INFLUENCE OF THE SHEAR WALL AREA TO FLOOR AREA RATIO ON THE SEISMIC PERFORMANCE OF EXISTING REINFORCED CONCRETE BUILDINGS

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ABSTRACT

INFLUENCE OF SHEAR WALL AREA TO FLOOR AREA RATIO ON THE SEISMIC PERFORMANCE OF EXISTING REINFORCED CONCRETE BUILDINGS

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An analytical study is performed to evaluate the influence of shear wall area to floor area ratio on the behavior of existing mid-rise reinforced concrete buildings under earthquake loading. The seismic performance of five existing school buildings with shear wall ratios between 0.00% and 2.50% in both longitudinal and transverse directions and their strengthened counterparts are evaluated. Based on the structural properties of the existing buildings, additional buildings with varying shear wall ratios are designed. Consequently, twenty four buildings with different floor plans, number of stories, cross-sectional properties of the members and material strengths are acquired. Nonlinear time-history analyses are performed for all buildings by utilizing the software program, SAP2000 v14.2.0. under seven different ground motion records. The results indicated that roof drifts and plastic deformations reduce with increasing shear wall ratios, but the rate of decrease is lower for higher shear wall ratios. Buildings with 1.00% shear wall ratio have significantly lower roof drifts and plastic deformations when compared to buildings with 0.00% or 0.50% shear wall ratio. Roof drifts and plastic deformations are minimized when the shear wall ratio is increased to 1.50%. After this limit, addition of shear walls has only a slight effect on the seismic performance of the analyzed buildings.

Keywords: Shear Wall Ratio, Seismic Performance, Reinforced Concrete Structures, Nonlinear Time History Analysis

ÖZ

PERDE DUVAR ALANININ KAT ALANINA ORANININ MEVCUT BETONARME BİNALARIN DEPREM YÜKLERİ ALTINDAKİ YAPISAL PERFORMANSLARINA ETKİSİ

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Betonarme perde duvar alanlarının kat alanına oranlarının, mevcut orta katlı betonarme binaların deprem yükleri altındaki davranışlarına etkisini değerlendirmek için analitik bir çalışma yapılmıştır. Birbirine dik her iki yönde %0.00 ve %2.50 arasında değişen betonarme perde duvar oranlarına sahip, beş mevcut okul binası ve bunların güçlendirilmiş hallerinin yapısal performansları incelenmiştir. Mevcut binaların yapısal özellikleri göz önüne alınarak farklı perde oranları olan yeni binalar tasarlanmıştır. Böylece kat planları, kat sayıları, yapısal elemanlarının kesit özellikleri ve malzeme dayanımları farklı olan yirmi dört bina oluşturulmuştur. Bu binaların doğrusal olmayan zaman tanım alanı analizleri, SAP2000 v14.2.0 yazılımı kullanılarak yedi farklı yer hareketi kaydı altında yapılmıştır. Sonuçlar, perde duvar oranları arttıkça çatı katı ötelenmelerinin ve plastik deformasyonların azaldığını göstermiştir, fakat bu düşüş oranı, yüksek perde duvar oranları için daha azdır. Perde duvar oranı %1.00 olan bir bina, bu oranın %0.00 veya %0.50 olduğu binalara kıyasla oldukça düşük çatı katı ötelenmeleri ve plastik deformasyonlara sahiptir. Perde duvar oranı %1.50'a arttırıldığında, çatı katı ötelenmeleri ve plastik deformasyonlar minimize edilir. Fakat, kullanılan perde duvar oranı bu değeri aştığı takdirde, eklenen perde duvarların, binaların deprem yükleri altındaki performanslarına olan etkisi oldukça azalır.

Anahtar Sözcükler: Perde Duvar Oranı, Deprem Yükü Altındaki Yapı Performansı, Betonarme Yapılar, Doğrusal Olmayan Zaman Tanım Alanı Analizi

To my Dear Family

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First of all, I am grateful and I would like to thank my supervisor, Asst. Prof. Dr. Burcu Burak because of continuous supports, endless guidance and limitless encouragement during the investigation.

Then, I am deeply indebted to my family for support, patience and faith anytime when I need.

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A_c	: Gross cross-sectional area of column
$\sum A_{\chi}, A_{\chi o\lambda}$	: Total cross-sectional area of columns at the base
$\sum (A_{\chi o\lambda})_{\xi}$	: Summation of effective cross-sectional areas of all columns in X-direction
$\sum (A_{\chi o\lambda})_{\psi}$	: Summation of effective cross-sectional areas of all columns in Y-direction
A_{cw}	: Total cross-sectional area of reinforced concrete shear walls in the loading direction at the base
A_d	: Cross-sectional area of diagonal brace
A_{ft}	: Total floor area at the base of a building
A_g	: Gross cross-sectional area of the concrete section
$\sum A_{\gamma}$	: Total cross-sectional area of the shear walls at the base in the loading direction
A_{mw}	: Cross-sectional area of masonry walls in the loading direction at the base
$\sum (A_{\mu\omega})_{\xi}$	: Summation of effective cross-sectional areas of all masonry walls in X- direction
$\sum (A_{\mu\omega})_{\psi}$	: Summation of effective cross-sectional areas of all masonry walls in Y-direction
A_{nx}	: Normalized lateral strength indices in X-direction
A_{ny}	: Normalized lateral strength indices in Y-direction
A_o	: Effective ground acceleration coefficient
A_p	: Floor area of a typical story
$\sum A_{\pi}$	: Total floor area at the base of a building
A_s	: Shear area of the concrete section
$\sum (A_{\sigma\omega})_{\xi}$	: Summation of effective cross-sectional areas of all shear walls in X-direction
$\sum (A_{\sigma\omega})_{\psi}$	: Summation of effective cross-sectional areas of all shear walls in Y-direction
A_w, A_{wall}	: Gross cross-sectional area of shear wall
$\sum A_{\omega}$	: Total cross-sectional area of the structural walls at the base
A_{web}	: Area of the beam web
b_e	: Effective slab width
b_{ex}	: Effective overhanging flange width on each side of the web of the beam section
CI	: Column index
E_c	: Modulus of elasticity of concrete
EI_{eff}	: Effective flexural stiffness
EI_g	: Gross flexural stiffness
f_c’	: Compressive strength of concrete
f_{ctd}	: Design tensile strength of concrete
f_{ywd}	: Design yield strength of the confinement reinforcement
g	: Gravitational acceleration
h_w, H	: Height of the shear wall
I	: Importance factor

I_c	: Moment of inertia of the column
$\sum(I_{\chi o\lambda})_{\xi}$	: Summation of moment of inertias of all columns about their centroidal X-axis
$\sum(I_{\chi o\lambda})_{\psi}$	: Summation of moment of inertias of all columns about their centroidal Y-axis
I_{gw}	: Moment of inertia of gross cross-section of the shear wall
I_{nx}	: Normalized indices about X-axis
I_{ny}	: Normalized indices about Y-axis
$\sum(I_{\sigma\omega})_{\xi}$	: Summation of moment of inertias of all shear walls about their centroidal X-axis
$\sum(I_{\sigma\omega})_{\psi}$	: Summation of moment of inertias of all shear walls about their centroidal Y-axis
l_b	: Span length of the beam
l_p	: Plastic hinge length
l_{tb}	: Transverse distance between beams
I_w, I_{wall}	: Moment of inertia of shear wall
L_w, l_w, D	: Length of shear wall
mnlsi	: Minimum normalized lateral strength indices
mnlstfi	: Minimum normalized lateral stiffness indices
n	: Number of stories
nrs	: Normalized redundancy score
or	: Overhang ratio
P	: Axial load
R	: Earthquake load reduction factor
ssi	: Soft story indices
S(T)	: Spectrum coefficient
t_b	: Slab thickness
V_r	: Shear resistance
V_t	: Total base shear acting on the building
w	: Natural frequency
W	: Total weight of the building including live load participation factor
WI	: Wall index
β	: Newmark's Method parameter, Beta
γ	: Newmark's Method parameter, Gamma
$\rho_{\sigma\eta}$	: Volumetric percentage of horizontal reinforcement of the shear walls
ζ	: Damping ratio
ASCE	: American Society of Civil Engineers
ATC	: Applied Technology Council
CP	: Collapse Prevention
IO	: Immediate Occupancy
LS	: Life Safety
P	: Shear Wall
PGA	: Peak Ground Acceleration
S	: Column
SW	: Shear Wall Ratio
TEC	: Turkish Earthquake Code

CHAPTER 1

INTRODUCTION

1.1 General

Turkey is an earthquake prone country and in the past, earthquakes caused significant damage and loss of life and assets. Istanbul, which has nearly fourteen million population, has a high earthquake hazard risk when compared to other cities in Europe. Based on the information provided by the Turkish Earthquake Code (2007) and TMMOB Chamber of Civil Engineers Report (2010), 66% of the total domains, 71% of the total population and 68% of total the municipalities in Turkey are at the first and second seismic zones. If the third and fourth seismic zones are also considered, approximately 92% of the total domains of Turkey is under the threat of earthquakes. Therefore, increasing the earthquake resistance of structures is essential. One of the most efficient methods to improve the seismic performance of the buildings is the use of properly designed and detailed reinforced concrete shear walls. Therefore, many experimental and analytical studies have been performed to investigate the behavior of shear walls under earthquake loading and their effect on the seismic performance of reinforced concrete structures.

In the past, high intensity earthquakes occurred in Turkey such as Kocaeli (1999), Düzce (1999), Erzincan (1939), Gediz (1970) Earthquakes damage lots of structures, especially public buildings. According to TMMOB Chamber of Civil Engineers Report (2010), a significant percentage of the hospitals and schools in Izmir and Istanbul should be retrofitted following the requirements of Turkish Earthquake Code (2007). In the strengthening of reinforced concrete buildings, addition of reinforced concrete shear walls is commonly utilized all over the world to increase the lateral load capacity and the stiffness of the structure.

Reinforced concrete buildings with substantial amount of reinforced concrete shear walls exhibited satisfactory performance under severe earthquakes such as Nicaragua (1972), Chile (1960), Armenia (1988), Venezuela (1967) Earthquakes without significant damage even when the shear walls had poor detailing or constructed with low strength materials (EERI Report (2005)). Badaux and Peter (2000) stated that shear wall buildings have considerable stiffness, lateral resistance and limited interstory distortions. Fintel (1995) noted that even when cracking was observed in shear walls, they were very efficient in controlling structural and nonstructural damage in buildings during the Chile Earthquake. Thus, using adequate shear wall area to floor area ratios is essential to have improved seismic resistance of reinforced concrete buildings.

1.2 Objective and Scope

Construction industry is one of the most important and booming industries in the whole world but especially in the developing countries like Turkey. In Turkey, the number of structures which were constructed in the last twenty five years, is more than the number of structures which had been constructed earlier. The buildings constructed in the last twenty five years are mostly masonry and reinforced concrete buildings that have poor detailing and low construction quality (TMMOB Chamber of Civil Engineers Report (2010)). Since Turkey is in a highly seismic zone, most of these buildings are damaged under earthquake loading or have insufficient capacity. Therefore, they should be strengthened based on the current requirements of the Turkish Earthquake Code (2007).

As mentioned earlier, addition of shear walls is one of the most efficient solutions to improve the seismic performance of a building. Shear walls are load bearing members of the structural system which carry the lateral loads induced by the earthquakes and they provide substantial energy dissipation capacity. Since the use of shear walls limits the roof and interstory drifts, the observed structural damage under earthquake loading is minimized. In this study, the influence of shear walls on the seismic performance of structures is investigated. For this purpose, five different mid-rise existing school buildings and their strengthened counterparts are inspected. This analytical study focuses on 3 to 5 story buildings, since a prior study by Burak and Çömlekoğlu (2012) indicated that the seismic performance of 5-story buildings is significantly affected by the variation in shear wall area to floor area ratio. Based on the structural properties of the existing school buildings, additional buildings are designed with increasing shear wall ratios. Thus, twenty four different buildings are modeled that have different shear wall ratios, floor plans, torsional irregularities, cross-sectional properties of members and number of stories. The floor area of the selected existing buildings varies from 320 m^2 to 777 m^2 and the floor height is in between 3.10 m and 3.45 m. The shear wall ratios range between 0.00% and 2.50% in both longitudinal and transverse directions of the building plans. The software program, SAP2000 v14.2.0 (2009) is utilized to perform nonlinear time history analysis of all buildings under seven different ground motion records.

The seismic performance of the buildings are evaluated by considering the average results obtained from the application of the selected ground motion records in terms of the observed roof drifts and plastic deformations in the members, percentage of the yielded members, base shear versus roof drift relationships and the percentage of the base shear force carried by the shear walls.

1.3 Thesis Outline

Seismic behavior of mid-rise existing reinforced concrete buildings with varying shear wall ratios is presented in this thesis. Information on the influence of shear walls on the seismic performance of structures and the objectives of this analytical study are provided in Chapter 1. Literature review is presented in Chapter 2, which involves classification and description of shear walls and the analytical modeling methods for shear wall structures. In addition, the relationship between shear wall indices and drift is mentioned in this chapter. Chapter 3 specifies the description and structural properties of the existing and designed school buildings including the material strengths, cross-sectional properties of the members, dimensions, applied loads, and the analytical modeling procedure followed to model all buildings in detail. Moreover, the selection of seven different ground motion time histories that are applied to the structures is introduced in this chapter. Chapter 4 discusses the analytical results and the findings of this study. Chapter 5 provides conclusions and recommendations for future research.

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CHAPTER 2

LITERATURE REVIEW

2.1 General Properties, Description and Classification of Shear Walls

Two main types of structural systems, which are concrete frame systems and concrete frame-wall systems, are used by civil engineers to resist external vertical and horizontal loads for concrete structures. ATC 40 (1996) states that both vertical and horizontal loads are carried by frames in concrete frame systems; but in concrete frame-wall systems, shear walls are generating the lateral resistance of the building and also these members can carry some local vertical loads. In Eurocode 8 (2003), structural systems of reinforced concrete buildings are divided into six categories as frame system, dual system, ductile wall system, system of large lightly reinforced wall, inverted pendulum system and torsionally flexible system. On the other hand, in Turkish Earthquake Code (2007), three different structural systems are mentioned as concrete frame systems, concrete wall systems with or without openings and concrete frame-wall systems. Concrete frame-wall systems are recommended by academicians, researchers and engineers especially in earthquake prone regions to increase seismic resistance and stiffness of the structures by using shear walls. Gulkan and Utkutuğ (2003) indicated that reinforced concrete shear wall buildings have not collapsed after severe earthquakes and most of these buildings satisfied immediate occupancy acceptance criteria after severe earthquakes.

Experimental and analytical research demonstrated that concrete frame-wall buildings have displayed better seismic performance and resistance compared to concrete frame systems (Bertore (1987)). Fintel (1995) concluded that buildings with shear walls had superior performance under Caracas Earthquake in Venezuela. Seismic performance of the building, which is the performance of the building when subjected to earthquake loading, is based on strength, stiffness and deformation capacity of the building. The use of reinforced concrete shear walls increases the stiffness of the structures and therefore limits the observed distortion and drift values. Furthermore, configuration of the shear walls is important and if shear walls are located symmetrically with respect to the center of mass of the building, there will be a uniform distribution of inelastic deformations during seismic activities.

Turkish Earthquake Code (2007) defines shear walls as vertical load carrying members which have a minimum length to thickness ratio of 7 and a minimum thickness of 0.2 m. However, Gulkan and Utkutuğ (2003) stated that a member with a thickness of 0.2 m and length of 1.4 m cannot be accepted as a shear wall for a 5 story building or a 14 m tall building due to the ratio of wall height to wall length which is 10. Some building codes also classify the shear walls based on the aspect ratios of the shear walls defined as the ratio of height, h_w to the length, l_w. Depending on the aspect ratio of shear walls, ASCE 41 (2007) classifies shear walls as squat shear walls or short walls which have an aspect ratio of 1.5 or less, slender shear walls which have an aspect ratio of 3.0 or more and intermediate shear walls which have an aspect ratio in between 1.5 and 3.0. On the other hand, ATC 40 (1996) states that squat shear walls have a height to length ratio of 2 or less, and slender shear walls have an aspect ratio of 4 or more. Aejaz and Wight (1990) defined the aspect ratio of low rise or squat walls as 0.5 or less and of long or slender walls as 2.0 or more. As expected, the behavior of squat shear walls and slender shear walls are controlled by shear and flexure, respectively and that of intermediate shear walls are influenced by combined shear and flexure. In general, the shear walls, which are more likely to fail under shear, are called squat walls and the ones, which are more likely to fail under flexure, are called slender walls. Typical side views of squat and slender walls can be seen in Figure 2.1.

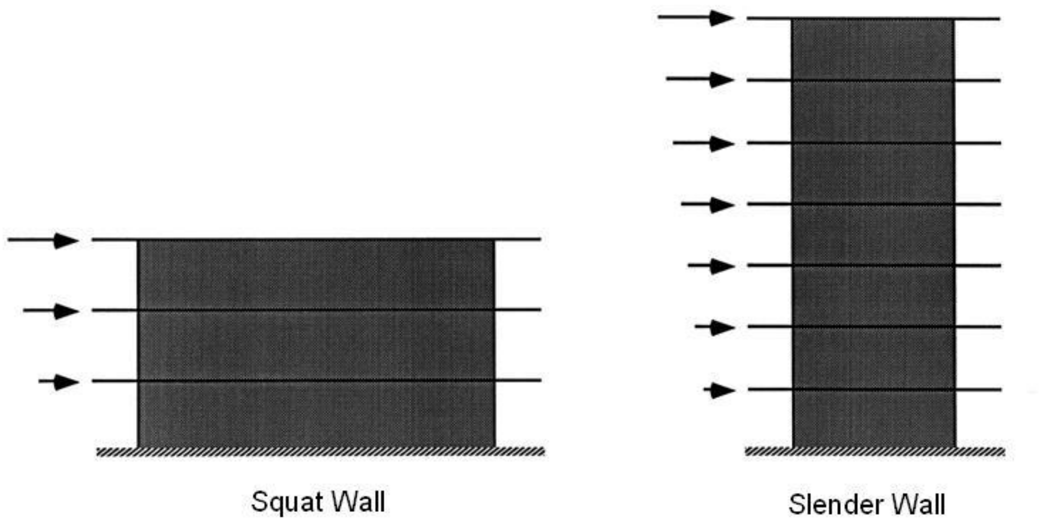


Figure 2.1 Types of Structural Walls (Aejaz and Wight (1990))

Shear failure of walls is undesirable because it is a brittle failure type and thus, the main topic of research on squat shear walls is to change mode of failure from shear to flexure. Also, experimental studies on deep beams are utilized to understand the performance of squat shear walls, because these types of shear walls have geometrical properties similar to deep beams. By properly designing and detailing web reinforcement for squat shear walls, ductility demands of squat walls can be reduced and inelastic flexural response can be obtained (Aejaz and Wight (1990)). On the other hand, slender shear walls fail under flexure and can be used in medium to high-rise buildings. Oesterle et al. (1979) demonstrated that the inelastic deformation capacity of buildings with slender shear walls is sufficient and as expected theoretically; plastic hinges of the wall specimens had been formed at the base due to the yielding of flexural reinforcement. Proper detailing of horizontal and vertical reinforcement of slender shear walls increase the stiffness and lateral resistance of the reinforced concrete shear walls and hence the reinforced concrete buildings; but also, the necessity of diagonal reinforcement is considered by Illiya and Bertero (1980). The test results showed that adding diagonal reinforcement to shear wall specimens improves the seismic behavior (Illiya and Bertero (1980)).

Another structural wall type is coupled walls (Figure 2.2). Coupled wall systems have huge openings due to architectural or technical needs and these systems consist of shear walls and beams connecting the shear walls which are called coupling beams. These beams absorb energy and provide energy dissipation capacity and for improving the seismic performance of these structural systems, energy dissipation capacity of the coupling beams should be increased (Aejaz and Wight (1990)). Therefore, reinforcement detailing of the coupling beams is extremely important to improve seismic behavior and coupling beams with diagonal reinforcement provide higher strength, stiffness and energy dissipation capacity to the structural system compared to the ones with conventional reinforcement according to ASCE 41 (2007) recommendations. Like coupled wall systems, in pierced wall systems in which there are small openings on the structural walls; but these openings do not influence the seismic performance of the walls significantly (Aejaz and Wight (1990)). Figure 2.2 shows different structural wall systems below.

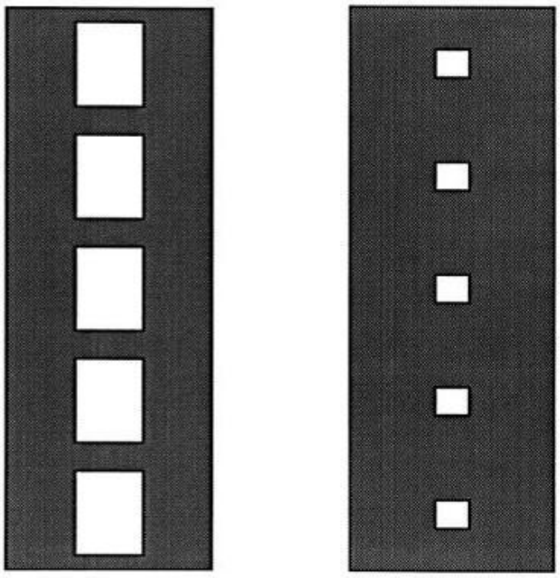


Figure 2.2 Types of Structural Wall Systems (Aejaz and Wight (1990))

Reinforced concrete shear walls are also classified according to the cross-sectional shape of walls as rectangular, flanged, barbell, channel, T or L-shaped, etc. structural walls. Barbell shaped shear walls are reinforced concrete shear walls with large stiff boundary elements and thin webs which are exposed to excessive shear forces; but properly detailed and confined boundary elements can resist higher shear forces and axial loads to delay inelastic bar buckling and to retain shear strength unlike the web of the structural walls. At high stress and deformation levels, web crushing can be observed on the structural walls (Oesterle et al. (1979)). Furthermore, Oesterle et al. (1979) demonstrated that properly designed and detailed boundary elements can improve the strain capacity of concrete, increase the shear capacity and stiffness of the wall and prevent inelastic buckling of vertical reinforcement. The seismic behavior of rectangular structural walls is broadly studied; however, there is a lack of information on the behavior of nonrectangular reinforced concrete shear walls such as channel, T and L-shaped walls. Due to architectural purposes, these types of walls are located around hallways or elevator shafts. The mode of failure for T-shaped shear walls is combined shear and flexure, concrete crushing occurs at the bottom of the web and longitudinal web reinforcement reaches its deformation limit (Pin-Le and Qing-Ling (2011)). T-shaped structural walls can have higher lateral load resistance compared to rectangular and L-shaped shear walls, but load bearing capacity of T-shaped shear walls is insufficient. On the other hand, T-shaped shear walls have adequate ductility, energy dissipation and deformation capacities. Therefore, increasing longitudinal reinforcement ratio of the web edge is the most effective method to enhance the load bearing capacity of these types of walls (Pin-Le and Qing-Ling (2011)). The most critical direction of channel-shaped or U-shaped structural walls is the diagonal direction (Beyer et al. (2008)). The diagonal direction of these structural walls should also be considered to obtain accurate analytical results while designing channel-shaped structural walls (Beyer et al. (2008)). Finally, the L-shaped shear walls exhibit a more improved seismic performance than that of rectangular shear walls and are usually used at the corners of the buildings (Pin-Le and Qing-Ling (2011)).

Typical deficiencies that can be observed in the design of reinforced concrete wall-frame systems are vertical discontinuity, weak stories, shear cracking, and diagonal tension/compression, etc. (ATC 40 (1996)). Shear walls of a structure should be placed from the foundation level up with no vertical discontinuity (Figure 2.3) in the building. If columns or shear walls are removed to create space for parking and shops, a weak story is formed, which is a very common deficiency in Turkey. In these types of structures, the stiffness and strength vary from floor to floor significantly (ATC 40 (1996)).

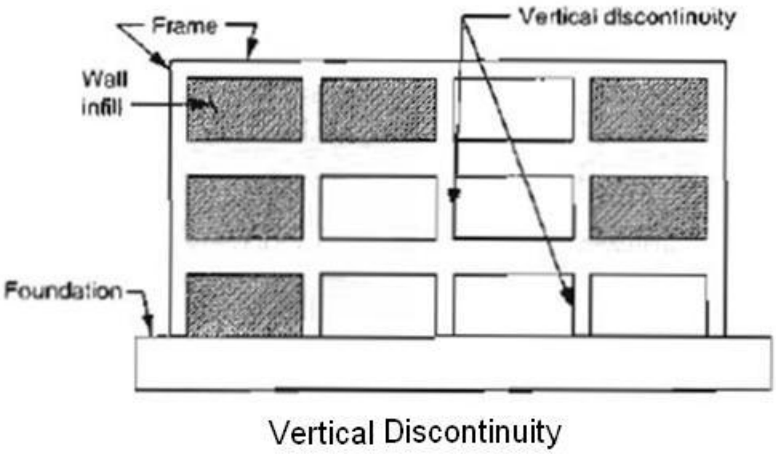


Figure 2.3 Typical Vertical Discontinuity (ATC 40 (1996))

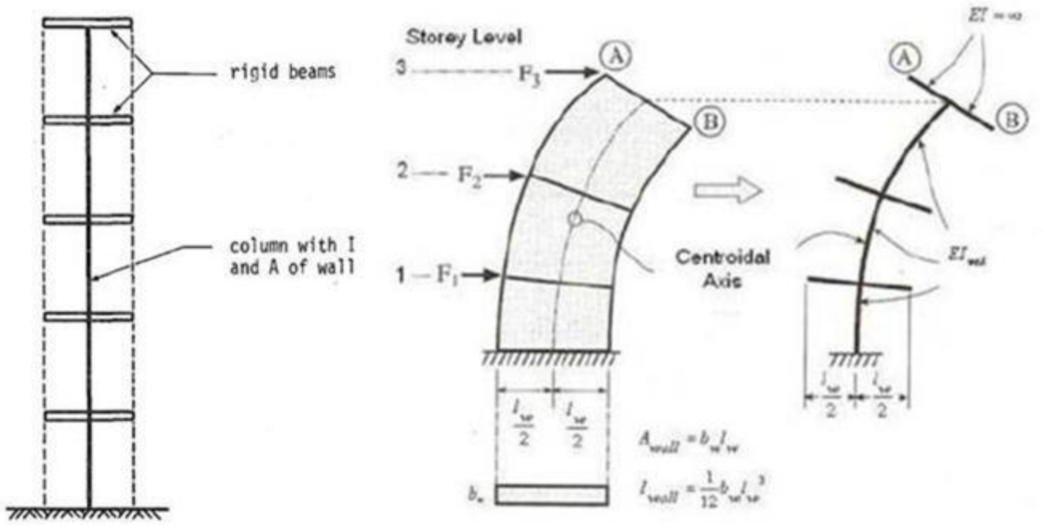
2.2 Modeling of Shear Walls

Reinforced concrete shear walls provide great stiffness and seismic resistance to the structure and limit the interstory drifts. Therefore, shear walls are widely used in high seismicity regions in the design of new structures and the rehabilitation of existing ones. It is important to accurately model the nonlinear behavior of the structural walls. Galal and Sokkary (2008) stated that while modeling structural walls, cross-sectional dimensions, aspect ratio, axial-flexure interaction, reinforcement ratio, bond properties, reinforcement detailing of the boundary elements, influence of connecting members, rigid-body rotation and the flexural capacity based on the shear capacity of the shear wall should be considered. Flexural response of the shear walls is easier to predict, however it is harder to represent combined flexural and shear response of the structural walls accurately. The analytical models of reinforced concrete shear walls can be classified into two categories as microscopic and macroscopic models. Macroscopic models are based on the test results and observations that consider the overall response of a shear wall using constituents of the wall such as concrete, reinforcement and bond between concrete and reinforcement. On the other hand, microscopic models are based on solid mechanics and consider the local behavior of the structural walls in detail. And finally, meso models are placed between these two major model groups and have some similar properties as both macroscopic and microscopic models (Linde (1993)).

The commonly used macroscopic models are equivalent beam element model, vertical line element model which includes varying number of springs, truss element model, braced frame analogy and braced wide column analogy. The microscopic methods can be categorized as finite element method and fiber method. Due to the simplicity, efficiency and practicality of macroscopic models, these models are generally utilized in analysis.

2.2.1 Macroscopic Models of Shear Walls

One of the commonly used macroscopic models is the equivalent beam element model, also named as the wide column analogy. In this model, the shear wall is defined as a line element at its centroidal axis, which has the same moment of inertia and cross-sectional area of the shear wall, and infinitely rigid beams placed at the floor levels of the structure connect this line element to the adjoining members. In Figure 2.4.a, simple representation of wide column analogy model is given. Rigid beam length is taken as half the shear wall length. In this model, it is assumed that plane sections at the floor levels remain plane after the application of lateral loads (Atımtay (2001)). The deformation of shear walls under lateral loading obtained by the equivalent beam element model can be seen in Figure 2.4.b. Equivalent beam element model is simple and have only a few degrees of freedom to compute the seismic response of shear walls; but shifting of the neutral axis due to flexural cracking and yielding of the wall reinforcement cannot be taken into account in this model and therefore, the strain distribution of the wall is unrealistic (Linde (1993)).



a.Wide Column Analogy (Smith et al. (1984)) b.Equivalent Beam Element Model (Atımtay (2001))

Figure 2.4 Equivalent Beam Element Method

Truss element model is another macroscopic model which consists of two vertical boundary truss elements at the ends of the shear walls, a horizontal rigid element represented as shear reinforcement and at least one diagonal truss element as shown in Figure 2.5. In this model, vertical boundary columns resist the acting moment, horizontal rigid beams carry the tension and diagonal truss elements carry compression under lateral loading (Galal and Sokkary (2008)). The shear wall is modeled as a statically determinate truss and the shear response of the walls under lateral loading is studied especially. Therefore, this model cannot predict the overall seismic response of the structural walls.

In the braced wide column analogy model, like wide column analogy, there are rigid beams at the floor levels and a column element at the centroidal axis of the structural wall; but in addition to those, there are diagonal braces with hinged ends connected to the beam elements in this model. Bending, shear and axial stiffness of the structural wall should be considered to have an accurate model and the stiffness of the columns and shear wall are determined following the recommendations by Smith and Girgis (1984). In this model, axial force, shear force and moment capacity of the structural wall are determined using bending moment, shear force and axial force on the column and axial force on the diagonal braces. Simple sketch of braced wide column analogy is given in Figure 2.6.a. Another macroscopic model by Smith and Girgis (1984) is the braced frame analogy shown in Figure 2.6.b. In this model, there are two column elements at each end of the shear wall, rigid beam elements at each floor level and diagonal braces with hinged ends. Like braced wide column analogy, bending, shear and axial stiffness of the shear walls are determined and forces on the elements are used to obtain the shear wall stresses. Braced frame analogy is demonstrated to be more accurate than the braced wide column analogy and both of these models are appropriate for planar and nonplanar shear walls.

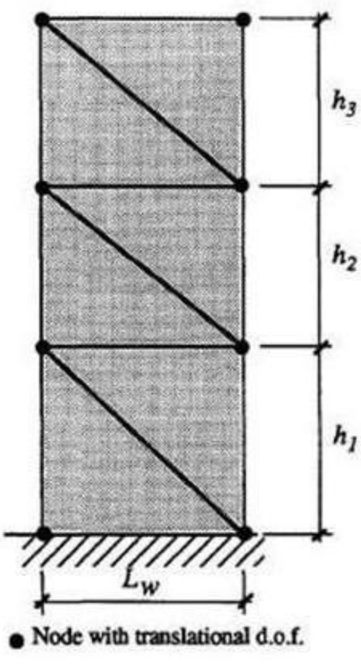
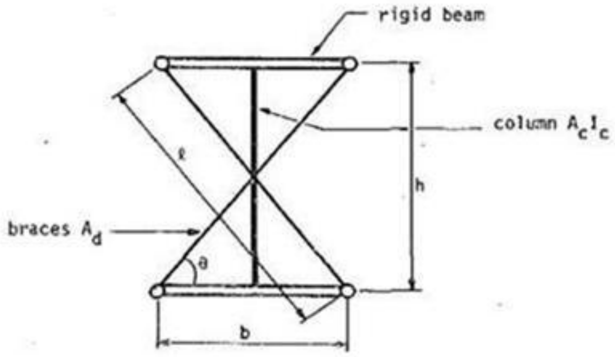
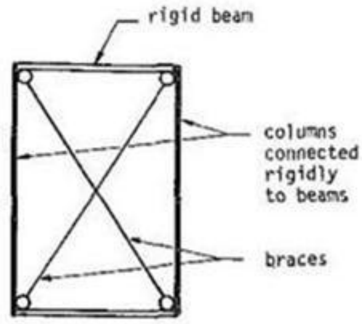


Figure 2.5 Truss Element Model (Linde (1993))





a.Braced Wide Column Analogy

b.Braced Frame Analogy

Figure 2.6 Braced Macroscopic Models of Shear Walls (Smith and Girgis (1984))

Moreover, continuum method is used as a macroscopic approach for multistory buildings to determine the maximum roof displacement and maximum interstory drift ratio during earthquakes. In this model, Miranda and Reyes (2002) used a flexural cantilever beam and a shear cantilever beam with nonuniform lateral stiffness distribution along the height of the structural wall. In-plane representation of the structural system is given in Figure 2.7, where the connecting links are assumed to be axially rigid beams. Therefore, the horizontal deflections at each floor level are the same under lateral loads. With the use of continuum model, it is shown that the ratio of spectral displacement to maximum roof displacement is not significantly affected by varying lateral stiffness along the height of the multistory building; but the ratio of maximum interstory drift ratio to roof drift ratio is influenced slightly (Miranda et al. (2002)).

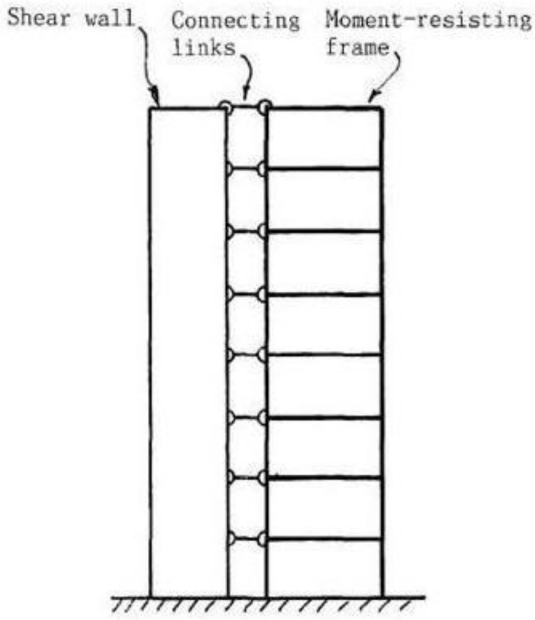


Figure 2.7 Continuum Model for a Multistory Building (Miranda and Reyes (2002))

Three vertical line element model (TVLEM) by Kabeyasawa et al. (1983) consists of horizontal rigid beams at each floor level, like equivalent beam element model and braced wide column model, two vertical truss elements at each end of the shear wall having the axial stiffness of boundary columns and one central vertical line element representing shear wall web. In this model, five springs are placed on three vertical line elements as shown in Figure 2.8. Nonlinear axial springs are used for each vertical truss element at the ends of the wall, which stand for the axial stiffness of the boundary elements. Horizontal, vertical and rotational springs are located on the base of the central vertical element. Horizontal spring represents the shear capacity of the shear wall and flexural capacity is represented by the rotational spring at the base of the central vertical element and axial springs of vertical truss elements at the ends of the shear wall. Three vertical line element model determines deformation and strength of the shear wall under bending by the help of two outer vertical truss elements and one central vertical line element. One of the outer vertical truss elements carries tension and the other one carries compression under lateral loads. Bending deformation of the shear wall is determined based on the extension of the boundary column, which carries tension. Three vertical line element model can be used to determine both the overall behavior of the structural system and the member behavior of the reinforced concrete shear wall (Kabesayawa et al. (1983)).

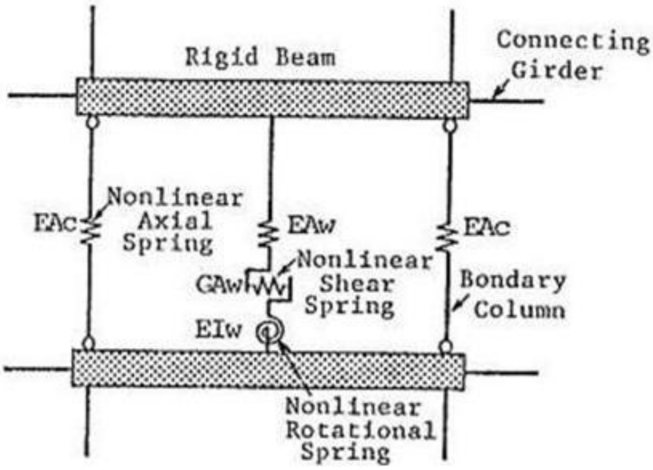
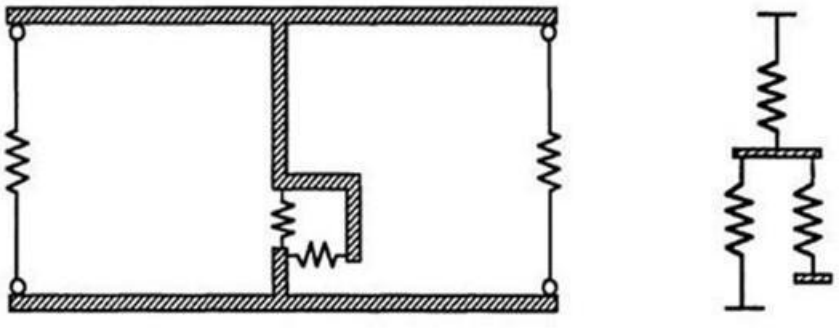


Figure 2.8 Three Vertical Line Element Model (Kabesayawa et al. (1983))

The three vertical line element model by Linde (1993) is similar to the one by Kabesayawa et al. (1983) but without the rotational spring at the base of the central line element. There are four axial springs in this model as shown in Figure 2.9.a. In this model, two outer vertical springs with the central vertical spring define the flexural behavior of the shear wall and the horizontal spring at the base of the central line element provide the shear behavior. With the use of three vertical line element model by Linde (1993), nonlinear performance of the shear wall can be properly predicted. Another three vertical line element model by Vulcano and Bertero (1986) is called the axial element in series model (AESM), because axial springs are connected in series (Figure 2.9.b). Axial stiffness of the boundary elements of the wall and the bond between reinforcement and concrete is represented by the upper one-component element and the lower two-component element stands for the axial stiffness of the boundary elements of the wall with no bond between reinforcement and concrete (Vulcano and Bertero (1986)). This model predicts the flexural response of the structural wall, but shear behavior cannot be investigated using axial element in series model (AESM).



a. Three Vertical Line Element Model　　　　b. Axial Element in Series Model

Figure 2.9 Vertical Line Element Models (Linde (1993))

In the multiple vertical line element model (MVLEM) by Vulcano et al. (1986), rotational and vertical springs at the base of the central line element are removed and multiple vertical springs at multiple vertical trusses are placed into the model (Figure 2.10). Outer vertical truss elements at the end of the shear wall, which simulate the boundary elements of the wall, and rigid beams at the floor levels are considered in the model like three vertical line element model by Kabeyasawa et al. (1983) and Linde (1993); but only a horizontal axial spring is located at the central line element of the model to provide the inelastic shear behavior of the wall. Other axial vertical springs provide the combined axial-flexure behavior of the shear wall. In multiple vertical line element method, gradual yielding of the vertical reinforcement of the shear wall can be examined more accurately and more realistically compared to the three vertical line element methods; but due to having multiple vertical springs, this model is relatively more complicated.

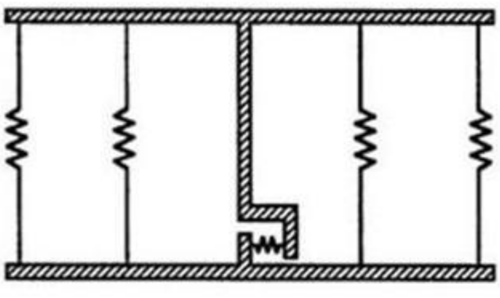
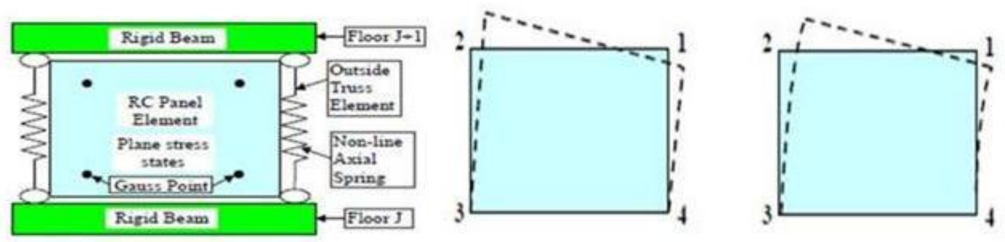


Figure 2.10 Multiple Vertical Line Element Method by Vulcano et al. (1986)

The panel element model by Chen et al. (2000) can predict the overall behavior of shear walls under lateral loading. In this model, there are two outer vertical truss elements with axial springs at each end of the shear wall and infinitely rigid beams at each floor level (Figure 2.11.a). Also, a panel, which represents the shear wall web, is used as both isoparametric and incompatible rectangular element in the models (Figures 2.11.b and 2.11.c). For shear walls that fail due to flexure, shear deformation of the structural wall is overestimated by using isoparametric element in panel element models, but incompatible elements in panel element models predict the shear and flexural deformations accurately compared to isoparametric elements. The analytical results of both elements show good correlation with the experimental results (Chen et al. (2000)).



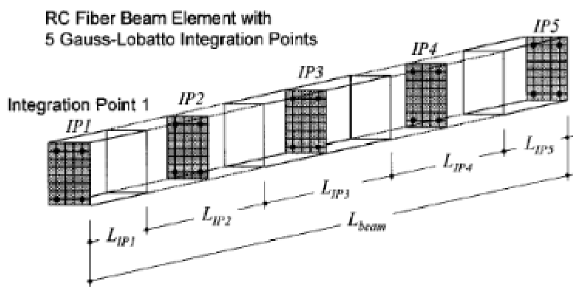
a. Panel Element Model b. Isoparametric Element c. Incompatible Element

Figure 2.11 Panel Element Model by Chen et al. (2000)

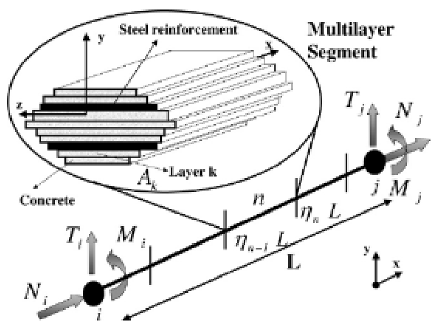
2.2.2 Microscopic Models of Shear Walls

Microscopic approach is also used to model the reinforced concrete structural walls in which the behavior of the individual materials, reinforcement and concrete, and the bond between them is examined. Microscopic approach is feasible when detailed evaluation of local response of shear walls is needed; but this approach is time consuming and limited to the seismic behavior of the individual elements of the structural system. There are a few types of microscopic models such as finite element model (FEM) and fiber model and representations of these models are given in Figures 2.12.a and 2.12.b.

Finite element method (FEM) is commonly used to determine the seismic behavior of the structural walls using finite number of small elements and this method obtains both global and local behavior of the shear wall. Moreover, Lepage et al. (2006) stated that onset of yielding, yield strength, initial stiffness and displacement response of the shear wall can be determined by finite element method (FEM) under seismic loading. In fiber model, as in finite element method, the member is divided into several small elements to get the nonlinear behavior of the shear wall. By the use of the fiber model, moment curvature relationship of the structural wall at each load increment, axial load – bending moment relationship and flexibility distribution along the length of the shear wall can be determined (Galal et al. (2008)).



a.Fiber Method



b.Finite Element Method

Figure 2.12 Microscopic Methods (Galal et al. (2008))

2.3 Shear Wall Ratio of the Structures

Shear walls resist lateral loading due to earthquakes and improve the nonlinear behavior of the reinforced concrete buildings and EERI Report (2005) claims that even the shear walls with low material quality and poor detailing, provide significant seismic capacity to the structure. Therefore, using adequate amount of shear walls in the structural system is necessary to have a sufficient seismic resistance.

A simplified method for the evaluation of reinforced concrete low-rise monolithic buildings is recommended by Hassan and Sozen (1997) in which the shear wall ratios and column ratios are used to identify buildings with a high probability of a severe damage. Structural dimensions, shear wall ratio and column ratio are the only required parameters and this method was used on 46 buildings to propose an evaluation procedure. In Figure 2.13, X-axis and Y-axis stand for the column index (CI) and the wall index (WI), respectively. The wall index (WI) is the ratio of the addition of the total cross-sectional areas of reinforced concrete shear walls and a percentage of masonry walls at the base of the building in the loading direction to the total floor area at the base of a building.

Wall index (WI) can be obtained as:

$$WI = \frac{\mathrm{A_{cw}} + \frac{\mathrm{A_{mw}}}{10}}{\mathrm{A_{ft}}} \times 100 \tag{2.1}$$

where,

A_{cw} : Total cross-sectional area of reinforced concrete shear walls in the loading direction at the base,

A_{mw} : Cross-sectional area of masonry walls in the loading direction at the base,

A_{ft} : Total floor area at the base of a building.

The column index (CI) is the ratio of effective column area at the base of a building to the total floor area at the base of a building.

Column index (CI) is computed as:

$$CI = \frac{\frac{A_{col}}{2}}{A_{ft}} \times 100 \quad (2.2)$$

A_{col} : Total cross-sectional area of columns at the base.

Based on these equations, two boundary lines were drawn in Figure 2.13 showing vulnerability level of the buildings against earthquakes. On this graph, the area which is surrounded by the column index axis, wall index axis and Boundary 1, represents the most critical region in terms of vulnerability of the building and the direction of increasing damage is given in the figure. This method is simple and useful for rapid evaluation of existing buildings and preliminary design of new buildings.

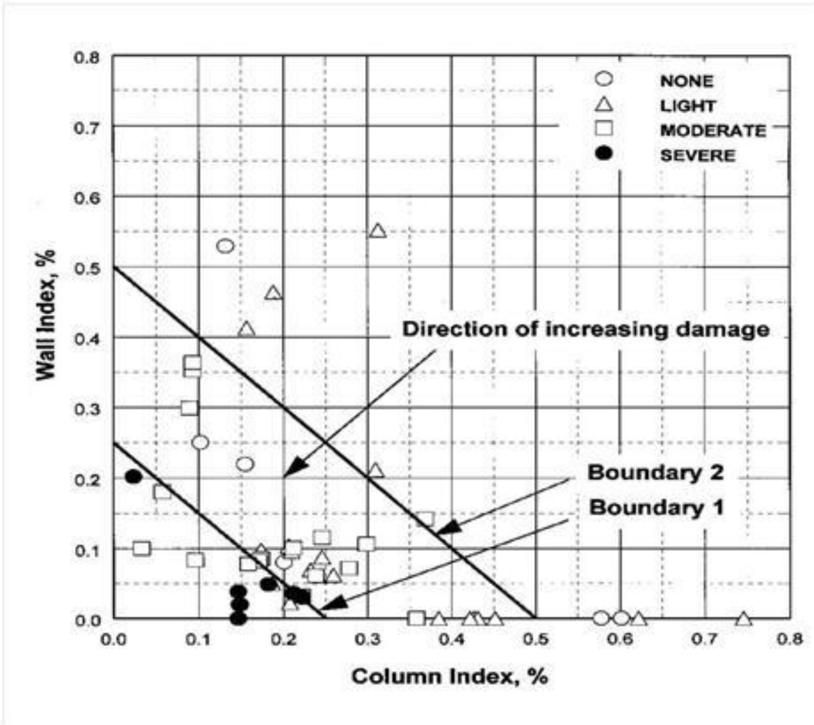


Figure 2.13 Evaluation Method Proposed by Hassan and Sozen (1997)

Ozcebe et al. (2003) also used shear wall ratios in the preliminary seismic vulnerability assessment of reinforced concrete structures. The parameters taken into account in this method are the number of stories (n), minimum normalized lateral stiffness indices (mnlstfi), minimum normalized lateral strength indices (mnlsi), normalized redundancy score (nrs), soft story indices (ssi) and overhang ratio (or). The minimum normalized lateral stiffness indices (mnlstfi) and minimum normalized lateral strength indices (mnlsi) are related to the cross-sectional properties of shear walls and columns.

Minimum normalized lateral stiffness indices (mnlstfi) is computed as:

$$\mathrm{MNLSTFI} = \min(I_{nx}, I_{ny}) \tag{2.3}$$

$$I_{nx} = \frac{\Sigma(\mathrm{I_{col}})_\mathrm{x} + \Sigma(\mathrm{I_{sw}})_\mathrm{x}}{\mathrm{A_{tf}}} \times 1000 \tag{2.4}$$

$$I_{ny} = \frac{\Sigma(\mathrm{I_{col}})_\mathrm{y} + \Sigma(\mathrm{I_{sw}})_\mathrm{y}}{\mathrm{A_{tf}}} \times 1000 \tag{2.5}$$

where,

$\sum(I_{col})_x$ and $\sum(I_{col})_y$: Summation of moment of inertias of all columns about their centroidal X- and Y-axes, respectively,

$\sum(I_{sw})_x$ and $\sum(I_{sw})_y$: Summation of moment of inertias of all shear walls about their centroidal X- and Y-axes, respectively,

I_{nx} and I_{ny} : Normalized indices about X- and Y-axes, respectively,

A_{ft} : Total floor area at the base of a building.

Minimum normalized lateral strength indices (mnlsi) is computed as:

$$\mathrm{MNLSI} = \max(A_{nx}, A_{ny}) \tag{2.6}$$

$$A_{nx} = \frac{\Sigma(\mathrm{A_{col}})_\mathrm{x} + \Sigma(\mathrm{A_{sw}})_\mathrm{x} + 0.1\Sigma(\mathrm{A_{mw}})_\mathrm{x}}{\mathrm{A_{tf}}} \times 1000 \tag{2.7}$$

$$A_{ny} = \frac{\Sigma(\mathrm{A_{col}})_\mathrm{y} + \Sigma(\mathrm{A_{sw}})_\mathrm{y} + 0.1\Sigma(\mathrm{A_{mw}})_\mathrm{y}}{\mathrm{A_{tf}}} \times 1000 \tag{2.8}$$

where,

$\sum(A_{col})_x$ and $\sum(A_{col})_y$: Summation of effective cross-sectional areas of all columns in X- and Y-directions, respectively,

$\sum(A_{sw})_x$ and $\sum(A_{sw})_y$: Summation of effective cross-sectional areas of all shear walls in X- and Y-directions, respectively,

$\sum(A_{mw})_x$ and $\sum(A_{mw})_y$: Summation of effective cross-sectional areas of all masonry walls in X- and Y-directions, respectively,

A_{nx} and A_{ny} : Normalized lateral strength indices in X- and Y-directions, respectively.

The results of preliminary seismic vulnerability assessment of reinforced concrete buildings using this method was checked using available vulnerability records of the 1992 Erzincan Earthquake and the 2002 Afyon Earthquake and very accurate classifications were made.

There are a set of empirical equations proposed by Ersoy (1999), which is used mostly for the preliminary design stage of the structures. These equations were developed based on the seismic vulnerability assessment of the buildings in the past earthquakes such as the 1992 Erzincan, 1995 Dinar and 1998 Ceyhan earthquakes. The two inequalities suggested by Ersoy (1999) are applicable for residential and office buildings that are up to 8 stories:

$$0.5\sum A_c + \sum A_w \geq 0.003\sum A_p \quad (2.9)$$

$$\sum A_w \geq 0.0002\sum A_p \geq 0.01A_p \quad (2.10)$$

where,

$\sum A_c$: Total cross-sectional area of the columns at the base,

$\sum A_w$: Total cross-sectional area of the structural walls at the base,

$\sum A_p$: Total floor area at the base in a building

Tekel (2006) also stated that due to the lack of information on the required shear wall ratio for the structures in the Turkish Earthquake Code (2007), a rule of thumb value, taken as 1.0%, should be used as shear wall ratios in both horizontal directions of a building. The evaluation of this rule of thumb value and the two inequalities of Turkish Earthquake Code (2007) on the shear wall ratio requirements were investigated by Tekel (2006):

$$\sum A_g / \sum A_p \leq 0.002 \quad (2.11)$$

$$V_t / \sum A_g \leq 0.5f_{ctd} \quad (2.12)$$

where,

$\sum A_g$: Total cross-sectional area of the shear walls at the base in the loading direction,

$\sum A_p$: Total floor area at the base of a building,

V_t : Total base shear acting on the building,

f_{ctd} : Design tensile strength of the concrete.

Moreover, the following two equations of Turkish Earthquake Code (2007) are examined by Tekel (2006):

$$V_t = S(T)A_oIW / R \quad (2.13)$$

$$V_r = A_{ch}(0.65f_{ctd} + \rho_{sh}f_{ywd}) \quad (2.14)$$

where,

$S(T)$: Spectrum coefficient,

A_o : Effective ground acceleration coefficient,

I : Importance factor,

W : Total weight of the building considering live load participation factor,

R : Earthquake load reduction factor,

V_r : Shear resistance,

ρ_{sh} : Volumetric horizontal reinforcement ratio of shear walls,

f_{ywd} : Design yield strength of the confinement reinforcement.

Based on the above mentioned Turkish Earthquake Code (2007) equations, Tekel (2006) investigated three cases, In the first case, the equality in Equation (2.11) is considered and following equation is developed:

$$\sum A_g / A_p = 0.002n \qquad (2.15)$$

where,

n : Number of stories,

A_p : Floor area of a typical story.

Table 2.1 is obtained using Equation (2.15) for reinforced concrete structures with shear walls and shear wall ratios based on the number of stories are given in Table 2.1. Structural walls are considered as the only lateral load resisting members of a structure and as it can be concluded from Table 2.1 and Equation (2.15), 1.0% shear wall ratio is sufficient for 5 story buildings to provide adequate seismic performance.

Table 2.1 Shear Wall Ratios based on Equation (2.15)

Number of Story (n)	Shear Wall Ratio (%)	Number of Story (n)	Shear Wall Ratio (%)
1	0.2	6	1.2
2	0.4	7	1.4
3	0.6	8	1.6
4	0.8	9	1.8
5	1	10	2

.

Second case investigates the equality of Equation (2.12) and following equation is derived:

$$\sum A_g / A_p = 0.0038n \qquad (2.16)$$

In this case, Equation (2.16) is utilized to obtain Table 2.2 and according to this table, a 3 story building with a shear wall ratio of %1.14 can resist earthquake loading, however the need for shear walls increases significantly with the increasing number of stories.

Table 2.2 Shear Wall Ratios based on Equation (2.16)

Number of Story (n)	Shear Wall Ratio (%)	Number of Story (n)	Shear Wall Ratio (%)
1	0.38	6	2.28
2	0.76	7	2.66
3	1.14	8	3.04
4	1.52	9	3.42
5	1.9	10	3.8

Following equation for the last case is determined, when the shear resistance is taken equal to the total base shear acting on the building ($V_r=V_t$):

$$\sum A_g / A_p = 0.0012n \tag{2.17}$$

Table 2.3 is formed using Equation (2.17) and this table indicates that 1.08% shear wall ratio is efficient to provide seismic resistance to a nine story building.

Table 2.3 Shear Wall Ratios based on Equation (2.17)

Number of Story (n)	Shear Wall Ratio (%)	Number of Story (n)	Shear Wall Ratio (%)
1	0.12	6	0.72
2	0.24	7	0.84
3	0.36	8	0.96
4	0.48	9	1.08
5	0.6	10	1.2

Based on the derived equations and obtained tables, Tekel (2006) stated that using different equations in the Turkish Earthquake Code (2007) may lead to different shear wall ratios for reinforced concrete buildings. Therefore, the most critical case should be selected.

2.4 Correlation between Shear Wall Ratio and Drift

Deformation capacity and seismic performance of reinforced concrete structures are two important parameters that provide seismic resistance and prevent excessive structural damage. Reinforced concrete structural walls have a significant role in providing high stiffness and deformation capacity to the buildings under earthquake loading. Moreover, information on the shear wall ratio of a building is required to determine the level of expected drifts, such as roof and interstory drifts, and these lateral drift values, can be used in the seismic evaluation to estimate the level of damage in a structural system. However, there is only a limited number of research studies on the correlation between shear wall ratios and drifts.

Gulkan et al. (2003) considered the elastic displacement response spectrum in the Turkish Earthquake Code (2007) to investigate the relationship between shear wall ratio and roof drift for reinforced buildings with shear walls of varying aspect ratios. It was observed that, there is a parabolic relationship between roof drift and shear wall ratio, for specified aspect ratios, h_w/l_w or H/D, where h_w is the height of the shear wall and l_w is the length of the shear wall (Figure 2.14).

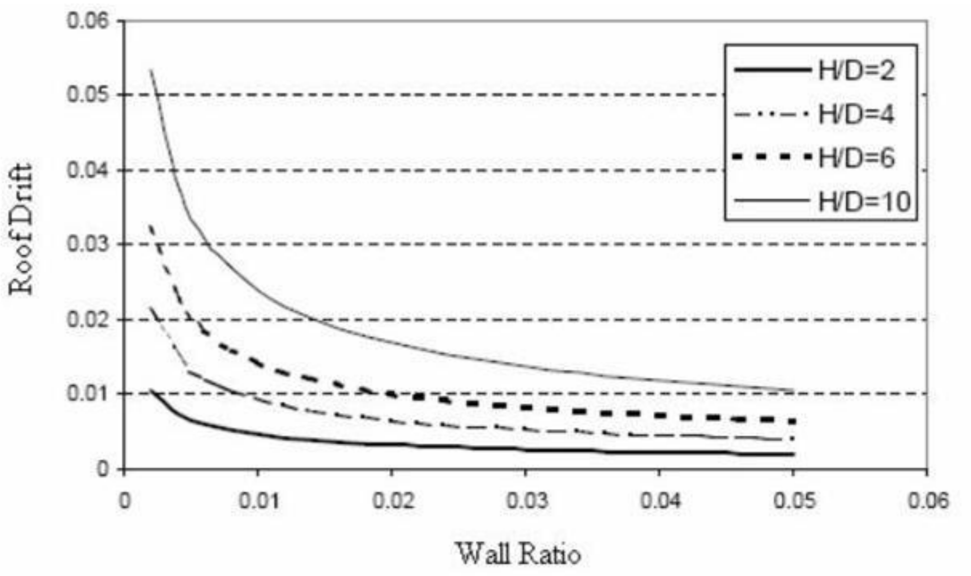


Figure 2.14 Roof Drifts vs. Wall Ratio Relationship by Gulkan et al. (2003)

Moreover, Gulkan et al. (2003) investigated the relationship between maximum compressive concrete strain observed in shear walls and shear wall ratios (Figure 2.15). From this figure, it can be observed that, higher shear wall ratios are required to satisfy the strain criterion for large aspect ratios and for high axial load ratios (N^*), when the maximum allowable strain level is 0.003. Around 1.50% shear wall ratio is required to have sufficient strain capacity for the most unfavorable conditions.

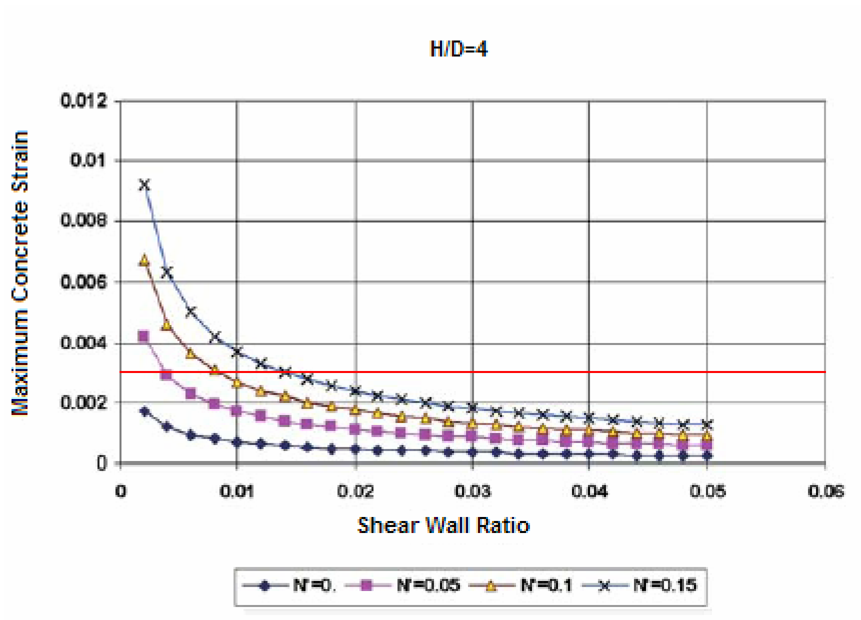


Figure 2.15 Maximum Concrete Strain vs. Shear Wall Ratio Relationship by Gulkan et al. (2003)

Wallace (1994) observed that the shear wall ratio, the aspect ratio, the configuration, the reinforcement ratio and the axial load on the shear walls influenced the wall-strain distribution. An approximate analytical procedure was developed to obtain the relationship between shear wall ratio and roof drift. First, elastic acceleration response spectrum is transformed to elastic displacement response spectrum. Then, the fundamental period of the structure was computed using cracked section stiffness of the members and elastic displacement response spectrum was used to obtain the elastic displacement of the structure at the fundamental period of the building. Finally, elastic displacement of the building was multiplied by a factor, which represented the difference between the displacements of a single degree of freedom system and the considered structural system, which was taken as 1.5, to obtain the roof drift. Figure 2.16 shows the outcome of this procedure for shear walls with different aspect ratios.

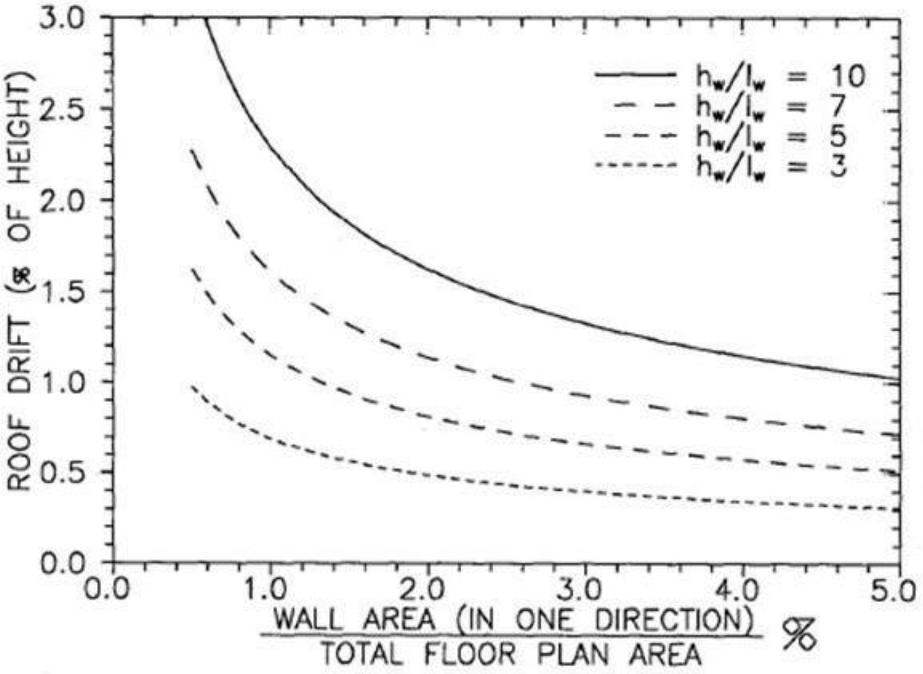


Figure 2.16 Roof Drift versus Wall Ratio by Wallace (1994)

More recent studies on the relationship between shear wall ratios and drifts were performed by Yakut and Soydas (2010), Canbolat et al. (2009) and Burak and Comlekoglu (2012). These analytical studies examined the influence of shear walls on the seismic performance of reinforced concrete buildings. Burak and Comlekoglu (2012) modeled 5 and 8 story reinforced concrete buildings with shear wall ratios ranging from 0.51% to 2.17% in both directions to investigate the effect of varying shear walls ratios on the seismic behavior of buildings. In this study, nonlinear time history analysis was used and the analytical results of building models compared with the experimental results of the full-scale seven-story reinforced concrete shear wall building that was tested in the U.S.-Japan Cooperative Research Program. It was concluded that the reinforced concrete buildings should have at least 1.00% shear wall ratio to control drifts and increasing the shear wall ratios beyond 1.50% did not improve the seismic performance of reinforced concrete buildings significantly.

Yakut and Soydas (2010) modeled low to mid-rise reinforced concrete buildings with 2, 5 and 8 stories that have 0.53% to 3.60% shear wall ratios, which are designed following the requirements of the Turkish Earthquake Code (2007) to determine shear wall ratio limits that can be used in the preliminary assessment and design of reinforced concrete buildings. Furthermore, the variation of roof and interstory drifts with increasing shear wall ratios was examined by linear elastic and nonlinear static pushover analyses of SAP2000 (2006) and the obtained results are compared with the results of the approximated procedures in the literature. The performances of these buildings are evaluated by linear elastic methods defined in the Turkish Earthquake Code (2007) by utilizing Probina Orion (2007). It was concluded that Turkish Earthquake Code (2007) is not adequate for the evaluation of the seismic performance and the interstory drift limitations of low to mid-rise reinforced concrete buildings. As a continuation of this study, Canbolat et al. (2009) showed that when the shear wall ratio is less than 1.5%, most of the vertical members underwent excessive deformations under high intensity earthquake loading and recommended shear wall ratio limits based on the number of stories of the buildings.

CHAPTER 3

ANALYTICAL MODELING OF SCHOOL BUILDINGS

3.1 Introduction

The aim of this analytical study is to investigate the influence of varying shear wall ratios on the mid-rise existing buildings that may or may not have torsional irregularities. Five different existing school buildings in Istanbul, which is one of the most earthquake prone cities in the world (TMMOB Chamber of Civil Engineers Report (2010)), are selected for this study. In this chapter, description of the existing school buildings and explanation of the analytical modeling of these structures are introduced. Material properties, beam, column and shear wall sectional properties, applied loads, dimensions and basic properties of these existing school buildings are given in "Description of the Existing School Buildings". Selected earthquake records, modeling procedure and methodology are given in "Explanation of the Analytical Modeling of the Buildings". Some of the selected school buildings were damaged after earthquakes and some of them were strengthened by adding shear walls or increasing the cross-sectional dimensions of the existing columns and shear walls. In this study, the strengthened school buildings are also modeled to investigate the improvement provided by the selected retrofit method. Moreover, new buildings, which have shear wall ratios in between that of before and after retrofit cases of the existing school buildings, are designed and analyzed to examine the influence of varying shear wall ratios on the seismic behavior of the overall structure and also the individual members. The information on the new buildings is given in "Description of Designed Buildings". Totally, twenty four school buildings are modeled by using the software program, SAP2000 v14.2.0 (2009).

3.2 Description of Existing School Buildings

Five different existing school buildings are analyzed in this study. In this chapter; material properties, beam, column and shear wall sectional properties, applied loads, dimensions and basic properties of these existing school buildings are presented. The selected school buildings are Güngören Haznedar Abdi İpekçi Primary School Block B, G.O.P. Ülkü Primary School Block B, Sarıyer MEV Dumlupınar Primary School, Fatih Gazi Primary School, Eminönü Çemberlitaş Anatolian High School Block A and two modified versions of Sarıyer MEV Dumlupınar Primary School designed following the requirements of the Turkish Earthquake Code (2007), Turkish Standards 498 (1987) and Turkish Standards 500 (2000).

3.2.1 Description of Güngören Haznedar Abdi İpekçi Primary School Block B

Güngören Haznedar Abdi İpekçi Primary School Block B is a four-story reinforced concrete frame-shear wall building. Measured concrete strength of the building is 7.2 MPa and reinforcement strength is 220 MPa according to the Assessment and Preliminary Report of this school building (2007). The building is located on a soil site of C-Z2 in Turkish Earthquake Code (2007), which corresponds to Class B in ASCE SEI 41-06 (Seismic Rehabilitation of Existing Buildings) (2006), and in the second seismic zone (Turkish Earthquake Code (2007)). The story height is 3.10 m and the floor area is 322 m^2. The plan view is given in Figure 3.1.

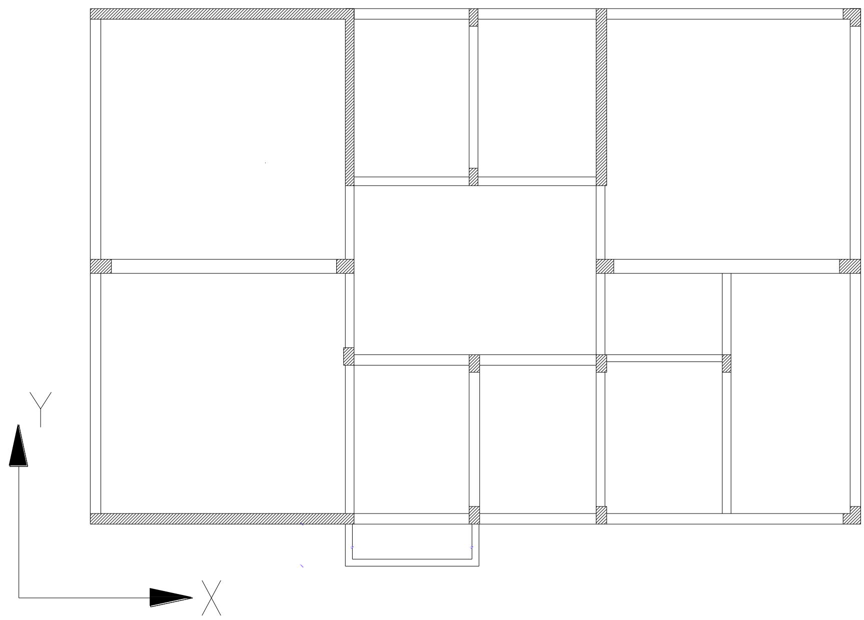


Figure 3.1 Plan View of Güngören Haznedar Abdi İpekçi Primary School Block B

There are four shear walls with three different cross sections in this school building with a shear wall ratio of 1.40% in the longitudinal direction and 0.86% in the transverse direction. In addition to these, there are six different column cross sections, but due to the variation of applied axial loads, totally, there are seventeen different types of columns. There are ten different beam cross sections, but due to the variation of beam length, distance between beams in the transverse direction and the thickness of the slabs, totally, there are twenty two different types of beams. The loads that are applied to the beams are taken as the dead load of the slab, floor cover and plaster, roof load and live loads of classes, senior common rooms, toilets, corridors, libraries and rooms of directors. After strengthening the building, Güngören Haznedar Abdi İpekçi Primary School Block B by adding reinforced concrete shear walls and increasing the sections of reinforced concrete columns, shear wall ratio in the longitudinal direction became 2.59% and in the transverse direction it became 2.07%. The plan view of the retrofitted building is given in Figure 3.2. There are three different cross sections for each of the added shear walls and the strengthened columns. Furthermore, this building is modeled to have shear wall ratios of 1.40% in the longitudinal direction and 1.48% in the transverse direction to examine the seismic behavior of the building with varying shear wall ratios. The columns of these generated buildings have the same cross sectional properties as the retrofitted building. The plan view of the generated building is given in Figure 3.3. The number of different types of reinforced concrete members and their cross sectional properties for the existing and retrofitted buildings are given in Table 3.1.

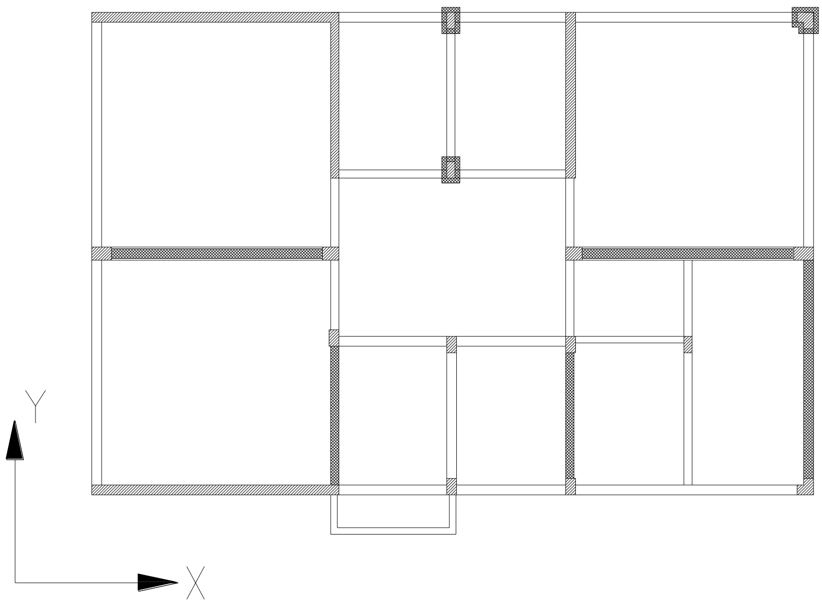


Figure 3.2 Plan View of the Retrofitted Güngören Haznedar Abdi İpekçi Primary School Block B

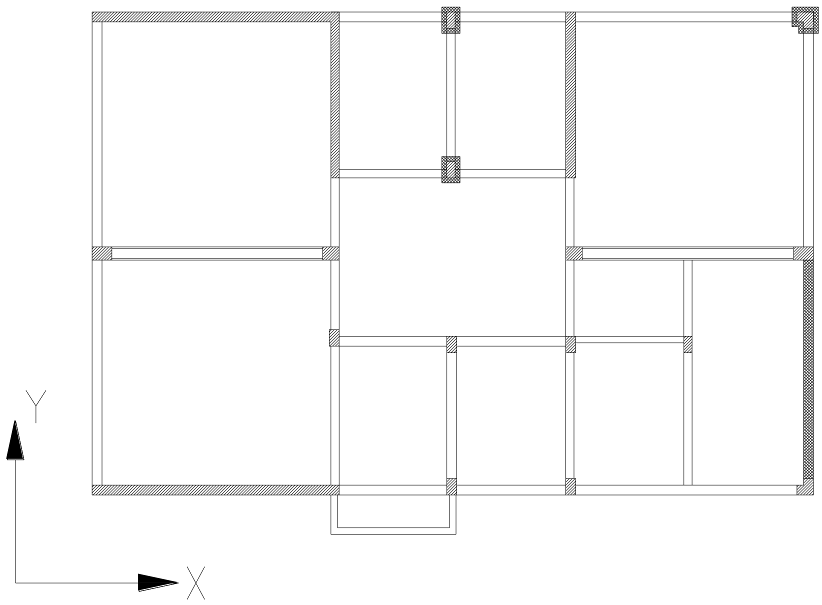


Figure 3.3 Plan View of the Generated Güngören Haznedar Abdi İpekçi Primary School Block B

Table 3.1 Properties of the Members in Existing and Retrofitted Case of Güngören Haznedar Abdi İpekçi Primary School Block B

Güngören Haznedar Abdi İpekçi Primary School Block B	Column		Beam		Shear Wall	
	Section(mm)	Reinforcement	Section(mm)	Reinforcement	Section(mm)	Long. Reinforcement
Existing Building	250x500	8ϕ14	200x400	2ϕ14 + 2ϕ14	250x5050	6ϕ14 + 52ϕ12 + 6ϕ14
	300x400	8ϕ14	200x500	2ϕ14 + 2ϕ14	300x5050	7ϕ14 + 52ϕ12 + 7ϕ14
	300x500	8ϕ14	200x700	2ϕ14 + 2ϕ14	300x7500	9ϕ14 + 76ϕ12 + 9ϕ14
	400x500	6ϕ16 + 2ϕ14	200x850	2ϕ14 + 2ϕ14	-	-
	400x600	6ϕ16 + 4ϕ14	250x400	3ϕ14 + 3ϕ14	-	-
	450x450	8ϕ14	250x500	3ϕ14 + 3ϕ14	-	-
	-	-	250x700	3ϕ14 + 3ϕ14	-	-
	-	-	300x500	3ϕ14 + 3ϕ14	-	-
	-	-	300x700	3ϕ14 + 3ϕ14	-	-
	-	-	400x700	3ϕ16 + 3ϕ16	-	-
Retrofitted Building (Additional Members)	550x800	20ϕ22	-	-	250x3830	15ϕ22 + 20ϕ12 + 15ϕ22
	600x700	18ϕ22	-	-	300x6400	21ϕ14 + 36ϕ12 + 21ϕ14
	800x800	24ϕ22	-	-	300x6650	21ϕ14 + 40ϕ12 + 21ϕ14

3.2.2 Description of G.O.P. Ülkü Primary School Block B

G.O.P. Ülkü Primary School Block B has similar properties with the building, Güngören Haznedar Abdi İpekçi Primary School Block B such as number of stories which is four, structural type of building which is reinforced concrete frame with shear walls, seismic zone which is second; but soil site is B-Z1 according to Assessment and Preliminary Report of the building (2007), which is defined as rock and firm soil in Turkish Earthquake Code (2007) and corresponds to Class B in ASCE SEI 41-06 (Seismic Rehabilitation of Existing Buildings) (2006). Measured concrete and reinforcement strengths of this building are 27.5 MPa and 220 MPa respectively. The story height for the two stories is 3.15 m for upper two stories is 3.10 m with a floor area of 320 m^2. Plan view of this building resembles the building, Güngören Haznedar Abdi İpekçi Primary School Block B (Figure 3.4).

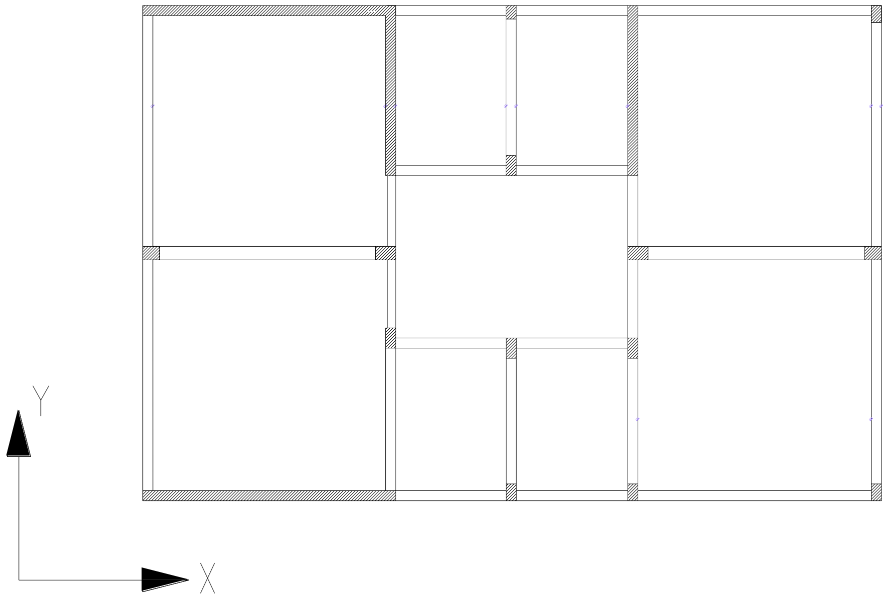


Figure 3.4 Plan View of G.O.P. Ülkü Primary School Block B

The shear wall ratios in the longitudinal and the transverse directions of this school building are 1.41% and 0.87%, respectively. This building has experienced only minor damage, therefore, it is not retrofitted. The properties of the members in the existing reinforced concrete building are shown in Table 3.2.

Table 3.2 Properties of the Members in G.O.P. Primary School Block B

G.O.P. Ülkü Primary School Block B	Column		Beam		Shear Wall	
	Section(mm)	Reinforcement	Section(mm)	Reinforcement	Section(mm)	Long. Reinforcement
Existing Building	300x400	8ϕ14	250x450	3ϕ14 + 3ϕ14	250x5050	6ϕ14 + 50ϕ12 + 6ϕ14
	300x500	8ϕ14	300x450	4ϕ14 + 4ϕ14	300x5050	7ϕ14 + 50ϕ12 + 7ϕ14
	300x600	10ϕ14	300x500	4ϕ14 + 4ϕ14	300x7500	13ϕ14 + 70ϕ12 + 13ϕ14
	400x500	8ϕ16	300x550	4ϕ14 + 4ϕ14	-	-
	400x600	8ϕ18	300x650	4ϕ14 + 4ϕ14	-	-
	-	-	400x650	4ϕ18 + 4ϕ18	-	-

3.2.3 Description of Sarıyer MEV Dumlupınar Primary School

Sarıyer MEV Dumlupınar Primary School is a four-story reinforced concrete frame-shear wall building in the third seismic zone in a C-Z2 soil site. The story height is 3.10 m, the area of first floor is 777 m^2 and the areas of other floors are 751 m^2. It can be seen from Figure 3.5 that this building has eleven bays in the longitudinal direction (X-direction) and three bays in the transverse direction (Y-direction). Measured concrete compressive strength of the building is 13.9 MPa and reinforcement strength is 220 MPa. Basic information of this building is obtained from Assessment and Preliminary Report of the building (2007).

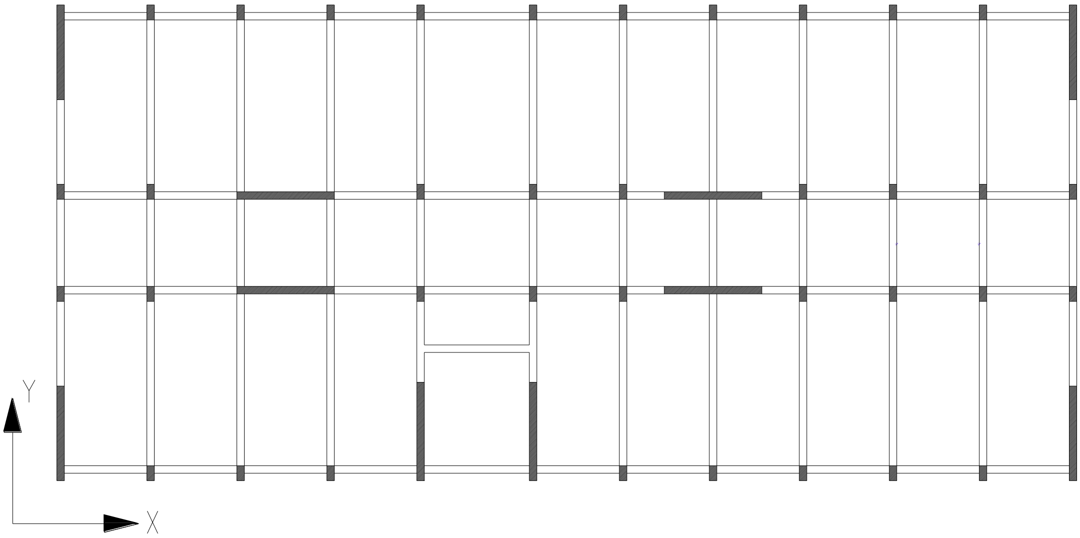


Figure 3.5 Plan View of Sarıyer MEV Dumlupınar Primary School

There are four shear walls in the longitudinal direction with a shear wall ratio of 0.62% and three different shear wall cross sections in the transverse direction corresponding to 0.93% shear wall ratio. There are two different beam sections and two different column sections. Sarıyer MEV Dumlupınar Primary School was also strengthened by increasing the dimensions of some columns and by adding shear walls. Four shear walls were added in the longitudinal direction and two shear walls were added in the transverse direction of this building. Therefore, shear wall ratio in the longitudinal direction and in the transverse direction becomes 1.15% and 1.46%, respectively for after retrofit case of this building. Plan view of the retrofitted building and member properties of the existing and retrofitted case of this building is shown in Figure 3.6 and in Table 3.3, respectively.

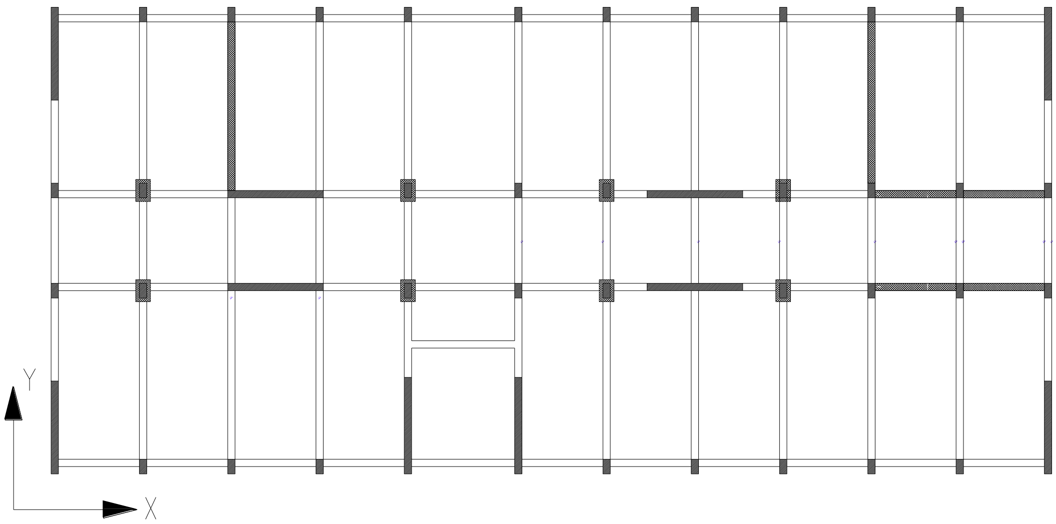


Figure 3.6 Plan View of the Retrofitted Sarıyer MEV Dumlupınar Primary School

Table 3.3 Properties of the Members in Existing and Retrofitted Case of Sarıyer MEV Dumlupınar Primary School

Sarıyer MEV Dumlupınar Primary School	Column		Beam		Shear Wall	
	Section(mm)	Reinforcement	Section(mm)	Reinforcement	Section(mm)	Long. Reinforcement
Existing Building	250x350	6ϕ14	250x400	2ϕ14 + 2ϕ14	300x3800	6ϕ14 + 34ϕ12 + 6ϕ14
	300x600	10ϕ14	300x600	3ϕ14 + 3ϕ14	300x3900	6ϕ14 + 34ϕ12 + 6ϕ14
	-	-	-	-	300x3950	6ϕ14 + 36ϕ12 + 6ϕ14
Retrofitted Building (Additional Members)	600x900	20ϕ22	-	-	300x3300	13ϕ22 + 20ϕ12 + 13ϕ22
	-	-	-	-	300x6600	21ϕ14 + 42ϕ12 + 21ϕ14

3.2.4 Description of Fatih Gazi Primary School

Fatih Gazi Primary School is the only three-story reinforced concrete building that has a frame-shear wall structural system in this study. According to Assessment and Preliminary Report of this building (2007), it is in C-Z2 soil site and in the second seismic zone. The floor area is 628 m^2, story height is 3.45 m and Fatih Gazi Primary School has fifteen bays in the longitudinal direction (X-direction) and three bays in the transverse direction (Y-direction) as shown in Figure 3.7. Similar to other school buildings used in this study, measured reinforcement strength of the building is 220 MPa, but, the concrete strength is 18.2 MPa.

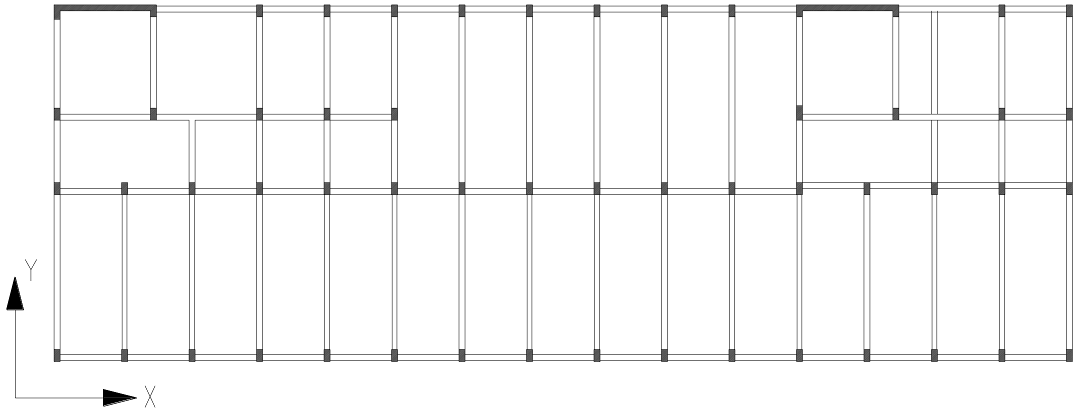


Figure 3.7 Plan View of Fatih Gazi Primary School

In this building, there is only one column cross section and only one shear wall section that is placed in the longitudinal direction (X-direction) with a shear wall ratio of 0.34% in X-direction. Moreover, there are three different beam sections. Three different sections of shear walls were used for strengthening the building and after retrofit, the shear wall ratio in the longitudinal direction and in the transverse direction (Y-direction) becomes 1.44% and 1.33%, respectively (Figure 3.8). Two other cases with different shear wall ratios were also modeled to study the influence of shear wall ratio on the seismic behavior of the structure. For the first case the shear wall ratios are 0.34% and 0.59% in X- and Y-directions, respectively (Figure 3.9) and for the second case, that are 0.95% and 1.07% in X- in Y-directions, respectively (Figure 3.10). Table 3.4 shows the properties of reinforced concrete members in Fatih Gazi Primary School.

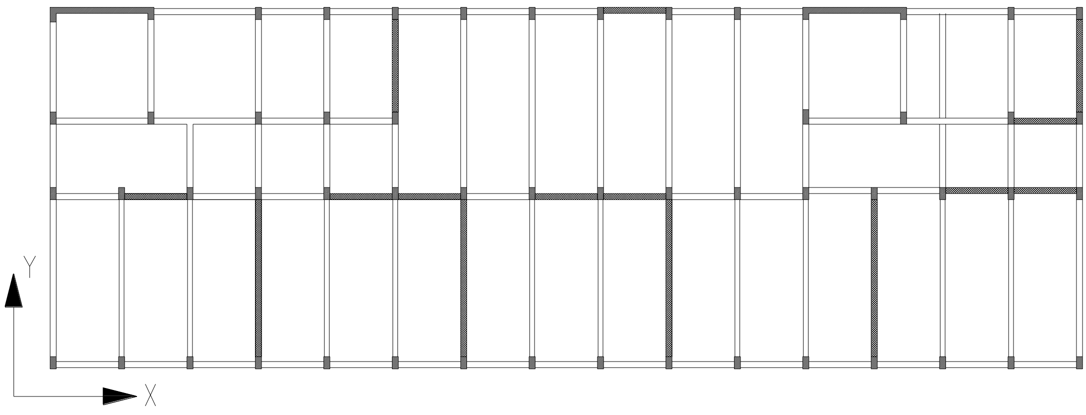


Figure 3.8 Plan View of the Retrofitted Case of Fatih Gazi Primary School

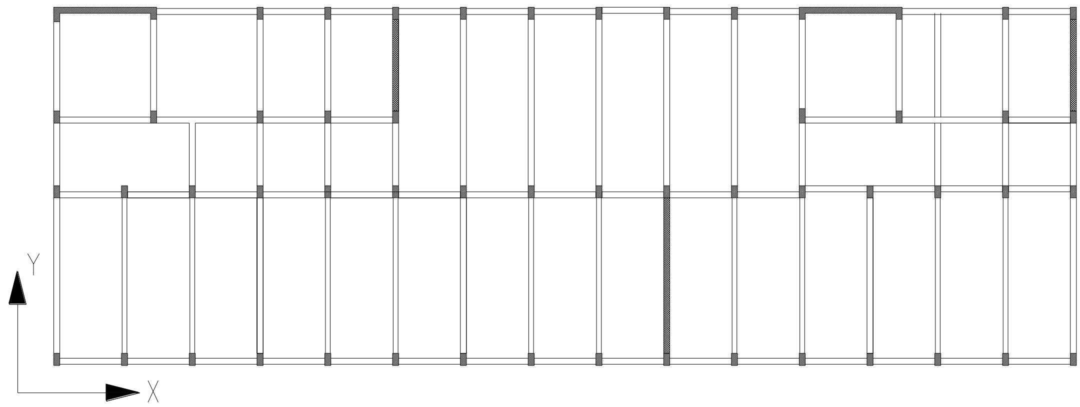


Figure 3.9 Plan View of the Generated Case of Fatih Gazi Primary School with 0.50%-0.50% Shear Wall Ratio

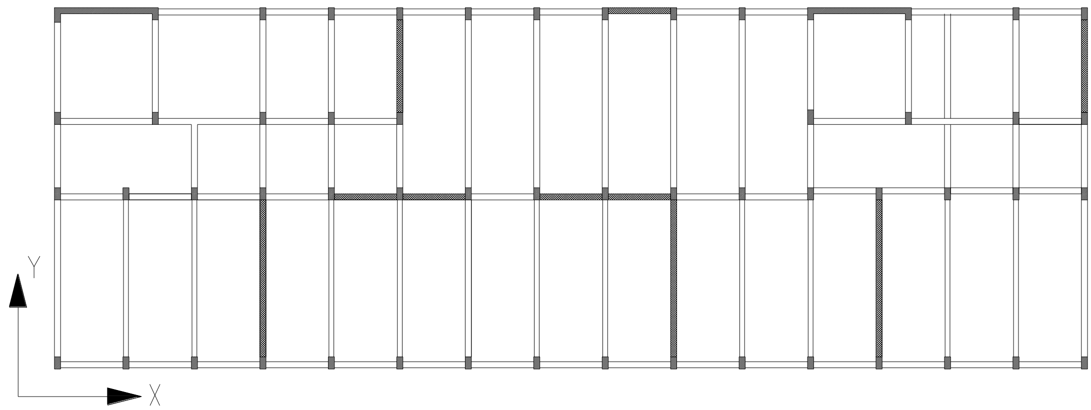


Figure 3.10 Plan View of the Generated Case of Fatih Gazi Primary School with 1.00%-1.00% Shear Wall Ratio

Table 3.4 Properties of the Members in Existing and Retrofitted Case of Fatih Gazi Primary School

Fatih Gazi Primary School	**Column**		**Beam**		**Shear Wall**	
	Section(mm)	**Reinforcement**	**Section(mm)**	**Reinforcement**	**Section(mm)**	**Long. Reinforcement**
Existing Building	250x500	4ϕ16 + 2ϕ14	250x400	2ϕ12 + 2ϕ12	250x4250	6ϕ14 + 42ϕ12 + 6ϕ14
	-	-	250x700	2ϕ14 + 2ϕ14	-	-
	-	-	250x900	2ϕ16 + 2ϕ16	-	-
Retrofitted Building (Additional Members)	-	-	-	-	250x3800	12ϕ22 + 20ϕ12 + 12ϕ22
	-	-	-	-	250x6450	18ϕ14 + 38ϕ12 + 18ϕ14
	-	-	-	-	300x2550	9ϕ14 + 12ϕ12 + 9ϕ14

3.2.5 Description of Eminönü Çemberlitaş Anatolian High School Block A

Eminönü Çemberlitaş Anatolian High School Block A is a reinforced concrete frame building which has five stories. Measured strength concrete and reinforcement is 12.9 MPa and 220 MPa respectively. The floor area is 346 m^2, the story height is 3.10 m and there are five bays in the longitudinal direction (X-direction) and three bays in the transverse direction (Y-direction) (Figure 3.11). According to Turkish Earthquake Code (2007) and Assessment and Preliminary Report (2007), this building is in C-Z2 soil site and the first seismic zone.

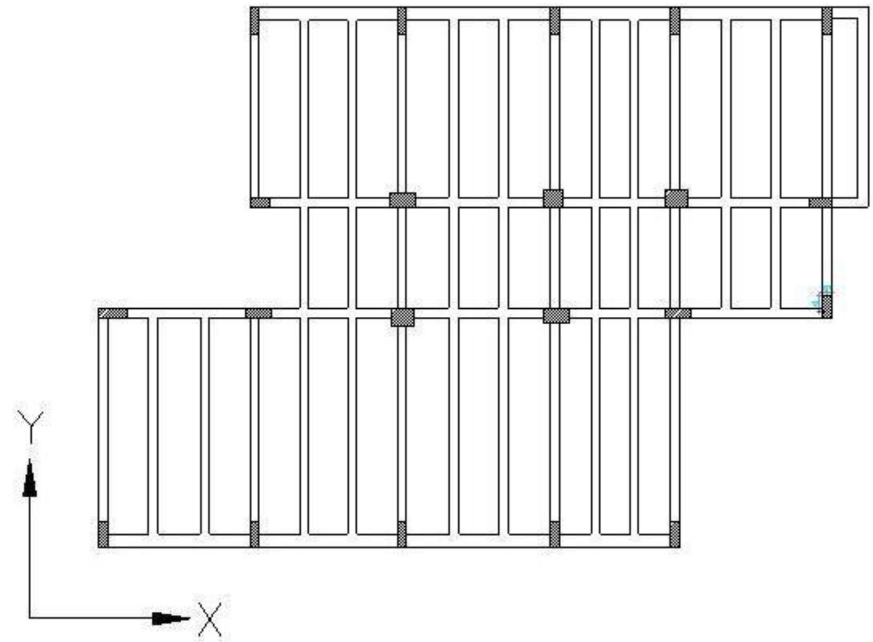


Figure 3.11 Plan View of Eminönü Çemberlitaş Anatolian High School Block A

This school building has four different sections of beams and nineteen different sections of columns without shear walls. This building was also strengthened by adding reinforced concrete shear walls and the shear wall ratio for the retrofitted case is 0.70% in the longitudinal direction and 1.37% in the transverse direction. Two more buildings with the same plan are designed and modeled. The first one has 0.47% shear wall ratio in the longitudinal direction and 0.38% in the transverse direction. The second building has 0.70% shear wall ratio in the longitudinal direction and 0.99% in the transverse direction. Plan view of the retrofitted case and generated cases are given in Figures 3.12, 3.13 and 3.14. The member properties of Eminönü Çemberlitaş Anatolian High School Block A are given in Table 3.5.

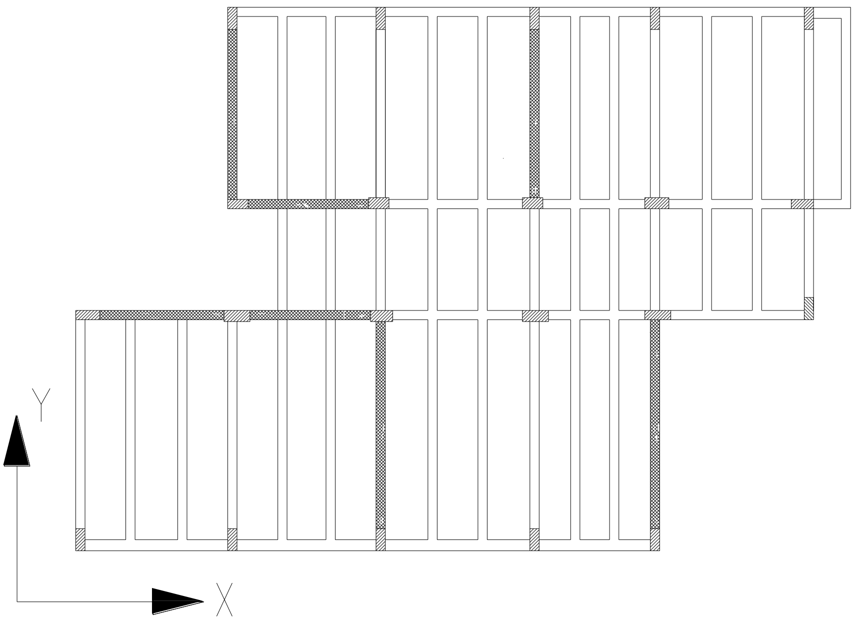


Figure 3.12 Plan View of the Retrofitted Case of Eminönü Çemberlitaş Anatolian High School Block A

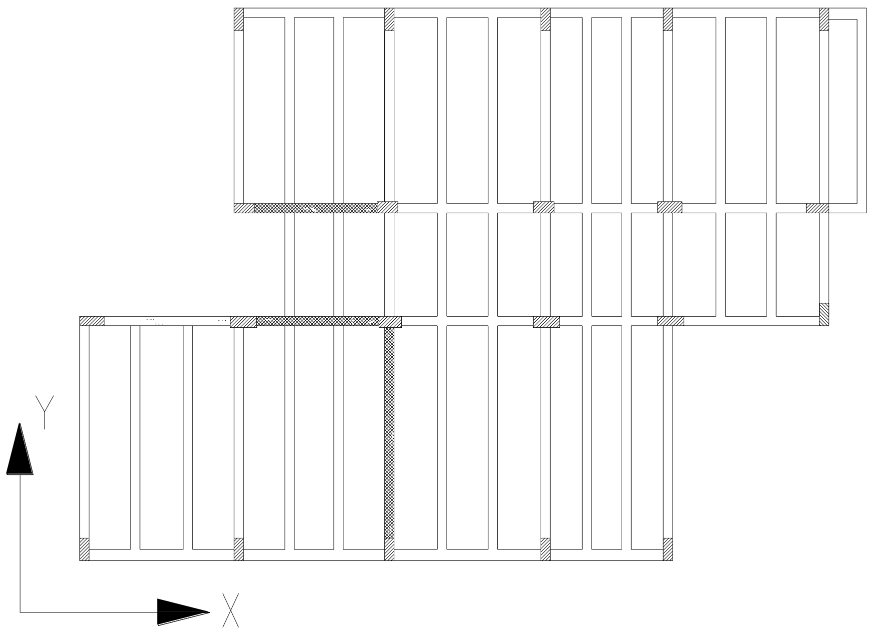


Figure 3.13 Plan View of the Generated Case of Eminönü Çemberlitaş Anatolian High School Block A with 0.50%-0.50% Shear Wall Ratio

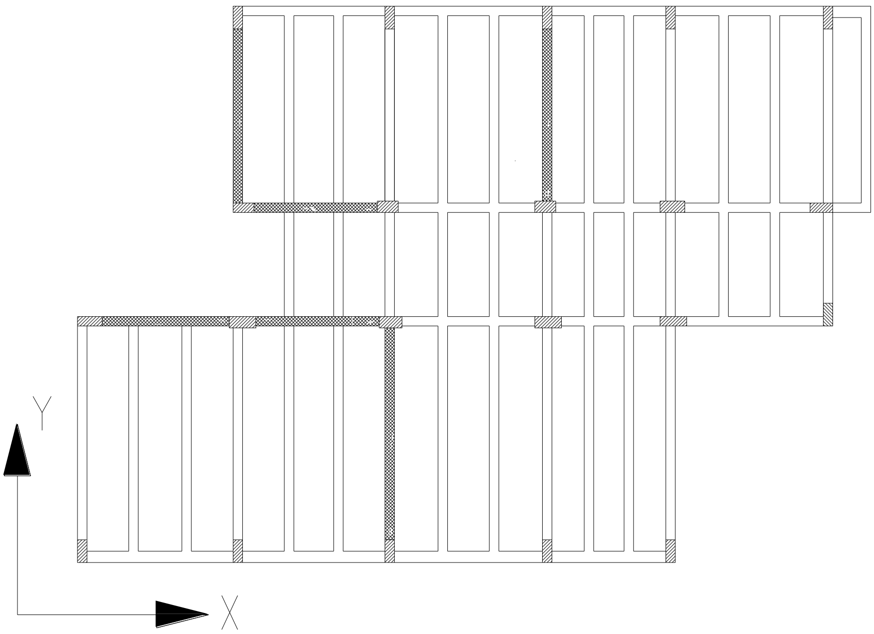


Figure 3.14 Plan View of the Generated Case of Eminönü Çemberlitaş Anatolian High School Block A with 1.00%-1.00% Shear Wall Ratio

Table 3.5 Properties of the Members in Existing and Retrofitted Case of Eminönü Çemberlitaş Anatolian High School Block A

Eminönü Çemberlitaş Anatolian High School Block A	Column		Beam		Shear Wall	
	Section(mm)	Reinforcement	Section(mm)	Reinforcement	Section(mm)	Long. Reinforcement
Existing Building	250x400	6ϕ14	250x600	2ϕ16 + 2ϕ16	-	-
	250x450	6ϕ14	250x750	3ϕ16 + 3ϕ16	-	-
	250x500	8ϕ14	300x600	3ϕ16 + 3ϕ16	-	-
	250x550	12ϕ12	350x600	3ϕ16 + 3ϕ16	-	-
	250x600	8ϕ14	-	-	-	-
	250x650	12ϕ14	-	-	-	-
	250x700	10ϕ14	-	-	-	-
	250x750	10ϕ14	-	-	-	-
	300x550	12ϕ12	-	-	-	-
	300x600	12ϕ14	-	-	-	-
	300x650	12ϕ14	-	-	-	-
	300x700	12ϕ14	-	-	-	-
	400x700	10ϕ16 + 2ϕ14	-	-	-	-
	500x550	12ϕ16	-	-	-	-
	500x600	12ϕ16	-	-	-	-
	500x650	14ϕ16	-	-	-	-
Retrofitted Building (Additional Members)	-	-	-	-	250x3250	12ϕ22 + 16ϕ12 + 12ϕ22
	-	-	-	-	250x4200	14ϕ14 + 24ϕ12 + 14ϕ14
	-	-	-	-	250x5300	16ϕ14 + 32ϕ12 + 16ϕ14

3.2.6 Description of Designed Buildings

Additional analyses are performed on buildings that are designed based on the structural properties of Sarıyer MEV Dumlupınar Primary School to investigate the influence of varying shear wall ratios on the seismic performance of the overall structure and individual members. As mentioned earlier, Sarıyer MEV Dumlupınar Primary School is a four-story reinforced concrete frame-shear wall building. The soil class, seismic zone, story height, number of stories, column and beam cross sectional dimensions and configurations, applied loads and plan of the building are kept the same in the designed buildings, but material properties are modified to be representative for the existing stock of buildings designed following the requirements of the Turkish Earthquake Code (2007). Shear wall ratio is increased step by step to observe the improvement in the behavior at each level of increase. Buildings are designed following the requirements of Turkish Earthquake Code (2007), Turkish Standards 498 (1987) and Turkish Standards 500 (2000).

As a result, five reinforced concrete buildings with increasing shear wall ratios of 0.00%, 0.50%, 1.00%, 1.50% and 2.00% in both longitudinal and transverse direction, are newly designed with concrete and steel grades of C20 and S420.

Equivalent static load method is used to design each of these buildings and therefore, importance factor (I), effective ground acceleration coefficient (A_0), earthquake load reduction factor (R) and spectrum coefficient (S(T)) are found separately for each building according to Turkish Earthquake Code (2007) to determine the equivalent static load coefficient. Then, with the help of SAP2000 v14.2.0 (2009), axial and shear forces and bending moments of each member are computed. Plan views of these five buildings can be seen in Figures 3.15 through 3.19.

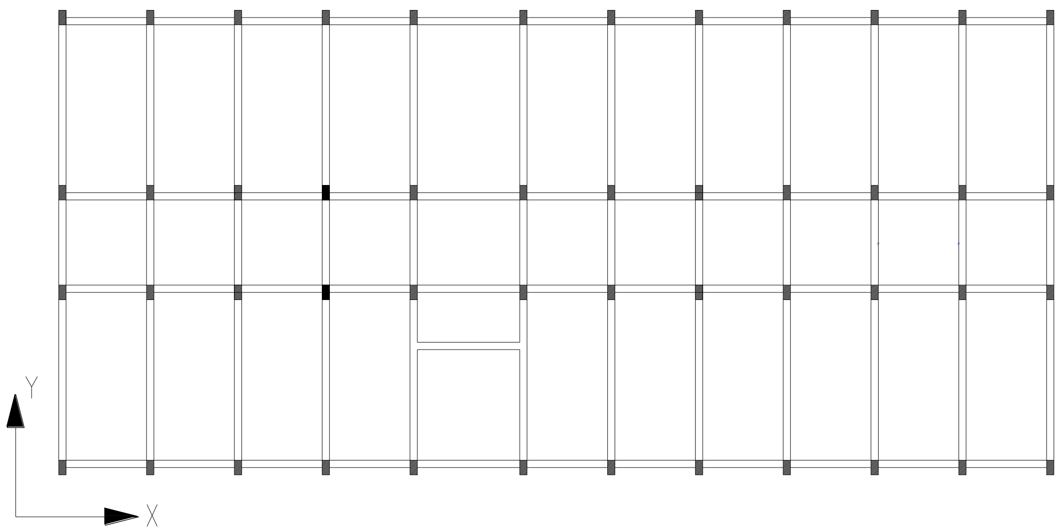


Figure 3.15 Plan View of the Designed Building with 0.0%-0.0% Shear Wall Ratio

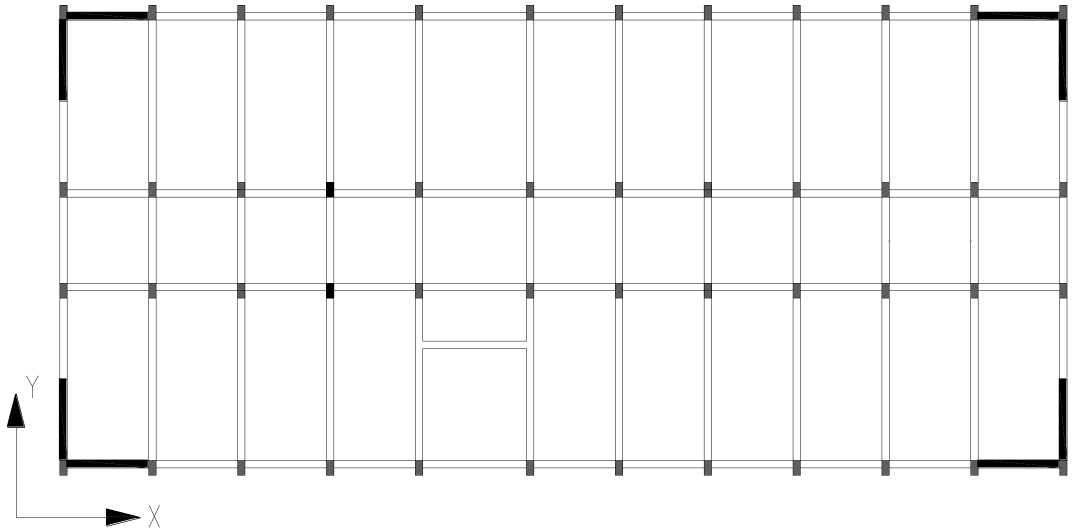


Figure 3.16 Plan View of the Designed Building with 0.5%-0.5% Shear Wall Ratio

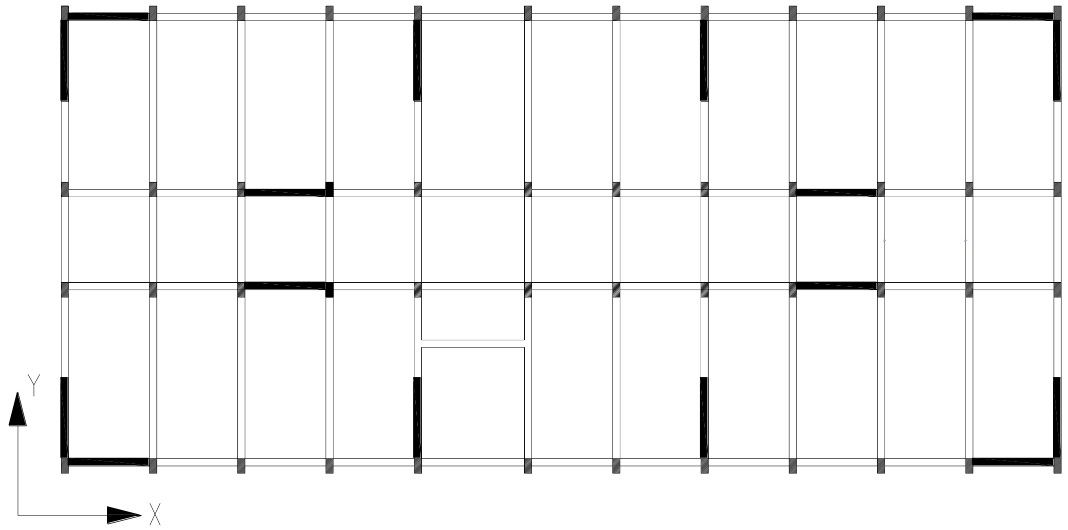


Figure 3.17 Plan View of the Designed Building with 1.0%-1.0% Shear Wall Ratio

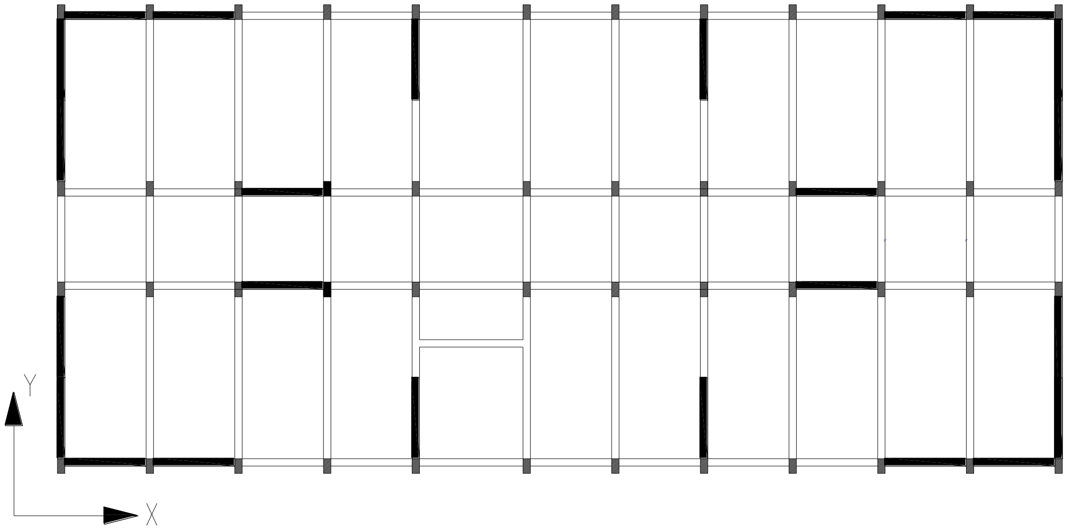


Figure 3.18 Plan View of the Designed Building with 1.5%-1.5% Shear Wall Ratio

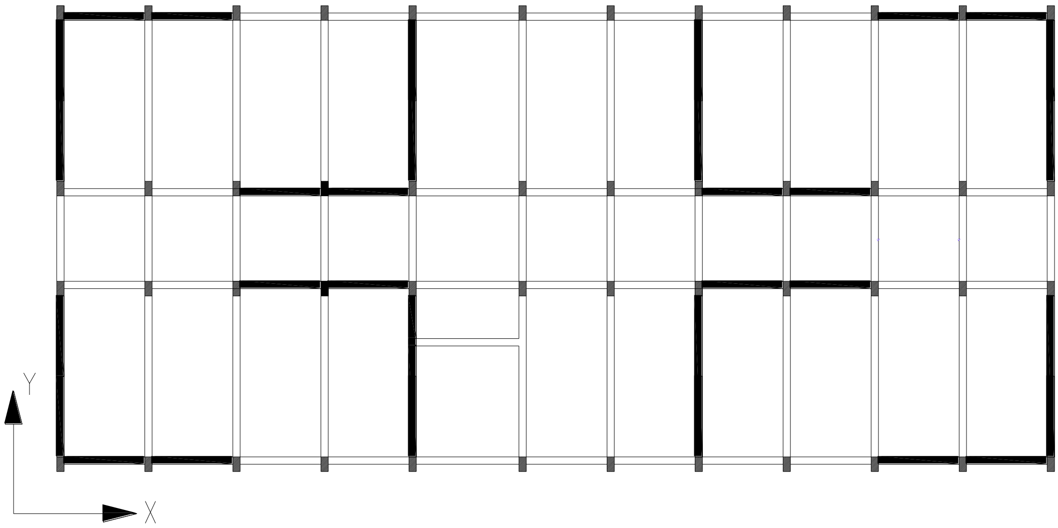


Figure 3.19 Plan View of the Designed Building with 2.0%-2.0% Shear Wall Ratio

Another building is designed and analyzed based on the structural properties of Sarıyer MEV Dumlupınar Primary School, where the rectangular columns of the first set of designed buildings are rotated 90 degrees to reduce the stiffness in the transverse direction where the larger component of the earthquake record is applied and create a weaker building. For the first set of designed buildings (first case), larger cross sectional dimension of the column, 300x600 mm is in the transverse direction (y-direction) of the building similar to the existing building. On the other hand, for the second set of designed buildings (second case), all the columns are rotated 90 degrees and therefore, larger dimension of the column sections is in the longitudinal direction (X-direction) of the building. Like the first set of designed buildings, five reinforced concrete buildings with increasing shear wall ratios are modeled with the use of SAP2000 v14.2.0 (2009).

3.2.7 Summary of Structural Characteristics of the Buildings used in the Analytical Study

As mentioned earlier, in this study five existing school buildings, which have different structural properties such as story heights, floor areas, number of stories, layouts, etc., are selected. General information about these existing school buildings is given in Table 3.6 below. Totally twenty-four school buildings that include the before and after retrofit cases of the selected school buildings and some generated buildings are analyzed to investigate the effect of varying shear wall ratios on the seismic performance of the structures by nonlinear direct integration time history analysis. The shear wall ratios of the existing school buildings vary between 0.0% and 2.5% in both directions. The component of the earthquake records with higher peak ground acceleration (PGA) values is applied to the weak direction of the buildings, except for second set of designed buildings, to examine the difference between the analytical results obtained from the models. Shear wall ratios of the existing buildings for before and after retrofit cases are shown in the Table 3.7. These ratios for the generated buildings that have in between shear wall ratios of before and after retrofit cases of the existing school buildings are given in Table 3.8. In Tables 3.7 and 3.8, the column "Class" is used for the classification of the group that the building is considered to be a part of while discussing the analytical results, i.e. 1.5-1.0 means a building having shear wall ratios of 1.5% and 1.0% in x and y directions, respectively. The shear wall ratios of the designed buildings that have C20 and S420 as the material grades are given in Table 3.9. In this table, "First Case" refers to the case where the column orientations are the same as the existing building and "Second Case" is for 90 degree rotated columns as explained in the section "Description of the Designed Buildings".

Table 3.6 General Properties of Existing School Buildings

Buildings	Number of Stories	Soil Class	Seismic Zone	Floor Area (m^2)	Floor Height (m)	Concrete Strength (MPa)	Yield Strength (MPa)
Güngören Haznedar Abdi İpekçi Primary School Block B	4	Z2	2	322 (1-2-3-4)	3.10 (1-2-3-4)	7.2	220
G.O.P. Ülkü Primary School Block B	4	Z1	2	320 (1-2-3-4)	3.15(1-2)-3.10(3-4)	27.5	220
Sarıyer MEV Dumlupınar Primary School	4	Z2	3	777(1)-751(2-3-4)	3.10 (1-2-3-4)	13.9	220
Fatih Gazi Primary School	3	Z2	2	628 (1-2-3)	3.45 (1-2-3)	18.2	220
Eminönü Çemberlitaş Anatolian High School Block A	5	Z2	1	346 (1-2-3-4-5)	3.10 (1-2-3-4-5)	12.9	220

Table 3.7 Shear Wall Ratios of Before and After Retrofit Cases of the Existing Buildings

Existing and Retrofitted Buildings	**Before Retrofit Case (Shear Wall Ratio(%)=SWR)**			**After Retrofit Case (Shear Wall Ratio(%)=SWR)**		
	SW_x	**SW_y**	**Class**	**SW_x**	**SW_y**	**Class**
Güngören Haznedar Abdi İpekçi Primary School Block B	1.40	0.86	1.5-1.0	2.59	2.07	2.5-2.0
G.O.P. Ülkü Primary School Block B	1.41	0.87	1.5-1.0	Not Retrofitted		
Sarıyer MEV Dumlupınar Primary School	0.62	0.93	0.5-1.0	1.15	1.46	1.0-1.5
Fatih Gazi Primary School	0.34	0.00	0.5-0.0	1.44	1.33	1.5-1.5
Eminönü Çemberlitaş Anatolian High School Block A	0.00	0.00	0.0-0.0	0.70	1.37	1.0-1.5

Table 3.8 Shear Wall Ratios in between Before and After Retrofit Cases of the Generated Buildings

Generated Buildings	**Case 1 (Shear Wall Ratio(%)=SWR)**			**Case 2 (Shear Wall Ratio(%)=SWR)**		
	SW_x	**SW_y**	**Class**	**SW_x**	**SW_y**	**Class**
Güngören Haznedar Abdi İpekçi Primary School Block B	1.40	1.48	1.5-1.5	None		
Fatih Gazi Primary School	0.34	0.51	0.5-0.5	0.95	1.03	1.0-1.0
Eminönü Çemberlitaş Anatolian High School Block A	0.47	0.38	0.5-0.5	0.70	0.99	1.0-1.0

Table 3.9 Shear Wall Ratios of the Designed Buildings

Designed Buildings	Shear Wall Ratio(%)=SWR				
	1	**2**	**3**	**4**	**5**
Sarıyer MEV Dumlupınar Primary School (First Case, Original Columns)	0.0-0.0	0.5-0.5	1.0-1.0	1.5-1.5	2.0-2.0
Sarıyer MEV Dumlupınar Primary School (Second Case, Rotated Columns)	0.0-0.0	0.5-0.5	1.0-1.0	1.5-1.5	2.0-2.0

3.3 Analytical Modeling

Twenty four school buildings are modeled by using SAP2000 v14.2.0 (2009) to investigate the improvement in the seismic performance when the shear wall ratio is increased. In this chapter, analytical modeling of the selected existing school buildings and the generated ones is explained in detail.

3.3.1 Selection of Ground Motion Records

Seven different earthquake records are selected from Pacific Earthquake Engineering Research Center (PEER) (2010) and Modern Geological Hazard Monitoring System (GoeNet) (2012) websites. The selected earthquake records have peak ground acceleration (PGA) values ranging from 0.152 g to 0.821 g with no impulse in the acceleration time history and velocity time history . Only, Kocaeli and Düzce records belong to near field earthquakes according to ATC 40 (1996). Closest distance to known seismic source for the other earthquake records is greater than 15 km and therefore, they belong to far field earthquakes. Basic information about these ground motion records is given in Table 3.10.

Table 3.10 Basic Information on Selected Earthquake Records

Earthquake	Year	Magnitude	Epicenter(km)	PGA_X(g)	PGA_Y(g)	PGV_X(cm/s)	PGV_Y(cm/s)
Kocaeli, Turkey (Izmit)	1999	7.8	5.31	0.152	0.220	22.6	29.8
Imperial Valley, USA (El Centro Array #6)	1979	6.5	27.47	0.410	0.439	64.9	109.8
Northridge, USA (Sun Valley-Roscoe Blvd)	1994	6.7	12.35	0.303	0.443	22.1	38.2
Christchurch, New Zealand (Lincoln Crop & Food Research)	2010	7.1	30.00	0.387	0.462	43.1	79.5
Düzce, Turkey (Düzce)	1999	7.1	1.61	0.348	0.535	60.0	83.5
Kobe, Japan (Takarazuka)	1995	6.9	38.60	0.693	0.694	68.3	85.3
Chichi, Nantou, Taiwan (CHY028)	1999	7.6	32.67	0.653	0.821	72.8	67.0

All of these ground motion records, which are acceleration time histories with different peak ground accelerations and characteristics are applied to all building models using SAP2000 v.14.2.0 (2009) considering 5% damping ratio based on ASCE 41 (2007) requirements. The acceleration time histories of all the selected ground motion records are provided in Figure 3.20. From this figure, it can be observed that the elastic response spectrum of Chi-Chi Earthquake is considerably high compared to the others. Chi-Chi Earthquake is selected to examine the seismic performance of the reinforced concrete buildings under severe earthquakes. In this figure, the elastic response spectrum generated for the building models according to Turkish Earthquake Code (2007) is also plotted with a bold line. Y-direction of the analyzed buildings is selected as the weak direction where there are lower number of load carrying frames or lower stiffness shear walls and columns compared to the transverse direction. Therefore, in this study, earthquake record components with higher peak ground acceleration values are applied to Y-directions of the school buildings and the orthogonal components of the records are applied to X-direction of the buildings, except for second case of the designed buildings.

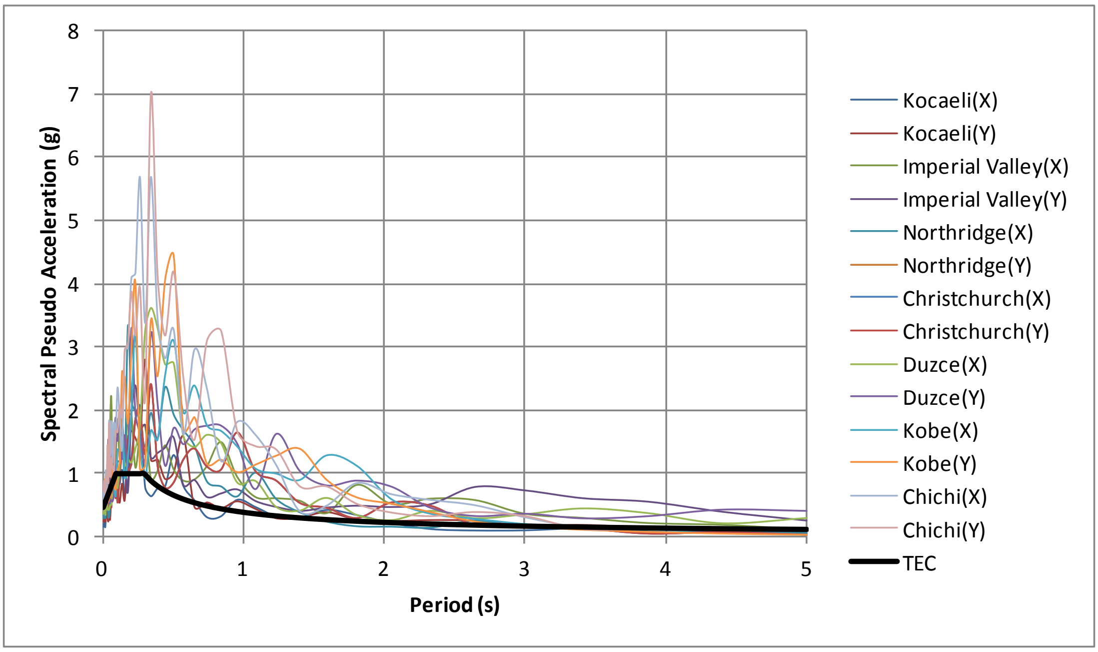


Figure 3.20 Response Spectra of the Selected Earthquake Records

Turkish Earthquake Code (2007) requires that if only three ground motion records are used in the analysis of the buildings, maximum analytical results should be considered, but if seven different earthquake records are utilized, the average analytical results can be considered in assessing the seismic behavior of the buildings. Furthermore, the total duration of selected ground motion records cannot be less than neither 15 seconds nor 5 times of the fundamental period of the building and the average of the spectral acceleration value of the earthquake records corresponding to zero period cannot be less than the multiplication of gravitational acceleration, g and effective acceleration coefficient, A_0. Moreover, 90% of the elastic spectral acceleration values, $S_{ae}(T)$ in between 0.2 and 2.0 times of the fundamental period of the building in the direction of the earthquake loading should be more than the average of spectral acceleration values of the earthquake records with 5% damping ratio according to Turkish Earthquake Code (2007). All the code requirements are satisfied in the selection of the seven ground motion records.

3.3.2 Applied Loads

The loads that are applied to the beams are taken as the dead load of the slab, floor cover and plaster, roof load and live loads of classes, senior common rooms, toilets, corridors, libraries and rooms of directors. Magnitudes of the floor cover and plaster load and the roof load are obtained from Assessment and Preliminary Reports of these existing school buildings as 2.0 kN/m^2 and 5.0 kN/m^2. The dead load of the slab varies with its thickness, therefore, for every building; dead load of the slab is calculated by multiplying the thickness of the slab with unit weight of the concrete which is 24 kN/m^3 (Assessment and Preliminary Report (2007)). Furthermore, live loads depend on the architectural drawings of these buildings (Assessment and Preliminary Report (2007)) and applied live load of class-senior common room-toilet, corridor-library and room of director is taken as 3.50 kN/m^2, 5.00 kN/m^2 and 2.00 kN/m^2 , respectively.

All the above mentioned loads are calculated for each slab at each floor and they are transferred as a distributed line loads to the adjoining beams. The load combinations from these loads are used as initial conditions for each earthquake loading. It should be mentioned that the influence of dead and live loads on the seismic response of the buildings cannot be neglected because of the varying capacities of the load carrying members based on the vertical loads acting on them during earthquakes.

3.3.3 Analytical Modeling of the School Buildings

Twenty-four buildings that consist of the before and after retrofit cases of the selected school buildings and some generated buildings as explained earlier are analyzed to investigate the effect of increasing shear wall ratios on the inelastic behavior of the structure under earthquake loading. The load carrying members of these buildings can be classified into shear walls, columns and beams. The moment versus rotation responses and axial load versus moment interaction diagrams of all members are obtained by using the software program, Response 2000 (2001). In the nonlinear analysis of buildings, beams and columns are modeled as line elements and shear walls are modeled as equivalent beam elements in SAP2000 v14.2.0 (2009). Detailed information on the modeling of the structure and the individual members is presented below.

In this research study, nonlinear direct integration time-history analysis is performed taking into account the damping of the structure to assess the seismic behavior. Chopra (2007) recommended two different approaches of classical damping, which are Rayleigh Damping and Caughey Damping, to model structural damping of multistory buildings. Rayleigh Damping has two different damping constants, which are mass-proportional damping constant and stiffness-proportional damping constant, to construct the damping matrix of the structure. The damping constants are calculated with the use of natural frequency, w and damping ratio, ζ and utilized in the nonlinear models. Newmark's Method is considered as the time-stepping method in the models of these school buildings. Newmark's equation with the parameters, gamma, γ and beta, β, taken as 0.5 and 0.25, respectively, corresponding to the assumption of constant average acceleration is utilized.

The applied dead and live loads are considered as lumped masses at each floor level. To obtain the mass of a floor, the dead loads are multiplied by 1.0 and the live loads are multiplied by 0.60 for school buildings according to the Table 2.7 in Turkish Earthquake Code (2007). Moreover, joint constraints are used at the slabs to construct rigid diaphragms at each floor level. Rigid diaphragms force the constrained joints of the floor level to move together as a planar diaphragm.

All existing school buildings are placed in first to third seismic zones and soil sites Z1 or Z2 according to Turkish Earthquake Code (2007). Soil-structure interaction is not taken into account and the ground floor columns and shear walls are modeled as having fixed supports as boundary conditions in SAP2000 v14.2.0 (2009).

3.3.3.1 Analytical Modeling of the Beams

Reinforced concrete beams are modeled as line elements in SAP2000 v14.2.0 (2009) in this study. Assessment and Preliminary Reports of the existing school buildings are used to obtain the cross sectional dimensions and the reinforcement detailing of the beams. Slab thicknesses, t_b, beam span lengths, l_b and transverse distances between beams, l_{tb} are also taken from these reports to compute the effective slab width of T-beams, which have slabs on both sides, and L-beams, which have slab on one side only, following the requirements of ACI 318 (2004).

The effective slab width of the T-beams is specified as the minimum of the following expressions:

1) Total flange width of the T-beam, which is also known as the effective slab width, b_e, cannot be greater than one-quarter of the beam span length, l_b,
2) The effective overhanging flange width on each side of the web, b_{ex}, cannot be greater than eight times of the slab thickness, t_b,
3) The effective overhanging flange width on each side of the web, b_{ex}, cannot be greater than one-half of the transverse distance between beam webs, l_{tb}.

Whereas, the overhanging flange width, b_{ex}, of the L-beams cannot be greater than the minimum of the following expressions:

1) one-twelfth of the beam span length, l_b,
2) six times the slab thickness, t_b,
3) one-half of the transverse distance between beam webs, l_{tb}.

The cross sectional properties of the beams such as cross-sectional area, A_g, moment of inertia, I, and shear area, A_s are determined. According to Section 11 of ACI 318 (2004), shear area of T-beams and L-beams are taken as five-sixth of the web area of the beam, A_{web}. Cracked behavior of the beam is considered during inelastic modeling; therefore, the moment of inertia of the web, I_{web} is input for both T-beams and L-beams based on ASCE 41 (2007) to SAP2000 v14.2.0 (2009).

The beam elements are modeled as cracked elastic line elements with zero length plastic hinges and rigid end zones at each end. The length of the rigid end zone is taken as half the column width in the direction of the beam. The plastic hinges of the beams are placed at each column face and modeled with Moment M_3 hinge in SAP2000 v14.2.0 (2009). Based on the requirements of ASCE 41 (2007), hinge length, l_p , is taken as half the effective flexural depth of the beam, d. The moment-curvature diagrams of each beam are obtained by using Response 2000 (2001) and these graphs are idealized by using the generalized force-deformation relationship for concrete elements or components given in ASCE 41 (2007) (Figure 3.21). Furthermore, a spreadsheet is prepared using Microsoft Office Excel 2007 to equate the areas added and taken off while obtaining the quadlinear generalized moment-curvature diagrams and an example for this spreadsheet is given in Appendix A.1. The parameters for the use of generalized force-deformation relationship for concrete elements or components are presented below as Table 3.11. Moreover, the acceptance criteria for the beams are acquired from the same table. An example of obtaining the generalized moment-curvature diagram for a Moment M_3 hinge in SAP2000 v14.2.0 (2009) is provided in Appendix A.2.

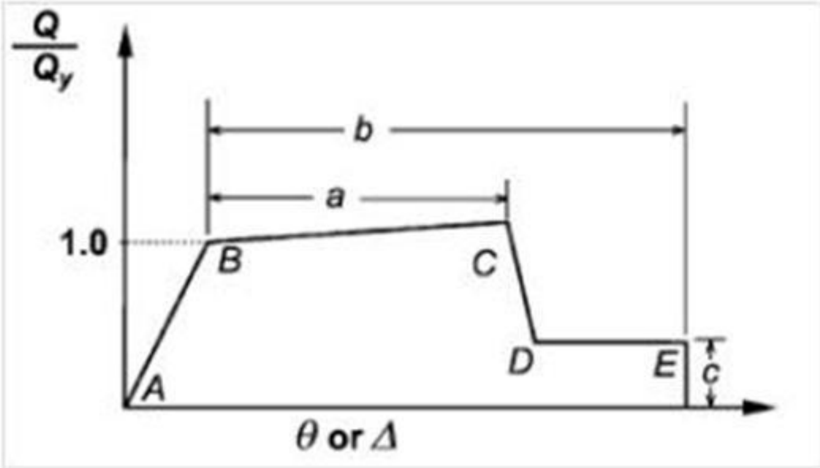


Figure 3.21 Generalized Force-Deformation Relationship of Concrete Elements or Components of ASCE 41 (2007)

Table 3.11 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures for Reinforced Concrete Beams from Table 6.7 of ASCE 41 (2007)

Conditions			Modeling Parameters[3]			Acceptance Criteria[3, 4]				
						Plastic Rotations Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotations Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01

3.3.3.2 Analytical Modeling of the Columns:

In reinforced concrete structures, one of the most important members of the structural system is columns since they carry the vertical loads and their failure lead to structural failure. In this study, columns are modeled as line elements with zero length plastic hinges and rigid end zones at each end, but at the ground floor level, rigid end zones are not placed at the fixed supports of the building models. Rigid end zone lengths are taken equal to half the beam height and the plastic hinge lengths are considered to be equal to half the effective depth of the column in the loading direction. The cross sectional dimensions of the columns and the longitudinal and transverse reinforcement ratios in the columns are obtained from the Assessment and Preliminary Reports of the existing school buildings. Response 2000 (2001) is used to find the moment-curvature and interaction diagrams of the columns. The generalized force-deformation relationship of elements or components (Figure 3.21) based on ASCE 41 (2007) requirements and a new spreadsheet produced by using Microsoft Office Excel 2007 is utilized to obtain the idealized moment-curvature diagram that is input to the SAP2000 v14.2.0 (2009) model. This spreadsheet is given in Appendix A.3 and an example of moment-curvature diagram of Interacting P-M_2-M_3 hinge in SAP2000 v14.2.0 (2009) is also provided in Appendix A.4. Another spreadsheet is prepared using Microsoft Office Excel 2007 to obtain the interaction diagrams of columns (Appendix A.5) and an example interaction diagram for an Interacting P-M_2-M_3 hinge in SAP2000 v14.2.0 (2009) is supplied in Appendix A.6. While constructing the interaction diagrams, the axial forces due to the combination of dead and live loads are considered, therefore, the same column has different interaction diagrams at different floor levels. The modeling parameters of generalized moment-curvature relationship and numerical acceptance criteria of columns that are considered and in this study are presented in Table 3.12 below.

Table 3.12 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures for Reinforced Concrete Columns from Table 6.8 of ASCE 41 (2007)

Conditions		Modeling Parameters[3]			Acceptance Criteria[3, 4]				
					Plastic Rotations Angle, radians				
					Performance Level				
		Plastic Rotations Angle, radians		Residual Strength Ratio		Component Type			
						Primary		Secondary	
		a	b	c	IO	LS	CP	LS	CP
Condition i. [1]									
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$								
≤ 0.1	≥ 0.006	0.035	0.060	0.2	0.005	0.026	0.035	0.045	0.060
≥ 0.6	≥ 0.006	0.010	0.010	0.0	0.003	0.008	0.009	0.009	0.010
≤ 0.1	= 0.002	0.027	0.034	0.2	0.005	0.020	0.027	0.027	0.034
≥ 0.6	= 0.002	0.005	0.005	0.0	0.002	0.003	0.004	0.004	0.005

Figure 3.22 gives the comparison of measured flexural rigidities from laboratory column tests with the flexural rigidity values proposed by ASCE 41 (2007) to find the effective moment of inertia, I_{eff}, of the column sections. In this figure, EI_{eff} is the effective flexural rigidity of the column, EI_g is the gross flexural rigidity of the column, P is the axial load applied to the column, A_g is the gross cross-

sectional area of the column and f_c' is the concrete compressive strength.

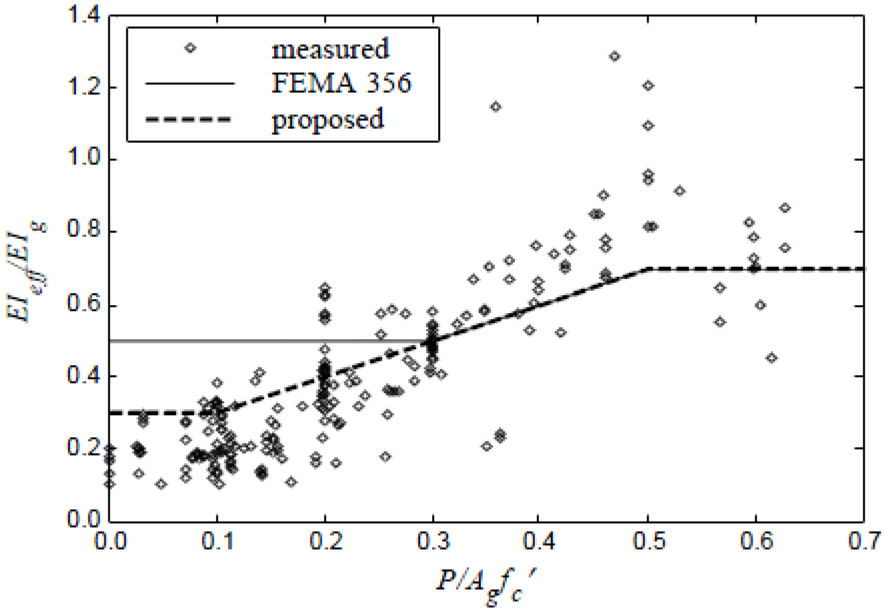


Figure 3.22 Comparison of Measured Flexural Rigidities with the Flexural Rigidity Values Proposed by Figure 1 of ASCE 41 (2007)

3.3.3.3 Analytical Modeling of the Shear Walls:

Reinforced concrete shear walls are both vertical and lateral load carrying members in reinforced concrete structures and as mentioned in the previous chapters, these members are very influential on the seismic performance of structures. In this study, structural walls are modeled by wide column analogy and therefore, structural walls of the models are formed of line elements. For each shear wall, there are rigid beams at each floor level and columns with the same cross-sectional properties as the shear walls at the centroidal axis of the shear wall. Detailed information on the wide column analogy is provided in Chapter 2. Like columns, rigid end zones are located at each end of the shear walls; but there are no rigid end zones at the ground floor level where the shear walls have fixed supports.

Assessment and Preliminary Reports of the existing school buildings are used to determine the cross sectional dimensions of the shear walls and the horizontal and vertical reinforcement ratios. Response 2000 (2001) is utilized to obtain the moment-curvature and interaction diagrams of the plastic hinges, which are placed at each end of the line element at the face of the rigid beams. Interacting P-M_2 or Interacting P-M_3 are selected as the plastic hinges for the shear walls in SAP2000 v14.2.0 (2009). Two new spreadsheets using Microsoft Office Excel 2007 are developed to find the generated moment-curvature relationship of shear walls (Figure 3.21) and the interaction diagrams. Examples of these diagrams obtained from Response 2000 (2001) are provided in Appendix A.7 and Appendix A.8 and examples of the spreadsheets of are given in Appendix A.9 and Appendix A.10. Table 3.13 based on ASCE 41 requirements is considered to obtain modeling parameters of the generated moment-curvature diagrams and the acceptance criteria of shear walls. As for the case of columns, while determining the moment-curvature diagrams of shear walls in Response 2000 (2001), axial forces acting on the shear walls due to the combination of dead and live loads are considered.

Table 3.13 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures of Reinforced Concrete Shear Walls from Table 6.18 of ASCE 41 (2007)

Conditions			Plastic Hinge Rotation (radians)		Residual Strength Ratio	Acceptable Plastic Hinge Rotation[4,5,6] (radians)				
						Performance Level				
							Component Type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Shear walls and wall segments										
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{V}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]								
≤ 0.1	≤ 34	Yes	0.015	0.020	0.75	0.005	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40	0.004	0.008	0.010	0.010	0.015
≥ 0.25	≤ 34	Yes	0.009	0.012	0.60	0.003	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30	0.0015	0.003	0.005	0.005	0.010
≤ 0.1	≤ 34	No	0.008	0.015	0.60	0.002	0.004	0.008	0.008	0.015
≤ 0.1	≥ 6	No	0.006	0.010	0.30	0.002	0.004	0.006	0.006	0.010
≥ 0.25	≤ 34	No	0.003	0.005	0.25	0.001	0.002	0.003	0.003	0.005
≥ 0.25	≥ 6	No	0.002	0.004	0.20	0.001	0.001	0.002	0.002	0.004

For the cracked behavior of shear walls, the flexural rigidity value proposed in Table 6.5 of ASCE 41 (2007) as $0.5E_cI_{gw}$ is considered, where E_c is the modulus of elasticity of concrete and I_{gw} is the moment of inertia of the gross shear wall section.

CHAPTER 4

ANALYTICAL RESULTS OF THE BUILDING MODELS

4.1 Introduction

The software program, SAP2000 v14.2.0 (2009) is used to analyze all the existing and designed reinforced concrete buildings by using nonlinear time-history analysis. Seven different earthquake records are utilized in all building models to evaluate the seismic behavior of these structures following the Turkish Earthquake Code (2007) requirements. The analytical results are provided first for the designed buildings, the floor plans of which are based on the existing school building, Sarıyer MEV Dumlupınar Primary School. Since the shear wall ratios of these buildings vary between 0.00% and 2.00% in both longitudinal and transverse directions, it is easier to identify the effect of this ratio on the seismic performance. As mentioned in Chapter 3, for the first set of designed buildings (first case), larger cross sectional dimension of the column is in the transverse direction (Y-direction) of the building similar to the existing building. For the second set of designed buildings (second case), all the columns are rotated 90 degrees and therefore, larger dimension of the column sections is in the longitudinal direction (X-direction) of the building. Then, the analytical results for the existing school buildings, the shear wall ratios of which vary between 0.0% and 2.5% in both directions, are provided.

4.2 Analytical Results of the Designed Buildings

4.2.1 Modal Periods of the Designed Buildings

Shear wall area to floor area ratio is a factor that influences the overall stiffness of a reinforced concrete building. Therefore, modal periods of the buildings change with varying shear wall ratios, as expected. Figure 4.1 shows the variation of modal periods with respect to increasing shear wall ratios for the designed buildings in both x and y loading directions. From this figure, it can be observed that, modal periods of the reinforced concrete buildings decrease with increasing shear wall ratios. This decrease is quite significant for buildings with lower shear wall ratios when compared to the ones with higher shear wall ratios.

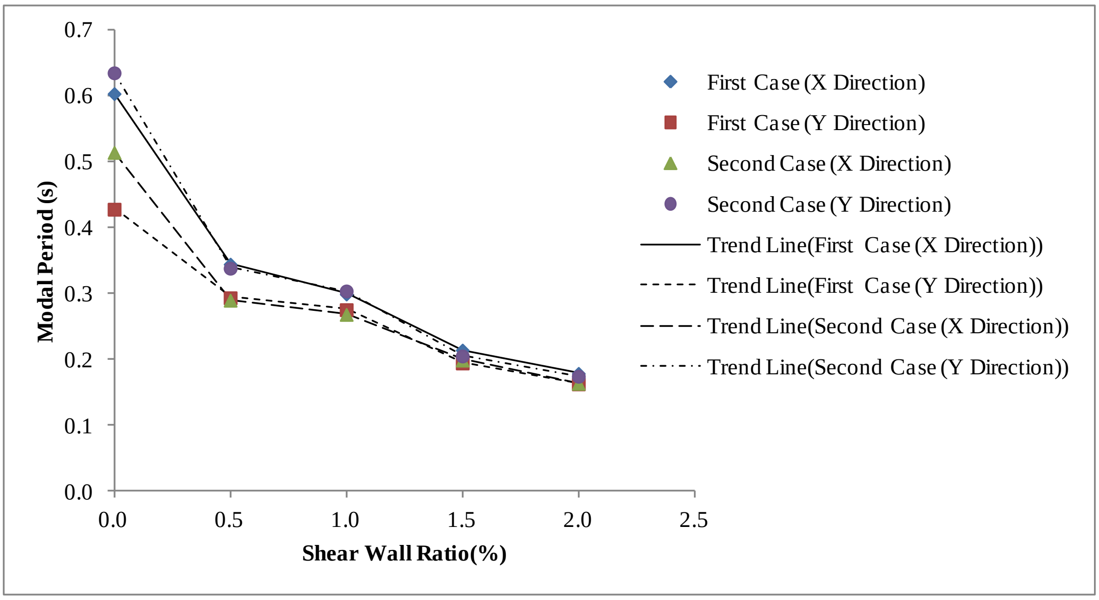


Figure 4.1 Modal Period vs. Shear Wall Ratio of the Designed Buildings

4.2.2 Analytical Results of the Designed Building-First Case

The existing school building, Sariyer MEV Dumlupinar Primary School is designed to have C20 and S420 as concrete and reinforcement grades to represent the mean material strengths used currently in Turkey. Five reinforced concrete buildings with different shear wall ratios are designed following the requirements of Turkish Earthquake Code (2007), TS 498 (1987) and TS 500 (2000) with shear wall ratios of 0.00%, 0.50%, 1.00%, 1.50% and 2.00% in both longitudinal and transverse directions.

While designing these four-story structures, the floor plan, story height, number of stories, configuration and cross-sections of the reinforced concrete beams and columns are kept the same with the existing school building. However, the material properties, reinforcement detailing and shear wall configurations are modified. Shear walls are placed as symmetrically as possible to prevent having torsional irregularity in the building as can be observed from Figures 3.15 through 3.19 in Chapter 3. Torsional irregularity is eliminated for the buildings with shear walls, but the designed frame building with no shear walls has torsional irregularity according to Table 2.1 and Figure 2.1 of the Turkish Earthquake Code (2007):

$$(\Delta_i)_{ort} = \frac{1}{2}\left[(\Delta_i)_{max} + (\Delta_i)_{min}\right] \quad (4.1)$$

where,

(Δ_i) : Interstory drift of the ith floor of the building,

$(\Delta_i)_{ort}$: Average interstory drift of the ith floor of the building,

$$\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{ort} \quad (4.2)$$

where,

η_{bi} : Torsional irregularity coefficient of i^{th} floor of the building.

Additional eccentricity effects are taken into account while calculating interstory drifts used in this equation. If torsional irregularity coefficient of any floor of the building is greater than 1.2, there is torsional irregularity at the building like the designed building with no reinforced concrete shear walls. Torsional irregularity coefficients of first, third and fourth floor of this building are 1.22, 1.24 and 1.29 respectively in the Y-direction.

4.2.2.1 Base Shear Carried by Reinforced Concrete Shear Walls

The percentage of the total base shear force carried by reinforced concrete shear walls of the buildings with increasing shear wall ratios are compared to determine the contribution of shear walls in carrying the applied lateral loads. Figures 4.2 and 4.3 show the shear wall contribution when the maximum base shear force is reached for each earthquake record in the X- and Y-directions, respectively. The trend lines passing through the average data points represent the variation of percentage of base shear force carried by shear walls with increasing shear wall ratios.

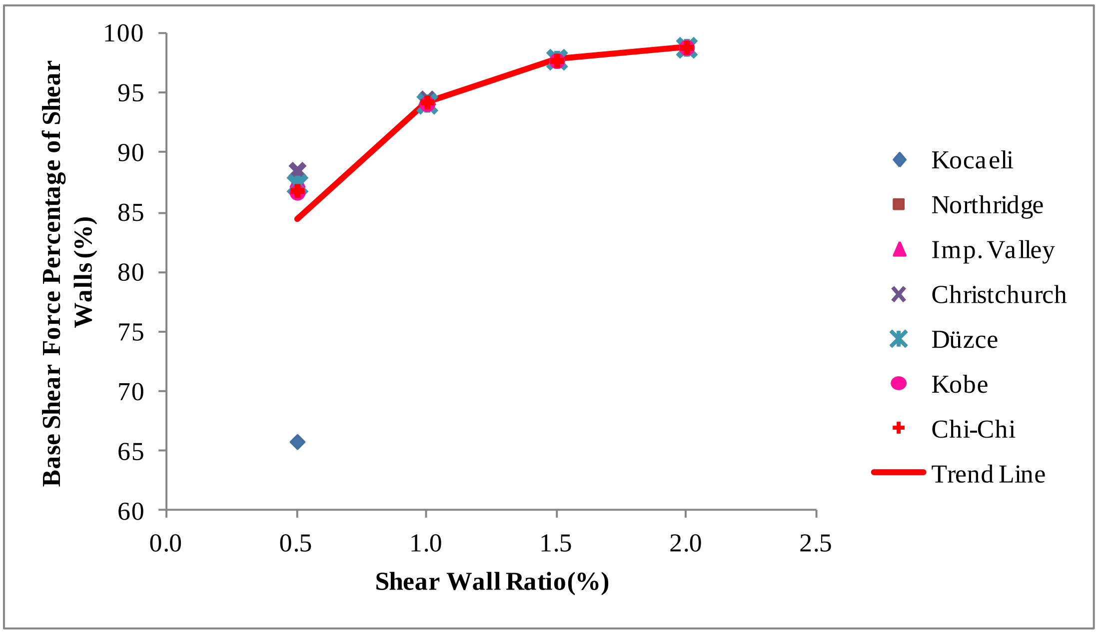


Figure 4.2 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio in the X-direction of the Designed Buildings

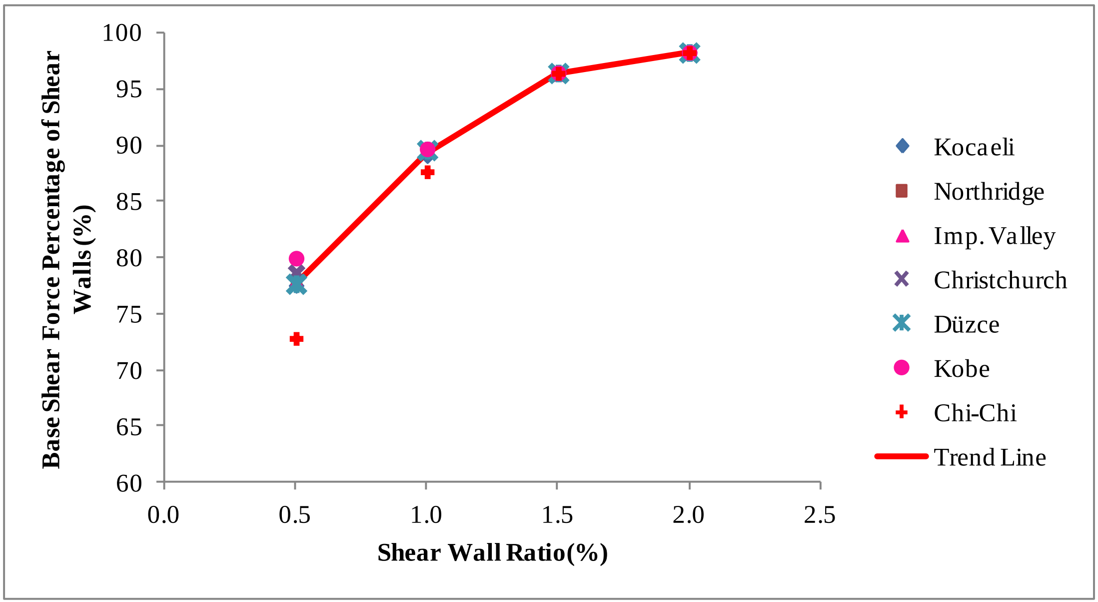


Figure 4.3 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio in the Y-direction of the Designed Buildings

It can be easily observed from these figures that the base shear percentage carried by shear walls increase with increasing shear wall ratios, but this trend reduces for high shear wall ratios, especially greater than 1.50%. Base shear carried by shear walls for each earthquake record is nearly 95% for the designed building with 1.00% reinforced concrete shear wall ratio in the X-direction and after this point, the increase is insignificant for higher shear wall ratios. On the other hand, when the shear wall ratio is only 0.50%, almost 15% and 25% of the total base shear was carried by columns in the X and Y-directions. These figures verify that the columns of wall-frame structures, especially with low shear wall ratios should be designed to carry some percentage of base shear. Furthermore, there is at most 10% difference in the shear wall contribution to carrying base shear when buildings with 1.00%, 1.50% and 2.00% shear wall ratios are compared. Therefore, increasing the shear wall ratio significantly does not always mean the shear walls will contribute more in carrying the lateral loads.

4.2.2.2 Roof Drift

Roof drift vs. shear wall ratio graphs of the designed building in the X and Y-directions are given in Figures 4.4 and 4.5, respectively. These results are obtained by using maximum roof drifts at the peak earthquake records and the trend lines are also shown in these figures.

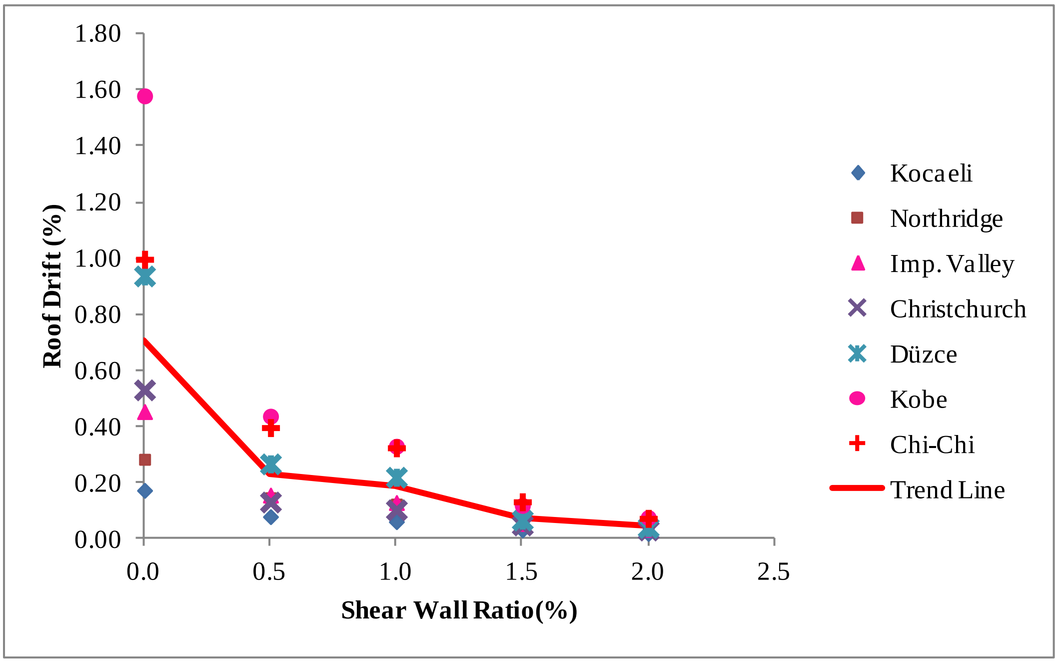


Figure 4.4 Roof Drift vs. Shear Wall Ratio in the X-direction of the Designed Buildings

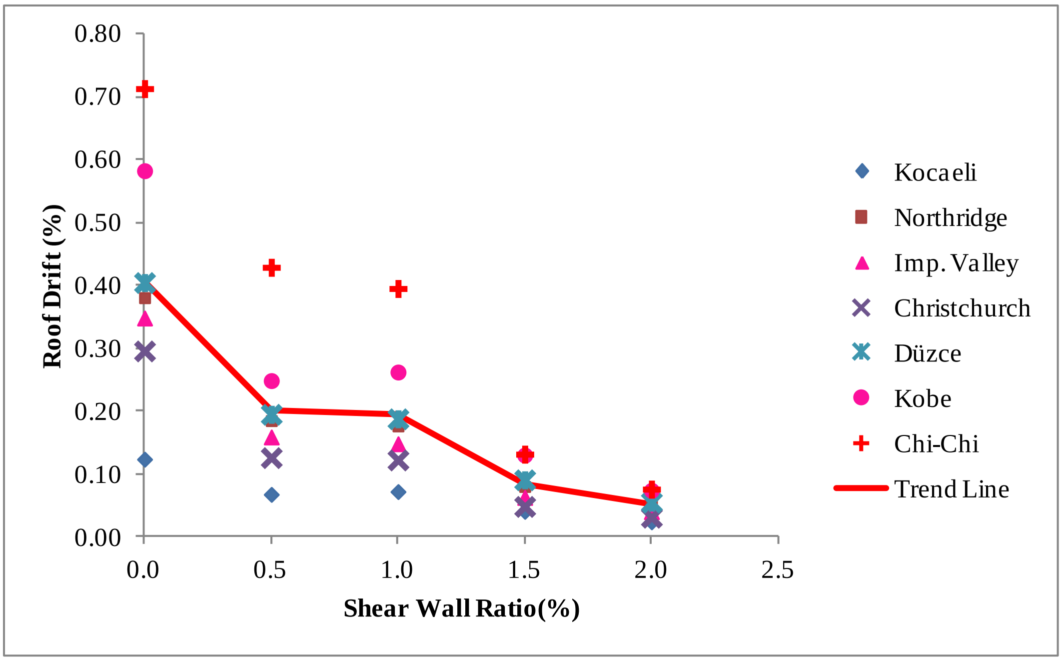


Figure 4.5 Roof Drift vs. Shear Wall Ratio in the Y-direction of the Designed Buildings

In Figure 4.4, the roof drift obtained by applying the Kobe Earthquake is greater than that of the Chi-Chi Earthquake due to the larger PGA value of the X-component of the Kobe Earthquake compared to the Chi-Chi Earthquake. However, the PGA value of the Y-component of Kobe Earthquake is smaller than that of Chi-Chi Earthquake and thus, the roof drift of Chi-Chi Earthquake in the Y-direction is greater than that of Kobe Earthquake (Figure 4.5). Moreover, the proximity of the PGA values of the Northridge, Imperial Valley and Christchurch Earthquakes, the roof drifts of the designed buildings under these earthquake records are close to each other.

As expected, the roof drifts of the designed buildings reduce with increasing shear wall ratios and the roof drifts of the buildings in the X-direction is greater than in the Y-direction because of the orientation of the reinforced concrete columns. The trend lines indicate that the average roof drifts of the designed buildings with 0.50% and 1.00% shear wall ratios are close to each other in both directions and the roof drift of the designed building with 1.50% shear wall ratio is notably smaller than this value. However, it should be noted that significant plastic deformations, which lead to the formation of a failure mechanism, are observed in the buildings with 0.50% shear wall ratios compared to the ones with 1.00% shear wall ratios especially under the earthquakes which have higher PGA values such as Kobe and Chi-Chi Earthquakes. Moreover, there is only negligible plastic deformations in the buildings with 1.50% shear wall ratio, therefore, it can be stated that there could only be negligible plastic deformations in the designed reinforced concrete buildings which have 1.50% and 2.00% shear wall ratios.

In this analytical study, the earthquake that has the lowest PGA values is the Kocaeli Earthquake. Therefore, none of the designed buildings, even the one with no shear walls have any plastic deformations and the roof drifts are significantly low under this earthquake loading (Figures 4.6 through 4.10). However, as the PGA values increase, starting from Northridge Earthquake, plastic deformations are observed in the reinforced concrete buildings with 0.50% and 1.00% shear wall ratios and roof drifts increase (Figures 4.11 through 4.15). For the buildings with shear wall ratios higher than 1.00%, no plastic deformation is detected.

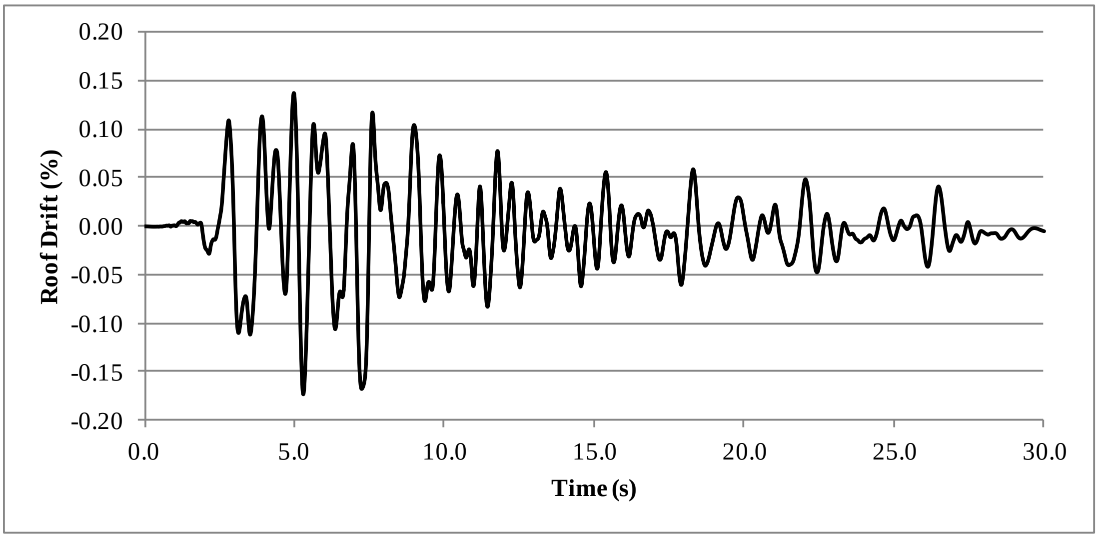


Figure 4.6 Roof Drift vs. Time in the X-direction of the Designed Building with no Shear Wall under Kocaeli Earthquake

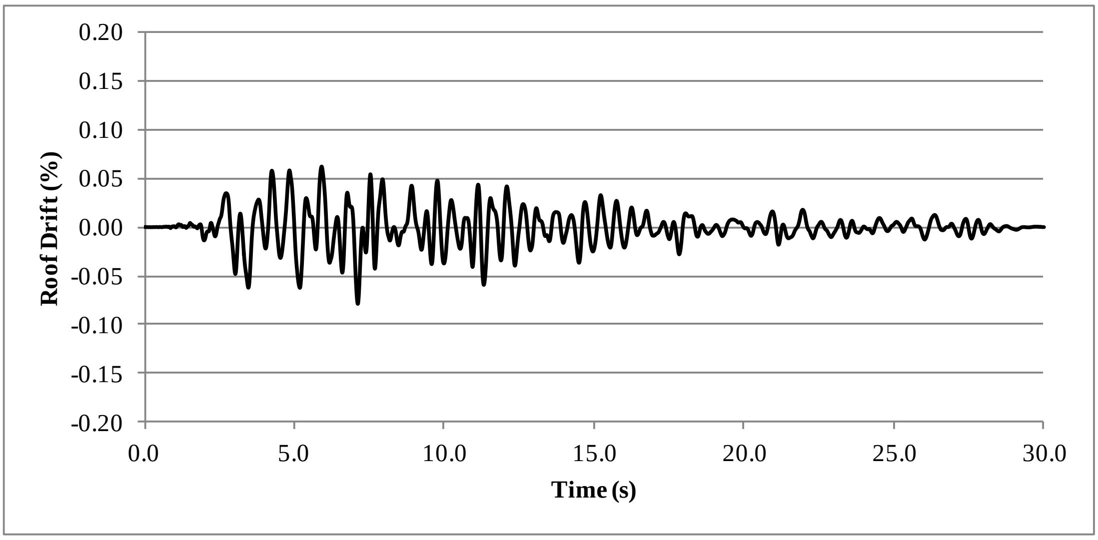


Figure 4.7 Roof Drift vs. Time in the X-direction of the Designed Building with 0.50% Shear Wall Ratio under Kocaeli Earthquake

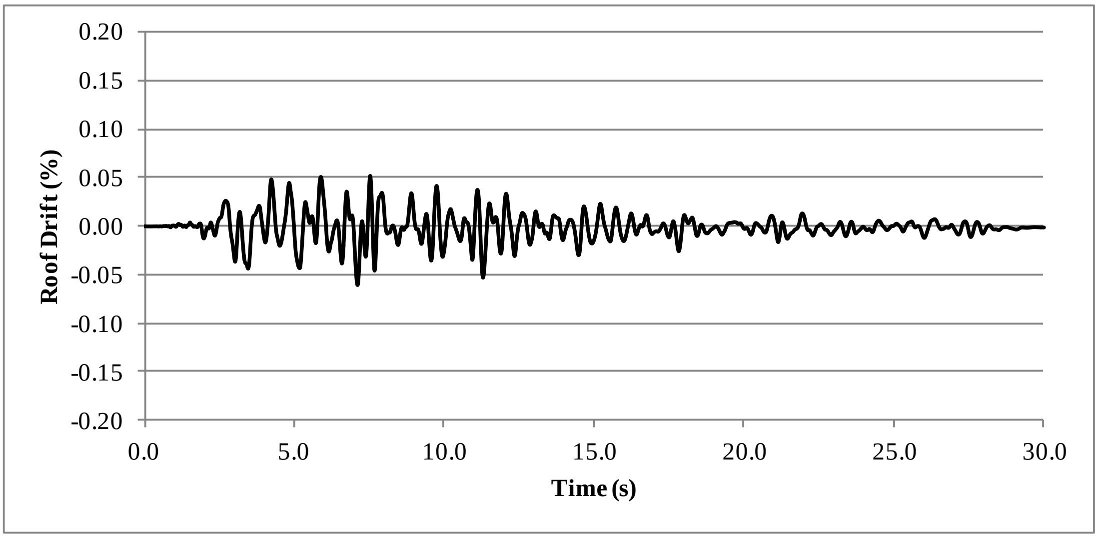


Figure 4.8 Roof Drift vs. Time in the X-direction of the Designed Building with 1.00% Shear Wall Ratio under Kocaeli Earthquake

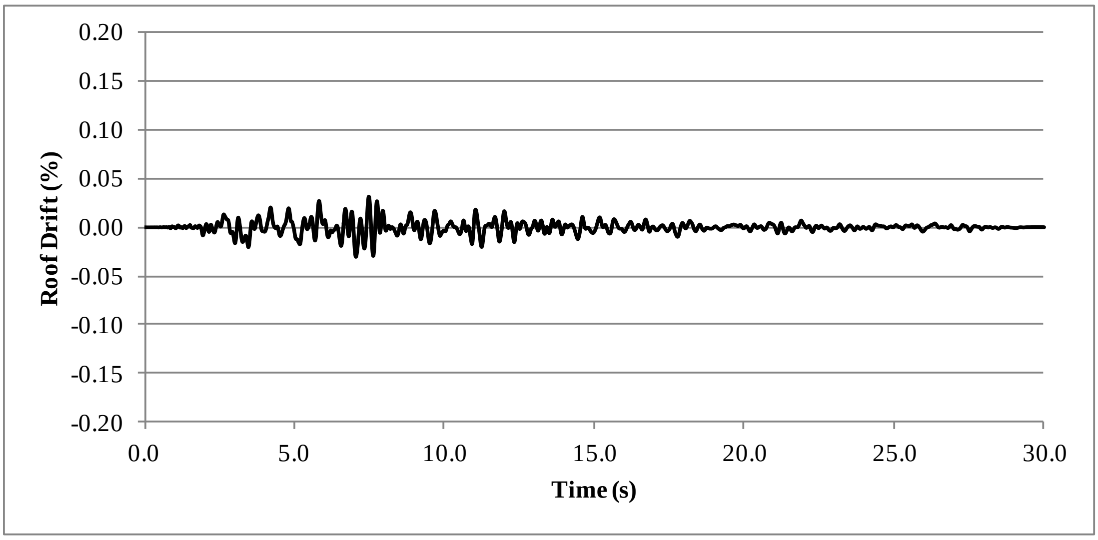


Figure 4.9 Roof Drift vs. Time in the X-direction of the Designed Building with 1.50% Shear Wall Ratio under Kocaeli Earthquake

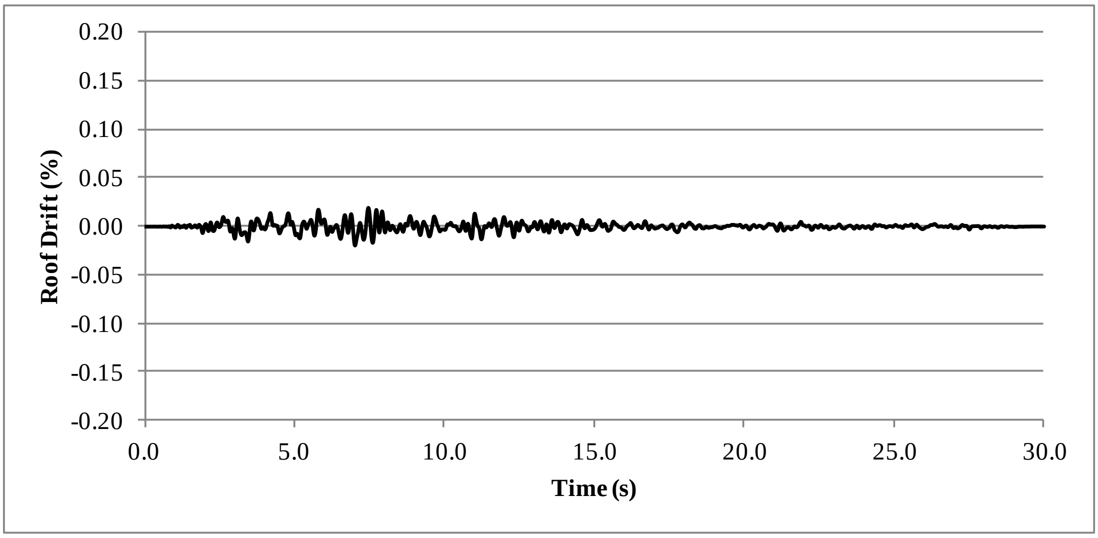


Figure 4.10 Roof Drift vs. Time in the X-direction of the Designed Building with 2.00% Shear Wall Ratio under Kocaeli Earthquake

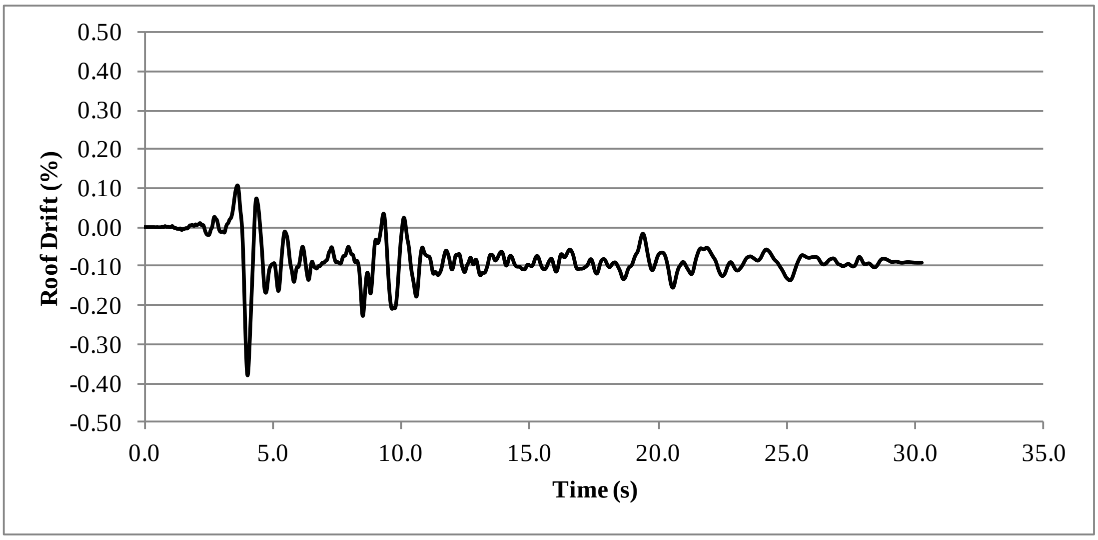


Figure 4.11 Roof Drift vs. Time in the Y-direction of the Designed Building with no Shear Wall under Northridge Earthquake

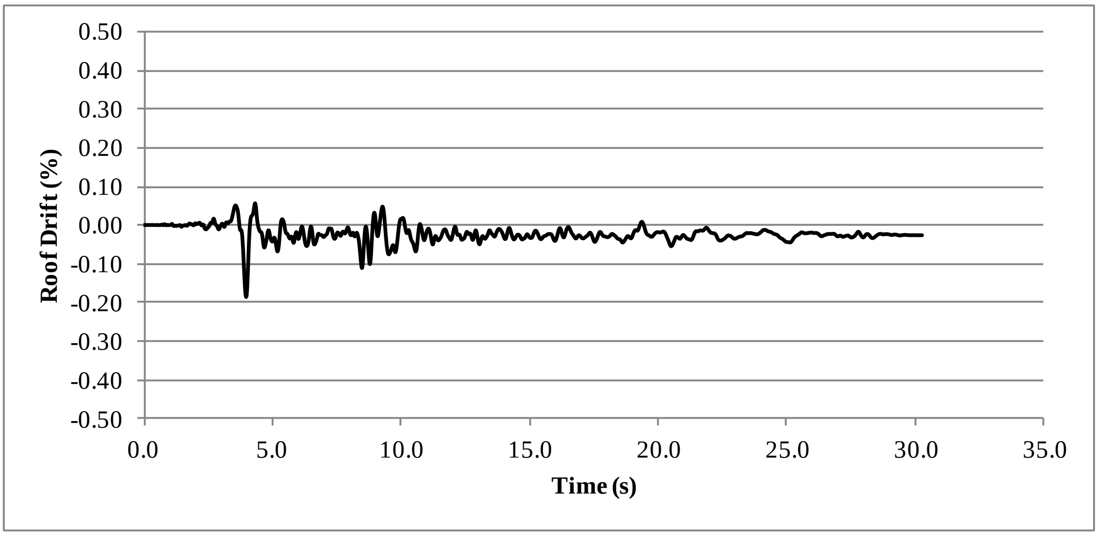


Figure 4.12 Roof Drift vs. Time in the Y-direction of the Designed Building with 0.50% Shear Wall Ratio under Northridge Earthquake

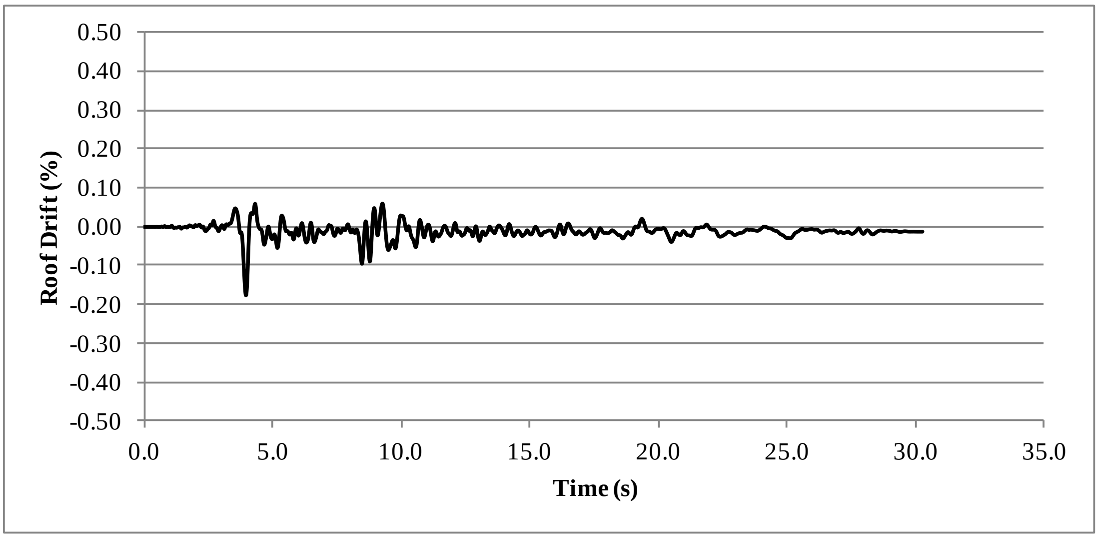


Figure 4.13 Roof Drift vs. Time in the Y-direction of the Designed Building with 1.00% Shear Wall Ratio under Northridge Earthquake

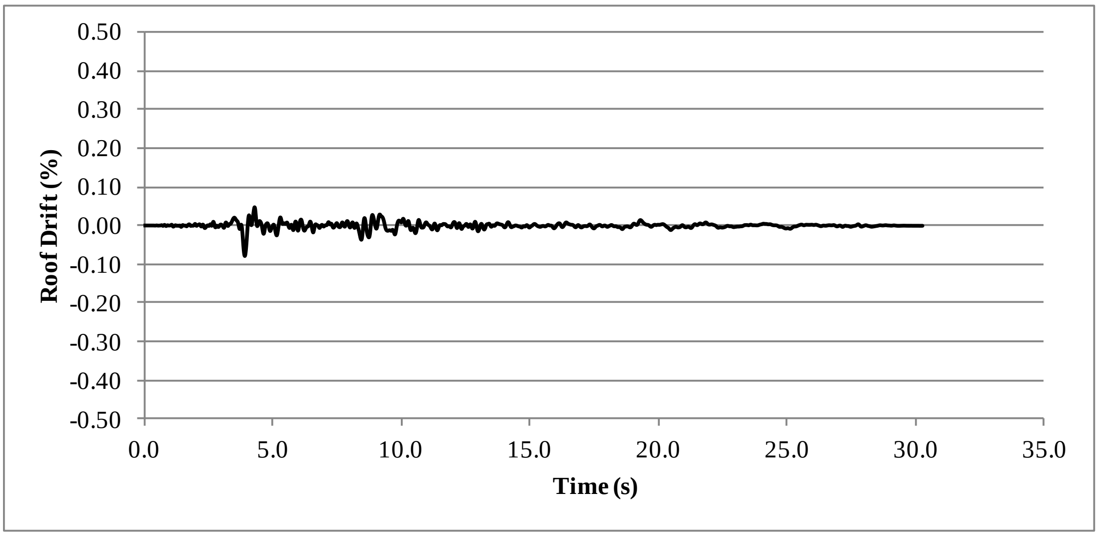


Figure 4.14 Roof Drift vs. Time in the Y-direction of the Designed Building with 1.50% Shear Wall Ratio under Northridge Earthquake

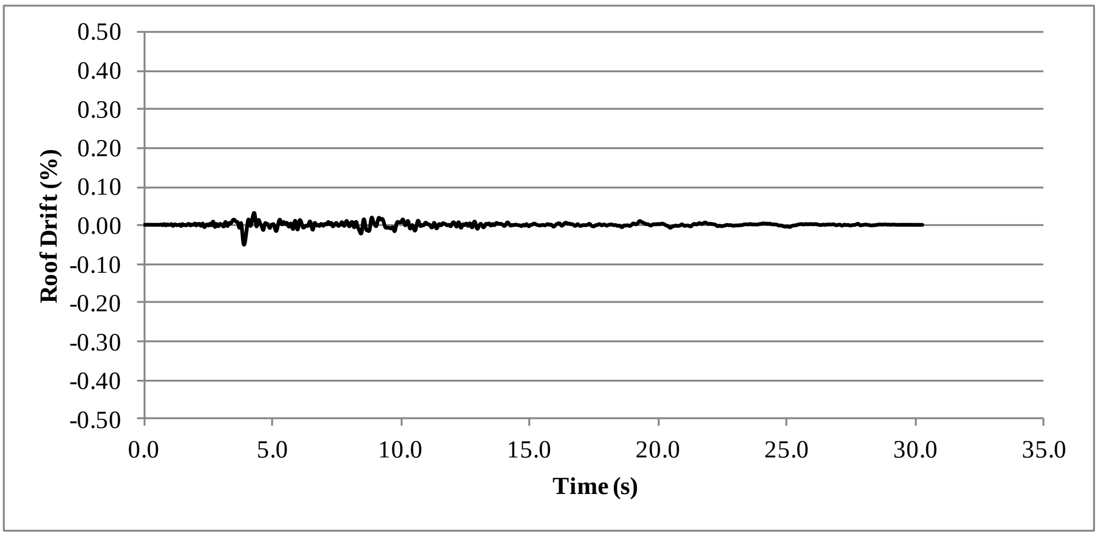


Figure 4.15 Roof Drift vs. Time in the Y-direction of the Designed Building with 2.00% Shear Wall Ratio under Northridge Earthquake

The roof drift vs. time graphs of the designed buildings in the Y-direction are given in Figures 4.16, through 4.20 under Kobe Earthquake, which has the highest PGA values in this direction. In Figure 4.16, the behavior of the structure for the overall duration of the earthquake cannot be provided, because of the failure of this building which has no shear wall at the first peak. The permanent drift of the building with 0.50% shear wall ratio due to the significant plastic deformations is easily noticeable compared to the buildings with higher shear wall ratios under Kobe Earthquake. It can be concluded from these figures that, using 1.50% or higher shear wall ratios minimizes the plastic deformation of the reinforced concrete buildings. The maximum roof drift values of all building models, under seven selected earthquakes are given in Appendix A.11.

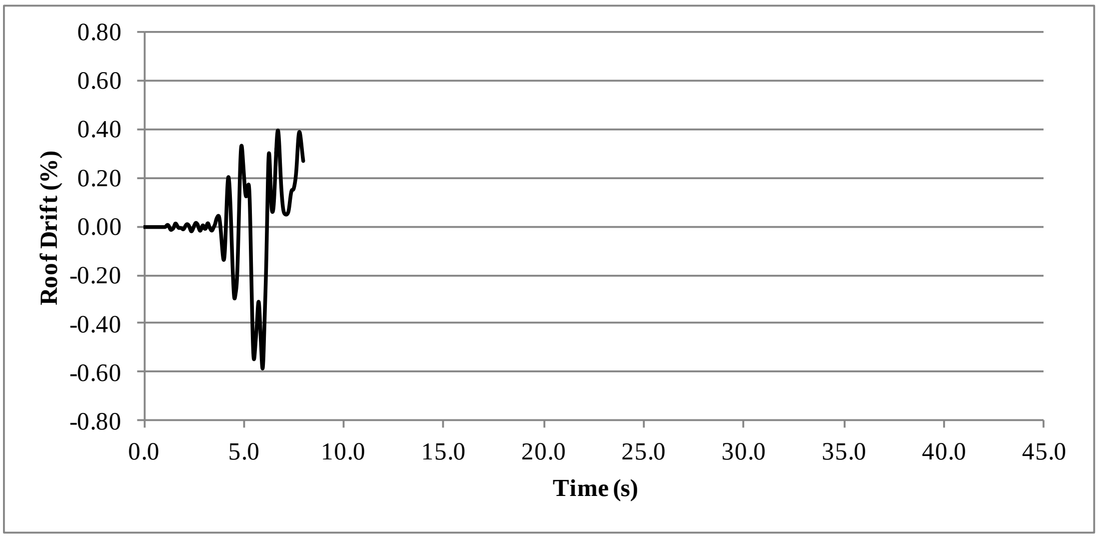


Figure 4.16 Roof Drift vs. Time in the Y-direction of Designed Building with no Shear Wall under Kobe Earthquake

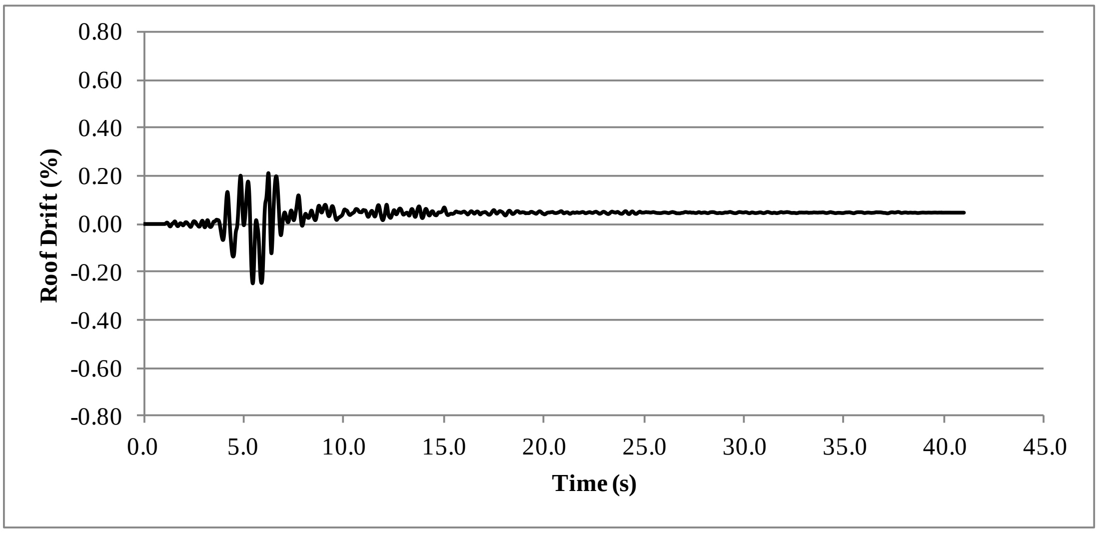


Figure 4.17 Roof Drift vs. Time in the Y-direction of the Designed Building with 0.50% Shear Wall Ratio under Kobe Earthquake

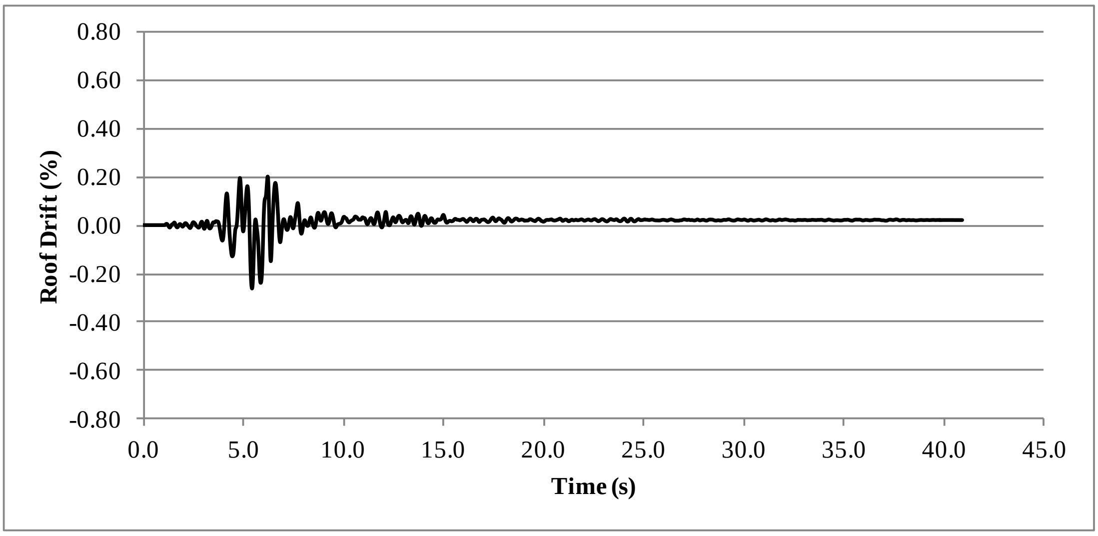


Figure 4.18 Roof Drift vs. Time in the Y-direction of the Designed Building with 1.00% Shear Wall Ratio under Kobe Earthquake

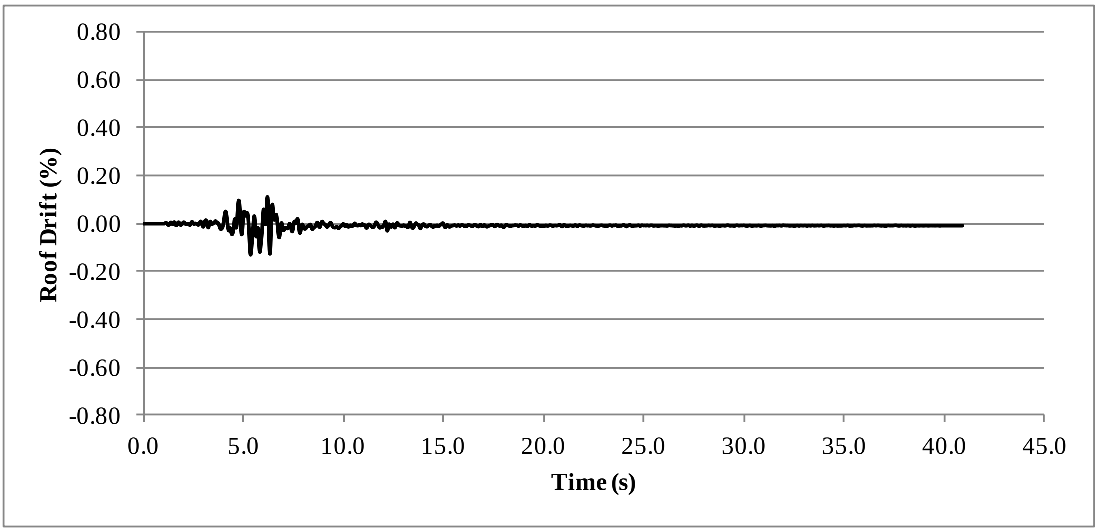


Figure 4.19 Roof Drift vs. Time in the Y-direction of the Designed Building with 1.50% Shear Wall Ratio under Kobe Earthquake

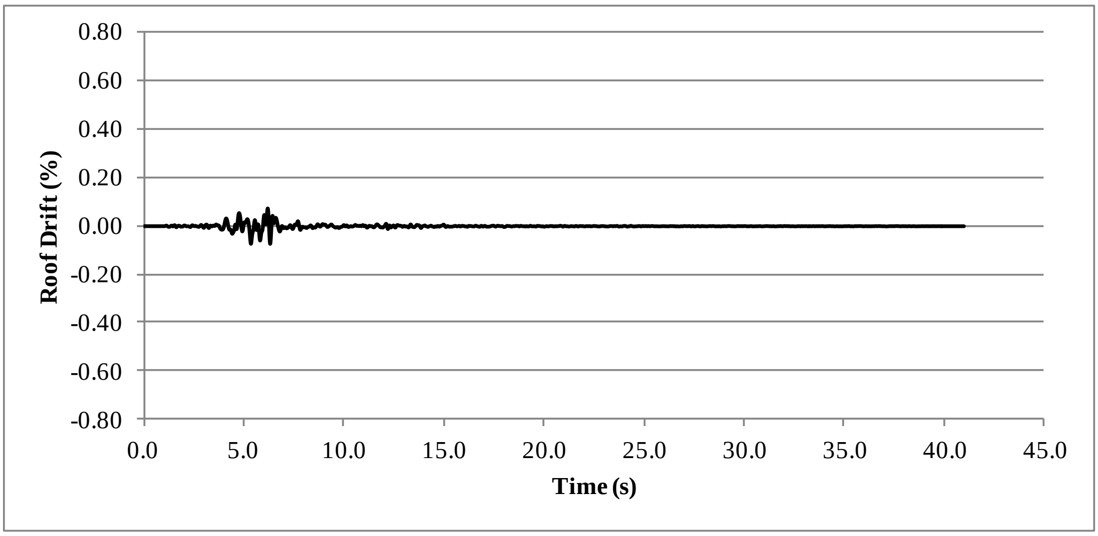


Figure 4.20 Roof Drift vs. Time in the Y-direction of the Designed Building with 2.00% Shear Wall Ratio under Kobe Earthquake

4.2.2.3 Base Shear versus Roof Drift Relationship

First, the Kocaeli Earthquake with smallest PGA values is utilized to investigate the base shear vs. roof drift relationship of designed buildings with 0.00% to 2.00% shear wall ratios in the X-direction and the responses are given in Figures 4.21 through 4.25. In these figures, elastic oscillation can be observed for all buildings under Kocaeli Earthquake record. The roof drifts of the buildings decrease and the base shears increase with increasing shear wall ratios.

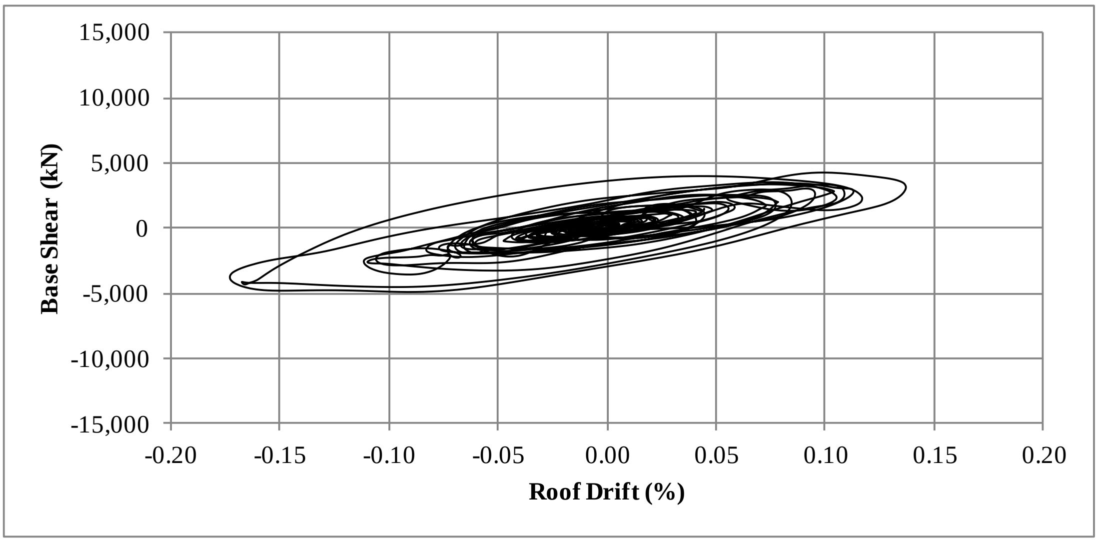


Figure 4.21 Base Shear vs. Roof Drift Relationship in the X-direction of the Designed Building with no Shear Wall under Kocaeli Earthquake

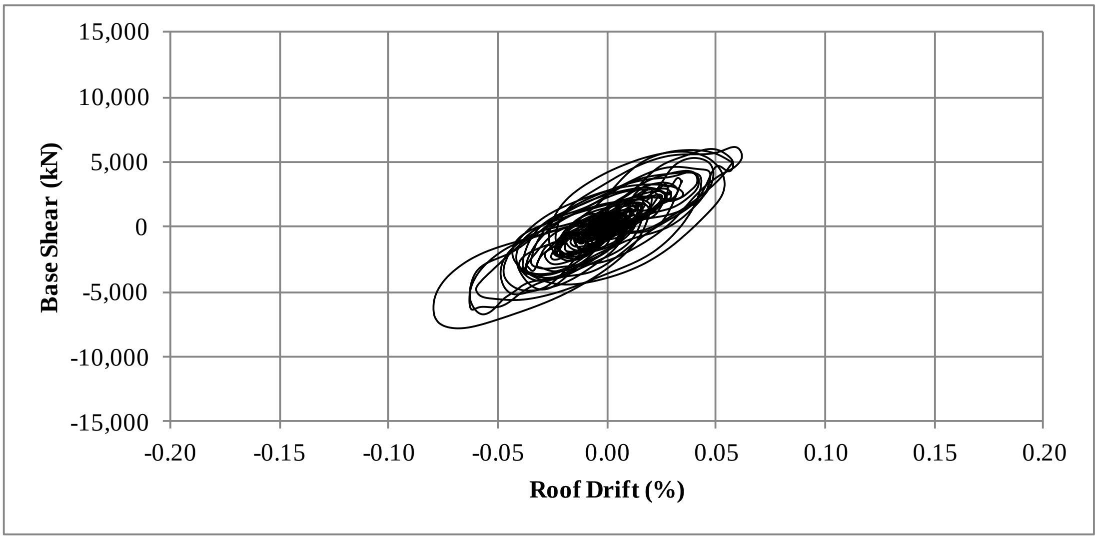


Figure 4.22 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 0.50% Shear Wall Ratio under Kocaeli Earthquake

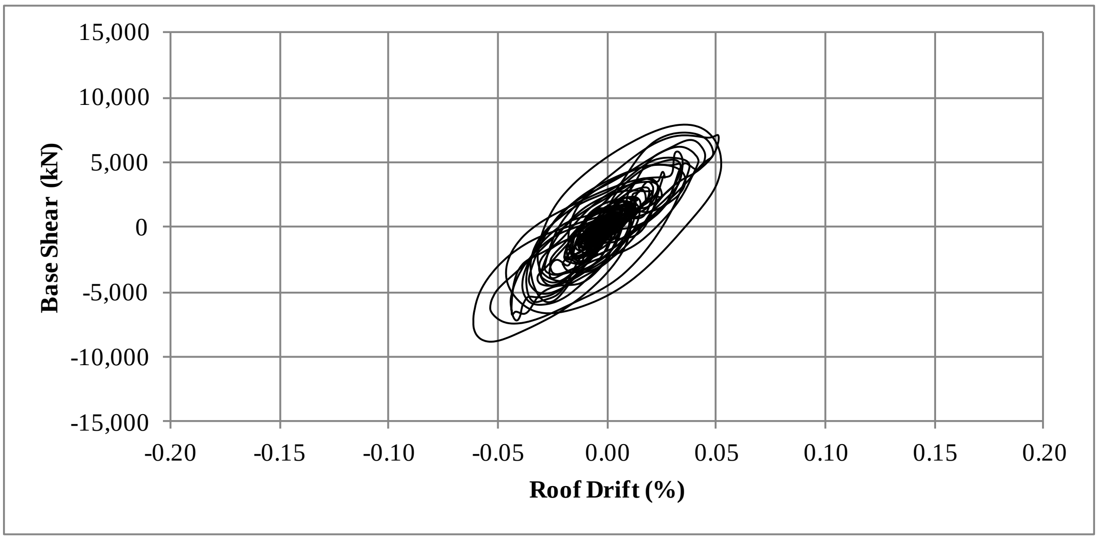


Figure 4.23 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 1.00% Shear Wall Ratio under Kocaeli Earthquake

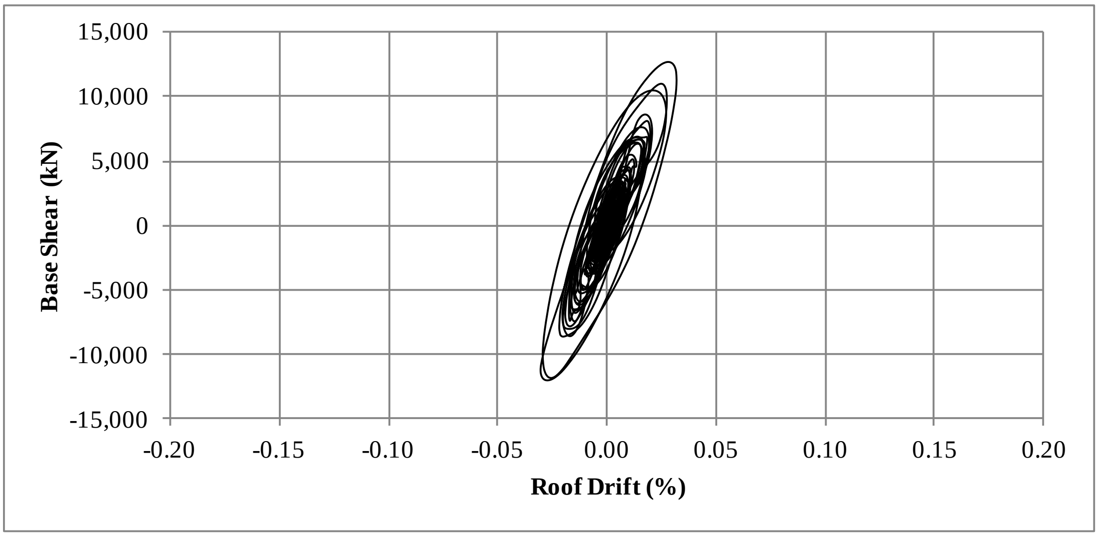


Figure 4.24 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 1.50% Shear Wall Ratio under Kocaeli Earthquake

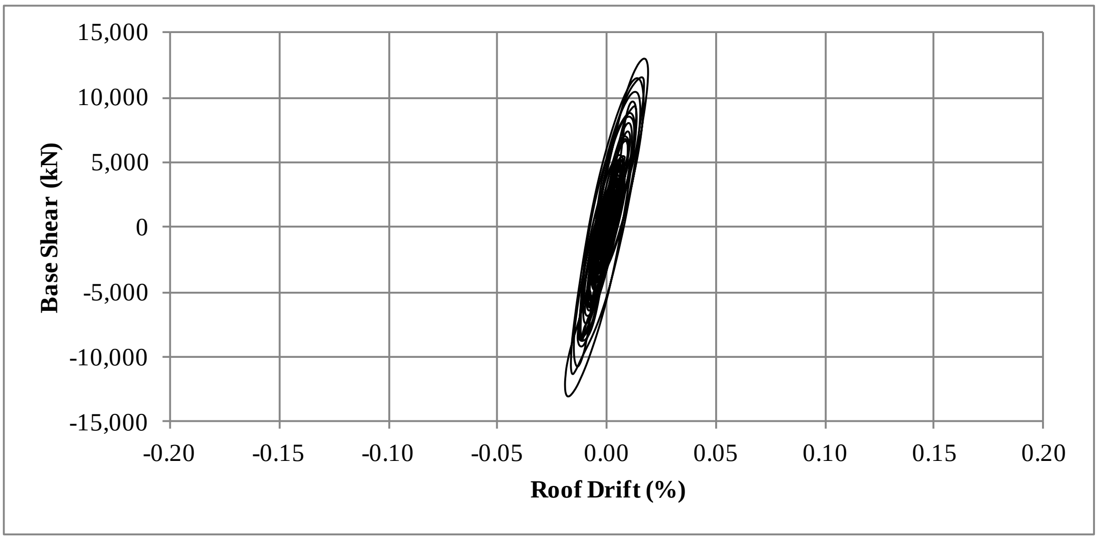


Figure 4.25 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 2.00% Shear Wall Ratio under Kocaeli Earthquake

Under Imperial Valley Earthquake plastic deformations are initializing and base shear force and roof drift values are greater than the ones under Kocaeli Earthquake (Figures 4.26 through 4.30). As expected, base shear force and roof drift values under Chi-Chi Earthquake is the largest because of the highest PGA values. Furthermore, the plastic deformations become more significant for the buildings with 0.00%, 0.50% and 1.00% shear wall ratios (Figures 4.31, 4.32 and 4.33). However, negligible plastic deformations can be observed for the designed buildings with shear wall ratios higher than 1.00% (Figures 4.34 and 4.35).

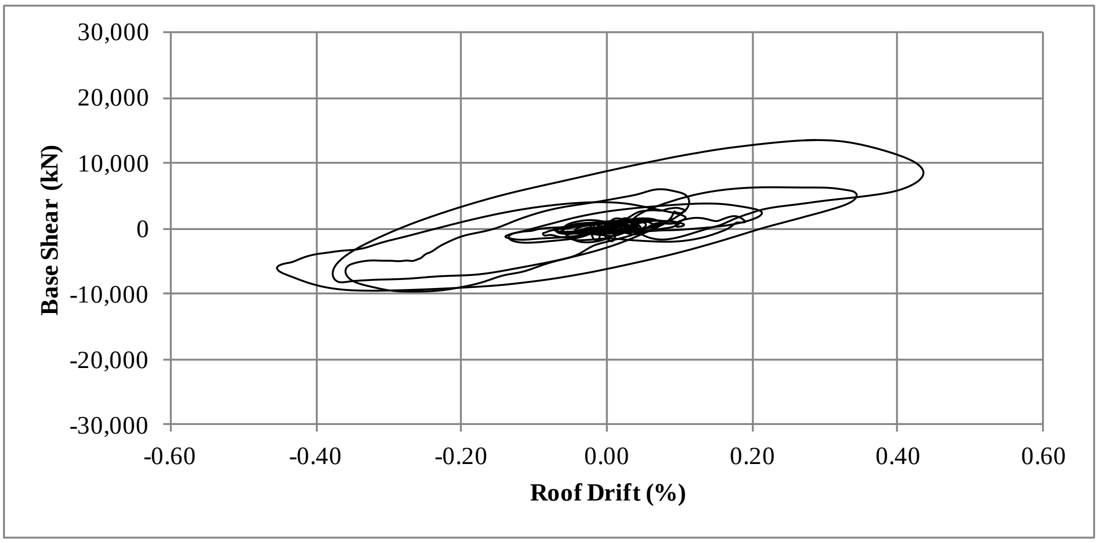


Figure 4.26 Base Shear vs. Roof Drift in the X-direction of the Designed Building with no Shear Wall under Imperial Valley Earthquake

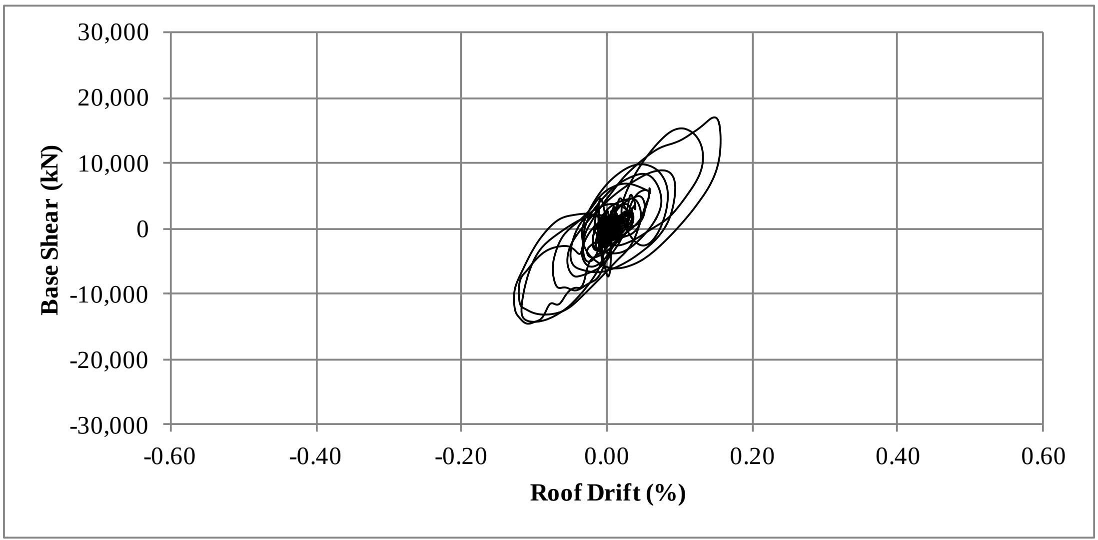


Figure 4.27 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 0.50% Shear Wall Ratio under Imperial Valley Earthquake

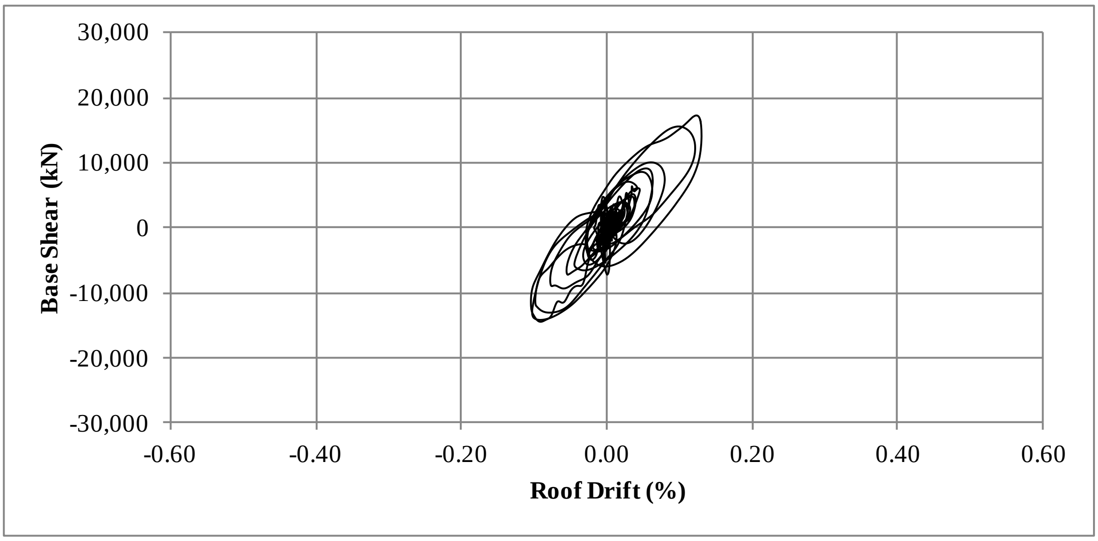


Figure 4.28 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 1.00% Shear Wall Ratio under Imperial Valley Earthquake

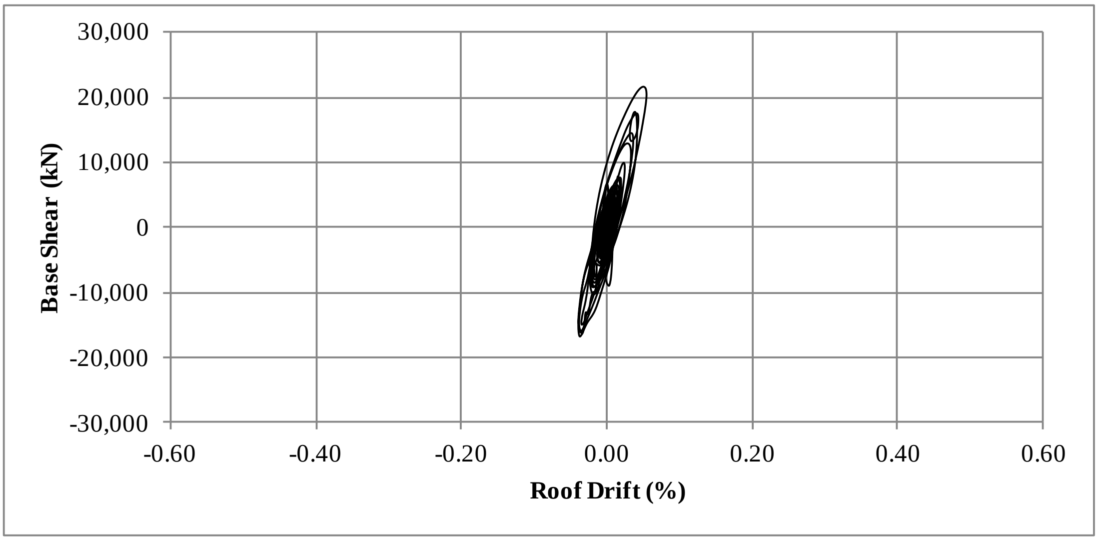


Figure 4.29 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 1.50% Shear Wall Ratio under Imperial Valley Earthquake

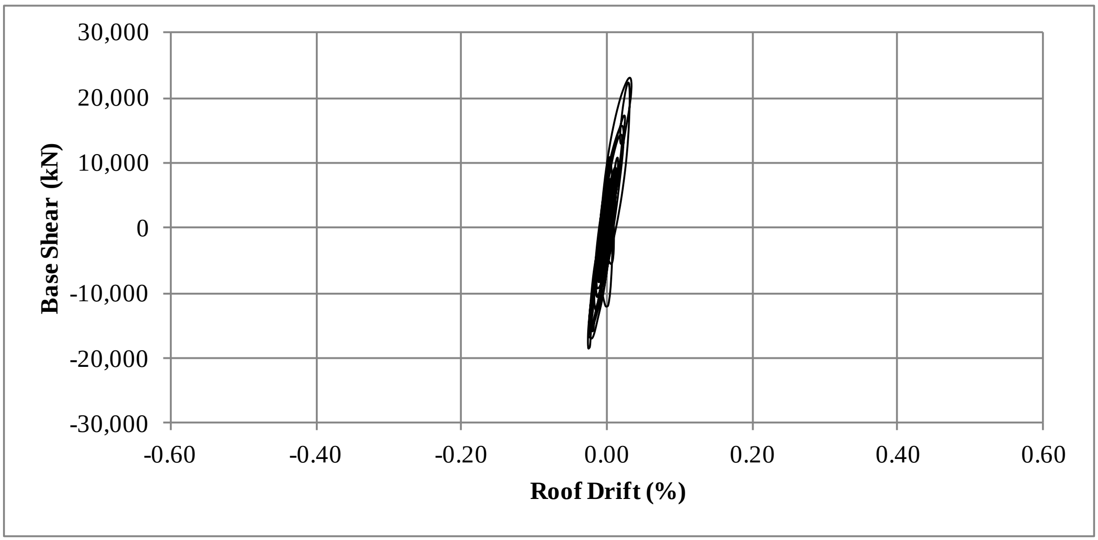


Figure 4.30 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 2.00% Shear Wall Ratio under Imperial Valley Earthquake

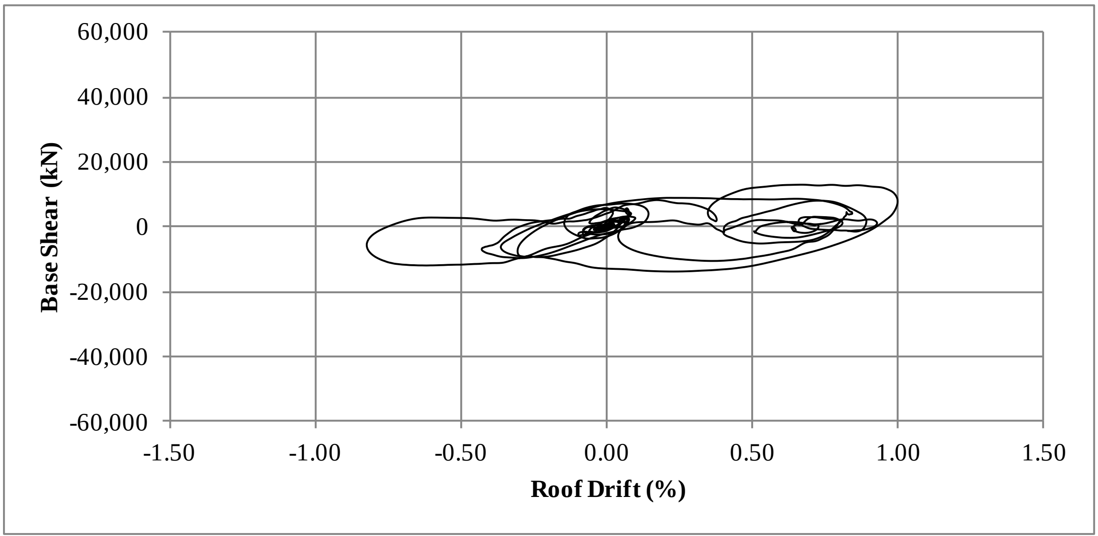


Figure 4.31 Base Shear vs. Roof Drift in the X-direction of the Designed Building with no Shear Wall under Chi-Chi Earthquake

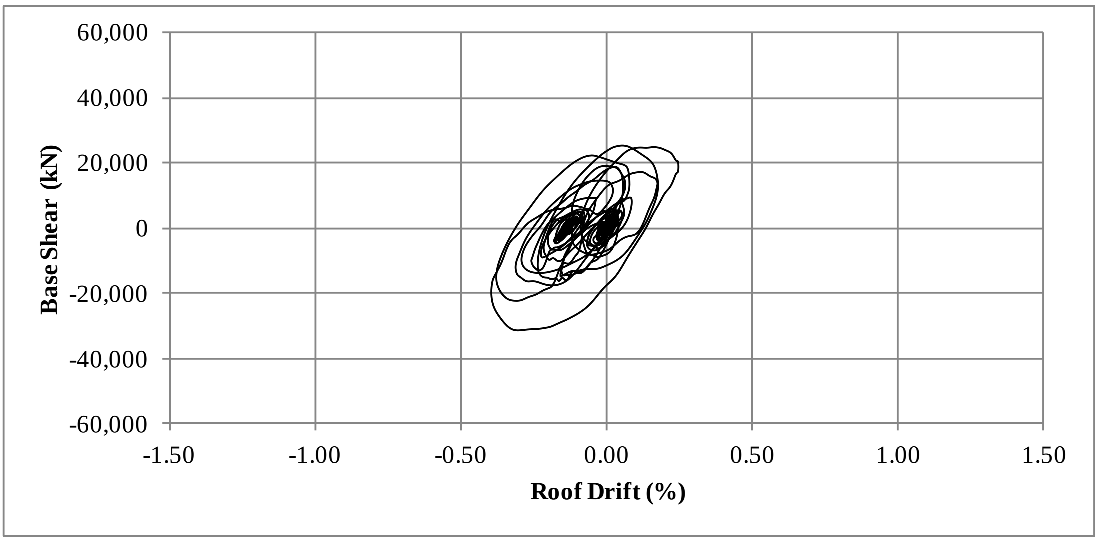


Figure 4.32 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 0.50% Shear Wall Ratio under Chi-Chi Earthquake

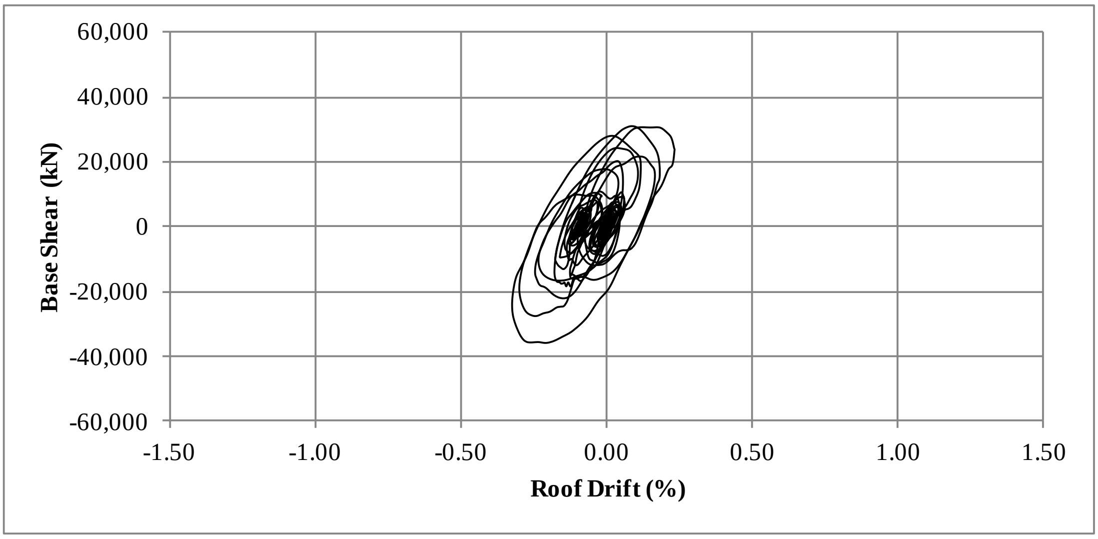


Figure 4.33 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 1.00% Shear Wall Ratio under Chi-Chi Earthquake

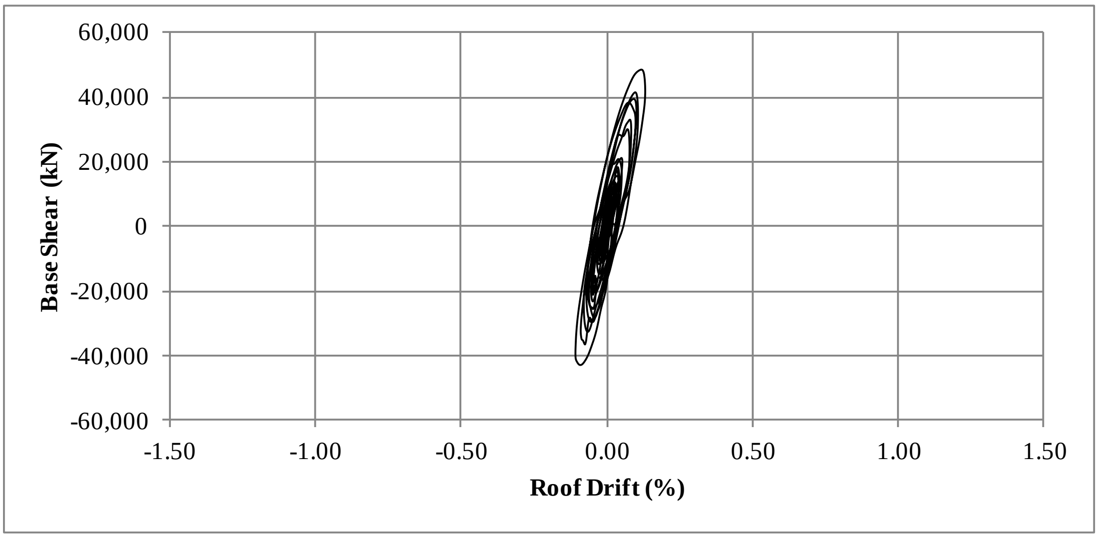


Figure 4.34 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 1.50% Shear Wall Ratio under Chi-Chi Earthquake

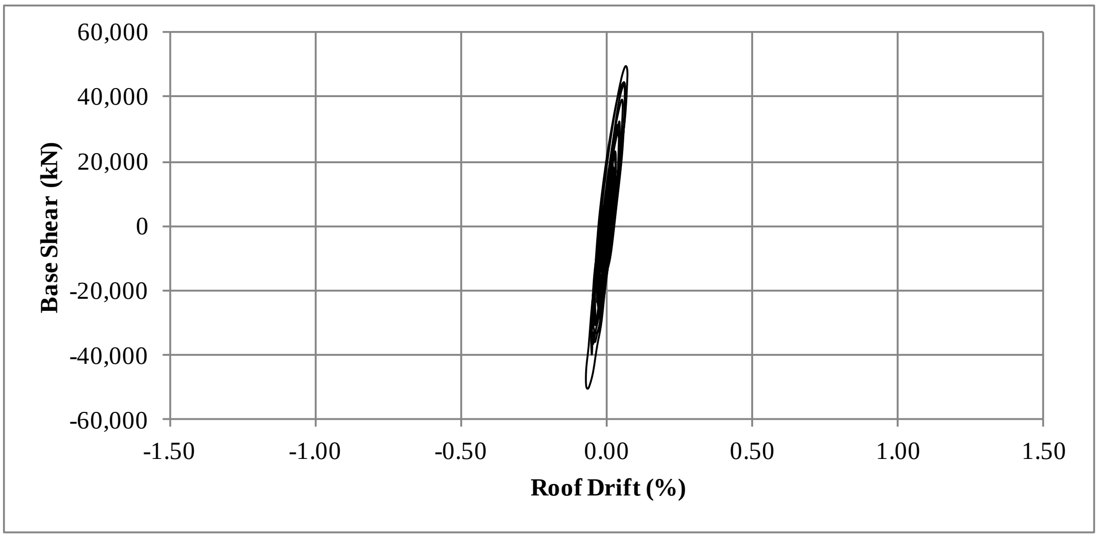


Figure 4.35 Base Shear vs. Roof Drift in the X-direction of the Designed Building with 2.00% Shear Wall Ratio under Chi-Chi Earthquake

The relationship between maximum base shear and maximum roof drift of the designed buildings under selected earthquake records is also investigated and given in Figures 4.36, through 4.39 for buildings with 0.00% and 0.50% shear wall ratios in both principal directions. The maximum base shear vs. maximum roof drift relationship for the buildings with 1.00%, 1.50% and 2.00% shear wall ratios under selected earthquake records is provided in Appendix A.12, A.13 and A.14. The maximum roof drifts are determined at the instant when maximum base shear forces are obtained for each earthquake record. These figures give envelope curves for the buildings under increasing PGA values of the earthquake records in both directions as if a pushover analysis is performed on the structures. With increasing shear wall ratios, the increase of base shear forces and the decrease of roof drift can also be observed. Furthermore, the highest maximum roof drift values in the X-direction are under Kobe Earthquake, because the X- component of this earthquake record has the highest PGA value. On the other hand, Chi-Chi Earthquake has the highest PGA value in the Y-direction and therefore, the maximum roof drifts of the buildings in this direction correspond to this earthquake record. The maximum roof drifts in the X-direction are higher when compared to the ones in the Y-direction due to the orientation of the columns; therefore these buildings are redesigned, as referred to Second Case, to examine the effect of column stiffness on the structural performance. The maximum base shear and maximum roof drift values of all designed buildings under selected earthquakes are given in tabular form in Appendix A.11.

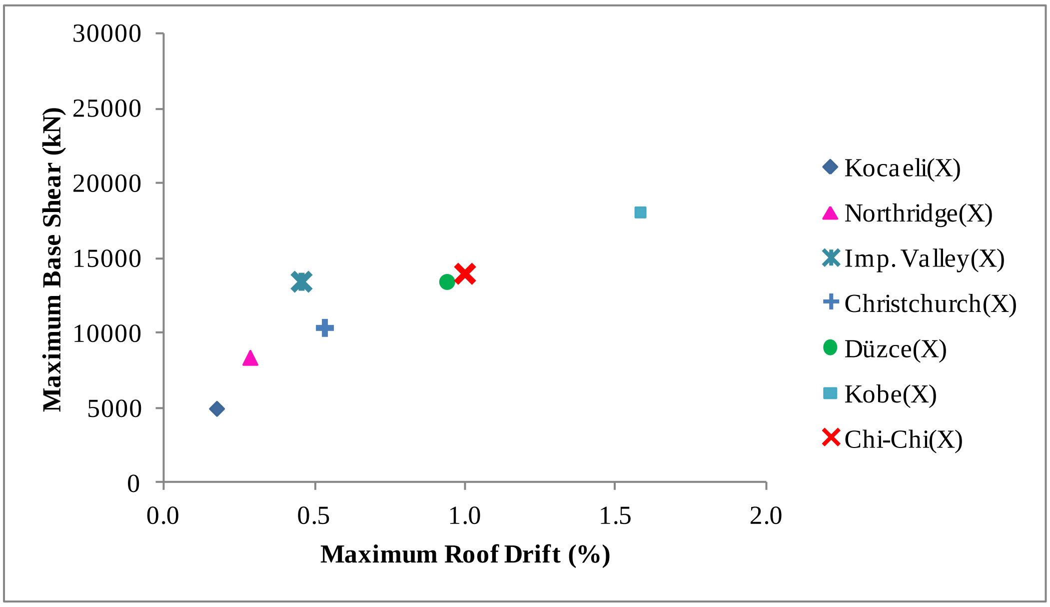


Figure 4.36 Maximum Base Shear vs. Maximum Roof Drift in the X-direction of the Designed Building with no Shear Wall under All Earthquake Records

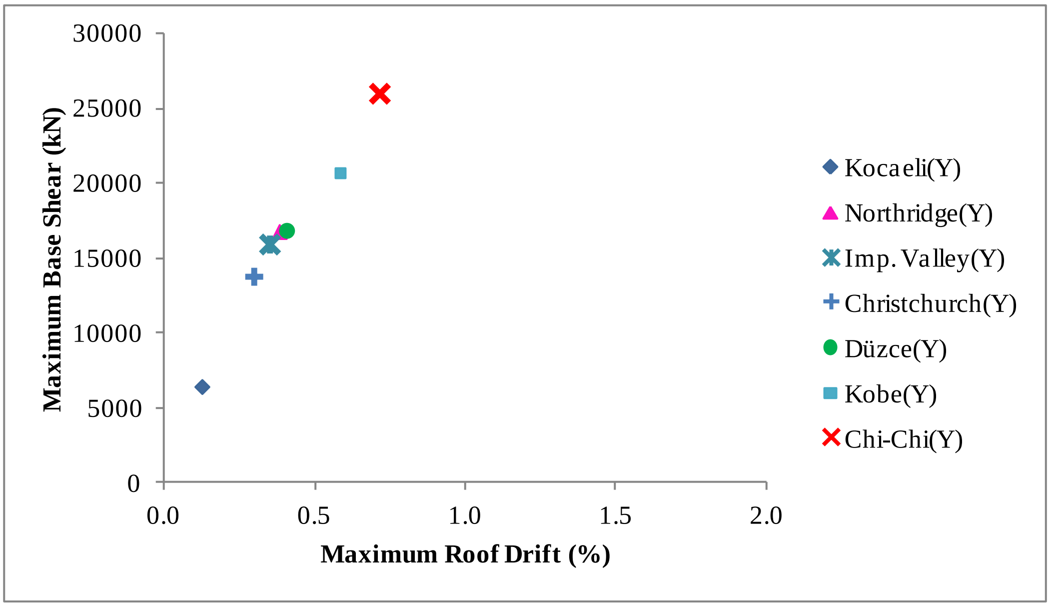


Figure 4.37 Maximum Base Shear vs. Maximum Roof Drift in the Y-direction of the Designed Building with no Shear Wall under All Earthquake Records

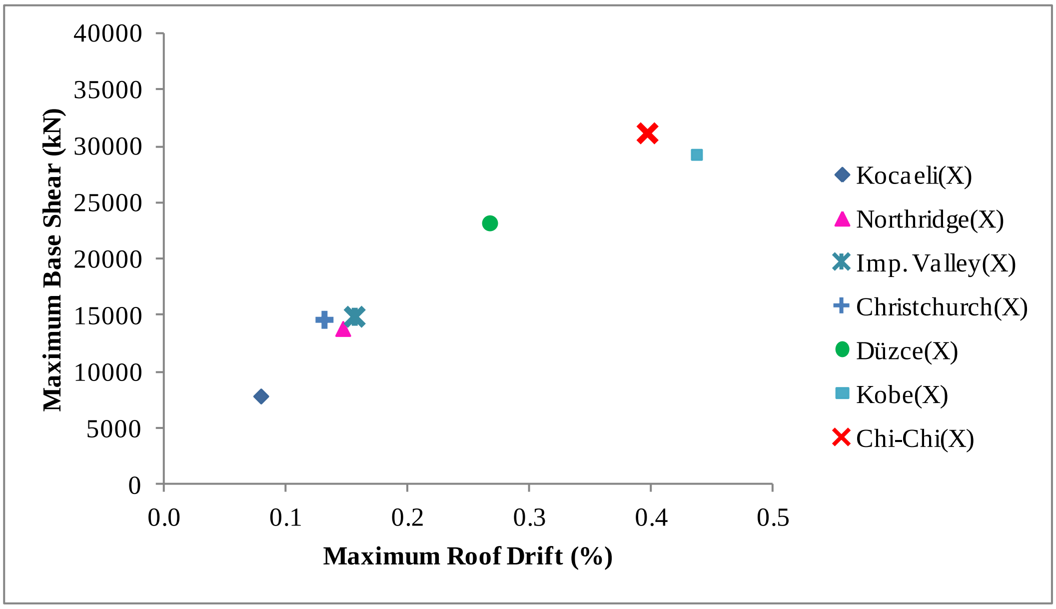


Figure 4.38 Maximum Base Shear vs. Maximum Roof Drift in the X-direction of the Designed Building with 0.50% Shear Wall Ratio under All Earthquake Records

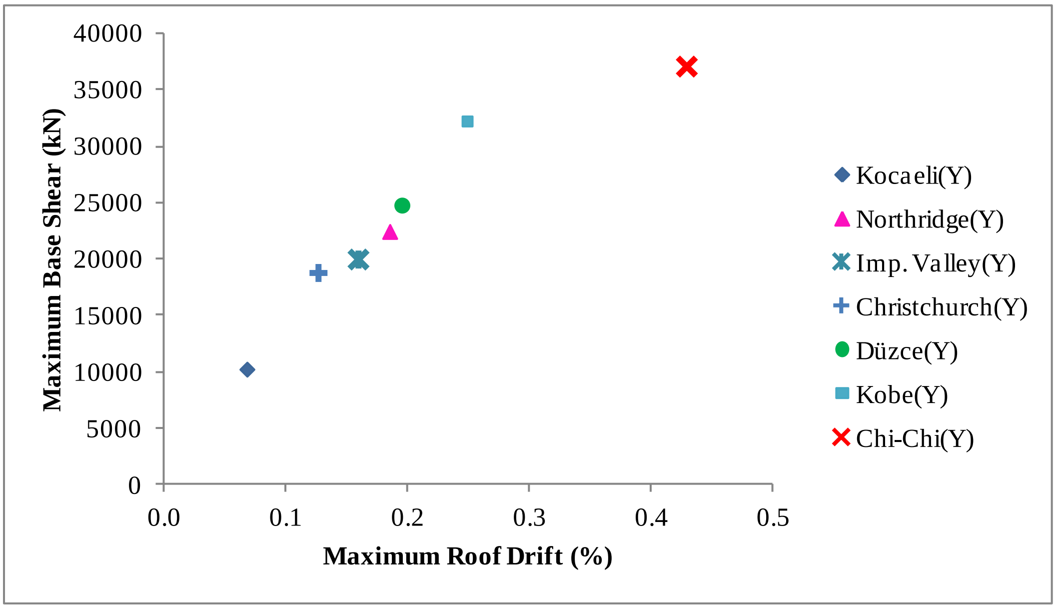


Figure 4.39 Maximum Base Shear vs. Maximum Roof Drift in the Y-direction of the Designed Building with 0.50% Shear Wall Ratio under All Earthquake Records

4.2.2.4 Comparison of the Seismic Performance of the Designed Buildings

4.2.2.4.1 Yielding of Members

The seismic performance of the designed buildings in terms of inelastic behavior observed in the individual members is also investigated. For this purpose, percentage of the yielded reinforced concrete members, such as beams, columns and shear walls at each floor level under three selected earthquakes, Düzce, Kobe and Chi-Chi Earthquakes, that have the highest PGA values in both directions are obtained at the time the building reaches its maximum base shear.

Tables 4.1 and 4.2 show the percentage of yielded members of the designed buildings in the X and Y-directions, respectively. From these tables, it can be observed that, the percentage yielded members decrease with increasing shear wall ratios. The buildings with 1.50% and 2.00% shear wall ratios have only a limited number of yielded vertical elements, which reduces the risk of formation of a failure mechanism. As expected, almost all members of the designed building with no shear wall are yielded under the selected earthquakes in both directions. The yielding does not penetrate into the third and fourth story levels of the reinforced concrete shear walls in any one of the designed buildings.

The yielding of the beams is not significantly affected by the varying shear wall ratios; however, there is a slight reduction in the percentage of beams that are yielded with increasing shear wall ratios under selected earthquakes in both directions. Nevertheless, the percentage of yielded members increase with increasing PGA values of earthquake records, for example the percentage of the yielded members under Kobe Earthquake is greater than the one under Düzce Earthquake for each building.

Table 4.1 Percentage of the Yielded Members for the Designed Buildings in the X-direction

X-Direction			Percentage of Yielded Members (%)			
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**	**Story 4**
Düzce	**0.0-0.0**	**Columns**	100.0	100.0	100.0	50.0
	0.5-0.5	**Columns**	0.0	0.0	0.0	0.0
	1.0-1.0	**Columns**	0.0	0.0	0.0	0.0
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	97.8	97.8	8.9
	0.5-0.5	**Beams**	61.0	80.5	70.3	9.8
	1.0-1.0	**Beams**	50.0	67.6	51.4	51.4
	1.5-1.5	**Beams**	29.4	51.5	39.4	15.2
	2.0-2.0	**Beams**	0.0	20.7	17.2	13.8
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	75.0	0.0	0.0	0.0
	1.0-1.0	**Shear Walls**	25.0	0.0	0.0	0.0
	1.5-1.5	**Shear Walls**	0.0	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0
Kobe	**0.0-0.0**	**Columns**	100.0	100.0	100.0	100.0
	0.5-0.5	**Columns**	90.5	100.0	100.0	95.0
	1.0-1.0	**Columns**	90.0	89.3	82.1	75.0
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	97.8	77.8
	0.5-0.5	**Beams**	95.2	97.6	97.6	58.5
	1.0-1.0	**Beams**	94.7	97.2	97.2	75.7
	1.5-1.5	**Beams**	64.7	69.7	57.6	48.5
	2.0-2.0	**Beams**	63.3	69.0	69.0	62.1
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	100.0	75.0	0.0	0.0
	1.0-1.0	**Shear Walls**	100.0	37.5	0.0	0.0
	1.5-1.5	**Shear Walls**	33.3	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0
Chi-Chi	**0.0-0.0**	**Columns**	100.0	100.0	100.0	72.9
	0.5-0.5	**Columns**	100.0	97.5	72.5	77.5
	1.0-1.0	**Columns**	90.0	71.4	60.7	60.7
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	97.8	8.9
	0.5-0.5	**Beams**	90.5	97.6	90.2	24.4
	1.0-1.0	**Beams**	84.2	91.9	86.5	64.9
	1.5-1.5	**Beams**	85.3	66.7	60.6	60.6
	2.0-2.0	**Beams**	63.3	62.1	62.1	51.7
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	100.0	50.0	0.0	0.0
	1.0-1.0	**Shear Walls**	50.0	25.0	0.0	0.0
	1.5-1.5	**Shear Walls**	25.0	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	18.8	0.0	0.0	0.0

Table 4.2 Percentage of Yielded Members for the Designed Buildings in Y-direction

Y-Direction			Percentage of Yielded Members (%)			
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**	**Story 4**
Düzce	**0.0-0.0**	**Columns**	100.0	100.0	100.0	50.0
	0.5-0.5	**Columns**	0.0	0.0	0.0	0.0
	1.0-1.0	**Columns**	0.0	0.0	0.0	0.0
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	100.0	5.3
	0.5-0.5	**Beams**	100.0	100.0	100.0	18.4
	1.0-1.0	**Beams**	82.5	84.2	78.9	34.2
	1.5-1.5	**Beams**	41.7	41.2	41.2	17.6
	2.0-2.0	**Beams**	12.5	13.3	13.3	13.3
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	75.0	25.0	0.0	0.0
	1.0-1.0	**Shear Walls**	25.0	0.0	0.0	0.0
	1.5-1.5	**Shear Walls**	12.5	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0
Kobe	**0.0-0.0**	**Columns**	100.0	100.0	100.0	100.0
	0.5-0.5	**Columns**	90.5	100.0	100.0	95.0
	1.0-1.0	**Columns**	90.0	89.3	82.1	75.0
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	100.0	78.9
	0.5-0.5	**Beams**	100.0	100.0	100.0	71.1
	1.0-1.0	**Beams**	100.0	100.0	100.0	60.5
	1.5-1.5	**Beams**	50.0	52.9	44.1	23.5
	2.0-2.0	**Beams**	21.9	33.3	30.0	20.0
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	100.0	100.0	0.0	0.0
	1.0-1.0	**Shear Walls**	100.0	50.0	0.0	0.0
	1.5-1.5	**Shear Walls**	25.0	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0
Chi-Chi	**0.0-0.0**	**Columns**	100.0	100.0	100.0	72.9
	0.5-0.5	**Columns**	100.0	97.5	72.5	77.5
	1.0-1.0	**Columns**	90.0	71.4	60.7	60.7
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	100.0	100.0
	0.5-0.5	**Beams**	100.0	100.0	100.0	97.4
	1.0-1.0	**Beams**	100.0	100.0	100.0	92.1
	1.5-1.5	**Beams**	44.4	50.0	44.1	17.6
	2.0-2.0	**Beams**	28.1	33.3	33.3	20.0
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	100.0	50.0	0.0	0.0
	1.0-1.0	**Shear Walls**	100.0	25.0	0.0	0.0
	1.5-1.5	**Shear Walls**	25.0	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0

4.2.2.4.2 Effect of Shear Wall Percentage on Earthquake Load Reduction Factor (R)

Earthquake load reduction factors of the first case of designed buildings with varying shear wall ratios are obtained following the requirements of Turkish Earthquake Code (2007) by utilizing the following equation:

$$R = 10-\alpha_s, \qquad (4.3)$$

where,

α_s : Base shear percentage carried by high ductility shear walls.

First, total base shear forces and base shear forces carried by shear walls in both longitudinal and transverse directions are determined by equivalent static load method. Then, base shear percentages carried by high ductility shear walls are obtained and earthquake load reduction factors are calculated according to Equation 4.3 to study the effect of varying shear wall percentage on the earthquake load reduction factor as shown in Table 4.3.

Table 4.3 Effect of Varying Shear Wall Percentage on the Earthquake Load Reduction Factor of the First Case of Designed Building based on the requirements of Turkish Earthquake Code (2007)

	Shear Wall Ratio (%)	Direction	Total Base Shear (kN)	Base Shear Carried by Shear Walls (kN)	Base Shear Percentage Carried by Shear Walls (α_s)	R
Designed Building (First Case)	0.0-0.0	X	2458	0	0.00	8.00
	0.0-0.0	Y	2458	0	0.00	8.00
	0.5-0.5	X	4012	3509	0.87	6.50
	0.5-0.5	Y	4013	3046	0.76	6.96
	1.0-1.0	X	4601	4335	0.94	6.23
	1.0-1.0	Y	4601	4072	0.89	6.46
	1.5-1.5	X	5238	5116	0.98	6.09
	1.5-1.5	Y	5238	5022	0.96	6.16
	2.0-2.0	X	5873	5798	0.99	6.05
	2.0-2.0	Y	5873	5754	0.98	6.08

In this table, the earthquake load reduction factors of the first case of designed building with no shear wall are selected as 8 based on Table 2.5 in Turkish Earthquake Code (2007). On the other hand, earthquake load reduction factors of the first case of designed buildings including shear walls are obtained as values rather close to 6 especially for buildings with shear wall ratios higher than 0.50%, which indicates the shear walls dominate the seismic behavior of the buildings.

4.2.2.4.3 Drift Limitations

Maximum interstory drift values for each story level of the first case of designed buildings with varying shear wall ratios are obtained following the requirements of Turkish Earthquake Load (2007 as shown in Table 4.4

Table 4.4 Drift Values of the First Case of Designed Buildings based on Turkish Earthquake Code (2007)

	Shear Wall Ratio (%)	Direction	R	Maximum Story Displacement at i^{th} story level (d_{imax}) (mm)	Maximum Interstory Drift at i^{th} story level (δ_{imax}) (%)
Designed Building (First Case)	0.0-0.0	X	8.00	3.00	0.77
	0.0-0.0	Y	8.00	1.60	0.41
	0.5-0.5	X	6.50	1.30	0.27
	0.5-0.5	Y	6.96	1.00	0.22
	1.0-1.0	X	6.23	1.00	0.20
	1.0-1.0	Y	6.46	0.90	0.19
	1.5-1.5	X	6.09	0.50	0.10
	1.5-1.5	Y	6.16	0.40	0.08
	2.0-2.0	X	6.05	0.40	0.08
	2.0-2.0	Y	6.08	0.30	0.06

It can be observed from this table that, for all the shear wall ratios of the first case of designed buildings in both longitudinal and transverse directions, the maximum drift values are lower than the drift limit provided in the specifications of Turkish Earthquake Code (2007) as 2.0 %.

4.2.3 Analytical Results of Designed Building-Second Case

As mentioned in Chapter 3, based on the existing building, Sarıyer MEV Dumlupınar Primary School, another set of buildings, referred to as the second case, are designed that have the same properties as the first case, except for the column orientations are designed following the requirements of Turkish Earthquake Code (2007), TS 498 (1987) and TS 500 (2000). In this case, all the reinforced concrete columns are rotated 90 degrees to increase the stiffness in the X-direction of the designed buildings. Shear wall configurations for the second case are the same with the first case to prevent torsional irregularity and enable the comparison of the two cases. Based on the Table 2.1 and Figure 2.1 of the Turkish Earthquake Code (2007), there is no torsional irregularity in any of the designed buildings for the second case.

4.2.3.1 Base Shear Carried by Reinforced Concrete Shear Walls

Figures 4.40 and 4.41 present the variation of the base shear carried by shear walls with increasing shear wall ratios in X and Y-directions, respectively and the corresponding trend lines. From these figures, it can easily be observed that, the percentage base shear carried by the shear walls for the first case in X and Y-directions almost switch for the second case due to the column orientation. Therefore, it can be concluded that the orientation of the columns is an important parameter that influences contribution of the reinforced concrete shear walls in carrying the applied lateral loads. Depending on their orientation, the columns of the designed buildings with 0.50% shear wall ratio can carry approximately %25 of the total base shear. Therefore, it cannot be assumed that, reinforced concrete shear walls are carrying almost all the base shear of the reinforced concrete building with 0.50% shear wall ratio.

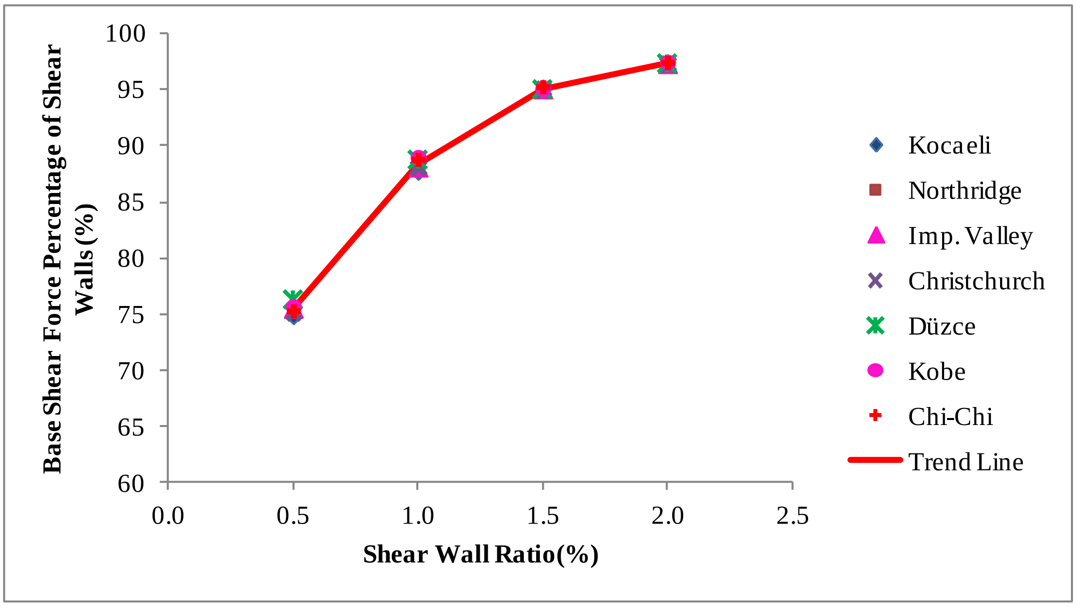


Figure 4.40 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio in the X-direction of the Second Case of the Designed Buildings

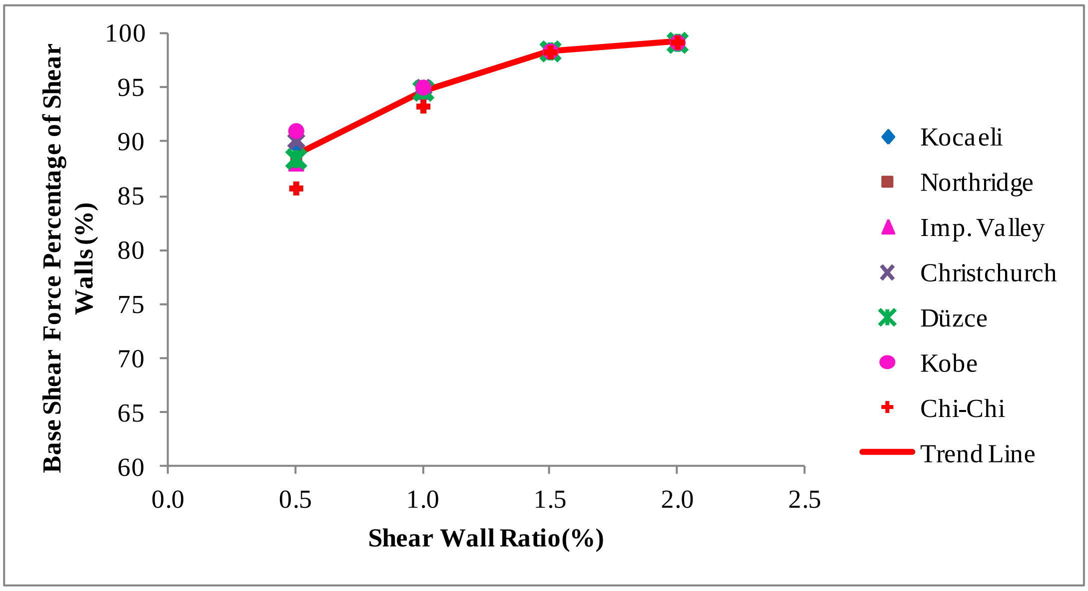


Figure 4.41 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio in the Y-direction of the Second Case of the Designed Buildings

4.2.3.2 Roof Drift

Figures 4.42 and 4.43 show the relationship between the roof drift and shear wall ratio of the designed buildings for the second case in the X and Y-directions, respectively with the corresponding trend lines. As for the first case of the designed buildings, the roof drifts reduce with increasing shear wall ratios for the second case, but the roof drifts in the X-direction are lower than the ones in the Y-direction due to the column orientation. The trend lines of the roof drift variation are nearly linear between the shear wall ratios of 0.50% and 1.50% in both directions and after 1.50% the reduction in the roof drift is insignificant.

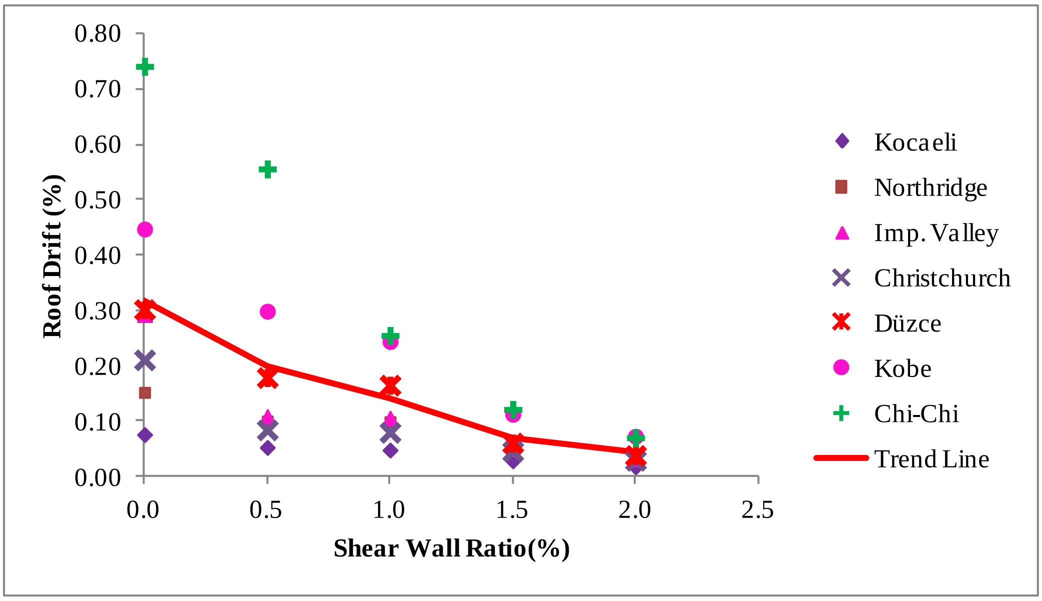


Figure 4.42 Roof Drift vs. Shear Wall Ratio in the X-direction of the Second Case of the Designed Buildings

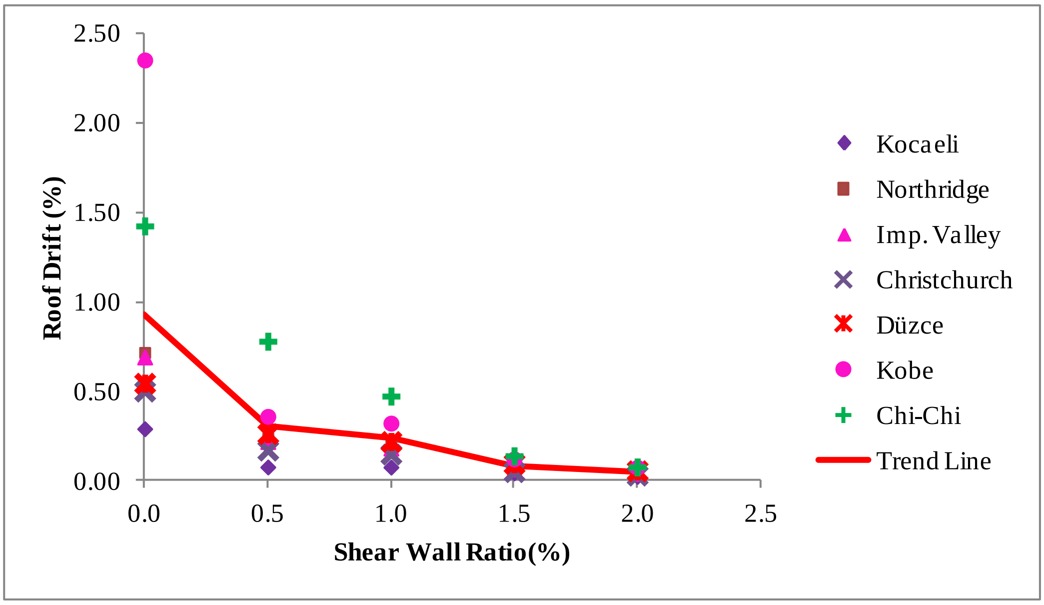


Figure 4.43 Roof Drift vs. Shear Wall Ratio in the Y-direction of the Second Case of the Designed Buildings

Moreover, significant plastic deformations are also observed for this case of buildings with 0.50% shear wall ratios under severe earthquakes such as Kobe and Chi-Chi Earthquakes; however the plastic deformations reduce with increasing shear wall ratios as shown in Figures 4.44, 4.45 and 4.46. These figures indicate that the permanent plastic deformation of the building with 1.00% shear wall ratio is significantly low when compared to the one with 0.50% shear wall ratio under Kobe Earthquake in the Y-direction and the plastic deformation of the building with 1.50% shear wall ratio is negligible. The maximum roof drifts of the second case of designed buildings are also provided in Appendix A.11.

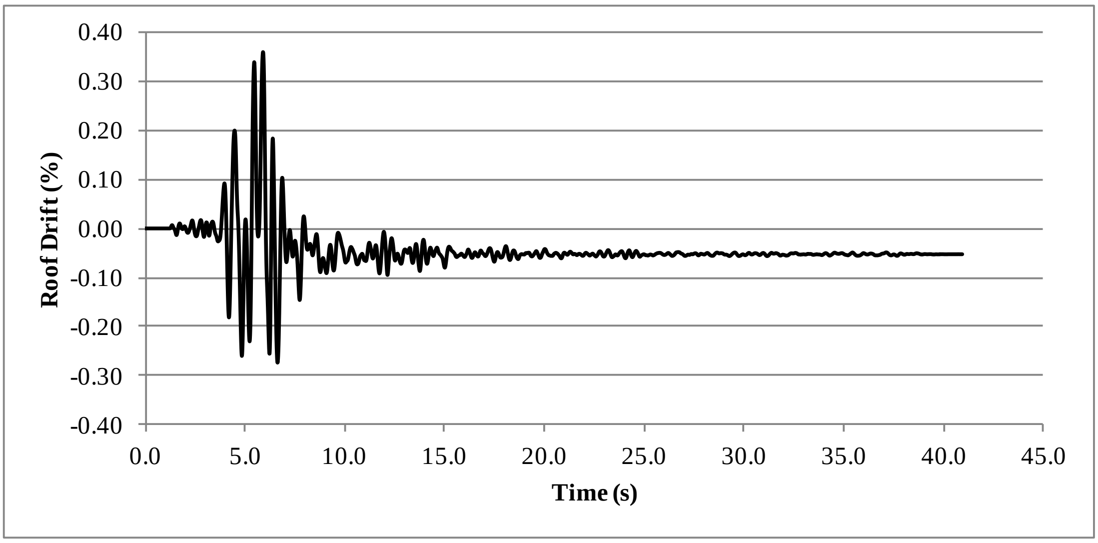


Figure 4.44 Roof Drift vs. Time in the Y-direction of the Second Case of the Designed Building with 0.50% Shear Wall Ratio under Kobe Earthquake

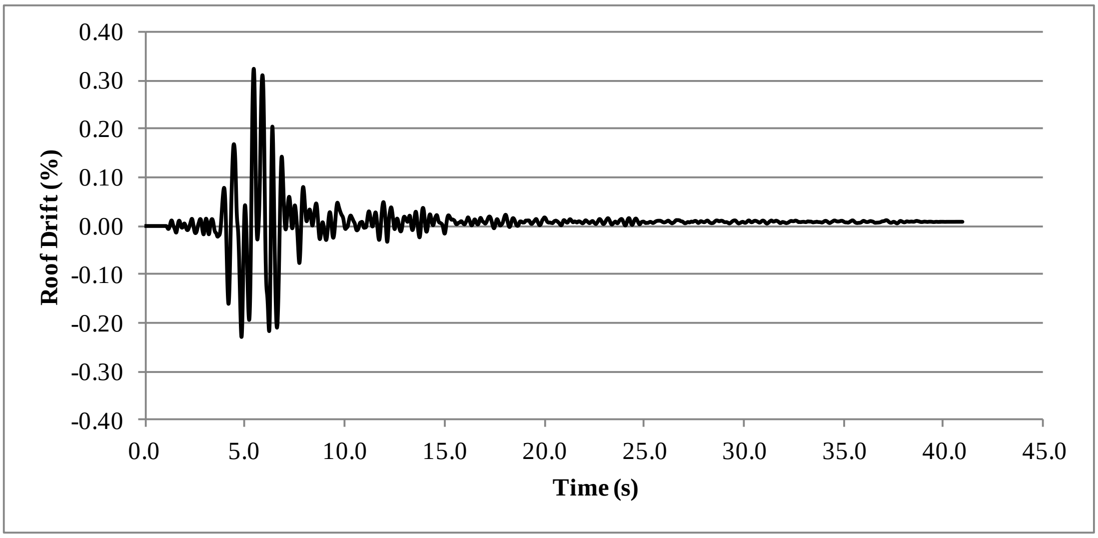


Figure 4.45 Roof Drift vs. Time in the Y-direction of the Second Case of the Designed Building with 1.00% Shear Wall Ratio under Kobe Earthquake

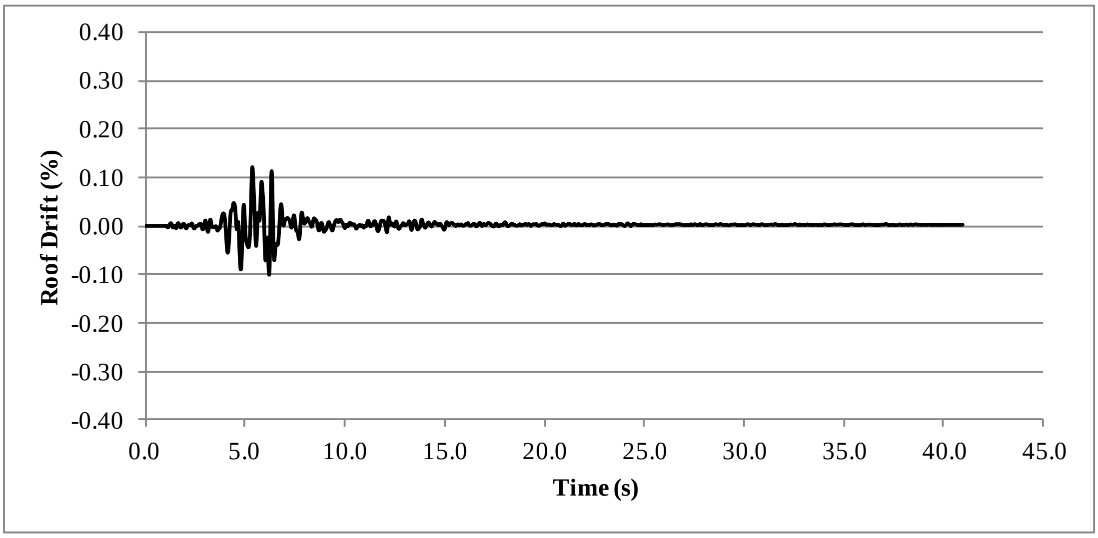


Figure 4.46 Roof Drift vs. Time in the Y-direction of the Second Case of the Designed Building with 1.50% Shear Wall Ratio under Kobe Earthquake

4.2.3.3 Base Shear versus Roof Drift Relationship

Base shear versus roof drift relationship for the second case is shown in Figures 4.47 and 4.48 for the designed buildings with no shear wall and highest shear wall ratio. In these figures, the roof drift decrease and base shear increase with increasing shear wall ratio under Chi-Chi Earthquake in the Y-direction as for the first case. The plastic deformations in the building with no shear wall are quite noticeable, whereas the building with 2.0% shear wall ratio behaves almost elastically.

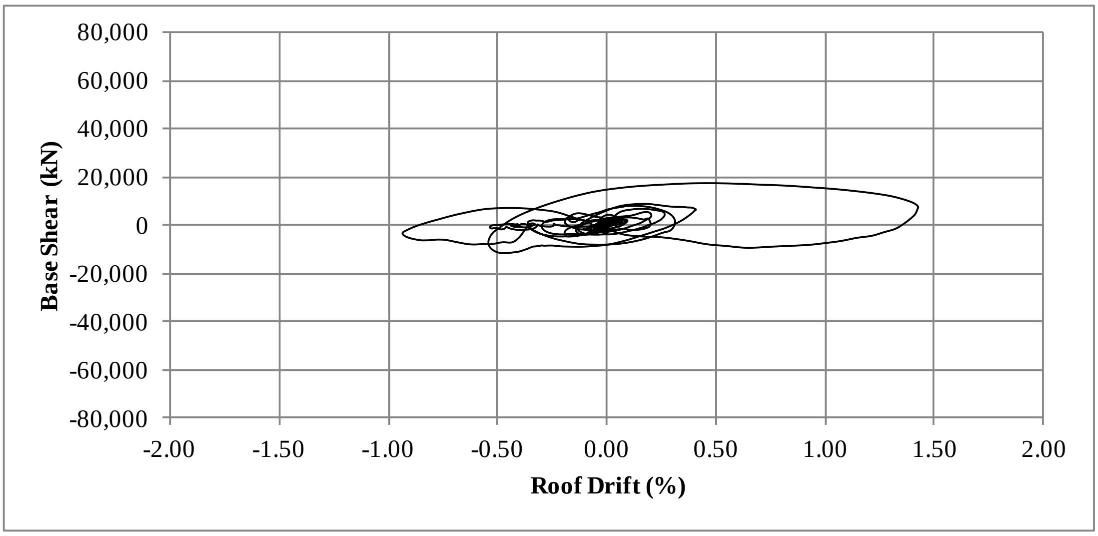


Figure 4.47 Base Shear vs. Roof Drift in the Y-direction of the Second Case of the Designed Building with no Shear Wall under Chi-Chi Earthquake

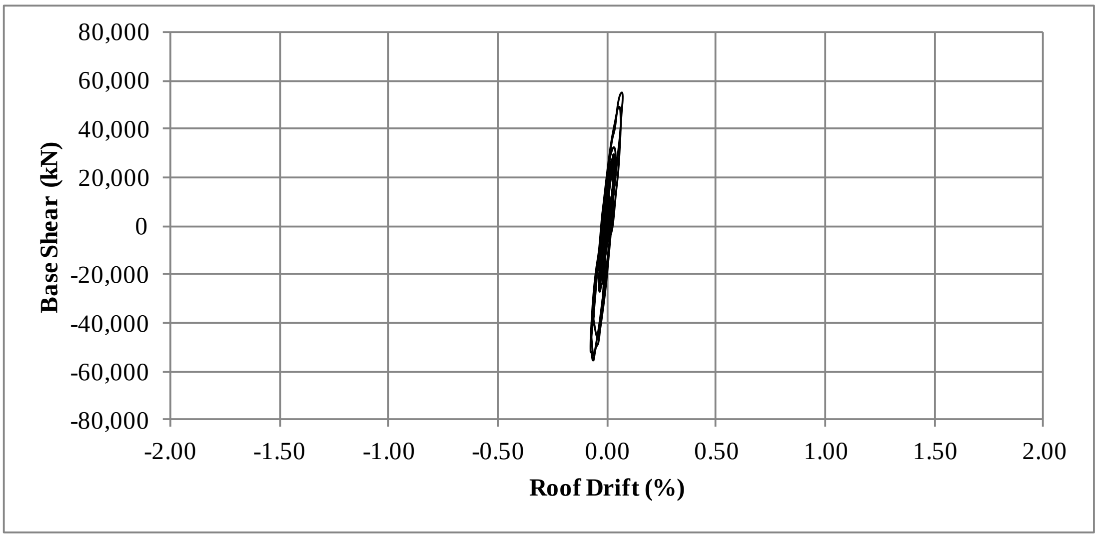


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Figure 4.48 Base Shear vs. Roof Drift in the Y-direction of the Second Case of the Designed Building with 2.00% Shear Wall Ratio under Chi-Chi Earthquake

The maximum base shear vs. maximum roof drift graphs of the building with 1.00% shear wall ratio under selected earthquakes in X and Y-directions are given in Figures 4.49 and 4.50 to observe the structural performance under earthquakes with increasing PGA values. This relationship for the designed buildings with other shear wall ratios is also provided in Appendix A.15, through A.18. As expected, base shear forces increase and roof drifts decrease with increasing earthquake intensity and the relationship is almost linear in both X and Y-directions. The maximum roof drift and maximum base shear values of each designed building under selected earthquakes in both directions are given in Appendix A.11.

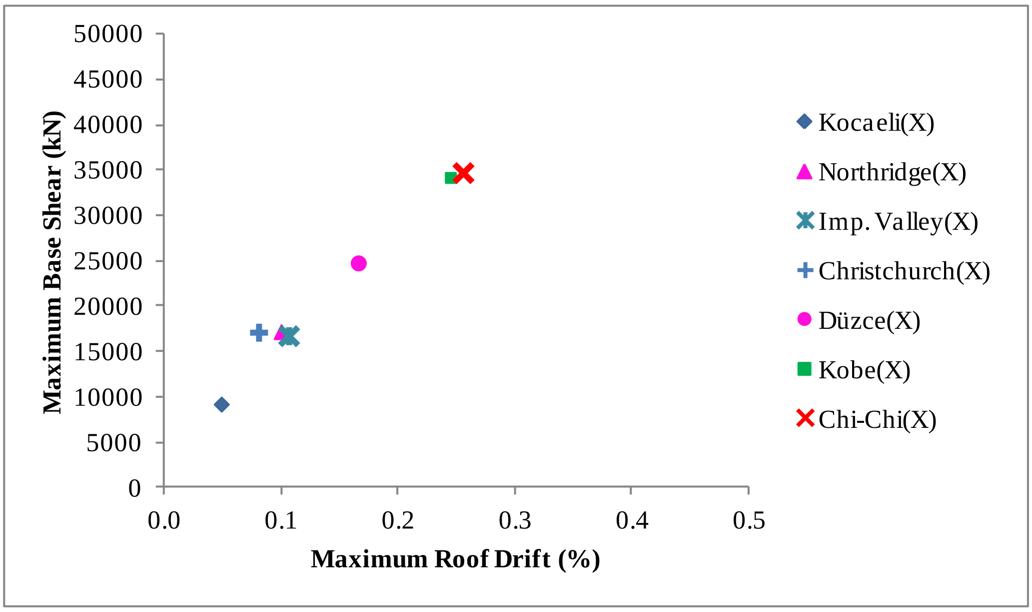

Figure 4.49 Maximum Base Shear vs. Maximum Roof Drift in the X-direction of the Second Case of the Designed Building with 1.00% Shear Wall Ratio under All Earthquake Records

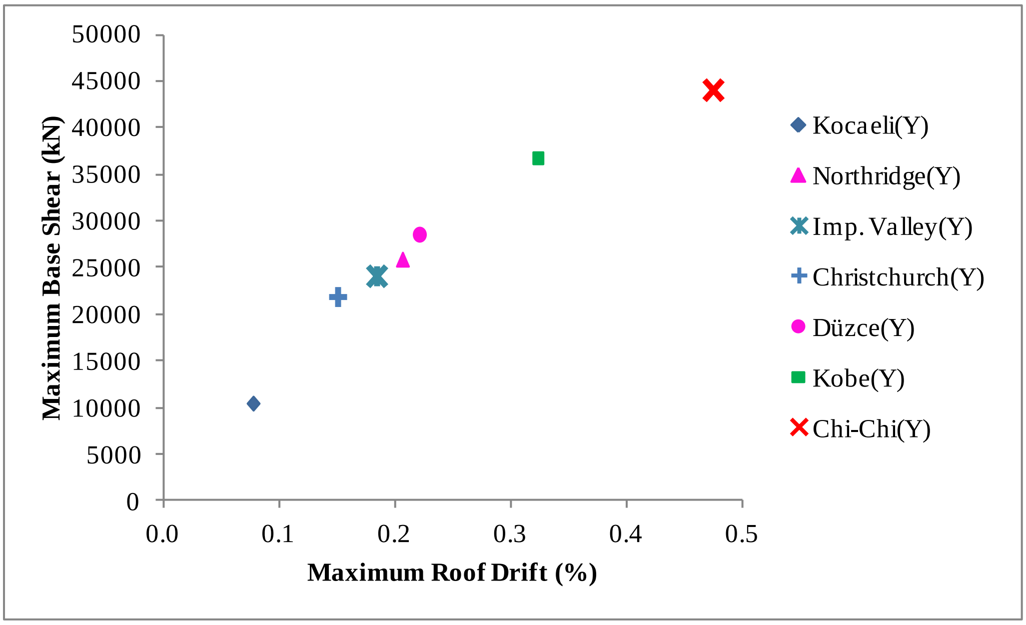

Figure 4.50 Maximum Base Shear vs. Maximum Roof Drift in the Y-direction of the Second Case of the Designed Building with 1.00% Shear Wall Ratio under All Earthquake Records

4.2.3.4 Comparison of the Seismic Performance of the Designed Buildings

4.2.3.4.1 Yielding of Members

Tables 4.5 and 4.6 show the percentage of the yielded members in the designed buildings for the second case in X and Y-directions. As it can be observed from these tables, the variation of the percentage of yielded vertical members of the designed buildings for the second case is very similar to the first case. However, unlike the first case, the percentage of the yielded beams has a more regular distribution. The percentage of the yielded beams decrease with increasing shear wall ratios and earthquake intensity, as expected. Moreover, the percentage of the yielded beams of the upper stories is smaller than that of the ground floors.

Table 4.5 Percentage of Yielded Members for the Second Case of the Designed Buildings in the X-direction

X-Direction			Percentage of Yielded Members (%)			
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**	**Story 4**
Düzce	0.0-0.0	Columns	100.0	75.0	75.0	4.2
	0.5-0.5	Columns	61.9	50.0	37.5	10.0
	1.0-1.0	Columns	40.0	42.9	35.7	7.1
	1.5-1.5	Columns	0.0	0.0	0.0	0.0
	2.0-2.0	Columns	0.0	0.0	0.0	0.0
	0.0-0.0	Beams	97.8	97.8	91.1	0.0
	0.5-0.5	Beams	95.2	97.6	97.6	48.8
	1.0-1.0	Beams	89.5	94.6	91.9	54.1
	1.5-1.5	Beams	52.9	66.7	57.6	33.3
	2.0-2.0	Beams	36.7	51.7	58.6	20.7
	0.0-0.0	Shear Walls	-	-	-	-
	0.5-0.5	Shear Walls	100.0	50.0	0.0	0.0
	1.0-1.0	Shear Walls	25.0	12.5	0.0	0.0
	1.5-1.5	Shear Walls	0.0	0.0	0.0	0.0
	2.0-2.0	Shear Walls	0.0	0.0	0.0	0.0
Kobe	0.0-0.0	Columns	100.0	100.0	100.0	91.6
	0.5-0.5	Columns	100.0	100.0	65.0	60.0
	1.0-1.0	Columns	90.0	67.9	60.7	57.1
	1.5-1.5	Columns	0.0	0.0	0.0	0.0
	2.0-2.0	Columns	0.0	0.0	0.0	0.0
	0.0-0.0	Beams	100.0	100.0	100.0	100.0
	0.5-0.5	Beams	95.2	97.6	97.6	95.1
	1.0-1.0	Beams	94.7	97.3	97.3	91.9
	1.5-1.5	Beams	88.2	93.9	87.9	63.6
	2.0-2.0	Beams	80.0	89.7	89.7	72.4
	0.0-0.0	Shear Walls	-	-	-	-
	0.5-0.5	Shear Walls	100.0	100.0	0.0	0.0
	1.0-1.0	Shear Walls	100.0	37.5	0.0	0.0
	1.5-1.5	Shear Walls	33.3	0.0	0.0	0.0
	2.0-2.0	Shear Walls	25.0	0.0	0.0	0.0
Chi-Chi	0.0-0.0	Columns	100.0	100.0	100.0	100.0
	0.5-0.5	Columns	100.0	100.0	100.0	100.0
	1.0-1.0	Columns	83.3	78.6	71.4	78.6
	1.5-1.5	Columns	0.0	0.0	0.0	0.0
	2.0-2.0	Columns	0.0	0.0	0.0	0.0
	0.0-0.0	Beams	100.0	97.8	97.8	31.1
	0.5-0.5	Beams	97.6	100.0	100.0	100.0
	1.0-1.0	Beams	94.7	97.3	97.3	83.8
	1.5-1.5	Beams	88.2	93.9	93.9	63.6
	2.0-2.0	Beams	73.3	86.2	79.3	62.1
	0.0-0.0	Shear Walls	-	-	-	-
	0.5-0.5	Shear Walls	100.0	100.0	75.0	0.0
	1.0-1.0	Shear Walls	50.0	25.0	12.5	0.0
	1.5-1.5	Shear Walls	33.3	0.0	0.0	0.0
	2.0-2.0	Shear Walls	12.5	0.0	0.0	0.0

Table 4.6 Percentage of Yielded Members for the Second Case of the Designed Buildings in the Y-Direction

Y-Direction			Percentage of Yielded Members (%)			
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**	**Story 4**
Düzce	**0.0-0.0**	**Columns**	100.0	75.0	75.0	4.2
	0.5-0.5	**Columns**	61.9	50.0	37.5	10.0
	1.0-1.0	**Columns**	40.0	42.9	35.7	7.1
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	100.0	5.3
	0.5-0.5	**Beams**	95.0	100.0	100.0	26.3
	1.0-1.0	**Beams**	42.1	84.2	60.5	42.1
	1.5-1.5	**Beams**	27.8	23.5	23.5	20.6
	2.0-2.0	**Beams**	12.5	13.3	13.3	13.3
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	100.0	50.0	0.0	0.0
	1.0-1.0	**Shear Walls**	75.0	12.5	0.0	0.0
	1.5-1.5	**Shear Walls**	0.0	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0
Kobe	**0.0-0.0**	**Columns**	100.0	100.0	100.0	91.6
	0.5-0.5	**Columns**	100.0	100.0	65.0	60.0
	1.0-1.0	**Columns**	90.0	67.9	60.7	57.1
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	100.0	100.0
	0.5-0.5	**Beams**	100.0	100.0	100.0	50.0
	1.0-1.0	**Beams**	100.0	100.0	100.0	52.6
	1.5-1.5	**Beams**	27.8	29.4	23.5	23.5
	2.0-2.0	**Beams**	15.6	20.0	20.0	20.0
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	100.0	100.0	0.0	0.0
	1.0-1.0	**Shear Walls**	100.0	50.0	0.0	0.0
	1.5-1.5	**Shear Walls**	50.0	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0
Chi-Chi	**0.0-0.0**	**Columns**	100.0	100.0	100.0	100.0
	0.5-0.5	**Columns**	100.0	100.0	100.0	100.0
	1.0-1.0	**Columns**	83.3	78.6	71.4	78.6
	1.5-1.5	**Columns**	0.0	0.0	0.0	0.0
	2.0-2.0	**Columns**	0.0	0.0	0.0	0.0
	0.0-0.0	**Beams**	100.0	100.0	100.0	84.2
	0.5-0.5	**Beams**	100.0	100.0	100.0	71.1
	1.0-1.0	**Beams**	100.0	100.0	97.4	55.6
	1.5-1.5	**Beams**	27.8	29.4	23.5	26.5
	2.0-2.0	**Beams**	15.6	20.0	20.0	20.0
	0.0-0.0	**Shear Walls**	-	-	-	-
	0.5-0.5	**Shear Walls**	100.0	100.0	0.0	0.0
	1.0-1.0	**Shear Walls**	100.0	25.0	0.0	0.0
	1.5-1.5	**Shear Walls**	25.0	0.0	0.0	0.0
	2.0-2.0	**Shear Walls**	0.0	0.0	0.0	0.0

4.2.3.4.2 Effect of Shear Wall Percentage on Earthquake Load Reduction Factor (R)

Like the first case of designed buildings, influence of varying shear wall percentage on the earthquake load reduction factor of the second case of designed buildings is investigated and the results are provided in Table 4.7. All the base shear percentages carried by high ductility shear walls of the first case of designed buildings including shear walls in both directions are obtained to be higher than 75%. However, for the second case of designed buildings, the designed building with 0.50% shear wall ratio in X-direction is lower than 75% base shear percentage and thus, earthquake load reduction factor of this building is 7, which shows combined shear wall-frame behavior. The earthquake load reduction factors for buildings with higher than 0.50% shear wall ratios are also around 6 similar to the first case of designed buildings.

Table 4.7 Effect of Varying Shear Wall Percentage on the Earthquake Load Reduction Factor of the Second Case of Designed Building based on the requirements of Turkish Earthquake Code (2007)

	Shear Wall Ratio (%)	Direction	Total Base Shear (kN)	Base Shear Carried by Shear Walls (kN)	Base Shear Percentage Carried by Shear Walls (α_s)	R
Designed Building (Second Case)	0.0-0.0	X	2360	0	0.00	8.00
	0.0-0.0	Y	2360	0	0.00	8.00
	0.5-0.5	X	4012	2950	0.74	7.00
	0.5-0.5	Y	4012	3546	0.88	6.46
	1.0-1.0	X	4601	4027	0.88	6.50
	1.0-1.0	Y	4601	4351	0.95	6.22
	1.5-1.5	X	5238	4950	0.95	6.22
	1.5-1.5	Y	5238	5148	0.98	6.07
	2.0-2.0	X	5873	5695	0.97	6.12
	2.0-2.0	Y	5873	5823	0.99	6.03

4.2.3.4.3 Drift Limitations

Maximum displacement and drift values are obtained from the building models based on Turkish Earthquake Code (2007) by using earthquake load reduction factors. Table 4.8 indicates that like the first case of designed buildings, the second case of designed buildings satisfy the requirements of Turkish Earthquake Code (2007) on the drift limitations

Table 4.8 Drift Values of the Second Case of Designed Buildings Turkish Earthquake Code (2007)

	Shear Wall Ratio (%)	Direction	R	Maximum Story Displacement at ith story level (d_{imax}) (mm)	Maximum Interstory Drift at ith story level (δ_{imax}) (%)
Designed Building (Second Case)	0.0-0.0	X	8.00	1.10	0.28
	0.0-0.0	Y	8.00	3.40	0.88
	0.5-0.5	X	7.00	0.90	0.20
	0.5-0.5	Y	6.46	1.20	0.25
	1.0-1.0	X	6.50	0.80	0.17
	1.0-1.0	Y	6.22	1.00	0.20
	1.5-1.5	X	6.22	0.40	0.08
	1.5-1.5	Y	6.07	0.40	0.08
	2.0-2.0	X	6.12	0.30	0.06
	2.0-2.0	Y	6.03	0.30	0.06

4.3 Analytical Results of the Existing School Buildings

In addition to the designed buildings, five different existing school reinforced concrete buildings are analyzed by using SAP2000 v14.2.0 (2009). As mentioned in Chapter 3, the existing school buildings are selected to have different structural properties such as story heights, floor areas, number of stories and layouts to investigate the influence of varying shear wall ratios on the seismic performance of different types of existing buildings. Moreover, some of the selected existing buildings were strengthened after suffering from earthquake loading and these strengthened buildings are also examined. Some additional buildings are also generated with shear wall ratios in between before and after retrofit cases to observe the effect of increasing shear wall ratios.

First the Sarıyer MEV Dumlupınar Primary School is analyzed, which is selected as the basis for the designed buildings. This building has four stories, 0.50-1.00% shear wall ratio, i.e., 0.50% in the X-direction and 1.00% in the Y-direction. The average concrete compressive strength and reinforcement yield strength for this building are 13.9 MPa and 220 MPa, respectively. This building was strengthened by the addition of reinforced concrete columns and shear walls and shear wall ratio of this building was increased to 1.0-1.5% for the strengthened case. The members added to all existing buildings have 25 MPa concrete compressive strength and 420 MPa reinforcement yield strength. The original and strengthened buildings have torsional irregularity in the Y-direction based on the Turkish Earthquake Code (2007), the torsional irregularity coefficients of which are provided in Table 4.9.

Table 4.9 Torsional Irregularity Coefficients for the Existing and Designed Buildings

Building	Shear Wall Ratio (%)	Torsional Irregularity Coefficient									
		X-Direction					Y-Direction				
		Story 1	Story 2	Story 3	Story 4	Story 5	Story 1	Story 2	Story 3	Story 4	Story 5
Sarıyer MEV Dumlupınar Primary School (Designed Building) (First Case)	0.0-0.0	1.02	1.02	1.03	1.04	-	1.22	1.19	1.24	1.29	-
	0.5-0.5	1.00	1.00	1.04	1.05	-	1.09	1.11	1.06	1.14	-
	1.0-1.0	1.00	1.00	1.05	1.06	-	1.11	1.07	1.13	1.14	-
	1.5-1.5	1.00	1.00	1.00	1.00	-	1.10	1.00	1.14	1.14	-
	2.0-2.0	1.00	1.00	1.00	1.00	-	1.00	1.00	1.10	1.10	-
Sarıyer MEV Dumlupınar Primary School (Designed Building) (Second Case)	0.0-0.0	1.10	1.10	1.06	1.00	-	1.15	1.14	1.15	1.17	-
	0.5-0.5	1.00	1.00	1.06	1.08	-	1.07	1.09	1.04	1.14	-
	1.0-1.0	1.11	1.00	1.00	1.00	-	1.09	1.11	1.05	1.11	-
	1.5-1.5	1.00	1.00	1.00	1.14	-	1.10	1.00	1.00	1.14	-
	2.0-2.0	1.00	1.00	1.00	1.10	-	1.00	1.00	1.00	1.10	-
Sarıyer MEV Dumlupınar Primary School	0.5-1.0 (Existing B.)	1.03	1.00	1.02	1.04	-	1.25	1.17	1.19	1.17	-
	1.0-1.5 (Retrofitted B.)	1.00	1.05	1.00	1.05	-	1.33	1.33	1.27	1.33	-
Fatih Gazi Primary School	0.5-0.0 (Existing B.)	1.32	1.16	1.06	-	-	1.21	1.18	1.16	-	-
	0.5-0.5 (Generated B.)	1.11	1.09	1.08	-	-	1.29	1.16	1.05	-	-
	1.0-1.0 (Generated B.)	1.07	1.05	1.03	-	-	1.06	1.12	1.17	-	-
	1.5-1.5 (Retrofitted B.)	1.00	1.00	1.03	-	-	1.29	1.33	1.44	-	-
Eminönü Çemberlitaş Anatolian High School Block A	0.0-0.0 (Existing B.)	1.08	1.11	1.12	1.09	1.09	1.13	1.13	1.17	1.00	1.09
	0.5-0.5 (Generated B.)	1.17	1.22	1.22	1.22	1.20	1.87	1.86	1.80	1.72	1.59
	1.0-1.0 (Generated B.)	1.51	1.64	1.68	1.68	1.68	1.76	1.75	1.74	1.72	1.68
	1.0-1.5 (Retrofitted B.)	1.29	1.42	1.45	1.48	1.51	1.26	1.25	1.26	1.27	1.28
G.O.P. Ülkü Primary School	1.5-1.0 (Existing B.)	1.17	1.11	1.10	1.05	-	1.08	1.11	1.14	1.13	-
Güngören Haznedar Abdi İpekçi Primary School Block B	1.5-1.0 (Existing B.)	1.17	1.05	1.14	1.10	-	1.08	1.08	1.14	1.12	-
	1.5-1.5 (Generated B.)	1.08	1.05	1.09	1.05	-	1.44	1.45	1.44	1.44	-
	2.5-2.0 (Retrofitted B.)	1.00	1.00	1.08	1.00	-	1.47	1.60	1.53	1.60	-

Fatih Gazi Primary School is modeled next, which is a three story building having concrete compressive strength and reinforcement yield strength of 18.2 MPa and 220 MPa, respectively. The shear wall ratio of this existing building is 0.50-0.00% and this building was strengthened by adding only reinforced concrete shear walls. Therefore, shear wall ratio was increased to 1.50-1.50% for the strengthened case. Two more building models with the same structural properties but different shear wall ratios are generated to study the seismic behavior of the existing building with varying shear wall ratios. The shear wall ratios of these generated buildings are selected as 0.50-0.50% and 1.00-1.00%. Fatih Gazi Primary School has torsional irregularity for all buildings except for the building with 1.00-1.00% shear wall ratio, the torsional irregularity coefficients of which are provided in Table 4.5.

Then, the five-story existing reinforced concrete building, Eminönü Çemberlitaş Anatolian High School Block A is inspected. For this building, the concrete compressive strength is 12.9 MPa and the reinforcement yield strength is 220 MPa and no shear walls exist in the original floor plan. Like the other existing buildings, this building was strengthened by shear walls and the shear wall ratio became 1.00-1.50%. Two more buildings with 0.50-0.50% and 1.00-1.00% shear wall ratios are generated and torsional irregularity checks are performed for each building. Except the original building, torsional irregularity exists for each reinforced concrete building as shown in Table 4.5.

Finally, two four-story existing buildings with similar structural plans are examined, Güngören Haznedar Abdi İpekçi Primary School Block B and G.O.P. Ülkü Primary School Block B. The reinforcement yield strength for both buildings is 220 MPa. The concrete compressive strength of G.O.P. Ülkü Primary School Block B is 27.5 MPa, but for Güngören Haznedar Abdi İpekçi Primary School Block B, this value is 7.2 MPa. These two buildings are selected to investigate if the concrete strength influences the improvement in the seismic performance provided by increasing the shear wall ratio. The shear wall ratio of both existing buildings is 1.50-1.00%. G.O.P. Ülkü Primary School

Block B was not strengthened and there is no torsional irregularity in this building. On the other hand, Güngören Haznedar Abdi İpekçi Primary School Block B was strengthened and shear wall ratio became 2.50-2.00%. One more reinforced concrete building is generated, the shear wall ratio of which is selected as 1.50-1.50%. Güngören Haznedar Abdi İpekçi Primary School Block B has torsional irregularity for all cases, except for the original building like the Eminönü Çemberlitaş Anatolian High School Block A.

4.3.1 Base Shear Carried by Reinforced Concrete Shear Walls

Figures 4.51 and 4.52 show the base shear percentage carried by shear walls vs. shear wall ratio graphs for all cases of the Fatih Gazi Primary School under the selected earthquakes in X and Y-directions. Like the designed buildings, adding reinforced concrete shear walls to the existing buildings increases base shear percentage carried by shear walls and after reaching 1.00% shear wall ratio in both directions, the increase in base shear percentage of shear walls is insignificant. Base shear percentage of reinforced concrete columns of the existing building with 0.50% shear wall ratio in both directions is in the range of 8.7-27.9%. However, to expect such high contributions from columns to carry the applied load is undesirable for buildings with shear walls under severe earthquakes. The concentration at 0.50% shear wall ratio in Figure 4.51 is due to the existence of two different buildings with 0.50% shear wall ratio in the X-direction. In Appendix A.19, through A.21, the graphs of base shear percentage carried by shear walls vs. shear wall ratio of other existing school buildings in X and Y-directions are provided. Based on these figures, it can be concluded that the contribution of the shear walls in carrying base shear has a similar trend for all buildings with different floor plans, number of stories, structural members and material properties.

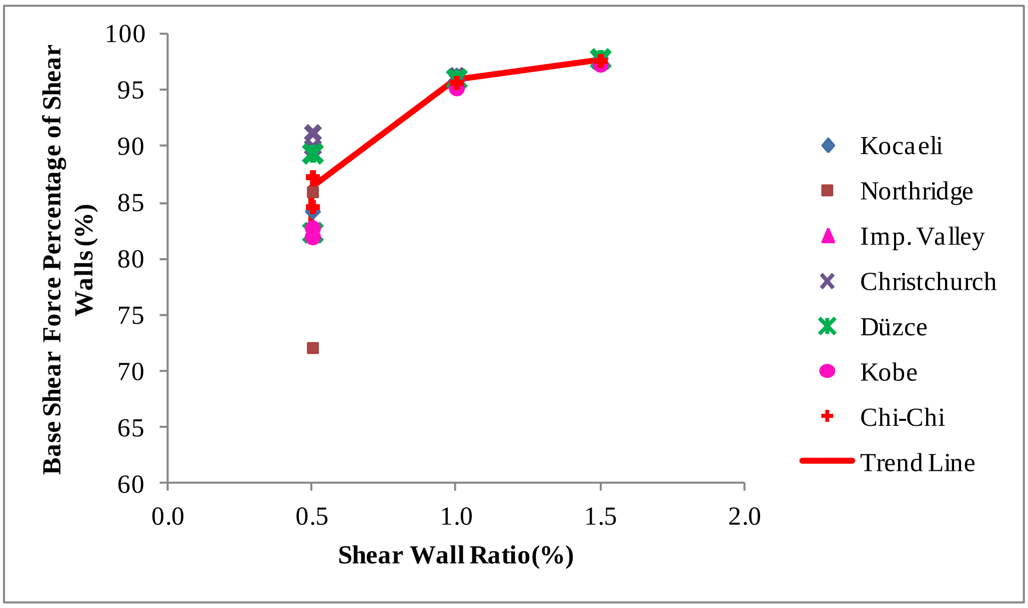

Figure 4.51 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio in the X-direction of Fatih Gazi Primary School

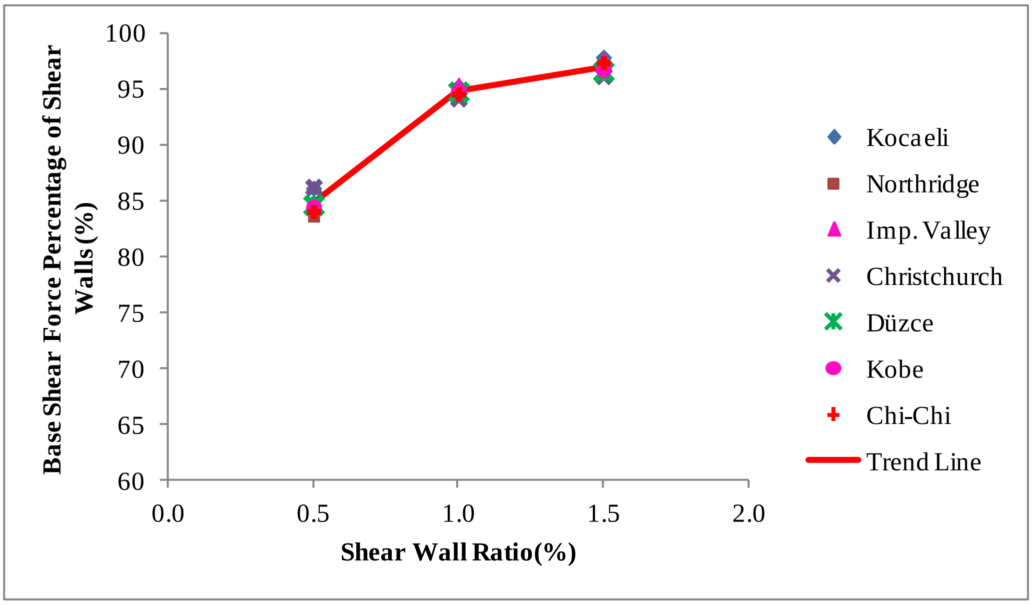

Figure 4.52 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio in the Y-direction of Fatih Gazi Primary School

4.3.2 Roof Drift

Roof drift versus shear wall ratio relationships for all cases of the Fatih Gazi Primary School under selected earthquakes in X and Y-directions are given in Figures 4.53 and 4.54. The roof drifts of the buildings decrease with increasing shear wall ratios in both directions and this effect is degrading for buildings with higher shear wall ratios. Especially, the difference between the roof drifts of the buildings with 1.00% and 1.50% shear wall ratios is significantly low when compared to the difference between the roof drifts of the buildings with 0.50% and 1.00% shear wall ratios. As an example, the roof drift of the existing building with 0.50-0.50% shear wall ratio under Kobe Earthquake in X-direction is 0.74%, which is three times higher than the roof drift of the building with 1.00-1.00% shear wall ratio, 0.24%. This proves the efficiency of using 1.00% shear wall ratio when compared to 0.50% shear wall ratio while strengthening the existing building, Fatih Gazi Primary School. The roof drift versus shear wall ratio relationships for other existing buildings are given in Appendix A.22 and A.23.

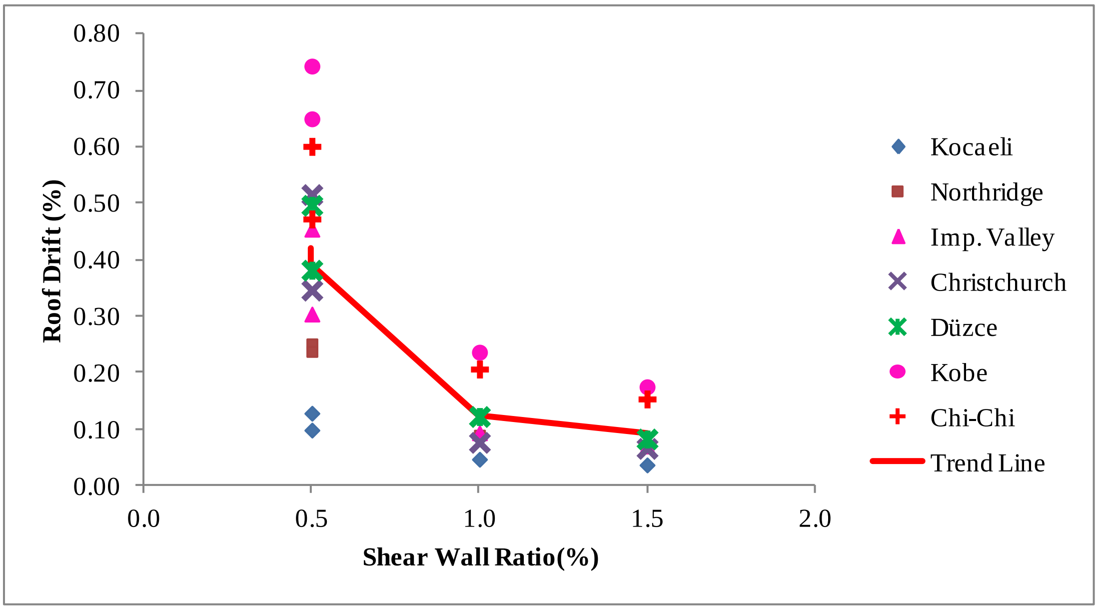

Figure 4.53 Roof Drift vs. Shear Wall Ratio in the X-direction of Fatih Gazi Primary School

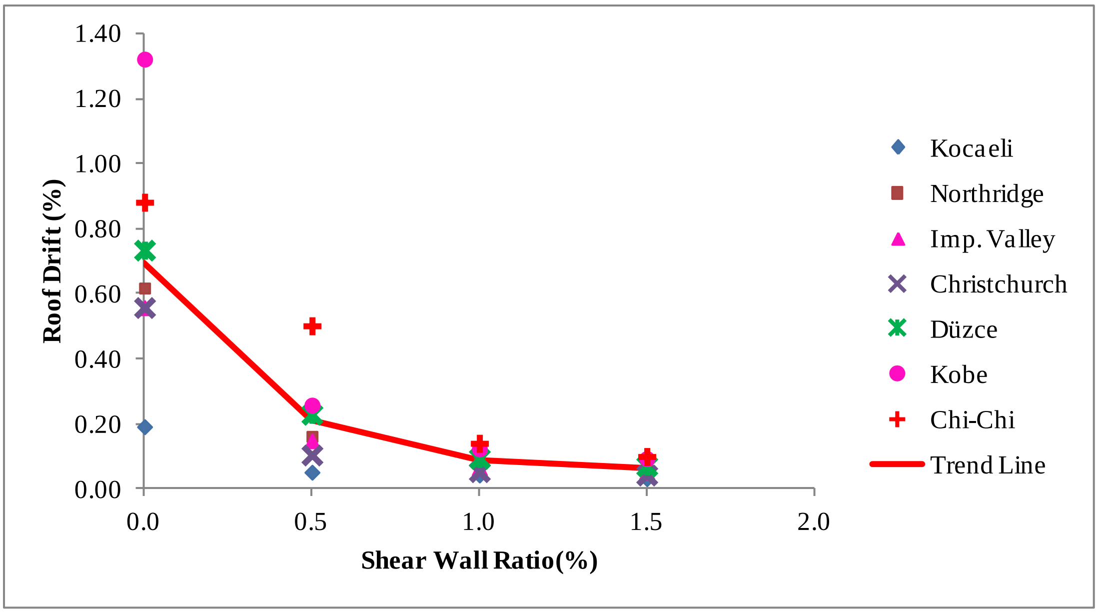

Figure 4.54 Roof Drift vs. Shear Wall Ratio in the Y-direction of Fatih Gazi Primary School

As expected, the roof drifts of Fatih Gazi Primary School decrease with increasing shear wall ratios and they increase with increasing PGA values of earthquakes in both directions generally, except for one case. In X-direction, the roof drifts of the building with 0.50-0.00% shear wall ratio under severe earthquake loadings are smaller than the roof drifts of the buildings with 0.50-0.50% shear wall ratio as can be observed from Figures 4.55 and 4.56 for the Chi-Chi Earthquake. Shear wall ratios of these buildings are the same in the X-direction, but addition of shear walls in the Y-direction increased the total weight and stiffness of the building only in the Y-direction. Therefore, the earthquake load in the X-direction increases, which results in higher roof drifts in this direction. Moreover, Figures 4.57 and 4.58 show the roof drift versus time relationship of the buildings with 1.00-1.00% and 1.50-1.50% shear wall ratios in the X-direction under Chi-Chi Earthquake and from these figures, it can be observed that, plastic deformation of these buildings with 1.00-1.00% and 1.50-1.50% shear wall ratios is negligible.

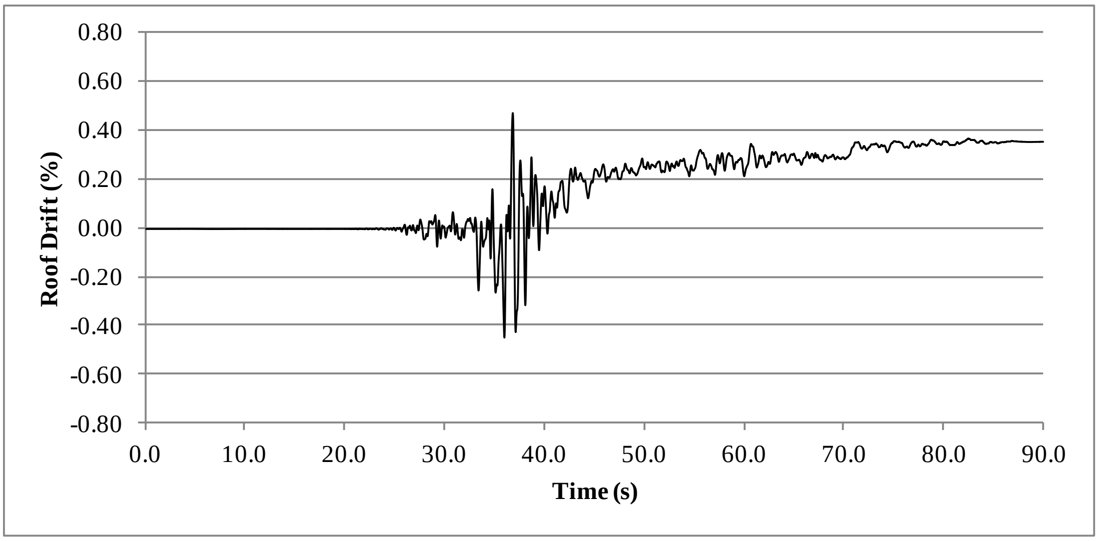

Figure 4.55 Roof Drift vs. Time in the X-direction of Fatih Gazi Primary School with 0.50-0.00% Shear Wall Ratio under Chi-Chi Earthquake

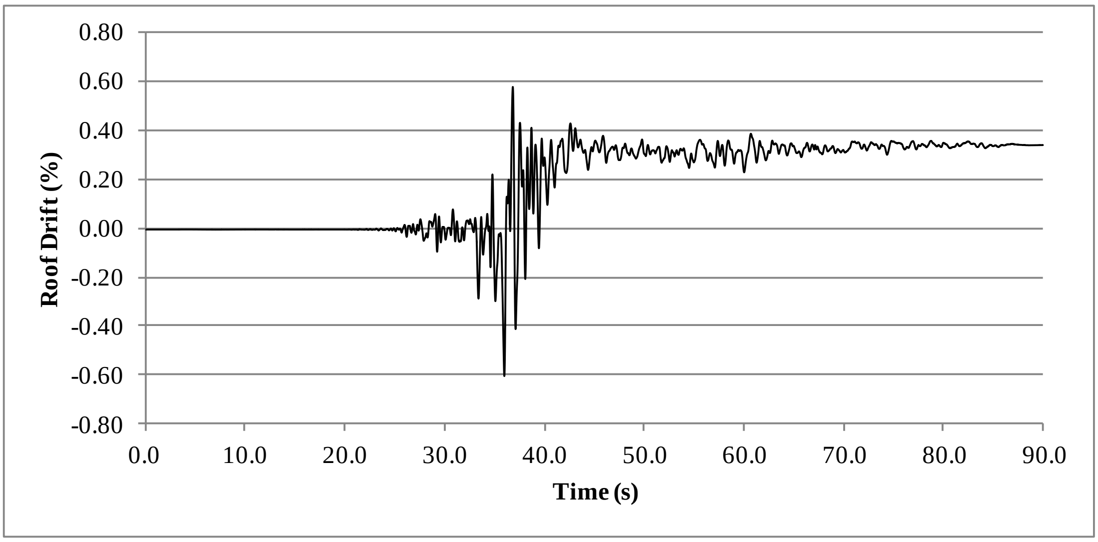

Figure 4.56 Roof Drift vs. Time in the X-direction of Fatih Gazi Primary School with 0.50-0.50% Shear Wall Ratio under Chi-Chi Earthquake

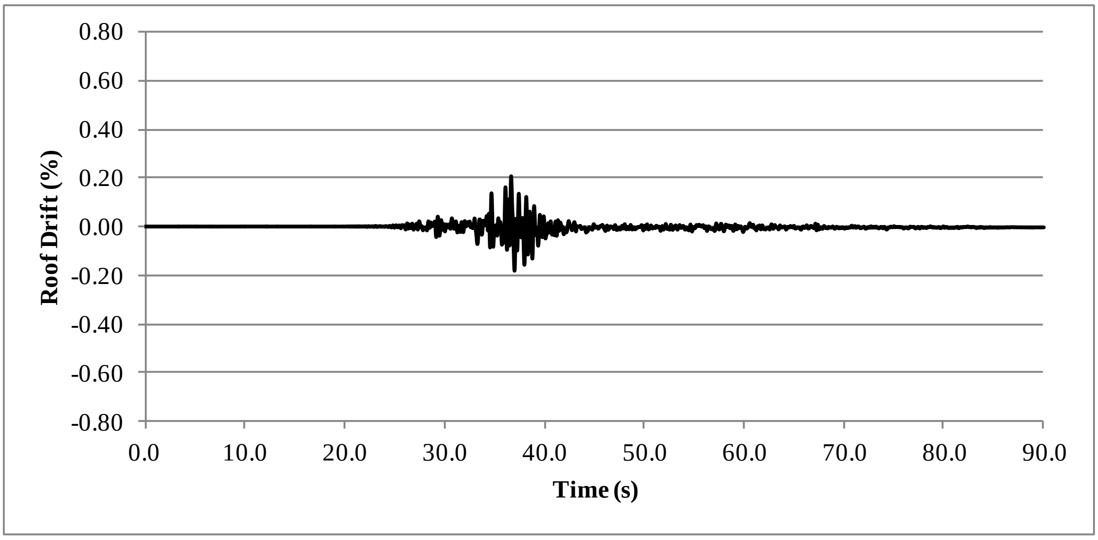

Figure 4.57 Roof Drift vs. Time in the X-direction of Fatih Gazi Primary School with 1.00-1.00% Shear Wall Ratio under Chi-Chi Earthquake

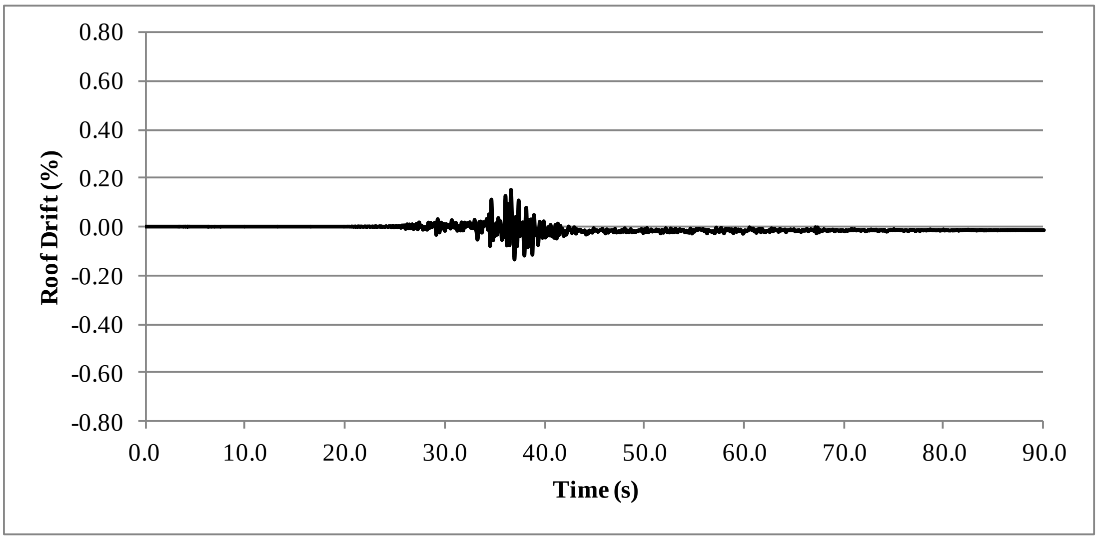

Figure 4.58 Roof Drift vs. Time in the X-direction of Fatih Gazi Primary School with 1.50-1.50% Shear Wall Ratio under Chi-Chi Earthquake

As for Fatih Gazi Primary School, the roof drifts of Eminönü Çemberlitaş Anatolian High School Block A with 1.00-1.00% shear wall ratio in the X-direction under selected earthquakes are lower than the roof drifts of this building with 1.00-1.50% shear wall ratio and the roof drifts of Güngören Haznedar Abdi İpekçi Primary School Block B with 1.50-1.00% shear wall ratio in the X-direction under selected earthquakes are lower than the roof drifts of this building with 1.50-1.50% shear wall ratio. The maximum roof drift values of all buildings under selected earthquakes are provided in Appendix A.11. The roof drift vs. shear wall ratio graphs of Eminönü Çemberlitaş Anatolian High School Block A under selected earthquakes in both directions are given in Figures 4.59 and 4.60. Unlike Fatih Gazi Primary School, Eminönü Çemberlitaş Anatolian High School Block A with 0.50-0.50% and 1.0-1.0% shear wall ratios have minor difference in the roof drifts, but under high intensity earthquakes, especially for the Kobe and Chi-Chi Earthquakes, the difference cannot be neglected.

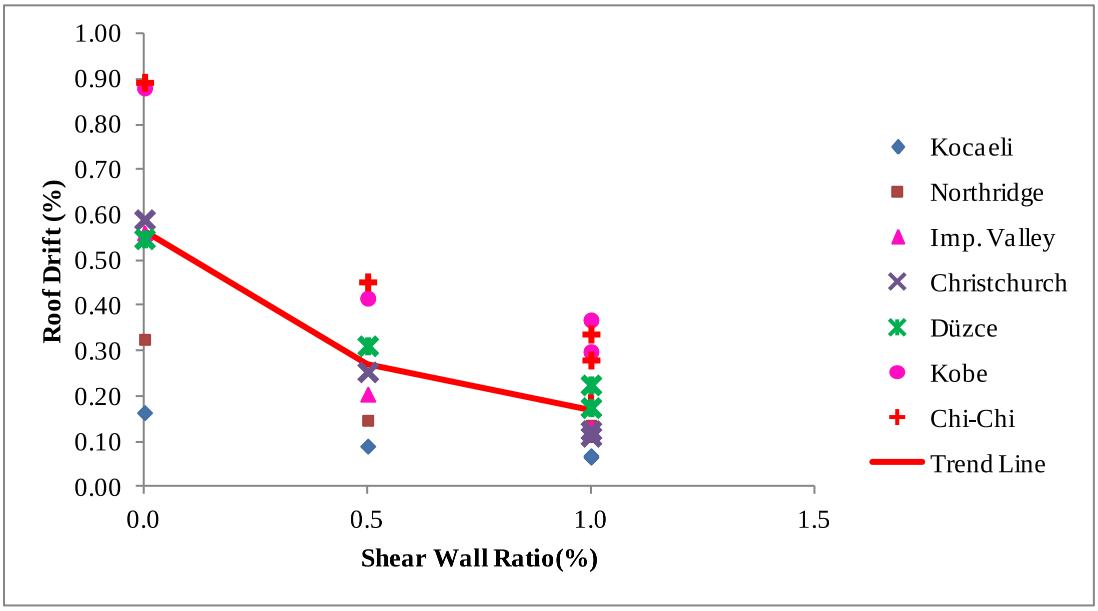

Figure 4.59 Roof Drift vs. Shear Wall Ratio in the X-direction of Eminönü Çemberlitaş Anatolian High School Block A

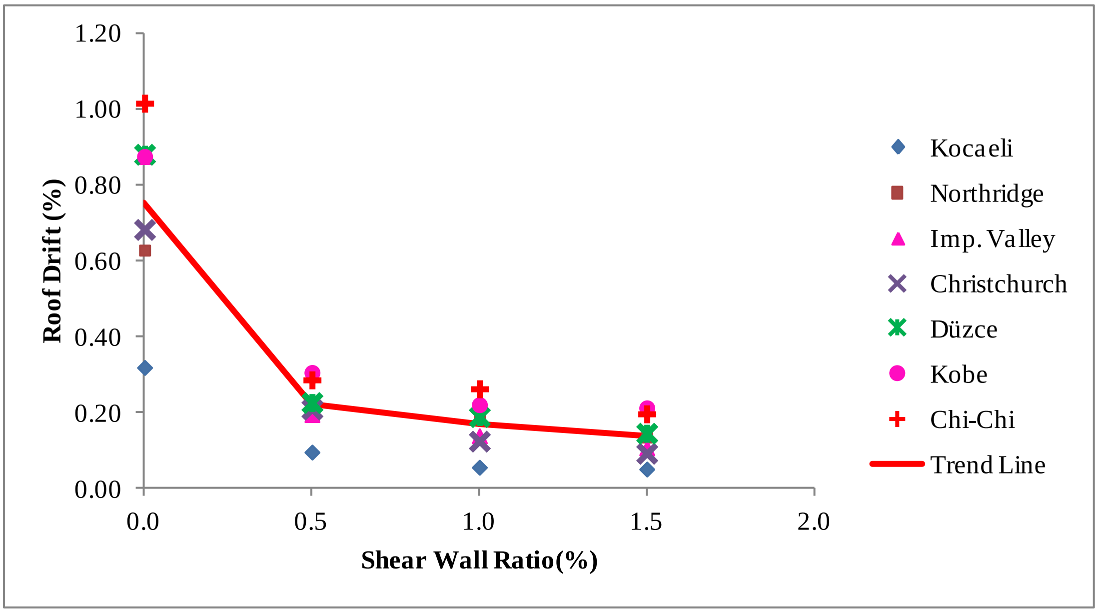

Figure 4.60 Roof Drift vs. Shear Wall Ratio in the Y-direction of Eminönü Çemberlitaş Anatolian High School Block A

4.3.3 Base Shear versus Roof Drift Relationship

Base shear versus roof drift relationships of existing school buildings have similar trends as the designed buildings. Roof drift values of the existing reinforced concrete buildings decrease and base shear values increase with increasing shear wall ratios. The maximum base shear vs. maximum roof drift graphs of Eminönü Çemberlitaş Anatolian High School Block A with 1.00-1.00% shear wall ratio under selected earthquakes in X- and Y-directions are given in Figures 4.61 and 4.62, respectively. Furthermore, this relationship for all existing buildings is provided in Appendix A.24 through A.36. Maximum base shear and maximum roof drift relationships of existing buildings also have similar trends as the designed buildings. Maximum roof drift and maximum base shear force values increase almost linearly with increasing earthquake intensity.

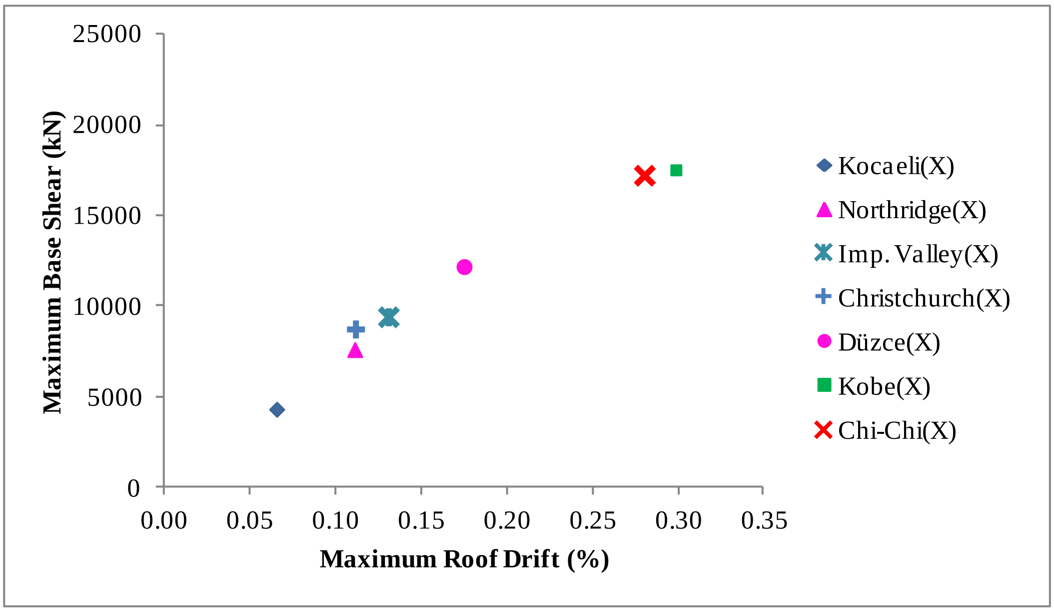

Figure 4.60 Maximum Base Shear vs. Maximum Roof Drift in the X-direction of Eminönü Çemberlitaş Anatolian High School Block A with 1.00% Shear Wall Ratio under All Earthquake Records

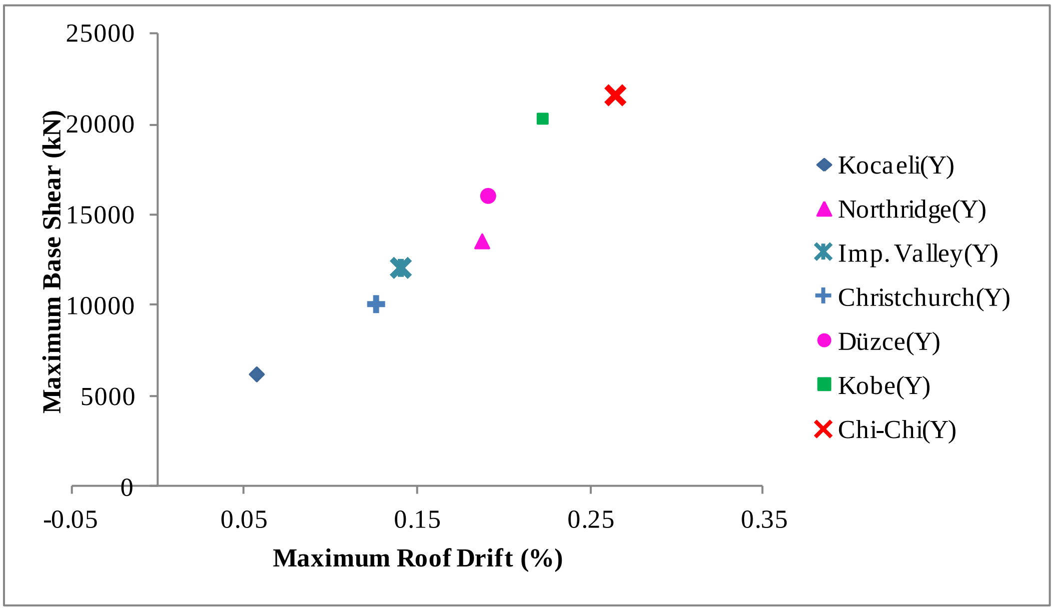

Figure 4.61 Maximum Base Shear vs. Maximum Roof Drift in the X-direction of Eminönü Çemberlitaş Anatolian High School Block A with 1.00% Shear Wall Ratio under All Earthquake Records

4.3.4 Comparison of the Seismic Performance of the Existing Buildings

4.3.4.1.1 Yielding of Members

Percentage of yielded members for the existing building, Sarıyer MEV Dumlupınar Primary School in X and Y-directions is given in Tables 4.10 and 4.11, respectively. The percentage of the yielded members for this building also decreases with increasing shear wall ratio and the decreasing PGA values of the selected earthquakes. Moreover, the percentage of the yielded members in the upper floors is smaller than the that of the ground floors in both directions. Based on these tables, it can be stated that, strengthening the existing building has not reduced the percentage of yielded members significantly under severe earthquakes of Kobe and Chi-Chi, but the retrofit is more effective for the Düzce Earthquake.

Table 4.10 Percentage of the Yielded Members for Sarıyer MEV Dumlupınar Primary School in the X-direction

X-Direction			Percentage of Yielded Members (%)			
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**	**Story 4**
Düzce	**0.5-1.0 (Existing B.)**	**Columns**	100.0	100.0	100.0	0.0
	1.0-1.5 (Retrofitted B.)	**Columns**	0.0	0.0	0.0	0.0
	0.5-1.0 (Existing B.)	**Beams**	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	**Beams**	57.9	67.6	67.6	59.5
	0.5-1.0 (Existing B.)	**Shear Walls**	100.0	100.0	50.0	0.0
	1.0-1.5 (Retrofitted B.)	**Shear Walls**	25.0	12.5	0.0	0.0
Kobe	**0.5-1.0 (Existing B.)**	**Columns**	100.0	100.0	100.0	0.0
	1.0-1.5 (Retrofitted B.)	**Columns**	73.3	71.4	64.3	0.0
	0.5-1.0 (Existing B.)	**Beams**	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	**Beams**	94.7	97.3	97.3	81.1
	0.5-1.0 (Existing B.)	**Shear Walls**	100.0	100.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	**Shear Walls**	100.0	62.5	0.0	0.0
Chi-Chi	**0.5-1.0 (Existing B.)**	**Columns**	100.0	100.0	100.0	0.0
	1.0-1.5 (Retrofitted B.)	**Columns**	93.3	53.6	42.9	0.0
	0.5-1.0 (Existing B.)	**Beams**	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	**Beams**	92.1	94.6	94.6	70.3
	0.5-1.0 (Existing B.)	**Shear Walls**	100.0	75.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	**Shear Walls**	100.0	37.5	0.0	0.0

Table 4.11 Percentage of Yielded Members for Sarıyer MEV Dumlupınar Primary School in the Y-direction

Y-Direction			Percentage of Yielded Members (%)			
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**	**Story 4**
Düzce	**0.5-1.0 (Existing B.)**	**Columns**	100.0	100.0	100.0	0.0
	1.0-1.5 (Retrofitted B.)	**Columns**	0.0	0.0	0.0	0.0
	0.5-1.0 (Existing B.)	**Beams**	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	**Beams**	100.0	100.0	100.0	100.0
	0.5-1.0 (Existing B.)	**Shear Walls**	100.0	100.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	**Shear Walls**	100.0	25.0	0.0	0.0
Kobe	**0.5-1.0 (Existing B.)**	**Columns**	100.0	100.0	100.0	0.0
	1.0-1.5 (Retrofitted B.)	**Columns**	73.3	71.4	64.3	0.0
	0.5-1.0 (Existing B.)	**Beams**	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	**Beams**	100.0	100.0	100.0	100.0
	0.5-1.0 (Existing B.)	**Shear Walls**	100.0	100.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	**Shear Walls**	100.0	25.0	0.0	0.0
Chi-Chi	**0.5-1.0 (Existing B.)**	**Columns**	100.0	100.0	100.0	0.0
	1.0-1.5 (Retrofitted B.)	**Columns**	93.3	53.6	42.9	0.0
	0.5-1.0 (Existing B.)	**Beams**	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	**Beams**	100.0	100.0	100.0	100.0
	0.5-1.0 (Existing B.)	**Shear Walls**	100.0	100.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	**Shear Walls**	100.0	25.0	0.0	0.0

The percentage of the yielded members for the three-story reinforced concrete building, Fatih Gazi Primary School is given in Tables 4.12 and 4.13. As mentioned earlier for Sarıyer MEV Dumlupınar Primary School, percentage of the yielded members in both directions shows regular distribution like the designed buildings. These tables indicate that, almost all the reinforced concrete beams are yielded in all buildings under the selected earthquakes. When 1.00% shear wall ratio is selected for strengthening the original building, the percentage of yielded vertical members reduced, but 1.50% shear wall ratio is more effective in minimizing the percentage of yielded vertical members and therefore, 1.50% is the selected ratio in strengthening this existing building in 2007. The tables providing the percentage of yielded members for other existing buildings are given in Appendix A.37, through A.40.

Table 4.12 Percentage of Yielded Members for Fatih Gazi Primary School in the X-direction

X-Direction			Percentage of Yielded Members (%)		
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**
Düzce	**0.5-0.0 (Existing B.)**	**Columns**	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	**Columns**	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	**Columns**	2.9	8.8	8.8
	1.5-1.5 (Retrofitted B.)	**Columns**	3.7	3.7	3.7
	0.5-0.0 (Existing B.)	**Beams**	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	**Beams**	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	**Beams**	100.0	100.0	100.0
	1.5-1.5 (Retrofitted B.)	**Beams**	100.0	100.0	100.0
	0.5-0.0 (Existing B.)	**Shear Walls**	100.0	100.0	0.0
	0.5-0.5 (Generated B.)	**Shear Walls**	100.0	100.0	0.0
	1.0-1.0 (Generated B.)	**Shear Walls**	28.6	0.0	0.0
	1.5-1.5 (Retrofitted B.)	**Shear Walls**	0.0	0.0	0.0
Kobe	**0.5-0.0 (Existing B.)**	**Columns**	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	**Columns**	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	**Columns**	67.6	44.1	32.4
	1.5-1.5 (Retrofitted B.)	**Columns**	14.8	11.1	0.0
	0.5-0.0 (Existing B.)	**Beams**	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	**Beams**	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	**Beams**	100.0	100.0	100.0
	1.5-1.5 (Retrofitted B.)	**Beams**	100.0	100.0	100.0
	0.5-0.0 (Existing B.)	**Shear Walls**	100.0	100.0	0.0
	0.5-0.5 (Generated B.)	**Shear Walls**	100.0	100.0	0.0
	1.0-1.0 (Generated B.)	**Shear Walls**	100.0	57.1	0.0
	1.5-1.5 (Retrofitted B.)	**Shear Walls**	100.0	18.2	0.0
Chi-Chi	**0.5-0.0 (Existing B.)**	**Columns**	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	**Columns**	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	**Columns**	41.2	23.5	20.6
	1.5-1.5 (Retrofitted B.)	**Columns**	11.1	11.1	7.4
	0.5-0.0 (Existing B.)	**Beams**	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	**Beams**	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	**Beams**	100.0	100.0	100.0
	1.5-1.5 (Retrofitted B.)	**Beams**	100.0	100.0	100.0
	0.5-0.0 (Existing B.)	**Shear Walls**	100.0	100.0	0.0
	0.5-0.5 (Generated B.)	**Shear Walls**	100.0	100.0	0.0
	1.0-1.0 (Generated B.)	**Shear Walls**	85.7	42.9	0.0
	1.5-1.5 (Retrofitted B.)	**Shear Walls**	81.8	0.0	0.0

Table 4.13 Percentage of Yielded Members for Fatih Gazi Primary School in the Y-direction

Y-Direction			Percentage of Yielded Members (%)		
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**
Düzce	0.5-0.0 (Existing B.)	Columns	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	2.9	8.8	8.8
	1.5-1.5 (Retrofitted B.)	Columns	3.7	3.7	3.7
	0.5-0.0 (Existing B.)	Beams	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Beams	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Beams	100.0	100.0	77.1
	1.5-1.5 (Retrofitted B.)	Beams	97.1	100.0	88.2
	0.5-0.0 (Existing B.)	Shear Walls	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	80.0	0.0	0.0
	1.5-1.5 (Retrofitted B.)	Shear Walls	66.7	0.0	0.0
Kobe	0.5-0.0 (Existing B.)	Columns	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	67.6	44.1	32.4
	1.5-1.5 (Retrofitted B.)	Columns	14.8	11.1	0.0
	0.5-0.0 (Existing B.)	Beams	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Beams	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Beams	100.0	100.0	94.3
	1.5-1.5 (Retrofitted B.)	Beams	100.0	100.0	91.2
	0.5-0.0 (Existing B.)	Shear Walls	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	80.0	0.0	0.0
	1.5-1.5 (Retrofitted B.)	Shear Walls	33.3	0.0	0.0
Chi-Chi	0.5-0.0 (Existing B.)	Columns	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	41.2	23.5	20.6
	1.5-1.5 (Retrofitted B.)	Columns	11.1	11.1	7.4
	0.5-0.0 (Existing B.)	Beams	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Beams	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Beams	100.0	100.0	82.9
	1.5-1.5 (Retrofitted B.)	Beams	100.0	100.0	88.2
	0.5-0.0 (Existing B.)	Shear Walls	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	66.7	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	60.0	0.0	0.0
	1.5-1.5 (Retrofitted B.)	Shear Walls	16.7	0.0	0.0

4.3.4.1.2 Drift Limitations

For existing reinforced concrete buildings, earthquake load reduction factors are equal to 4 according to Table 2.5 in the Turkish Earthquake Code (2007) due to the low ductility of these buildings. Like designed buildings, maximum interstory drift values are determined and given in Table 4.14. From this table, it can be concluded that, drift values of all existing buildings satisfy the requirements of Turkish Earthquake Code except Eminönü Çemberlitaş Anatolian High School Block A. Maximum interstory drift value of this reinforced concrete building with no shear walls in the Y-direction exceeds the drift limitation of Turkish Earthquake Code (2007) given as 2.0%.

Table 4.14 Drift Values of the First Case of Designed Buildings following the requirements of Turkish Earthquake Code (2007)

	Shear Wall Ratio (%)	Direction	R	Maximum Story Displacement at i^th story level (d_{imax}) (mm)	Maximum Interstory Drift at i^th story level (δ_{imax}) (%)
Güngören Haznedar Abdi İpekçi Primary School Block B	1.5-1.0	X	4	1.20	0.15
	1.5-1.0	Y	4	2.50	0.32
	1.5-1.5	X	4	1.20	0.15
	1.5-1.5	Y	4	2.40	0.31
	2.5-2.0	X	4	0.70	0.09
	2.5-2.0	Y	4	2.40	0.31
G.O.P. Ülkü Primary School Block B	1.5-1.0	X	4	1.10	0.14
	1.5-1.0	Y	4	2.40	0.31
Sarıyer MEV Dumlupınar Primary School	0.5-1.0	X	4	2.60	0.34
	0.5-1.0	Y	4	2.20	0.28
	1.0-1.5	X	4	1.10	0.14
	1.0-1.5	Y	4	1.40	0.18
Fatih Gazi Primary School	0.5-0.0	X	4	5.30	0.68
	0.5-0.0	Y	4	7.40	0.95
	0.5-0.5	X	4	5.60	0.72
	0.5-0.5	Y	4	2.60	0.34
	1.0-1.0	X	4	2.10	0.27
	1.0-1.0	Y	4	1.40	0.18
	1.5-1.5	X	4	1.50	0.19
	1.5-1.5	Y	4	1.40	0.18
Eminönü Çemberlitaş Anatolian High School Block A	0.0-0.0	X	4	10.00	1.29
	0.0-0.0	Y	4	15.80	2.04
	0.5-0.5	X	4	6.20	0.80
	0.5-0.5	Y	4	10.70	1.38
	1.0-1.0	X	4	5.80	0.75
	1.0-1.0	Y	4	7.40	0.95
	1.0-1.5	X	4	6.50	0.84
	1.0-1.5	Y	4	3.80	0.49

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Summary

The influence of varying shear wall area to floor area ratios on the seismic behavior of reinforced concrete buildings is investigated in this study. First, five existing school buildings that have different number of stories, floor plans, cross-sectional areas of the members and material properties are selected and modeled. The retrofitted cases of these existing buildings are also investigated. Furthermore, one of the selected existing buildings is designed following the requirements of the Turkish Earthquake Code (2007), TS 498 (1987) and TS 500 (2000) considering different shear wall ratios. To represent the general building stock in Turkey, the material grades are selected as C20 and S420 for the designed buildings. Next, the orientation of the columns of this existing reinforced concrete building is changed and the building is redesigned with different shear wall ratios to explore the effect of column stiffness on the seismic behavior. Totally, twenty four mid-rise reinforced concrete building models that have shear wall ratios between 0.00% and 2.50% in both directions are modeled and analyzed by using nonlinear time-history analyses of the widely utilized software program, SAP2000 v14.2.0 (2009).

Seven different ground motion records with a wide range of PGA values are selected to perform nonlinear time-history analyses of each building model. The average values of the analytical results under the selected earthquake records are used in determining the seismic performance of the structures. The variations of roof drifts, base shear forces, base shear percentage carried by shear walls and the percentage of yielded members with increasing shear wall ratios are investigated for all buildings.

5.2 Conclusions

The influence of the shear wall area to floor area ratio on the seismic performance of existing reinforced concrete buildings is investigated in this study and the following conclusions are obtained:

- The analytical results prove that roof drift values of reinforced concrete buildings are significantly affected by the addition of reinforced concrete shear walls. Roof drifts reduce with increasing shear wall ratios, but the rate of decrease is lower for higher shear wall ratios. Designed and existing buildings with 1.00% shear wall ratio have significantly lower roof drifts when compared to buildings with 0.00% or 0.50% shear wall ratio. After the limit of 1.50% shear wall ratio, addition of shear walls has only a slight effect on the reduction of roof drift values.

- For some existing and designed reinforced concrete buildings, there is no substantial change in the roof drift values when the shear wall ratio is increased from 0.50% to 1.00%. If the analytical results are examined in detail, even for those buildings, the seismic performance of the structure is significantly affected by the varying shear wall ratio. Considerable plastic deformations are observed in the members of buildings with 0.50% shear wall ratio, especially for the ones with torsional irregularities. To limit the plastic deformations in an existing or a newly designed structure, which may lead to the formation of a failure mechanism, at least 1.00% shear wall ratio should be employed. Buildings with shear wall ratios of 1.50% or more, undergo negligible plastic deformations. The existing buildings constructed with considerably low concrete and reinforcement strength require further strengthening by using high shear wall ratios to prevent plastic deformations under severe earthquake loading. Therefore, material strength of existing buildings is an important factor, in selection of the required shear wall ratio for the strengthening of these buildings.

- The percentage of yielded members can be used as an indicator for the extend of plastic deformations. It is observed that as the shear wall ratio increases, the number of yielded members reduces under earthquake loading. Almost no vertical members have yielded for both the existing and newly designed buildings with 1.50% and higher shear wall ratios even under high intensity earthquakes, which eliminate the possible formation of a failure mechanism. On the other hand, the percentage of the yielded beams is not significantly affected by the variation of the shear wall ratios.

- Both existing and newly designed reinforced concrete buildings should have at least 1.00% shear wall area to floor area ratio to reduce the roof drift and plastic deformations. If 1.50% shear wall ratio is provided, the roof drift values are minimized and the structure almost behaves elastically. Increasing the shear wall ratio further attracts higher lateral loads without any significant increase in the seismic performance.

- The variation of roof drifts with increasing shear wall ratios in only one principal direction is also examined for some existing buildings in this study. When the shear wall ratio in one direction is kept constant and the one in the orthogonal direction is increased, the roof drift values increase for the direction where the shear wall ratio remains the same. When the shear wall ratio of the building is increased even in only one principal direction, the total weight and the stiffness of the building and thus the applied lateral forces increase which results in the amplification of the roof drift especially under severe earthquake loading.

- The maximum base shear and maximum roof drift values increase almost linearly, when the PGA values of the earthquake records increase for both existing and newly designed buildings.

- The analytical results indicate that the percentage of the base shear force carried by shear walls increase with increasing shear wall ratio for both existing and designed buildings, as expected. However, as the shear wall ratio gets higher, rate of increase in the base shear percentage of shear walls decrease. For the investigated buildings, the average base shear force carried by the shear walls in buildings with 0.50% and 1.00% shear wall ratios is around 80% and 95%, respectively, for the ones with 1.50% and 2.00% shear wall ratios this percentage is almost the same in the range of 95-100%. Therefore, it can be stated that, the shear walls of the reinforced concrete buildings with 1.00% or higher shear wall ratios carry almost all the total base shear under earthquake loading reducing the demand on columns. On the other hand, the base shear carried by the columns of the buildings with 0.50% shear wall ratio is approximately 20%, which bring out the need for ductile detailing of these members in the shear wall-frame structures with 0.50% or lower shear wall ratios under earthquake loading.

- The percentage of the base shear carried by the shear walls is also influenced by the stiffness of the columns in the shear wall-frame structures. First and second cases of the designed buildings have the same structural properties expect for the column orientations. The Y-direction of the first case and the X-direction of the second case is stiffer than the orthogonal direction. The shear wall contribution in carrying lateral loads is effected by around 10% in both directions due to the orientation of the columns. It is observed that if the placement of columns increases the stiffness of a structure predominantly in one direction, higher shear forces will be attracted to those columns and earthquake demands will increase, which has to be taken into account in the design process.

- Torsional irregularity may result in the formation of undesired failure mechanisms in reinforced concrete structures. It is demonstrated that by positioning shear walls to reduce the drift values observed in a reinforced concrete building with an unsymmetrical plan, torsional irregularity can be prevented or minimized.

5.3 Recommendations for Future Research

The influence of reinforced concrete shear walls on the seismic behavior of mid-rise existing reinforced concrete buildings is investigated in this analytical study. Further investigations should be performed on different parameters and methodologies, to improve and support the findings of this study. Reinforced concrete buildings with L-shaped or T-shaped structural walls should be examined to recognize the effects of non-rectangular shear wall on the seismic performance. This analytical study focuses on 3 to 5 story buildings, since a prior study by Burak and Comlekoglu (2012) denoted that the seismic performance of 5-story buildings is significantly affected by the variation in shear wall area to floor area ratio. In future studies, the number of stories can be increased to determine the influence of shear walls on the behavior of high-rise existing buildings against under earthquake loading. In this analytical study, it is observed that the placement of new shear walls is difficult for buildings with shear wall ratios higher than 1.50% due to architectural requirements. The limiting shear wall ratio, for which the shear wall thickness should be increased rather than adding new shear walls, can be investigated. The effect of material strength is another parameter that should be explored. The use of high strength or low strength materials and the degradation of bond when the concrete compressive strength is below the code limitations should be investigated. Existing reinforced concrete theatre or cinema halls can be inspected to study the seismic performance of structures that have longer spans. Other analytical methods, such as pushover analysis or response spectrum analysis, can be utilized to compare the obtained results with the nonlinear time history analysis. Performance levels of the members can be investigated to obtain the seismic performance of reinforced concrete buildings in further detail.

In this analytical study, rigid diaphragms are used at each story level and adequate transfer of load from shear walls to the slabs is considered. Reinforced concrete buildings which have inadequate connection between slabs and shear walls can be examined in further detail. Moreover, the effect of foundation type on the seismic behavior and the soil-structure interaction can be investigated.

Different software programs or analytical procedures in SAP2000 v14.2.0 (2009) can be researched, such as changing the type of the time history analysis from direct integration to modal analysis.

In this analytical study, seven different earthquake records are applied to the building models. Different ground motions can be utilized to check the effect of impact loading or different soil conditions on the seismic performance of reinforced concrete buildings. In the selection of the earthquake records, other parameters such as the effective duration, energy content and frequency content of the earthquakes can be taken into account.

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APPENDIX A

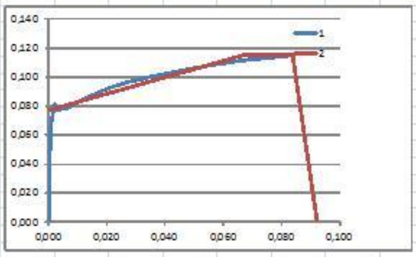
PROCEDURE FOLLOWED IN THE ANALYTICAL MODEL

A.1 Prepared Spreadsheet to obtain the Moment-Curvature Diagram of the Beams

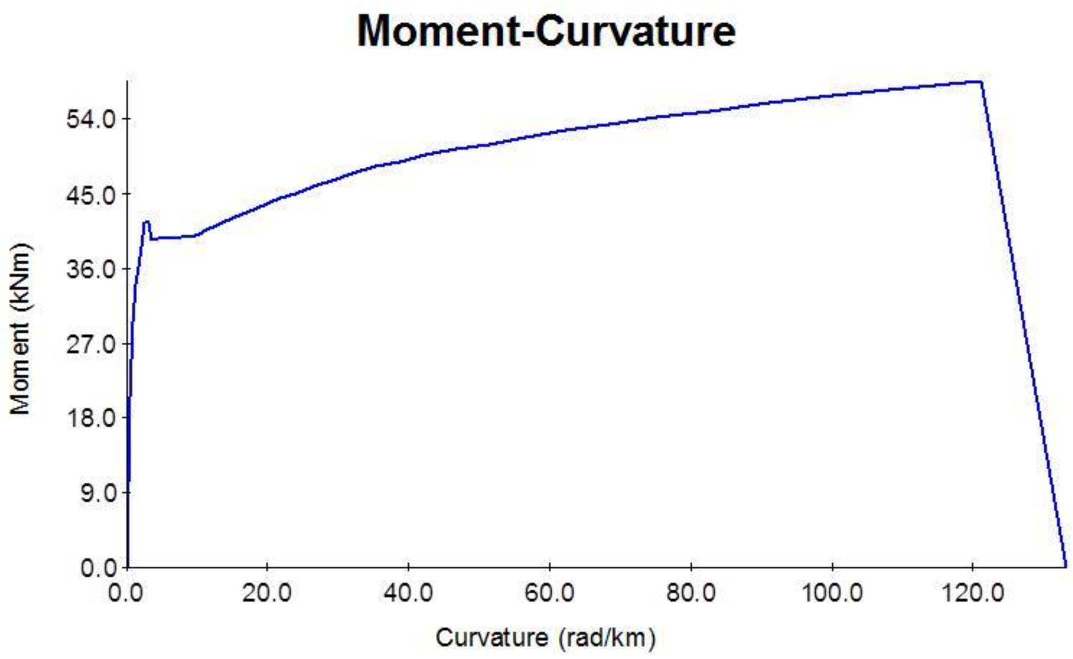
Example:

A1total =	8.866	M2=	115.182	M3=	0.000	C2-3=	0.084	C4=	0.092	C2=	0.067	C3=	0.084
A2total =	8.859	M1=	77.000	C1=	0.000	A1/A2=	1.000764						

C	M	C	M	C-1	M-1	A1	M2	C2	C-2	M-2
0.000 ,	-0.000	0.000	-0.000	0.000	0.000	3.614574	0	0	0.000	77.000
0.204 ,	35.437	0.204	35.437	0.204	35.437	14.53323	0	0	66.879	115.182
0.511 ,	59.242	0.511	59.242	0.511	59.242	19.6926	0	0	83.808	115.182
0.819 ,	68.632	0.819	68.632	0.819	68.632	21.39861	0	0	92.189	0.000
1.127 ,	70.320	1.127	70.320	1.127	70.320	22.28237	0	0		
1.434 ,	74.842	1.434	74.842	1.434	74.842	23.85368	0	0		
1.742 ,	80.052	1.742	80.052	1.742	80.052	24.82218	0	0		
2.050 ,	81.131	2.050	81.131	2.050	81.131	24.25438	0	0		
2.357 ,	76.878	2.357	76.878	2.357	76.878	23.70907	0	0		
2.665 ,	77.077	2.665	77.077	2.665	77.077	23.76882	0	0		
2.973 ,	77.266	2.973	77.266	2.973	77.266	23.74952	0	0		
3.280 ,	77.454	3.280	77.454	3.280	77.454	25.51296	0	0		
3.609 ,	77.640	3.609	77.640	3.609	77.640	27.97776	0	0		
3.969 ,	77.792	3.969	77.792	3.969	77.792	30.91876	0	0		
4.366 ,	77.970	4.366	77.970	4.366	77.970	34.10741	0	0		
4.803 ,	78.128	4.803	78.128	4.803	78.128	37.53792	0	0		
5.283 ,	78.280	5.283	78.280	5.283	78.280	41.44583	0	0		
5.812 ,	78.415	5.812	78.415	5.812	78.415	45.60647	0	0		
6.393 ,	78.578	6.393	78.578	6.393	78.578	50.39378	0	0		
7.032 ,	79.149	7.032	79.149	7.032	79.149	55.94615	0	0		
7.735 ,	80.015	7.735	80.015	7.735	80.015	62.29075	0	0		
8.509 ,	80.943	8.509	80.943	8.509	80.943	69.30289	0	0		
9.360 ,	81.951	9.360	81.951	9.360	81.951	77.18256	0	0		
10.296 ,	82.989	10.296	82.989	10.296	82.989	85.97604	0	0		
11.325 ,	84.117	11.325	84.117	11.325	84.117	95.98549	0	0		
12.458 ,	85.319	12.458	85.319	12.458	85.319	107.0127	0	0		
13.708 ,	86.589	13.708	86.589	13.708	86.589	119.5341	0	0		
15.074 ,	87.786	15.074	87.786	15.074	87.786	133.27	0	0		
16.581 ,	89.082	16.581	89.082	16.581	89.082	148.798	0	0		
18.239 ,	90.409	18.239	90.409	18.239	90.409	166.253	0	0		
20.063 ,	91.886	20.063	91.886	20.063	91.886	185.7656	0	0		
22.069 ,	93.324	22.069	93.324	22.069	93.324	207.5363	0	0		
24.276 ,	94.747	24.276	94.747	24.276	94.747	231.6992	0	0		
26.704 ,	96.109	26.704	96.109	26.704	96.109	258.5094	0	0		
29.374 ,	97.531	29.374	97.531	29.374	97.531	288.6629	0	0		
32.312 ,	98.972	32.312	98.972	32.312	98.972	322.1307	0	0		
35.543 ,	100.428	35.543	100.428	35.543	100.428	359.5973	0	0		
39.097 ,	101.934	39.097	101.934	39.097	101.934	401.6274	0	0		
43.007 ,	103.502	43.007	103.502	43.007	103.502	448.6094	0	0		
47.308 ,	105.105	47.308	105.105	47.308	105.105	501.0253	0	0		
52.038 ,	106.745	52.038	106.745	52.038	106.745	559.914	0	0		
57.242 ,	108.441	57.242	108.441	57.242	108.441	625.6332	0	0		
62.966 ,	110.159	62.966	110.159	62.966	110.159	699.1685	0	0		
69.263 ,	111.905	69.263	111.905	69.263	111.905	780.9619	0	0		
76.189 ,	113.611	76.189	113.611	76.189	113.611	871.5869	0	0		
83.808 ,	115.182	83.808	115.182	83.808	115.182	482.6702	115.182	83.808		
92.189 ,	0.000	92.189	0.000	92.189	0.000		0	0		



A.2 Example Moment-Curvature Diagram of a Beam

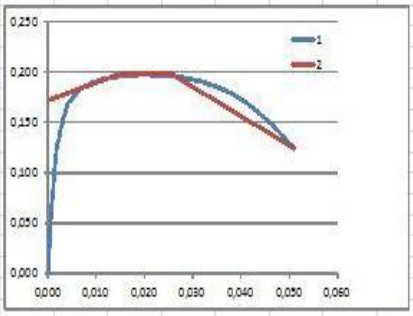


A.3 Prepared Spreadsheet to obtain the Moment-Curvature Diagram of the Columns

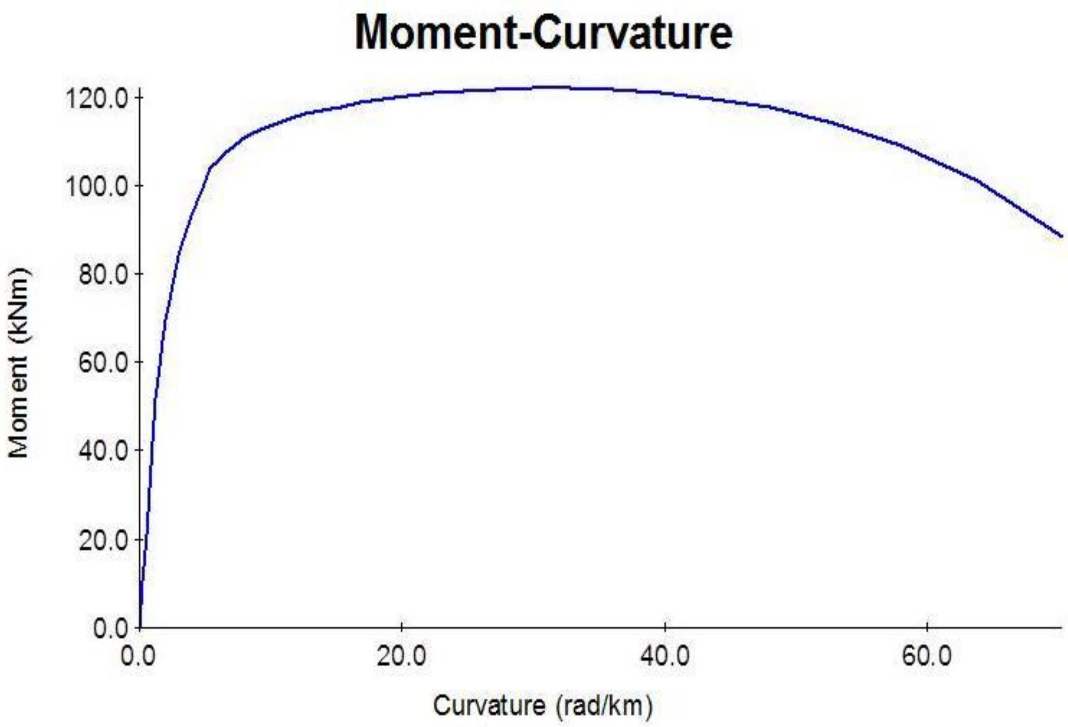
Example:

A1total =	8.974	M2=	197.821	M3=	123.714	C2-3=	0.020	C4=	0.051	C2=	0.015	C3=	0.026
A2total =	8.974	M1=	172.195	C1=	0.000	A1/A2=	1.000011						

C	M	C	M	C-1	M-1	A1	M2	C2	C-2	M-2
-0.000 ,	0.000	-0.000	0.000	0.000	0.000	1.6318	0	0	0.000	172.195
0.200 ,	16.318	0.200	16.318	0.200	16.318	4.8936	0	0	14.774	197.821
0.400 ,	32.618	0.400	32.618	0.400	32.618	8.1495	0	0	25.609	197.821
0.600 ,	48.877	0.600	48.877	0.600	48.877	11.3956	0	0	51.095	123.714
0.800 ,	65.079	0.800	65.079	0.800	65.079	14.6286	0	0		
1.000 ,	81.207	1.000	81.207	1.000	81.207	17.6289	0	0		
1.200 ,	95.082	1.200	95.082	1.200	95.082	20.1083	0	0		
1.400 ,	105.951	1.400	105.951	1.400	105.951	22.0712	0	0		
1.600 ,	114.761	1.600	114.761	1.600	114.761	23.6919	0	0		
1.800 ,	122.158	1.800	122.158	1.800	122.158	25.0486	0	0		
2.000 ,	128.328	2.000	128.328	2.000	128.328	26.1729	0	0		
2.200 ,	133.401	2.200	133.401	2.200	133.401	29.90493	0	0		
2.420 ,	138.462	2.420	138.462	2.420	138.462	34.11462	0	0		
2.662 ,	143.477	2.662	143.477	2.662	143.477	38.8102	0	0		
2.928 ,	148.329	2.928	148.329	2.928	148.329	44.20154	0	0		
3.221 ,	153.388	3.221	153.388	3.221	153.388	50.30574	0	0		
3.543 ,	159.070	3.543	159.070	3.543	159.070	57.31862	0	0		
3.897 ,	164.764	3.897	164.764	3.897	164.764	65.24798	0	0		
4.287 ,	169.841	4.287	169.841	4.287	169.841	73.45102	0	0		
4.716 ,	172.588	4.716	172.588	4.716	172.588	82.00628	0	0		
5.187 ,	175.634	5.187	175.634	5.187	175.634	91.91801	0	0		
5.706 ,	178.578	5.706	178.578	5.706	178.578	102.7369	0	0		
6.277 ,	181.271	6.277	181.271	6.277	181.271	114.6135	0	0		
6.905 ,	183.740	6.905	183.740	6.905	183.740	127.5738	0	0		
7.595 ,	186.039	7.595	186.039	7.595	186.039	141.9911	0	0		
8.354 ,	188.114	8.354	188.114	8.354	188.114	158.0253	0	0		
9.190 ,	189.957	9.190	189.957	9.190	189.957	175.1871	0	0		
10.109 ,	191.319	10.109	191.319	10.109	191.319	194.1201	0	0		
11.120 ,	192.697	11.120	192.697	11.120	192.697	215.0108	0	0		
12.232 ,	194.013	12.232	194.013	12.232	194.013	238.0056	0	0		
13.455 ,	195.208	13.455	195.208	13.455	195.208	263.1358	0	0		
14.800 ,	196.077	14.800	196.077	14.800	196.077	291.0557	0	0		
16.281 ,	196.976	16.281	196.976	16.281	196.976	321.1328	0	0		
17.909 ,	197.536	17.909	197.536	17.909	197.536	353.8445	0	0		
19.699 ,	197.821	19.699	197.821	19.699	197.821	389.4995	197.821	19.699		
21.669 ,	197.610	21.669	197.610	21.669	197.610	427.652	0	0		
23.836 ,	197.085	23.836	197.085	23.836	197.085	468.3511	0	0		
26.220 ,	195.827	26.220	195.827	26.220	195.827	510.7079	0	0		
28.842 ,	193.729	28.842	193.729	28.842	193.729	554.0895	0	0		
31.726 ,	190.487	31.726	190.487	31.726	190.487	596.5589	0	0		
34.899 ,	185.535	34.899	185.535	34.899	185.535	634.379	0	0		
38.389 ,	178.006	38.389	178.006	38.389	178.006	660.8992	0	0		
42.228 ,	166.302	42.228	166.302	42.228	166.302	662.7041	0	0		
46.450 ,	147.627	46.450	147.627	46.450	147.627	630.1895	0	0		
51.095 ,	123.714	51.095	123.714	51.095	123.714		0	0		



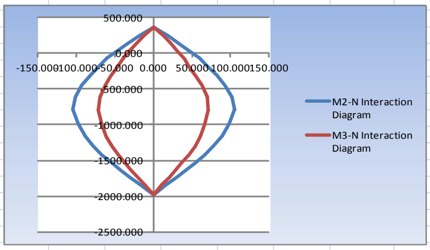
A.4 Example Moment-Curvature Diagram of a Column



A.5 Prepared Spreadsheet to obtain the Interaction Diagram of the Columns

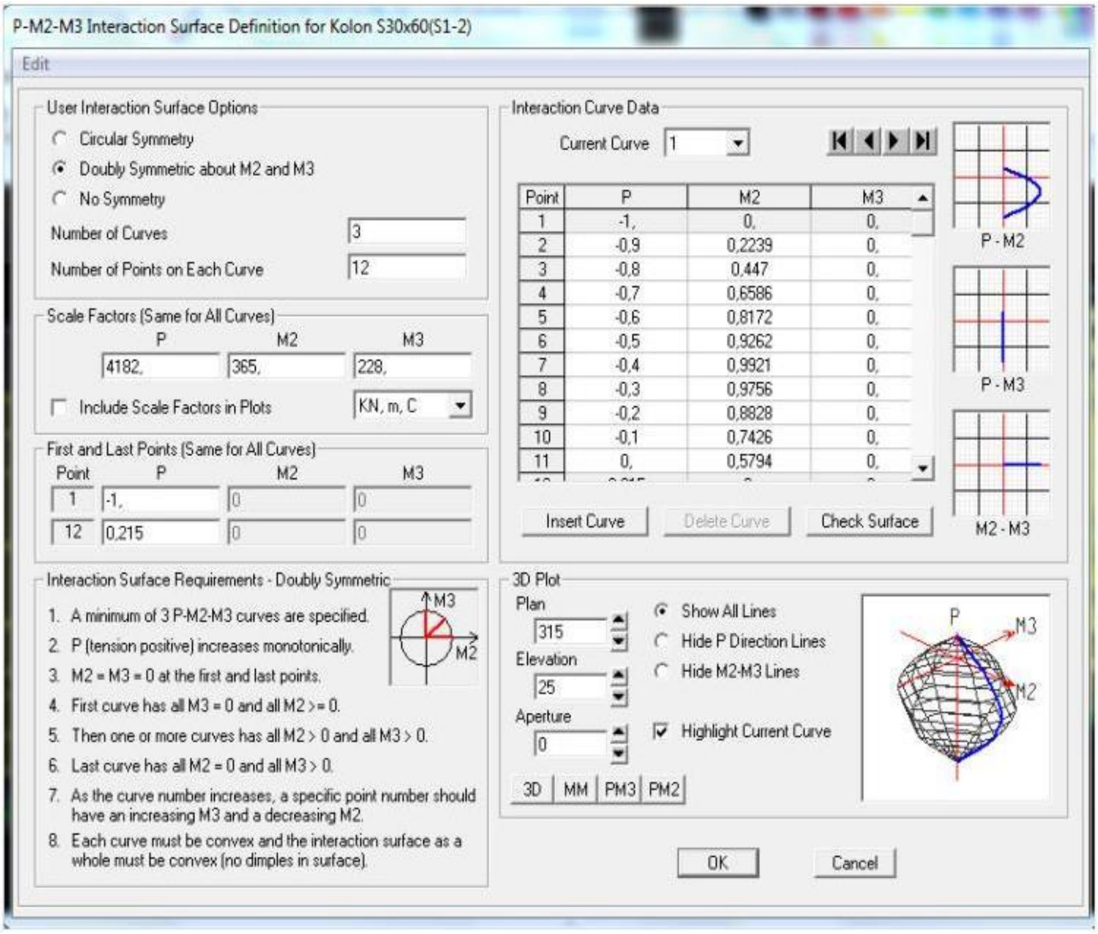
Example:

Pmak=	-1982.065	M2mak=	104.828	M3mak=	71.253		Curve 1			Curve 1			Curve 1			Curve2		
			M2	N	M2(kNm)	N(kN)	P(kN)	M2(kNm)	M3(kNm)	P(kN)	M2(kNm)	M3(kNm)	P(kN)	M2(kNm)	M3(kNm)	P(kN)	M2(kNm)	M3(kNm)
104.828 , -787.038			104.828	-787.038	104.828	-787.038	-0.397	1.000	0.000									
101.583 , -608.113			101.583	-608.113	101.583	-608.113	-0.307	0.969	0.000									
92.705 , -449.699			92.705	-449.699	92.705	-449.699	-0.227	0.884	0.000									
81.939 , -313.930			81.939	-313.930	81.939	-313.930	-0.158	0.782	0.000									
71.038 , -190.778			71.038	-190.778	71.038	-190.778	-0.096	0.678	0.000									
60.129 , -70.162			60.129	-70.162	60.129	-70.162	-0.035	0.574	0.000									
47.284 , 32.400			47.284	32.400	47.284	32.400	0.016	0.451	0.000									
33.515 , 134.723			33.515	134.723	33.515	134.723	0.068	0.320	0.000									
18.208 , 246.334			18.208	246.334	18.208	246.334	0.124	0.174	0.000									
0.000 , 361.021			0.000	361.021	0.000	361.021	0.182	0.000	0.000	-1.000	0.000	0.000	-1.000	0.000	0.000	-1.000	0.0000	0.0000
-18.208 , 246.334			-18.208	246.334	-18.208	246.334	0.124	-0.174	0.000	-0.932	-0.157	0.000	-0.900	-0.230	0.000	-0.900	-0.2128	-0.0871
-33.515 , 134.723			-33.515	134.723	-33.515	134.723	0.068	-0.320	0.000	-0.874	-0.291	0.000	-0.800	-0.456	0.000	-0.800	-0.4217	-0.1743
-47.284 , 32.400			-47.284	32.400	-47.284	32.400	0.016	-0.451	0.000	-0.822	-0.410	0.000	-0.700	-0.663	0.000	-0.700	-0.6127	-0.2544
-60.129 , -70.162			-60.129	-70.162	-60.129	-70.162	-0.035	-0.574	0.000	-0.769	-0.523	0.000	-0.600	-0.821	0.000	-0.600	-0.7584	-0.3163
-71.038 , -190.778			-71.038	-190.778	-71.038	-190.778	-0.096	-0.678	0.000	-0.716	-0.635	0.000	-0.500	-0.930	0.000	-0.500	-0.8590	-0.3576
-81.940 , -313.932			-81.940	-313.932	-81.940	-313.932	-0.158	-0.782	0.000	-0.654	-0.745	0.000	-0.400	-0.998	0.000	-0.400	-0.9222	-0.3818
-92.705 , -449.695			-92.705	-449.695	-92.705	-449.695	-0.227	-0.884	0.000	-0.581	-0.847	0.000	-0.300	-0.967	0.000	-0.300	-0.8932	-0.3716
-101.583 , -608.107			-101.583	-608.107	-101.583	-608.107	-0.307	-0.969	0.000	-0.495	-0.934	0.000	-0.200	-0.844	0.000	-0.200	-0.7799	-0.3211
-104.828 , -787.038			-104.828	-787.038	-104.828	-787.038	-0.397	-1.000	0.000	-0.397	-1.000	0.000	-0.100	-0.684	0.000	-0.100	-0.6320	-0.2572
-97.954 , -981.615			-97.954	-981.615	-97.954	-981.615	-0.495	-0.934	0.000	-0.307	-0.969	0.000	0.000	-0.490	0.000	0.000	-0.4525	-0.1842
-88.798 , -1152.167			-88.798	-1152.167	-88.798	-1152.167	-0.581	-0.847	0.000	-0.227	-0.884	0.000	0.182	0.000	0.000	0.182	0.0000	0.0000
-78.129 , -1295.620			-78.129	-1295.620	-78.129	-1295.620	-0.654	-0.745	0.000	-0.158	-0.782	0.000						
-66.606 , -1418.460			-66.606	-1418.460	-66.606	-1418.460	-0.716	-0.635	0.000	-0.096	-0.678	0.000						
-54.780 , -1524.629			-54.780	-1524.629	-54.780	-1524.629	-0.769	-0.523	0.000	-0.035	-0.574	0.000						
-42.953 , -1628.699			-42.953	-1628.699	-42.953	-1628.699	-0.822	-0.410	0.000	0.016	-0.451	0.000						
-30.467 , -1732.080			-30.467	-1732.080	-30.467	-1732.080	-0.874	-0.291	0.000	0.068	-0.320	0.000						
-16.457 , -1846.834			-16.457	-1846.834	-16.457	-1846.834	-0.932	-0.157	0.000	0.124	-0.174	0.000						
-0.000 , -1982.065			-0.000	-1982.065	0.000	-1982.065	-1.000	0.000	0.000	0.182	0.000	0.000						
16.457 , -1846.834			16.457	-1846.834	16.457	-1846.834	-0.932	0.157	0.000									
30.467 , -1732.080			30.467	-1732.080	30.467	-1732.080	-0.874	0.291	0.000									
42.953 , -1628.699			42.953	-1628.699	42.953	-1628.699	-0.822	0.410	0.000									
54.780 , -1524.629			54.780	-1524.629	54.780	-1524.629	-0.769	0.523	0.000									
66.606 , -1418.460			66.606	-1418.460	66.606	-1418.460	-0.716	0.635	0.000									
78.129 , -1295.620			78.129	-1295.620	78.129	-1295.620	-0.654	0.745	0.000									
88.798 , -1152.167			88.798	-1152.167	88.798	-1152.167	-0.581	0.847	0.000									
97.953 , -981.639			97.953	-981.639	97.953	-981.639	-0.495	0.934	0.000									
104.828 , -787.038			104.828	-787.038	104.828	-787.038	-0.397	1.000	0.000									

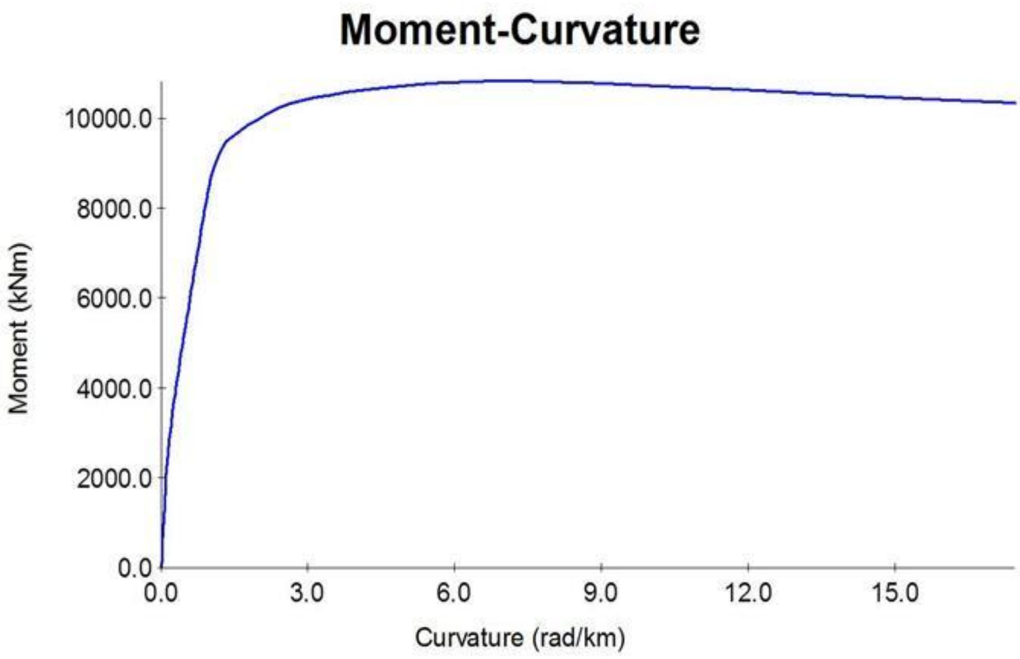


							Curve 3			Curve 3					
			M3	N	M3(kNm)	N(kN)	P(kN)	M2(kNm)	M3(kNm)	P(kN)	M2(kNm)	M3(kNm)			
71.253 , -795.470			71.253	-795.470	71.253	-795.470	-0.397	0.000	1.000						
69.367 , -612.264			69.367	-612.264	69.367	-612.264	-0.307	0.000	0.974						
62.840 , -453.006			62.840	-453.006	62.840	-453.006	-0.227	0.000	0.882						
55.115 , -319.407			55.115	-319.407	55.115	-319.407	-0.158	0.000	0.774						
47.446 , -199.799			47.446	-199.799	47.446	-199.799	-0.096	0.000	0.666						
40.111 , -80.259			40.111	-80.259	40.111	-80.259	-0.035	0.000	0.563						
31.644 , 24.344			31.644	24.344	31.644	24.344	0.016	0.000	0.444						
22.611 , 134.095			22.611	134.095	22.611	134.095	0.068	0.000	0.317						
12.304 , 248.472			12.304	248.472	12.304	248.472	0.124	0.000	0.173						
0.000 , 360.580			0.000	360.580	0.000	360.580	0.182	0.000	0.000	-1.000	0.000	0.000	-1.000	0.000	0.000
-12.304 , 248.472			-12.304	248.472	-12.304	248.472	0.124	0.000	-0.173	-0.932	0.000	-0.155	-0.900	0.000	-0.228
-22.611 , 134.095			-22.611	134.095	-22.611	134.095	0.068	0.000	-0.317	-0.874	0.000	-0.287	-0.800	0.000	-0.456
-31.644 , 24.344			-31.644	24.344	-31.644	24.344	0.016	0.000	-0.444	-0.822	0.000	-0.408	-0.700	0.000	-0.665
-40.119 , -80.264			-40.119	-80.264	-40.119	-80.264	-0.035	0.000	-0.563	-0.769	0.000	-0.523	-0.600	0.000	-0.827
-47.446 , -199.799			-47.446	-199.799	-47.446	-199.799	-0.096	0.000	-0.666	-0.716	0.000	-0.637	-0.500	0.000	-0.935
-55.115 , -319.407			-55.115	-319.407	-55.115	-319.407	-0.158	0.000	-0.774	-0.654	0.000	-0.750	-0.400	0.000	-0.998
-62.840 , -453.006			-62.840	-453.006	-62.840	-453.006	-0.227	0.000	-0.882	-0.581	0.000	-0.854	-0.300	0.000	-0.972
-69.367 , -612.264			-69.367	-612.264	-69.367	-612.264	-0.307	0.000	-0.974	-0.495	0.000	-0.940	-0.200	0.000	-0.839
-71.253 , -795.470			-71.253	-795.470	-71.253	-795.470	-0.397	0.000	-1.000	-0.397	0.000	-1.000	-0.100	0.000	-0.672
-66.962 , -990.908			-66.962	-990.908	-66.962	-990.908	-0.495	0.000	-0.940	-0.307	0.000	-0.974	0.000	0.000	-0.482
-60.836 , -1163.039			-60.836	-1163.039	-60.836	-1163.039	-0.581	0.000	-0.854	-0.227	0.000	-0.882	0.182	0.000	0.000
-53.413 , -1307.422			-53.413	-1307.422	-53.413	-1307.422	-0.654	0.000	-0.750	-0.158	0.000	-0.774			
-45.362 , -1429.074			-45.362	-1429.074	-45.362	-1429.074	-0.716	0.000	-0.637	-0.096	0.000	-0.666			
-37.259 , -1531.923			-37.259	-1531.923	-37.259	-1531.923	-0.769	0.000	-0.523	-0.035	0.000	-0.563			
-29.103 , -1634.183			-29.103	-1634.183	-29.103	-1634.183	-0.822	0.000	-0.408	0.016	0.000	-0.444			
-20.471 , -1734.979			-20.471	-1734.979	-20.471	-1734.979	-0.874	0.000	-0.287	0.068	0.000	-0.317			
-11.046 , -1843.983			-11.046	-1843.983	-11.046	-1843.983	-0.932	0.000	-0.155	0.124	0.000	-0.173			
-0.000 , -1982.474			-0.000	-1982.474	0.000	-1982.474	-1.000	0.000	0.000	0.182	0.000	0.000			
11.046 , -1843.983			11.046	-1843.983	11.046	-1843.983	-0.932	0.000	0.155						
20.471 , -1734.979			20.471	-1734.979	20.471	-1734.979	-0.874	0.000	0.287						
29.103 , -1634.183			29.103	-1634.183	29.103	-1634.183	-0.822	0.000	0.408						
37.259 , -1531.923			37.259	-1531.923	37.259	-1531.923	-0.769	0.000	0.523						
45.362 , -1429.074			45.362	-1429.074	45.362	-1429.074	-0.716	0.000	0.637						
53.413 , -1307.422			53.413	-1307.422	53.413	-1307.422	-0.654	0.000	0.750						
60.836 , -1163.039			60.836	-1163.039	60.836	-1163.039	-0.581	0.000	0.854						
66.962 , -990.908			66.962	-990.908	66.962	-990.908	-0.495	0.000	0.940						
71.253 , -795.470			71.253	-795.470	71.253	-795.470	-0.397	0.000	1.000						

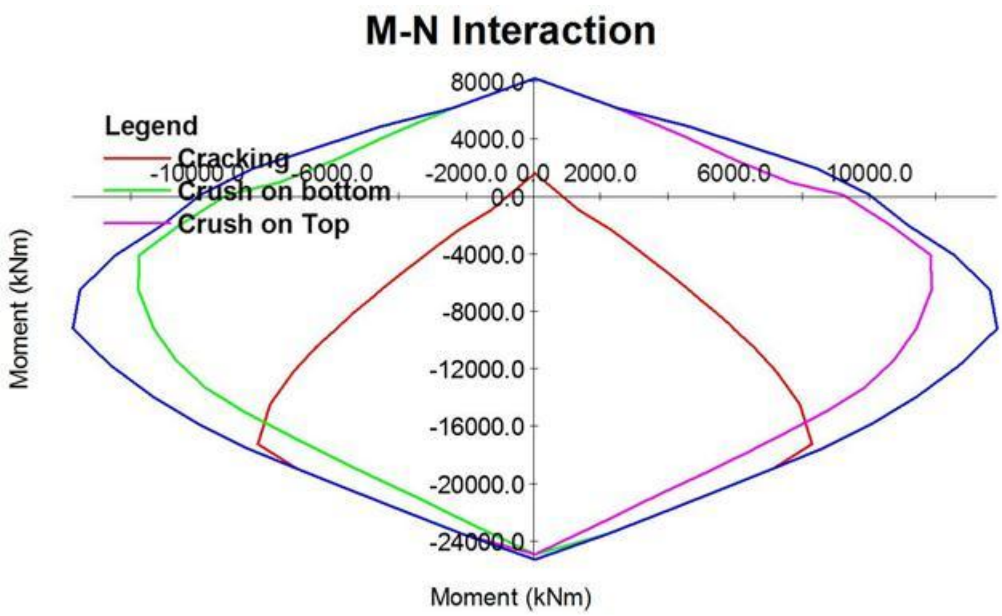
A.6 Example Interaction Diagram for Interacting P-M2-M3 Hinge of SAP2000 v14.2.0 (2009)



A.7 Example Moment-Curvature Diagram of a Shear Wall



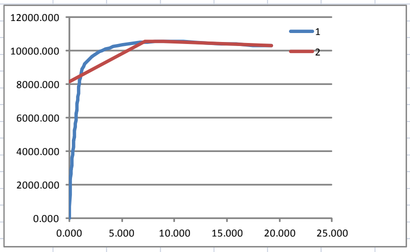
A.8 Example Interaction Diagram of a Shear Wall



A.9 Prepared Spreadsheet to obtain the Moment-Curvature Diagram of the Shear Walls

Example:

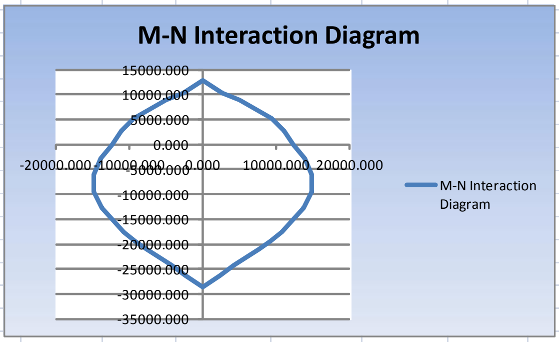
						A1total=	192.824	M2=	10564.739	M3=	10297.330	C2-3=	0.009	C4=	0.019	C2=	0.007	C3=	0.009
						A2total=	192.823	M1=	8167.133	C1=	0.000	A1/A2=	1.000003						
C	M	C	M	C-1	M-1	A1	M2	C2	C-2	M-2									
-0.000 , 0.000		-0.000	0.000	0.000	0.000	10.5459	0	0	0.000	8167.133									
0.030 , 703.060		0.030	703.060	0.030	703.060	32.66902	0	0	7.163	10564.739									
0.061 , 1404.619		0.061	1404.619	0.061	1404.619	50.73563	0	0	8.954	10564.739									
0.091 , 1977.756		0.091	1977.756	0.091	1977.756	64.89311	0	0	19.194	10297.330									
0.121 , 2348.451		0.121	2348.451	0.121	2348.451	77.39879	0	0											
0.152 , 2645.019		0.152	2645.019	0.152	2645.019	83.25987	0	0											
0.182 , 2905.639		0.182	2905.639	0.182	2905.639	90.77714	0	0											
0.212 , 3146.170		0.212	3146.170	0.212	3146.170	97.81223	0	0											
0.242 , 3374.645		0.242	3374.645	0.242	3374.645	108.0508	0	0											
0.273 , 3596.372		0.273	3596.372	0.273	3596.372	111.1343	0	0											
0.303 , 3812.578		0.303	3812.578	0.303	3812.578	117.5666	0	0											
0.333 , 4025.196		0.333	4025.196	0.333	4025.196	128.0825	0	0											
0.364 , 4238.193		0.364	4238.193	0.364	4238.193	130.2366	0	0											
0.394 , 4444.248		0.394	4444.248	0.394	4444.248	136.418	0	0											
0.424 , 4650.283		0.424	4650.283	0.424	4650.283	147.3505	0	0											
0.455 , 4856.203		0.455	4856.203	0.455	4856.203	148.7743	0	0											
0.485 , 5062.083		0.485	5062.083	0.485	5062.083	154.9459	0	0											
0.515 , 5267.642		0.515	5267.642	0.515	5267.642	161.0936	0	0											
0.545 , 5471.929		0.545	5471.929	0.545	5471.929	172.7945	0	0											
0.576 , 5676.101		0.576	5676.101	0.576	5676.101	173.326	0	0											
0.606 , 5878.965		0.606	5878.965	0.606	5878.965	179.402	0	0											
0.636 , 6081.168		0.636	6081.168	0.636	6081.168	191.6296	0	0											
0.667 , 6282.033		0.667	6282.033	0.667	6282.033	191.4648	0	0											
0.697 , 6482.290		0.697	6482.290	0.697	6482.290	197.4492	0	0											
0.727 , 6680.990		0.727	6680.990	0.727	6680.990	210.2508	0	0											
0.758 , 6883.580		0.758	6883.580	0.758	6883.580	209.5409	0	0											
0.788 , 7085.815		0.788	7085.815	0.788	7085.815	215.5766	0	0											
0.818 , 7285.959		0.818	7285.959	0.818	7285.959	221.5508	0	0											
0.848 , 7484.094		0.848	7484.094	0.848	7484.094	235.0663	0	0											
0.879 , 7681.471		0.879	7681.471	0.879	7681.471	233.3706	0	0											
0.909 , 7876.569		0.909	7876.569	0.909	7876.569	239.1971	0	0											
0.939 , 8069.904		0.939	8069.904	0.939	8069.904	252.9771	0	0											
0.970 , 8251.198		0.970	8251.198	0.970	8251.198	249.2659	0	0											
1.000 , 8366.527		1.000	8366.527	1.000	8366.527	852.3332	0	0											
1.100 , 8680.137		1.100	8680.137	1.100	8680.137	968.441	0	0											
1.210 , 8927.882		1.210	8927.882	1.210	8927.882	1089.484	0	0											
1.331 , 9080.124		1.331	9080.124	1.331	9080.124	1216.379	0	0											
1.464 , 9211.284		1.464	9211.284	1.464	9211.284	1361.797	0	0											
1.611 , 9316.561		1.611	9316.561	1.611	9316.561	1508.294	0	0											
1.772 , 9420.010		1.772	9420.010	1.772	9420.010	1676.872	0	0											
1.949 , 9527.698		1.949	9527.698	1.949	9527.698	1870.45	0	0											
2.144 , 9656.407		2.144	9656.407	2.144	9656.407	2079.524	0	0											
2.358 , 9778.393		2.358	9778.393	2.358	9778.393	2319.815	0	0											
2.594 , 9881.053		2.594	9881.053	2.594	9881.053	2570.641	0	0											
2.853 , 9969.454		2.853	9969.454	2.853	9969.454	2851.026	0	0											
3.138 , 10037.746		3.138	10037.746	3.138	10037.746	3163.417	0	0											
3.452 , 10111.406		3.452	10111.406	3.452	10111.406	3499.91	0	0											
3.797 , 10177.930		3.797	10177.930	3.797	10177.930	3879.768	0	0											
4.177 , 10241.903		4.177	10241.903	4.177	10241.903	4292.679	0	0											
4.595 , 10297.231		4.595	10297.231	4.595	10297.231	4741.945	0	0											
5.054 , 10364.837		5.054	10364.837	5.054	10364.837	5254.063	0	0											
5.560 , 10402.209		5.560	10402.209	5.560	10402.209	5798.14	0	0											
6.116 , 10454.411		6.116	10454.411	6.116	10454.411	6401.502	0	0											
6.727 , 10499.770		6.727	10499.770	6.727	10499.770	7076.005	0	0											
7.400 , 10528.476		7.400	10528.476	7.400	10528.476	7801.217	0	0											
8.140 , 10555.893		8.140	10555.893	8.140	10555.893	8596.097	0	0											
8.954 , 10564.739		8.954	10564.739	8.954	10564.739	9461.182	10564.739	8.954											
9.850 , 10553.970		9.850	10553.970	9.850	10553.970	10389.19	0	0											
10.835 , 10540.83		10.835	10540.83	10.835	10540.830	11399.98	0	0											
11.918 , 10511.77		11.918	10511.77	11.918	10511.770	12511.76	0	0											
13.110 , 10481.11		13.110	10481.11	13.110	10481.110	13708.66	0	0											
14.421 , 10432.18		14.421	10432.18	14.421	10432.180	15015.26	0	0											
15.863 , 10393.43		15.863	10393.43	15.863	10393.430	16439.51	0	0											
17.449 , 10337.35		17.449	10337.35	17.449	10337.350	18003.76	0	0											
19.194 , 10297.33		19.194	10297.33	19.194	10297.330		0	0											



A.10 Prepared Spreadsheet to obtain the Interaction Diagram of the Shear Walls

Example:

Pmak= -28574.734 Mmak=	14849.606					Curve 1			Curve 1		Curve 1	
	M2	N	M(kNm)	N(kN)	P(kN)	M2(kNm)	M3(kNm)	P(kN)	M2(kNm)	P(kN)	M2(kNm)
14807.966 , -9602.312	14807.966	-9602.312	14807.966	-9602.312	-0.336	0.997	0.000				
14849.606 , -6109.148	14849.606	-6109.148	14849.606	-6109.148	-0.214	1.000	0.000				
13881.721 , -2858.119	13881.721	-2858.119	13881.721	-2858.119	-0.100	0.935	0.000				
12343.363 , -89.234	12343.363	-89.234	12343.363	-89.234	-0.003	0.831	0.000				
11043.077 , 2763.458	11043.077	2763.458	11043.077	2763.458	0.097	0.744	0.000				
9326.708 , 5240.771	9326.708	5240.771	9326.708	5240.771	0.183	0.628	0.000				
7210.705 , 7067.894	7210.705	7067.894	7210.705	7067.894	0.247	0.486	0.000				
5018.925 , 8808.344	5018.925	8808.344	5018.925	8808.344	0.308	0.338	0.000				
2623.096 , 10261.235	2623.096	10261.235	2623.096	10261.235	0.359	0.177	0.000				
-0.099 , 12818.587	-0.099	12818.587	-0.099	12818.587	0.449	0.000	0.000	-1.000	0.000	-1.000	0.000
-2623.223 , 10261.147	-2623.223	10261.147	-2623.223	10261.147	0.359	-0.177	0.000	-0.918	-0.155	-0.900	-0.190
-5019.080 , 8809.232	-5019.080	8809.232	-5019.080	8809.232	0.308	-0.338	0.000	-0.852	-0.284	-0.800	-0.388
-7208.420 , 7069.903	-7208.420	7069.903	-7208.420	7069.903	0.247	-0.485	0.000	-0.794	-0.401	-0.700	-0.581
-9329.708 , 5237.683	-9329.708	5237.683	-9329.708	5237.683	0.183	-0.628	0.000	-0.738	-0.512	-0.600	-0.738
-11042.751 , 2761.759	-11042.751	2761.759	-11042.751	2761.759	0.097	-0.744	0.000	-0.679	-0.620	-0.500	-0.858
-12343.298 , -88.964	-12343.298	-88.964	-12343.298	-88.964	-0.003	-0.831	0.000	-0.612	-0.723	-0.400	-0.952
-13881.765 , -2858.212	-13881.765	-2858.212	-13881.765	-2858.212	-0.100	-0.935	0.000	-0.534	-0.822	-0.300	-0.998
-14849.599 , -6109.110	-14849.599	-6109.110	-14849.599	-6109.110	-0.214	-1.000	0.000	-0.443	-0.921	-0.200	-0.992
-14807.965 , -9602.313	-14807.965	-9602.313	-14807.965	-9602.313	-0.336	-0.997	0.000	-0.336	-0.997	-0.100	-0.935
-13671.098 , -12666.594	-13671.098	-12666.594	-13671.098	-12666.594	-0.443	-0.921	0.000	-0.214	-1.000	0.000	-0.828
-12200.821 , -15245.668	-12200.821	-15245.668	-12200.821	-15245.668	-0.534	-0.822	0.000	-0.100	-0.935	0.449	0.000
-10732.117 , -17481.397	-10732.117	-17481.397	-10732.117	-17481.397	-0.612	-0.723	0.000	-0.003	-0.831		
-9207.687 , -19391.383	-9207.687	-19391.383	-9207.687	-19391.383	-0.679	-0.620	0.000	0.097	-0.744		
-7603.203 , -21096.148	-7603.203	-21096.148	-7603.203	-21096.148	-0.738	-0.512	0.000	0.183	-0.628		
-5947.442 , -22685.231	-5947.442	-22685.231	-5947.442	-22685.231	-0.794	-0.401	0.000	0.247	-0.485		
-4222.811 , -24334.942	-4222.811	-24334.942	-4222.811	-24334.942	-0.852	-0.284	0.000	0.308	-0.338		
-2308.456 , -26223.789	-2308.456	-26223.789	-2308.456	-26223.789	-0.918	-0.155	0.000	0.359	-0.177		
-0.000 , -28574.734	-0.000	-28574.734	0.000	-28574.734	-1.000	0.000	0.000	0.449	0.000		
2308.456 , -26223.789	2308.456	-26223.789	2308.456	-26223.789	-0.918	0.155	0.000				
4222.811 , -24334.942	4222.811	-24334.942	4222.811	-24334.942	-0.852	0.284	0.000				
5947.442 , -22685.231	5947.442	-22685.231	5947.442	-22685.231	-0.794	0.401	0.000				
7603.203 , -21096.148	7603.203	-21096.148	7603.203	-21096.148	-0.738	0.512	0.000				
9207.687 , -19391.383	9207.687	-19391.383	9207.687	-19391.383	-0.679	0.620	0.000				
10732.117 , -17481.397	10732.117	-17481.397	10732.117	-17481.397	-0.612	0.723	0.000				
12200.821 , -15245.668	12200.821	-15245.668	12200.821	-15245.668	-0.534	0.822	0.000				
13671.098 , -12666.594	13671.098	-12666.594	13671.098	-12666.594	-0.443	0.921	0.000				
14807.965 , -9602.313	14807.965	-9602.313	14807.965	-9602.313	-0.336	0.997	0.000				



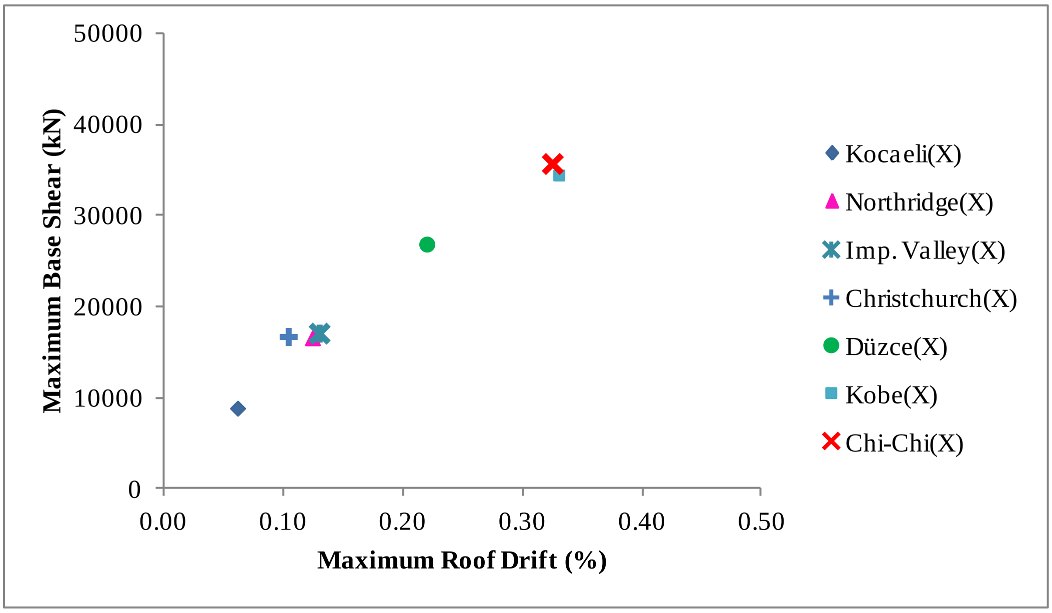
A.11 Maximum Base Shear and Maximum Roof Drift Values

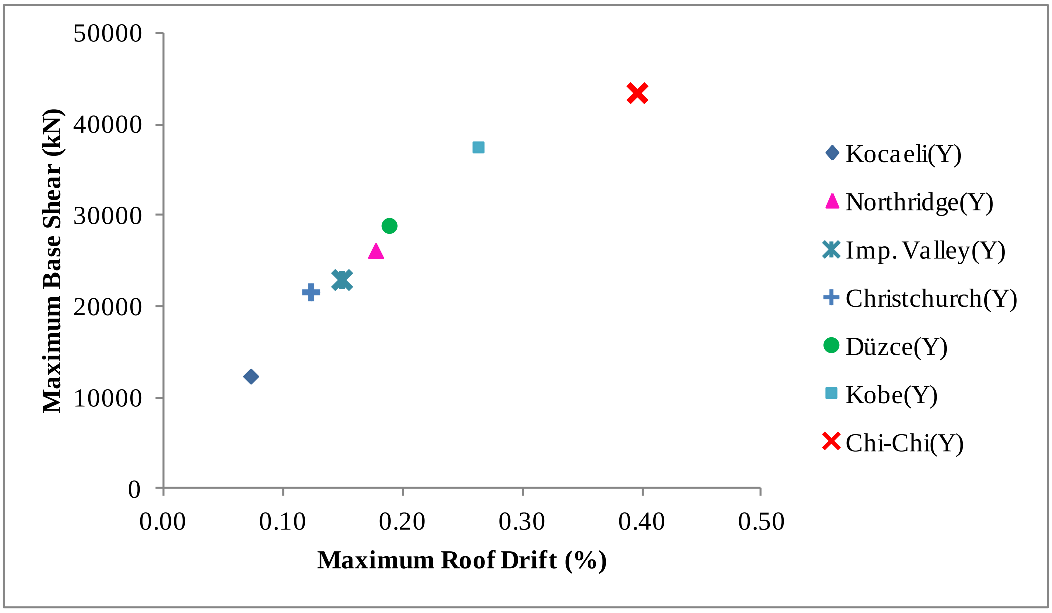
Building	Earthquake (X Direction)	Max. Base Shear(kN)	Max. Roof Drift(%)	Earthquake (Y Direction)	Max. Base Shear(kN)	Max. Roof Drift(%)
Sarıyer MEV Dumlupınar Primary School (Designed Building) (First Case) (0.0-0.0)	Kocaeli(X)	4981.0	0.17	Kocaeli(Y)	6437.1	0.12
	Northridge(X)	8391.8	0.28	Northridge(Y)	16800.7	0.38
	Imp. Valley(X)	13478.2	0.45	Imp. Valley(Y)	15965.1	0.35
	Christhurch(X)	10393.3	0.53	Christhurch(Y)	13809.3	0.30
	Duzce(X)	13465.2	0.94	Duzce(Y)	16878.8	0.40
	Kobe(X)	18112.7	1.58	Kobe(Y)	20730.9	0.58
	Chichi(X)	14012.4	1.00	Chichi(Y)	26044.4	0.71
Sarıyer MEV Dumlupınar Primary School (Designed Building) (First Case) (0.5-0.5)	Kocaeli(X)	7880.0	0.08	Kocaeli(Y)	10261.3	0.07
	Northridge(X)	13871.3	0.15	Northridge(Y)	22483.6	0.19
	Imp. Valley(X)	14954.1	0.16	Imp. Valley(Y)	20036.2	0.16
	Christhurch(X)	14683.4	0.13	Christhurch(Y)	18842.6	0.13
	Duzce(X)	23247.3	0.27	Duzce(Y)	24821.4	0.20
	Kobe(X)	29324.5	0.44	Kobe(Y)	32288.6	0.25
	Chichi(X)	31235.8	0.40	Chichi(Y)	37157.0	0.43
Sarıyer MEV Dumlupınar Primary School (Designed Building) (First Case) (1.0-1.0)	Kocaeli(X)	8865.4	0.06	Kocaeli(Y)	12380.5	0.07
	Northridge(X)	16606.0	0.12	Northridge(Y)	26189.5	0.18
	Imp. Valley(X)	17120.9	0.13	Imp. Valley(Y)	22997.3	0.15
	Christhurch(X)	16750.2	0.10	Christhurch(Y)	21644.9	0.12
	Duzce(X)	26909.1	0.22	Duzce(Y)	28945.6	0.19
	Kobe(X)	34506.8	0.33	Kobe(Y)	37576.6	0.26
	Chichi(X)	35769.6	0.32	Chichi(Y)	43543.4	0.40
Sarıyer MEV Dumlupınar Primary School (Designed Building) (First Case) (1.5-1.5)	Kocaeli(X)	12618.2	0.03	Kocaeli(Y)	17320.2	0.04
	Northridge(X)	22037.0	0.05	Northridge(Y)	33345.5	0.08
	Imp. Valley(X)	21497.4	0.05	Imp. Valley(Y)	27443.2	0.06
	Christhurch(X)	22122.4	0.05	Christhurch(Y)	22679.5	0.05
	Duzce(X)	26393.0	0.07	Duzce(Y)	38323.7	0.09
	Kobe(X)	43622.3	0.12	Kobe(Y)	47494.0	0.13
	Chichi(X)	48240.3	0.13	Chichi(Y)	50794.5	0.13
Sarıyer MEV Dumlupınar Primary School (Designed Building) (First Case) (2.0-2.0)	Kocaeli(X)	13043.0	0.02	Kocaeli(Y)	18054.6	0.02
	Northridge(X)	25060.9	0.04	Northridge(Y)	37051.1	0.05
	Imp. Valley(X)	22983.1	0.03	Imp. Valley(Y)	30391.6	0.04
	Christhurch(X)	23834.0	0.03	Christhurch(Y)	25602.4	0.03
	Duzce(X)	27853.2	0.04	Duzce(Y)	40967.3	0.06
	Kobe(X)	51898.7	0.08	Kobe(Y)	54215.3	0.07
	Chichi(X)	50306.9	0.07	Chichi(Y)	55908.9	0.08
Sarıyer MEV Dumlupınar Primary School (Designed Building) (Second Case) (0.0-0.0)	Kocaeli(X)	6321.0	0.08	Kocaeli(Y)	6884.8	0.29
	Northridge(X)	11081.4	0.15	Northridge(Y)	14164.5	0.72
	Imp. Valley(X)	13623.8	0.29	Imp. Valley(Y)	13248.6	0.69
	Christhurch(X)	11548.2	0.21	Christhurch(Y)	11548.2	0.50
	Duzce(X)	14026.1	0.30	Duzce(Y)	12458.1	0.55
	Kobe(X)	22916.1	0.45	Kobe(Y)	14666.2	2.36
	Chichi(X)	22807.1	0.74	Chichi(Y)	17280.3	1.43
Sarıyer MEV Dumlupınar Primary School (Designed Building) (Second Case) (0.5-0.5)	Kocaeli(X)	7894.5	0.05	Kocaeli(Y)	7984.6	0.08
	Northridge(X)	14955.9	0.10	Northridge(Y)	22108.9	0.24
	Imp. Valley(X)	14268.8	0.11	Imp. Valley(Y)	21123.0	0.22
	Christhurch(X)	15013.9	0.08	Christhurch(Y)	18665.4	0.17
	Duzce(X)	22197.5	0.18	Duzce(Y)	24056.2	0.26
	Kobe(X)	29607.2	0.30	Kobe(Y)	30101.8	0.36
	Chichi(X)	29047.1	0.56	Chichi(Y)	37168.6	0.78
Sarıyer MEV Dumlupınar Primary School (Designed Building) (Second Case) (1.0-1.0)	Kocaeli(X)	9209.7	0.05	Kocaeli(Y)	10431.3	0.08
	Northridge(X)	17206.6	0.10	Northridge(Y)	25864.7	0.21
	Imp. Valley(X)	16760.4	0.11	Imp. Valley(Y)	24085.1	0.18
	Christhurch(X)	17147.4	0.08	Christhurch(Y)	21858.4	0.15
	Duzce(X)	24795.0	0.17	Duzce(Y)	28543.8	0.22
	Kobe(X)	34224.2	0.24	Kobe(Y)	36735.0	0.32
	Chichi(X)	34756.0	0.26	Chichi(Y)	44034.2	0.47

Building	Earthquake (X Direction)	Max. Base Shear(kN)	Max. Roof Drift(%)	Earthquake (Y Direction)	Max. Base Shear(kN)	Max. Roof Drift(%)
Sarıyer MEV Dumlupınar Primary School (Designed Building) (Second Case) (1.5-1.5)	Kocaeli(X)	12527.2	0.03	Kocaeli(Y)	17709.3	0.04
	Northridge(X)	21994.3	0.05	Northridge(Y)	33596.8	0.09
	Imp. Valley(X)	21344.8	0.05	Imp. Valley(Y)	27412.3	0.07
	Christhurch(X)	22054.5	0.05	Christhurch(Y)	22704.4	0.05
	Duzce(X)	25909.8	0.06	Duzce(Y)	38650.4	0.10
	Kobe(X)	43831.2	0.11	Kobe(Y)	47609.3	0.12
	Chichi(X)	47111.8	0.12	Chichi(Y)	51282.5	0.14
Sarıyer MEV Dumlupınar Primary School (Designed Building) (Second Case) (2.0-2.0)	Kocaeli(X)	12974.7	0.02	Kocaeli(Y)	18146.4	0.02
	Northridge(X)	25012.3	0.04	Northridge(Y)	37197.9	0.05
	Imp. Valley(X)	22984.4	0.03	Imp. Valley(Y)	30479.0	0.04
	Christhurch(X)	23605.7	0.03	Christhurch(Y)	25449.5	0.03
	Duzce(X)	27763.3	0.04	Duzce(Y)	41118.5	0.06
	Kobe(X)	51830.2	0.07	Kobe(Y)	54268.9	0.08
	Chichi(X)	50055.1	0.07	Chichi(Y)	55812.1	0.08
Sarıyer MEV Dumlupınar Primary School (0.5-1.0)	Kocaeli(X)	7800.2	0.09	Kocaeli(Y)	8307.3	0.08
	Northridge(X)	11449.5	0.15	Northridge(Y)	20185.9	0.29
	Imp. Valley(X)	14441.3	0.20	Imp. Valley(Y)	19105.2	0.25
	Christhurch(X)	13920.3	0.22	Christhurch(Y)	17228.5	0.22
	Duzce(X)	17081.2	0.40	Duzce(Y)	21897.2	0.39
	Kobe(X)	25832.2	0.53	Kobe(Y)	26539.3	0.38
	Chichi(X)	26766.1	0.53	Chichi(Y)	32730.7	0.58
Sarıyer MEV Dumlupınar Primary School (1.0-1.5)	Kocaeli(X)	9679.3	0.04	Kocaeli(Y)	17072.6	0.06
	Northridge(X)	18672.3	0.08	Northridge(Y)	28259.7	0.13
	Imp. Valley(X)	18610.0	0.10	Imp. Valley(Y)	24000.0	0.09
	Christhurch(X)	19459.5	0.07	Christhurch(Y)	19964.6	0.07
	Duzce(X)	26713.1	0.11	Duzce(Y)	32622.0	0.14
	Kobe(X)	36672.2	0.24	Kobe(Y)	39964.7	0.17
	Chichi(X)	36349.0	0.22	Chichi(Y)	42525.9	0.26
Fatih Gazi Primary School (0.5-0.0)	Kocaeli(X)	4054.3	0.10	Kocaeli(Y)	4726.1	0.19
	Northridge(X)	7200.1	0.25	Northridge(Y)	11948.6	0.62
	Imp. Valley(X)	8891.8	0.45	Imp. Valley(Y)	10234.7	0.56
	Christhurch(X)	7407.8	0.52	Christhurch(Y)	8483.2	0.56
	Duzce(X)	10252.1	0.50	Duzce(Y)	11392.8	0.74
	Kobe(X)	14765.8	0.65	Kobe(Y)	14549.0	1.32
	Chichi(X)	15174.1	0.47	Chichi(Y)	16139.0	0.88
Fatih Gazi Primary School (0.5-0.5)	Kocaeli(X)	4855.0	0.13	Kocaeli(Y)	7026.7	0.05
	Northridge(X)	8072.1	0.24	Northridge(Y)	15988.2	0.16
	Imp. Valley(X)	10262.1	0.30	Imp. Valley(Y)	13865.7	0.15
	Christhurch(X)	8503.5	0.35	Christhurch(Y)	12532.9	0.10
	Duzce(X)	10716.6	0.38	Duzce(Y)	17703.2	0.23
	Kobe(X)	16763.7	0.74	Kobe(Y)	20995.6	0.26
	Chichi(X)	16548.8	0.60	Chichi(Y)	24784.5	0.50
Fatih Gazi Primary School (1.0-1.0)	Kocaeli(X)	7120.4	0.05	Kocaeli(Y)	10688.4	0.04
	Northridge(X)	13149.3	0.09	Northridge(Y)	20402.6	0.09
	Imp. Valley(X)	13189.2	0.09	Imp. Valley(Y)	16576.9	0.07
	Christhurch(X)	12902.2	0.08	Christhurch(Y)	13558.2	0.05
	Duzce(X)	17540.1	0.12	Duzce(Y)	23420.8	0.09
	Kobe(X)	25130.7	0.24	Kobe(Y)	29222.9	0.12
	Chichi(X)	27608.2	0.21	Chichi(Y)	30095.8	0.14
Fatih Gazi Primary School (1.5-1.5)	Kocaeli(X)	8034.2	0.04	Kocaeli(Y)	11259.8	0.03
	Northridge(X)	15038.7	0.07	Northridge(Y)	22233.7	0.07
	Imp. Valley(X)	14914.8	0.07	Imp. Valley(Y)	18043.2	0.05
	Christhurch(X)	14780.0	0.07	Christhurch(Y)	14620.7	0.04
	Duzce(X)	18101.5	0.08	Duzce(Y)	25126.2	0.07
	Kobe(X)	29281.8	0.18	Kobe(Y)	32172.5	0.09
	Chichi(X)	32114.4	0.15	Chichi(Y)	33971.9	0.10

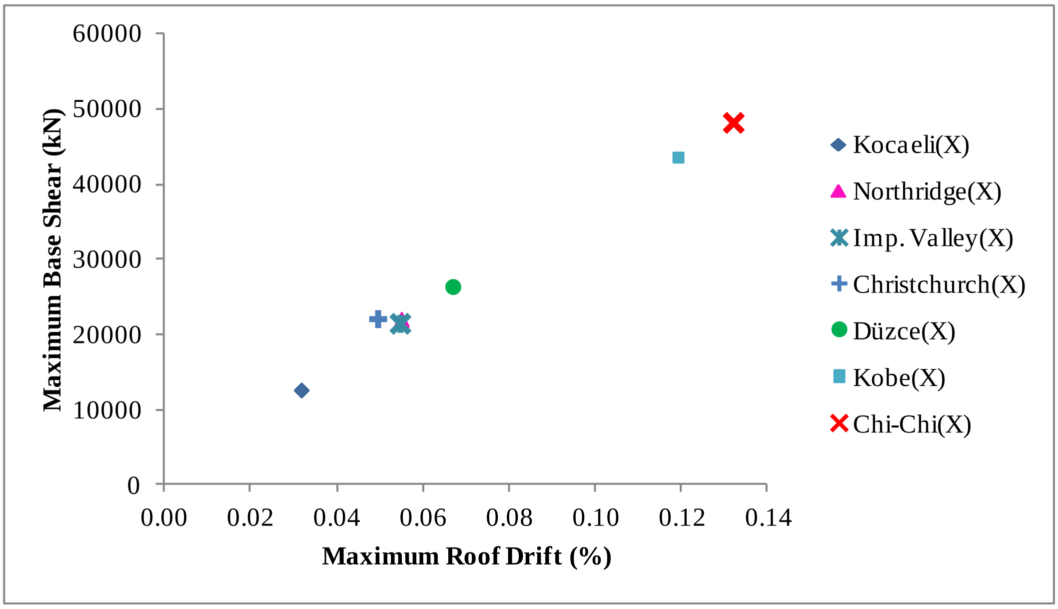
Building	Earthquake (X Direction)	Max. Base Shear(kN)	Max. Roof Drift(%)	Earthquake (Y Direction)	Max. Base Shear(kN)	Max. Roof Drift(%)
Eminönü Çemberlitaş Anatolian High School Block A (0.0-0.0)	Kocaeli(X)	2171.2	0.16	Kocaeli(Y)	3269.7	0.32
	Northridge(X)	3554.7	0.33	Northridge(Y)	6075.3	0.63
	Imp. Valley(X)	4799.4	0.56	Imp. Valley(Y)	6060.0	0.88
	Christhurch(X)	4362.7	0.59	Christhurch(Y)	4387.8	0.68
	Duzce(X)	4262.5	0.55	Duzce(Y)	6224.2	0.88
	Kobe(X)	7826.8	0.88	Kobe(Y)	6935.5	0.88
	Chichi(X)	7048.9	0.89	Chichi(Y)	6618.9	1.02
Eminönü Çemberlitaş Anatolian High School Block A (0.5-0.5)	Kocaeli(X)	3205.1	0.09	Kocaeli(Y)	4628.3	0.10
	Northridge(X)	5937.4	0.15	Northridge(Y)	10960.3	0.22
	Imp. Valley(X)	7626.6	0.20	Imp. Valley(Y)	9961.1	0.20
	Christhurch(X)	7203.0	0.25	Christhurch(Y)	8741.1	0.21
	Duzce(X)	9563.7	0.31	Duzce(Y)	12323.1	0.23
	Kobe(X)	12281.6	0.42	Kobe(Y)	15609.3	0.31
	Chichi(X)	12723.8	0.45	Chichi(Y)	18091.6	0.29
Eminönü Çemberlitaş Anatolian High School Block A (1.0-1.0)	Kocaeli(X)	4322.1	0.07	Kocaeli(Y)	6236.2	0.06
	Northridge(X)	7617.9	0.11	Northridge(Y)	13583.4	0.19
	Imp. Valley(X)	9424.7	0.13	Imp. Valley(Y)	12116.3	0.14
	Christhurch(X)	8753.8	0.11	Christhurch(Y)	10114.8	0.13
	Duzce(X)	12192.8	0.17	Duzce(Y)	16090.4	0.19
	Kobe(X)	17533.5	0.30	Kobe(Y)	20355.2	0.22
	Chichi(X)	17235.9	0.28	Chichi(Y)	21649.7	0.26
Eminönü Çemberlitaş Anatolian High School Block A (1.0-1.5)	Kocaeli(X)	4612.1	0.07	Kocaeli(Y)	7178.3	0.05
	Northridge(X)	8117.4	0.14	Northridge(Y)	14620.7	0.14
	Imp. Valley(X)	9889.6	0.13	Imp. Valley(Y)	12967.3	0.11
	Christhurch(X)	9170.5	0.12	Christhurch(Y)	11090.9	0.09
	Duzce(X)	13476.9	0.23	Duzce(Y)	17548.8	0.15
	Kobe(X)	18126.9	0.37	Kobe(Y)	21752.8	0.21
	Chichi(X)	17989.7	0.34	Chichi(Y)	23807.8	0.20
G.O.P. Ülkü Primary School Block B (1.5-1.0)	Kocaeli(X)	3831.1	0.03	Kocaeli(Y)	4541.9	0.06
	Northridge(X)	6682.5	0.07	Northridge(Y)	9994.0	0.19
	Imp. Valley(X)	6806.5	0.06	Imp. Valley(Y)	8634.3	0.17
	Christhurch(X)	7532.7	0.06	Christhurch(Y)	7591.5	0.12
	Duzce(X)	8778.8	0.10	Duzce(Y)	11401.5	0.22
	Kobe(X)	14269.2	0.10	Kobe(Y)	13576.7	0.31
	Chichi(X)	14039.7	0.16	Chichi(Y)	16153.6	0.35
Güngören Haznedar Abdi İpekçi Primary School Block B (1.5-1.0)	Kocaeli(X)	4415.1	0.03	Kocaeli(Y)	4371.5	0.08
	Northridge(X)	7151.7	0.09	Northridge(Y)	9457.9	0.36
	Imp. Valley(X)	7197.4	0.08	Imp. Valley(Y)	9061.3	0.29
	Christhurch(X)	8155.7	0.09	Christhurch(Y)	7523.3	0.26
	Duzce(X)	9716.1	0.14	Duzce(Y)	10871.8	0.54
	Kobe(X)	14157.8	0.15	Kobe(Y)	14834.0	0.56
	Chichi(X)	14763.7	0.23	Chichi(Y)	15187.7	0.66
Güngören Haznedar Abdi İpekçi Primary School Block B (1.5-1.5)	Kocaeli(X)	4591.4	0.03	Kocaeli(Y)	4808.7	0.07
	Northridge(X)	7958.6	0.10	Northridge(Y)	11217.8	0.29
	Imp. Valley(X)	7997.4	0.08	Imp. Valley(Y)	10685.5	0.23
	Christhurch(X)	8851.2	0.06	Christhurch(Y)	8982.0	0.17
	Duzce(X)	10473.5	0.11	Duzce(Y)	12934.7	0.38
	Kobe(X)	15734.7	0.20	Kobe(Y)	16137.3	0.44
	Chichi(X)	15938.6	0.22	Chichi(Y)	17786.4	0.50
Güngören Haznedar Abdi İpekçi Primary School Block B (2.5-2.0)	Kocaeli(X)	5137.5	0.02	Kocaeli(Y)	5375.4	0.06
	Northridge(X)	9760.7	0.04	Northridge(Y)	14091.4	0.23
	Imp. Valley(X)	9768.4	0.04	Imp. Valley(Y)	13261.2	0.21
	Christhurch(X)	10912.3	0.04	Christhurch(Y)	12097.4	0.15
	Duzce(X)	11895.4	0.05	Duzce(Y)	15772.6	0.26
	Kobe(X)	22147.0	0.08	Kobe(Y)	20291.7	0.29
	Chichi(X)	20329.6	0.09	Chichi(Y)	22487.1	0.49

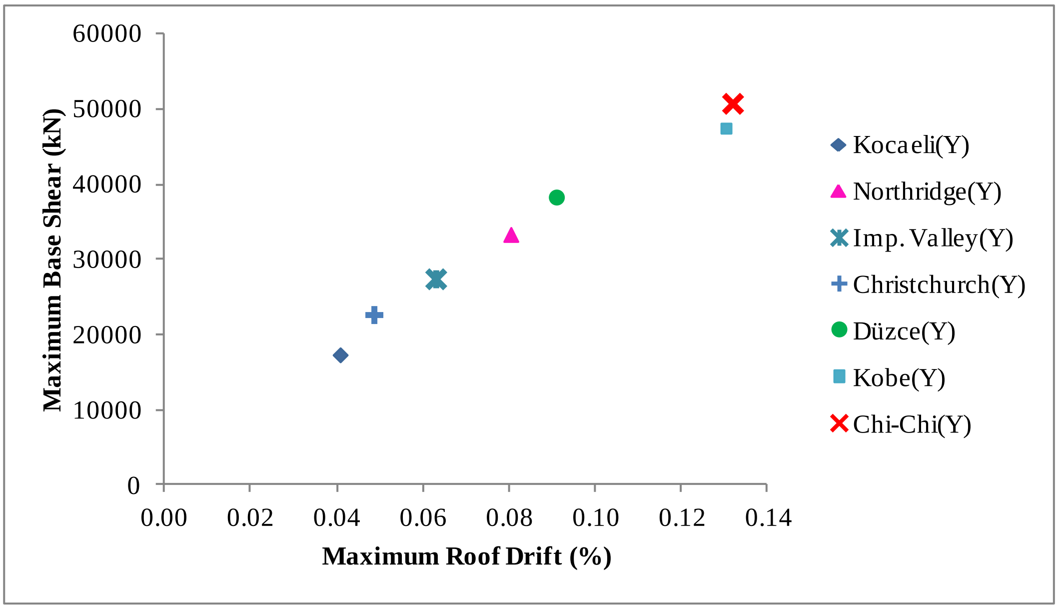
A.12 Maximum Base Shear vs. Maximum Roof Drift for the Designed Building with 1.00% Shear Wall Ratio for the First Case in the X and Y-Directions



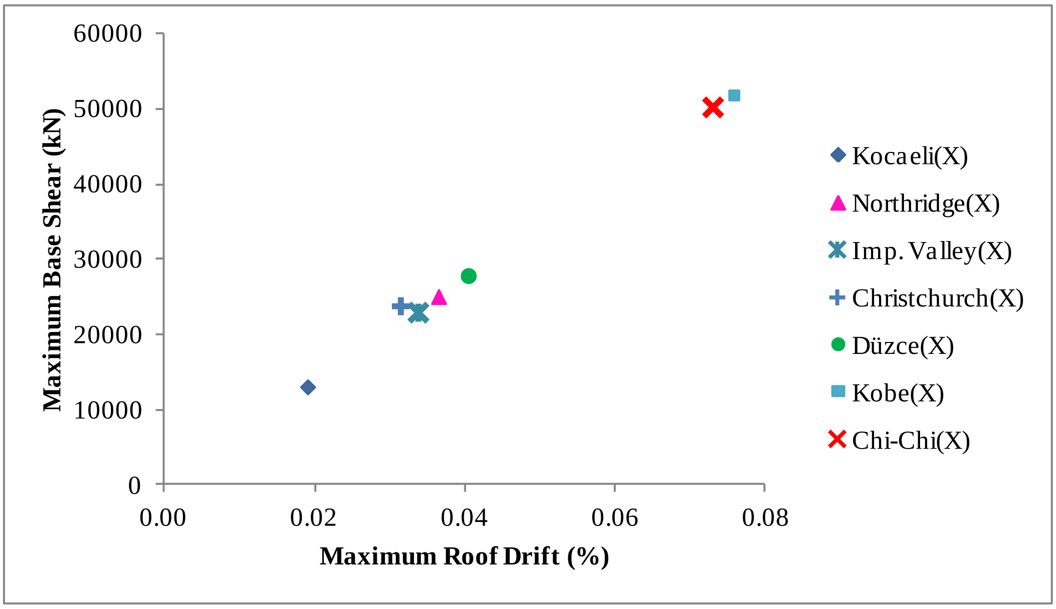


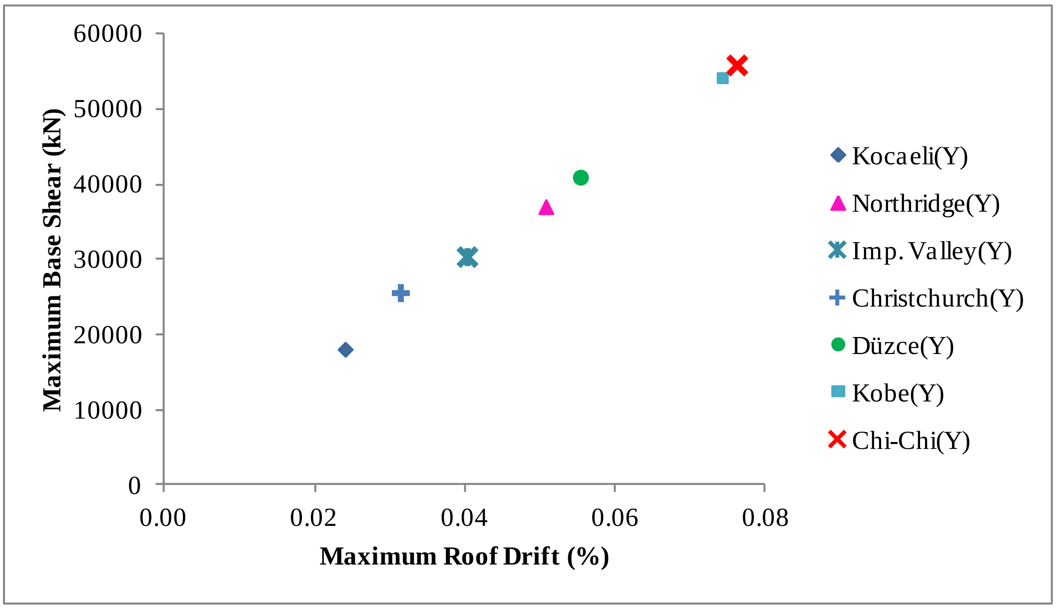
A.13 Maximum Base Shear vs. Maximum Roof Drift for the Designed Building with 1.50% Shear Wall Ratio for the First Case in the X and Y-Directions



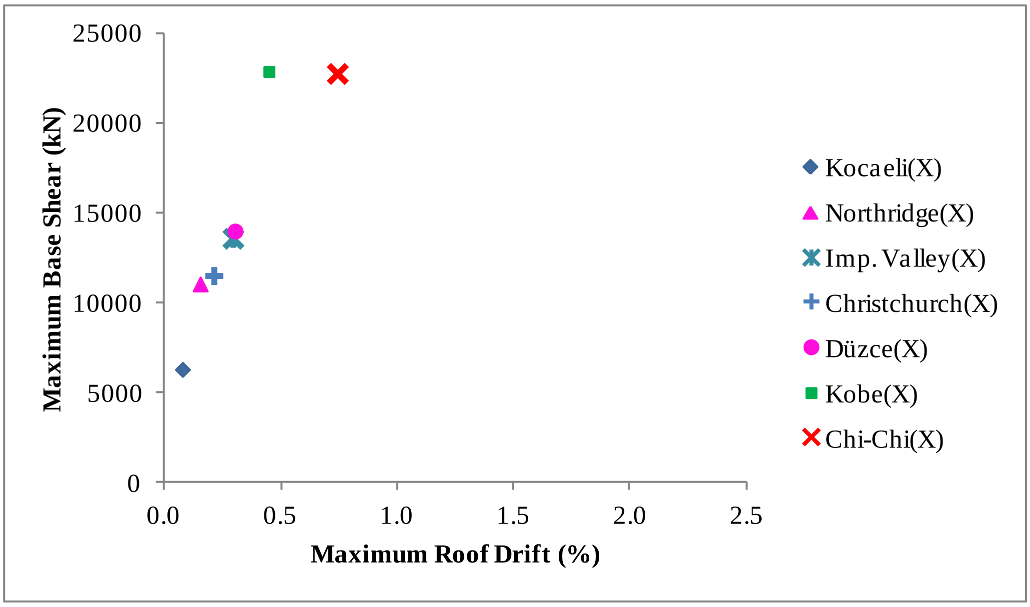


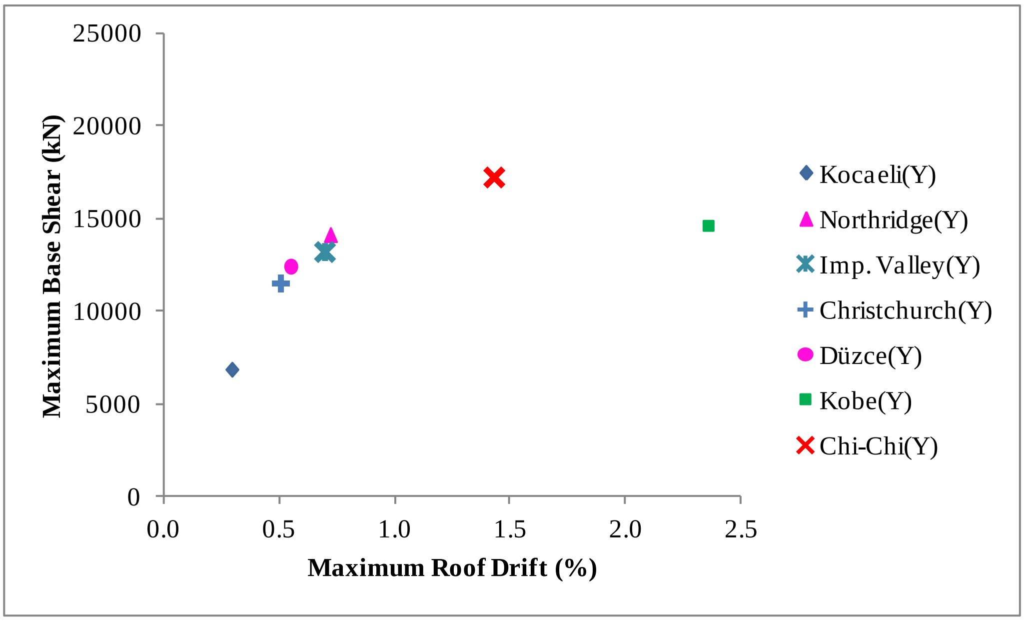
A.14 Maximum Base Shear vs. Maximum Roof Drift for the Designed Building with 2.00% Shear Wall Ratio for the First Case in the X and Y-Directions



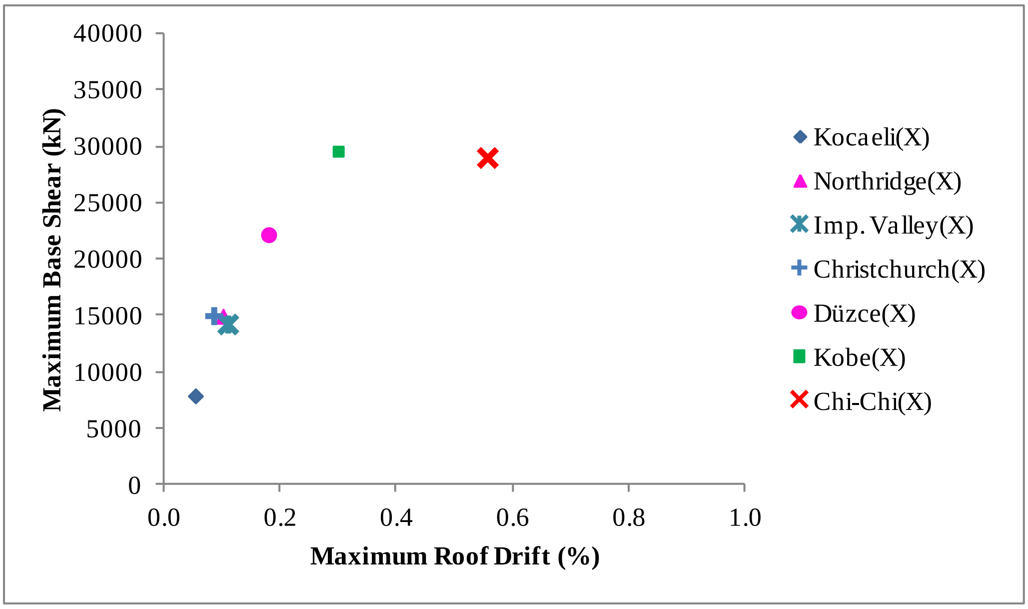


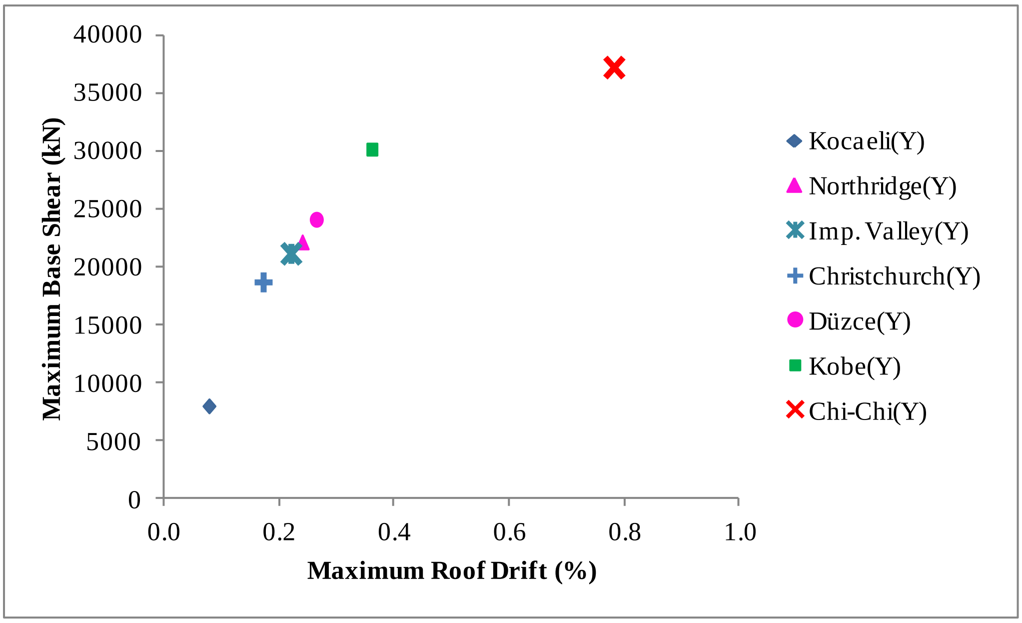
A.15 Maximum Base Shear vs. Maximum Roof Drift for the Designed Building with no Shear Wall for the Second Case in the X and Y-Directions



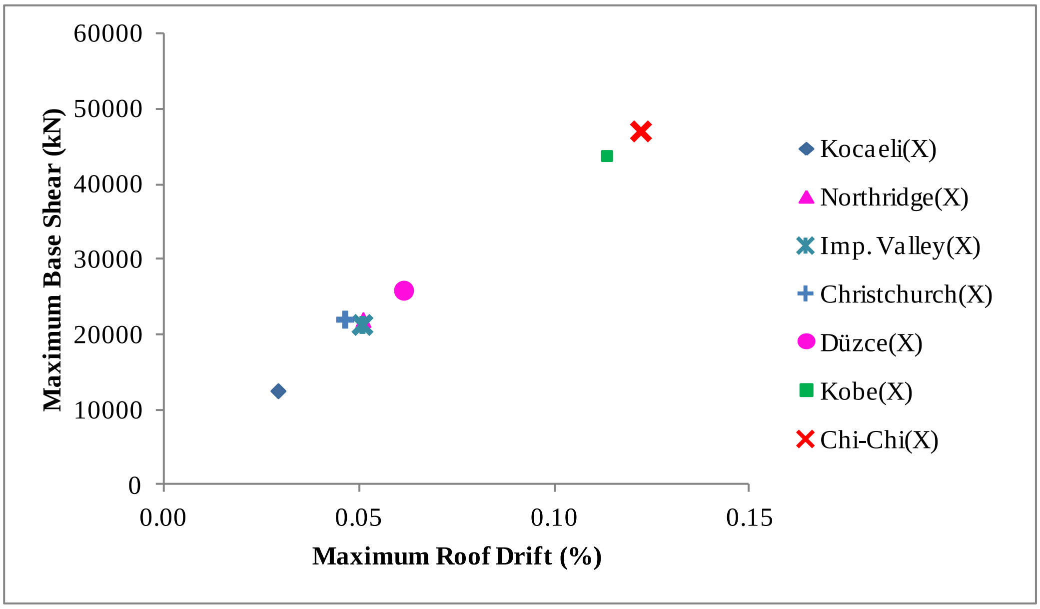


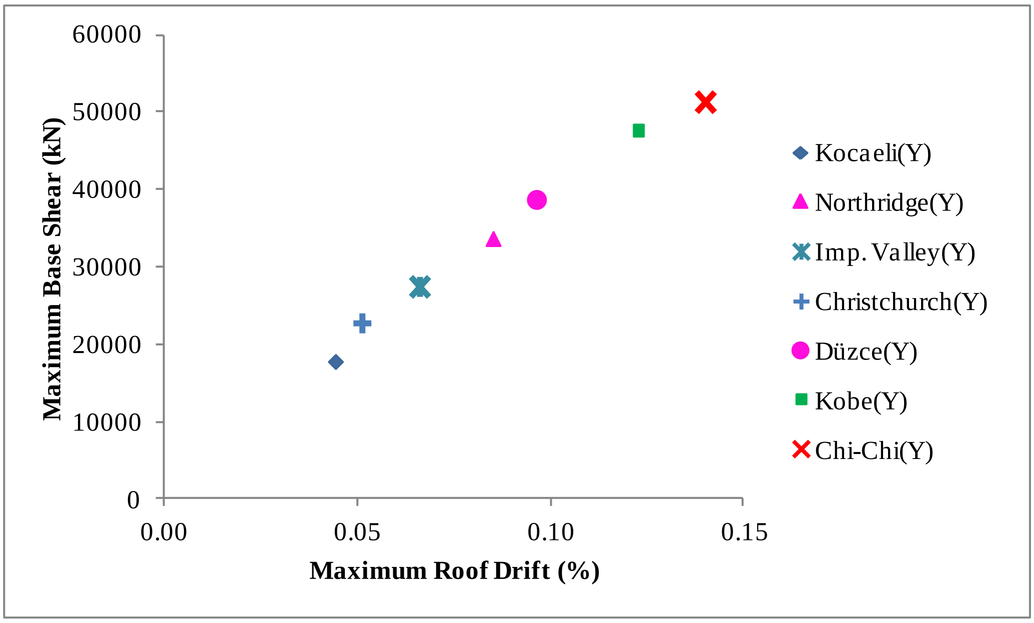
A.16 Maximum Base Shear vs. Maximum Roof Drift for the Designed Building with 0.50% Shear Wall Ratio for the Second Case in the X and Y-Directions



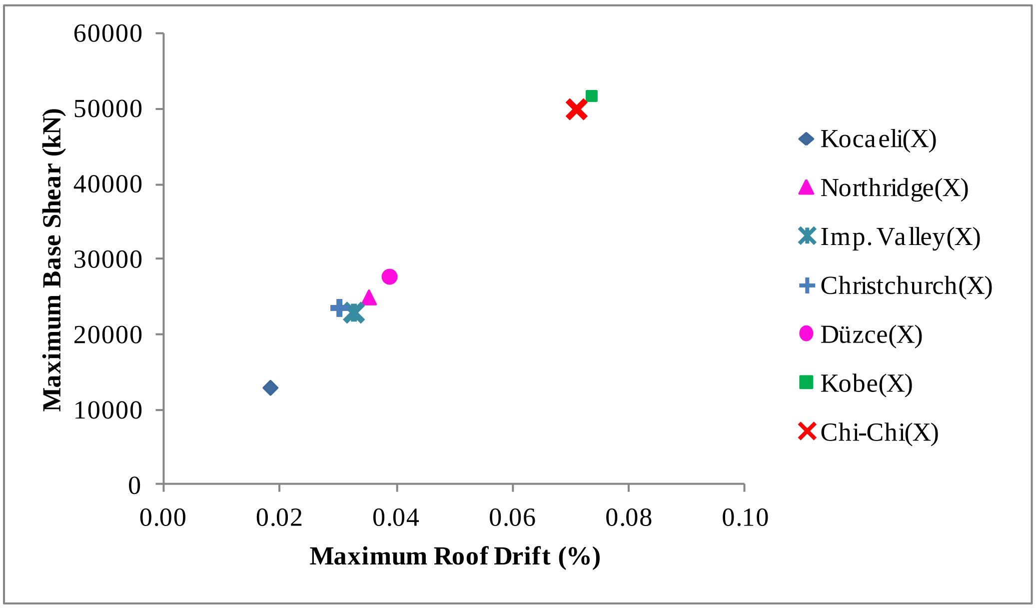


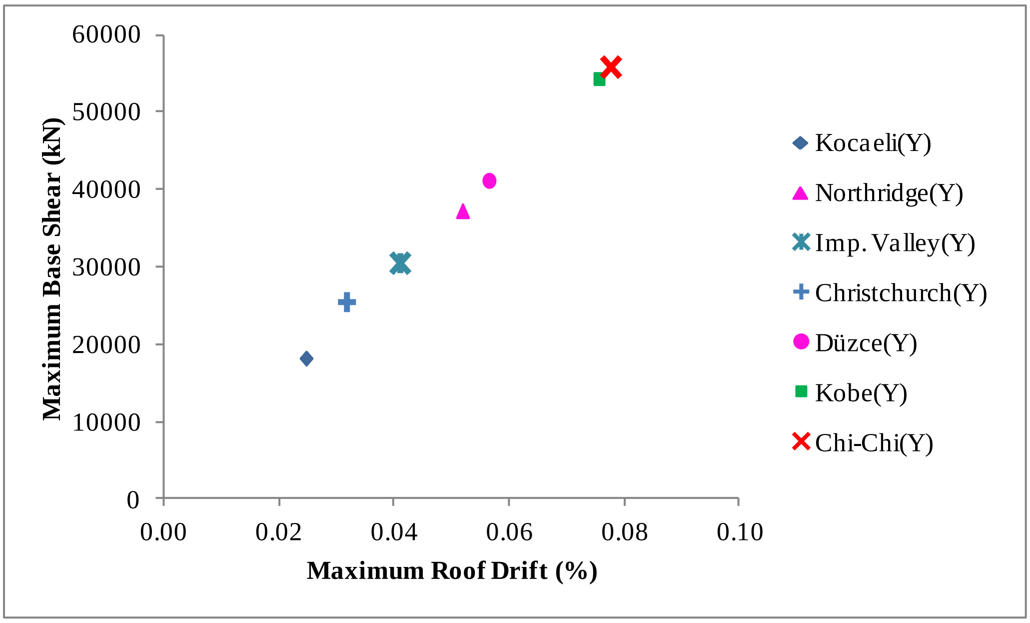
A.17 Maximum Base Shear vs. Maximum Roof Drift for the Designed Building with 1.50% Shear Wall Ratio for the Second Case in the X and Y-Directions



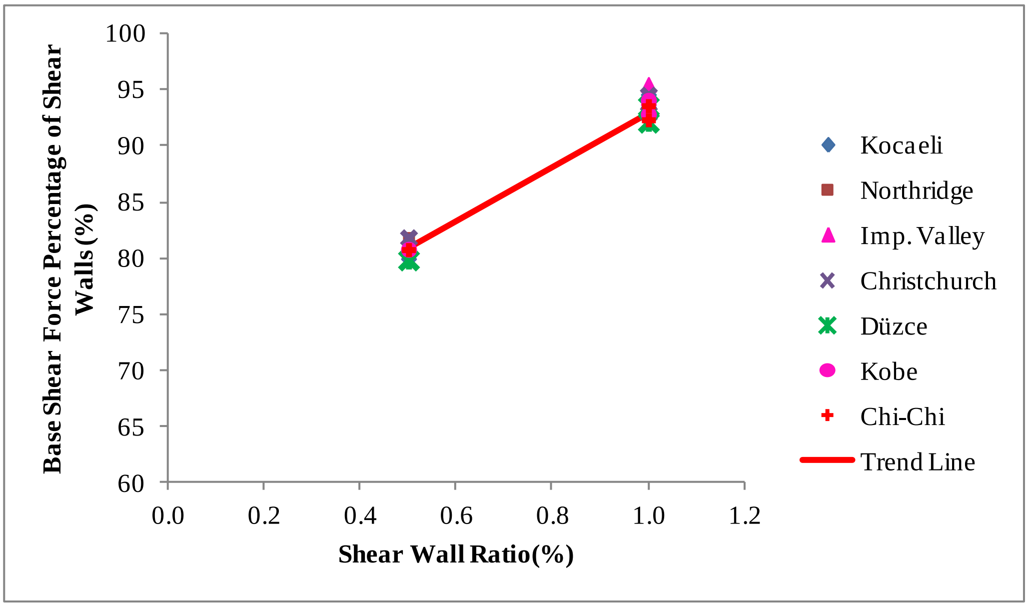


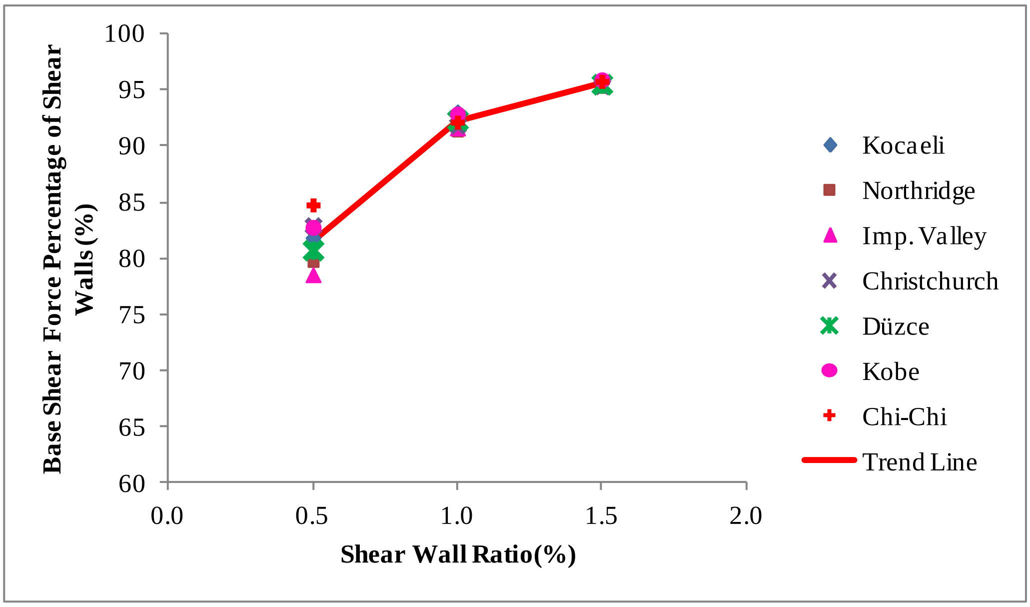
A.18 Maximum Base Shear vs. Maximum Roof Drift for the Designed Building with 2.00% Shear Wall Ratio for the Second Case in the X and Y-Directions



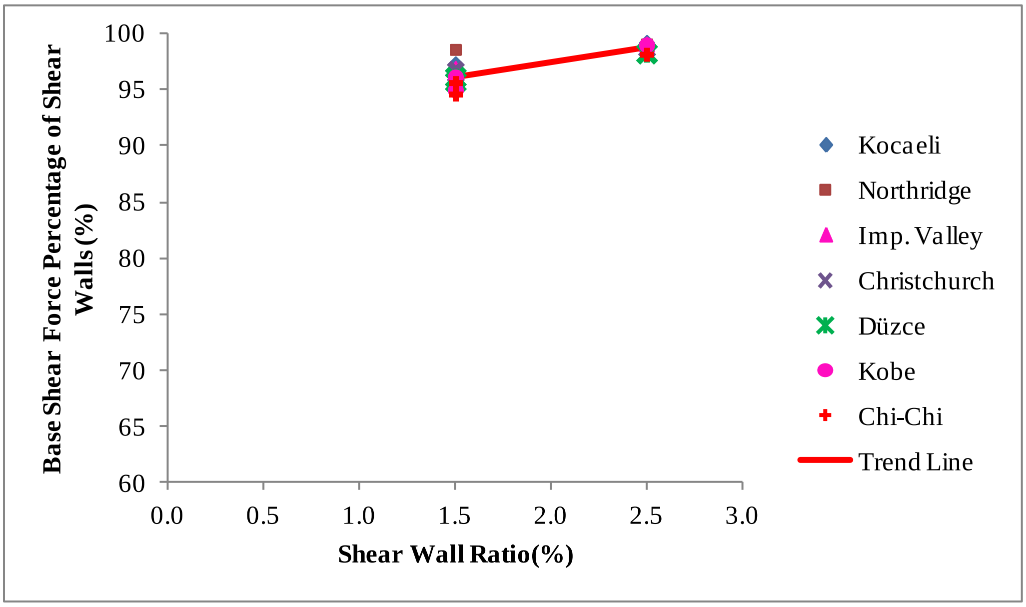


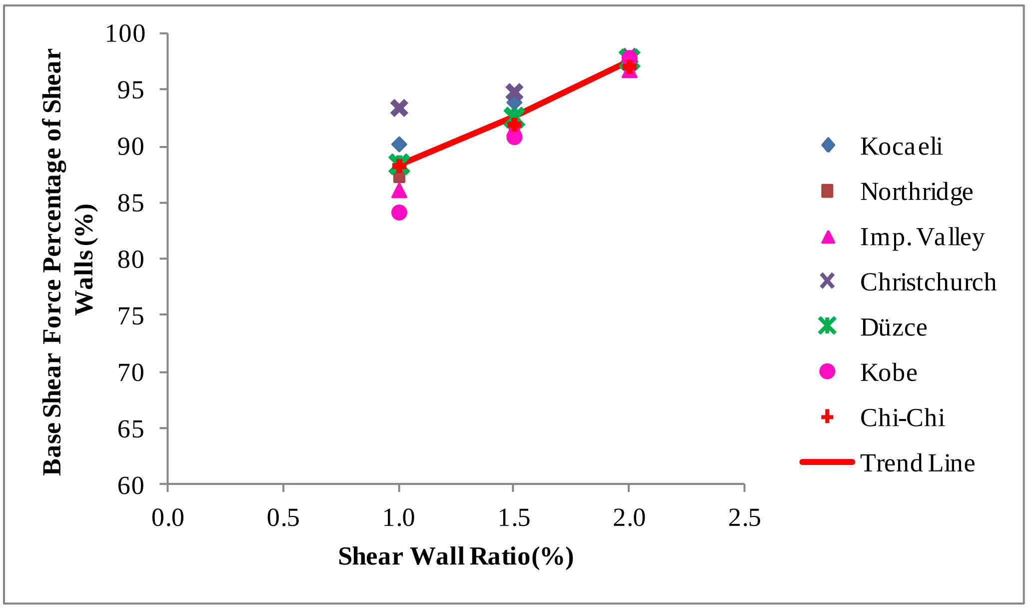
A.19 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio for Eminönü Çemberlitaş Anatolian High School Block A in the X and Y-Directions



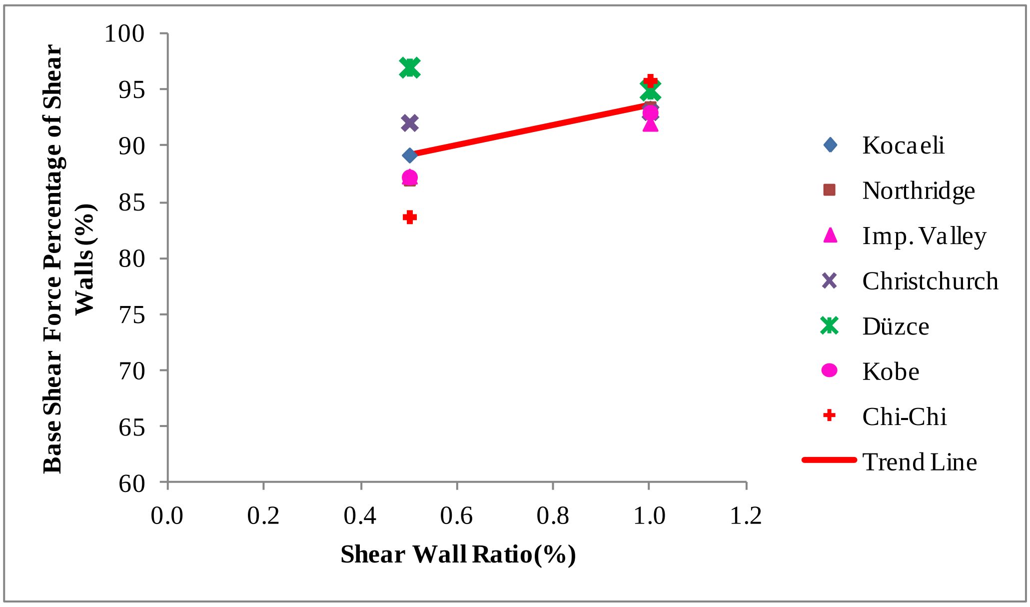


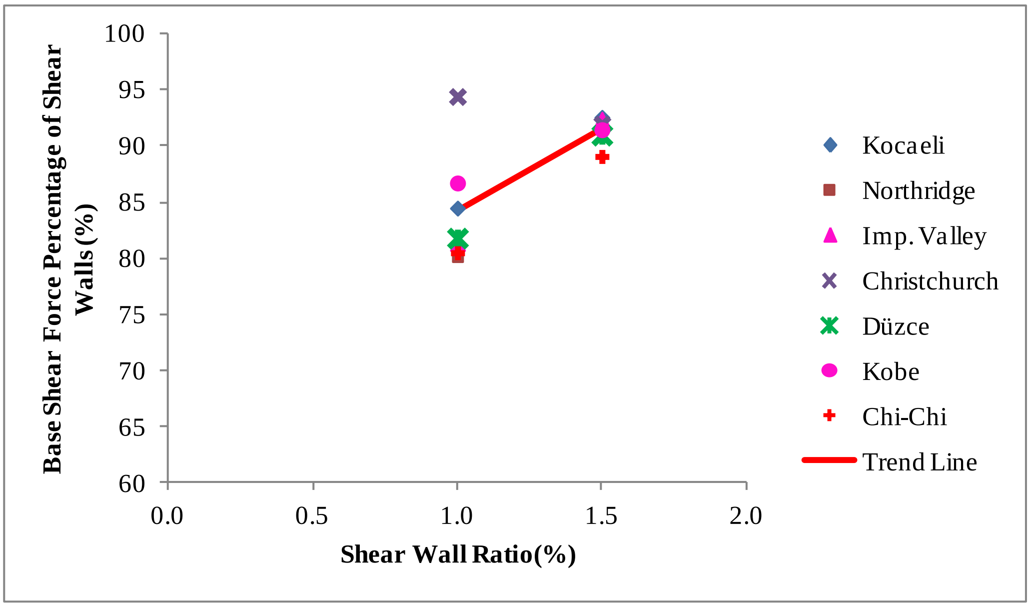
A.20 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio for Güngören Haznedar Abdi İpekçi Primary School Block B in the X and Y-Directions



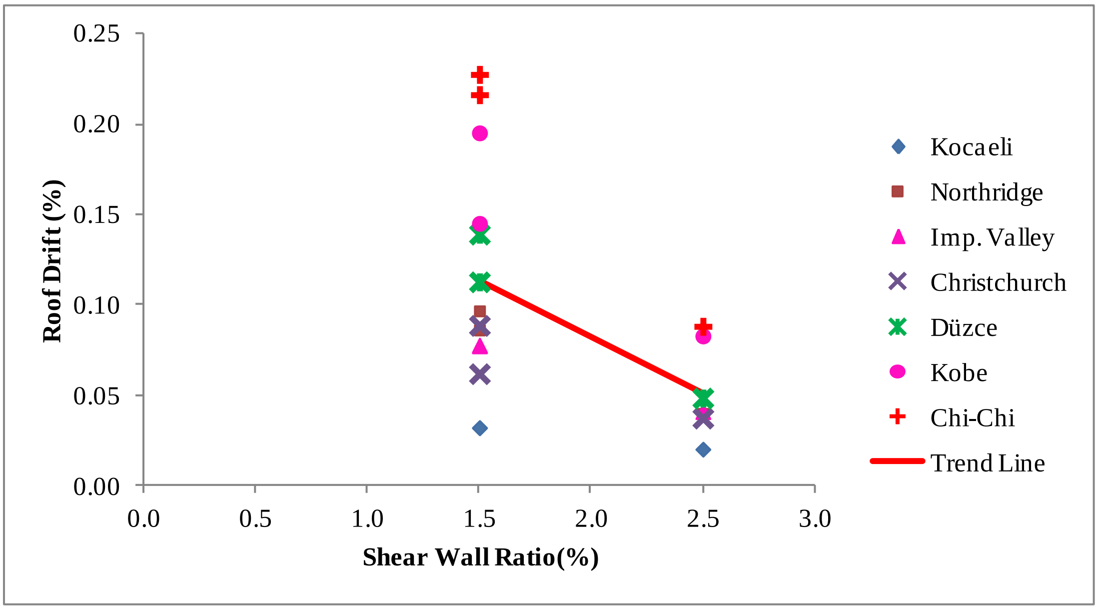


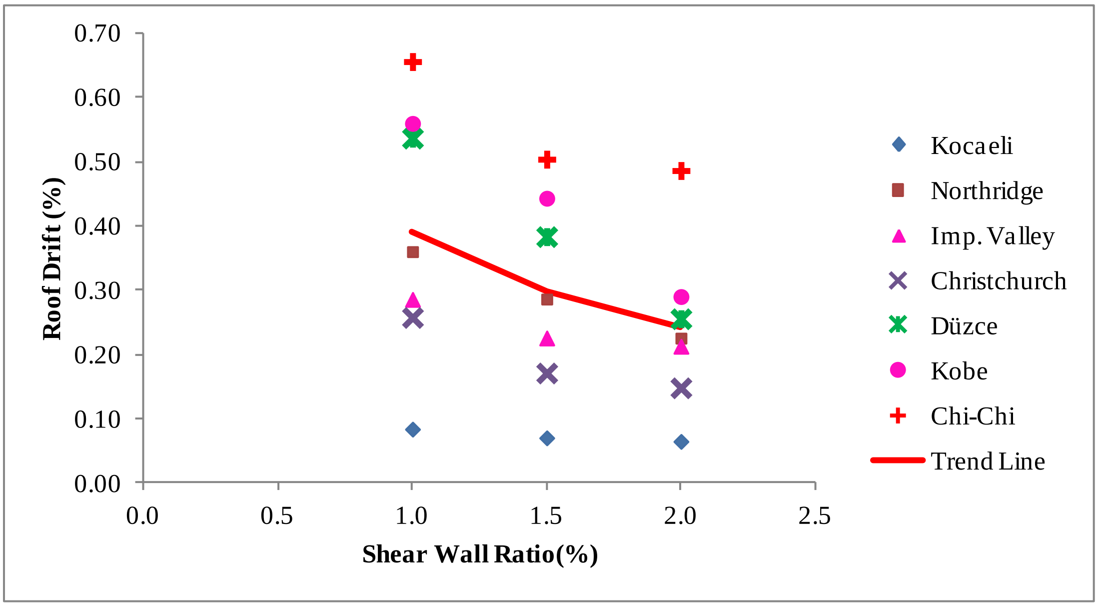
A.21 Base Shear Percentage Carried by Shear Walls vs. Shear Wall Ratio for Sarıyer MEV Dumlupınar Primary School in the X and Y-Directions



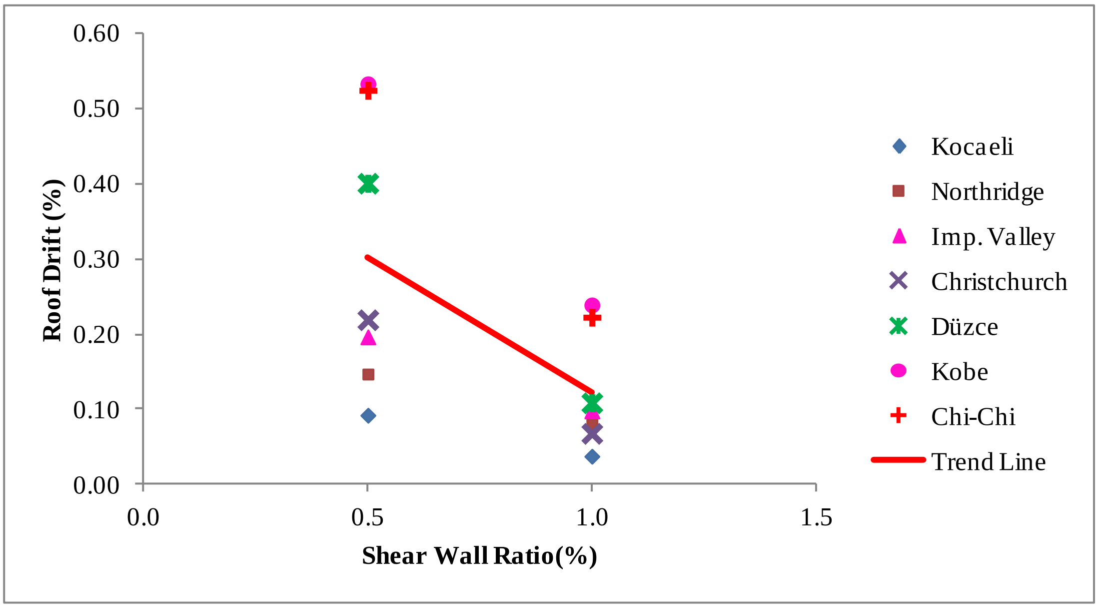


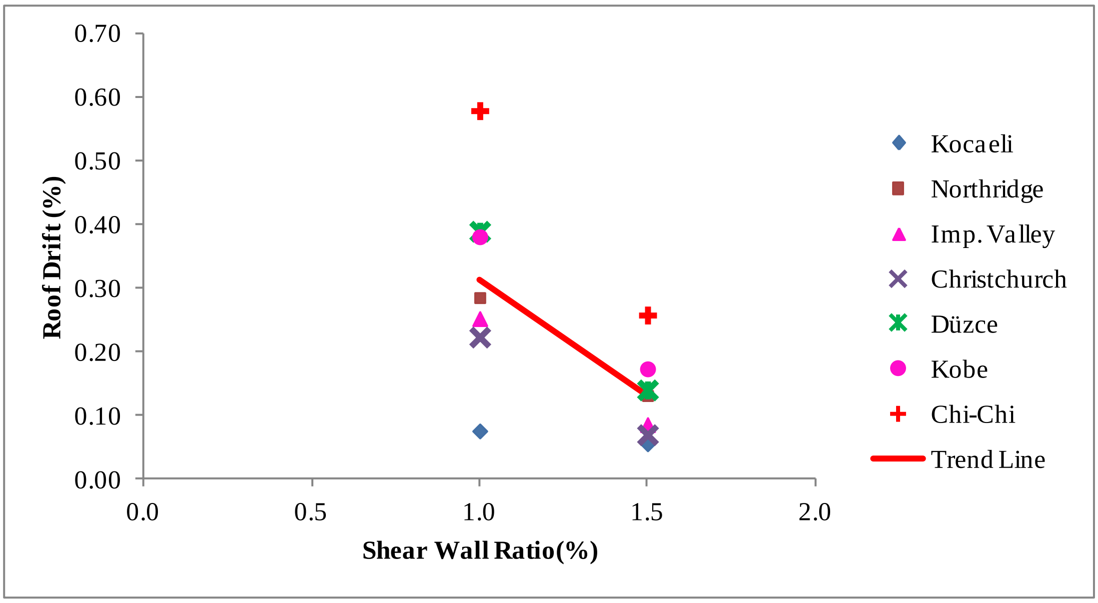
A.22 Roof Drift vs. Shear Wall Ratio for Güngören Haznedar Abdi İpekçi Primary School Block B in the X and Y-Directions



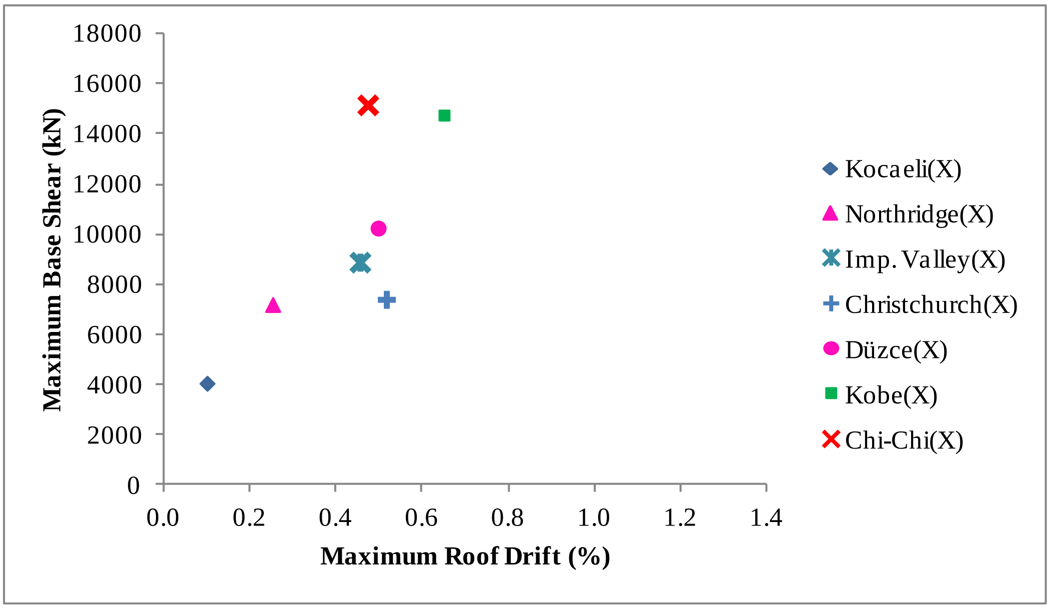


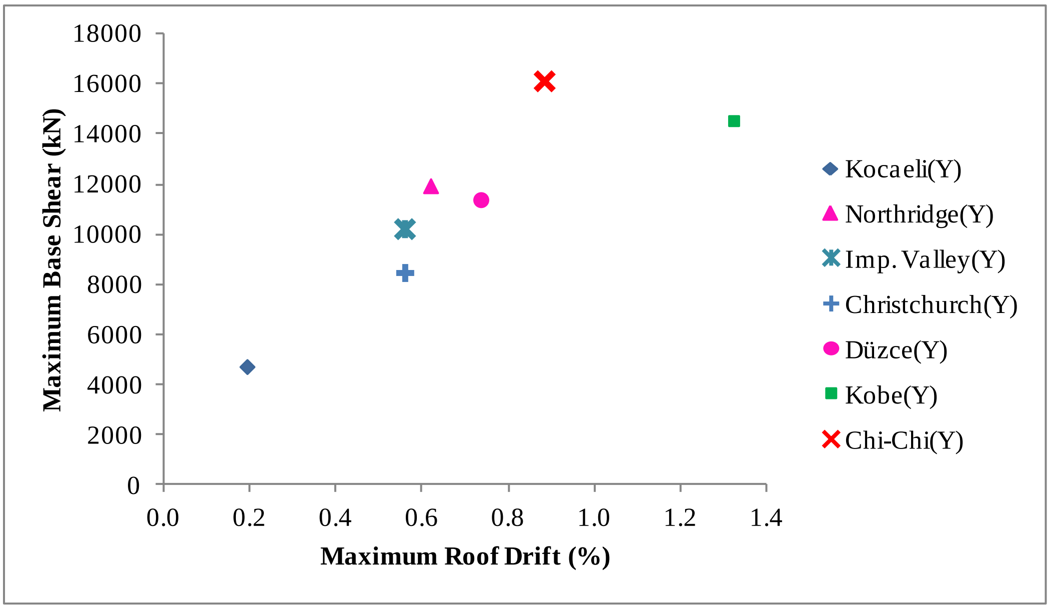
A.23 Roof Drift vs. Shear Wall Ratio for Sarıyer MEV Dumlupınar Primary School in the X and Y-Directions



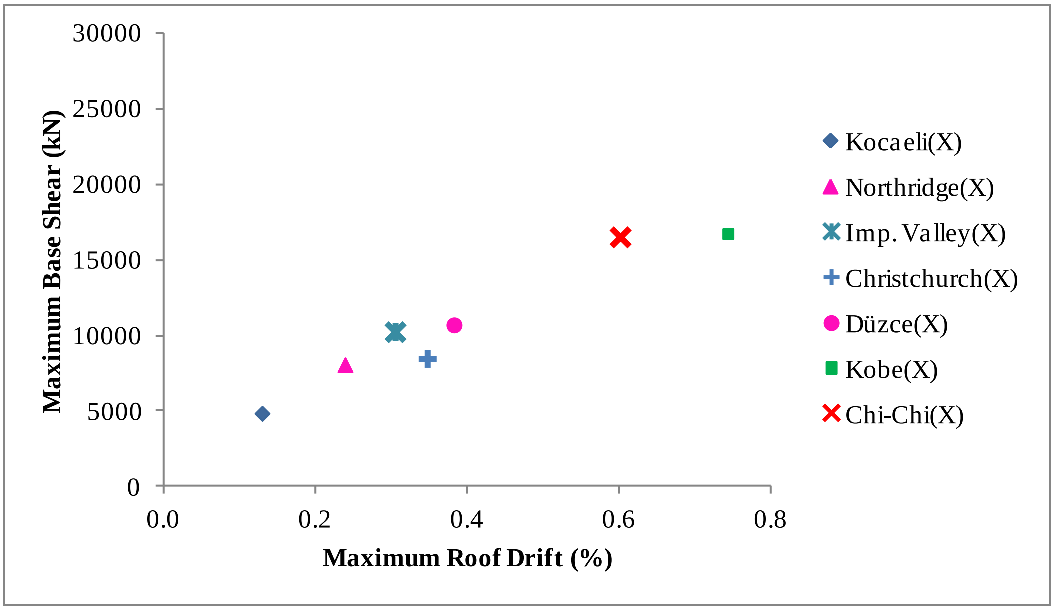


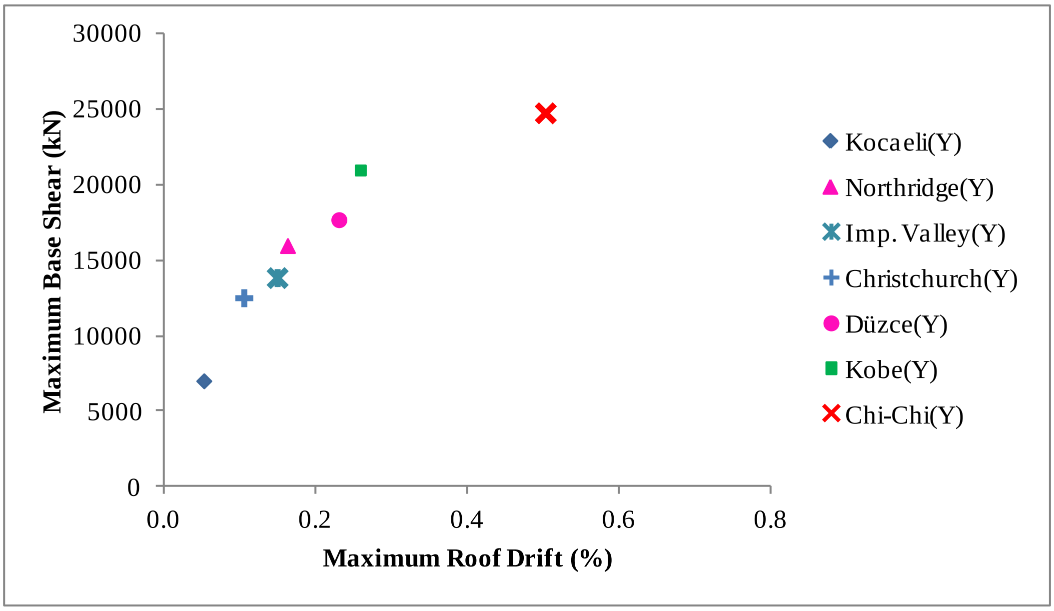
A.24 Maximum Base Shear vs. Maximum Roof Drift for Fatih Gazi Primary School with 0.50-0.00% Shear Wall Ratio in the X and Y-Directions



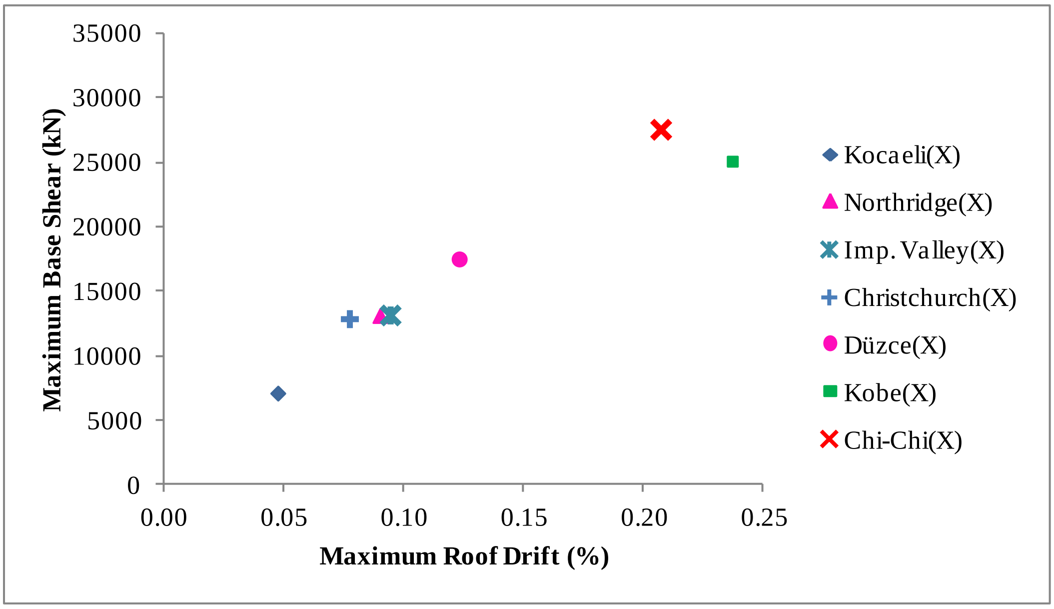


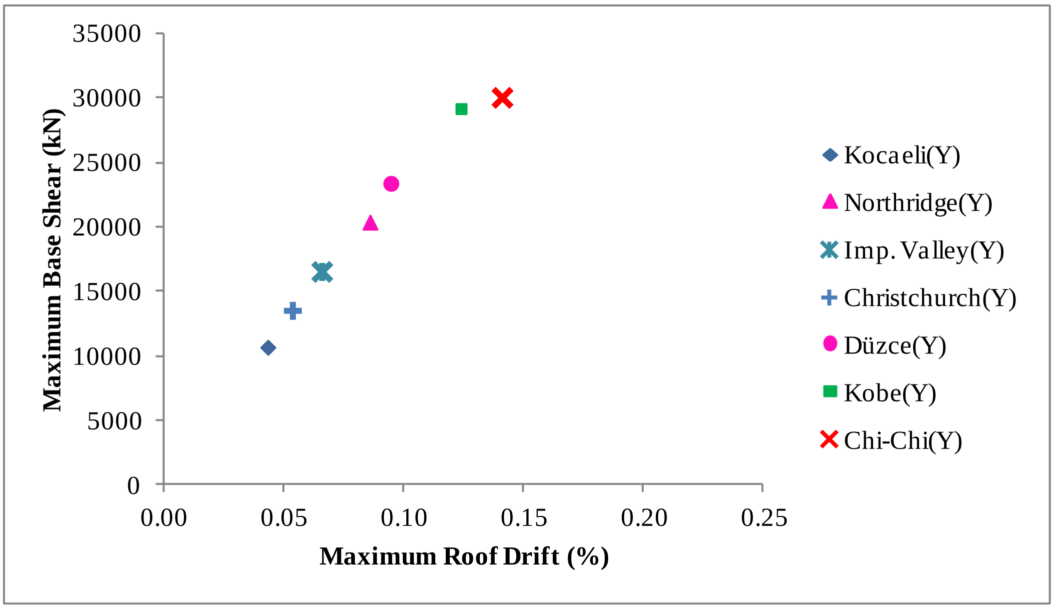
A.25 Maximum Base Shear vs. Maximum Roof Drift for Fatih Gazi Primary School with 0.50-0.50% Shear Wall Ratio in the X and Y-Directions



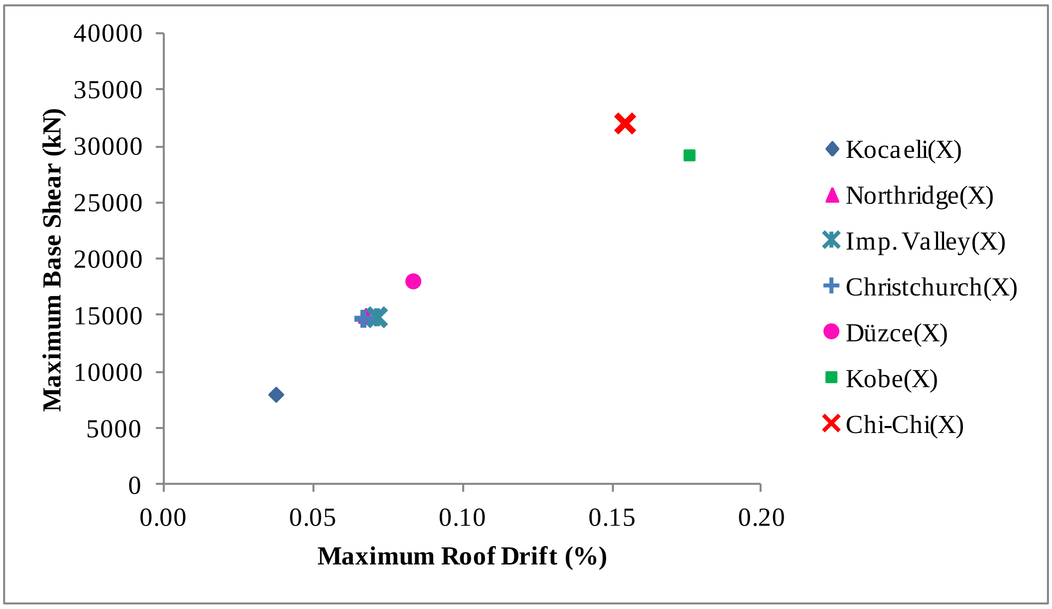


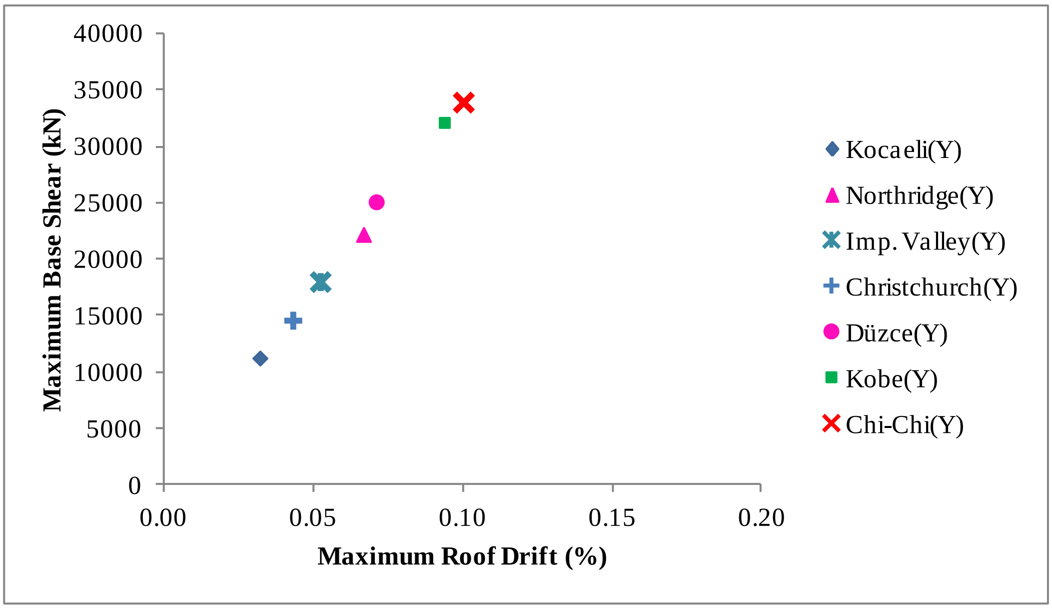
A.26 Maximum Base Shear vs. Maximum Roof Drift for Fatih Gazi Primary School with 1.00-1.00% Shear Wall Ratio in the X and Y-Directions



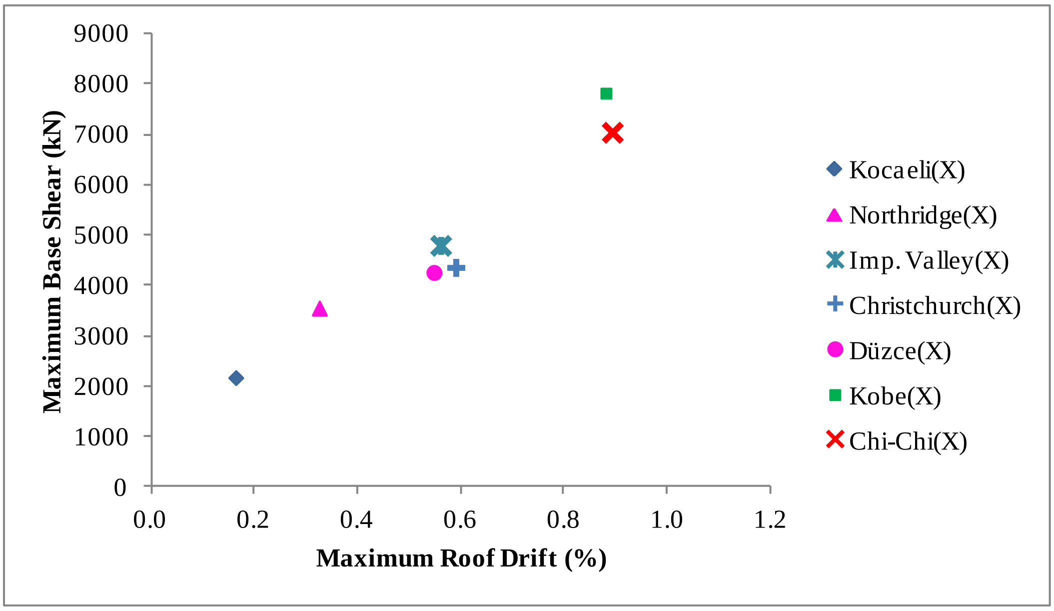


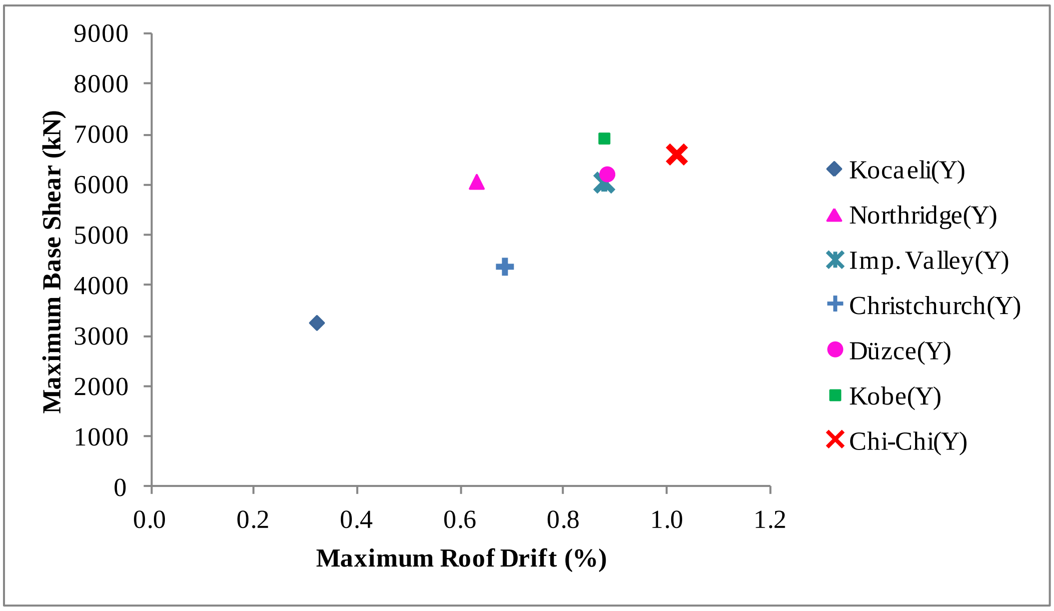
A.27 Maximum Base Shear vs. Maximum Roof Drift for Fatih Gazi Primary School with 1.50-1.50% Shear Wall Ratio in the X and Y-Directions



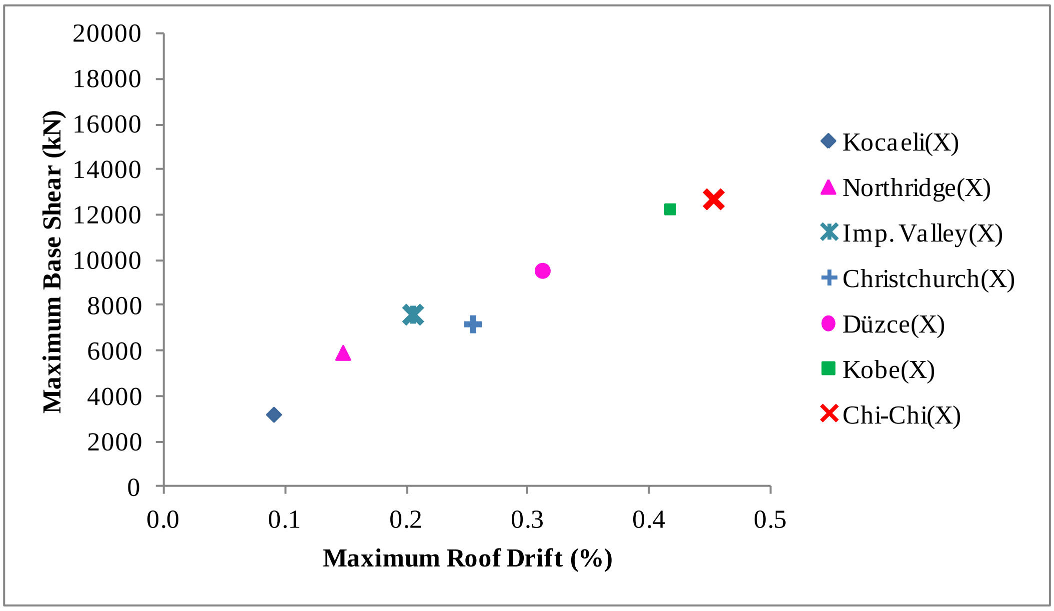


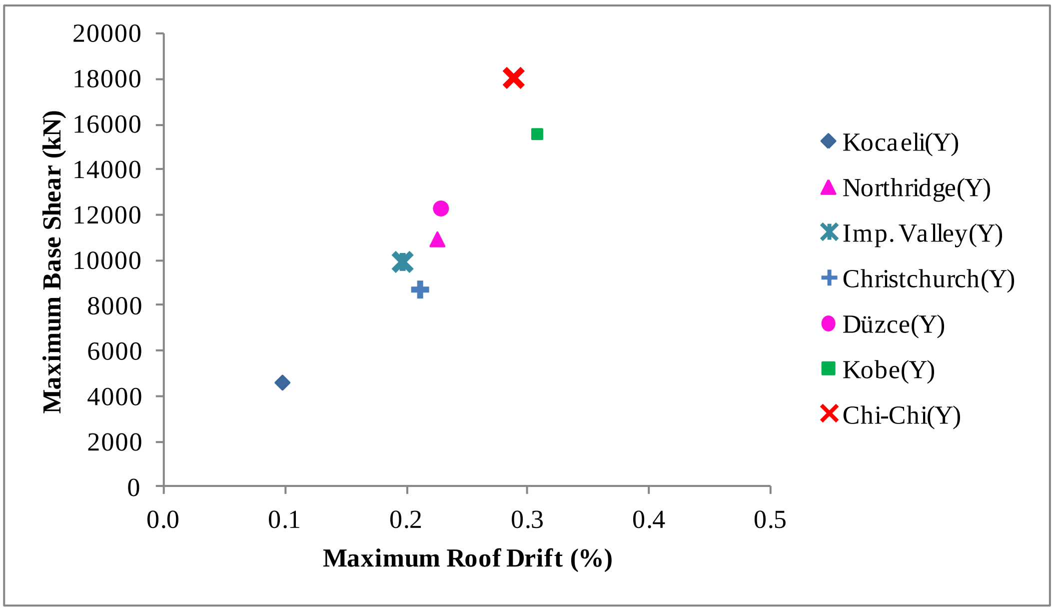
A.28 Maximum Base Shear vs. Maximum Roof Drift for Eminönü Çemberlitaş Anatolian High School Block A with no Shear Wall in the X and Y-Directions



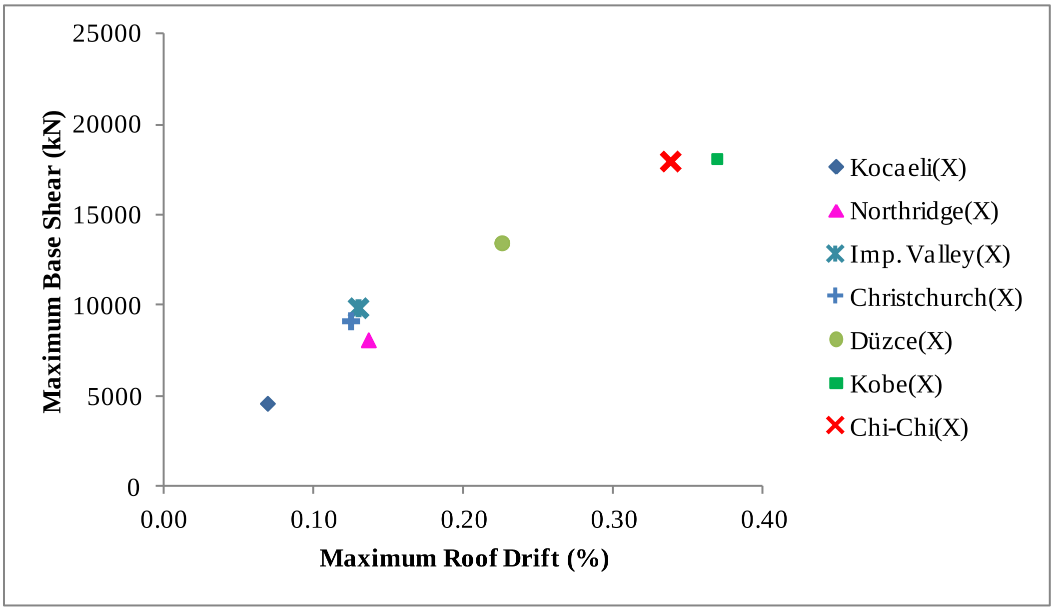


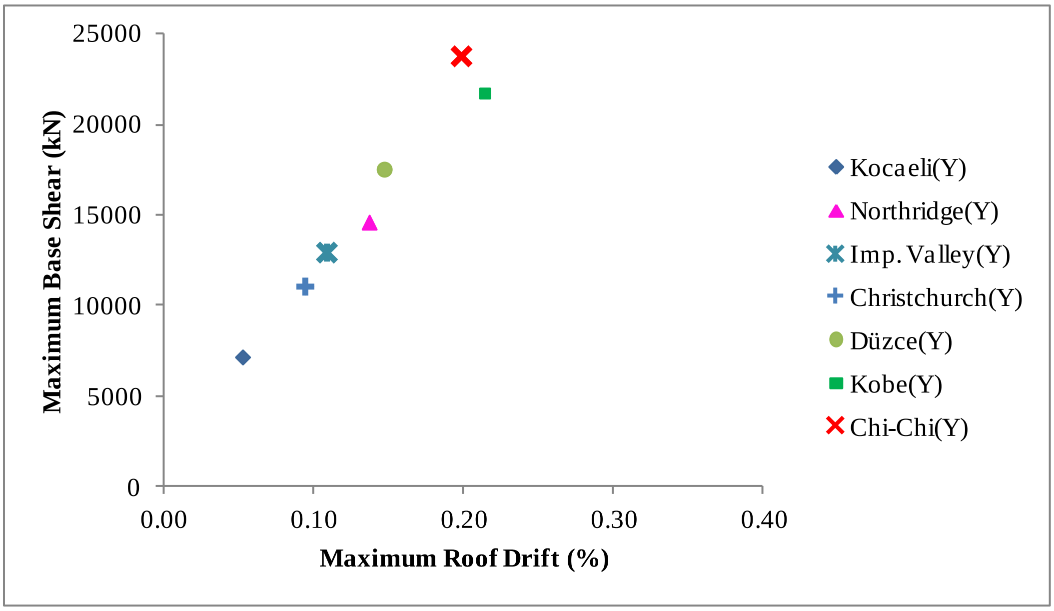
A.29 Maximum Base Shear vs. Maximum Roof Drift for Eminönü Çemberlitaş Anatolian High School Block A with 0.50-0.50% Shear Wall Ratio in the X and Y-Directions



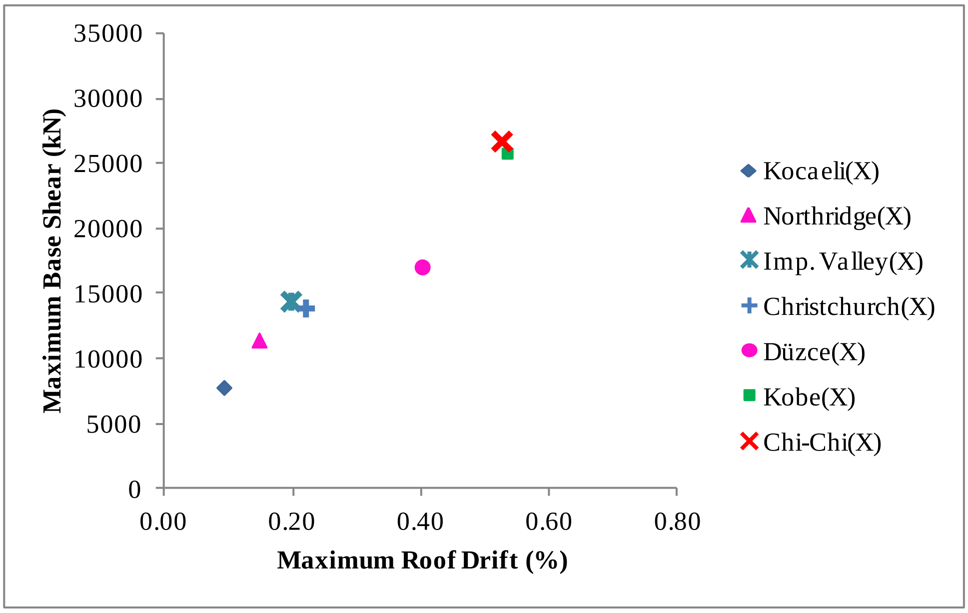


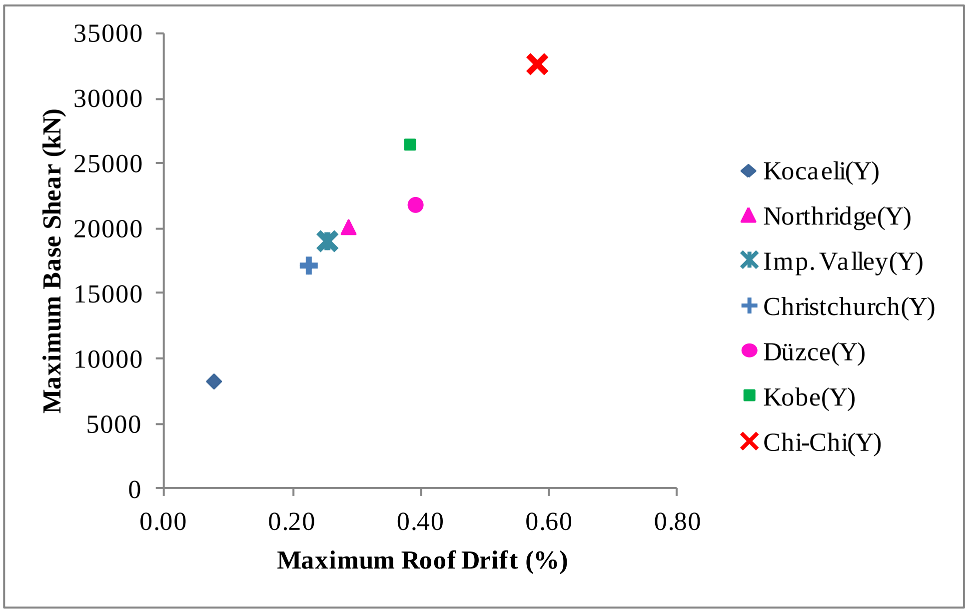
A.30 Maximum Base Shear vs. Maximum Roof Drift for Eminönü Çemberlitaş Anatolian High School Block A with 1.00-1.50% Shear Wall Ratio in the X and Y-Directions



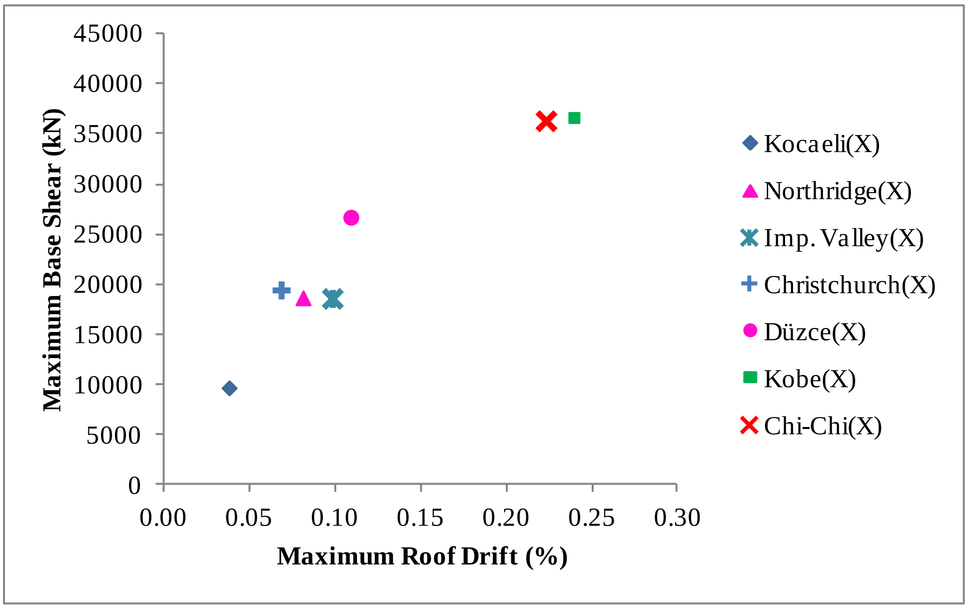


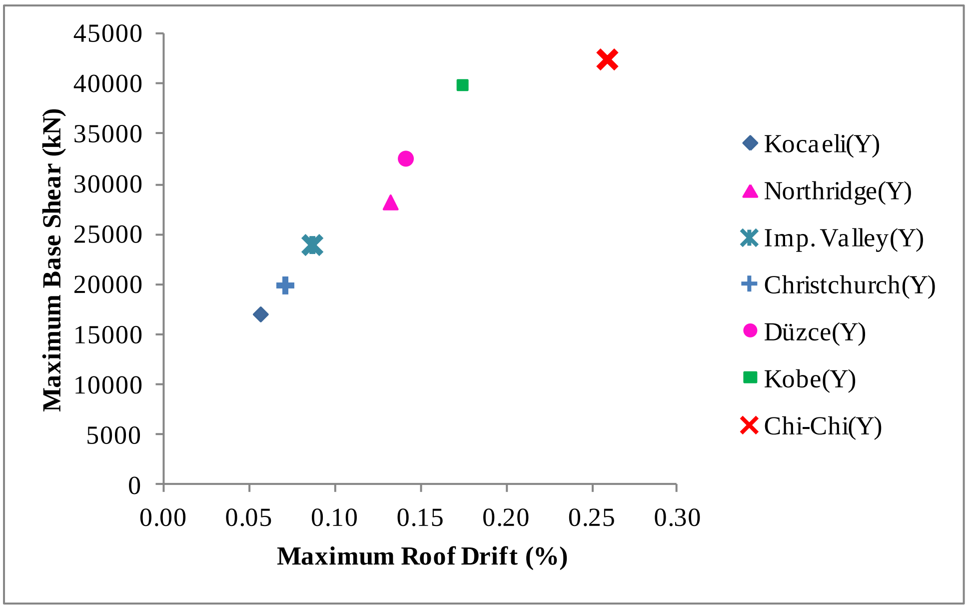
A.31 Maximum Base Shear vs. Maximum Roof Drift for Sarıyer MEV Dumlupınar Primary School with 0.50-1.00% Shear Wall Ratio in the X and Y-Directions



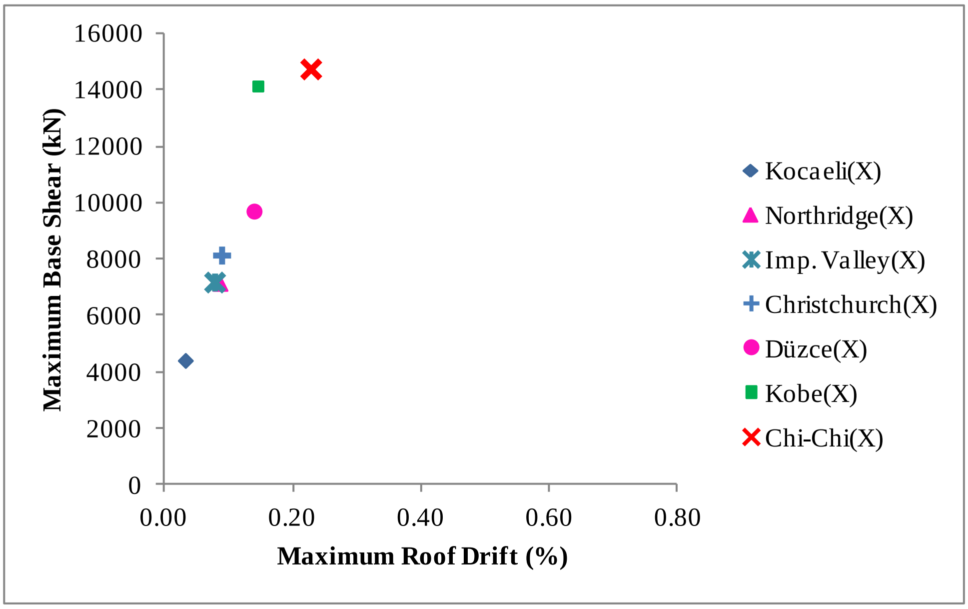


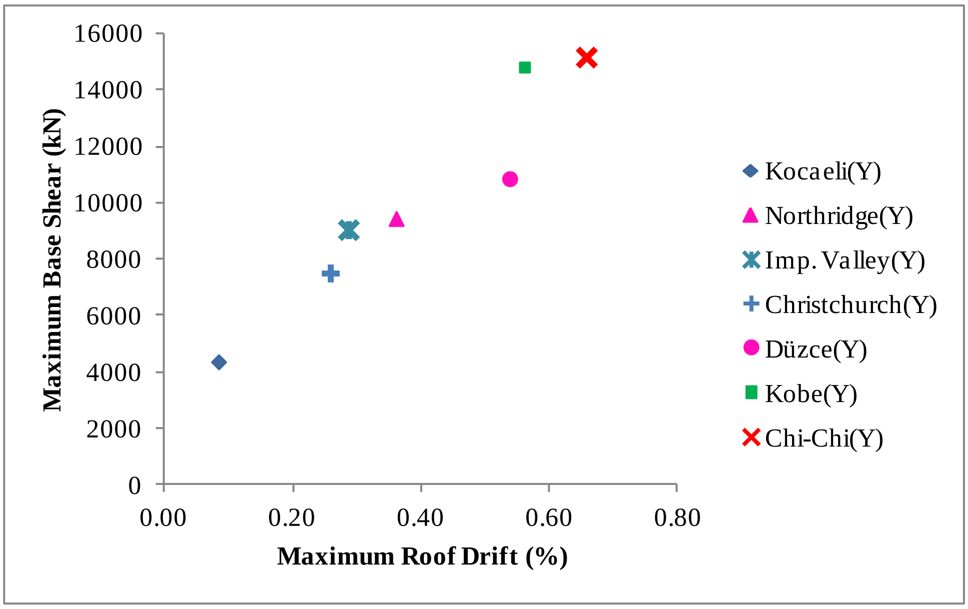
A.32 Maximum Base Shear vs. Maximum Roof Drift for Sarıyer MEV Dumlupınar Primary School with 1.00-1.50% Shear Wall Ratio in the X and Y-Directions



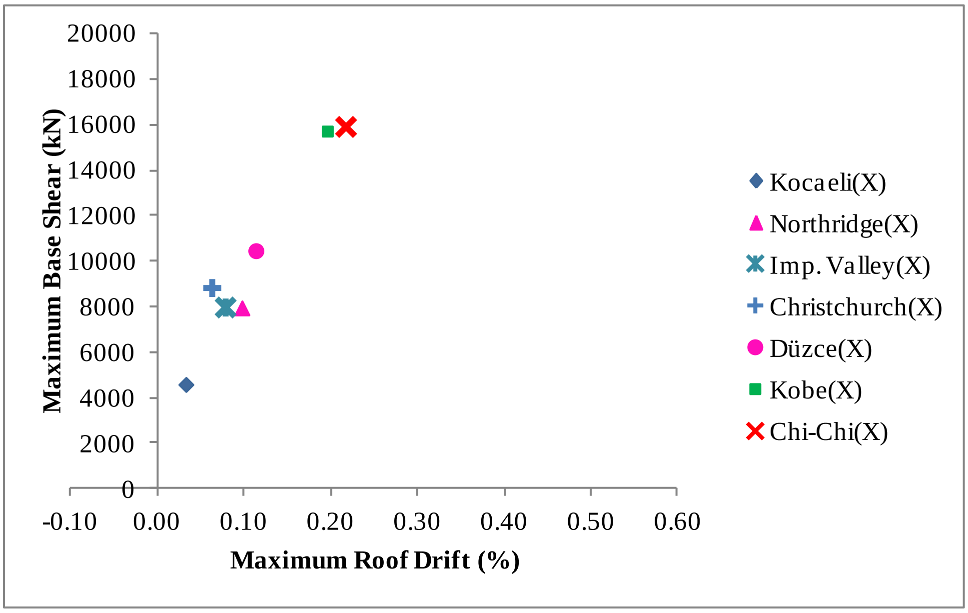


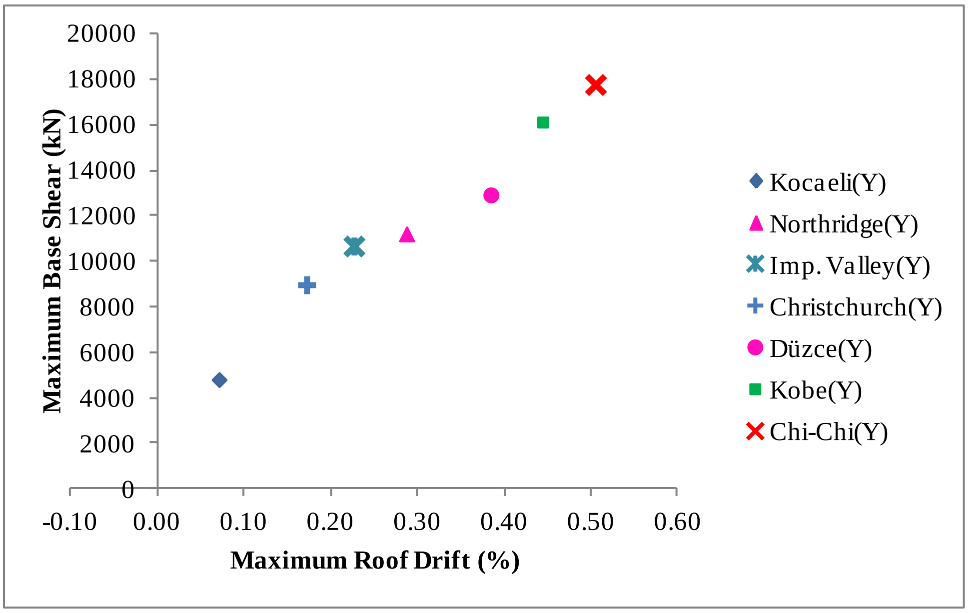
A.33 Maximum Base Shear vs. Maximum Roof Drift for Güngören Haznedar Abdi İpekçi Primary School Block B with 1.50-1.00% Shear Wall Ratio in the X and Y-Directions



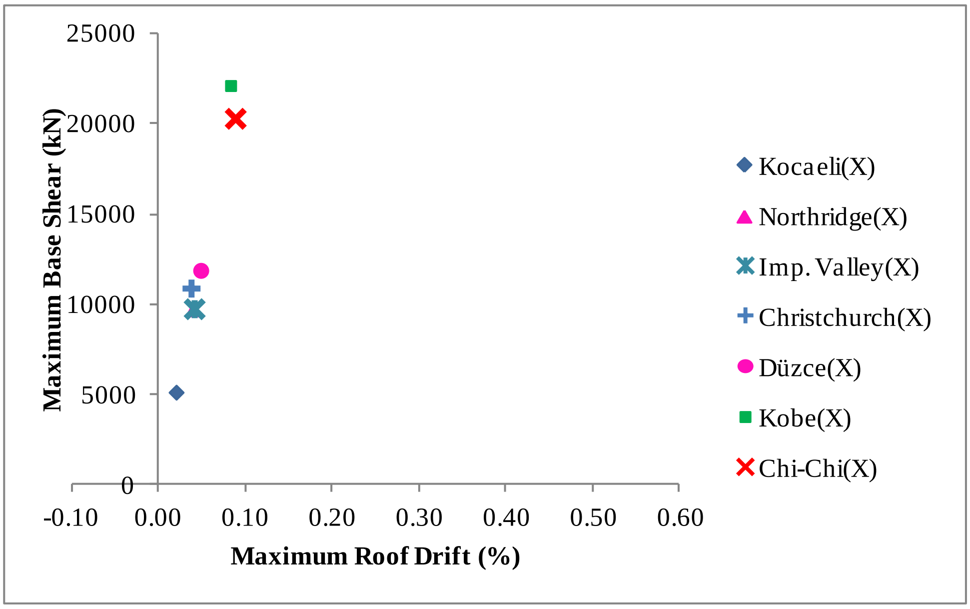


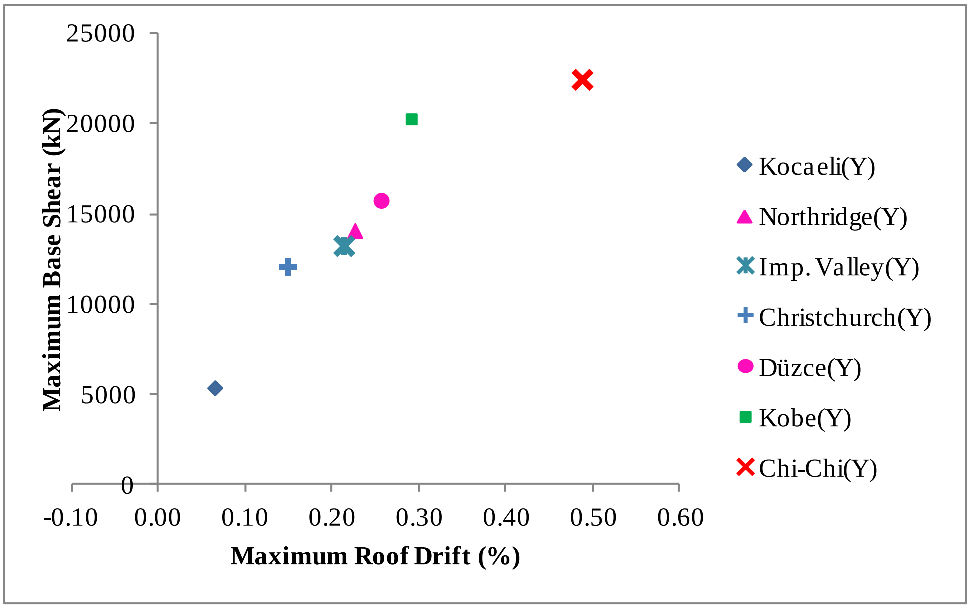
A.34 Maximum Base Shear vs. Maximum Roof Drift for Güngören Haznedar Abdi İpekçi Primary School Block B with 1.50-1.50% Shear Wall Ratio in the X and Y-Directions



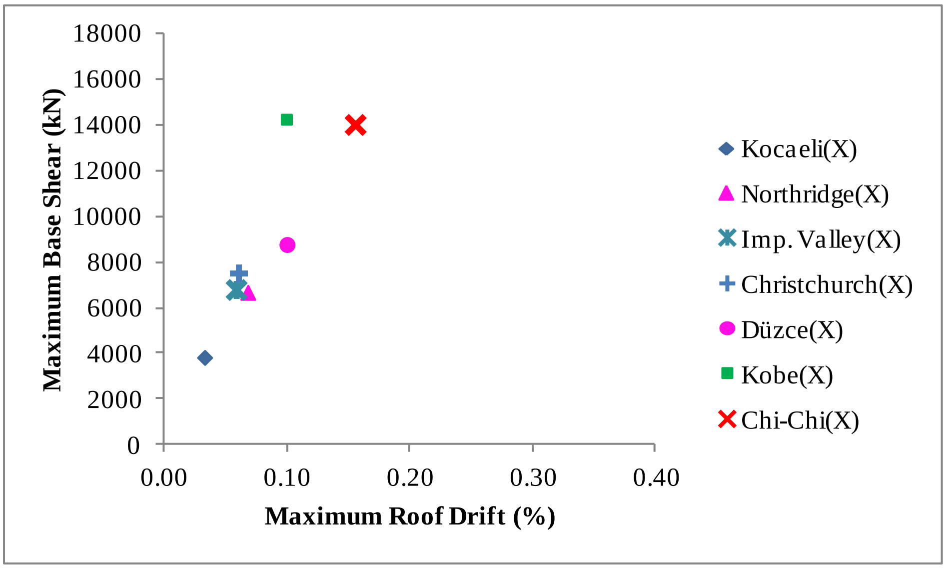


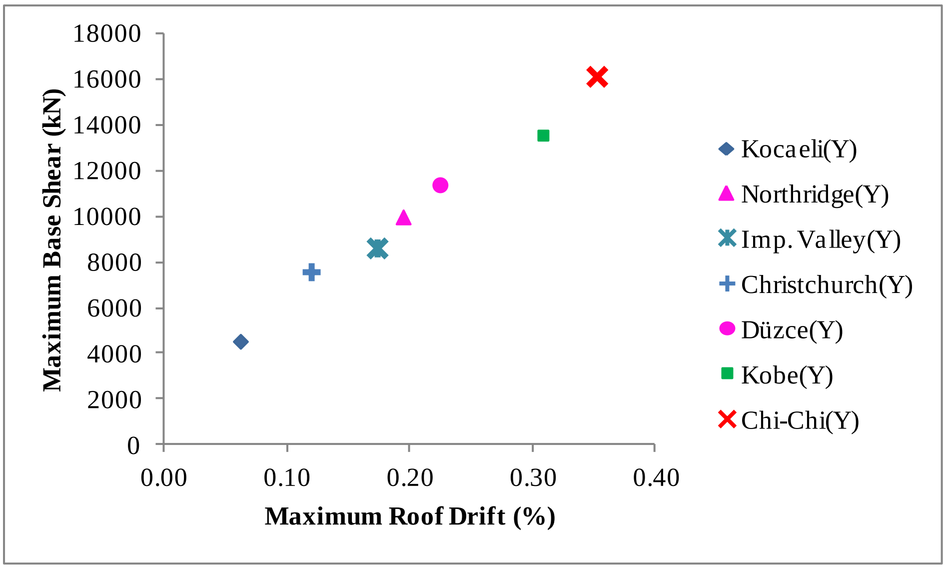
A.35 Maximum Base Shear vs. Maximum Roof Drift for Güngören Haznedar Abdi İpekçi Primary School Block B with 2.50-2.00% Shear Wall Ratio in the X and Y-Directions





A.36 Maximum Base Shear vs. Maximum Roof Drift for G.O.P. Ülkü Primary School Block B with 1.50-1.00% Shear Wall Ratio in the X and Y-Directions





A.37 Percentage of Yielded Members for Güngören Haznedar Abdi İpekçi Primary School Block B in the X-Direction

X-Direction			Percentage of Yielded Members (%)			
Record	Shear Wall Percentage (%)	Member Type	Story 1	Story 2	Story 3	Story 4
Düzce	1.5-1.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Columns	66.7	72.7	72.7	72.7
	2.5-2.0 (Retrofitted B.)	Columns	16.7	20.0	20.0	20.0
	1.5-1.0 (Existing B.)	Beams	87.5	100.0	100.0	91.7
	1.5-1.5 (Generated B.)	Beams	87.5	100.0	100.0	91.7
	2.5-2.0 (Retrofitted B.)	Beams	87.5	100.0	100.0	91.7
	1.5-1.0 (Existing B.)	Shear Walls	100.0	100.0	0.0	0.0
	1.5-1.5 (Generated B.)	Shear Walls	100.0	100.0	0.0	0.0
	2.5-2.0 (Retrofitted B.)	Shear Walls	50.0	50.0	0.0	0.0
Kobe	1.5-1.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Columns	66.7	72.7	72.7	72.7
	2.5-2.0 (Retrofitted B.)	Columns	16.7	20.0	20.0	20.0
	1.5-1.0 (Existing B.)	Beams	93.8	100.0	100.0	91.7
	1.5-1.5 (Generated B.)	Beams	93.8	100.0	100.0	91.7
	2.5-2.0 (Retrofitted B.)	Beams	92.3	100.0	100.0	91.7
	1.5-1.0 (Existing B.)	Shear Walls	100.0	100.0	100.0	0.0
	1.5-1.5 (Generated B.)	Shear Walls	100.0	100.0	50.0	0.0
	2.5-2.0 (Retrofitted B.)	Shear Walls	75.0	50.0	0.0	0.0
Chi-Chi	1.5-1.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Columns	75.0	72.7	72.7	72.7
	2.5-2.0 (Retrofitted B.)	Columns	33.3	40.0	40.0	40.0
	1.5-1.0 (Existing B.)	Beams	100.0	100.0	100.0	91.7
	1.5-1.5 (Generated B.)	Beams	93.8	100.0	100.0	91.7
	2.5-2.0 (Retrofitted B.)	Beams	93.8	100.0	100.0	91.7
	1.5-1.0 (Existing B.)	Shear Walls	100.0	100.0	50.0	0.0
	1.5-1.5 (Generated B.)	Shear Walls	100.0	100.0	50.0	0.0
	2.5-2.0 (Retrofitted B.)	Shear Walls	50.0	50.0	0.0	0.0

A.38 Percentage of Yielded Members for Güngören Haznedar Abdi İpekçi Primary School Block B in the Y-Direction

Y-Direction			Percentage of Yielded Members (%)			
Record	Shear Wall Percentage (%)	Member Type	Story 1	Story 2	Story 3	Story 4
Düzce	1.5-1.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Columns	66.7	72.7	72.7	72.7
	2.5-2.0 (Retrofitted B.)	Columns	16.7	20.0	20.0	20.0
	1.5-1.0 (Existing B.)	Beams	87.5	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Beams	86.7	100.0	100.0	100.0
	2.5-2.0 (Retrofitted B.)	Beams	84.6	100.0	100.0	100.0
	1.5-1.0 (Existing B.)	Shear Walls	100.0	100.0	50.0	0.0
	1.5-1.5 (Generated B.)	Shear Walls	66.7	66.7	33.3	0.0
	2.5-2.0 (Retrofitted B.)	Shear Walls	60.0	40.0	0.0	0.0
Kobe	1.5-1.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Columns	66.7	72.7	72.7	72.7
	2.5-2.0 (Retrofitted B.)	Columns	16.7	20.0	20.0	20.0
	1.5-1.0 (Existing B.)	Beams	87.5	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Beams	87.5	100.0	100.0	100.0
	2.5-2.0 (Retrofitted B.)	Beams	84.6	100.0	100.0	100.0
	1.5-1.0 (Existing B.)	Shear Walls	100.0	100.0	50.0	50.0
	1.5-1.5 (Generated B.)	Shear Walls	100.0	66.7	0.0	0.0
	2.5-2.0 (Retrofitted B.)	Shear Walls	60.0	20.0	0.0	0.0
Chi-Chi	1.5-1.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Columns	75.0	72.7	72.7	72.7
	2.5-2.0 (Retrofitted B.)	Columns	33.3	40.0	40.0	40.0
	1.5-1.0 (Existing B.)	Beams	100.0	100.0	100.0	100.0
	1.5-1.5 (Generated B.)	Beams	100.0	100.0	100.0	100.0
	2.5-2.0 (Retrofitted B.)	Beams	100.0	100.0	100.0	100.0
	1.5-1.0 (Existing B.)	Shear Walls	100.0	100.0	50.0	0.0
	1.5-1.5 (Generated B.)	Shear Walls	100.0	66.7	33.3	0.0
	2.5-2.0 (Retrofitted B.)	Shear Walls	80.0	60.0	0.0	0.0

A.39 Percentage of Yielded Members for Eminönü Çemberlitaş Anatolian High School Block A in the X-Direction

X-Direction			Percentage of Yielded Members (%)				
Record	**Shear Wall Percentage (%)**	**Member Type**	**Story 1**	**Story 2**	**Story 3**	**Story 4**	**Story 5**
Düzce	0.0-0.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	Columns	90.0	90.0	90.0	90.0	90.0
	0.0-0.0 (Existing B.)	Beams	100.0	98.1	98.1	98.1	92.5
	0.5-0.5 (Generated B.)	Beams	93.6	100.0	100.0	97.9	97.9
	1.0-1.0 (Generated B.)	Beams	95.5	95.5	95.5	95.5	90.9
	1.0-1.5 (Retrofitted B.)	Beams	88.6	93.2	93.2	88.6	90.9
	0.0-0.0 (Existing B.)	Shear Walls	-	-	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	50.0	0.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	66.7	66.7	0.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	Shear Walls	66.7	33.3	0.0	0.0	0.0
Kobe	0.0-0.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.0-0.0 (Existing B.)	Beams	98.1	98.1	96.2	96.2	90.6
	0.5-0.5 (Generated B.)	Beams	93.6	95.7	95.7	95.7	91.5
	1.0-1.0 (Generated B.)	Beams	95.5	95.5	95.5	95.5	93.2
	1.0-1.5 (Retrofitted B.)	Beams	95.5	95.5	95.5	95.5	95.5
	0.0-0.0 (Existing B.)	Shear Walls	-	-	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	50.0	0.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	100.0	66.7	33.3	0.0	0.0
	1.0-1.5 (Retrofitted B.)	Shear Walls	100.0	66.7	33.3	0.0	0.0
Chi-Chi	0.0-0.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.0-0.0 (Existing B.)	Beams	96.2	98.1	98.1	96.2	90.6
	0.5-0.5 (Generated B.)	Beams	93.6	95.7	95.7	95.7	91.5
	1.0-1.0 (Generated B.)	Beams	95.5	95.5	95.5	95.5	88.6
	1.0-1.5 (Retrofitted B.)	Beams	95.5	95.5	95.5	95.5	95.5
	0.0-0.0 (Existing B.)	Shear Walls	-	-	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	50.0	50.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	66.7	33.3	0.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	Shear Walls	66.7	33.3	0.0	0.0	0.0

A.40 Percentage of Yielded Members for Eminönü Çemberlitaş Anatolian High School Block A in the Y-Direction

Y-Direction			Percentage of Yielded Members (%)				
Record	Shear Wall Percentage (%)	Member Type	Story 1	Story 2	Story 3	Story 4	Story 5
Düzce	0.0-0.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	Columns	90.0	90.0	90.0	90.0	90.0
	0.0-0.0 (Existing B.)	Beams	100.0	100.0	100.0	84.6	64.1
	0.5-0.5 (Generated B.)	Beams	94.7	89.5	86.8	92.1	89.5
	1.0-1.0 (Generated B.)	Beams	74.3	69.4	69.4	66.7	52.8
	1.0-1.5 (Retrofitted B.)	Beams	72.9	65.7	65.7	57.1	48.6
	0.0-0.0 (Existing B.)	Shear Walls	-	-	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	100.0	0.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	66.7	0.0	0.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	Shear Walls	25.0	0.0	0.0	0.0	0.0
Kobe	0.0-0.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.0-0.0 (Existing B.)	Beams	100.0	100.0	100.0	51.3	41.0
	0.5-0.5 (Generated B.)	Beams	65.8	57.9	57.9	55.3	44.7
	1.0-1.0 (Generated B.)	Beams	61.1	58.3	58.3	58.3	44.4
	1.0-1.5 (Retrofitted B.)	Beams	60.0	60.0	57.1	34.3	28.6
	0.0-0.0 (Existing B.)	Shear Walls	-	-	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	100.0	0.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	100.0	66.7	0.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	Shear Walls	100.0	50.0	0.0	0.0	0.0
Chi-Chi	0.0-0.0 (Existing B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.5-0.5 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.0 (Generated B.)	Columns	100.0	100.0	100.0	100.0	100.0
	1.0-1.5 (Retrofitted B.)	Columns	100.0	100.0	100.0	100.0	100.0
	0.0-0.0 (Existing B.)	Beams	100.0	92.3	92.3	79.5	61.5
	0.5-0.5 (Generated B.)	Beams	68.4	71.1	60.5	55.3	55.3
	1.0-1.0 (Generated B.)	Beams	63.9	58.3	58.3	55.5	55.5
	1.0-1.5 (Retrofitted B.)	Beams	62.9	57.1	54.3	54.3	54.3
	0.0-0.0 (Existing B.)	Shear Walls	-	-	-	-	-
	0.5-0.5 (Generated B.)	Shear Walls	100.0	100.0	0.0	0.0	0.0
	1.0-1.0 (Generated B.)	Shear Walls	100.0	33.3	0.0	0.0	0.0
	1.0-1.5 (Retrofitted B.)	Shear Walls	100.0	25.0	0.0	0.0	0.0