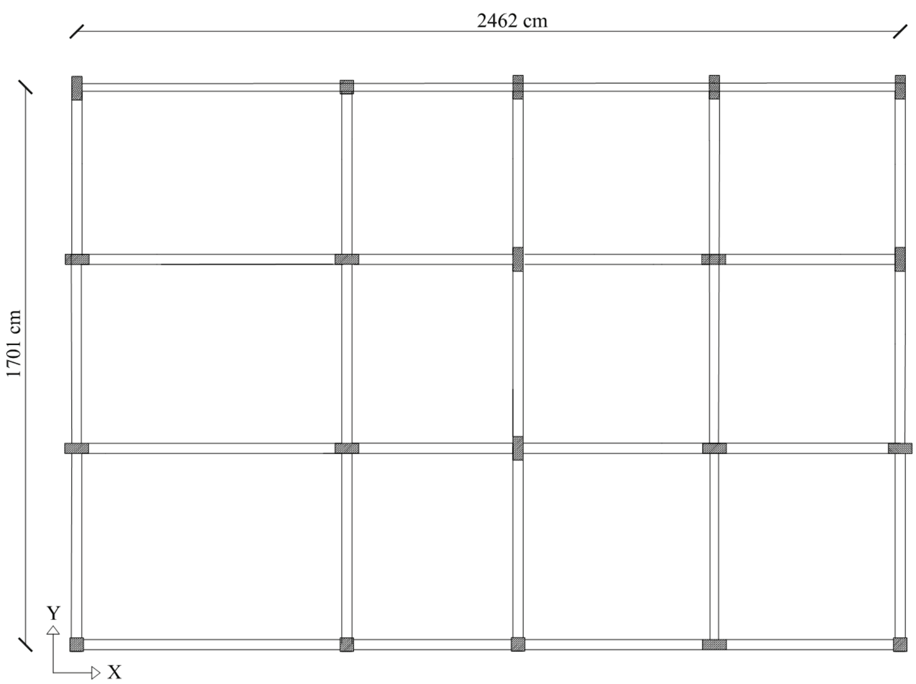

Figure 2.2. Floor plan of building B

- Building C

$$Discontinuity\ Factor\ X = 1 - \frac{48}{59} = 0.19$$

$$Discontinuity\ Factor\ Y = 1 - \frac{25}{41} = 0.39$$

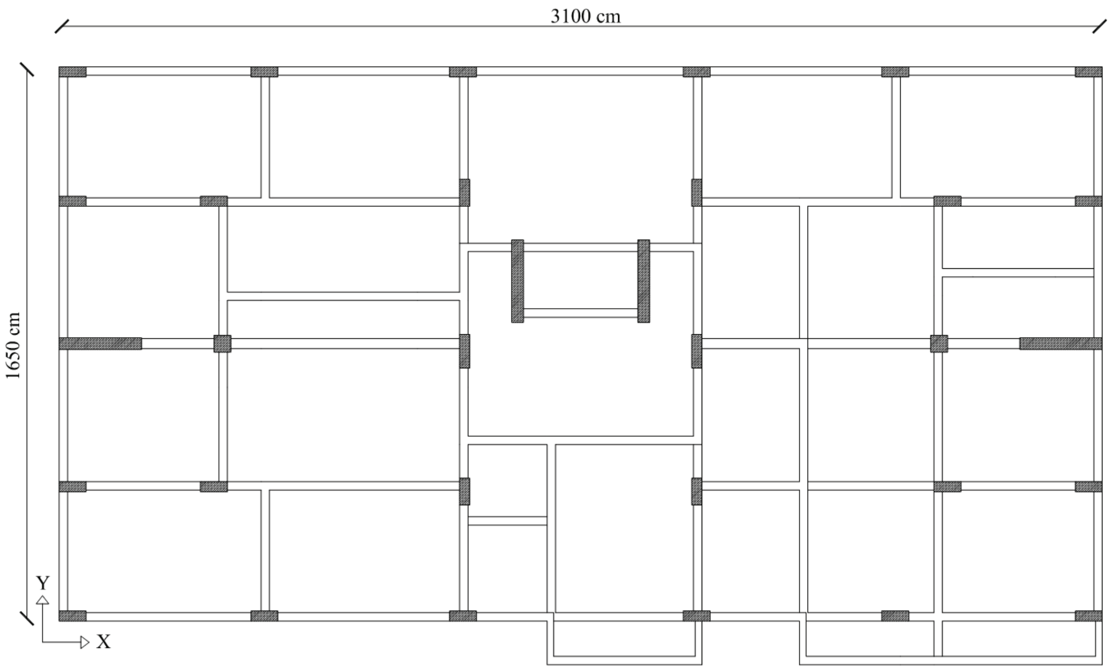

Figure 2.3. Floor plan of building C

2.3. The Performance Evaluation

TER 2007 specifies procedures to evaluate the performance of existing structures. The procedures span from relatively simple linear analysis to three-dimensional nonlinear time history analysis. The selection of the method is controlled through the effectiveness of the higher modes, torsional irregularity and the mass participation factor of the first mode. Due to the nature of the structures with high horizontal grid discontinuity, it is needed to perform the most advanced form of the evaluation procedures. The relevant steps of the procedure are summarized in this section.

2.3.1. General Rules for Seismic Analysis

In order to determine the seismic performance of the existing or reinforced buildings linear or nonlinear methods can be used according to Turkish Earthquake Regulation 2007. However, it is stated in the regulations that the performance evaluations carried out using these methods may not give similar results due to the differences between theoretical approaches. The 3-dimensional time-history analyses are selected as the type of the procedure due to the regulation demand for the high torsional (A1) irregularity of the irregular frame analyzed.

Building Importance Factor is not applied in the seismic analyses (I=1.0).

The combination of vertical and earthquake loads are taken into consideration in the seismic analyses and floor masses are determined with the full dead and a predefined portion of the live loads.

Seismic loads are applied in two orthogonal axes of the building in both directions.

Floors are defined as rigid diaphragm on the buildings and two horizontal translational and one vertical rotational degree of freedom are taken into consideration at each mass center without any eccentricity.

The uncertainties related to the buildings are taken into account with the knowledge level coefficients.

The interaction diagrams of the reinforced concrete sections under uniaxial or biaxial bending and axial force are evaluated as follows. Existing strengths of the concrete and reinforcing steel determined according to the knowledge levels are used in the seismic analyses. The maximum compression strain of concrete and the maximum tension strain of reinforcing steel may be taken as 0.0035 and 0.01, respectively. Interaction curves can be linearized properly and can be modeled as multi-linear or multi-plane diagrams.

Connection regions can be taken into account as infinitely rigid end zones.

Effective stiffness (EI_e) of the "cracked" sections are considered in the seismic analyses. For the beams (EI_e) = 0.4 EI, for the columns

$(EI_e) = 0.4EI$ if $N_D / (A_c * f_{cm}) \leq 0.10$

$(EI_e) = 0.8EI$ if $N_D / (A_c * f_{cm}) \geq 0.40$

and linear interpolation can be applied for the intermediate values.

2.4. Time History Analysis

In this thesis, considering that TER 2007 leaves the decisions for the selection the ground motions to the user with minimal requirements, the procedures with respect to nonlinear time-history analysis of TER 2018 is adopted. It should be mentioned that the selected procedure stays within the requirements defined for TER 2007.

Time history analysis is step-by-step analysis of the structure to be obtained its dynamic response using direct integration.

Considering the TER 2018 11 earthquakes with two components are taken into consideration for the seismic analyses of the buildings. The earthquakes are applied in two orthogonal directions (X and Y) simultaneously in the model. Later the analyses are reiterated by turning the directions of the earthquake records 90 degrees. Following the TER 2018 requirements maximum 3 earthquake records from the same earthquake is considered.

Deformation demand for the ductile sections and internal force demand for brittle structures are calculated according to the results from 22 (2x11) seismic analysis. The results are obtained considering the mean value of the largest absolute values from the performed analyses.

2.4.1. Suite of Ground Motion Pairs and Scaling

A suite of eleven ground motion pairs was considered for the nonlinear time-history analysis. These ground motions are listed in Table 2.1. As it is seen from the Table 2.1, eleven ground motions from eight different earthquakes are chosen for the seismic analyses carried out in this study. The ground motions records waveforms are presented from Figure 2.4 to 2.14.

The acceleration records related to above mentioned ground motions were scaled by the recommendations of the Turkish Earthquake Regulation 2018. The horizontal spectrum of each ground motion is obtained by the square root sum of the squares of the two horizontal components of each set of earthquake records. Then, the amplitudes of the average of the resulting spectra of all selected records between 0.2 T_p and 1.5 T_p (T_p = uncracked period of the structure) are scaled by the definition in TER 2018. According to the definition, the amplitudes of the average of the resulting spectra should be 1.3 times larger than the amplitudes in design spectrum for the periods mentioned.

In this study, TER 2007 design spectrum in accordance with Z3 soil conditions is considered and 1.3 times larger of it is used for determining scale factors of selected earthquakes. Scale factors of ground motions are demonstrated in Table 2.2.

Response spectra and the average response spectra of scaled ground motions are seen in Figure 2.15 and 2.16, respectively. As it can be seen from the Figures, the values of average response spectrum are larger than the specified design spectrum corresponding to Z3 soil type.

Table 2.1. Summary of eleven ground motions

Earthquake	Station	Date	Magnitude
Duzce-1	Duzce	12.11.1999	7.3
Duzce-2	Bolu	12.11.1999	7.3
Erzincan	Erzincan	13.03.1992	6.8
Kocaeli-1	Yarımca	17.08.1999	7.8
Kocaeli-2	Duzce	17.08.1999	7.8
Kocaeli-3	Iznik	17.08.1999	7.8
Imperial Valley	El Centro	19.05.1940	6.7
Kobe	Takarazuka	16.01.1995	6.9
LomaPrieta	WAHO	18.10.1989	7.1
Northridge	Saticoy	17.01.1994	6.8
Parkfield	Cholame	28.06.1966	6.0

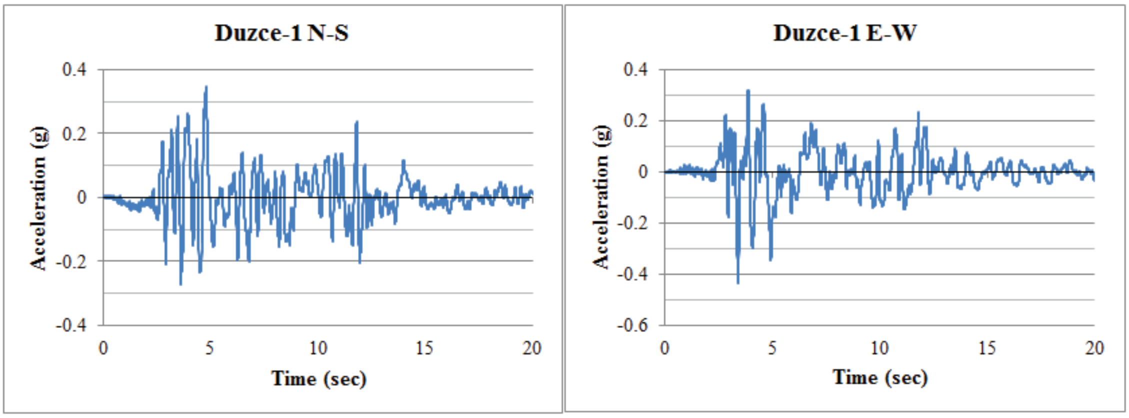

Figure 2.4. Duzce-1 ground motion pairs

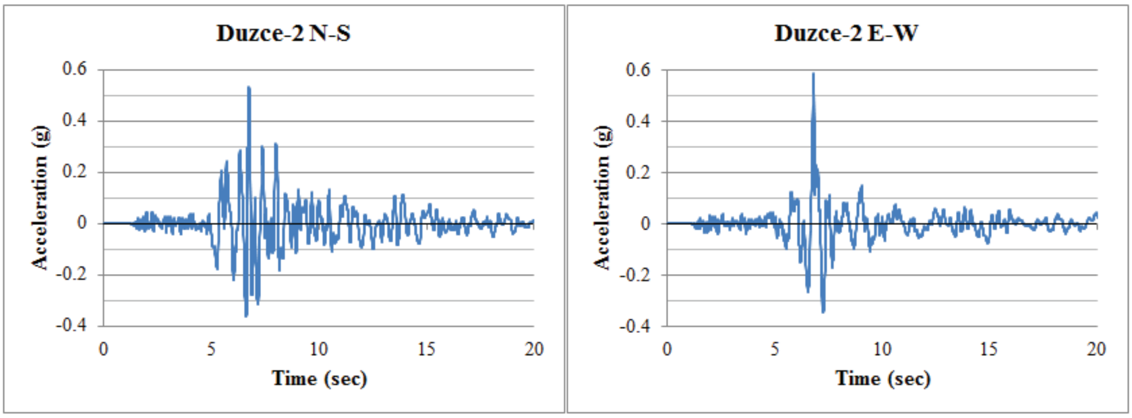

Figure 2.5. Duzce-2 ground motion pairs

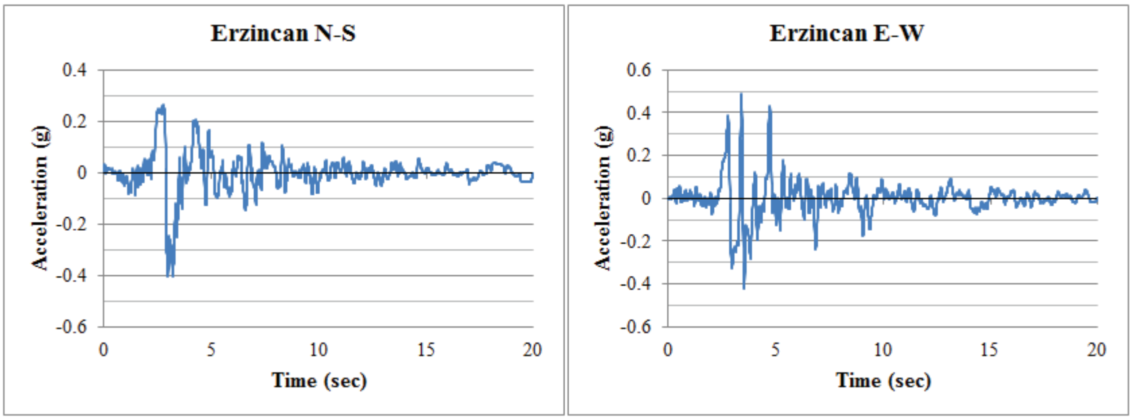

Figure 2.6. Erzincan ground motion pairs

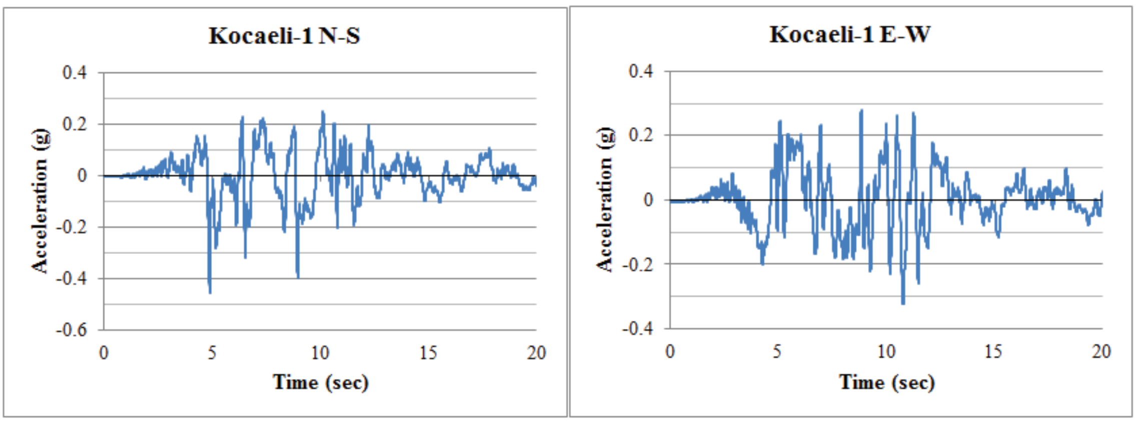

Figure 2.7. Kocaeli-1 ground motion pairs

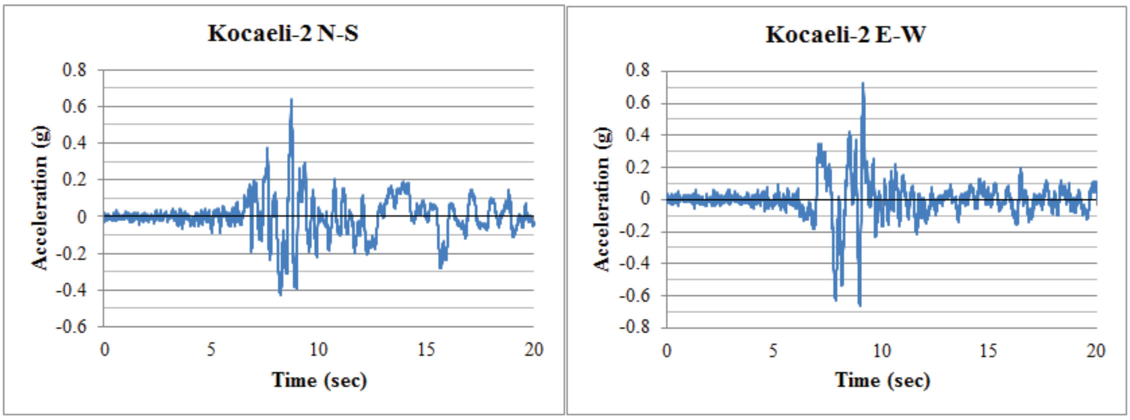

Figure 2.8. Kocaeli-2 ground motion pairs

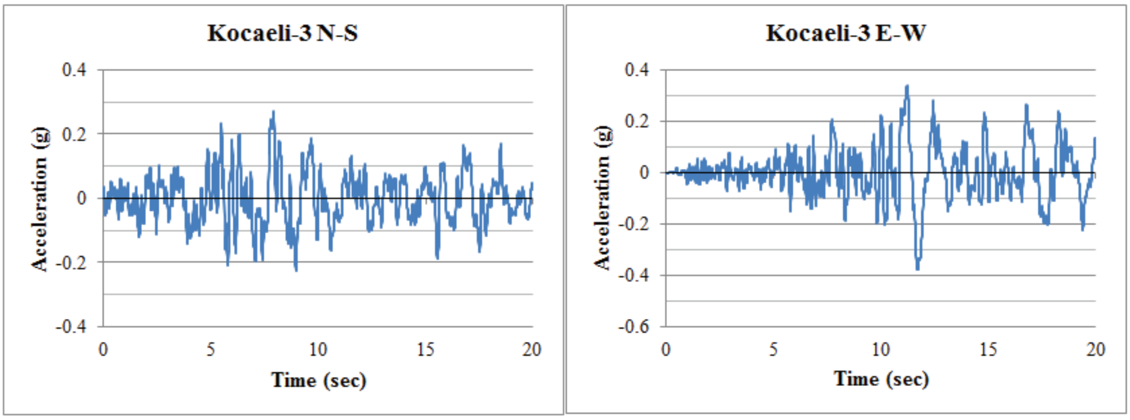

Figure 2.9. Kocaeli-3 ground motion pairs

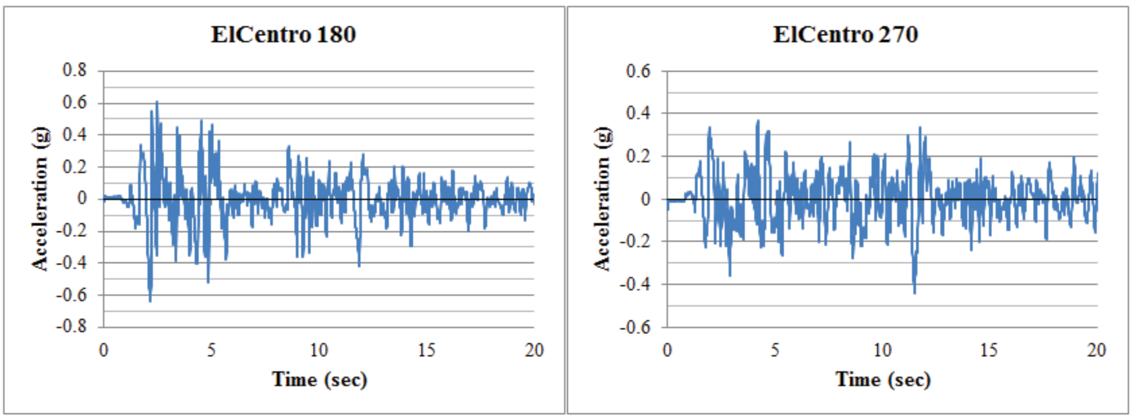

Figure 2.10. El Centro ground motion pairs

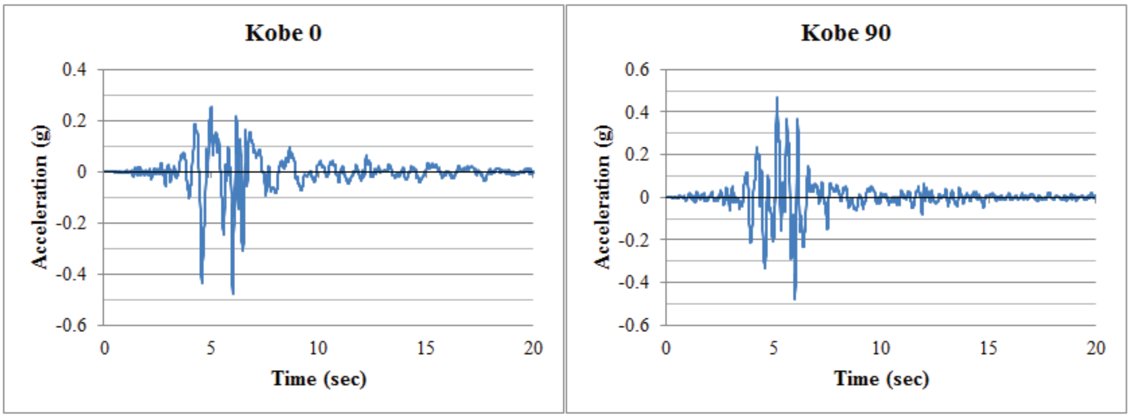

Figure 2.11. Kobe ground motion pairs

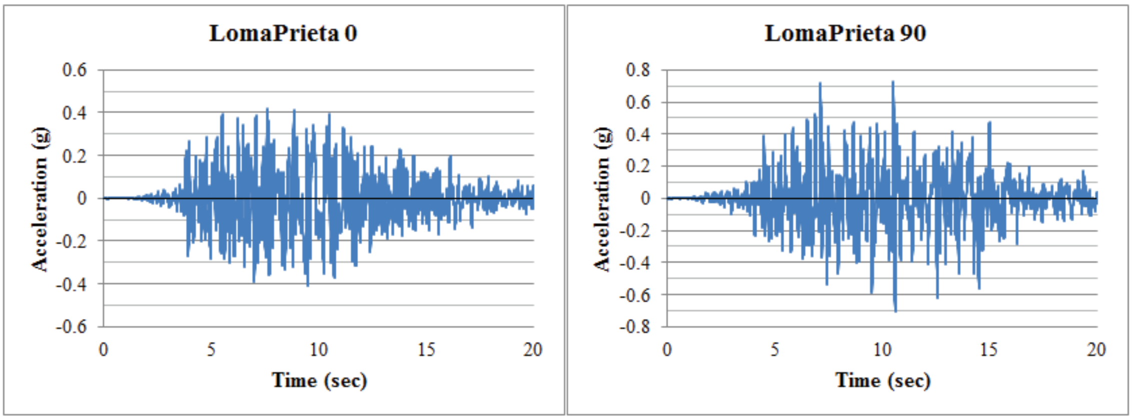

Figure 2.12. LomaPrieta ground motion pairs

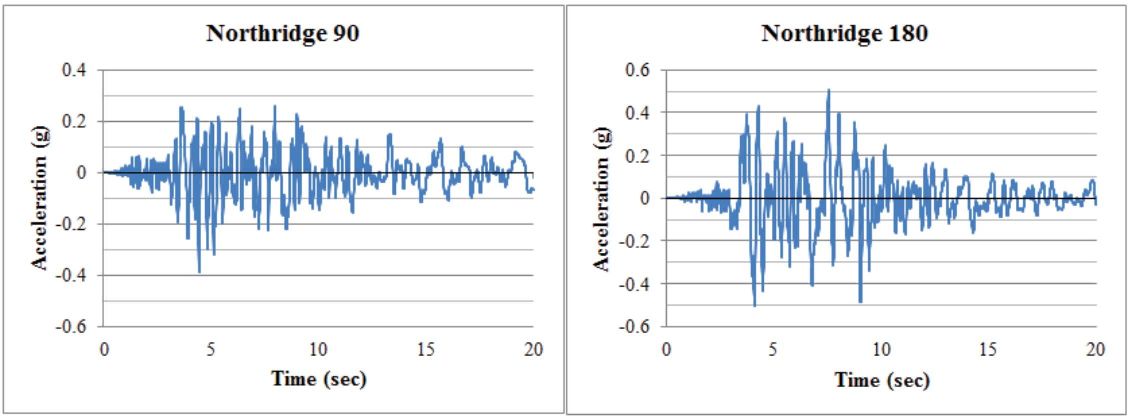

Figure 2.13. Northridge ground motion pairs

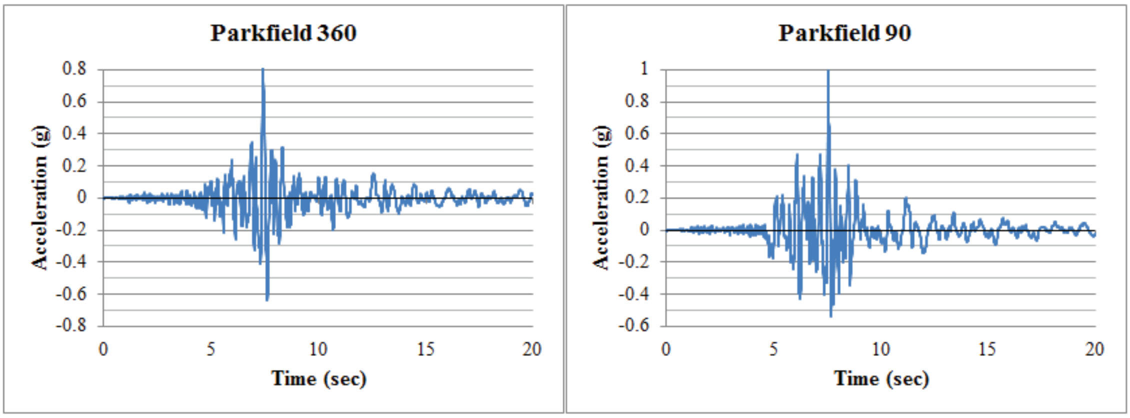

Figure 2.14. Parkfield ground motion pairs

Table 2.2. Scale factors of ground motions

Earthquake	Scale Factor
Duzce-1	0.85
Duzce-2	0.71
Erzincan	1.04
Kocaeli-1	1.41
Kocaeli-2	1.89
Kocaeli-3	2.95
Imperial Valley	2.04
Kobe	0.69
LomaPrieta	1.14
Northridge	1.06
Parkfield	2.25

2.5. Determining Strain Demands

The structural model in SAP2000 is developed using concentrated hinges at the member ends. The analysis results give the plastic rotations of these concentrated hinges as output. The rotation to curvature transformation is performed dividing the plastic rotation values to predefined plastic hinge length by the TER 2007. The plastic curvature demand, ϕ_p is calculated with respect to plastic rotation θ_p obtained from time history analysis as shown in Equation 2.2. The length of plastic hinge (L_p) is taken as the half of the section length (h) in the considered direction ($L_p = 0.5$ h).

$$\emptyset_p = \frac{\theta_p}{L_p} \quad \text{(Eq 2.2)}$$

The total curvature demand, ϕ_t is calculated by summing up the plastic curvature demand, ϕ_p and yield curvature, ϕ_y as shown in Equation 2.3.

$$\emptyset_t = \emptyset_y + \emptyset_p \quad \text{(Eq 2.3)}$$

By moment-curvature analysis, the compression strain in concrete and the tension strain in reinforcing steel in reinforced concrete members are determined in order to compare those strain values with the limiting strain values to provide information about the damage level of member ends.

The limiting strain values of the cross-sections related to the damage levels are introduced as follows:

For the Minimum Damage Limit (ML);

$(\varepsilon_{cu})_{MN} = 0.0035 \quad ; \quad (\varepsilon_s)_{MN} = 0.01$

For the Safety Limit (SL);

$(\varepsilon_{cu})_{SL}$=0.0035+0.01(ρ_s / ρ_{sm}) $\leq$ 0.0135 ; $(\varepsilon_s)_{SL}$=0.04

For the Collapsing Limit (CL);

$(\varepsilon_{cu})_{CL}$=0.004+0.013(ρ_s / ρ_{sm}) $\leq$ 0.018 ; $(\varepsilon_s)_{SL}$=0.06

2.6. Damage Limits

According to the TER 2007, three limit conditions have been defined for ductile elements based on cross-sections of them. These are Minimum Damage Limit (ML), Safety Limit (SL) and Collapsing Limit (CL), respectively. Minimum damage limit defines the onset of the beyond-elastic behavior, safety limit defines the limit of the beyond-elastic behavior of the cross-section with safely providing its strength and collapsing limit defines the limit of pre-collapse behavior of the cross-section. As shown in Figure 2.1, the cross-sections deformations of which before ML are in the Light Damage Region, deformations of which between ML and SL are in the Moderate

Damage Region, deformations of which between SL and CL are in the Severe Damage Region and deformations of which beyond CL are in the Collapsing Region.

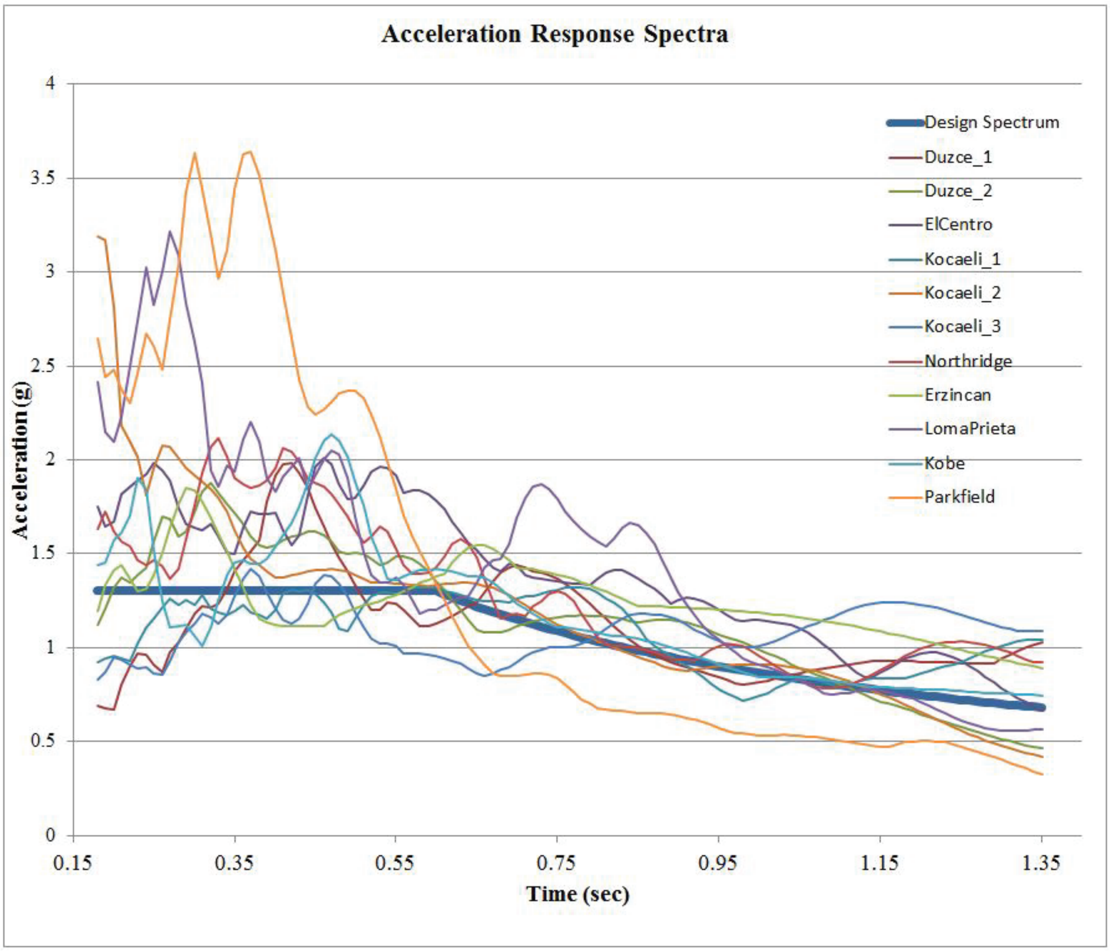

Figure 2.15. Response spectra of ground motion pairs (SRSS)

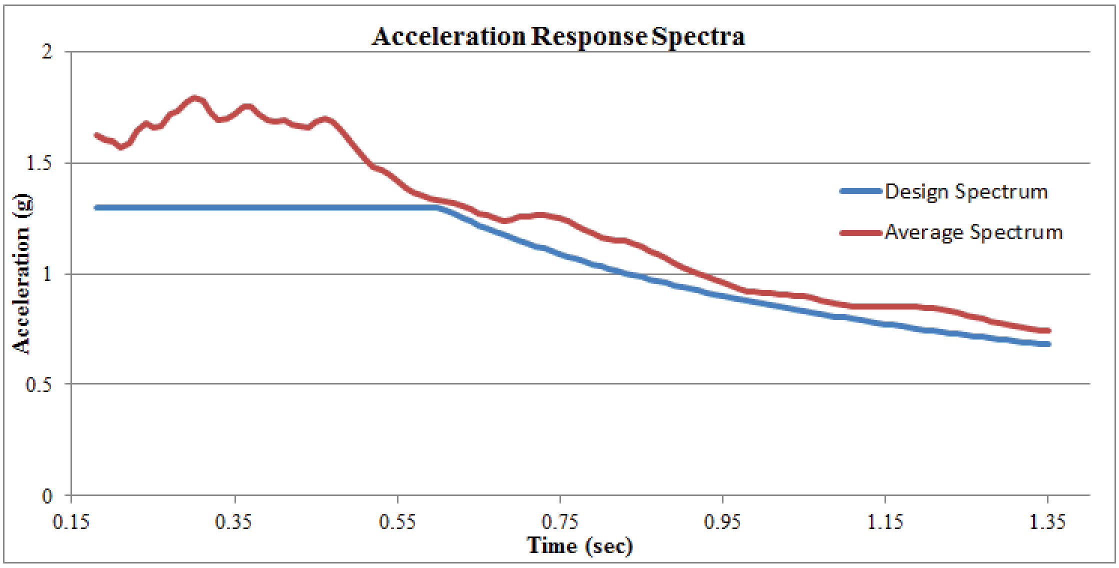

Figure 2.16. Average response spectra of ground motions

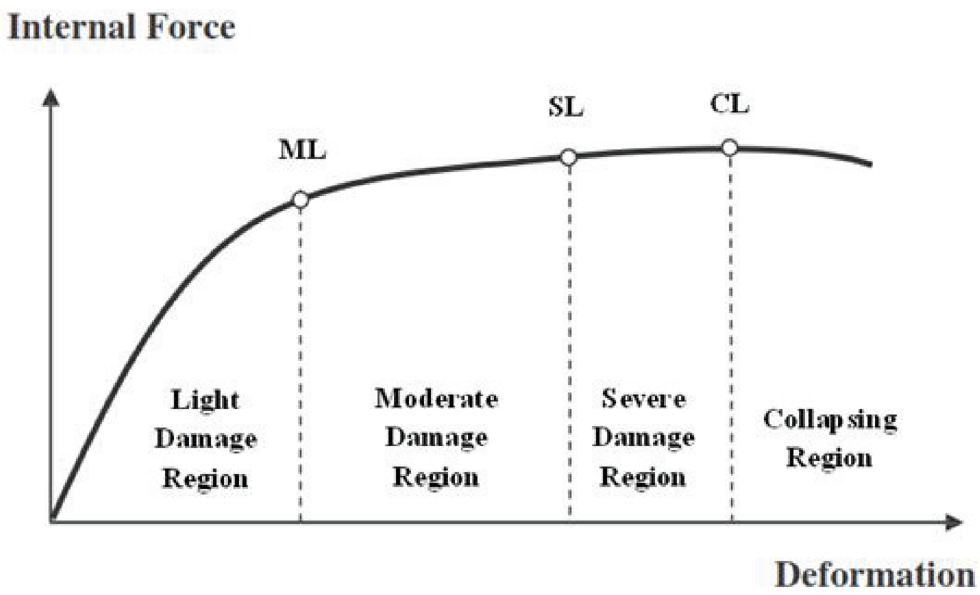

Figure 2.17. Cross-sectional damage levels
(Source: TER 2007)

2.7. Estimation of the Building Performance

The seismic performance of the building is determined based on the condition of the damages under applied seismic excitation and defined with different performance levels. These are Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP) and Collapse (C), respectively.

2.7.1. Immediate Occupancy Performance Level

In the aftermath of the seismic analyses, 10% of the beams in any floors can be in the Moderate Damage Region providing that the rest of the structural members are in Light Damage Zone. Considering the brittle structural members have to be strengthened, such buildings can be agreed to be in the Immediate Occupancy Performance Level.

2.7.2. Life Safety Performance Level

In the aftermath of the seismic analyses, 30% of the beams and a lot of columns in any floors can be in the Severe Damage Region; however shear contributions of the so-called columns must be lower than 20% of the columns in any floors. For the top floor shear contribution of the severely-damaged columns can be 40% percent of the columns in this story. All other structural members must be in the Light Damage or Moderate Damage Region on the condition that shear contribution of the columns which have moderately-damaged top and bottom sections must be lower than 30% of the columns in any floors. Considering the brittle structural members have to be strengthened, such buildings can be agreed to be in the Life Safety Performance Level.

2.7.3. Collapse Prevention Performance Level

In the aftermath of the seismic analyses, 20% of the beams in any floors can be in the Collapsing Region. All other structural members must be in the Light Damage, Moderate Damage Region or Severe Damage Region on the condition that shear contribution of the columns which have moderately-damaged top and bottom sections must be lower than 30% of the columns in any floors. Considering the brittle structural members have to be strengthened, such buildings can be agreed to be in the Collapse Prevention Performance Level. Buildings in this performance level are risky to survive and have to be retrofitted considering seismic rehabilitation is economical or not.

2.7.4. Collapse Performance Level

If the building does not satisfy Collapse Prevention Performance Level, Collapse Performance Level is valid for this building. Buildings in this performance level are greatly risky to survive and have to be retrofitted. However the seismic rehabilitation for such type of buildings may not be economical.

2.7.5. Targeted Performance Level for the Buildings

In TER 2007, there are three different types of earthquake levels used in the performance evaluation of the buildings in terms of probability of exceedance in 50 years. The minimum performance targets according to these earthquake levels are given in Table 2.2.

- Earthquake level with a 50% probability of exceedance in 50 years has the coordinates of acceleration spectrum as approximately the half of the spectrum shown in Figure 2.2. The return period of this earthquake is approximately 72 years.

- Earthquake level with a 10% probability of exceedance in 50 years has the coordinates of acceleration spectrum as the spectrum shown in Figure 2.2. The return period of this earthquake is approximately 475 years.

- Earthquake level with a 2% probability of exceedance in 50 years has the coordinates of acceleration spectrum as approximately 1.5 times of the spectrum shown in Figure 2.3. The return period of this earthquake is approximately 2475 years.

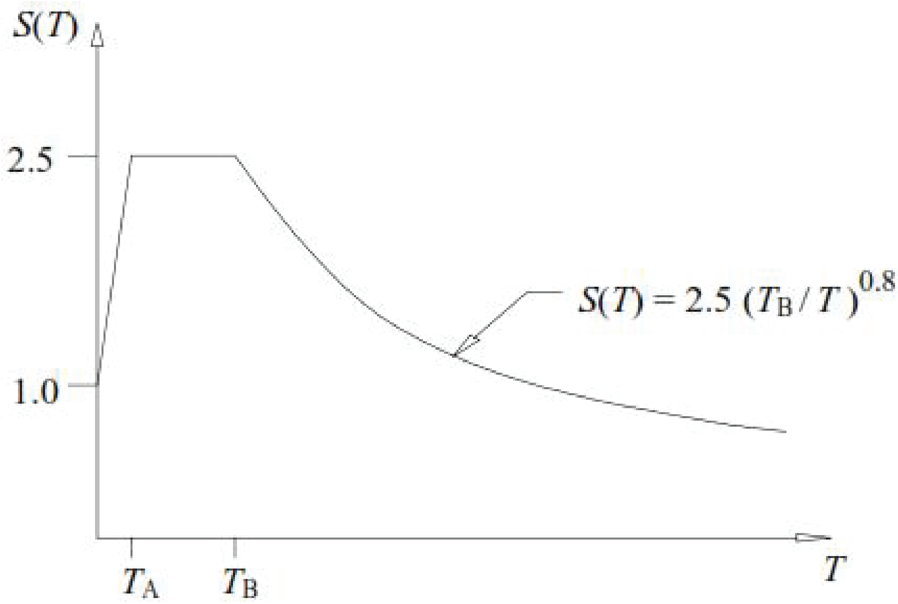

Figure 2.18. Design Spectrum in TER 2007

Table 2.3. Targeted performance levels for the buildings
(Source: TER 2007)

Purpose of Occupancy and Type of Building	Probability of exceedance		
	50% in 50 years	10% in 50 years	2% in 50 years
The buildings to be utilized immediately after the earthquake: Hospitals, health facilities, fire fighting buildings, communication and power facilities, transportation stations, governorate, county and municipality administration buildings, first aid and emergency planning stations	-	IO	LS
Intensively and long-term occupied Buildings: Schools, dormitories, boarding houses, military barracks, prisons, museums, etc.	-	IO	LS
Intensively and short-term occupied Buildings: Cinema, theatre, concert saloons, cultural centers, sport facilities.	IO	LS	-
Buildings containing hazardous materials: Buildings containing or storing toxic, explosive and flammable materials, etc.	-	IO	CP
Other buildings: Buildings after than above defined buildings (Residential and office buildings, hotels, tourustic facilities, industrial structures, etc.)	-	LS	-

CHAPTER 3

CASE STUDY

3.1. Introduction

As a case study, two 5-story reinforced concrete structures are modeled in order to investigate the effects of the grid discontinuity irregularity using nonlinear time history analysis. The case study buildings are designed as moment frames without shear walls and meet the requirements of Turkish Standard 500 (Requirements for design and construction of reinforced concrete structures, TS 500) and Turkish Earthquake Regulation (TER 2007). The first building has heavy horizontal grid discontinuity that demonstrates irregularity in plan while the second one is the control case with a uniform horizontal grid structure. The case study structure with the high horizontal grid irregularity is adopted from an existing building and the uniform structure is modified version of it to avoid irregularity. Therefore, the masses of the frames are very close to each other.

This chapter gives the description of the case study structures and the results of the performance analysis performed by TER 2007. The performance analysis is done through nonlinear time history analysis using SAP2000 software.

3.2. Description of the Case Study Structures

The selected case study building is located in Izmir. The location selection is done to have high seismic demand and Izmir is a high seismic region in Turkey. The selected building has heavy horizontal grid discontinuity in its original form. But some minor modifications are done to avoid unnecessary complexity. Afterwards, the building is adopted as irregular and well-framed structures by making alterations on the beam and column orientations and seismic performance evaluations are carried out. Case study structures have 5 stories and have a uniform story height of 3 m. The planar dimensions of irregular and well-framed structures are (24.72 x 16.82) m^2 and (24.62 x 16.82) m^2, respectively. Both of the buildings meet the requirements of TS 500 and

TER 2007. The soil type of the structures is Z3 according to TER 2007. The other parameters related to the structures are given as follows;

- Building Importance Factor : 1.0
- Seismic Zone : 1 (A_o = 0.4)
- Concrete Quality : C30 (E_c = 33000 Mpa)
- Steel Grade : S420

For the case study structures the same load configurations are taken into consideration for both normal and roof floors. The parameters with respect to structure loads are given as follows;

- Total Dead Load : 6.24 kN/m^2 (including slab weight)
- Live Load : 2 kN/m^2
- Live Load Participation Factor : 0.3

In the modeling of both buildings, rigid panel zones are assumed and fixed support condition is taken into account at the foundation level. At the story levels rigid diaphragm assumption is considered and combined movement of the constrained joints is provided. The plastic hinges according to the moment-curvature relationship of the cross-sections are constituted at the beam and column ends.

3.2.1. Case Study 1 – Irregular Structure

The floor plan of the irregular structure is given in Figure 3.1. Even though it has frame discontinuities, it meets the requirements of TS 500 and TER 2007. The 3D model of the irregular structure is shown in Figure 3.2. Slabs are not modeled; however the slab loads are considered in the model and rigid diaphragms are constituted at the story levels. The dimensions of the beams are 30/50 cm, the dimensions of the columns are 30/70, 70/30, 40/40 cm and they are shown in Table 3.1 and in Table 3.2, respectively. The reinforcing steel details can be seen in Table 3.1 and in Table 3.2 for beams and columns, respectively. The longitudinal stirrup spacing is 9 cm for all beams and 8 cm for all columns for the confinement zones of the sections in the irregular structure.

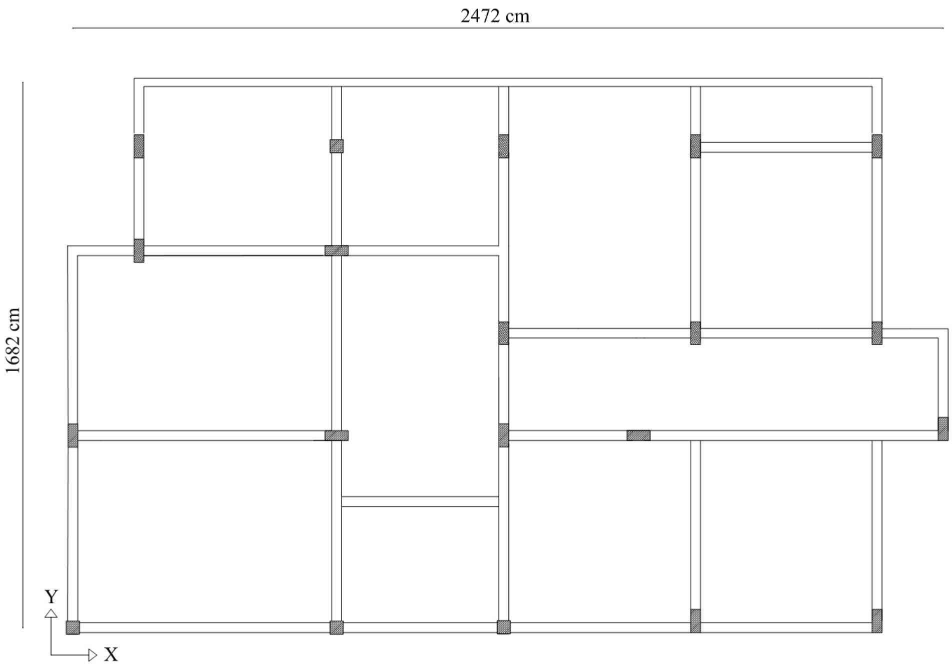

Figure 3.1. Floor plan of irregular structure

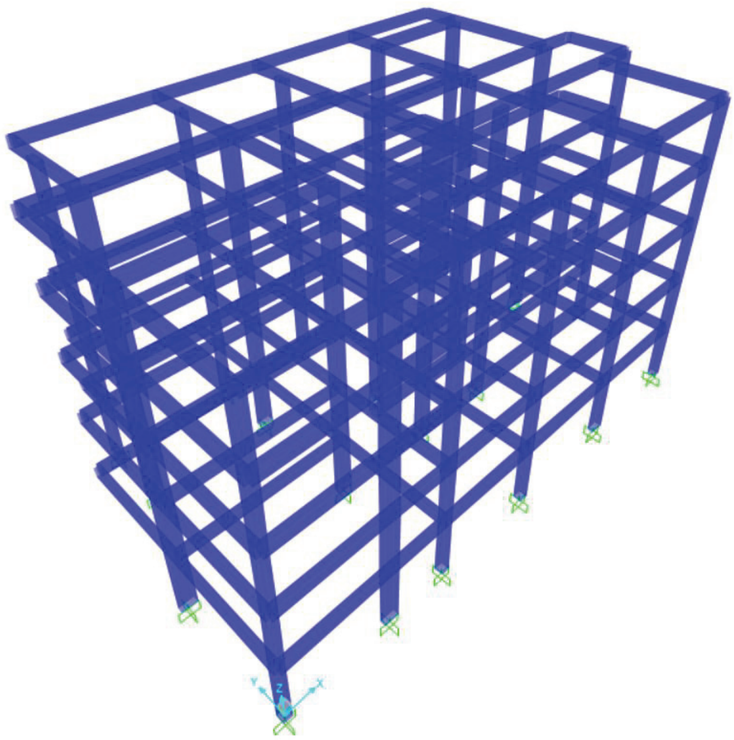

Figure 3.2. 3D model of the irregular structure

Table 3.1. Dimensions and reinforcements of the beams (irregular frame)

Beams	**Width (cm)**	**Height (cm)**	**Top Reinforcement**		**Bottom Reinforcement**	
B_1-2-3-4	25	50	4	ϕ20	3	ϕ16
B_5	30	50	6	ϕ20	6	ϕ16
B_6	30	50	3	ϕ20	3	ϕ20
B_7-8	30	50	5	ϕ26	4	ϕ26
B_9-10	30	50	4	ϕ26	4	ϕ26
B_11	30	50	4	ϕ26	4	ϕ26
B_12	30	50	5	ϕ26	5	ϕ26
B_13-14-15	35	50	5	ϕ26	5	ϕ26
B_16	35	50	5	ϕ26	4	ϕ26
B_17	30	50	5	ϕ18	4	ϕ18
B_18-19-20-21-22	30	50	5	ϕ20	4	ϕ20
B_23-24	30	50	6	ϕ18	4	ϕ18
B_25	30	50	5	ϕ26	5	ϕ26
B_26	30	50	3	ϕ26	3	ϕ26
B_27-28-29-30-31	30	50	7	ϕ20	5	ϕ18
B_32-33-34-35-36-37	30	50	5	ϕ26	4	ϕ26
B_38	30	50	5	ϕ26	4	ϕ20
B_39-40	30	50	5	ϕ26	5	ϕ20
B_41	30	50	5	ϕ20	4	ϕ20
B_42-43	30	50	5	ϕ26	5	ϕ20
B_44	30	50	6	ϕ20	6	ϕ20

Table 3.2. Dimensions and reinforcements of the columns (irregular frame)

Columns	**Width (cm)**	**Height (cm)**	**Reinforcement**
S01, S03, S04, S05, S08, S10, S13	70	30	18 ϕ18
S02, S16, S17 S18, S20	40	40	12 ϕ18
S06, S07, S09, S11, S12, S14, S15, S19	30	70	18 ϕ18

3.2.1.1. Structural Parameters of Case Study Building 1

Floor weights of the case study 1 building are 3700 kN per story considering live load reduction factor, 0.3.

The modal analysis resulted the first three fundamental modes of the building as tabulated in the Table 3.3 considering the uncracked and cracked conditions.

The effective stiffness of the members are calculated according to the Section 2.3.1. of the thesis, as defined by TER 2007. For the beams, effective stiffnesses are taken as 0.4 EI while for the columns, it is calculated in terms of axial load on the columns and they are shown in Table 3.4.

Table 3.3. Fundamental modes of case study building 1

	Periods	
	Uncracked	**Cracked**
$\mathbf{T_x}$	0.9	1.505
$\mathbf{T_y}$	0.751	1.229
$\mathbf{R_z}$	0.697	1.099

3.2.2. Case Study 2 – Structure with the Uniform Frame

The floor plan of the uniform framed structure (24.62 x 16.82 m^2) is given in Figure 3.3 and this building also meets the requirements of TS 500 and TER 2007. The 3D model of the uniform framed structure is shown in Figure 3.4. Slabs are not modeled; however the slab loads are considered in the model and rigid diaphragms are constituted at the story levels. The dimensions of the beams are 30/50 cm except three beams which are 30/70 cm, the dimensions of the columns are 30/70, 70/30, 40/40 cm which are shown in Table 3.5 and in Table 3.6, respectively. The concrete quality of all members in this building is C30 concrete the modulus elasticity of which is 33000 MPa. The reinforcing steel details can be seen in Table 3.5 and in Table 3.6 for beams and columns, respectively. The longitudinal stirrup spacing is 9 cm for all beams and 8 cm for all columns for the confinement zones of the sections in the uniform framed structure.

Table 3.4 Effective bending rigidities of the cracked sections of the columns

	Floors				
Columns	**1**	**2**	**3**	**4**	**5**
S01	0.4	0.4	0.4	0.4	0.4
S02	0.49	0.45	0.4	0.4	0.4
S03	0.41	0.4	0.4	0.4	0.4
S04	0.42	0.49	0.43	0.4	0.4
S05	0.4	0.4	0.4	0.4	0.4
S06	0.48	0.44	0.4	0.4	0.4
S07	0.59	0.52	0.46	0.4	0.4
S08	0.53	0.48	0.43	0.4	0.4
S09	0.55	0.49	0.43	0.4	0.4
S10	0.42	0.4	0.4	0.4	0.4
S11	0.49	0.45	0.4	0.4	0.4
S12	0.61	0.53	0.47	0.4	0.4
S13	0.56	0.5	0.44	0.4	0.4
S14	0.57	0.51	0.45	0.4	0.4
S15	0.44	0.4	0.4	0.4	0.4
S16	0.4	0.4	0.4	0.4	0.4
S17	0.4	0.4	0.4	0.4	0.4
S18	0.47	0.43	0.4	0.4	0.4
S19	0.49	0.45	0.4	0.4	0.4
S20	0.4	0.4	0.4	0.4	0.4

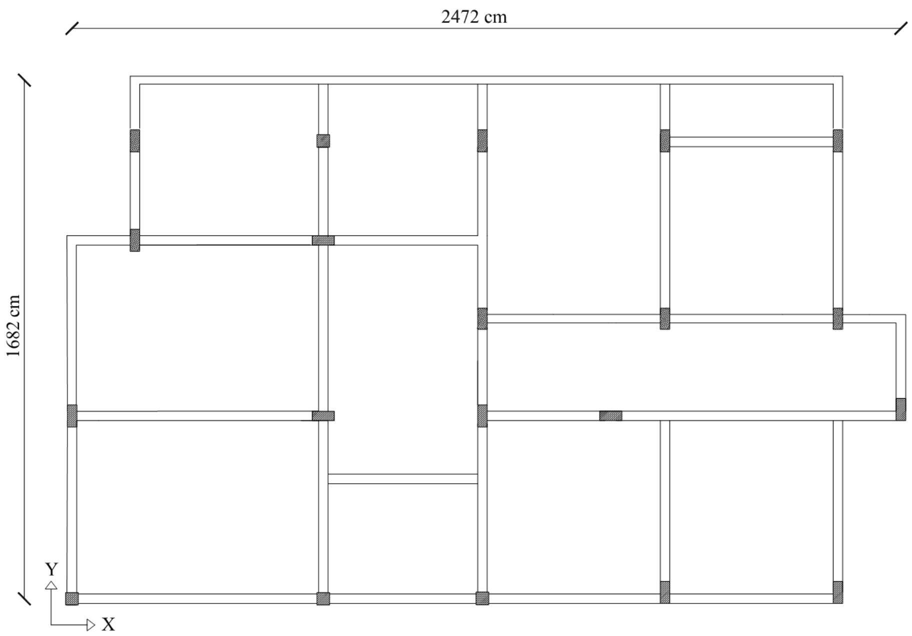

Figure 3.3. Floor Plan of Uniform-framed Structure

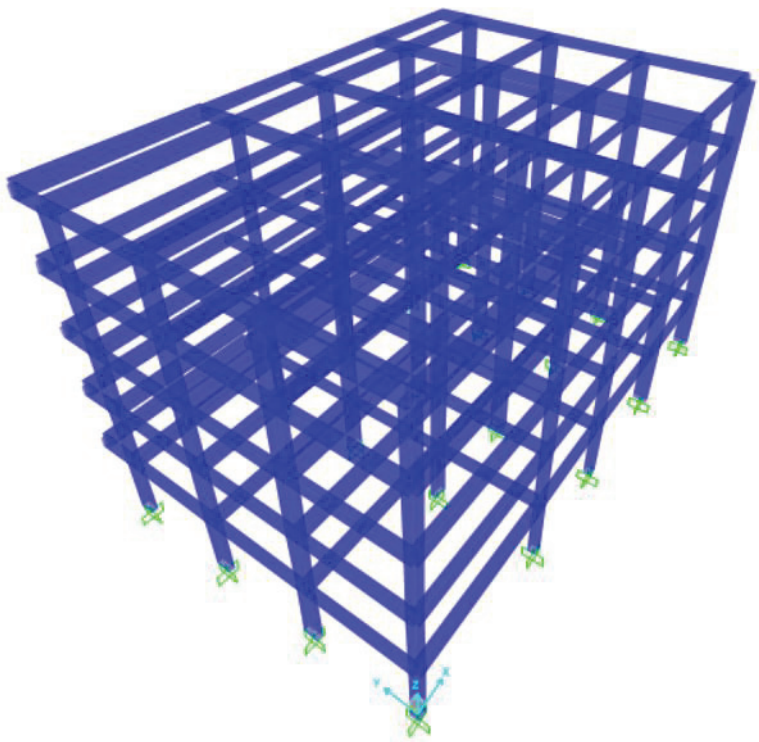

Figure 3.4. 3D Model of the Uniform-framed Structure

Table 3.5. Dimensions and reinforcements of the beams (uniform-framed)

Beams	Width (cm)	Height (cm)	Top Reinforcement		Bottom Reinforcement	
B_1	30	70	5	ϕ20	4	ϕ20
B_2-3-4	30	50	3	ϕ20	3	ϕ20
B_6-7-8	30	50	4	ϕ20	4	ϕ20
B_9	30	70	4	ϕ26	4	ϕ26
B_10-11-12	30	50	5	ϕ26	4	ϕ26
B_13	30	70	5	Φ26	4	Φ26
B_14-15-16	30	50	5	ϕ28	5	ϕ28
B_17	30	70	5	ϕ26	4	ϕ26
B_18-19-20	30	50	5	ϕ18	4	ϕ18
B_21-22-23-24	30	50	5	ϕ20	4	Φ20
B_25-26-27-28	30	50	5	ϕ26	5	ϕ26
B_29-30-31-32	30	50	5	ϕ26	5	ϕ26
B_33-34-35-36	30	50	5	ϕ26	5	ϕ26
B_37-38-39-40	30	50	5	ϕ20	4	ϕ20

Table 3.6. Dimensions and reinforcements of the columns (uniform-framed)

Columns	Width (cm)	Height (cm)	Reinforcement
S01, S03, S04, S05, S08, S10, S13	70	30	18 ϕ18
S02, S16, S18, S19, S21	40	40	12 ϕ18
S06, S07, S09, S11, S12, S14, S15, S17, S20	30	70	18 ϕ18

3.2.2.1. Structural Parameters of Case Study Building 2

Floor weights of the case study 1 building are 3868 kN per story considering live load reduction factor, 0.3.

The modal analysis resulted the fundamental three modes of the case study 1 building as tabulated in the Table 3.7 in terms of uncracked and cracked condition of the building.

The effective stiffness of the members are calculated according to the Section 2.5. For the beams, effective stiffnesses are taken as 0.4 EI while for the columns, it is calculated in terms of axial load on the columns and they are shown in Table 3.8.

Table 3.7. Fundamental modes of case study building 2

	Periods	
	Uncracked	Cracked
T_x	0.787	1.343
T_y	0.711	1.190
R_z	0.597	0.973

3.3. Modal Analysis Results

The comparisons are carried out for the first three fundamental modes of the case study structures. As can be seen in Table 3.9, the difference between modal characteristics is mostly due to the grid discontinuity phenomenon. Because of the improper frame type, the irregular structure has longer period values for the uncracked and cracked conditions as well.

Table 3.8. Effective bending rigidities of the cracked sections of the columns

	Effective Bending Rigidities				
Columns	**1**	**2**	**3**	**4**	**5**
S01	0.4	0.4	0.4	0.4	0.4
S02	0.49	0.45	0.4	0.4	0.4
S03	0.41	0.4	0.4	0.4	0.4
S04	0.42	0.4	0.4	0.4	0.4
S05	0.4	0.4	0.4	0.4	0.4
S06	0.48	0.44	0.4	0.4	0.4
S07	0.59	0.52	0.46	0.4	0.4
S08	0.53	0.48	0.43	0.4	0.4
S09	0.55	0.49	0.43	0.4	0.4
S10	0.42	0.4	0.4	0.4	0.4
S11	0.49	0.45	0.4	0.4	0.4
S12	0.61	0.54	0.47	0.4	0.4
S13	0.56	0.5	0.44	0.4	0.4
S14	0.57	0.51	0.45	0.4	0.4
S15	0.44	0.4	0.4	0.4	0.4
S16	0.4	0.4	0.4	0.4	0.4
S17	0.4	0.4	0.4	0.4	0.4
S18	0.47	0.43	0.4	0.4	0.4
S19	0.49	0.45	0.4	0.4	0.4
S20	0.4	0.4	0.4	0.4	0.4

Table 3.9 Period comparisons between case study structures

Periods					
Irregular Structure			**Uniform-framed Structure**		
	Uncracked	**Cracked**		**Uncracked**	**Cracked**
$\mathbf{T_x}$	0.90	1.51	$\mathbf{T_x}$	0.79	1.34
$\mathbf{T_y}$	0.75	1.23	$\mathbf{T_y}$	0.71	1.19
$\mathbf{R_z}$	0.70	1.1	$\mathbf{R_z}$	0.60	0.97

3.4. Time History Analysis Results

In order to evaluate the effect of the grid discontinuity on the performance of the case study building, time history analysis is performed. The analysis is performed with

SAP2000 software. The nonlinear model is developed defining the concentrated plastic hinges at the member ends. This approach needs the moment-rotation relations that represent plastic behavior at the end of the members. To develop the moment-rotation relations the moment-curvature relations of the cross-sections are calculated using external software. The rotation values are calculated using the predefined plastic hinge lengths. The plastic hinge length definition is taken from the Turkish Earthquake Regulations (TER 2007). It is half of the member depth for the beams and half of the member depth for the columns in terms of corresponding direction. So, plastic hinge lengths of weak and strong axis of the column are different according to its orientation. The corresponding plastic hinge definitions for each member are performed and applied to model. The effective stiffness definitions from TER 2007 are also used in modeling. Rigid diaphragms are defined for each individual floor level.

The ground motion couples that are defined in Chapter 2 are used time-history analysis. The maximum response instant of the buildings is taken to be the instant which the center of mass of the roof story reaches to the maximum displacement. The results are evaluated as defined in TER 2007. The individual value of a response parameter is needed to be calculated as the average of each 22-time history analysis considering 11 different earthquake data.

The performance evaluation procedure of the structure is based on the definitions in TER (2018). The evaluation is based on the percentages of the members with certain damage levels at each story. The damage levels are organized for each case study buildings. On the account of the fact that the severity of damage decreases at the upper storeys, the performance evaluations are performed only for the first and second storeys.

3.4.1. Evaluation of Damages at the Case Study Building 1

The time-history analysis results are evaluated considering the plastic hinges rotation levels occurred at the critical regions of the beams for the first and second stories of the case study building 1 (irregular structure). Evaluations are carried out related to strain limits for the concrete and reinforcement.

45% of the first story beams and 39% of second story beams are in the moderate damage region. The rest of the beams in the mentioned stories are in the light damage region. For the columns, the variation of the member damages between consecutive stories is more apparent and 25% of the columns in the second story are in the moderate damage region while 10% of the columns in the first story are in the same damage condition.

An example related to evaluation of beam and column damage region for the case study building 1 is given as follows; B7-8 beam at the first story has 0.008797 rad plastic rotation according to the average value of the 22 earthquake set. Plastic hinge length of this beam can be taken as the half depth of the beam, Lp = 0.5 x 0.50 = 0.25 m. So, plastic curvature is plastic rotation / Lp = 0.008797 / 0.25 = 0.035165 rad/m. Finally, the total curvature is found by summing up plastic curvature with the yield curvature which is 0.0088 rad/m. Total curvature is 0.043965 rad/m for this beam and strain values for the concrete section and reinforcement are 0.00172 and 0.0166, respectively. According to these values, the beam is in the moderate damage region. S12 column in the second story has 0.002585 rad plastic rotation according to the average value of the 22 earthquake set as well. Plastic hinge length of the column is 0.15 m (Lp = 0.5 x 0.30 = 0.15 m). Plastic and total curvatures are calculated 0.017232 rad/m and 0.0349 rad/m, respectively. Strain values for this column related to the total curvature are 0.00715 for the concrete section and 0.0145 reinforcing steel and this condition demonstrates the column in relation is in the moderate damage region.

3.4.2. Evaluation of Damages at Case Study Building 2

Similar to first structure, the time-history analysis results are evaluated considering the plastic hinges rotation levels occurred at the critical regions of the beams for the first and second stories of the building. Again the evaluations are carried out related to strain limits for the concrete and reinforcement.

All beams for the first and second story of the case study building 2 are in the light damage region. In other words, there are not any beams having severely even moderately damaged. The same condition is valid for the columns as well and all columns for the first and second story of the uniform-framed structure are in the light damage region.

3.4.3. Comparing of the Performance of the Structures

As mentioned previously the irregular structure has moderately damaged beams and columns while all members of the uniform framed structure are only in the light damage region. Two columns in the first story and 5 columns in the second story have moderate damages for the case study building 1 (irregular structure) according to the time history analysis made by using 11 earthquake sets. The severity of the column damages is determined by the strain values related to reinforcing steel as the analysis mentioned above. The Figure 3.5 demonstrates the comparison of column damage states between case study building 1 and case study building 2 (uniform framed structure). The comparison is done for the 7 columns which have moderate damages at the irregular frame. The limiting line (Minimum Damage Limit (ML) = 0.01) in Figure 3.5 shows the limit strain value in terms of reinforcing steel and the columns having larger strain values than 0.01 are in moderate damage region while having smaller than 0.01 are in light damage region. As it is seen in the Figure 3.5 the so-called columns for the uniform framed structure have strain values below the limiting line and therefore they are in light damage region.

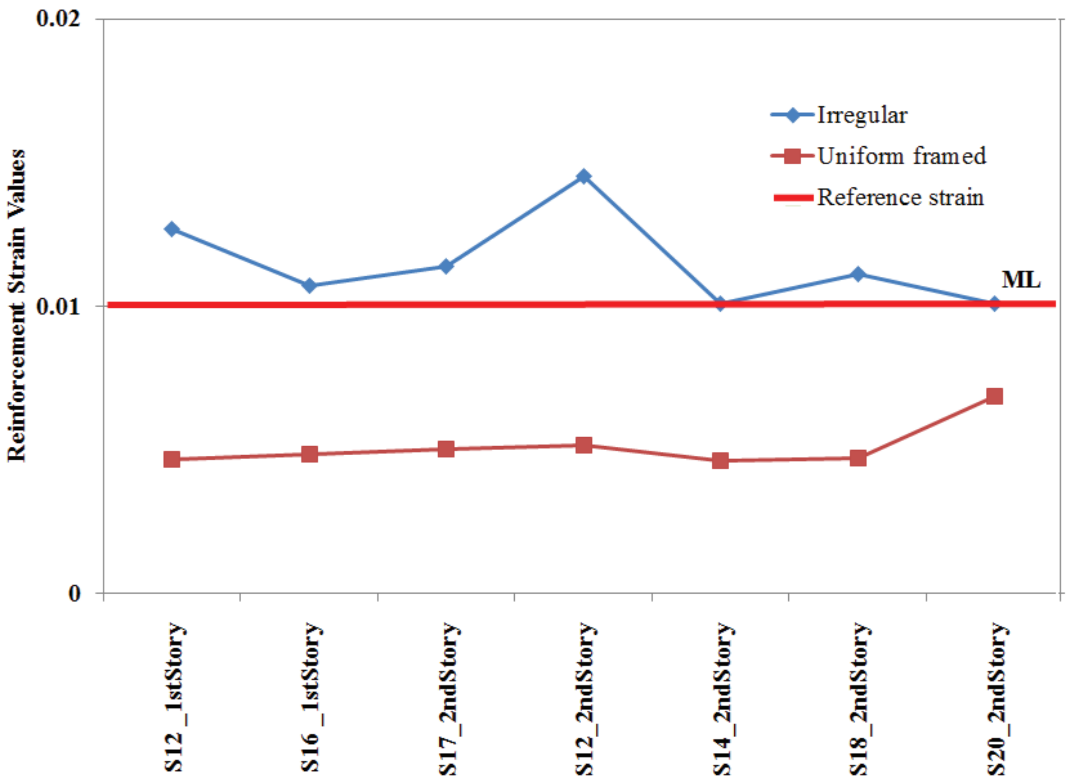

Figure 3.5. Comparison of Column Damage States

Another comparison between case study building 1 and 2 is related to top displacement values at the center of mass considering the 22 time history analyses. The average top displacement values considering the analysis regarding case study building 1 and 2 are 272 mm and 233 mm, respectively (Table 3.10). As it is expected the case study building 1 (irregular structure) has larger top displacement values than case study building 2 (uniform framed structure) because of the grid discontinuity phenomenon. This condition provides 17% larger displacement demand when compared between these two buildings.

Table 3.10 Comparison Top Displacement of the Structures

Earthquakes	**Top Displacement Values (mm)**	
	Irregular Structure	**Uniform Framed Structure**
Duzce-1_0	172.16	158.94
Duzce-1_90	261.5	187.82
Duzce-2_0	183.62	173.78
Duzce-2_90	170.45	189.97
Erzincan_0	452.53	373.72
Erzincan_90	389.82	343.48
Kocaeli-1_0	290.14	254.91
Kocaeli-1_90	274.82	223.38
Kocaeli-2_0	381.24	319.55
Kocaeli-2_90	553.34	433.27
Kocaeli-3_0	347.03	305.33
Kocaeli-3_90	423.92	351.32
Imperial Valley_0	226.93	215.96
Imperial Valley_90	214.19	197.88
Kobe_0	295.39	234.05
Kobe_90	253.08	199.21
LomaPrieta_0	132.54	124.5
LomaPrieta_90	178.93	172.66
Northridge_0	237.34	211.71
Northridge_90	283.39	240.71
Parkfield_0	130.06	102.67
Parkfield_90	125.67	112.12
Average	**271.73**	**233.04**

The case study building 1 (irregular structure) has 7 columns in moderate damage region and 45% of the first storey beams and 39% of second storey beams are in the moderate damage region as well. On the other hand there are not any beams and columns in the moderate damage region at the case study building 2 (uniform framed structure). So, case study building 1 has Life Safety performance while case study building 2 has Immediate Occupancy performance according to the TER 2007 definitions.

3.5. Grid Discontinuity Factor Case Study

In order to demonstrate the effects of grid discontinuity on the building stiffness, 5 different structures are modeled and numerically investigated. The floor plans of the models related to this case study are shown in Figures 3.6, 3.7, 3.8, 3.9 and 3.10 and as it could be observed in the figures all models have same dimensions (16m x 20 m). The dimensions of beams (30x50 cm) and columns (40x40 cm) are selected the same for all models in order to investigate the effects of grid discontinuity only. These structures have various grid discontinuity factors (GDF) and they are listed in Table 3.11.

The modal analysis of the investigated structures shows that there is a strong relation with the fundamental period of the structure and the grid discontinuity factor. It is observed that the period increases with the increase of the so-called factor. The relationship between grid discontinuity factor and the fundamental period is shown in graph in Figure 3.11. As it is seen in graph the grid discontinuity factor plays an important role regarding the stiffness characteristics of the buildings and leads to increase on the fundamental period of the structure.

Table 3.11 Grid Discontinuity Factors of Structures

	Grid Discontinuity Factor	Fundamental Period of the Structure
Structure_A	0	0.49
Structure_B	0.2	0.55
Structure_C	0.4	0.58
Structure_D	0.68	0.64
Structure_E	0.84	0.72

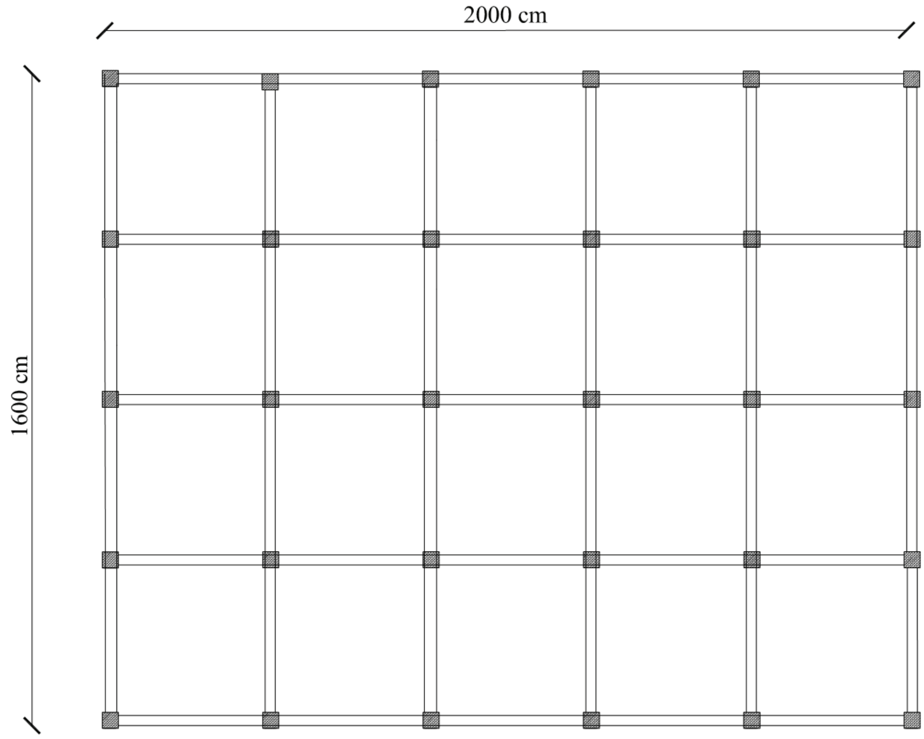

Figure 3.6. Floor plan of structure A (T=0.49 sec, GDF=0)

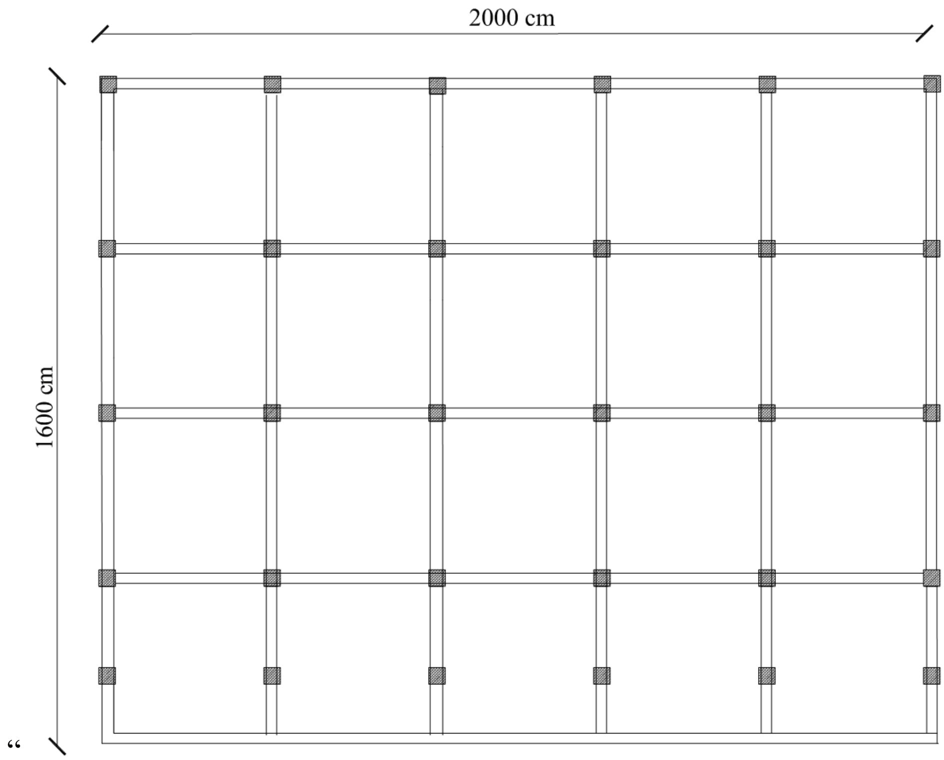

Figure 3.7. Floor plan of structure B (T=0.55 sec, GDF=0.2)

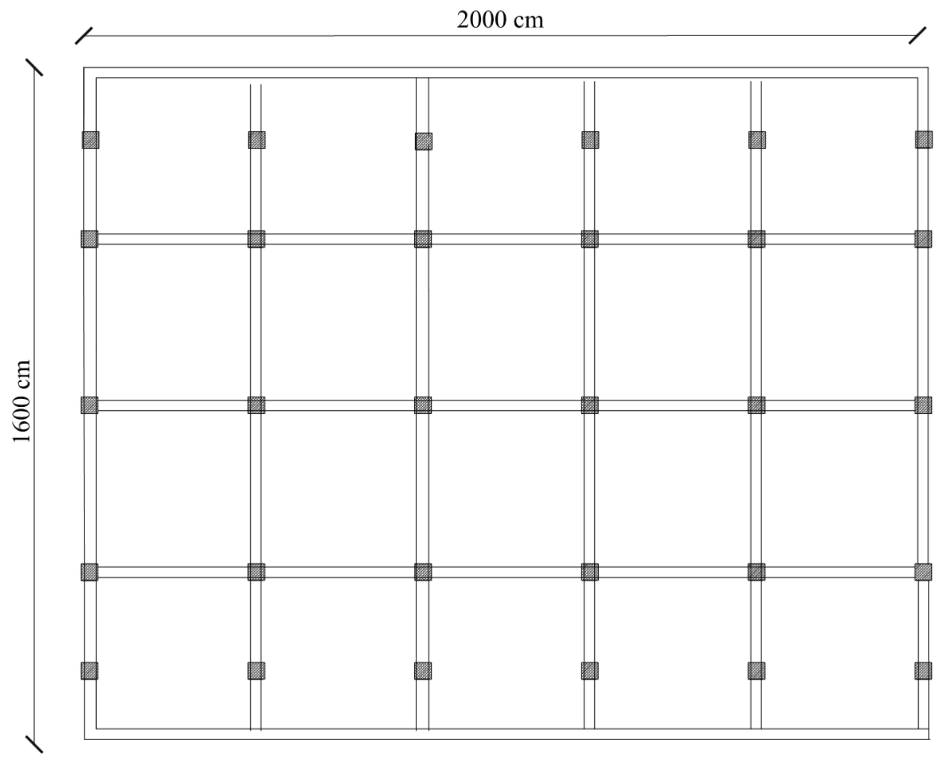

Figure 3.8. Floor plan of structure C (T=0.58 sec, GDF=0.4)

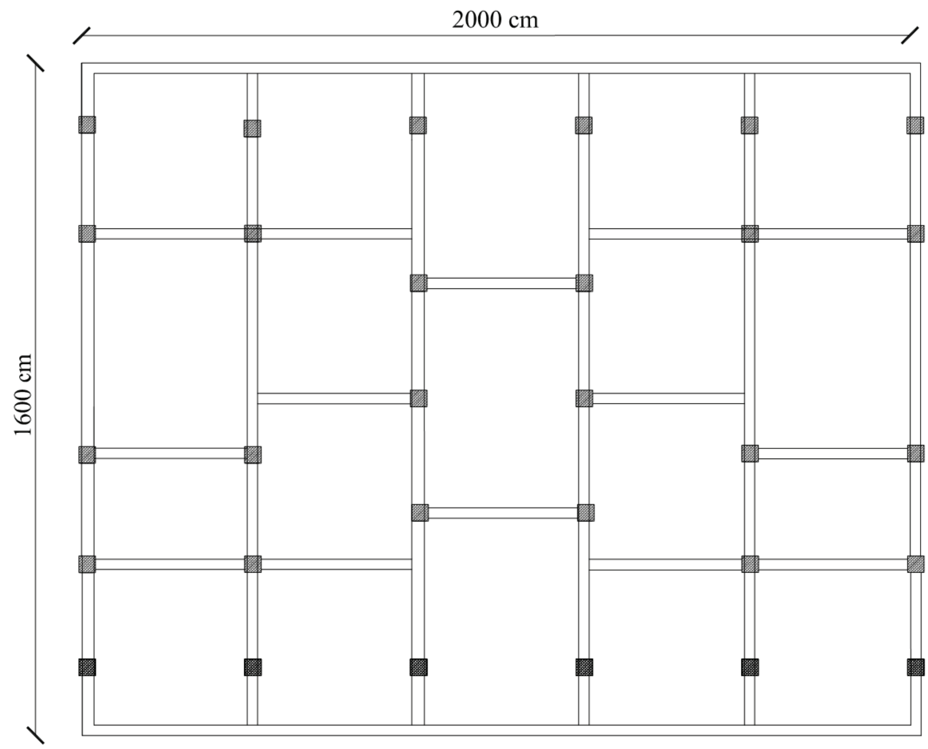

Figure 3.9. Floor plan of structure D (T=0.64 sec, GDF=0.64)

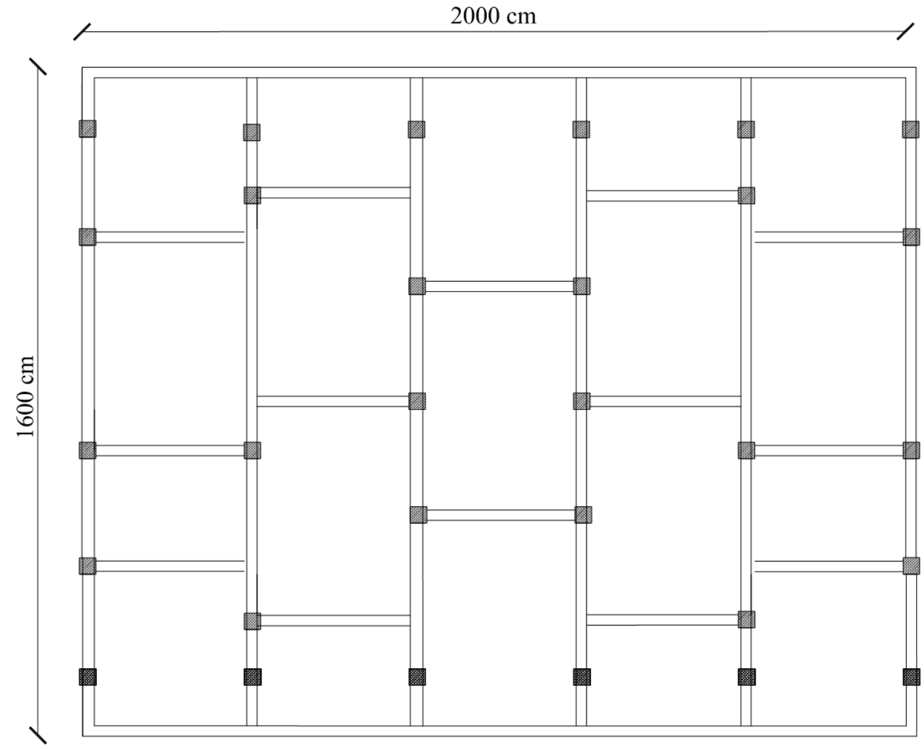

Figure 3.10. Floor plan of structure E (T=0.72 sec, GDF=0.84)

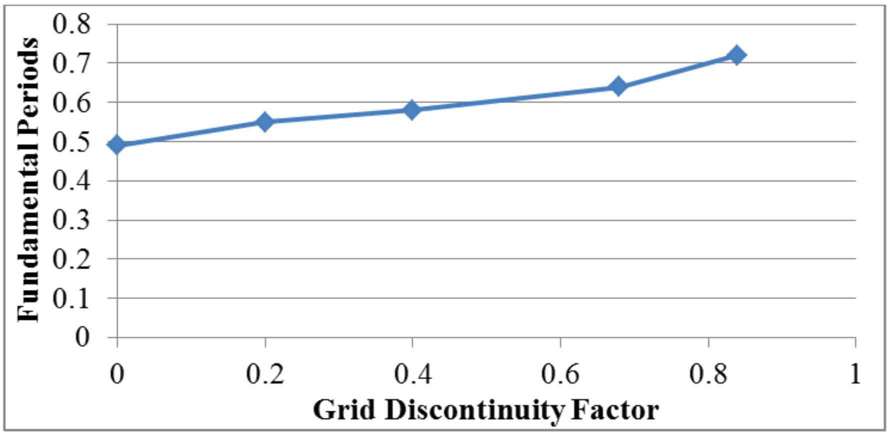

Figure 3.11. Fundamental period vs. grid discontinuity factor

CHAPTER 4

DISCUSSION OF RESULTS AND CONCLUSION

In this thesis, a performance evaluation study using nonlinear time-history analysis is carried out in accordance with TER 2007 in order to investigate the earthquake response of the reinforced concrete (RC) buildings having grid discontinuity irregularity in floor plan. However, TER 2018 recommendations are considered for the selecting and scaling of the ground motions. Eleven pairs of earthquake are taken into consideration for the seismic analyses of the buildings and the earthquakes are applied in both directions at the same time.

The grid irregularities in plan are a common defect for the structures in the Turkish building stock. It is partly due to architectural concerns but also due to lack of seismic knowledge both in the architects and the engineers. One of the main features in these buildings is the discontinuity of the beams lines. Although there are observations that some of the structures having discontinuous beams perform insufficiently in the seismic actions, there are no existing limitations or recommendations about it in the both 2007 and 2018 Turkish Earthquake Regulations (TER 2007, TER 2018).

The grid discontinuity observed to be decrease the stiffness of the structures. Considering the effect of the decrease in stiffness, it is expected to have an increase in the period of the structures. The studied buildings verified this condition. Based on the "equivalent displacement rule" the increase in period expected to cause an increase in the deformation demands of the structure. In Chapter 3, it is shown that the uncracked period of the studied structures varies maximum about 47%. The fundamental period of the structure is closely related to the grid discontinuity factor and it increases with the increase of the so-called factor.

Two case study buildings are investigated as irregular and uniform framed structures. The irregular structure's period that is calculated on effective stiffness have an increase of 13% as compared with the uniform framed structure. The performed time-history analysis results show that the displacement demand increase 17%. Roof drift at the center of mass increases from 233 to 272 mm.

The case study building 1 (irregular structure) has 7 columns in moderate damage region and 45% of the first storey beams and 39% of second storey beams are in the moderate damage region as well. On the other hand there are not any beams and columns in the moderate damage region at the case study building 2 (uniform framed structure). So, case study building 1 has Life Safety performance while case study building 2 has Immediate Occupancy performance according to the Turkish Earthquake Regulation (TER 2007) definitions.

As it is seen from two case studies, the variation of grid discontinuity factor effects the stiffness characteristics of the structure and plays an important role for the earthquake behavior of the structures. Especially, a distinct change in the performance point is observed although there is a small variation between periods of two case study buildings. Further study is needed to determine the effects of larger period variations.

In the consequence of evaluations based on member damage level, the buildings with grid discontinuities are more vulnerable to earthquakes than those without grid discontinuity irregularities. Further study is needed to define a procedure to mitigate the vulnerability created by the horizontal grid discontinuity.

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