RESPONSE OF ASYMMETRIC ISOLATED BUILDINGS UNDER BI-DIRECTIONAL EXCITATIONS OF NEAR-FAULT GROUND MOTIONS

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RESPONSE OF ASYMMETRIC ISOLATED BUILDINGS UNDER BI-DIRECTIONAL EXCITATIONS OF NEAR-FAULT GROUND MOTIONS

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ABSTRACT

RESPONSE OF ASYMMETRIC ISOLATED BUILDINGS UNDER BI-DIRECTIONAL EXCITATIONS OF NEAR-FAULT GROUND MOTIONS

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Isolator displacements, floor accelerations, roof displacements, base shear and torsional moments are basic parameters that are considered in the design of seismically isolated structures. The aim of this study is to evaluate the effects of bi-directional earthquake excitations of near fault records on the response of base isolated structures in terms of basic parameters mentioned above. These parameters computed from nonlinear response history analysis (RHA) and they are compared with the parameters computed from equivalent lateral force procedure (ELF). Effect of asymmetry in superstructure is also examined considering mass eccentricity at each floor level. Torsional amplifications in isolator displacements, floor accelerations, roof displacements and base shear are compared for different level of eccentricities. Two buildings with different story heights are used in the analyses.

The building systems are modeled in structural analysis program SAP2000. The scaling of ground motion data are taken from the study of “Response of Isolated Structures Under Bi-directional Excitations of Near-fault ground Motions” (Ozdemir, 2010). Each ground motion set (fault normal and fault parallel) are applied simultaneously for different range of effective damping of lead rubber bearing (LRB) and for different isolation periods.

Keywords: Seismic isolation, nonlinear response history analysis (RHA), near-fault ground motion, bi-directional excitation, lead rubber bearing.

ÖZ

SİSMİK İZOLASYON UYGULANAN ASİMETRİK YAPILARIN YAKIN KAYNAKLI VE ÇİFT DOĞRULTULU DEPREM HAREKETLERİ ALTINDAKİ DAVRANIŞLARININ İNCELENMESİ

FİTOZ, Hatice Eda
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Prof. Dr. Uğurhan AKYÜZ

Şubat 2012, 72 sayfa

İzolatör deplasmanları, kat ivmeleri, çatı ötelenmeleri,taban kesme kuvvetleri ve burulma momentleri sismik izolatörlü yapıların tasarlanmasında göze alınan önemli parametrelerdir. Bu çalışmanın amacı, sismik izolasyon uygulanan yapıların yakın kaynaklı ve her iki doğrultulu deprem hareketleri altındaki davranışlarının yukarıda bahsedilen parametreler bazında incelenmesidir. Doğrusal olmayan dinamik analizlerden elde edilen bu parametreler eşdeğer yanal kuvvet yöntemi yardımıyla hesaplanan değerler ile karşılaştırılmıştır. Üst yapıdaki düzensizliğin kıyaslanan değerler üzerindeki etkisi, her kat seviyesinde kütle eksenelliğinin olduğu kabul edilerek verilmiştir. Analizlerde farklı kütle eksenelliği değerleri incelenmiştir. Farklı kütle eksenellik değerleri için bulunan izolatör deplasmanlarında,kat ivmelerinde, çatı ötelenmelerinde ve taban kesme kuvvetlerinde oluşacak burulma etkisi de incelenmiştir. Analizlerde kat yüksekliği farklı iki betonarme bina, SAP2000 adlı yapısal analiz programı kullanılarak modellenmiştir. Bu tezde kullanılan deprem ölçeklendirme değerleri "Response of Isolated Structures Under Bi-directional Excitations of Near-fault ground Motions" (Ozdemir, 2010) adlı çalışmadan alınmıştır. Elde edilen sonuçlar farklı etkili sönümlenme değerleri ve izolasyon periyotları için incelenmiştir.

Anahtar Kelimeler: Sismik izolasyon, doğrusal olmayan dinamik analiz, yakın kaynaklı depremler, çift doğrultulu analiz, kurşun çekirdekli kauçuk yastık.

To my beautiful family and my mother…

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CHAPTER 1

INTRODUCTION

Reduction of earthquake damage is one of the big concerns of structural engineering philosophy. As a traditional seismic design approach, increasing strength or ductility of building systems could be a solution. In addition to classical approach, seismic isolation systems are introduced as a protection from earthquakes.

In base isolation, flexible and dissipative elements are placed at the interface between the foundation and the base of the structure. These isolators increase building flexibility and energy absorption capacity. The isolating system absorbs part of the earthquake energy before transferring it to the structure, by shifting the natural period of the isolated structure. This period shift results in a reduction in the inertial forces. As a result, the energy dissipated by the structural elements decreases (Agranovich and Ribakov, 2008).

Base isolation application has been increasing all over the world with the help of developing technologies. The analytical and experimental researches have also accelerated for many aspects of base isolation. However, in terms of torsional response of base-isolated structures, there are not many studies available in the literature (Tena-Colunga and Zambrana-Rojas, 2006).

1.1. Base Isolation Philosophy

In traditional seismic design, there are two different approaches to increase safety of buildings: increasing strength or increasing ductility of building systems. Increasing the strength is only possible with the reduction of ductility, which will cause an increase in floor acceleration. On the other hand, increasing the ductility causes an increase in vibrations.

Interstory drifts and floor accelerations are two primary causes of damage to the building systems. Interstory drift can reach high values in flexible structures, and high floor accelerations can be observed in stiff buildings Furthermore, floor accelerations generally increase through the height of buildings in a fixed base structure during an earthquake (Figure 1.1) (Mayes and Naeim, 2001). Isolating the structures from earthquakes by using seismic isolation systems lowers the floor accelerations (Figure 1.2).

The basic idea behind the seismic isolation is decoupling the structure from the earthquake excitations by placing dissipative and flexible elements at the isolation level. These elements reduce the transmission of seismic force from the ground to the superstructure by shifting the fundamental period of structure (Figure 1.3). This means that the inertial forces that affect the structure will be lower. However, the period shift should be in a reasonable limit since increasing period result in increasing displacement at isolation level. Damping of isolation system is the parameter that limits these excessive displacement (Figure 1.4) (Matsagar and Jangid, 2004; Constantinou et. al., 2007; Ozdemir 2010).

1.2. Types of Seismic Isolation Devices

There are two primary isolation systems: elastomeric bearings and friction sliding devices. For a proper seismic isolation, these two systems should have the following properties (Naeim and Kelly, 1999; Mayes and Naeim, 2001):

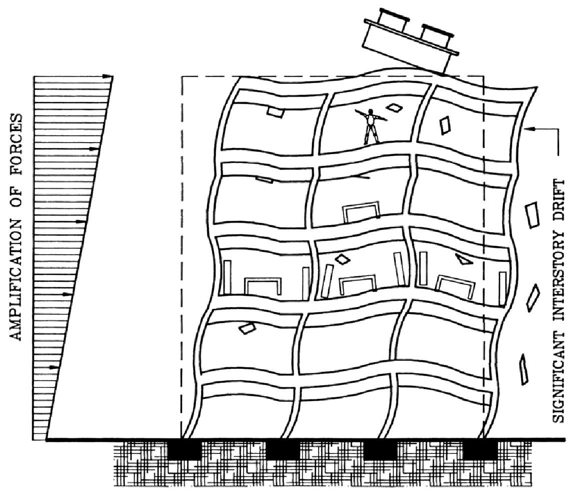

Figure 1.1 Conventional Structure (Mayes and Naeim, 2001).

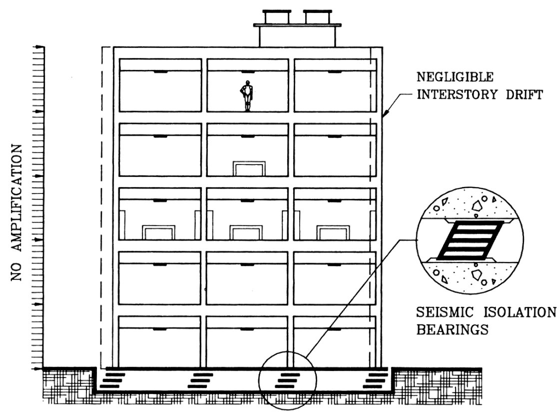

Figure 1.2 Base Isolated Structure (Mayes and Naeim, 2001)

- High lateral flexibility to increase the fundamental period sufficiently.
- Damping to prevent resonance in case of long period earthquakes.
- Rigidity for low lateral loads such as wind and minor earthquakes.
- Re-centring effect: to bring the structure its rest position.
- Resistant to tension if there is a risk of uplift.

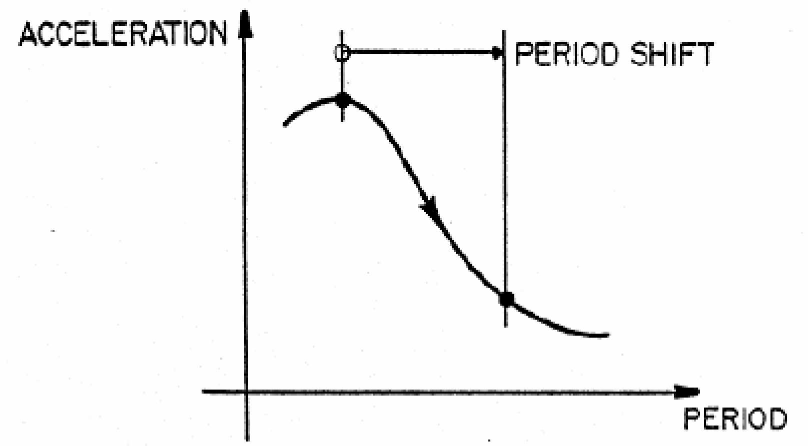

Figure 1.3 Effect of Period Shift in Isolated Structures on Accelerations (Constantinou et. al., 2007).

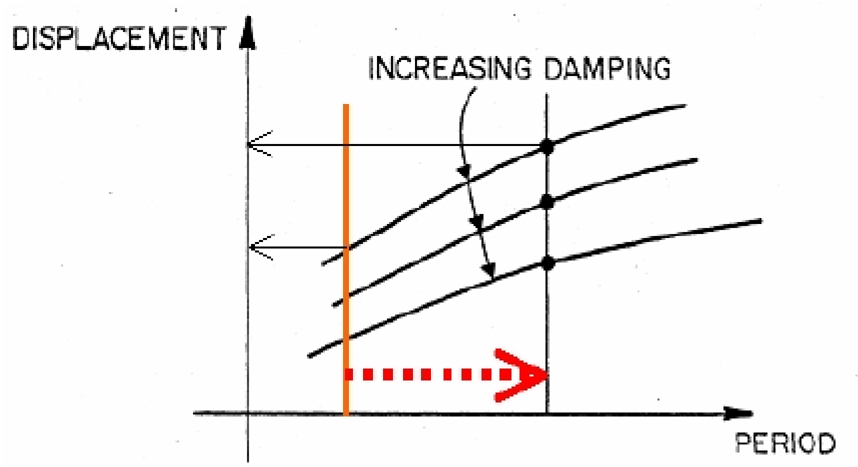

Figure 1.4. Effect of Period Shift in an Isolated Structure on Displacements (Constantinou et. al., 2007).

1.2.1. Elastomeric Bearings

Elastomeric bearings can be grouped into three groups.

i) Low Damping Natural and Synthetic Rubber Bearing (NRB)

ii) Lead Rubber Bearing (LRB)

iii) High Damping Rubber Bearing (HRB).

1.2.1.1 Low Damping Natural and Synthetic Rubber Bearing (NRB)

Two thick steel endplates, many thin steel shims and rubber plates are components of a low damping natural and synthetic rubber bearing (NRB) (Figure 1.5). High vertical stiffness are provided by steel shims. Moreover, these shims prevents excessive bulging of the rubber. On the other hand, elastomer control the horizontal stiffness of NRB. As a result, horizontal stiffness is relatively small. The behaviour of NRB system is approximately linear up to shear strains about 100%, with the damping range of 2–3% . In addition, the material is not subjected to creep, and long-term stability of the modulus is good. Another advantage of NRB is its manufacturing simplicity. It is easy to model. In addition, temperature, aging and history do not affect the mechanical response of these bearings. On the other hand, the disadvantage of NRBs is that they must be used with additional damping systems such as viscous dampers, steel bars, and frictional devices (Kelly and Naeim, 1999).

1.2.1.2 Lead Rubber Bearing (LRB)

A lead rubber bearing (LRB) is similar to low damping rubber bearing but it has lead core inserted into holes (Figure 1.6). This core increase the damping, control the displacements and supports weight of the structure (Kelly and Naeim, 1999). In this thesis, the selected isolator type is Lead Rubber Bearings. The mechanical properties of chosen LRB are given in Chapter 3.

1.2.1.3 High Damping Rubber Bearing (HDRB)

A HDRB is generally consisting of rubber layers and reinforcement steel shims. The HDRB bearing shows high stiffness at small strains (γ<20%). This tends to minimize the response to low-level earthquake and wind loads. Between the ranges of 20–120% shear strain, the stiffness is low. For larger strains, the stiffness increases. To sum up, a HDRB is stiff for small input, is nearly linear and highly flexible for design level and is able to restrict displacement over the design level (Kelly and Naeim, 1999).

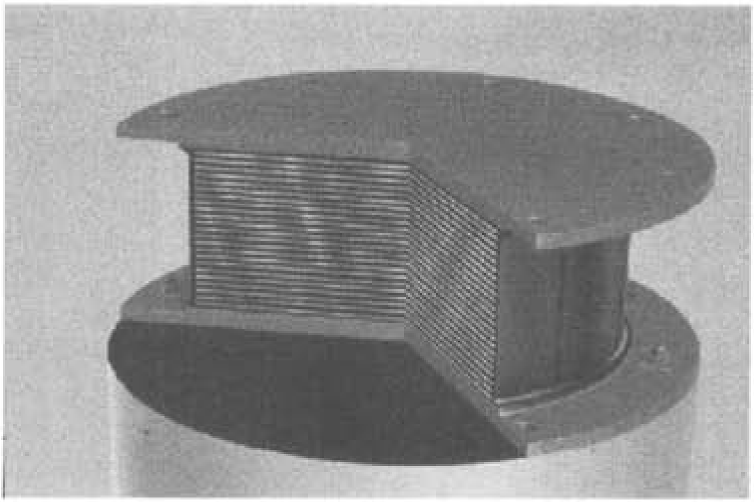

Figure 1.5 NRB (Kelly and Naeim, 1999).

A study on comparison between LRB and HDRB was done by Islam et al. (2011). The nonlinear response history and time history analyses were conducted and isolators were designed in accordance with UBC. The conclusion that was stated by authors was that HDRB was more effective in terms of base shears and isolator displacements. On the other hand, LRB was better at reducing floor accelerations that were decreasing the earthquake damage.

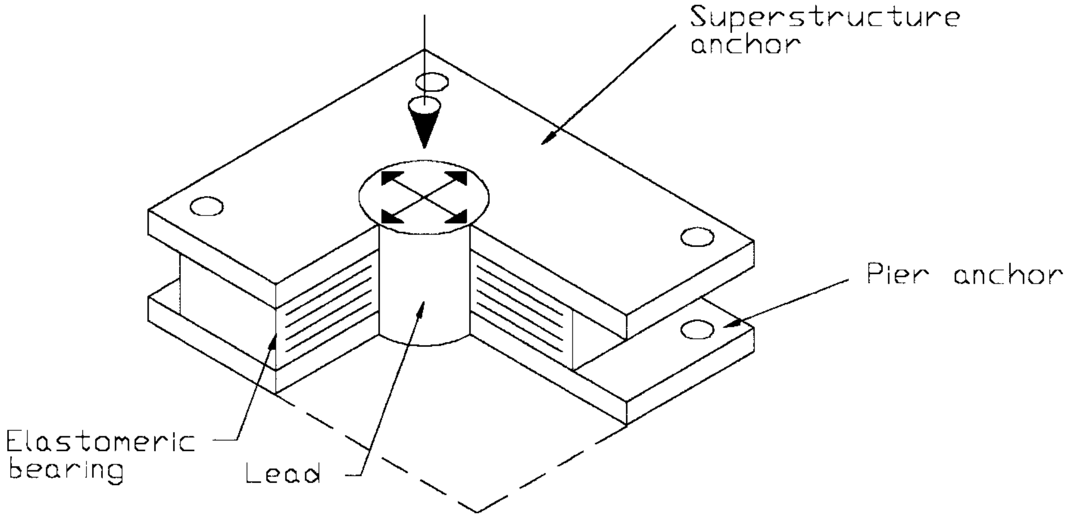

Figure 1.6 LRB (Kelly and Naeim, 1999).

1.2.2. Friction Sliding Devices

The second type of seismic isolators is sliding device. The basic idea behind sliding systems is limiting the transmission of shear force to the superstructure through friction. The sliding devices are effective for a wide range of frequency input. Since the frictional force is developed at the base, it is proportional to the mass of the structure and the center of mass and center of resistance of the sliding support coincides. Consequently, the torsional effects produced by the asymmetric building are diminished.

1.3. Development of Seismic Isolation and Recent Studies

In the late 1860's, David Stevenson, a Scottish engineer, had proposed a system to protect lighthouses from earthquake hazard. His systems, called "asymmetric joint", are composed of balls and cups. In the following years, about 1880, John Milne, referred as father of modern seismology, and the son of Stevenson, David A. Stevenson successfully, conducted the first test of seismic isolation (Tobriner, 2006).

One of the earliest examples of seismic isolation systems is proposed by Touaillon in 1870. His isolation system is composed of balls that are free to move and resting on brick structure (Figure 1.7). This free moving system protects superstructure from earthquake damage by decreasing the movement of superstructure. His idea was “In cities where buildings are built with very little space between them, the walls may be provided with springs or bumpers, made of India rubber or other suitable material to prevent injury or destruction from striking together (Tobriner, 2006).

In 1960, a medical doctor from England, J.A. Calantarients was also proposed a seismic isolation idea. He thought a system similar to Touaillon’s. However, his “free joint” is composed of fine sand, mica or talc. In this way, superstructure decouples from base (Kelly and Naeim, 1999).

After theoretical and experimental developments in base isolation systems, seismic isolation devices have been used in earthquake prone regions especially for important structures (schools, hospital etc.). At first, the rubber plates without steel sheets were used. As a result, lateral bulging was observed. With the developing technology, steel reinforcing plates with rubber layers have started to use in isolation devices. The usage of seismic isolation systems has become widespread with the development of multi layered elastomeric bearings (Kelly and Naeim, 1999).

Since seismic isolation is still a developing technology, there are many researches about the response of seismically isolated structures. (Jangid and Datta, 1995; Malhotra 1999; Matsagar and Jangid, 2004; Warn and Whittaker, 2004; Dicleli and Buddaram,2006; Huang et al., 2007). The studies has been focused on importance of selection and scaling of ground motion for both isolated and unisolated systems

(Naeim and Lew, 1995; Shome et al., 1998; Malhotra 1999; Chopra and Chintanapakdee, 2001; Stewart at al., 2001; Akkar and Gulkan, 2002) response of different isolation systems to ground excitation, discussion of simplified methods of codes in the prediction of seismic isolation systems (Kelly, 1999; Mayes and Naeim, 2001; Ramirez et al., 2002; Matsagar and Jangid, 2004; Pavlou and Constantinou, 2004; Warn and Whittaker, 2004; Guyader and Iwan, 2006). In this study, the emphasis will be given to the studies on the response of asymmetric isolated structures.

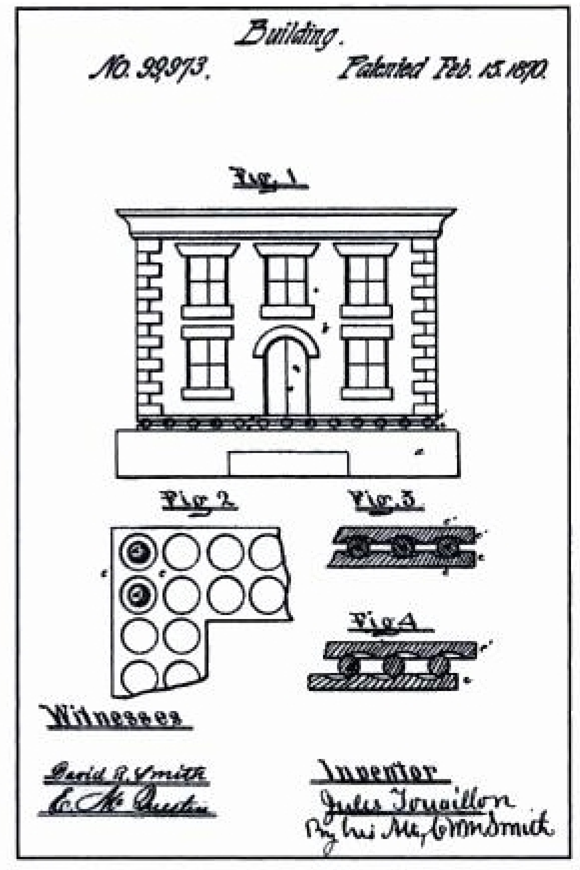

Figure 1.7 Touaillon’s proposed isolation system **(Tobriner, 2006).**

Torsional response of inelastic elastomeric isolation structure under bi-directional horizontal ground motion was investigated by Nagarajaiah et al. (1993a). The authors stated that although seismic isolation had decreasing effect on the response of buildings, asymmetry in superstructure had an important role on torsional response. Jangid and Datta (1995) investigated the displacement response of asymmetric one story seismically isolated structure. The results obtained from calculations were that the superstructure eccentricity did not have much effect on isolator displacements. Tena-Colunga and Zambrana-Rojas (2006) also investigated the nonlinear dynamic isolator displacements response of buildings .The authors stated that higher level of eccentricity results in higher displacement at the base level. Moreover, the authors concluded that increasing eccentricity has negative effects on design of seismic isolators.

Asymmetry can be managed by changing the center of mass or center of stiffness of buildings. In the study of Tena-Colunga and Escamilla-Cruz (2007), two different types of eccentricity were analyzed: mass and stiffness eccentricities. Firm soil bi-directional ground motion data was used in analysis. The conclusion remark of authors is that torsional amplifications were higher in buildings with mass eccentricity than building with stiffness eccentricity.

Kilar and Koren's study (2009) was about asymmetry effects of buildings on the seismic isolation. The eccentricity was given by changing the center of mass. The analyzed buildings were four story RC frame buildings. Selected bearing type was LRBs that were designed according to Eurocode 2 and 8. Nonlinear dynamic analyses were conducted under ten ground motion data in SAP 2000. Three different scaling types were applied. The authors investigated the effects of distribution of bearings on the displacement and rotational response of superstructure. In addition to that the proper distribution of bearings in plan was investigated to diminish the torsional amplification due to mass eccentricity. The authors concluded that although most of building codes suggested superposing the center of mass with center of isolators to decrease torsional response, this bearing implementation was not protective for superstructure. They stated that top displacements could be increased up to nearly 2 times when it is compared to symmetrical buildings.

Khante and Laukesh (2010) conducted another study. Their study was based on the effect of mass eccentricities of seismically isolated structure. 13-story concrete building was analyzed in ETABS. The selected levels of eccentricities were 5%, 10%, 15% and 20% of larger plan dimensions. The response spectrum and time history analyses were conducted in both uni-axial and bi-axial directions. The fixed base buildings were also analyzed and comparisons were done. The authors emphasized that no matter how high the eccentricity is, the response of seismically isolated structure is reduced in terms of maximum shear force, torsion, bending moment, lateral displacement, story drift, story acceleration and base shear when comparing with fixed base buildings.

Etadili and Sohrabi (2011) compared the response of seismic isolated asymmetric buildings with fixed base ones. Nonlinear time history analyses were conducted and three real ground motion records were selected in analyses (El Centro (1940), Tabas (1978), and Bam (2003). These records were scaled to the maximum acceleration of 0.4 g. 3 story and eight story steel structures supported by lead rubber bearings were analyzed in SAP 2000. The selected level of eccentricities were chosen randomly as 10%, 15% and 20% and T=1.5, T=2, T=2.5 and T=3.0 sec were selected as isolation periods. The authors stated that the seismically isolated asymmetric buildings were less prone to the destructive effect of torsion with respect to fixed base buildings. However, in large eccentricities the effectiveness of isolation on torsion disappeared. Moreover, isolator displacements became higher when level of eccentricity was higher. The increasing isolation period had also same effect on isolator displacement.

1.4. Scope of Study

This study has been conducted to evaluate the effects of bi-directional earthquake excitations of near fault records on the torsional response of asymmetric base isolated structures. Nonlinear response history analysis (RHA) is carried out for 11 different earthquakes recorded at soft soil. The ground motion data are taken from the study of Ozdemir (2010). Effects of asymmetry in superstructure are examined considering mass eccentricity at each floor level for the percentage of 5, 10 and 15 of the longest plan dimensions. Torsional amplifications in base shears, torsional moments, isolator displacements, floor displacements and accelerations are compared for different level of eccentricities. Amplifications due to bi-directional excitations and asymmetry in superstructure are also investigated. The isolation response of buildings that were calculated from Nonlinear RHA is compared with the isolator displacements calculated from simplified method. The structures are modeled as a 3-D three-story (3-S) and seven-story (7-S) buildings with concrete columns, beams and concrete slabs resting on the base isolation system. Lead-rubber bearings (LRBs) are chosen in analyses and the variables that are examined in this thesis are the effective damping of LRB and isolation periods.

CHAPTER 2

TYPES OF ANALYSIS FOR ISOLATON SYSTEM

2.1. Introduction

Although there are different codes for the design of seismically isolated structures, the main philosophy is the same:

The current codes define two levels of seismic hazards:

– The Design Basis Earthquake (DBE, 475 years period earthquakes)
– The Maximum Credible Earthquake (MCE, 2500 years period earthquakes) (Mayes and Naeim, 2001).

Two different procedures are permitted for the analysis of seismically isolated structures:
-Static Analysis
-Dynamic Analysis

2.2. Static Analysis

In general, a static analysis is necessary for all seismic isolation systems since this analysis gives a minimum level for design forces and design displacements. Furthermore, when dynamic analysis is required, it is also useful both for preliminary design of the isolation system and the superstructure. However, for design review, under certain circumstances it may be the only design method used (Kelly and Naeim, 1999). The static analyses are also carried out in this thesis when choosing the properties of bearings. The details are given in Chapter 4.

2.3. Dynamic Analysis

Contrary to static analysis, dynamic analysis can be used in all cases. Besides, if static analysis is not sufficient to represent the response of system, dynamic analysis should be used. The response spectrum and time history analysis may be the form of dynamic analysis (Mayes and Naeim, 2001). Site-specific spectra are required in the following cases:

- The isolated structure is located on soft soil.
- The isolated structure is within 10 km of a known active fault.
- The isolated structure period (MCE) is greater than 3 sec.

However, dynamic analysis is still needed if the effective period of the isolated structure for DBE (T_D) is greater than three times the elastic fixed-base period of the structure (Kelly and Naeim, 1999). All of the three parameters listed above are appropriate with the concept of this study, as a result nonlinear response history analysis (RHA) with the proper ground motion records is chosen as the analysis type.

Suitable acceleration records are needed to carry out nonlinear dynamic analyses. The selected earthquake ground motions should be consistent with site, soil, source characteristics and durations with DBE and MCE (Kelly and Naeim, 1999). In addition, if real ground motion data is not available, suitable simulated time histories could be used to constitute the required number of ground motion (Mayes and Naeim, 2001). Real acceleration time histories are used in this study.

2.3.1. Selection and Scaling of Ground Motions

Since the ground motion databases are widespread and available, the characteristics of the ground motion records can be obtained easily. When these records are examined, it is seen that there is a distinct difference between the ground motion data recorded at near-fault and far-fault regions. Near-fault records of major earthquakes have large displacement pulses from 0.5 m to more than 1.5 m with peak velocities of 0.5 m/sec or higher. Moreover, although acceleration time histories are generally smooth, in some cases they have also large pulses in near-fault records (Makris and Chang, 2000). Malhotra (1999) also stated that the ground motions recorded in near-fault regions have remarkable pulses in acceleration, velocity and displacement time histories.

In Figure 2.1(a), acceleration, velocity and displacement histories of Northridge, California, earthquake of 17 January 1994 are given. This data is recorded at a near-fault region and these records are belonging to fault normal component. There is a long period pulse in the acceleration time history that is consistent with velocity and displacement histories. On the other hand, the ground motion data recorded at Kern County, California, earthquake of 21 July 1952 is given in Figure 2.1(b). These records are far-fault region records and there is not such a distinct pulse in time histories. Due to the different characteristics of near-fault ground motion, the interest on the near-fault effect on structure response has increased (Chintanapakdee and Chopra, 2001). In this thesis, a set of near-fault ground motion records are used to examine near-fault effect.

Suitable acceleration records are needed to carry out non-linear dynamic analyses. These records should be compatible with predefined earthquake generally obtained from a probabilistic seismic hazard analysis. The time histories used in nonlinear RHA analyses have different effects on the response of structure. It is an important issue to obtain the expected inelastic response of a structure in earthquake engineering. As a result, selection and scaling of acceleration histories of ground motions are also crucial to have an idea about the real response of a structure under earthquake excitations. There are some parameters such as magnitude of earthquake, source-to-site distance and site classification when considering scaling and selection. (Hancock et al., 2008). The number of records used in analysis is also an important issue. The codes recommended that at least three pairs of horizontal ground motion recorded events should be used in time history analysis. If seven or more records are used then the average of the results could be used in design. On the other hand, if three records are chosen in design, then the maximum response of these three records should be used. Besides the codes' recommendations, there are some researches about the required number of ground motions. Shome et al. (1998) reported that in case of proper scaling of records, the number of ground motion records use in analyses could be decreased by a factor of four. In addition, the study of Bommer and Acevedo (2004) also stated that there is a possibility of reduction in the number of motions in case of reasonable selection and scaling.

In this thesis, the selection and scaling procedure of ground motion used in analyses are taken from the previous researches. The selection information is taken from previous studies carried out by Somerville et al. (1997), Akkar and Gulkan (2002), Pavlou and Constantinou (2004), Metin (2006) and Ozdemir (2010). The ground motion records used in analyses are listed in Table 2.1. The moment magnitude M_W, the closest distance to the fault rupture d, the peak ground acceleration (PGA), peak ground velocity (PGV) and the peak ground displacement (PGD) are also presented. The selected ground motions are recorded at soft soil.

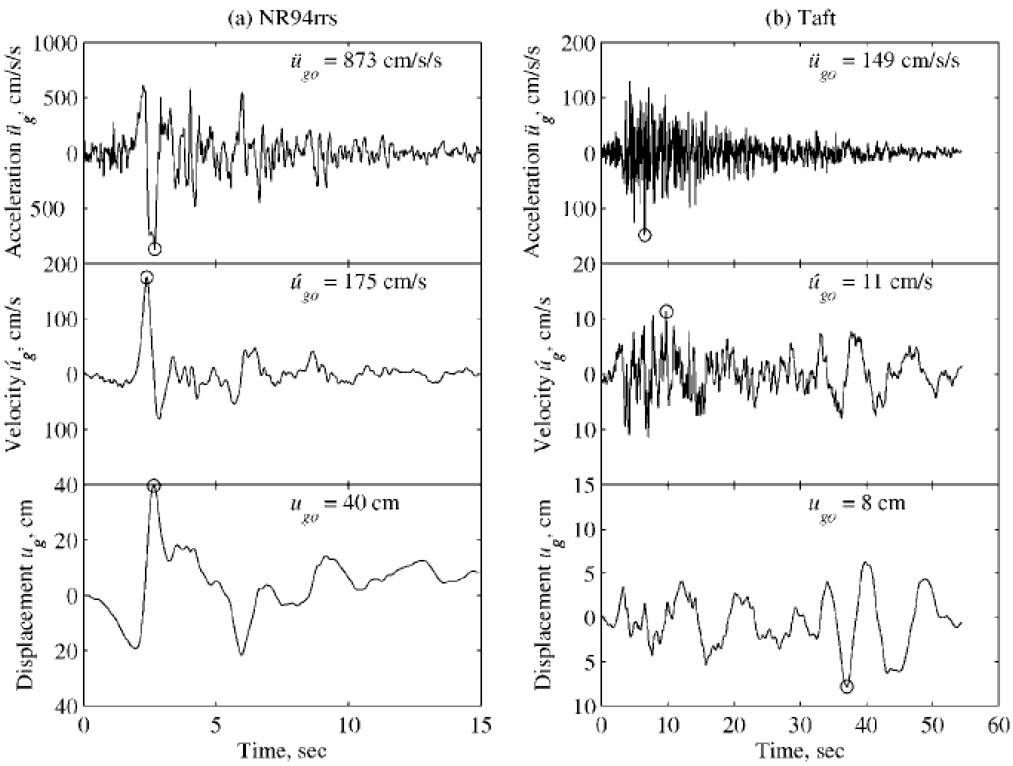

Figure 2.1 Ground motion records of fault normal component taken from (a) Rinaldi Receiving Station, 1994 Northridge earthquake and (b) Taft, 1952 Kern County earthquake. (Chintanapakdee and Chopra, 2001).

Selection of scaling procedure is important issue in the response of isolated buildings since it can result in overestimation or underestimation for nonlinear RHA. The important parameters for scaling procedure are summarized as follows (Ozdemir,2010):

- The period of structure should have no effect on scaling.
- A range of period should be considered instead of one single period.
- In order to determine seismic demands of the structures, the number of selected ground motions should be chosen as small as possible.
- It should also be applicable for both far field and near-field records.
- The distribution in the earthquake shaking for the selected characterization of the hazard should be preserved for the site of interest.
- Bidirectional effect of the records should be considered.
- Scale factors should be less than four.

The scaling procedure is also adopted from a previous study carried out by Constantinou and Ozdemir (2010). Their suggested scaling procedure is conducted in two phase. In first stage, the selected ground motions should be made compatible with the target spectrum that is called geo-mean scaling. This method is based on minimizing sum of the weighted squared errors between the geometric mean of the two horizontal components and the target spectral values at various periods by amplitude scaling. The geometric mean of the acceleration spectra of the two horizontal ground motion components is calculating and mean value of these spectrums correspond to each ground motion pairs is calculated. On the other hand, target spectrum values are taken from Turkish Earthquake Code (TEC) (2007) for the soft soil conditions. The corresponding spectral ordinates values for MCE are selected to be 1.5 times the ordinates of DE (design earthquake) spectra (ASCE, 2005; TEC, 2007). The soft soil definition is given by choosing spectrum characteristic periods T_A = 0.20 sec, and T_B = 0.6 sec. Target spectral accelerations are calculated by;

$$A(T) = \mathrm{AoIS}(T) \qquad \textbf{\textit{(2.1)}}$$

that is in accordance with TEC (2007). The selected values are :

Ao (effective ground acceleration coefficient) = 0.4
I (building importance factor) = 1.0
S(T) (spectrum coefficient) = 2.5 (for $T_A \leq T \leq T_B$)
A(T) is the spectral acceleration coefficient corresponding to 5% damped design spectrum normalized by the acceleration gravity. It is obtained as 1.5g between the spectrum characteristic periods for MCE. (Constantinou and Ozdemir, 2010).

The comparison of the target MCE spectra with the mean SRSS of the spectral components of the scaled ground motions are concerned after the first phase of scaling.

On the other hand, the requirements of ASCE (2005) should be checked for dynamic analysis in second part. Scaling of each pair of ground motions was concerned such that the average of the SRSS spectra from all ground motion pairs does not fall below 1.3 times the corresponding ordinate of the target response spectrum by more than 10% for each period between $0.5T_D$ and $1.25T_M$. T_D and T_M are the effective periods in the DE and the MCE, respectively. (Constantinou and Ozdemir, 2010).

The product of two scaling factor corresponding to each scaling stage is the final scale factor. These scaling factors are represented in Table 2.2.

Table 2.1 Characteristics of - near fault ground motions recorded at soft soil.

Earthquake	Station	Year	Magnitude (Mw)	d (km)	Component	PGA (g)	PGV (cm/sec)	PGD (cm)
Chi Chi (CC101)	TCU101	1999	7.6	2.1	N W	0.25 0.20	49.4 67.9	35.1 75.4
Erzincan (EE)	Erzincan	1992	6.7	4.4	NS EW	0.52 0.50	83.9 64.3	27.4 22.8
Imperial Valley (IVA4)	Array 4	1979	6.5	7.1	140 230	0.49 0.36	37.4 76.6	20.2 59.0
Imperial Valley (IVA5)	Array 5	1980	6.5	4.0	140 230	0.52 0.38	46.9 90.5	35.4 63.0
Imperial Valley (IVA6)	Array 6	1981	6.5	1.4	140 230	0.41 0.44	64.9 109.8	27.7 65.9
Imperial Valley (IVA10)	Array 10	1982	6.5	6.2	50 320	0.17 0.22	47.5 41.0	31.1 19.4
Kocaeli (KD)	Duzce	1999	7.5	15.4	180 270	0.31 0.36	58.8 46.4	44.1 17.6
Kocaeli (KY)	Yarimca	1999	7.5	4.8	60 330	0.27 0.35	65.7 62.1	57.0 51.0
Loma Prieta (LPCor)	Corralitos	1989	6.9	3.9	0 90	0.64 0.48	55.2 45.2	10.9 11.4
Loma Prieta (LPSar)	Saratoga	1990	6.9	8.5	0 90	0.51 0.32	41.2 42.6	16.2 27.5
Parkfield (PC)	Cholame 2	2004	6.0	14.3	90 360	0.60 0.37	63.3 44.1	14.1 8.9

Table 2.2 Scale factor of near-fault ground motions for soft soil records.

Ground Motion	Scale Factor (MCE)
CC101	2.43
EE	1.24
IVA4	1.75
IVA5	1.48
IVA6	1.24
IVA10	2.7
KD	1.74
KY	1.39
LPCor	2.2
LPSar	2.41
PC	1.73

CHAPTER 3

PROPERTIES OF ISOLATION SYSTEM AND SIMPLIFIED METHOD OF ANALYSIS

3.1. Properties of Isolation System – Mechanical Characteristics of Lead Rubber Bearings (LRBs)

In general, equivalent linear model is used for seismic isolation bearings when demand analysis is concerned. However, since more approximate estimation of bearing performance of isolation systems under maximum capable earthquakes (MCE) is needed, nonlinear demand analysis has been used extensively in recent years (Fenves et al, 2000).

In this study, the isolation system bearings are chosen as lead rubber bearings (LRBs). Bi-linear force–deformation relations can be used to model LRB systems without considering cycle-to-cycle deterioration. The idealized force–deformation relation of a LRB is shown in Figure 3.1 (Naeim and Kelly, 1999). Q is the characteristic strength, F_y is the yield force and D_y is the yield displacement. In addition, k_e and k_d are elastic and post-elastic stiffness of an isolator, respectively. Finally, D represents maximum lateral isolator displacement and F represents the maximum lateral force carried by the isolator. Actually, three of these parameters are sufficient to describe the model of LRB. Q, k_d and D_y are the initial parameters that describe the bearing characteristics.

Fixing the ratio of the initial stiffness to the post-yield stiffness is the recommended procedure in general (Kelly and Naeim, 1999). However, Makris and Chang (2000) stated that constant k_e/k_d value is inappropriate. In addition, Ryan and Chopra (2004) stated that fixing k_e/k_d value causes variations of yield deformation that is proportional to yield strength. However, it does not correctly represent the behavior of such systems. Recent studies' recommendation is to fix the yield deformation D_y instead of k_e/k_d value.

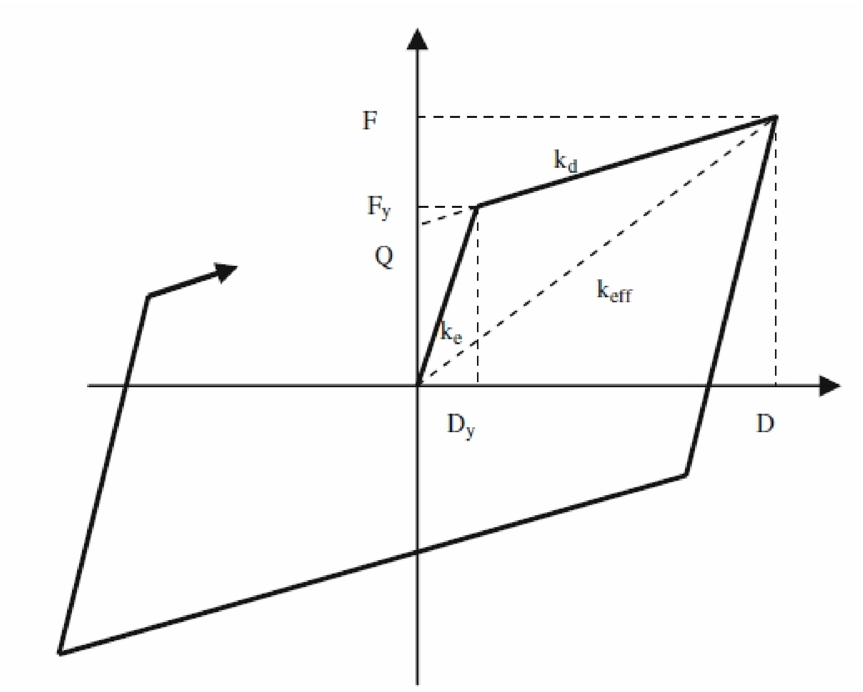

Figure 3.1 Bilinear forces – deformation relation of an isolator (Naeim and Kelly, 1999).

In the study of Constantinou and Ozdemir (2010), the yield displacement is taken as a constant value of 10 mm for bearings that is the indication of LRB systems rather than sliding systems. On the other hand, they also stated that the calculated isolation displacements and base shears are not affected significantly by the yield displacement. In this thesis, D_y is also taken as 10 mm.

The variable parameters used in this study are taken from the study of Ozdemir (2010) and represented in Table 3.1. These are the ratio of strength *Q* to weight *W* supported by isolators that is a measure of effective damping and the period *T* based on the post-elastic stiffness.

Table 3.1 Parameters for isolation systems considered.

Period, *T* (s)	3.5, 4.0, 4.5
Ratio strength to weight *Q*/*W*	0.08, 0.10, 0.12, 0.14
Yield displacement, Dy	10 mm

The selected parameters were chosen so that the base shear of each of these isolation systems does not exceed 30% of the weight of the structure in the maximum capable earthquake (MCE).

3.1.1. Equivalent Lateral Force Procedure (ELF)

Equivalent lateral force procedure (ELF) is defined in ASCE (2005). It is an iterative procedure and effective damping, effective stiffness and isolation period are used in calculations. Basic idea behind ELF is calculating the response of isolated buildings with higher effective damping values at isolation level by modifying 5% damped response spectrum (Ozdemir, 2010).

According to ELF, the effective stiffness k_{eff} and effective damping β_{eff} of a single-degree-of-freedom system are the representation of the isolated structure and given by:

$$k_{eff} = \frac{Q}{D} + k_d \qquad \textbf{(3.1)}$$

$$b_{eff} = \frac{4Q(D - D_y)}{2pk_{eff} D^2} \qquad \textbf{(3.2)}$$

As mentioned before, Q, kd and Dy are the selected initial parameters and the beginning of the ELF procedure start with the assumption of displacement D.

Afterward, the effective stiffness k_{eff} is calculated followed by the calculation of effective damping β_{eff} and the effective period T_{eff} as represented through Equations 3.1 - 3.3, respectively. The numerator in the calculation of β_{eff} is the area of hysteretic loop that is the representation of the energy dissipated at each cycle.

$$T_{eff} = 2p\sqrt{\frac{W}{k_{eff}\, g}} \qquad \textbf{(3.3)}$$

where; W = weight carried by isolator,
g = gravitational acceleration.

In the calculation of design displacement, damping reduction factor, B is also needed. Simplified elastic methods for increasing damping values can be used by modifying the 5% damped elastic response spectrum. Elastic spectrum for damping values greater than 5% can simply be obtained by dividing the 5% spectrum by factor *B*. In Table 3.2, values of damping reduction factor in codes and specifications are given in terms of $\beta_{eff.}$ The mentioned codes and specifications are AASHTO (American Association of State Highway and Transportation Officials, 1999), NEHRP (Building Seismic Safety Council, 2003), ASCE (American Society of Civil Engineers, 2005), Eurocode 8 (European Committee for Standardization, 2005) and recommendations in FEMA 440 (Applied Technology Council, 2005). In this study, B values are chosen from ASCE (2005).

Table 3.2 Damping reduction factor B in codes and specifications.

β_{eff} %	AASHTO, ASCE	NEHRP	FEMA 440	EUROCODE 8
≤ 2	0.8	0.8	0.8	0.8
5	1.0	1.0	1.0	1.0
10	1.2	1.2	1.2	1.2
20	1.5	1.5	1.5	1.6
30	1.7	1.7 or 1.8	1.8	1.9
40	1.9	2.1	2.1	2.1
50	2.0	2.4	2.4	2.4

Finally, the isolation system displacement is calculated from the response spectrum for period T_{eff} and damping β_{eff} . The design displacement is calculated as

$$D = \frac{gS_a T_{eff}^2}{4p^2 B} \qquad \textbf{\textit{(3.4)}}$$

where S_a is the spectral acceleration for the corresponding T_{eff}. The 5% damped response spectra adopted from TEC 2007 and illustrated in Figure 3.2.

As mentioned before, since the procedure is iterative, it ends when assumed displacement value is close enough to calculated one. The calculated displacement value corresponds to the uni-directional excitation and symmetrical systems. There should be adoption for bi-directional excitations for the calculation of design displacement, $D_{bi\text{-}dir.}$ shown as follows

$$D_{bi-dir} = D(\sqrt{(1^2 + 0.3^2)}) \qquad \textbf{\textit{(3.5)}}$$

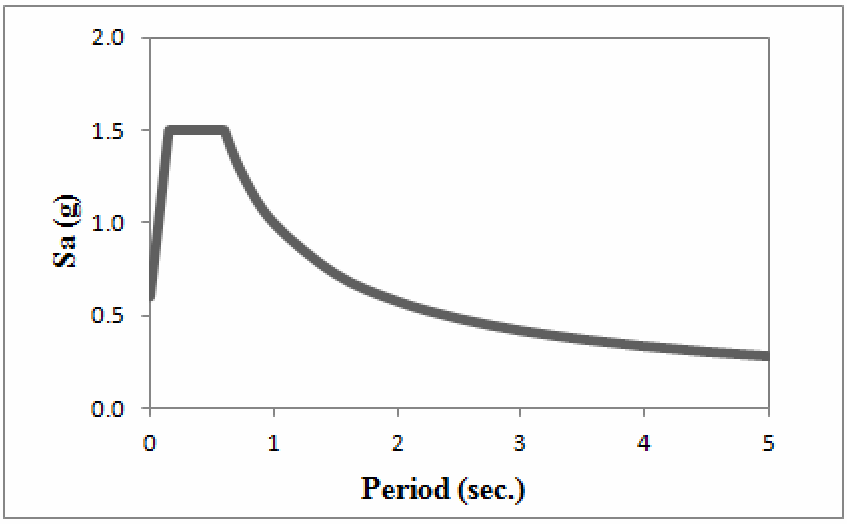

Figure 3.2 Adopted Response Spectra Used in Analysis (TEC(2007)).

$D_{bi\text{-}dir}$ is calculated by Eqn. (3.5) in accordance with 100%+30% rule as per ASCE (2005). This is the demonstration of 100% of the ground motion in critical direction and 30% of the ground motion on the other horizontal direction (Ozdemir, 2010).

Moreover, another adaptation should be done due to asymmetric systems considering the effect of eccentricity of superstructure on the isolator displacement. D_{ELF} is the representation of final design displacements and calculated as:

$$D_{ELF} = D_{bi-dir}(1 + y\frac{12e}{b^2 + d^2}) \qquad \textbf{\textit{(3.6)}}$$

where y is the distance from center of rigidity to a corner perpendicular to the critical direction, b and d are the plan dimensions of the system and e is the actual eccentricity plus 5% accidental eccentricity from the longest plan direction. If the eccentricity effect $(1 + y\frac{12e}{b^2 + d^2})$ is smaller than the value of 1.1, it must be taken as 1.1. Figure 3.3 represents the plan dimensions for calculations of D_{ELF}.

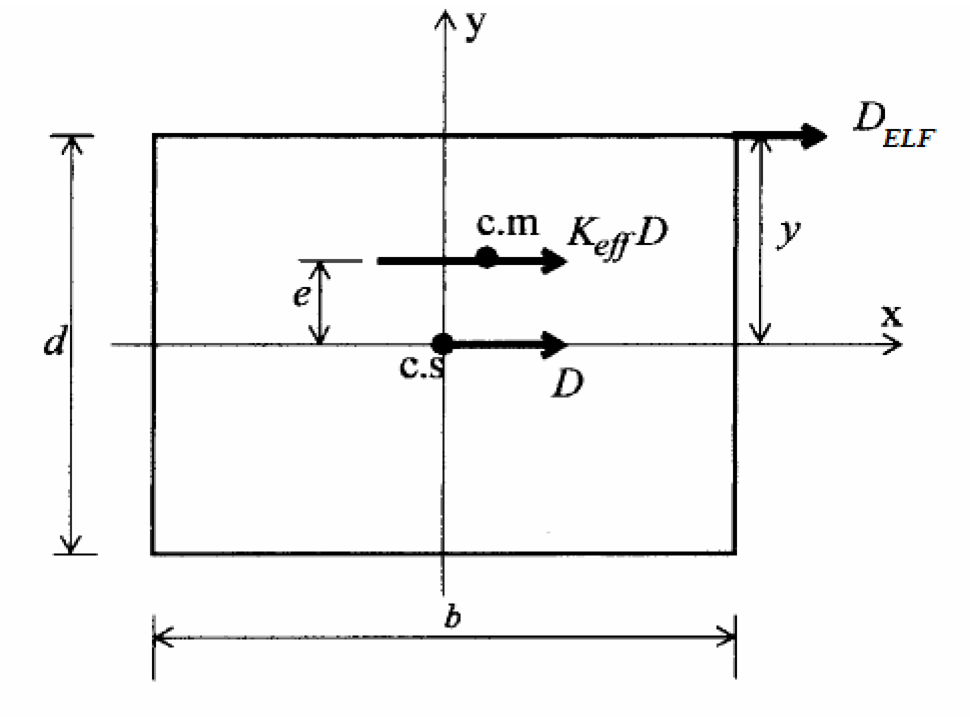

Figure 3.3 Plan dimensions for calculations of D_{ELF} (Naeim and Kelly, 1999).

3.2. Properties of Analyzed Structure

In this study, the structure is idealized as three-dimensional three-story (3-S) and seven story (7-S) buildings with concrete columns, beams and concrete slabs resting on the base isolation system. The plan dimensions of the RC buildings are 16m x 12m. Story heights of the structure are 3.0 m and equal for each story. The plan dimensions and isolation system joint numbering of building system are given in Figure 3.4 (a) and (b), respectively.

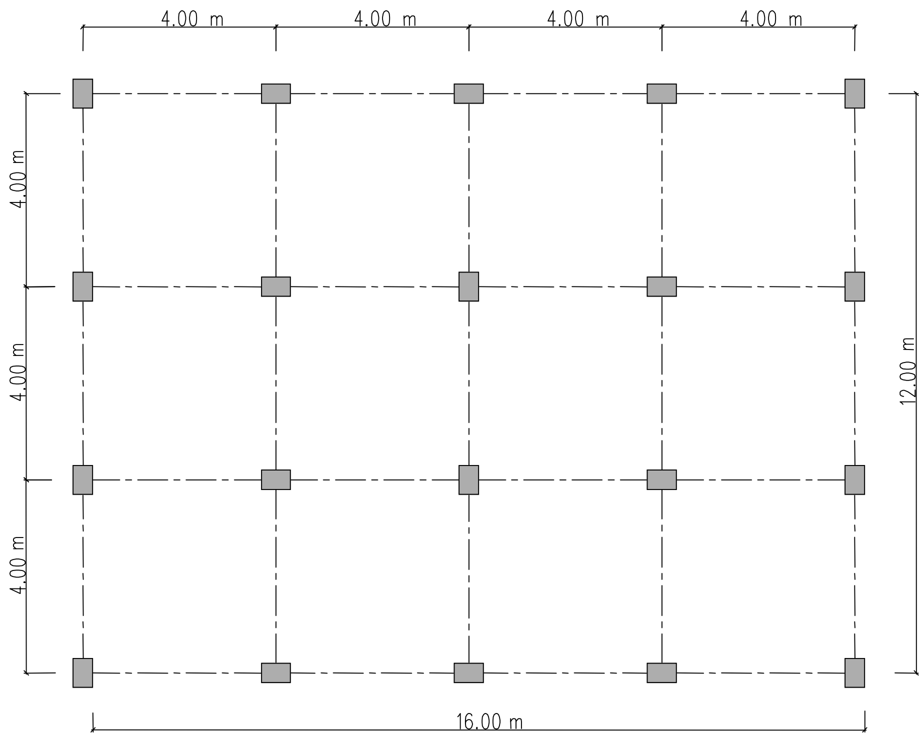

Figure 3.4 Idealized model of isolated RC building: plan

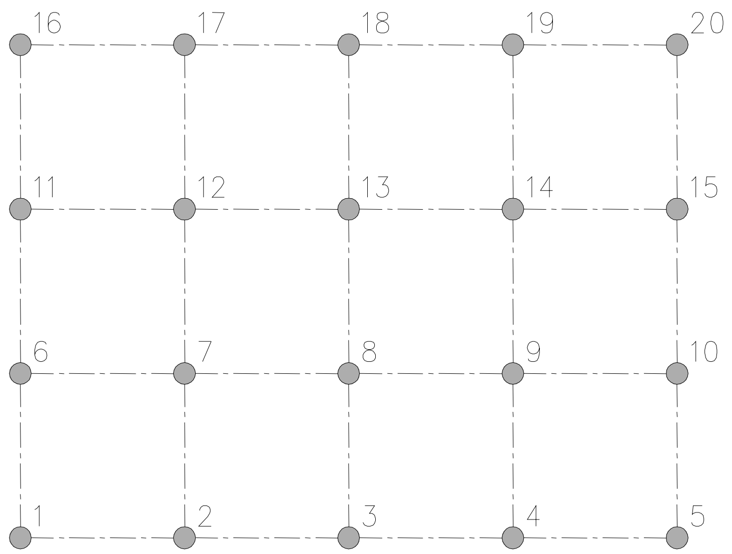

Figure 3.5 Idealized model of isolated RC building: isolation system joint numbering.

Although the system is symmetric and centric behavior is expected, mass eccentricities in superstructure are also taken into account in both of the horizontal directions (bi-directional eccentricity). The values of eccentricities are taken as 5%, 10% and 15% of plan dimensions. There are four different cases for location of these eccentricities as shown Figure 3.5. Since, cases 1 and 2 are same with cases 3 and 4, the evaluations and comparisons are done taking cases 1 and 2 into consideration.

Column dimensions are 40cm x 60cm, typical beam dimensions are 30cm x 50cm. The material properties used in analyses are summarized in Table 3.3. The distributed dead and live load values are 0.500 t/m^2 and 0.200 t/m^2, respectively. These values are consistent with the Turkish Earthquake Code (2008) and related Turkish standards. Finally, total weights of the structures are 3870 kN and 8170 kN for 3-S and 7-S buildings, respectively.

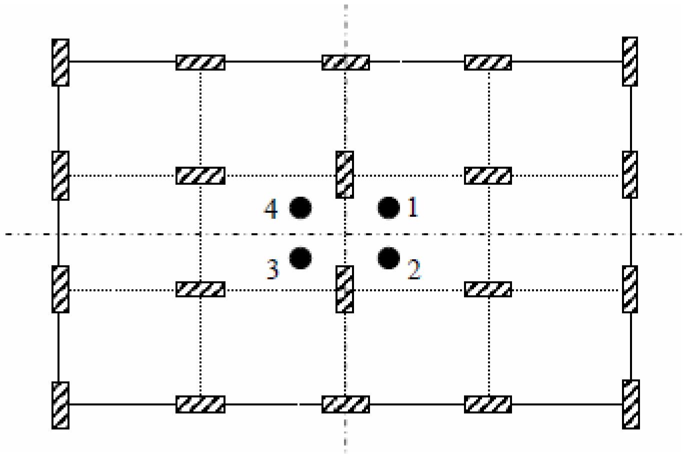

Figure 3.6 Eccentricity location of superstructure (Ozdemir, 2010).

Table 3.3 Mechanical properties of concrete material.

Modulus of elasticity, *E*	30000 MPa
Poisson ratio, ν	0.2
Compression strength, *fc*	25 MPa

The bearings were modeled as nonlinear link elements with bi-linear force-deformation relation available in SAP2000 (2008). The input data needed for utilizing the link elements properties are F_y, the ratio of post yield stiffness k_{eff} to initial stiffness k_e. Both of the RC buildings are supported by twenty isolators as illustrated in Figure 3.4 (b). With the isolator properties shown in Table 3.4 and 3.5, iterative analyses were conducted.

The analyses are conducted in SAP2000 (2008) to estimate the response of the isolated structures. The assumptions of modeling procedures are summarized as follows:

- All floors for all story level have three degree of freedoms: two translations and one rotation. This is due to their rigidity in their own plane.
- Superstructure frame members are taken as elastic.
- Soil-structure interaction effects on the response of structure are not taken into consideration.
- Although columns provide lateral stiffness, they are weightless and inextensible.
- Mass of a floor is equally distributed among the joints of that floor level.
- For the first three modes, the damping was specified as 2%. For the other higher modes, it was assigned as 5%. The logic behind the 2% damping is that it is conservative to choose damping lower since the seismically isolated superstructure behaves almost elastically.

Tables 3.6 and 3.7 show the periods of the first three fixed-base modes of the 3-S and 7-S buildings under elastic conditions, respectively.

Table 3.4 LRB properties used in analysis for 3-S buildings

Period T (sec.)	Q/W	Effective Stiffness k_{eff} (kN/m)	Effective Damping β_{eff}	Elastic Stiffness k_e (kN/m)	Yield Strength F_y (kN)	Post Yield Stiffness Ratio
3.5	0.08	145.61	0.171	2686.00	26.86	0.039
3.5	0.10	164.07	0.221	3332.00	33.32	0.032
3.5	0.12	185.47	0.267	3977.00	39.77	0.027
3.5	0.14	211.81	0.311	4622.00	46.22	0.023
4.0	0.08	117.92	0.196	2661.16	26.61	0.030
4.0	0.10	134.60	0.249	3307.16	33.07	0.025
4.0	0.12	155.32	0.298	3952.16	39.52	0.021
4.0	0.14	181.20	0.344	4597.16	45.97	0.018
4.5	0.08	98.04	0.217	2644.12	26.44	0.024
4.5	0.10	114.00	0.274	3290.12	32.90	0.019
4.5	0.12	134.45	0.327	3935.12	39.35	0.016
4.5	0.14	162.50	0.377	4580.12	45.80	0.014

Table 3.5 LRB properties used in analysis for 7-S buildings.

Period T (sec.)	Q/W	Effective Stiffness k_{eff} (kN/m)	Effective Damping β_{eff}	Elastic Stiffness k_e (kN/m)	Yield Strength F_y (kN)	Post Yield Stiffness Ratio
3.5	0.08	307.43	0.171	5671.78	56.72	0.039
3.5	0.10	346.35	0.221	7033.78	70.34	0.032
3.5	0.12	391.52	0.267	8394.78	83.95	0.027
3.5	0.14	447.13	0.311	9756.78	97.57	0.023
4.0	0.08	248.94	0.196	5619.33	56.19	0.030
4.0	0.10	284.13	0.249	6981.33	69.81	0.025
4.0	0.12	327.86	0.298	8342.33	83.42	0.021
4.0	0.14	382.52	0.344	9704.33	97.04	0.018
4.5	0.08	207.00	0.217	5583.37	55.83	0.024
4.5	0.10	240.66	0.274	6945.37	69.45	0.019
4.5	0.12	283.83	0.327	8306.37	83.06	0.016
4.5	0.14	343.04	0.377	9668.37	96.68	0.014

Table 3.6 The first three periods of modes of analyzed fixed-base 3-story building.

MODE	PERIOD (sec.)
1 (Translation)	0.18
2 (Translation)	0.18
3 (Rotation)	0.14

Table 3.7 The first three periods of modes of analyzed fixed-base 7-story building.

MODE	PERIOD (sec.)
1 (Translation)	0.45
2 (Translation)	0.44
3 (Rotation)	0.15

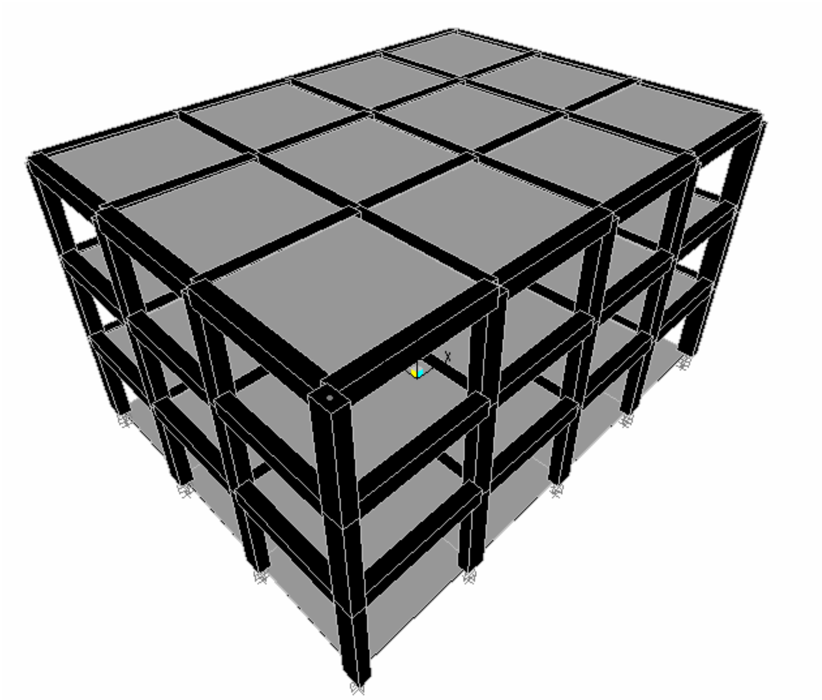

Figure 3.7 3-D model of 3-S RC structure in SAP2000.

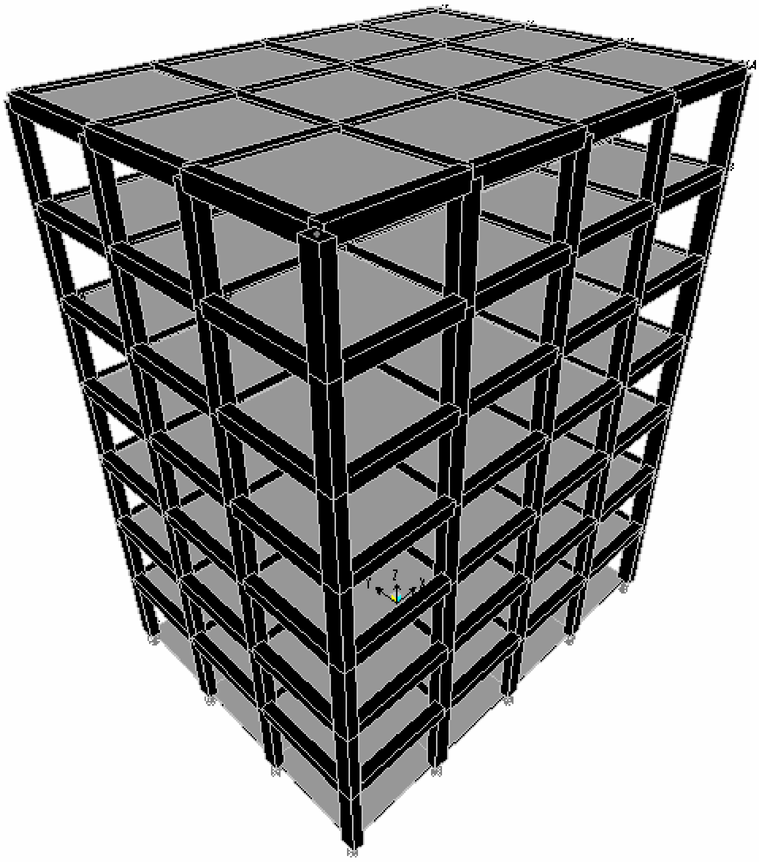

Figure 3.8 3-D model of 7-S RC structure in SAP2000.

CHAPTER 4

ANALYSES OF STRUCTURES

4.1. General Concepts

In this study, 1848 Nonlinear RHA was conducted under bi-directional earthquake excitations in structural analysis program SAP2000 (2008). The ground motion excitations are applied simultaneously like in Figure 4.1. The response of symmetric buildings for both combinations gives same results. On the other hand, asymmetric buildings are subjected to both combinations of ground motion excitations. The maximum response of these two combinations has been investigated.

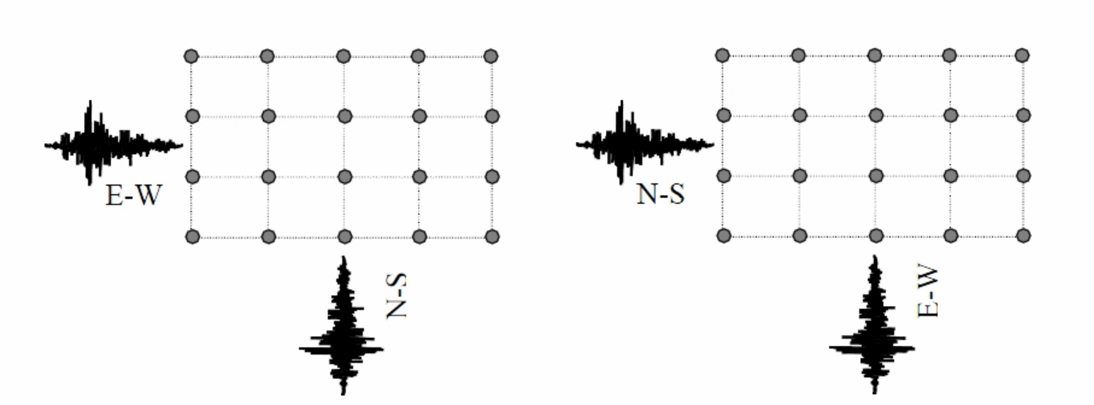

Figure 4.1 Application of bi-directional ground motion excitations (Ozdemir 2010).

As mentioned in Chapter 3, eccentricity in superstructure can be provided in two different ways: stiffness and mass eccentricity. The type of irregularity on superstructure is chosen as mass eccentricity for analyses. Therefore, seismically isolated buildings with eccentric superstructure are also subjected to real scaled ground motions. Then, the same procedures with symmetric buildings are applied for asymmetric buildings, when calculating mean values of isolator displacements. The selected level of mass eccentricity values are chosen as 5%, 10% and 15% of plan dimensions of floors in both horizontal directions. On the other hand, as previously described, there are four different cases for location of these eccentricities as shown in Figure 3.5. Since, cases 1 and 2 are approximately same with cases 3 and 4, the evaluations and comparisons are done taking cases 1 and 2 into consideration.

The torsional amplifications of superstructure asymmetry on seismically isolated buildings are investigated. The variations of base shears, torsional moments, isolator displacements, floor displacements and accelerations are compared based on the level of eccentricities. The discussions of results are summarized in the following sections.

4.2. Base Shear

Reduction of base shear is one of the basic purposes of base isolation system in design of buildings since it decreases earthquake damage. As a result, variation of base shear with different effective damping values, isolation periods and level of eccentricities are investigated in this thesis. Base shear results of Nonlinear RHA of near-fault ground motions under two horizontal orthogonal directions were calculated. Maximum of calculated base shear are taken and normalized with weight of analyzed buildings. V_{RHA} is the representation of maximum base shear.

The variation of normalized base shear as a function of effective damping and isolation periods for different level of eccentricities of 3-story (3-S) and 7-story (7-S) buildings are given in Figure 4.2 and 4.3, respectively.

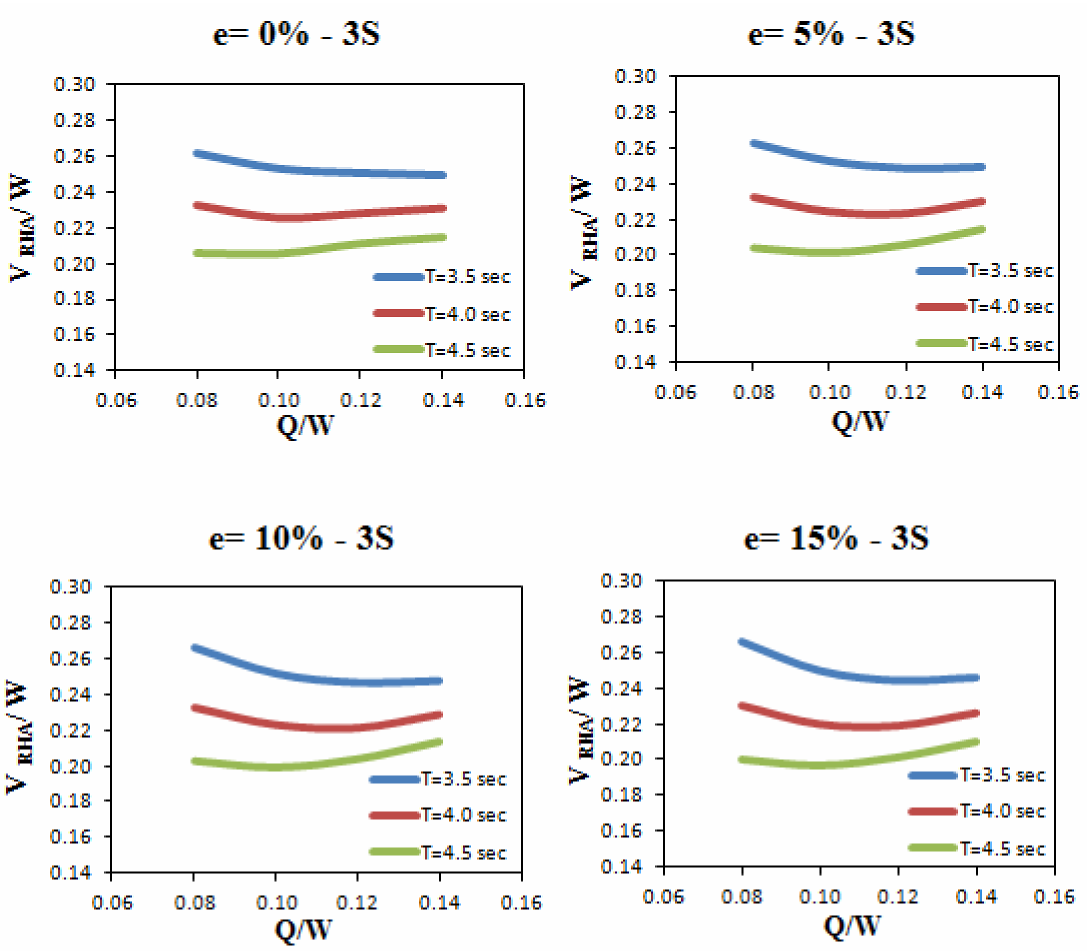

Figure 4.2 Variation of maximum base shears with damping for different level of eccentricities in 3- story superstructure.

According to nonlinear RHA results, alteration of normalized V_{RHA} is nearly same for each level of eccentricities and can be seen in Figures 4.2 and 4.3. The normalized base shear values are also similar for 3-S and 7-S for same periods, damping values and eccentricities. This means that change in superstructure has no

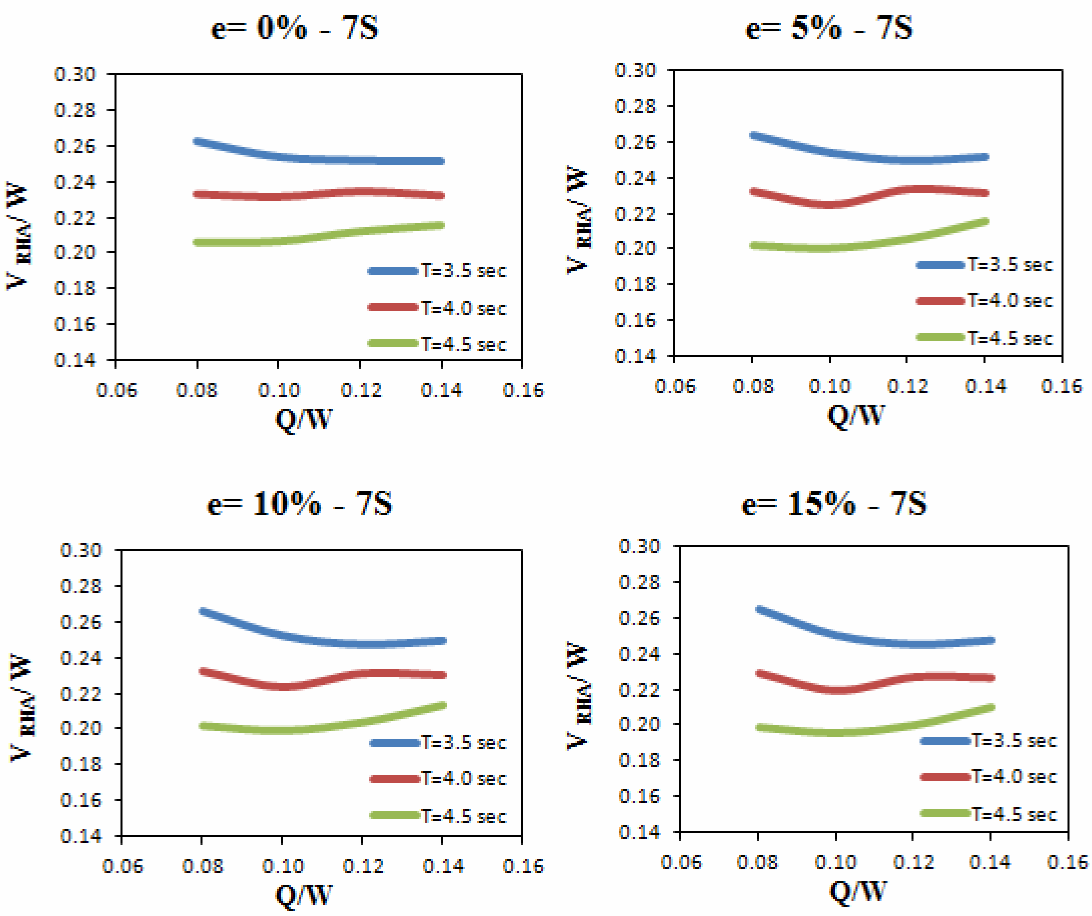

Figure 4.3 Variation of maximum base shears with damping for different level of eccentricities in 7- story superstructure.

effect on normalized base shear of isolated buildings. The variation of normalized base shear when Q/W =0.14 for different level of eccentricity of 3-S and 7-S buildings are given in Figure 4.4. The values of normalized base shear are very close to each other and decrease from 0.25W to 0.2 W. Another conclusion from the nonlinear RHA results of the analysis is that effective damping has no effect on base shear. On the other hand, the increase of isolation periods causes a decrease in base shear. However, it is very small and can be neglected. Finally, as mentioned before,

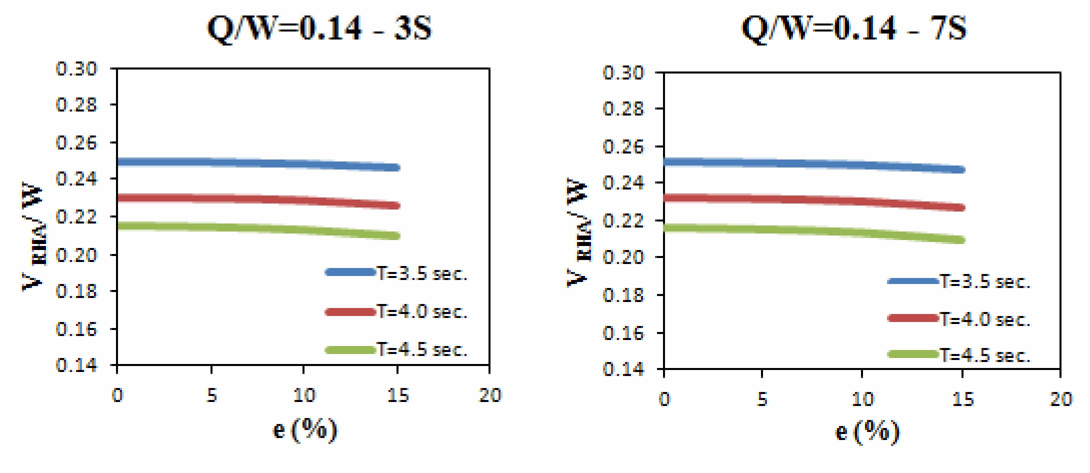

Figure 4.4 Variation of maximum base shears with different level of eccentricities for different isolation periods and Q/W =0.14 in 3-S and 7-S buildings.

the selected parameters were chosen so that the base shear of each of these isolation systems does not exceed 30% of the weight of the structure in the maximum considered earthquake (MCE) and results meets the desired condition.

4.3. Torsional Moments

One of the expected results of mass eccentricity is torsional moment in buildings since torsional moment is calculated as multiplication of base shear by eccentricity. The effects of different level of eccentricities with different isolation period and different effective damping values on torsional moment for 3-S and 7-S buildings are investigated in this study and the results are illustrated in Figures 4.5 and 4.6. $M_{torsional}$ is the representation of maximum torsional moment calculated from nonlinear RHA. As similar to base shear results, effective damping ratio has no effect on torsional moment. However, an increase in the eccentricity of building results in an increase in torsional moment. Although the torsional moment of symmetrical

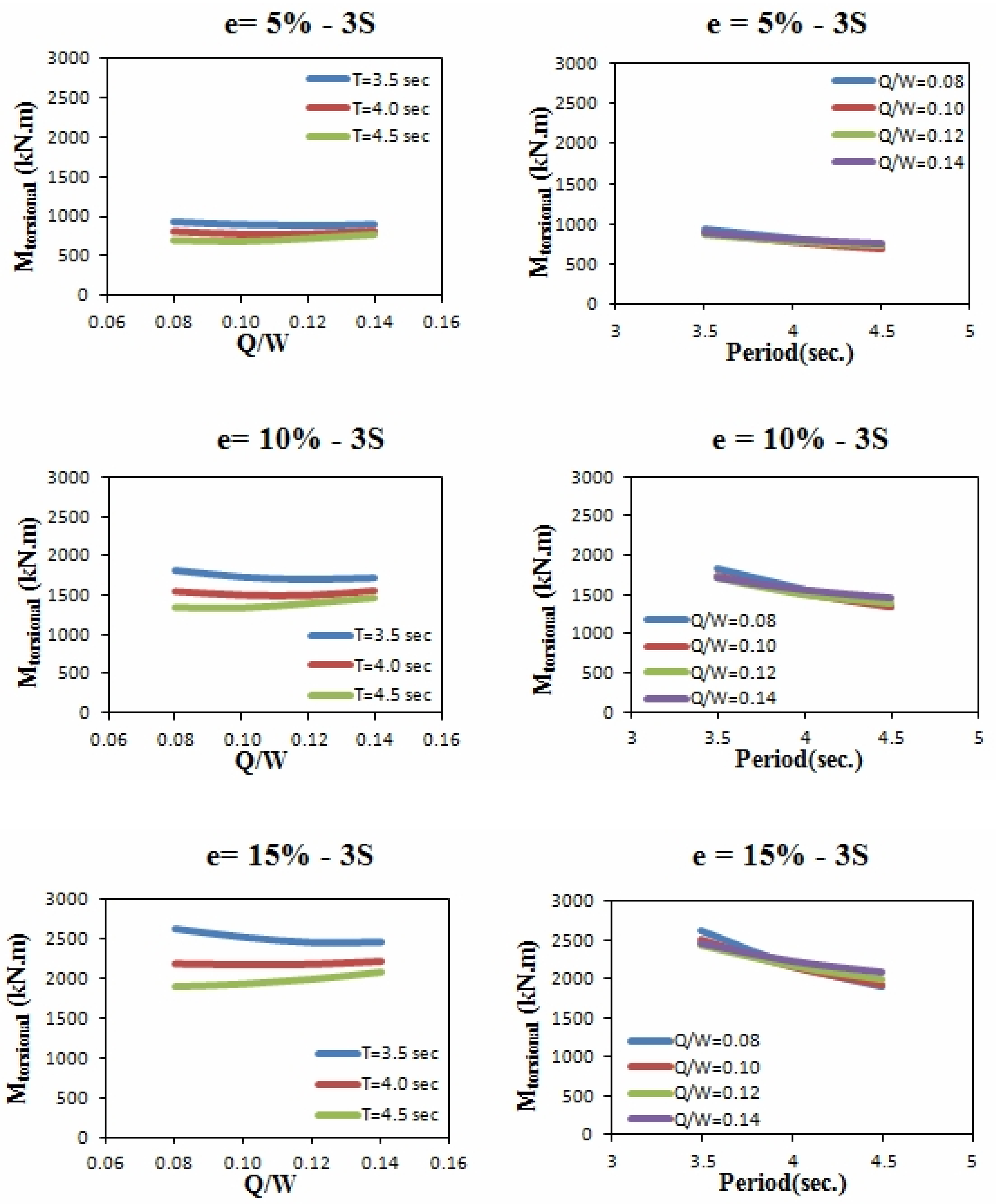

Figure 4.5 Variation of maximum torsional moment with effective damping and isolation periods for different level of eccentricities in 3- story (3-S) superstructure.

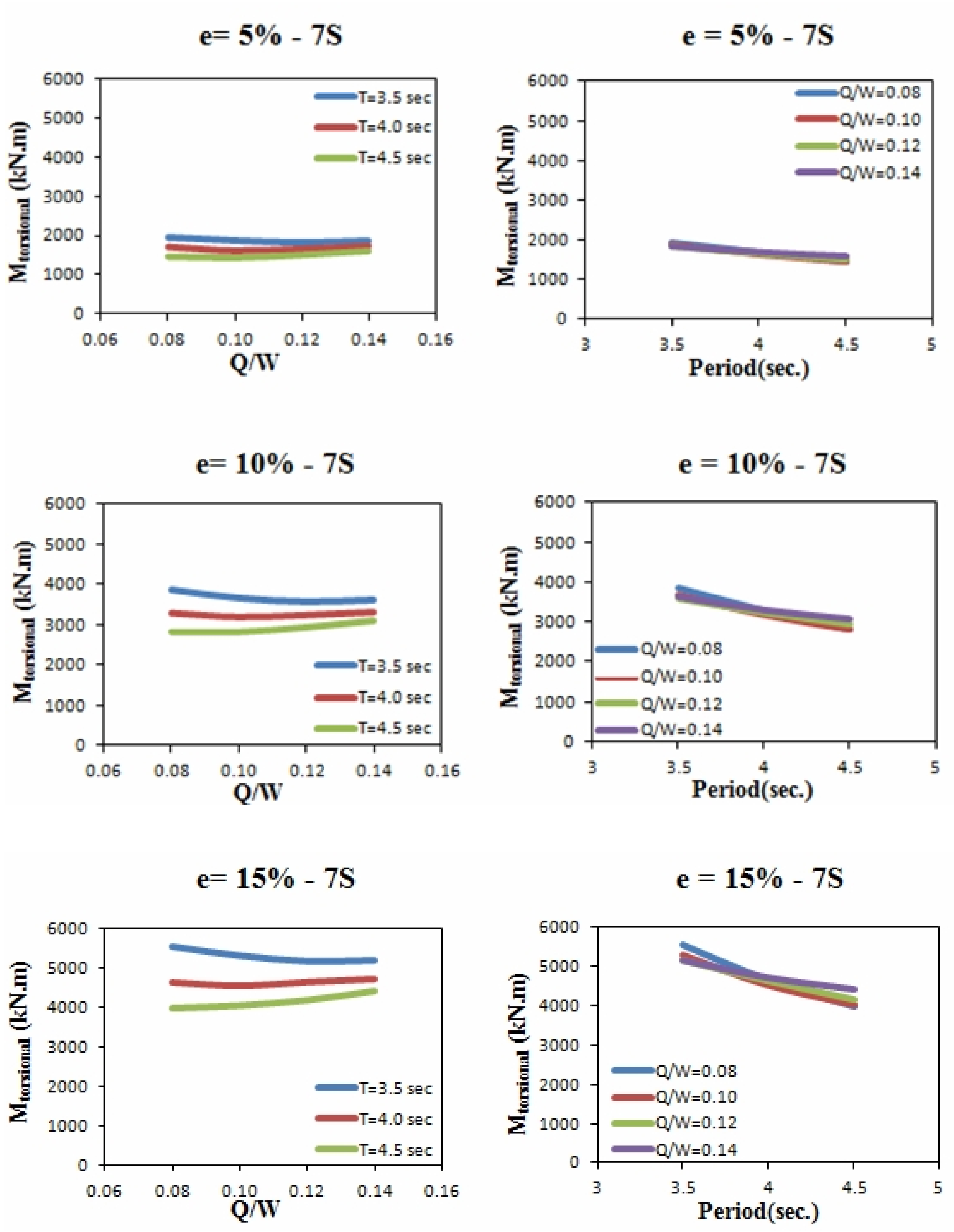

Figure 4.6 Variation of maximum torsional moment with effective damping and isolation periods for different level of eccentricities in 7- story (7-S) superstructure.

buildings is very close to zero, for buildings with 15% mass eccentricity these values reach about 2600 kN.m and 5550 kN.m for 3-S and 7-S buildings, respectively. Figure 4.7 shows the alteration of torsional moment with Q/W =0.14 for different level of eccentricity of 3-S and 7-S buildings. Decreasing isolation period also leads to increase in torsional moment. Moreover, increasing level of eccentricity also increases calculated torsional moments. $M_{torsional(e=10\%)}$ values of asymmetric 3-S and 7-S RC buildings are between 90%~96% higher than the values of $M_{torsional(e=5\%)}$. In addition to that, $M_{torsional(e=15\%)}$ values of buildings are between 170% and 183% of $M_{torsional(e=5\%)}$ for different isolation periods.

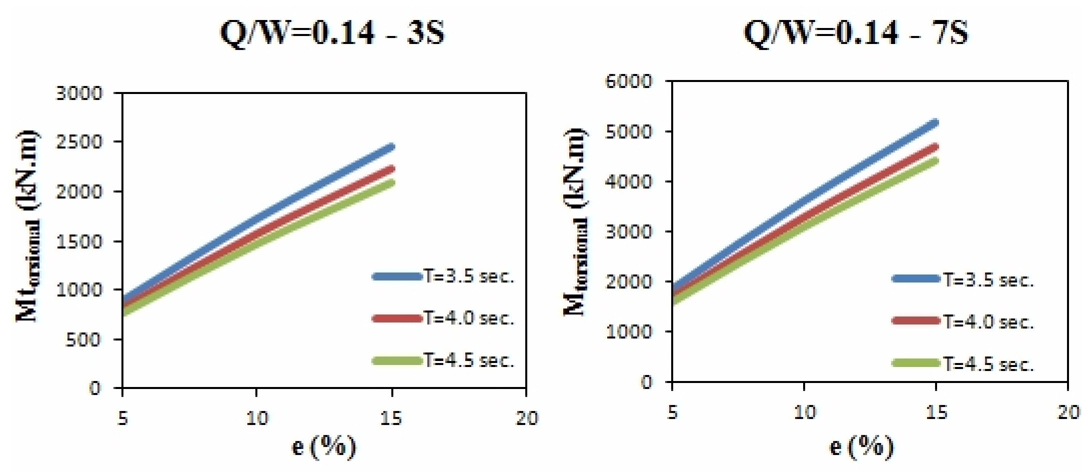

Figure 4.7 Variation of maximum torsional moment with different level of eccentricities for different isolation periods and Q/W =0.14 in 3-S and 7-S buildings.

4.4. Maximum Isolator Displacement

Isolator displacements of the base isolated structures were calculated by taking the SRSS of displacements in both horizontal orthogonal directions at each time step.

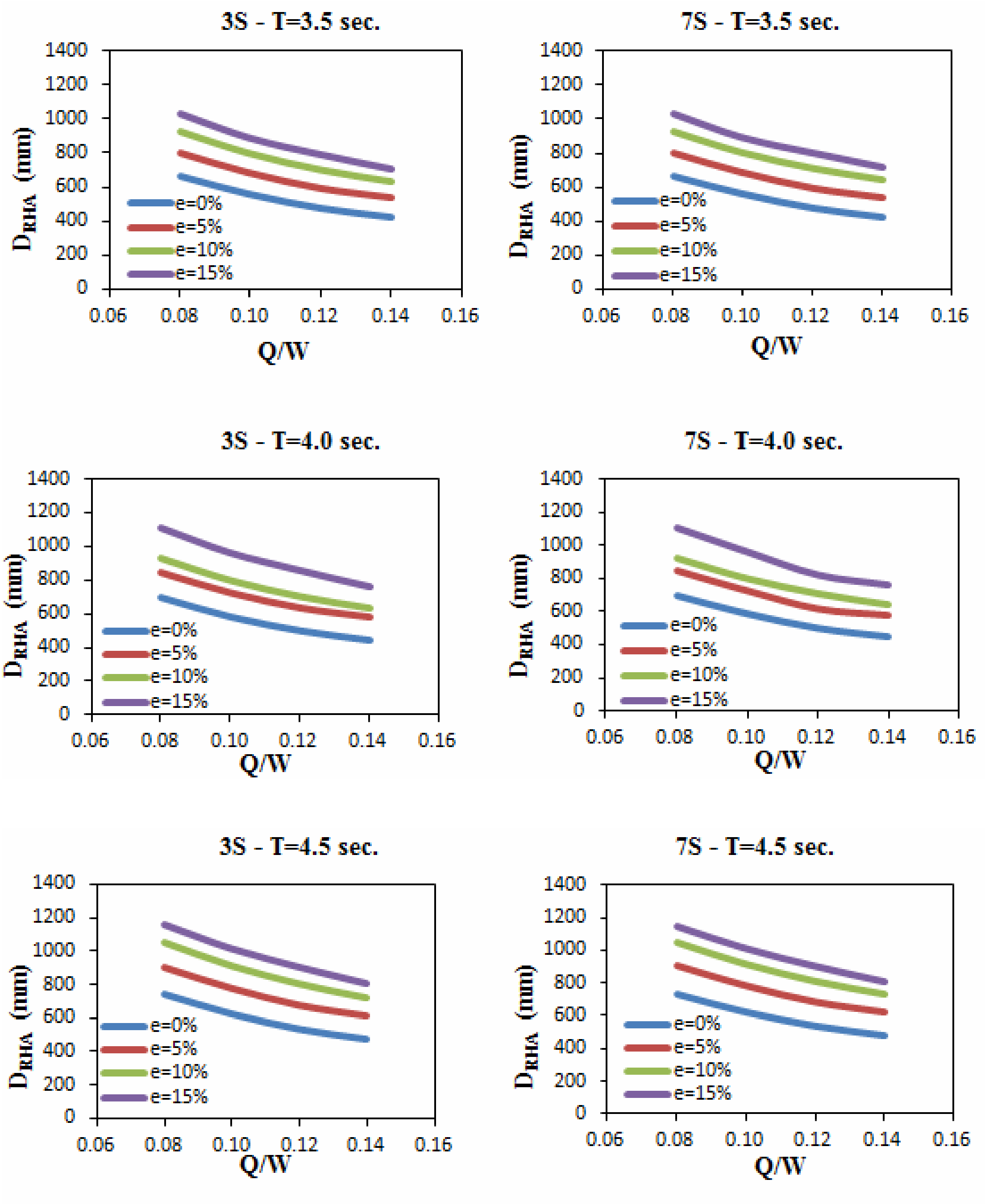

Figure 4.8 Variation of maximum isolator displacements with damping and eccentricities in superstructure.

Afterward, the maximum of calculated isolator displacements were taken for each ground motion pair. Finally, the mean values of maximum displacements of each earthquake record (Table 3.1) are calculated. D_{RHA} is the representation of mean isolator displacement.

In Figure 4.8, the variations of D_{RHA} are represented as functions of damping and mass eccentricity in superstructure for various isolation periods. It is clear that isolator displacement responses of 3-S and 7-S isolated buildings are very close to each other for same damping and same level of eccentricities. It indicates that isolator displacements are independent from superstructure in terms of story height. On the other hand, the results taken from nonlinear RHA demonstrate the inverse relation between damping (Q/W) and isolator displacements regardless of evaluated isolation period and eccentricity above basement. Figure 4.8 displays the decreasing tendency of isolator displacements due to increasing damping. Moreover, the amount of decrease is nearly same for different isolation periods. These decreases are about 36%, 32%, 32% and 31% for symmetric and asymmetric 3-S and 7-S RC buildings corresponding to e=5%, 10% and 15%, respectively. The nonlinear RHA results of isolator displacements of 3-S and 7-S buildings with 15% eccentricity are normalized

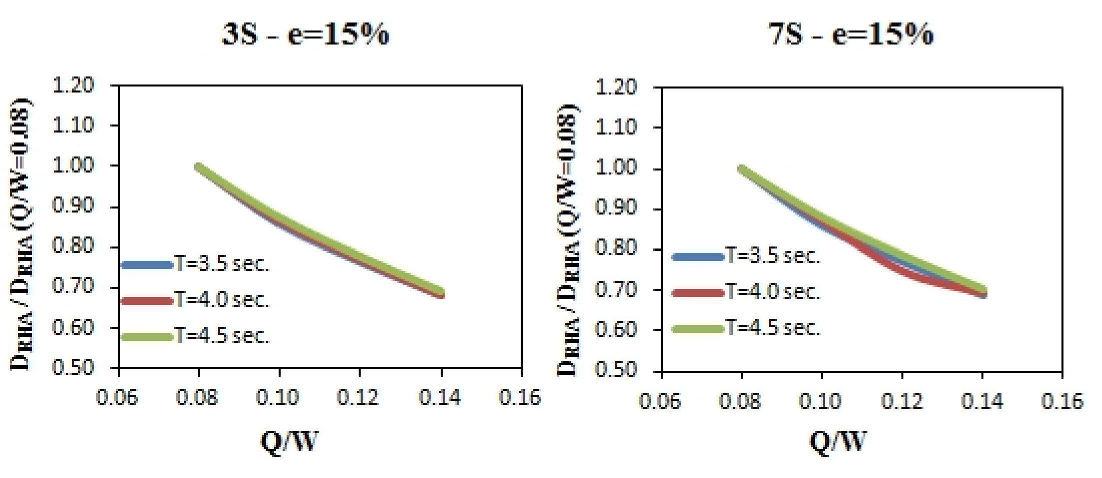

Figure 4.9 Normalized isolator displacements obtained from bi-directional excitations for 3- story buildings and each mass eccentricity.

with that of smallest effective damping (Q/W=0.08) and illustrated in Figure 4.9. The normalized displacements are coincides with each other.

Increasing isolation period has increasing effects on isolator displacements. With the variation of isolation period from 3.5 sec to 4.5 sec, D_{RHA} values increase about 12%, 14%, 14% and 13% for symmetric and asymmetric 3-S and 7-S RC buildings corresponding to e=5%, 10% and 15%, respectively.

The variation of $(D_{RHA})_{ecc}/D_{RHA}$ ratio for various effective damping values as a function of isolation periods for 3-S and 7-S buildings are illustrated in Figure 4.10 for each level of eccentricity. These ratios generally increase slightly when isolation periods increase for 3-S buildings. They are in between 1.15-1.30, 1.35-1.55 and 1.55-1.70 depending on Q/W ratio for e=5%, 10% and 15%, respectively. Figure 4.10, the decreasing tendency of $(D_{RHA})_{ecc}/D_{RHA}$ with increasing damping, observed in 3-S buildings, dissappears in 7-S buildings. The eccentricity increase diminishes the effect of damping. Moreover, the isolation periods become ineffective on $(D_{RHA})_{ecc}/D_{RHA}$.

In this thesis, the accuracy of simplified method in terms of isolator displacement is also investigated. In Figure 4.11, the horizontal and vertical axes show the isolator displacements calculated by equivalent lateral force procedure (D_{ELF}) and maximum resultant isolator displacements taken from nonlinear RHA (D_{RHA}), respectively. The solid lines imply the equality of D_{ELF} and D_{RHA} values for each eccentricity. It is clearly seen that almost all values are below the solid line that represents the conservation of simplified method. The overestimation of isolator displacements of ELF is about 10% and independent from the level of eccentricities, since different level of eccentricities are concern in calculations of isolator displacements in simplified method.

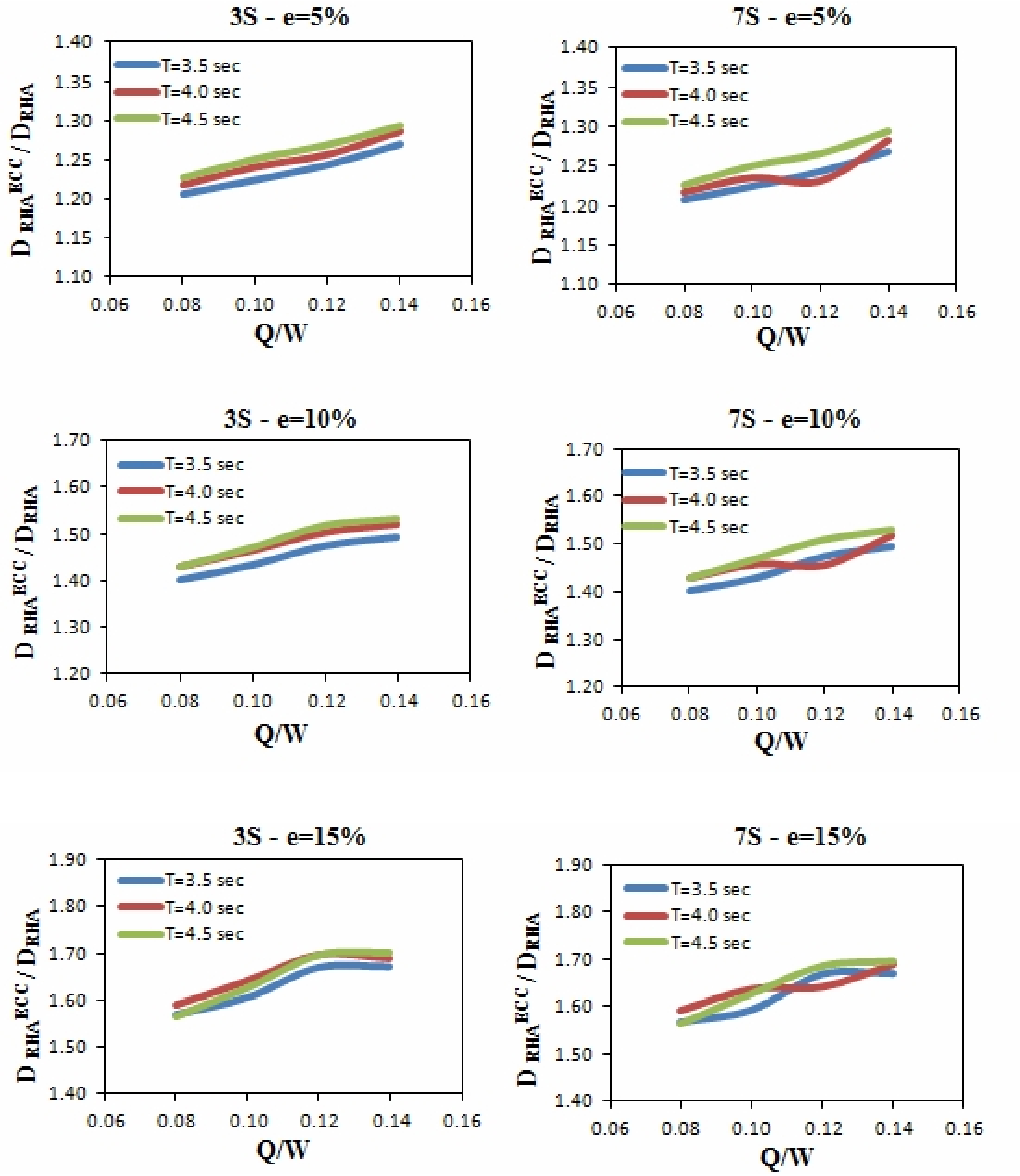

Figure 4.10 Amplifications in isolator displacements due to isolation period under increasing Q/W ratios.

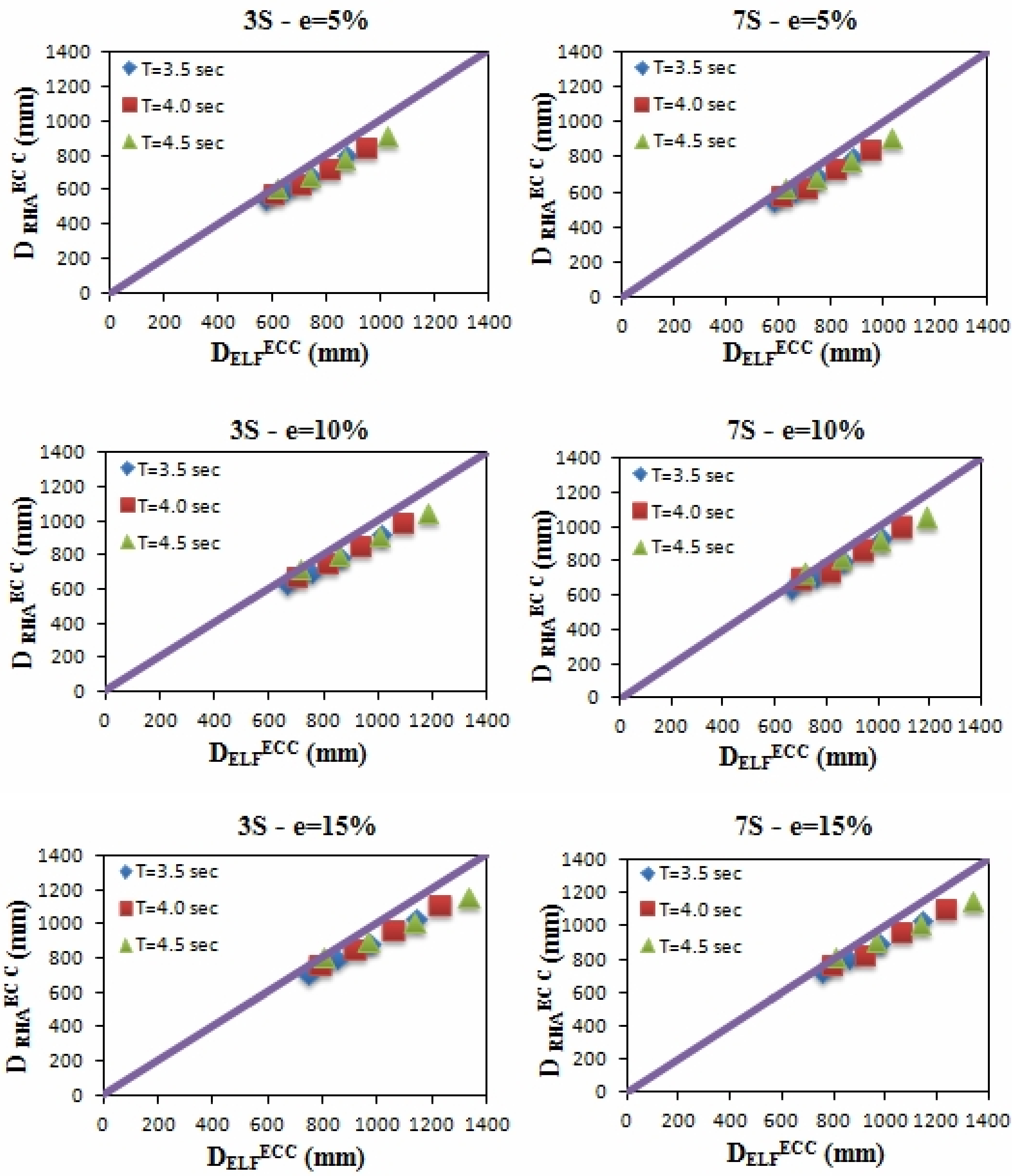

Figure 4.11 $D_{RHA}{}^{ECC}$ versus $D_{ELF}{}^{ECC}$ for 3-story and 7-story superstructure.

4.5. Floor Displacements

Calculation methods of floor displacements of 3-S and 7-S buildings are same as calculation of isolator displacements. SRSS of displacements are calculated in both horizontal orthogonal directions at each time step. Maximum of these displacements are taken and average of each selected ground motion displacement is calculated. The results are illustrated in Figure 4.12 and 4.13 for 3-S buildings and Figure 4.14 and 4.15 for 7-S buildings. Similar results are obtained for 3-S and 7-S buildings that is the indication of independency of floor displacements to superstructure in terms of story level. Increase in effective damping results in decrease in floor displacements for both 3-S and 7-S buildings. The higher the effective damping values are, the smaller the floor displacements are. On the other hand, there are remarkable increases in displacements with increasing isolation periods. Level of eccentricity is also an effective parameter that influences displacement response of buildings. Reflection of increasing level of eccentricity is increasing floor displacements. As expected, while story level is increasing, displacement increments between successive floors are very small.

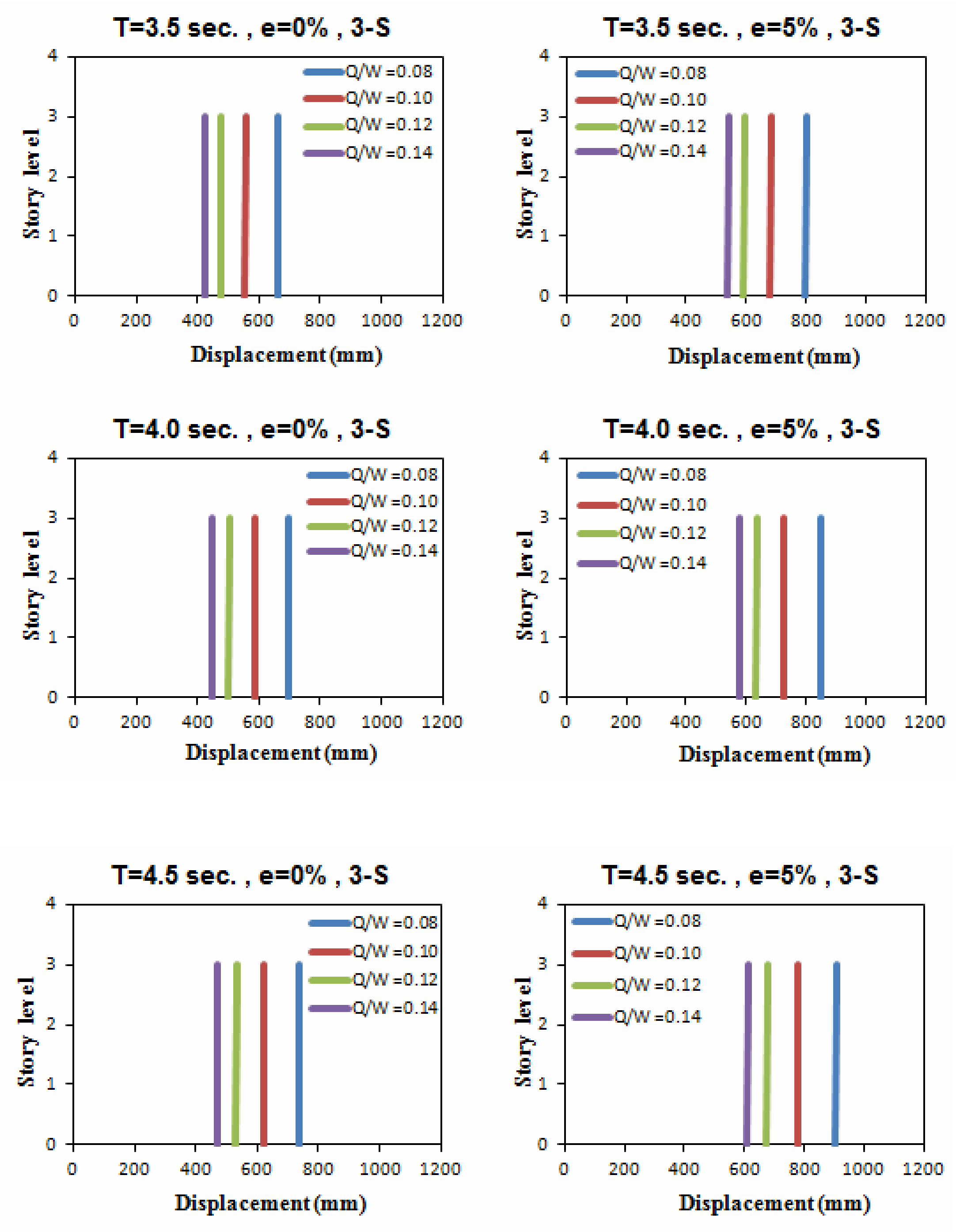

Figure 4.12 Variation of maximum displacements with story level and damping in 3-S buildings for e=0% and e=5%.

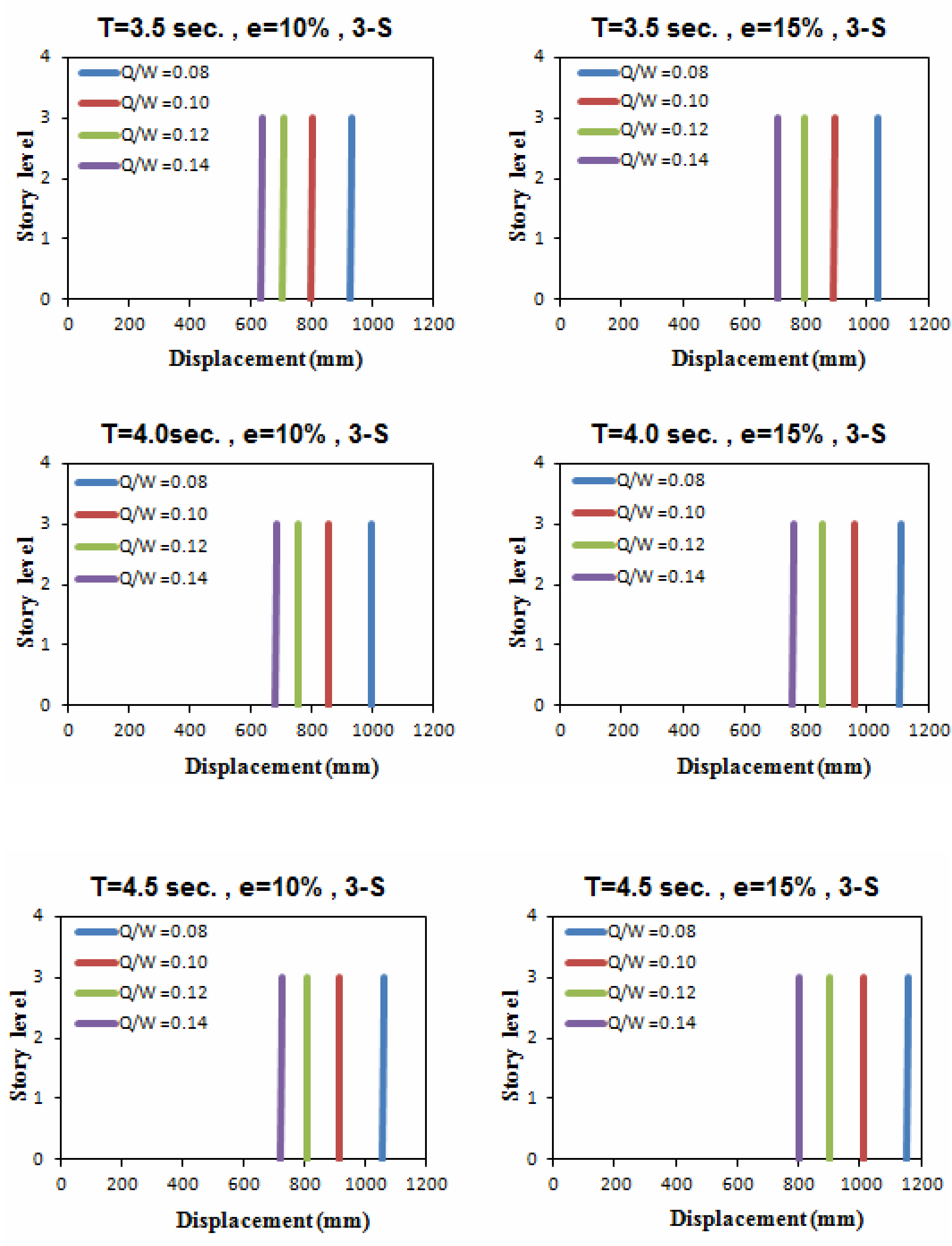

Figure 4.13 Variation of maximum displacements with story level and damping in 3-S buildings for e=10% and e=15%.

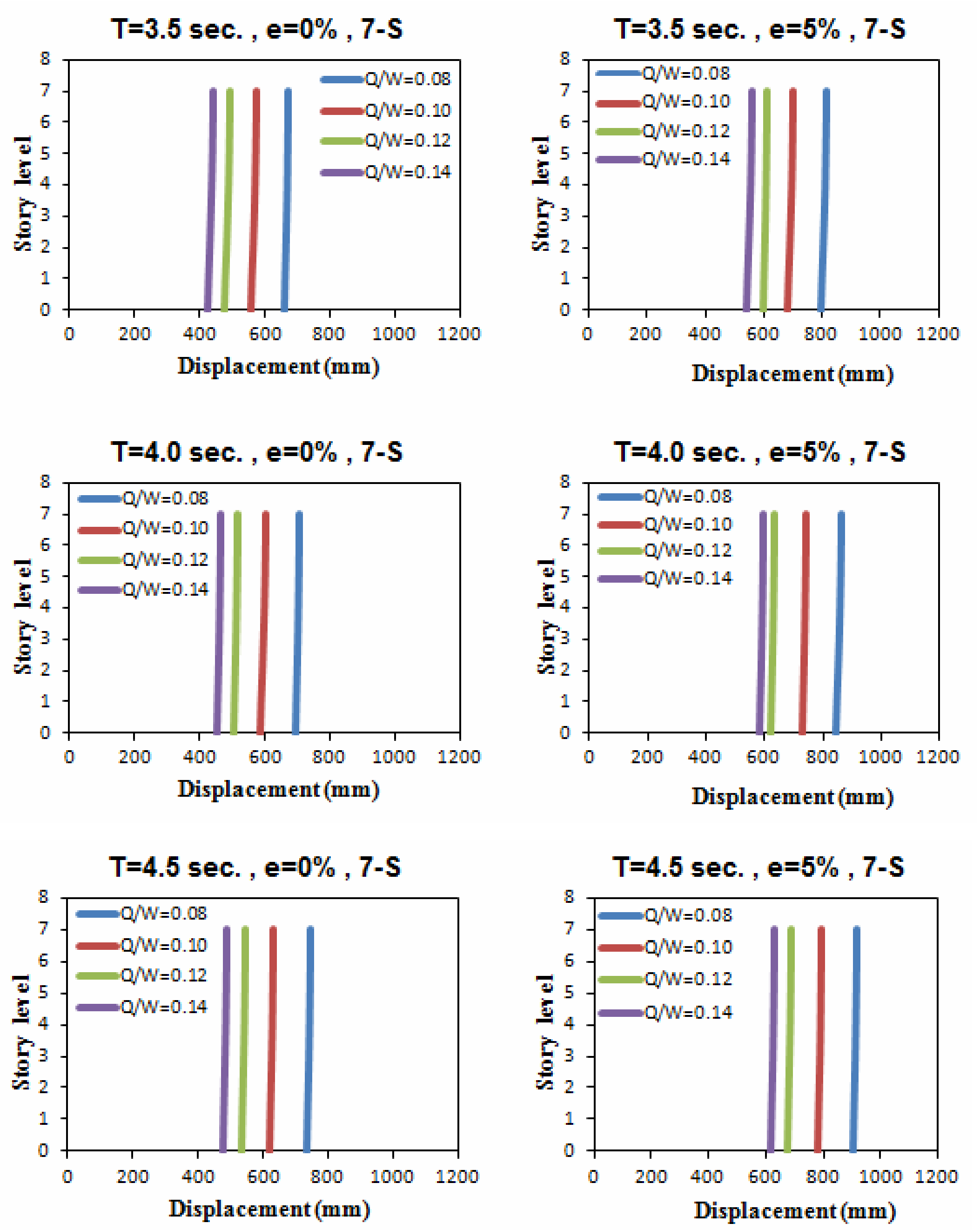

Figure 4.14 Variation of maximum displacements with story level and damping in 7-S buildings for e=0% and e=5%.

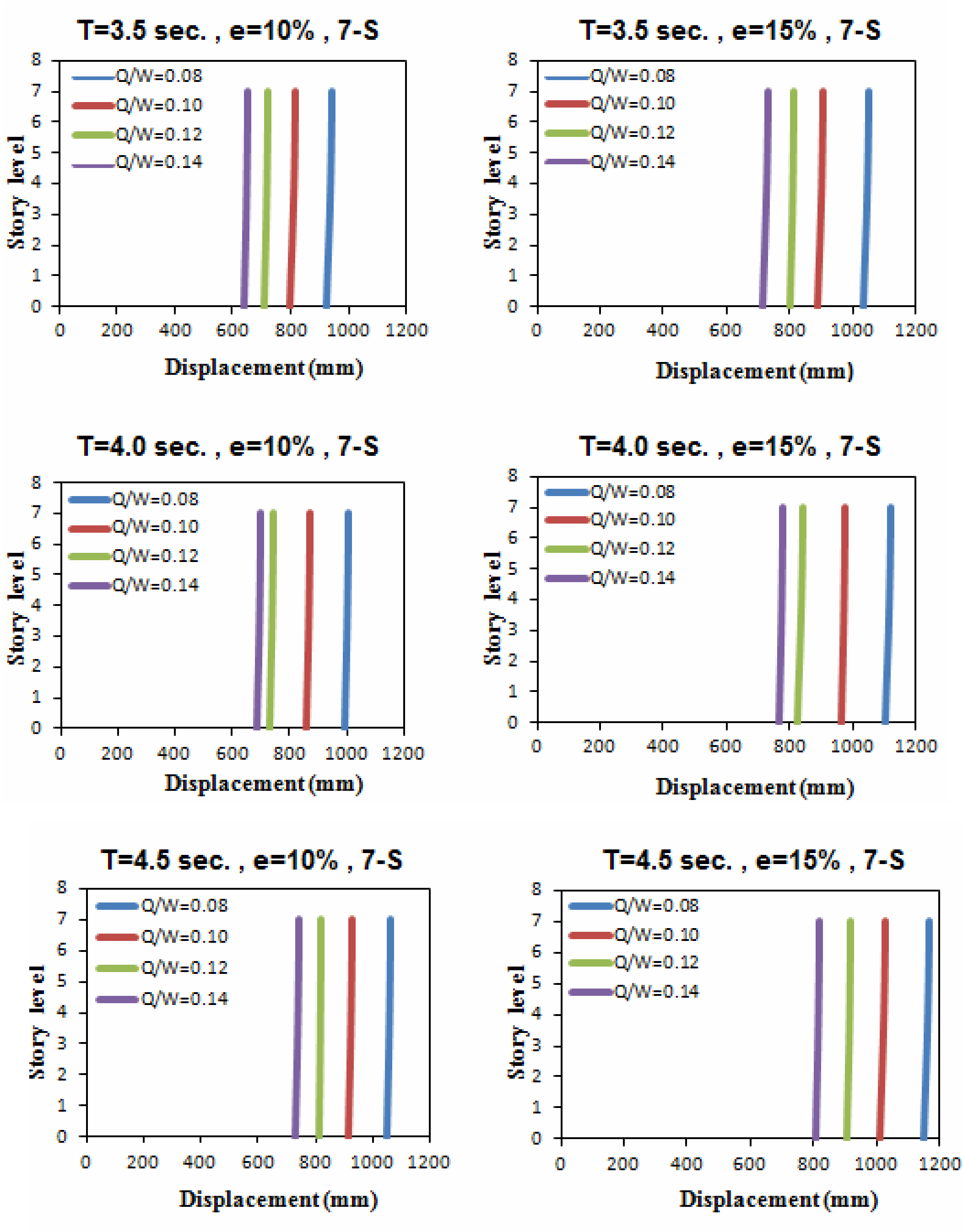

Figure 4.15 Variation of maximum displacements with story level and damping in 7-S buildings for e=10% and e=15%.

4.6. Floor Accelerations

Floor acceleration is another important parameter when earthquake damage is concerned. One of the main purposes of seismic isolation is to reduce the floor accelerations. In accordance with this intention, the variation of floor acceleration with different level of eccentricity, isolation period and effective damping are investigated in this study. Floor accelerations were calculated by taking maximum of accelerations in both horizontal orthogonal directions at each time step of selected ground motions. Then averages of these maximum accelerations of each ground motion were calculated. Acc is the representation of this maximum average acceleration.

The changes of floor acceleration with evaluated parameters are illustrated through Figures 4.16-4.19 for 3-S buildings and Figures 4.20-4.23 for 7-S buildings. An increase in isolation period results in a decrease in floor accelerations through height of buildings in 3-S RC symmetric buildings. In case of asymmetric 3-S structures, the decreasing effect of increasing periods becomes to disappear. In case of 7-S RC symmetric buildings, the decreasing effect of increasing isolation periods continue only for Q/W =0.08 and Q/W =0.10. For 7-S buildings with Q/W ratio equal to 0.12 and 0.14 and asymmetric 7-S buildings, this effect diminishes.

Another remarkable result of analyses is that a sudden increase is observed in floor accelerations when comparing $Acc_{(e=0\%)}$ with $Acc_{(e=5\%)}$ for 3-S buildings . The $Acc_{(e=5\%)}$ values are 1.95 ~3.17 times higher than $Acc_{(e=0\%)}$ values. The increase between Acc values become relatively slow when level of eccentricity start to increase. The $Acc_{(e=10\%)}$ and $Acc_{(e=15\%)}$ values are 1.05 ~1.36 and 1.02~1.14 times higher than $Acc_{(e=5\%)}$ $Acc_{(e=10\%)}$ values, respectively.

Instantaneous rises between the results of $Acc_{(e=0\%)}$ and $Acc_{(e=5\%)}$ are also seen for 7-S buildings. The $Acc_{(e=5\%)}$ / The $Acc_{(e=0\%)}$ values are changing between 1.22 and 2.62. The $Acc_{(e=10\%)}$ / $Acc_{(e=5\%)}$ and $Acc_{(e=15\%)}$ / The $_{(e=10\%)}$ ratios are between 0.97 and 1.30 , 1.01 and 1.24, respectively. It is clear that the increment of Acc through increasing eccentricity is relatively small with respect to 3-S buildings. Furthermore, the differences through the height of structure in 7-S buildings are more remarkable with respect to 3-S buildings. That is the indication of higher modes effects in 7-S buildings.

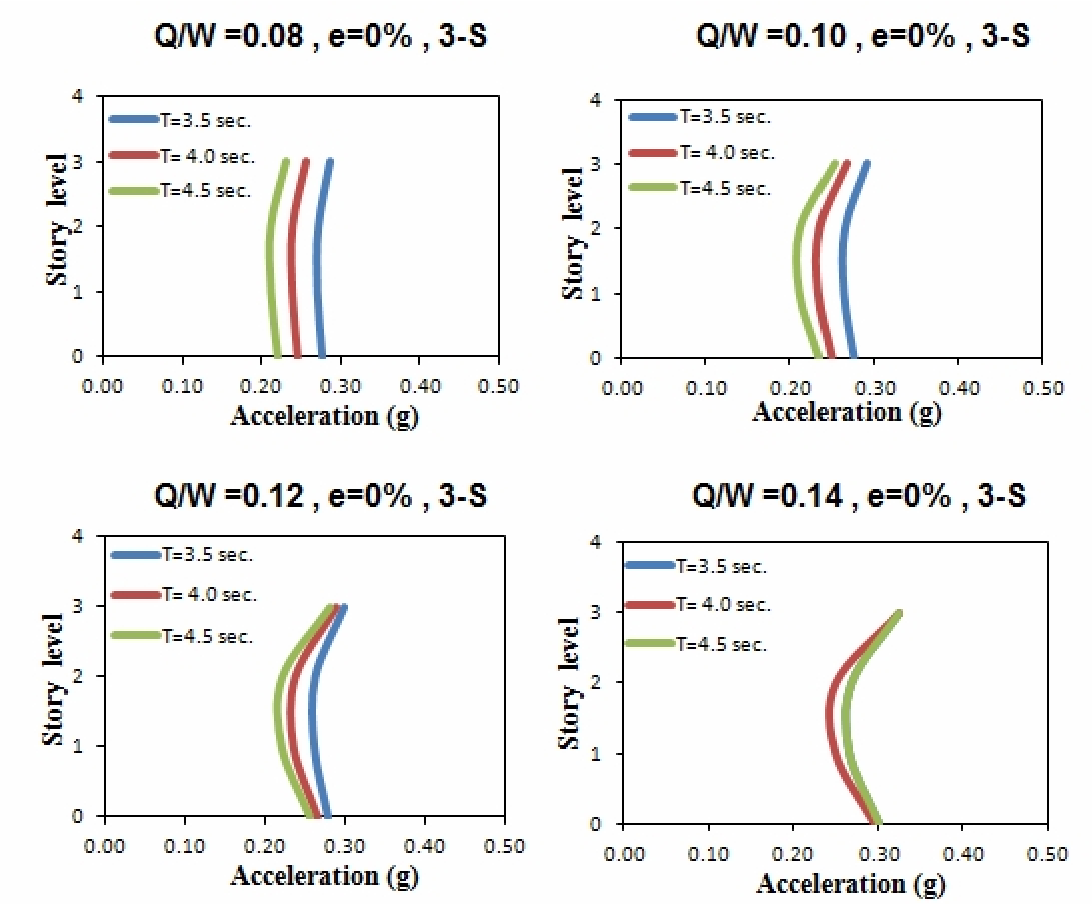

Figure 4.16 Variation of maximum floor accelerations with story level and isolation periods in 3-S buildings for e=0%.

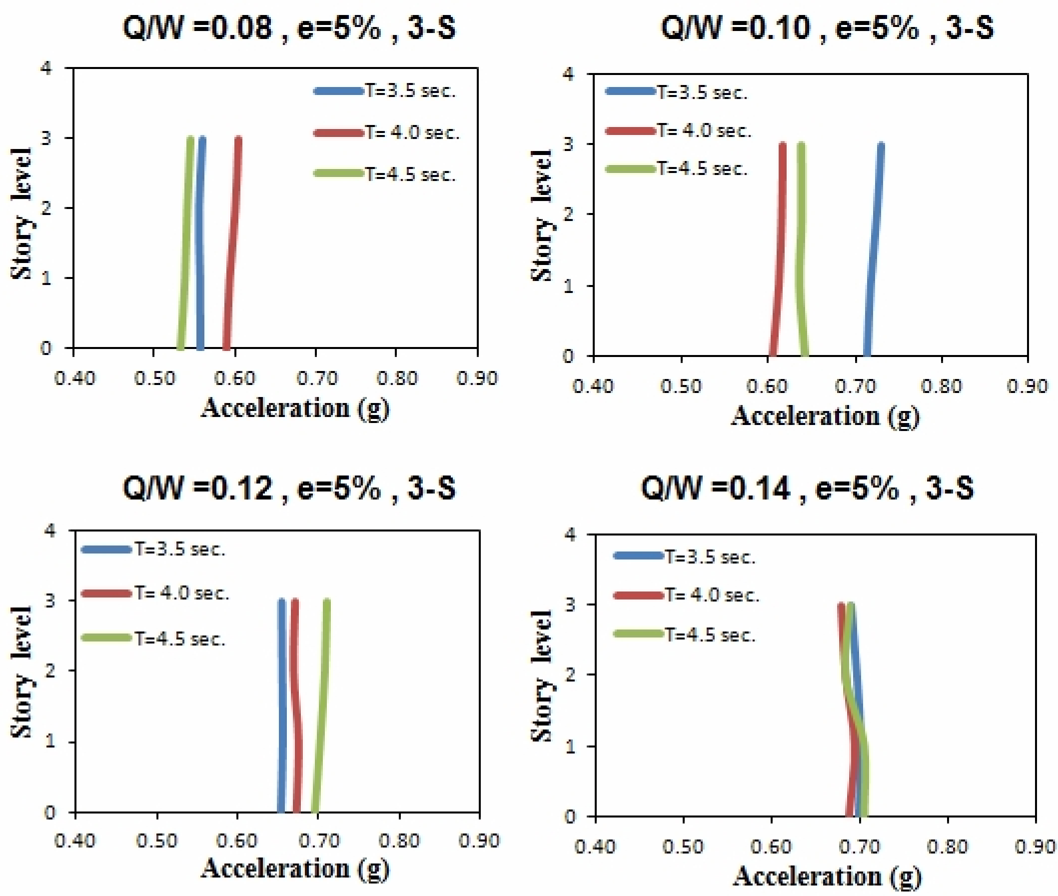

Figure 4.17 Variation of maximum floor accelerations with story level and isolation periods in 3-S buildings for e=5%.

The influence of increasing effective damping on floor accelerations is also increasing. The maximum floor accelerations for Q/W=0.14 are normalized with maximum floor accelerations of isolated building with Q/W =0.08. The results are tabulated in Table 4.1 for both 3-S and 7-S buildings. It is seen that the increasing effective damping also increases the floor accelerations' response of buildings. In symmetric buildings, the increase is directly proportional to isolation periods. This is in good accordance with the results obtained by Ozdemir (2010). On the other hand, with the increase in the level of eccentricity the increment percentage of maximum

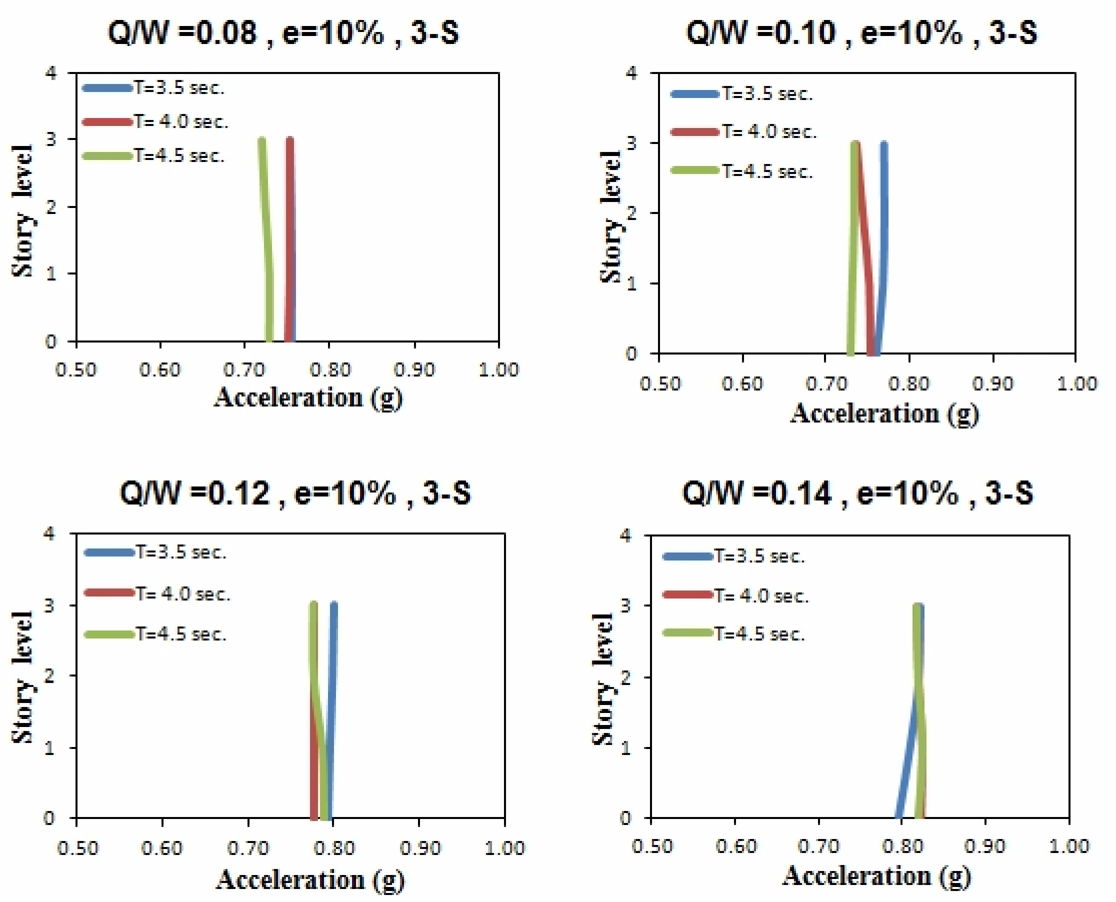

Figure 4.18 Variation of maximum floor accelerations with story level and isolation periods in 3-S buildings for e=10%.

Acc become independent from isolation periods as mentioned before. The percentage of increase of 7-S symmetric buildings' Acc is high relatively compared to 3-S symmetric buildings. On the other hand, the effect of superstructure in terms of number of story becomes ineffective with increasing effective damping.

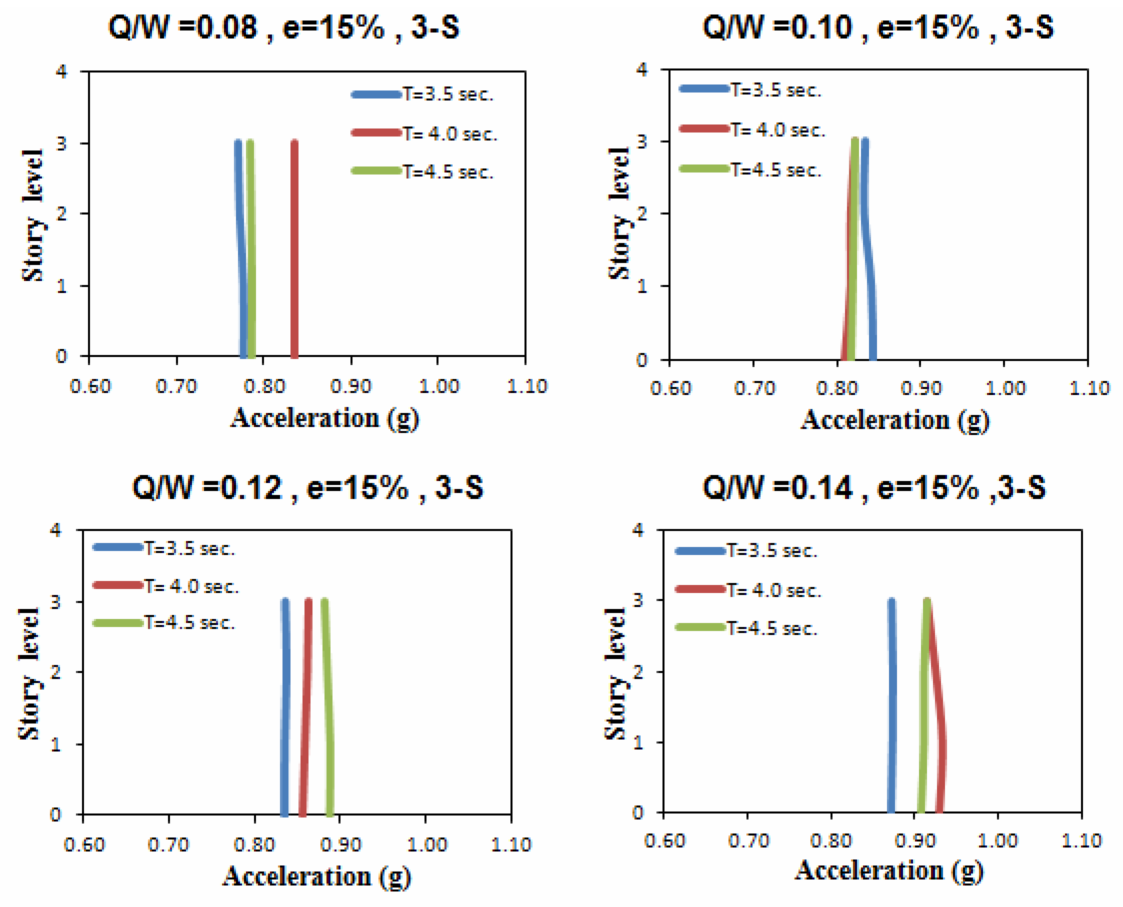

Figure 4.19 Variation of maximum floor accelerations with story level and isolation periods in 3-S buildings for e=15%.

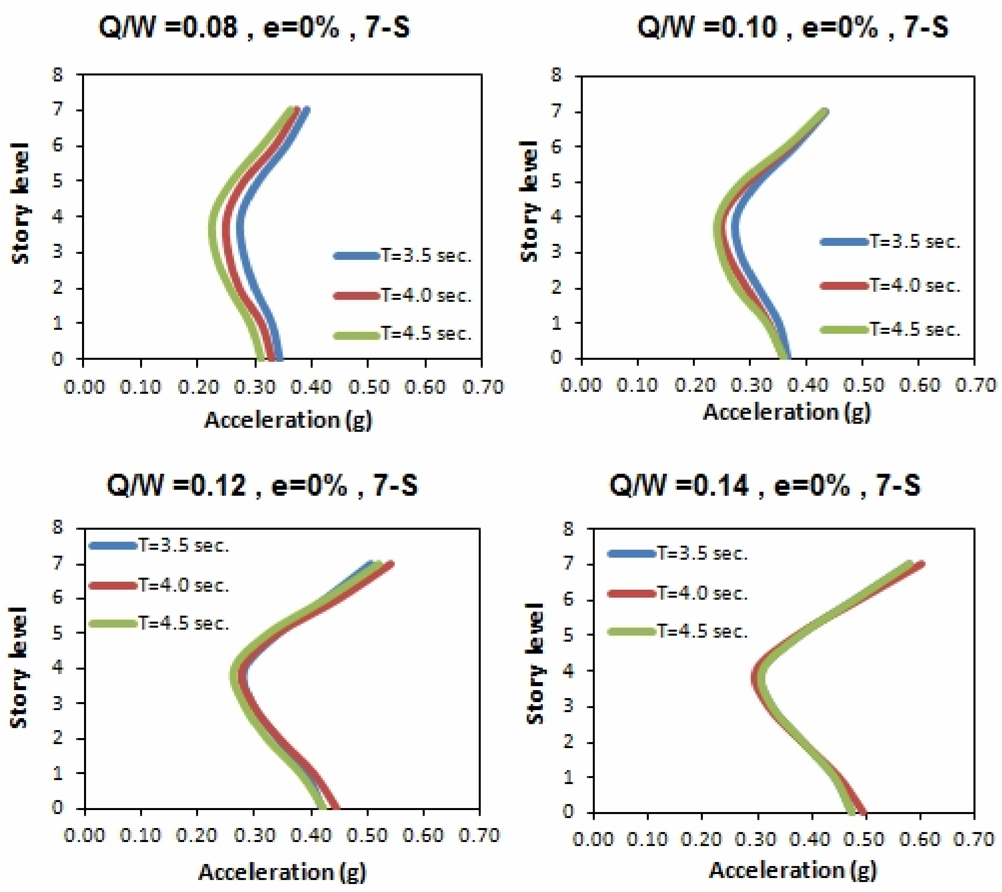

Figure 4.20 Variation of maximum floor accelerations with story level and isolation periods in 7-S buildings for e=0%.

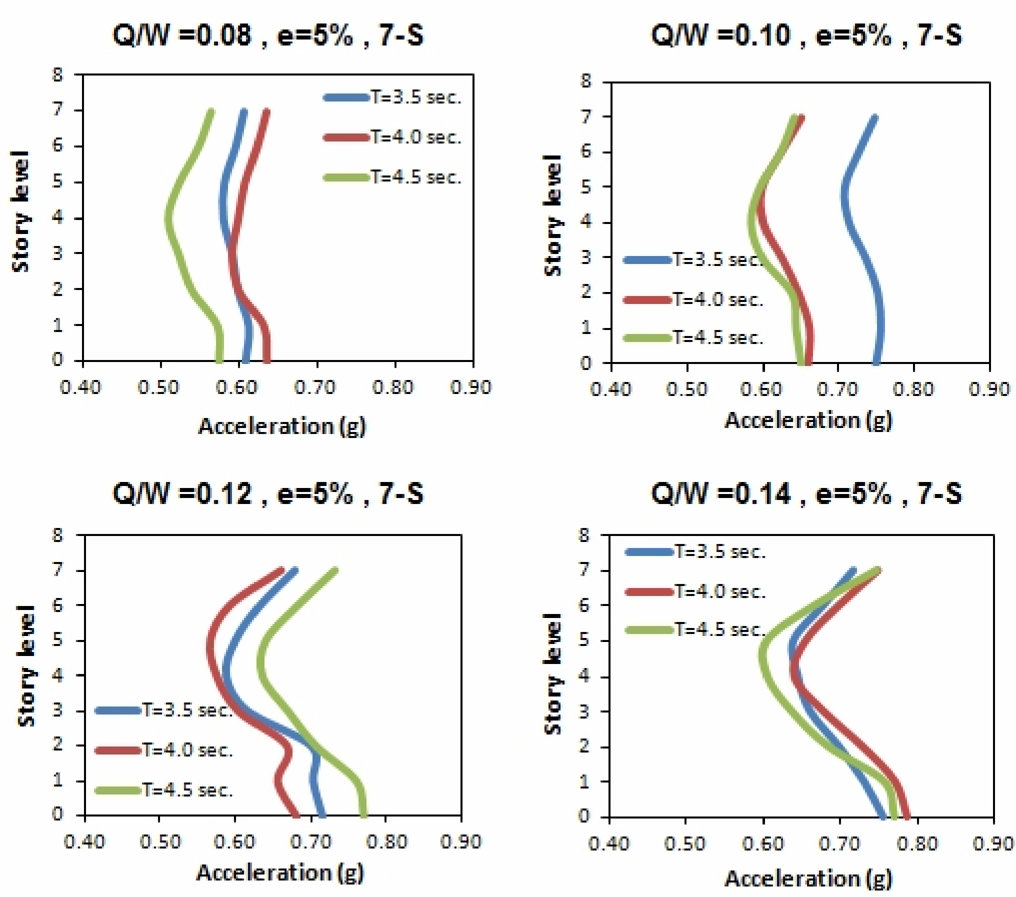

Figure 4.21 Variation of maximum floor accelerations with story level and isolation periods in 7-S buildings for e=5%.

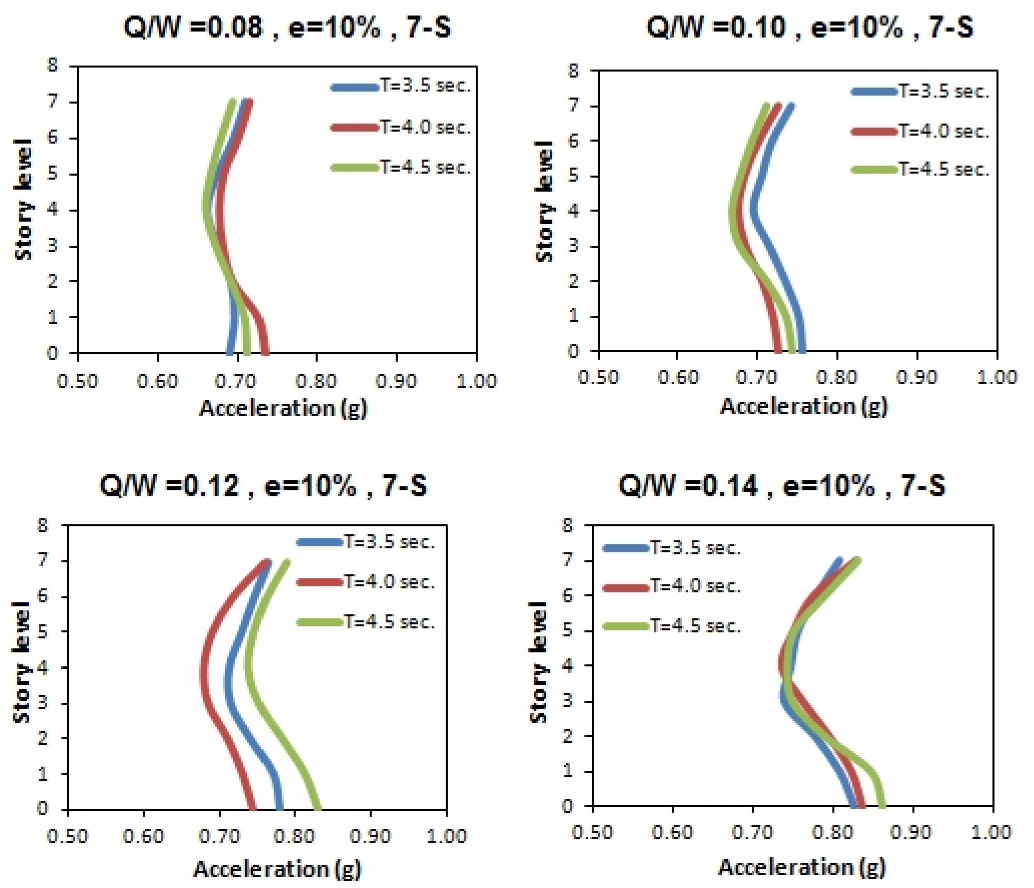

Figure 4.22 Variation of maximum floor accelerations with story level and isolation periods in 7-S buildings for e=10%.

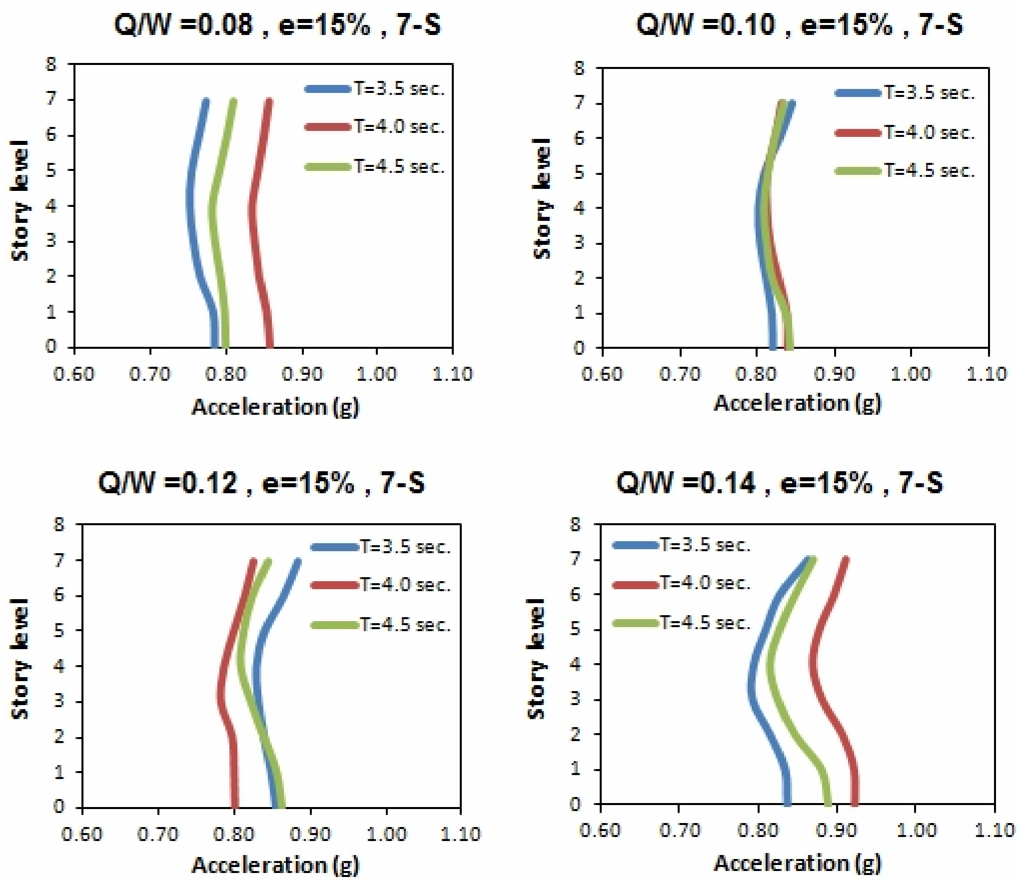

Figure 4.23 Variation of maximum floor accelerations with story level and isolation periods in 7-S buildings for e=15%.

Table 4.1 Percentage of increase in floor accelerations between Q/W =0.08 and Q/W=0.14 in 3-S and 7-S buildings.

3-S BUILDINGS			7-S BUILDINGS		
e %	T (sec.)	% of increase in accelerations from Q/W=0.08 to Q/W=0.14	e %	T (sec.)	% of increase in accelerations from Q/W=0.08 to Q/W=0.14
0	3.5	12.9	0	3.5	48.4
0	4	26.6	0	4	61.7
0	4.5	41.2	0	4.5	60.0
5	3.5	25.1	5	3.5	23.1
5	4	14.5	5	4	23.5
5	4.5	29.5	5	4.5	33.7
10	3.5	8.8	10	3.5	16.6
10	4	9.6	10	4	13.6
10	4.5	13.4	10	4.5	21.3
15	3.5	12.4	15	3.5	10.3
15	4	11.7	15	4	7.6
15	4.5	16.3	15	4.5	9.7

CHAPTER 5

SUMMARY AND CONCLUSION

This study has been conducted to evaluate the effects of bi-directional earthquake excitations of near fault records on the response of asymmetric base isolated buildings. The variation of maximum base shear, torsional moments, isolator displacements, floor accelerations and displacements are also compared based on the level of eccentricities. Non-linear response history analyses (RHA) are carried out for different range of effective damping of LRB and isolation period and for 11 different earthquake ground motions. The building systems are modeled in SAP2000 program. The isolation bearings are modeled as non-linear link element and superstructure is modeled as elastic. Effect of asymmetry in superstructure is examined considering different eccentricity at each floor level. Moreover, the isolator displacements computed from nonlinear response history analysis (RHA) are compared with the isolated displacements computed from simplified method.

Based on the numerical results, conclusions can be drawn as follows:

- The effect of level of eccentricity on normalized base shear is negligible for both 3-S and 7-S buildings. The change in superstructure in terms of story level is also an insignificant parameter for base shear response of isolated buildings. Moreover, effective damping is also an insignificant parameter for calculation of base shear. The normalized base shear values of buildings corresponding to Q/W =0.14 represent the independency of base shear to level of eccentricity. Although V_{RHA} / W ratio decreases from 0.25W to 0.20 W for Q/W=0.14 for 3-S and 7-S buildings, the decrease is small and can be neglected.

- The torsional amplification of moments is higher with increasing level of eccentricity. $M_{torsional(e=10\%)}$ values of asymmetric 3-S and 7-S RC buildings are about 1.95 times higher than the values of $M_{torsional(e=5\%)}$. Moreover, $M_{torsional(e=15\%)}$ values of buildings are between 170% and 183% of $M_{torsional(e=5\%)}$ for different isolation periods. In addition to that, decreasing isolation period also leads to increase in torsional moment.

- Isolator displacements are independent from superstructure in terms of story height since nonlinear RHA results of isolator displacements for 3-S and 7-S buildings are nearly same. On the other hand, increasing effective damping results in decreasing isolator displacements. These decreases are about 36%, 32%, 32% and 31% for symmetric and asymmetric 3-S and 7-S RC buildings corresponding to e=5%, 10% and 15%, respectively. These given ratios are independent from isolation periods.

- An increase in isolation period has increasing effects on isolator displacements. When isolation periods increase from 3.5 sec to 4.5 sec, D_{RHA} values increase about 12%, 14%, 14% and 13% for symmetric and asymmetric 3-S and 7-S RC buildings corresponding to e=5%, 10% and 15%, respectively.

- Simplified method of analysis (ELF) procedure is conservative in terms of isolator displacements. Since simplified method also considers the bi-directional and asymmetry effect, the overestimation of isolator displacements are nearly same and about 10% for each 3-S and 7-S symmetric and asymmetric RC buildings.

- $(D_{RHA})_{ecc}/D_{RHA}$ ratios generally increase slightly when isolation periods increase for 3-S buildings . They are in between 1.15-1.30, 1.35-1.55 and 1.55-1.70 depending on Q/W ratio for e=5%, 10% and15%, respectively. The decreasing tendency of $(D_{RHA})_{ecc}/D_{RHA}$ with increasing damping, observed in 3-S buildings, diminish in 7-S buildings. In addition to that, the isolation periods become ineffective on $(D_{RHA})_{ecc}/D_{RHA}$.

- Seismic isolation reduces the floor displacements with respect to base. However, the relative floor displacements are relatively low. Effective damping has decreasing effect on displacements. In contrast to that high eccentricities result in higher displacements.

- Increasing isolation periods has reducing effect on floor accelerations through the height of buildings in symmetric 3-S RC buildings. On the other hand, the decreasing effect of increasing isolation periods is only valid for 7-S RC symmetric buildings with Q/W ratio equals to 0.08 and 0.10.

- There are remarkable increases between floor accelerations of symmetric and asymmetric buildings. However, the rate of increase in accelerations decreases with increasing level of eccentricity for both 3-S and 7-S buildings. The $Acc_{(e=5\%)}/Acc_{(e=0\%)}$, $Acc_{(e=10\%)}/Acc_{(e=15\%)}$ and $Acc_{(e=15\%)}/Acc_{(e=10\%)}$ are between 1.95 ~3.17, 1.05 ~1.36 and 1.02 ~1.14 for 3-S buildings and 1.22 ~2.62, 0.97 ~1.30 and 1.01 ~1.24, respectively.

- The influence of increasing effective damping on floor accelerations is also increasing considering the response of both 3-S and 7-S buildings in terms of normalized floor accelerations corresponds to Q/W=0.14 with floor accelerations of isolation buildings with Q/W =0.08. In symmetric buildings, the increase is directly proportional to isolation periods for both 3-S and 7-S buildings. On the other hand, with the increase in the level of eccentricity the increment percentage of maximum Acc become independent from isolation periods. The percentage of increase of Acc for 7-S symmetric building is relatively high compared to 3-S symmetric buildings.

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