EFFECT OF SHEAR WALLS ON THE BEHAVIOR OF REINFORCED CONCRETE BUILDINGS UNDER EARTHQUAKE LOADING

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

HAKKI GÜRHAN ÇÖMLEKOĞLU

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

DECEMBER 2009

Approval of the thesis:

EFFECT OF SHEAR WALLS ON THE BEHAVIOR OF REINFORCED CONCRETE BUILDINGS UNDER EARTHQUAKE LOADING

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ABSTRACT

EFFECT OF SHEAR WALLS ON THE BEHAVIOR OF REINFORCED CONCRETE BUILDINGS UNDER EARTHQUAKE LOADING

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November 2009, 141 Pages

An analytical study was performed to evaluate the effect of shear wall ratio on the dynamic behavior of mid-rise reinforced concrete structures. The primary aim of this study is to examine the influence of shear wall area to floor area ratio on the dynamic performance of a building. Besides, the effect of shear wall configuration and area of existing columns on the seismic performance of the buildings were also investigated. For this purpose, twenty four mid-rise building models that have five and eight stories and shear wall ratios ranging between 0.51 and 2.17 percent in both directions were generated. These building models were examined by carrying out nonlinear time-history analyses using PERFORM 3D. The analytical model used in this study was verified by comparing the analytical results with the experimental results of a full-scale seven-story reinforced concrete shear wall building that was tested for U.S.-Japan Cooperative Research Program in 1981. In the analyses, seven different ground motion time histories were used and obtained data was averaged and utilized in the evaluation of the seismic performance. Main parameters affecting the overall performance were taken as roof and interstory drifts, their distribution throughout the structure and the base shear characteristics. The analytical results indicated that at least 1.0 percent shear wall ratio should be provided in the design of mid-rise buildings, in order to control observed drift. In addition; when the shear wall ratio increased beyond 1.5 percent, it was observed that the improvement of the seismic performance is not as significant.

Keywords: Earthquake, Shear Wall, Shear Wall Ratio, Drift, Base Shear, Nonlinear Time History Analysis

ÖZ

PERDE DUVARLARIN BETONARME BİNALARIN DEPREM YÜKÜ ALTINDAKİ DAVRANIŞINA ETKİSİ

Çömlekoğlu, Hakkı Gürhan
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Yrd. Doç. Dr. Burcu Burak

Aralık 2009, 141 Sayfa

Perde duvar oranının orta katlı binaların dinamik davranışı üzerindeki etkisini inceleyen analitik bir çalışma yapılmıştır. Bu çalışmanın ana amacı, perde duvar alanının kat alanına oranının binaların dinamik performansına olan etkisini incelemek olmuştur. Bunun yanı sıra, perde duvarların plandaki yerleşimi ve mevcut kolon alanlarının yapıların performansına olan etkileri de ayrıca incelenmiştir. Bu amaçla, perde duvar yüzdeleri her iki yönde 0.51 ve 2.17 arasında değişen, beş ve sekiz kattan oluşan yirmi dört adet orta katlı bina modeli oluşturulmuştur. Bu modeller nonlineer zaman-tanım alanı analizi ile incelenmiş ve analizler PERFORM 3D programı ile gerçekleştirilmiştir. Analizlerde kullanılan analitik modelin tutarlılığı 1981'de A.B.D.- Japonya Ortak Araştırma Projesi için test edilen, yedi katlı tam ölçekli betonarme perde duvarlı binaya ait deney sonuçları ile kontrol edilmiştir. Analizler yedi farklı deprem kaydı ile gerçekleştirilmiş ve elde edilen verilerin ortalaması binaların deprem yükü altındaki performansını değerlendirmede kullanılmıştır. Binaların, çatı ve göreli kat ötelemeleri ile bunların yapı genelindeki dağılımları ve taban kesme kuvveti özellikleri genel performansı etkileyen parametreler olarak değerlendirilmiştir. Araştırma sonunda orta katlı betonarme duvarların tasarımlarında, kat ötelemelerinin kontrol altında tutulması için en az % 1.0 oranında perde duvar kullanılması gerekliliği ortaya çıkmıştır. Buna ek olarak, perde duvarlar oranı % 1.5'un üzerine çıkartıldığı takdirde, bu orandaki artışın binanın deprem yükü altındaki performansına olan etkisi azalmaktadır.

Anahtar Kelimeler: Deprem, Perde Duvar, Perde Duvar Oranı, Kat Ötelemesi, Taban Kesme Kuvveti, Nonlineer Zaman-Tanım Alanı Analizi

To My Father

ACKNOWLEDGMENTS

I would like to express my deepest gratitude to my supervisor Assist. Prof. Dr. Burcu Burak for her guidance, advice, criticism, encouragements and insight throughout the research.

I would also like to thank Prof. Dr.-Ing. Uwe Dorka from Universitat Kassel, Germany for his supports during my stay for exchange study in Kassel.

I am deeply indebted to my family for the endless reliance, support and love they have given me throughout my life.

TABLE OF CONTENTS

LIST OF TABLES

TABLES

LIST OF FIGURES

FIGURES

LIST OF SYMBOLS

A_c: Gross cross-sectional area of column or shear wall

A_{ch}: Gross cross-sectional area of shear wall

A_d: Axial area of the diagonal brace

A_w, A_{wall}: Cross-sectional area of shear wall

$\sum A_c$: Total cross-sectional area of base storey columns

$\sum A_{col}$: Total cross-sectional area of columns above base

$\sum A_{cw}$: Total cross-sectional area of reinforced concrete walls in one horizontal direction at base

$\sum A_{ft}$: Total floor area above base in a building

$\sum A_g$: Sum of section areas of structural elements at any storey behaving as structural walls in the direction parallel to the earthquake direction considered

$\sum A_{mw}$: Total cross-sectional area of non-reinforced masonry filler walls in one horizontal direction at base

A_p: Plan area of one storey

$\sum A_p$: Sum of plan areas of all stories of building

$\sum A_w$: Total area of shear walls at a typical storey

A_0: Effective Acceleration Coefficient

b: Length of shear wall

D: length of shear wall

E: Modulus of elasticity of concrete

E_c: Concrete modulus of elasticity

f_c: Compressive strength of concrete

f_{ck}: Characteristic compressive strength of concrete

f_{ctd}: Design tensile strength of concrete

f_y: Yield strength of longitudinal reinforcement

f_{yk}: Characteristic yield strength of longitudinal reinforcement

G: Shear Modulus

g: Gravitational acceleration

H: Height of shear wall

h: Height of shear wall

I: Importance factor

I_c: Moment of inertia of column

I_w, I_{wall}: Moment of inertia of wall

L: Length of element

l: Length of diagonal brace

l_b: Length of beam

l_w: Shear wall length

N: Axial load

n: Number of stories

R: Earthquake load reduction factor

PsD: Pseudo dynamic

S(T): Spectrum Coefficient

T: Fundamental structural period

t_d: thickness of shear wall

V_t: Total shear force at the base of the structure

V_r: Shear resistant force

W: Total weight of building calculated by considering live load participation factor

WI: Wall Index

ATC: American Technology Council

ACI: American Concrete Institute

CI: Column Index

FEMA: Federal Emergency Management Agency

SEAOC: Structural Engineers Association of California

TEC: Turkish Earthquake Code

TS: Turkish Standards

TVLEM: Three-Vertical-Line-Element Model

3D: Three dimensional

ν: Poissons ratio

α_s: Shear stiffness reduction factor

ρ_{sh}: Percentage of lateral steel reinforcement

θ: Slope of diagonal

ε_{max}: Maximum strain

CHAPTER 1

INTRODUCTION

1.1. GENERAL

Earthquake is one of the most important threats that structures can face during their service life. Investigations after strong ground motions revealed that properly designed and detailed shear wall buildings showed relatively good performance in past earthquakes. Shear walls built in high seismic regions should be in compliance with special detailing requirements. However, prior observations indicated that even buildings that have high wall area to floor area ratios with walls that do not have special seismic detailing survived high magnitude earthquakes. These observations drew attention of both practical engineers and academic researchers to shear wall-frame buildings. Therefore, in order to minimize the loss after these possible disasters, the availability of experimental and analytical studies that add up to solid information on the seismic design approaches, encourage use of shear walls for earthquake resistant design. In addition, the features which are the simplicity of constructing shear walls due to practical reinforcement detailing and implementation at construction site make shear walls more popular in construction applications of earthquake prone countries. Consequently, shear walls are efficient, both in terms of construction cost and effectiveness in minimizing earthquake damage in buildings.

Fintel [9] investigated and reported on the behavior of modern structures under strong ground motions for a period starting with the Skopje Earthquake of 1963 through the Armenian Earthquake of 1988, in which no collapse and life loss occurred in the buildings containing shear walls even though cracking with various degrees of severity was observed in some cases. Excessive interstory distortions that caused shear failures of columns were indicated as the main reason for the collapse of the hundreds of reinforced concrete frame structures. Even when total collapse was not observed, large interstory distortions in frames caused significant property loss. Fintel [9] states that it should be the responsibility of engineers and architects to make certain that residential buildings are constructed with significant shear wall to floor area ratios. The impressive efficiency of shear walls in resisting strong earthquake motions is summarized in the quote of Fintel [9] “We cannot afford to build economical concrete buildings to resist severe earthquakes without shear walls.”

The ratio of shear wall area to floor area, the wall aspect ratio, and the wall configuration in plan are indicated as important parameters that affect the details of a shear wall for reinforced concrete design (Wallace [36]). However; among these parameters, shear wall ratio is also accepted as an essential parameter affecting the global performance of a building under severe ground motions. Therefore, wall ratio is set as a leading parameter to be investigated in this analytical study. The effect of shear wall ratio on structural vulnerability could be evaluated by the variation of different parameters such as roof or interstory drift with increasing shear wall ratio.

1.2. RESEARCH OBJECTIVES AND SCOPE

Investigations performed after earthquakes revealed that buildings designed with no shear walls are more vulnerable to collapse when they are subjected to strong ground motions. This outcome has drawn the attention of engineers to consider shear walls in design because of their efficiency in not only reducing construction cost but also minimizing earthquake damage.

Roof and interstory drifts are good indicators of expected damage of a building during earthquake loading. In earthquake resistant design; when a dual system is utilized, general approach is using a shear wall area to floor area ratio of about 1.0 percent as a rule of thumb. At this point, it is worth to mention that the relationship between drift and shear wall ratio is not studied extensively for reinforced concrete buildings under earthquake loading. Consequently, the deficiency of research on this relationship prevents current codes to provide recommendations on this parameter.

This study mainly concentrates on the influence of the shear wall area to floor area of a building on the seismic performance of reinforced concrete mid-rise buildings. Additionally, the effect of shear wall configuration and area of the existing columns on the dynamic behavior were also investigated. For this purpose, 24 building models that have 5 and 8 stories and shear wall to floor area ratios ranging between 0.51 % and 2.17 % for both directions were generated. Investigations of the structures were performed by inelastic analyses (nonlinear time-history analysis) by using a structural analysis program, PERFORM 3D v 4.0. [5]. In the analyses, 7 different ground motion time histories were used and average of the data obtained from each earthquake record was utilized in evaluation of the results. The change in seismic behavior of the buildings with different shear wall ratios and configurations and the influence of total column area to the performance of the system was evaluated in terms of inelastic roof drifts, interstory drifts and base shear responses. The outcomes on these performance parameters would enable the codes to provide recommendations that can be used in the preliminary design of shear wall systems.

1.3. THESIS OUTLINE

In this study, the influence of the shear wall area to floor area of a building on the seismic performance of reinforced concrete mid-rise buildings is examined. There are five chapters in this thesis. Chapter 1 provides brief information on the behavior of shear wall buildings under earthquake loading and sets the research objectives. Chapter 2 includes literature survey on description and modeling of shear walls, wall ratios and the relationship between wall ratio and drifts in reinforced concrete shear wall-frame buildings. Chapter 3 gives information on the description of structures including the full-scale seven-story test building of U.S.-Japan Cooperative Research Program which was utilized for verifying the analytical modeling technique and the 5 and 8 story model buildings generated for examining the effectiveness of the shear wall to floor area ratios as well as the effect of total column area and shear wall configuration on dynamic performance of the structures. Details of the element models utilized in analytical modeling and the selected ground motions for time history analyses are also given in Chapter 3. Results of the analyses and the corresponding outcomes are evaluated in Chapter 4. Conclusions of this study and recommendations for future research are presented in Chapter 5.

CHAPTER 2

LITERATURE REVIEW

2.1. DESCRIPTION OF SHEAR WALLS

The essential purpose of structural systems is to support gravity loads, such as dead loads, live loads and snow loads. In addition to these vertical loads, lateral loads caused by earthquake, wind or blast also acts on structures. These lateral loads may cause considerable amount of drift in the structural system which may induce high stresses in the load carrying members. Whenever certain limits for strength or deformation capacities of structural members are exceeded, partial or even total collapse of the structure is sometimes inevitable under these kinds of lateral loads. Therefore, having adequate stiffness as well as sufficient strength is crucial for a structure in order to survive lateral loading.

Shear walls resisting axial and shear forces under static and dynamic loading provide high strength and in-plane stiffness to buildings in the direction of their orientation. Consequently, these structural members significantly reduce roof and interstory drifts of a building under lateral loading and thereby reduce damage to the structure and its components. As other reinforced concrete members, shear walls should be designed properly to sustain their strength and stiffness without any critical deterioration while providing energy dissipation during linear and nonlinear response.

In current applications, shear walls are generally designed as planar. However, in some applications engineers utilize non-planar shear wall sections in their design for functionality as well as for architectural needs. Examples of shear walls with different cross sections used in the structures are given in Figure 2.1.

Box and H-Section shear walls are generally designed as central cores in high rise buildings, while L-Sections are popular in seismic strengthening applications, especially when the strengthening procedure is to be implemented from outside of the structure in order to keep the building in service during construction. In addition, a wide use of U-Sections as stair wells or elevator cores is observed in the building stock of Turkey.

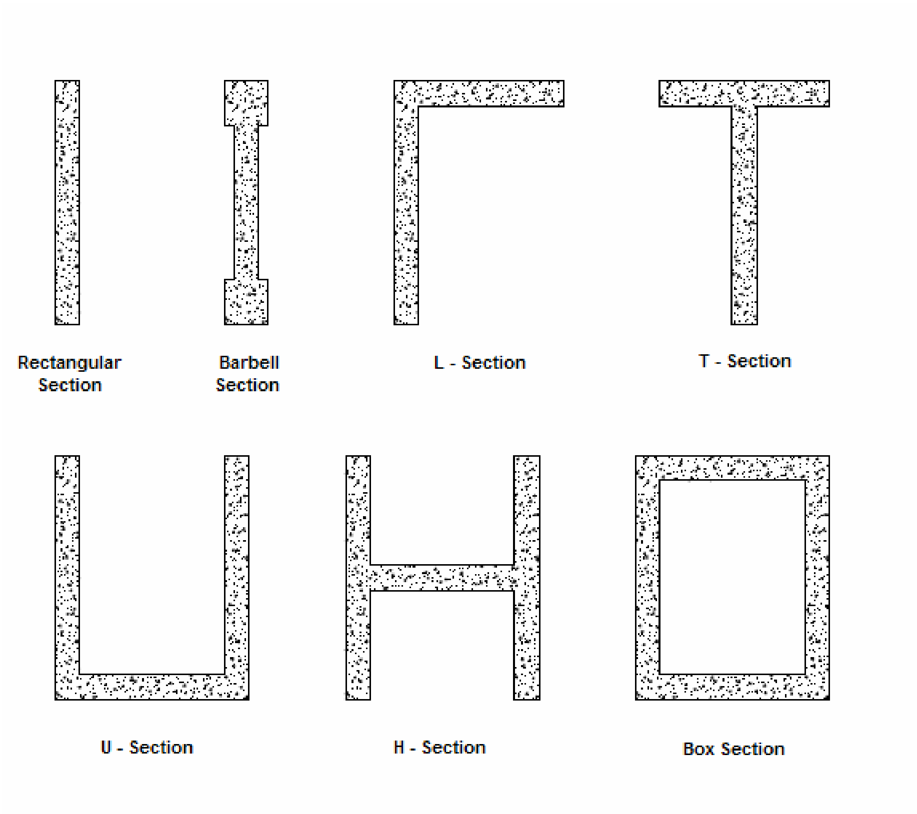

Figure 2.1. Typical Shear Wall Sections

According to the current reinforced concrete design code used in Turkey (TS500 [30]), shear walls are described as vertical load carrying members that have a minimum ratio of the longer to shorter dimension of 7. In addition, there is a minimum thickness requirement of the walls, set as 200 mm in TEC2007 [28].

Gülkan [10] comments on the description of the shear walls by discussing the differences between the Turkish code and international codes on the behavior and design of shear walls. A shear wall is described as a member that carries a great amount of lateral earthquake induced force along its frame. These members should have adequate stiffness and strength properties in order to interact with the frame it belongs to and fulfill the required construction details. The theory of structural analysis indicates that the main property of a wall, which shows it behaves as a shear wall, is the ratio of total height to its depth instead of ratio of the depth to the thickness [10]. It was observed that members defined as shear walls, fulfilling the requirement of the former codes including Turkish Earthquake Code (1998) [29], failed immediately due to lateral loading during strong ground motions, when ratio of the depth to the thickness was used to define a shear wall [10]. Therefore, wall dimensions in the vertical direction are emphasized to be critical.

2.2. MODELING OF SHEAR WALLS

Analytical techniques used to model reinforced concrete shear walls can be divided into two major groups; microscopic and macroscopic models. Microscopic models are generated by the use of finite element approach considering solid mechanics; however macroscopic models are based on the results of conducted tests. Although microscopic modeling techniques are more theoretical, their use in practice is limited for nonlinear dynamic analyses of multi-story buildings with reinforced concrete shear walls, especially for 3D analytical models. Because, those analyses require very detailed and complex models to provide the features of cyclic dynamic behavior, and also sophisticated finite element programs increase the time required for creating the structural model, computing and interpreting the analytical results. Microscopic models are more suitable for determining localized damage in a structure; however, more practical approaches emphasize prediction of global behavior rather than local behavior in current practice [14].

When compared to microscopic models, macroscopic models can be easily utilized in analytical models; however their usage is limited. Vulcano and Bertero [33] emphasize that the analytical results are only valid for the specific conditions, on which the derivation of the model is based. However, in general, macroscopic models show more consistent results in the analysis of structures that do not have any irregularities in plan or elevation. Three main types of macroscopic models developed for modeling reinforced concrete shear walls can be categorized as the equivalent beam element model, the braced wide column analogy model, and the vertical line element model.

In equivalent beam element method, which is also known as wide column analogy, reinforced concrete shear walls are replaced by an idealized frame structure consisted of a column and rigid beams located at floor levels. At the centroidal axis of the wall a column is defined that has the same moment of inertia and cross sectional area with the wall. Infinitely rigid beams, which have half the length of the wall, are located at each story level and link the center column to the connecting beams. Plane sections remain plane concept is valid for this model under lateral loading (Figure 2.2). An analytical model for the shear wall is shown in Figure 2.3.

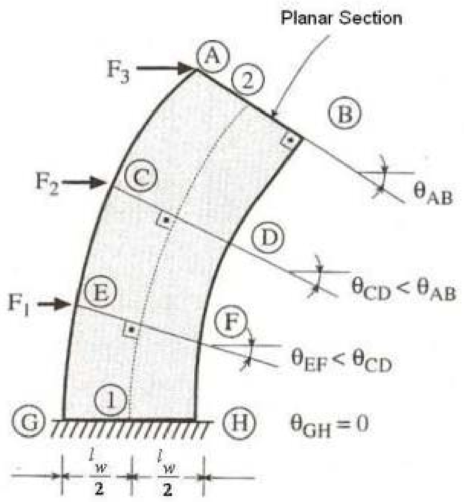

Figure 2.2. Deformation of Shear Wall under Lateral Loading [3]

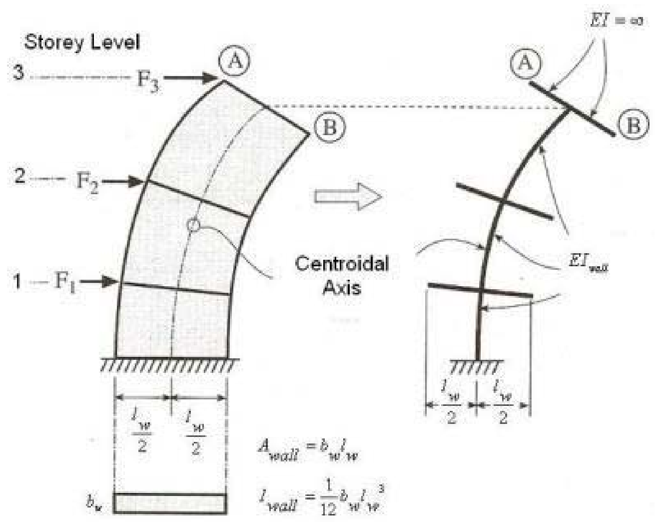

Figure 2.3. Shear Wall Model Proposed for Equivalent Beam Model [3]

In wide column analogy, rotations are assumed to take place about the centroidal axis of the element. This may introduce some drawbacks to the model, especially in nonlinear analyses. Important parameters, such as shift of the neutral axis of the cross section or rocking of the wall, leading to obtain a more realistic behavior of a shear wall member cannot be taken into consideration. However, due to its simplicity, the equivalent frame method is commonly used in practice for the analysis of multi story shear wall-frame structures.

The braced wide column analogy is similar to formerly described wide column analogy. However, in this model diagonal braces are added to the system. Rigid horizontal beams, equal to the length of the wall, are connected to a column at the centroidal axis. Diagonal braces with hinged ends are connected to the beams [22]. A typical braced wide column model represented as a single module is shown in Figure 2.4.

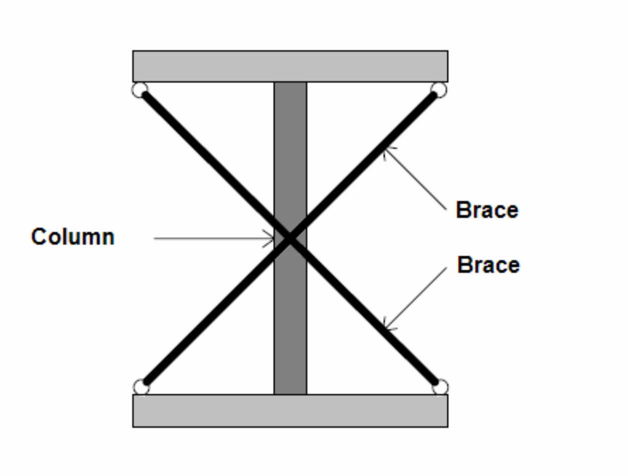

Figure 2.4. Braced Wide Column Module

The stiffness properties of the column; moment of inertia, I_c, and area of the cross section, A_c, and the stiffness properties of the braces; axial area of the diagonal brace, A_d, are determined from the following equations:

$$I_c = \frac{tb^3}{12} \tag{2.1}$$

$$\frac{12EI_c}{h^3} + \frac{2EA_d \cos^2\theta}{l} = \frac{btG}{h} \tag{2.2}$$

$$\frac{EA_c}{h} + \frac{2EA_d \sin^2\theta}{l} = \frac{EA_w}{h} \tag{2.3}$$

These equations are obtained by considering the equilibrium for the bending, shear and axial stiffness of corresponding wall segments, respectively. In these equations, t is the thickness and b is the length of the shear wall, E is the modulus of elasticity, h is the height of the shear wall, θ is the slope of the diagonal, l is the length of the diagonal brace, G is the shear modulus and A_w is the sectional area of the shear wall.

Akış [1] states that the probability of obtaining negative stiffness values for the column and braces for certain aspect ratios of the framework modules may be one of the deficiencies of the braced wide column analogy model since most of the frame analysis software cannot perform analysis with negative area and inertia values.

Kabeyasawa et al. [12] propose a wall member which is idealized as three vertical line elements with infinitely rigid beams at top and bottom floor levels (Figure 2.5). Two truss elements existing outside of the wall model represents the axial stiffness of the boundary columns. Central truss element has horizontal, vertical and rotational springs concentrated at the base level. The model simulates wall deformation under uniform bending and the resistance of the wall is lumped at the vertical spring of the central vertical one-component model and outer truss elements. Axial rigidity of the truss elements and the vertical spring of the central vertical element are determined by considering the related axial areas. Axial rigidity of these elements are reduced to 90 percent of their initial elastic stiffness values when the axial force due to gravity effect is overcome by the overturning effect. The yield tension load is taken as the sum of the forces carried by the longitudinal reinforcements of the elements at yield. The stiffness values for the elements are reduced to 0.1% of the initial one after tensile yielding. Axial load versus moment capacity interaction of the elements is not considered in the analysis.

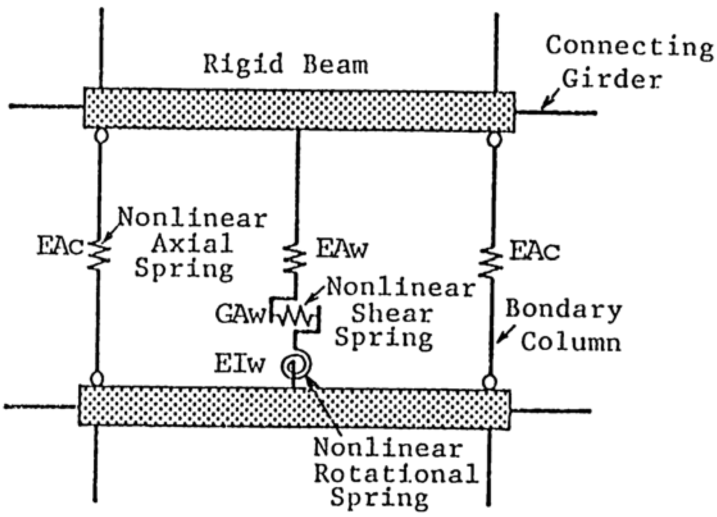

Figure 2.5. Shear Wall Model Proposed by Kabeyasawa et al. [12]

In the analytical model, lateral spring located in the central vertical element provides the shear resistance of the wall. A factor, α_s, which is the ratio of secant stiffness at the shear yield point to the elastic stiffness is used in the model for the reduction of shear stiffness. After shear yielding point, the stiffness is reduced to 0.1% of the initial elastic shear rigidity [12].

For the computation of rotations, moment distribution is assumed to be uniform along the story height set at a value equal to the moment at the critical wall section. Yielding moment is taken as fully plastic; in other words yield moment value about the centroid of the wall section is determined considering all the vertical wall reinforcement is yielded and the effect of gravity loads is not taken into account for this computation. The stiffness of the rotational spring after yielding is taken as 0.1 percent of the initial elastic stiffness [12].

For the two outside truss elements and the central vertical spring element of the shear wall model, axial-stiffness hysteresis model, which is given in Figure 2.6, is used. Origin-Oriented Hysteresis Model, in which the response point moves along a line connecting the origin and the previous maximum response point in positive and negative directions, is used for the rotational and horizontal springs located at the base of the central vertical element (Figure 2.7). When the response point exceeds the previous maximum point, a skeleton force-deformation relationship with a new maximum response point is followed. This model dissipates only a low amount of hysteresis energy. No hysteresis energy is dissipated when the response point oscillates within the region between positive and negative maximum response points. Stiffness changes as the sign of the response point changes and there is no residual deformation in this model [12].

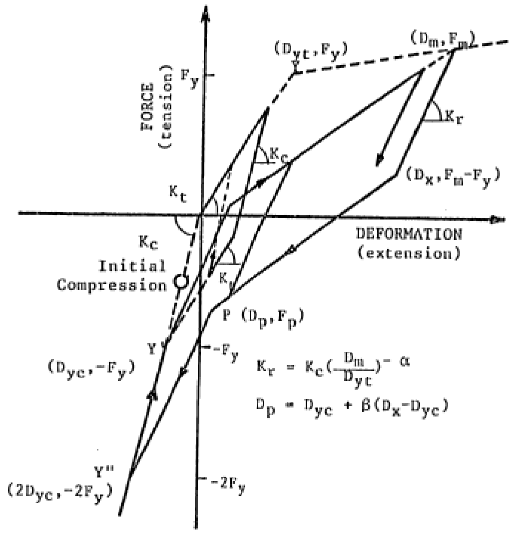

Figure 2.6. Axial Stiffness Hysteresis Model [12]

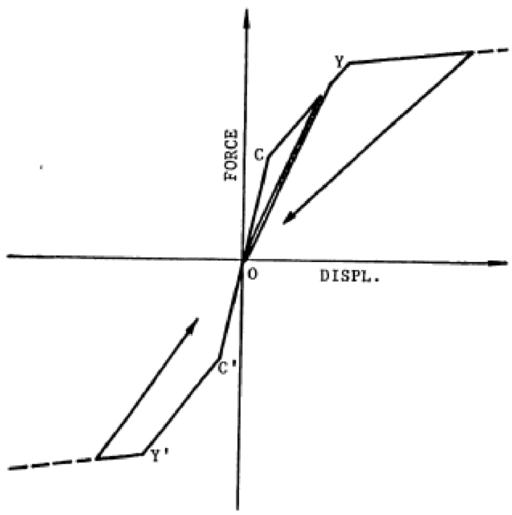

Figure 2.7. Origin-Oriented Hysteresis Model [12]

The Three-Vertical-Line-Element Model (TVLEM) proposed by Kabeyasawa et al [12] was modified by Vulcano et al. [34] by introducing additional axial springs. The use of at least four axial springs was proposed to improve the flexural response and the rotational spring located at the central vertical element was removed in this model. A simpler model was recommended by Linde [16] that has three axial springs and one horizontal spring (Figure 2.8). In this model a simple empirical mathematical formulation is developed to predict the nonlinear behavior more accurately. In 2002, Orakcal et al. [19] developed a multiple spring macroscopic model similar to the one that Vulcano et al. [34] proposed. There are two main differences between two analytical shear wall models. The first difference is that Orakcal et al. [19] increased the number of vertical springs in order to reflect the actual cross sectional details and the detailing of reinforcement to the model accurately. The other difference was that hysteretic constitutive laws of concrete and reinforcing steel were introduced for the springs. The flexural response of reinforced concrete shear wall members is generally represented precisely with macroscopic models. When flexure governs the dynamic behavior of the wall, moment curvature envelope can be obtained by using axial springs representing the boundary elements. However, most of the time, axial springs do not satisfactorily simulate the shear response. In order to predict the overall response, several shear models have been introduced by macroscopic model users by varying the model parameters to match the experimental results, such as changing the vertical location of the horizontal spring to indirectly account for shear behavior, but precise results can not be obtained [14].

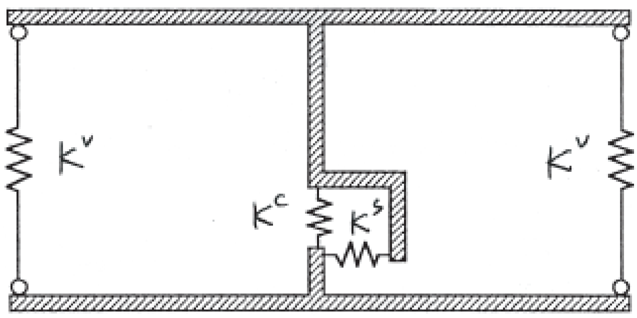

Figure 2.8. Shear Wall Model Proposed by Linde [16].

Kim et al. [14] state that one reason for the deficiency of some existing reinforced concrete shear wall models is that shear distortion developed in a wall results from flexural yielding in the plastic hinge zone and does not generally follow a typical reinforced concrete shear force versus shear distortion relationship. When flexure governs the behavior, the wall never reaches its full shear strength since typically no shear yielding occurs. First, flexural cracks in boundary regions appear and then inclined cracks develop across the web region of the walls followed by shear cracks observed at lower level walls which occur only towards the end. Therefore, shear distortion is mostly related to flexural cracks that develop earlier than shear cracks. A pinched shape is formed in the shear distortion behavior due to the opening and closing of cracks, as observed by Oesterle et al. [17], [18], Wang et al. [37], and Vallenas et al. [32].

Lepage et al. [15] proposed an alternative simplified analytical model for nonlinear response of flexural-yielding reinforced concrete walls. Finely meshed linear-response shell elements coupled with uniaxial line elements are used in this analytical model. A sketch of a model is given in Figure 2.9. At the expected plastic hinge region, the axial stiffness of the shell elements are adjusted by modifiers varying between 0 and 1 along a transition region and gradually transferred to and from the neighborhood line elements which are introduced longitudinally in between the shell element nodes. In this modeling technique, typical nonlinear response parameters are assigned to the line elements which generally extend for a distance equal to the length of the wall, corresponding to a multilinear plasticity model. For this purpose, graphs that represent nonlinear force-deformation relationship for both tension and compression responses are constructed by using the material characteristics of the shear wall member. The first slope of the graph defines the range of linear elastic response, whereas the remaining segments characterize the plastic deformations.

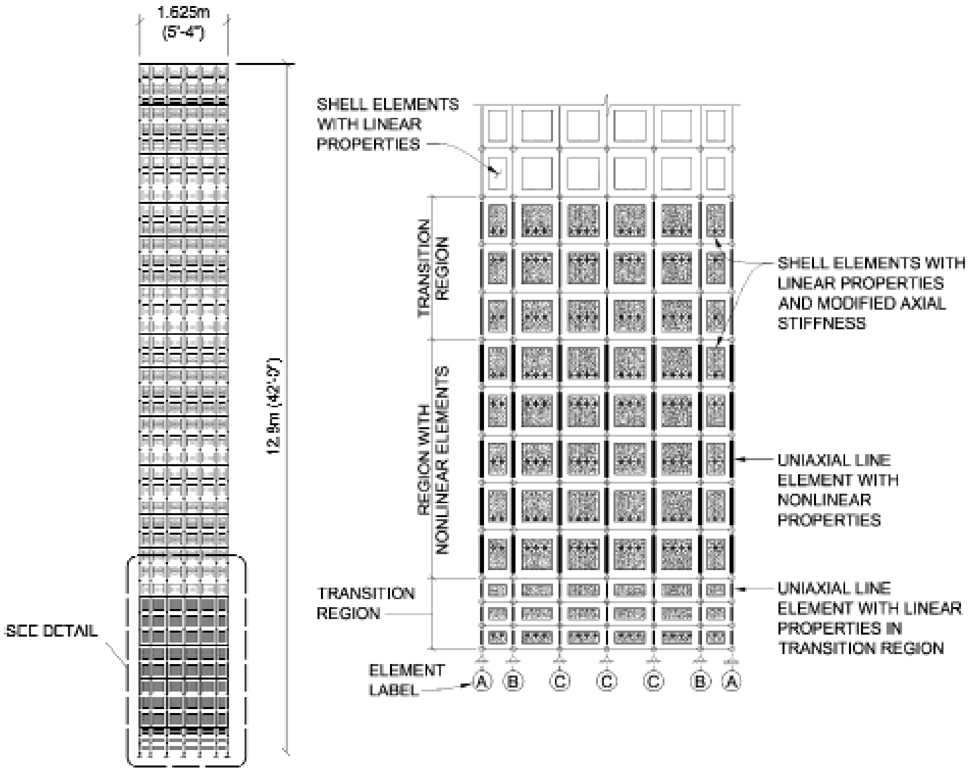

Figure 2.9. Shear Wall Model Proposed by Lepage et al. [15]

To represent compression response of the shear wall member, a force-deformation relationship characterizing the line element is developed using the tributary area. The compressive strength is defined by multiplying the tributary area by the compressive strength, f_c and the deformation is obtained from Lf_c / E_c where L is the length of the element and $E_c = 4700\sqrt{f_c}$ in MPa. Post-yield compression stiffness is taken as 0.1% of the initial stiffness.

To represent tension response, the yield force is determined by multiplying yield strength of the reinforcement, f_y by the tributary area. The deformation is derived by setting the slope equal to the initial slope used for compression loading. Identical initial slopes are assumed for both tension and compression loading to conveniently characterize the overall stiffness of the wall. Post-yield tension stiffness is set to 2%

of the stiffness associated with the area of steel only and it is indicated that for a strain ductility of 6, a steel stress of 1.1 times the yield value is reached.

Lepage et al. [15] propose this analytical model which is found to be successful in capturing the main response parameters such as, initial stiffness, onset of yielding, yield strength, and displacement response to perform practical nonlinear static or nonlinear dynamic analyses implemented in commercially available structural analysis programs.

Another approach for modeling of shear walls is utilizing fiber cross section components for the axial/bending layers. As being a more sophisticated model compared to the others, fiber model demands additional details in order to reflect a realistic behavior for the inelastic analysis of shear walls.

In this modeling technique, mechanic and hysteretic constitutive laws of concrete and reinforcing steel are introduced to the related fiber sections. The material properties of concrete and steel components such as, cracking and possible crushing compressive stresses of concrete and yield strength and possible buckling compressive stress of steel are assigned to fiber cross section components. Compound components of the shear wall are defined by assigning fiber sections to the vertical and horizontal axial/bending layers. General shear wall element is constructed by assembling compound components together by taking into account the number of fibers with their areas and locations as shown in Figure 2.10 [5].

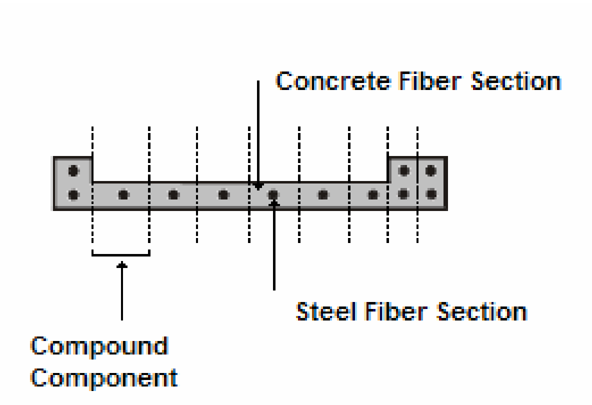

Figure 2.10. Shear Wall Element with Fiber Sections [5]

The cross sections for the axial/bending layers composed of inelastic fiber sections enable users to model a shear wall with more detailed parameters; however this requires extra time and consideration to generate a model which may be inaccurate for some applications.

2.3. SHEAR WALL RATIO

The ratio of shear wall area to floor area, named shortly as the shear wall ratio throughout this thesis report, is a parameter that is considered during preliminary earthquake resistant design in current practice. Shear wall ratio of a structure could be defined in two different ways. The total shear wall area at a typical story in a specific direction could be divided either to the floor area of that story or to the total floor plan area of the building.

Some approximate shear wall ratios are proposed in the literature to be used in preliminary design stages of shear wall-frame buildings. These ratios are generally based on empirical values that are obtained from building surveys performed after severe earthquakes or by force-based relations that are recommended by Ersoy [6] and Tekel [26].

Shear wall ratio of 1.0 % in each direction of a building, the value which is accepted by most of the design engineers as a rule of thumb and used in practice during preliminary design stages, is proposed by Ersoy [6] after the evaluation of seismic performance of the buildings during earthquakes like Erzincan (1992), Dinar (1995) and Ceyhan (1998) occurred in the near past of Turkey. Two inequality expressions are proposed by Ersoy [6] for residential and office buildings having number of stories up to 8:

$$0.5\sum A_c + \sum A_w \geq 0.003\sum A_p \quad (2.4)$$

$$\sum A_w \geq 0.002\sum A_p \geq 0.01 A_p \quad (2.5)$$

where,

$\sum A_c$: Total cross-sectional area of base story columns

$\sum A_w$: Total cross-sectional area of base story shear walls

$\sum A_p$: Total floor area

Tekel [26] evaluated the behavior of reinforced concrete structures having 1.0 % shear wall ratio by considering the Turkish Earthquake Code (TEC 2007 [28]) requirements. The main structure of this study is based on the three equations given below:

$$\sum A_g / \sum A_p \leq 0.002 \qquad \text{(Article 3.14 of TEC 2007)} \tag{2.6}$$

$$V_t / \sum A_g \leq 5 f_{ctd} \qquad \text{(Article 3.14 of TEC 2007)} \tag{2.7}$$

$$V_r = V_t \tag{2.8}$$

These equations are supported by two additional equations:

$$V_t = S(T) A_0 I W / R \tag{2.9}$$

$$V_r = A_{ch}(0.65 f_{ctd} + \rho_{sh} f_{ywd}) \tag{2.10}$$

Equation (2.9) is for computing the shear force exerted to a building during earthquake whereas Equation (2.10) is for determining the shear strength of a shear wall.

By using Equation 2.6, shear wall ratio can be represented as,

$$\sum A_g / A_p = 0.002n \tag{2.11}$$

Where n is the number of stories and A_p is the floor area of a single story. Table 2.1 gives the required shear wall ratios depending on the number of stories for the buildings where total shear force is carried by shear walls.

Table 2.1. Shear Wall Ratios for Varying Number of Stories Obtained by Equation 2.11

n, Number of Stories	Shear Wall Ratio%	n, Number of Stories	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	1.0	10	2.0

Considering Equation 2.11, a widely recommended shear wall ratio of 1.0 % is required for 5 story buildings.

Equation 2.7 is simplified to an equation with three variables which are A_g, A_p and n by plugging in the most commonly used values in practice, such as strength characteristics of construction materials, parameters used in the determination of response spectrum and etc. Shear wall ratios that depend on the number of stories are summarized in Table 2.2 and the simplified equation derived from Equation 2.12 is as follows:

$$\sum A_g / A_p = 0.0038n \qquad (2.12)$$

Table 2.2. Shear Wall Ratios for Varying Number of Stories Obtained by Equation 2.12

n Number of Stories	Shear Wall Ratio%	n Number of Stories	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	1.90	10	3.80

Table 2.2 shows that a shear wall ratio of 1% is required for buildings that have 3 stories based on Equation 2.12.

Equation 2.8 indicates that Equations 2.9 and 2.10 are equal, which means that total shear force exerted to a building is taken by shear walls only. The equilibrium is simplified, again, by plugging in most commonly used values in practice and Equation 2.13 is generated:

$$\sum A_g / A_p = 0.0012n \tag{2.13}$$

Table 2.3. Shear Wall Ratios for Varying Number of Stories Obtained by Equation 2.13

n Number of Stories	Shear Wall Ratio%	n Number of Stories	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	0.60	10	1.20

Table 2.3 shows that shear wall ratio of 1.0% is required for buildings that have 9 stories.

Investigations of Tekel [26] show that utilization of the equations derived following the requirements of TEC 2007 [28] is not likely to reach a distinct value for shear wall ratio. However, 1.0 % shear wall ratio is observed to be efficient for mid-rise buildings having number of stories ranging between 3 and 9 when the total shear force is carried only by shear walls.

A study was performed by Riddell et al. [21] to define the general features of the buildings located in Viña del Mar which experienced the Chile Earthquake (1985), magnitude of 7.8, and to detect the related earthquake damage. Data of 178 low and mid-rise buildings representing a stock of 322, of which 319 have shear walls, was used in the evaluation. Most of these buildings were designed with considerably high shear wall ratios, independent of number of stories, varying between 3.0 and 8.0 percent with an average of around 6.0 % (Figure 2.11). Investigations indicated that the design of the reinforced concrete shear walls was not in compliance with the current earthquake resistant design code requirements. Most of the shear walls were lightly reinforced without any boundary elements or special confinement details. Despite these facts, no damage was observed in nearly 90 percent of the buildings with shear walls during this severe earthquake. As an outcome of this study, it can be stated that the higher the shear wall ratio used in a building, the higher the possibility of having no damage in the structural system during a strong ground motion. However using an optimum shear wall ratio is essential in modern engineering design practice.

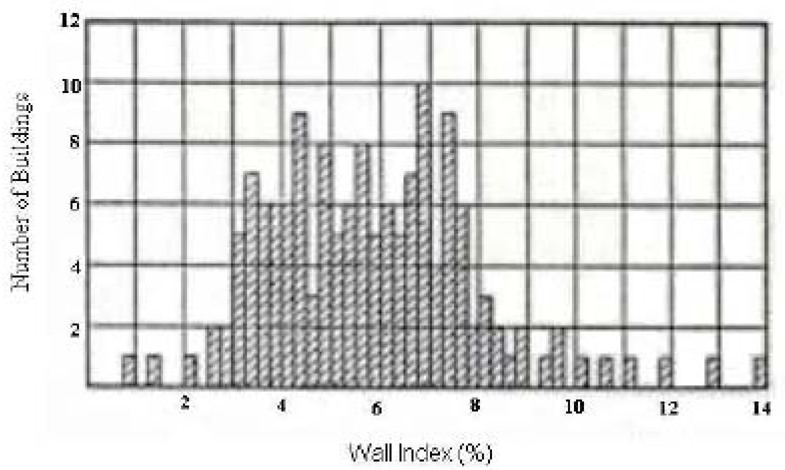

Figure 2.11. Shear Wall Index for Buildings in Viña del Mar [21]

A simplified method proposed by Hassan & Sözen [11] enables to rank an inventory of low-rise (up to 5 stories) monolithic reinforced concrete buildings based on their seismic vulnerability level from low to high by using column and wall indices. The method requires only structural dimensions as the input and is based on effective wall and column indices plotted in a two-dimensional form. The wall index including reinforced concrete and masonry infill walls is the ratio of the effective wall area at the base of the building to the total floor area above base. The column index is the ratio of the effective column area at base to the total floor area above base. The effective areas are proposed to be taken as the area of 100% of reinforced concrete walls, 10% of non-reinforced infill walls, and 50% of columns.

Two-dimensional plot in Figure 2.12 consists of wall index (WI) in y-axis and column index (CI) in x-axis.

Wall index is computed as follows:

$$WI = \frac{A_{cw} + \frac{A_{mw}}{10}}{A_{ft}} \times 100 \tag{2.14}$$

where,

A_{cw} = total cross-sectional area of reinforced concrete walls in one horizontal direction at base

A_{mw} = cross-sectional area of non-reinforced masonry filler walls in one horizontal direction at base

A_{ft} = total floor area above base in a building

Column index is obtained as:

$$CI = \frac{\frac{A_{col}}{2}}{A_{ft}} \times 100 \tag{2.15}$$

where,

A_{col} = total cross-sectional area of columns above base

A plot constructed for 46 institutional buildings that were subjected to 1992 Erzincan Earthquake that contains information on the observed damage is given in Figure 2.12.

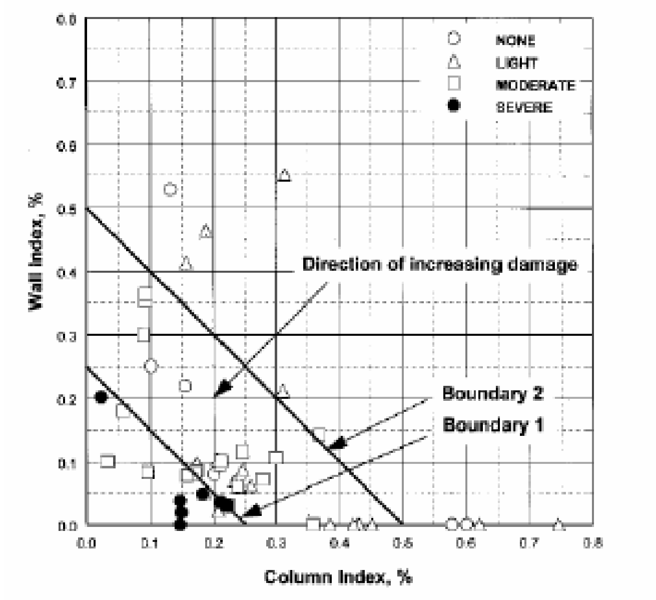

Figure 2.12. Proposed Evaluation Method by Hassan and Sözen [11]

In this procedure, the wall ratios are computed by considering total floor area of the building instead of considering a single story plan area. Boundary 1 and 2 are constructed on the graph without absolute basis. The triangular region formed between Boundary 1, column and wall index axes is accepted to be more critical than the outer region for a building by means of structural vulnerability. This procedure does not take into account some important parameters such as material quality, number of stories and structural frame details. However, due to its quick application and practicality, this method is very handy for evaluating buildings that are vulnerable to earthquakes.

2.4. RELATIONSHIP BETWEEN SHEAR WALL RATIO AND DRIFT

Lateral forces exerted by strong ground motions induce deformations on buildings leading to structural damage. Global deformations in a structure such as roof drift and interstory drifts are good indicators of expected damage of a building under earthquake loading. Even so, the relationship between drift and shear wall ratio have not been deeply investigated as of now. Independent of any shear wall ratio, current codes recommend certain limits for roof and interstory drifts obtained from both linear and nonlinear analysis. Eurocode 8 [7] limits the elastic design interstory drifts while SEAOC [24] and ATC 40 [2] have limits on inelastic interstory drifts for specified performance levels. TEC 2007 [28] also restricts the interstory drifts in linear elastic performance analysis.

An analytical procedure is proposed by Wallace [36] to observe the variation of roof drift with shear wall ratio. The procedure can be summarized as follows:

1) Elastic acceleration spectrum is converted to elastic displacement spectrum.
2) Fundamental period of the structure considering cracked sections is calculated by a proposed formula.
3) Elastic roof displacement of the structure is determined from the elastic displacement response spectrum constructed in Step 1.
4) Roof displacement is converted to roof drift which is later multiplied with a proposed factor of 1.5 in order to reflect the difference in behavior between a single degree of freedom system and the structural system that it represents.

Figure 2.13 is obtained by the procedure explained above for building models having different shear wall ratios. In this procedure, shear wall ratio is computed as the ratio of shear wall area to the total floor plan area of a building. In this graph, the effect of shear wall aspect ratio on the behavior is also given by using different curves. In the study by Wallace [36], it is stated that in addition to the determination of elastic drifts, the inelastic drift of a building can be obtained by utilizing inelastic acceleration spectrum following the same procedure proposed for the elastic roof drift analysis.

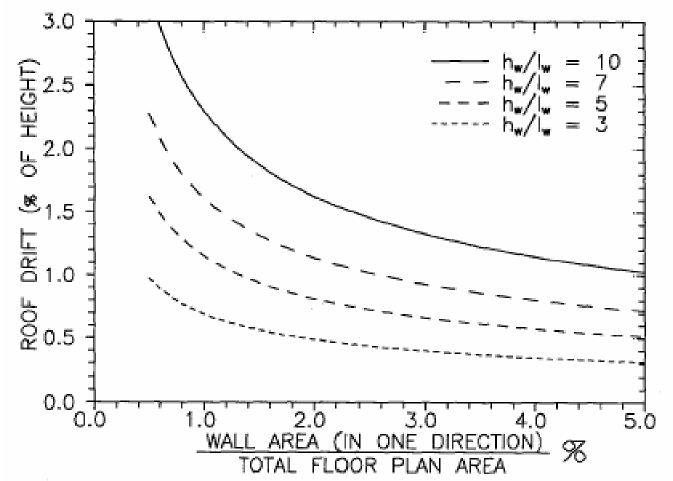

Figure 2.13. Roof Drift versus Shear Wall Ratio by Wallace [36]

Wallace and Moehle [35] investigated the response of shear wall buildings by using the same procedure that Wallace [36] proposed. The graph in Figure 2.14 presents the estimated periods of the buildings with different shear wall ratios. This study indicated that buildings with a shear wall ratio more than 1.5 % in the direction of loading, with a shear wall aspect ratio equal to or less than 5, are expected to experience roof drifts less than 1.0 percent under strong ground motions.

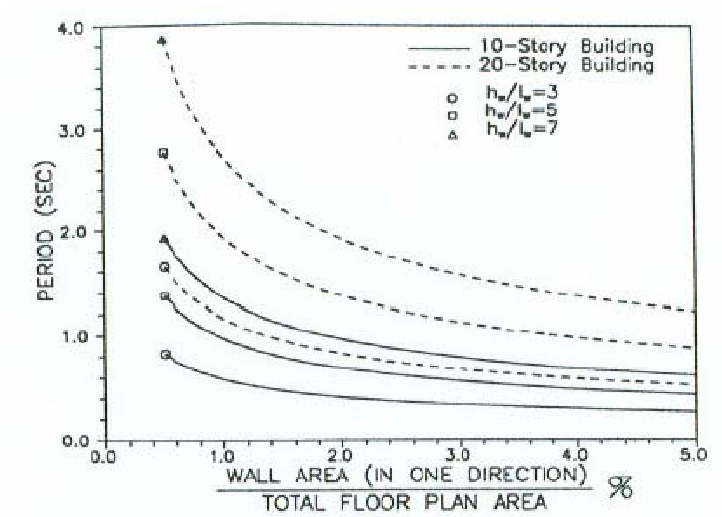

Figure 2.14. Estimated Fundamental Periods for Shear Wall Buildings [35]

Gülkan [10] investigated the relationship between roof drift and shear wall ratio by taking into account the shear wall aspect ratio, which is represented as H/D (Figure 2.15). The procedure followed in this study was similar to the one proposed by Wallace [36] except for the elastic displacement response spectrum utilized, which is based on TEC 2007 [28]. The study by Gülkan [10] emphasized the importance of a minimum shear wall ratio that should be used in the design of reinforced concrete buildings, especially for school buildings.

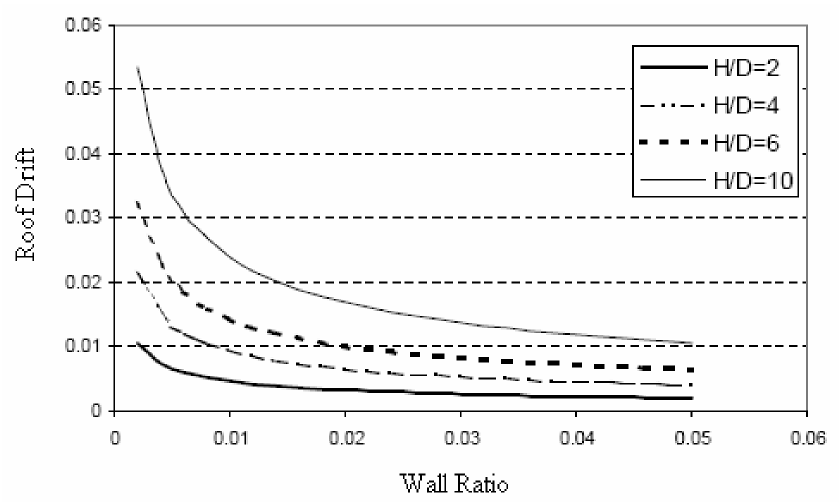

Figure 2.15. Estimated Roof Drift by Gülkan [10]

Gülkan [10] also examined a control criterion that could be used in the estimation of shear wall ratio. To keep the system damage under control, the global ductility demand of the shear wall was limited to a specific value. In other words, the ratio of tip displacement of the wall, δtop, to the tip displacement, when first yielding was observed in the wall, δy, shall not exceed a specific limit. For this purpose, Equation 2.16 of which the derivation is used and Figure 2.16 is obtained:

$$\frac{\delta_{top}}{\delta y} = 0.36\sqrt{\frac{1}{\rho}} \qquad (2.16)$$

where,

ρ is the shear wall ratio.

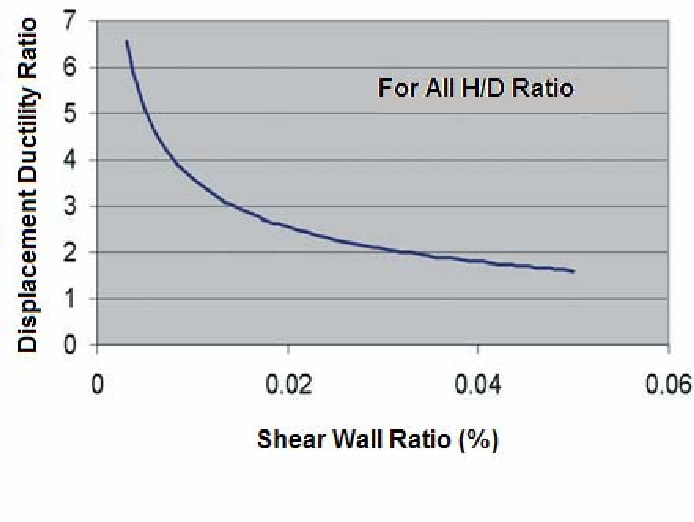

Figure 2.16. Variation of Displacement Ductility Ratio with Increasing Shear Wall Ratio [10]

For this equation, Gülkan [10] stated that the aspect ratio, H/D, of the shear wall is not critical for the calculations.

Maximum compressive concrete strain of shear wall member was also investigated as a control criterion in this study. When the strain is restricted to a level of 0.003, for large wall aspect ratios, H/D, and axial load levels, $N' = N / Dt_d f_{ck}$, this criteria is met with considerable amount of wall ratio. In Figures 2.17 and 2.18, strain level of 0.003 is shown with a horizontal line and based on the results, approximately 1.50 % shear wall ratio is required to satisfy the demand for the most unfavorable conditions of an axial load level of 0.15 and a shear wall aspect ratio of 10. As an expected outcome, shear wall members with reasonable aspect ratios and low axial load levels are likely to have lower damage, which can clearly be evaluated with ε_{max}.

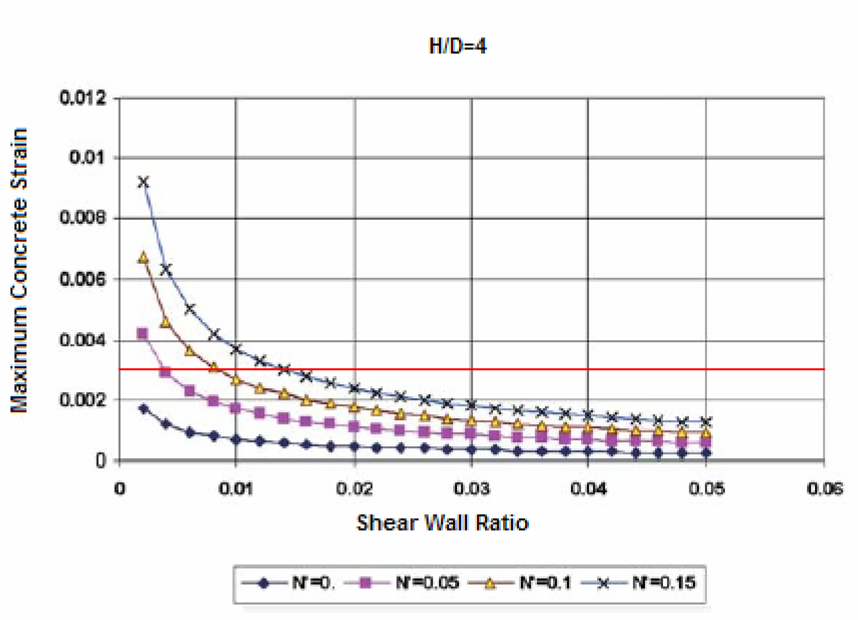

Figure 2.17. Estimation of Shear Wall Ratio for Constant Aspect Ratio and Variable Axial Load [10]

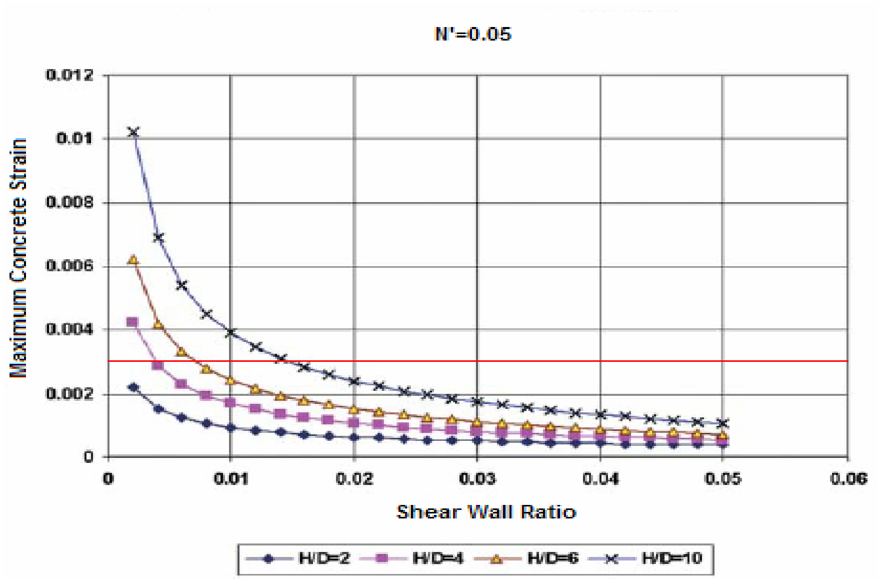

Figure 2.18. Estimation of Shear Wall Ratio for Constant Axial Load and Variable Aspect Ratio [10]

Soydaş [23] investigated the effect of varying shear wall ratios on the performance of reinforced concrete buildings to obtain limiting values that can be used in the preliminary assessment and design of buildings with shear walls. In this study, buildings with 2, 5 and 8 stories that have a shear wall ratios ranging between 0.53 and 3.60 percent were designed in compliance with Turkish Earthquake Code (TEC 2007 [28]) and the variation in roof and interstory drift with shear wall ratio was studied by both elastic and inelastic analyses. Additionally, the observed results were compared with several approximated procedures for drift calculations. In Figure 2.19, the variation of maximum interstory drift with shear wall ratio is given.

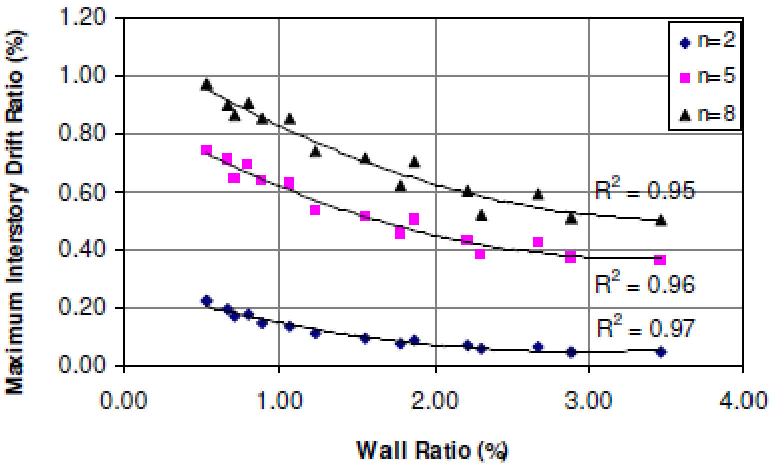

Figure 2.19. Variation of Calculated Maximum Interstory Drift with Shear Wall Ratio by Elastic Analysis [23]

According to the results of this study, interstory drifts showed a decreasing trend as the shear wall ratio increased up to 2 percent; however the maximum calculated interstory drift did not exceed 1.0 percent for the tallest building with the lowest shear wall ratio. Soydaş [23] concluded that the interstory drift limit proposed by TEC2007 [23] is not the primary effect to be considered in evaluation of performance of low to mid-rise shear wall-frame buildings.

The correlation between the seismic performance and shear wall ratio was also investigated by Soydaş [23]. The generated building models were assessed following the linear assessment procedure of the Turkish Earthquake Code (TEC 2007 [28]). The results obtained from the observation of variation in the roof and interstory drift with shear wall ratio were compared with the performance based limits given in the code. A weak trend between the wall ratio and the performance was observed by indicating an inconsistency between the design and assessment parts of the code. The limits given in Table 2.4 were proposed as minimum wall ratios for different number of stories by considering the performance level of the vertical members of the generated buildings for quick assessment and preliminary design stages. Life safety performance level of TEC2007 is indicated to be satisfied for all vertical members in the Y direction for all models and for vertical members of models that have wall ratios greater than 1.33 percent [4].

Table 2.4. Proposed Wall Ratios for Different Performance Levels [4]

	Wall Ratio (%)		
Number of Stories	IO	LS	CP
2	0.50	0.50	0.50
5	1.00	1.30	1.60
8	1.00	1.30	1.60

CHAPTER 3

ANALYTICAL MODELING

3.1. INTRODUCTION

In this chapter, description of the building models in terms of dimensions, sections, detailing, and material properties are presented. Element models used for columns, beams and shear walls throughout the analysis are explained in detail. First, information on the seven-story full-scale test building used for the verification of the analytical modeling approach is provided. Then, the thought process in the generation of the model buildings and selection of their structural detailing was explained. Next, description of the element models used in nonlinear dynamic time-history analyses is given. Finally, the details of the time histories of the selected ground motions used throughout the analyses are presented. The structural models of the buildings were prepared by using PERFORM 3D v 4.0 [5].

3.2. SEVEN-STORY FULL-SCALE TEST BUILDING

A full-scale seven-story reinforced concrete building was designed, constructed and tested at the Building Research Institute, Ministry of Construction, Japan, as part of the U.S.-Japan Cooperative Research Program Utilizing Large Scale Testing Facilities. The data from the PsD (Pseudo Dynamic) test applied on this building enables the comparison of the two analytical shear wall models considered to be used in this study, which are the fiber section and wide column analogy methods. Consequently, the most efficient model to be utilized in the analyses of the model buildings is selected by comparing the experimental data with the analytical results.

The building has three bays in the longitudinal direction, and two in the transverse direction (Figures 3.1 & 3.2). There is a shear wall in the plan of the building placed parallel to direction of loading, in the middle bay of the center frame. Columns are 500x500 mm throughout the structure. Girders parallel to the loading direction are 300x500 mm from the second floor to the roof level, whereas 300x450 mm transverse beams are used in the perpendicular direction. The average concrete cover from outer surface of the longitudinal reinforcement was 56 mm in columns, 48 mm at bottom of the beams and 59 mm at top of them. The 200 mm thick shear wall has boundary columns on both of its edges. The floor slab is 120 mm throughout the structure. Further information on the dimensions, reinforcement detailing, material properties and construction process of the test structure are reported by Wight et al. [38]. Example detailing for a column, beam and shear wall are given in Appendix A.2.

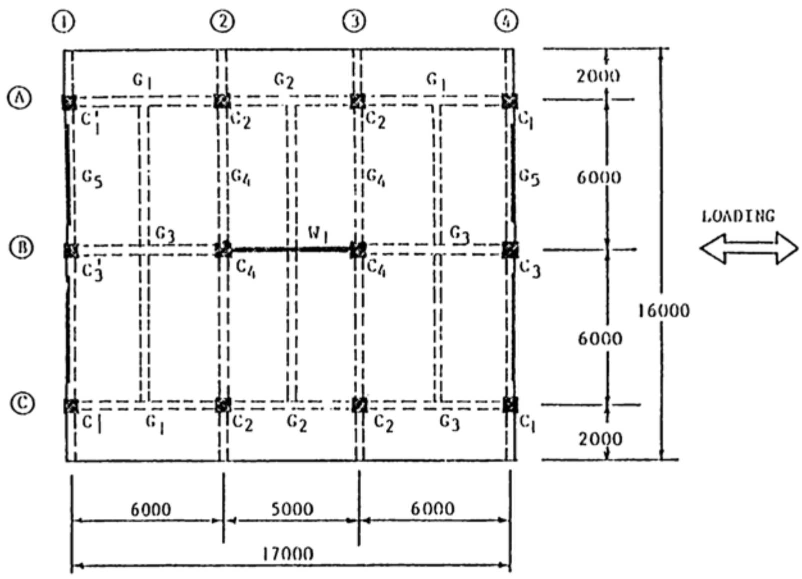

Figure 3.1. Plan of the Seven-Story Full-Scale Building [12]

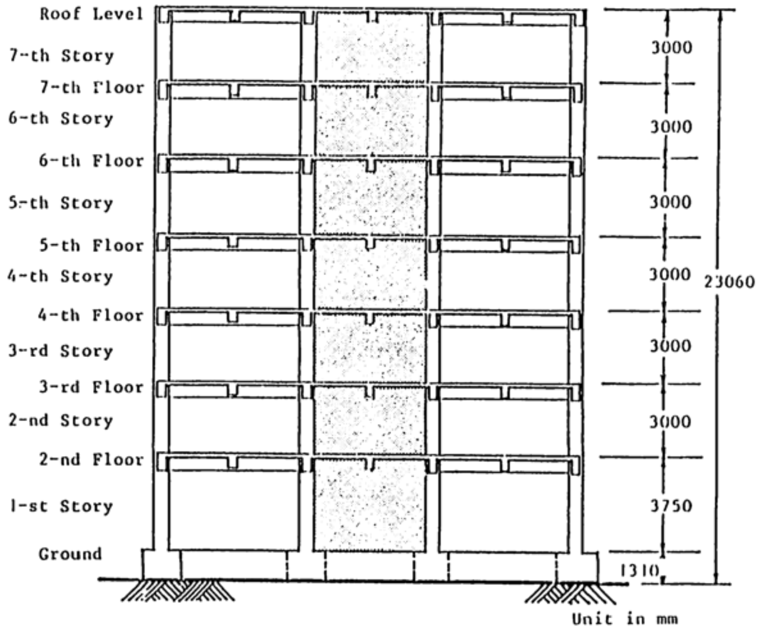

Figure 3.2. Side View of the Seven-Story Full-Scale Building [12]

The floor plan is rectangular, 17 m in the X-direction by 16 m in the Y-direction with a total floor area of 272 m^2. There are 3 frames in the X-direction and 4 frames in the Y-direction. The total height of the seven-story building is 21.75 m. The model is symmetrical in plan with respect to the centroidal X and Y axes.

The average compressive strength of concrete is 28.0, 28.90 and 16.30 MPa for Stories 1-4, Story 5 and Stories 6-7, respectively. The unit weight of concrete is 25 kN/m^3 and the modulus of elasticity of concrete, E_c, is 25000 MPa. The average yield strength of steel is 343 MPa (Wight et al. [38]).

3.3. BUILDING MODELS

The structural models used in the analyses were generated considering the parameters that are included in the database of buildings surveyed after Düzce (1999) and Bingöl (2003) Earthquakes by the Structural Engineering Research Unit (S.E.R.U.) of the Department of Civil Engineering in the Middle East Technical University (M.E.T.U.) [25]. The geometric properties of the mid-rise building models like the story height and total floor area were determined according to the average values obtained from this inventory, considering buildings having more than 3 stories, in order to reflect construction applications that are common in Turkey. Shear walls representing different wall area to floor area ratios and configurations were used on the same building plan in order to carry out this parametric study. Although the average values of the geometric properties were used in formation of building models, the design of these models was carried out following the Turkish Earthquake Code (TEC 2007) [28] requirements.

Twenty four building models that have five and eight stories with same floor plans but different shear wall areas and arrangements (Table 3.1) were generated for use in the nonlinear time-history analyses. Twenty models were examined for investigating the effect of shear wall ratio and two model pairs were examined to observe the effect of column dimensions and shear wall configuration on the seismic performance. Shear wall area to floor area ratio was determined by dividing total shear wall area in one direction to the floor plan area of one story ($\sum A_w/A_p$). In the building models, a range of wall area to floor area ratios between 0.51 and 2.17 percent was studied**.** Floor plans of model buildings with wall ratios of about 0.5 %, 1.0 %, 1.5 % and 2.0 % for both directions are given in Figures 3.3 to 3.6.

Table 3.1. Wall & Column Area to Floor Area Ratios of Model Buildings

Model ID	# of Stories	Shear Wall Ratio (%)		Column Ratio (%)
		X-dir	Y-dir	
1	5	0.51	0.51	1.25
2	5	0.51	1.01	1.06
3	5	1.01	1.01	0.88
4	5	0.51	1.52	0.88
5	5	1.01	1.52	0.69
6	5	1.55	1.52	0.51
7	5	0.51	2.12	0.88
8	5	1.01	2.12	0.69
9	5	1.55	2.12	0.51
10	5	2.17	2.12	0.51
11	8	0.51	0.51	1.25
12	8	0.51	1.01	1.06
13	8	1.01	1.01	0.88
14	8	0.51	1.52	0.88
15	8	1.01	1.52	0.69
16	8	1.55	1.52	0.51
17	8	0.51	2.12	0.88
18	8	1.01	2.12	0.69
19	8	1.55	2.12	0.51
20	8	2.17	2.12	0.51
21	5	1.01	1.01	0.49
22	5	1.01	1.01	1.37
23	5	1.01	1.06	1.11
24	5	1.01	1.06	1.02

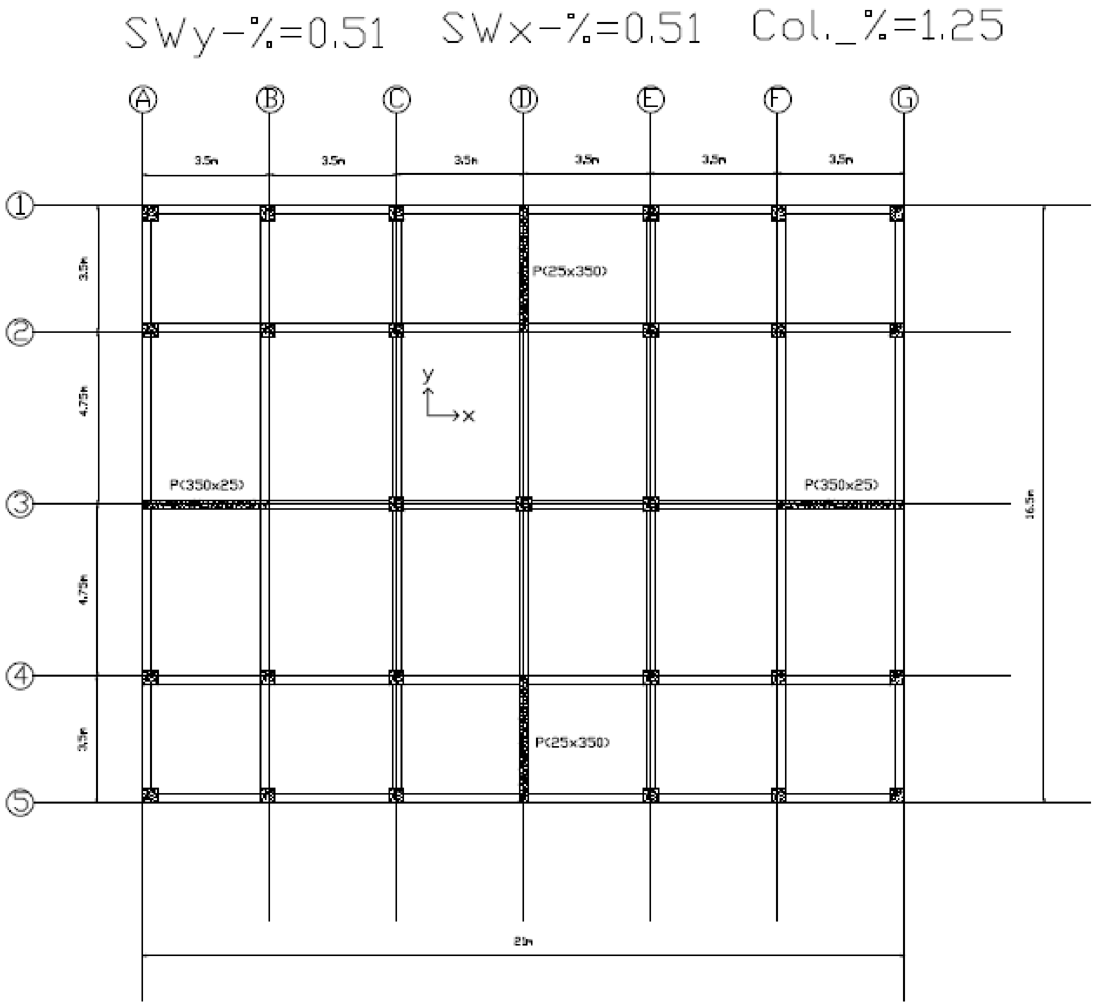

Figure 3.3. Plan of Model 1 with 0.5% Wall Area to Floor Area Ratio in Each Direction

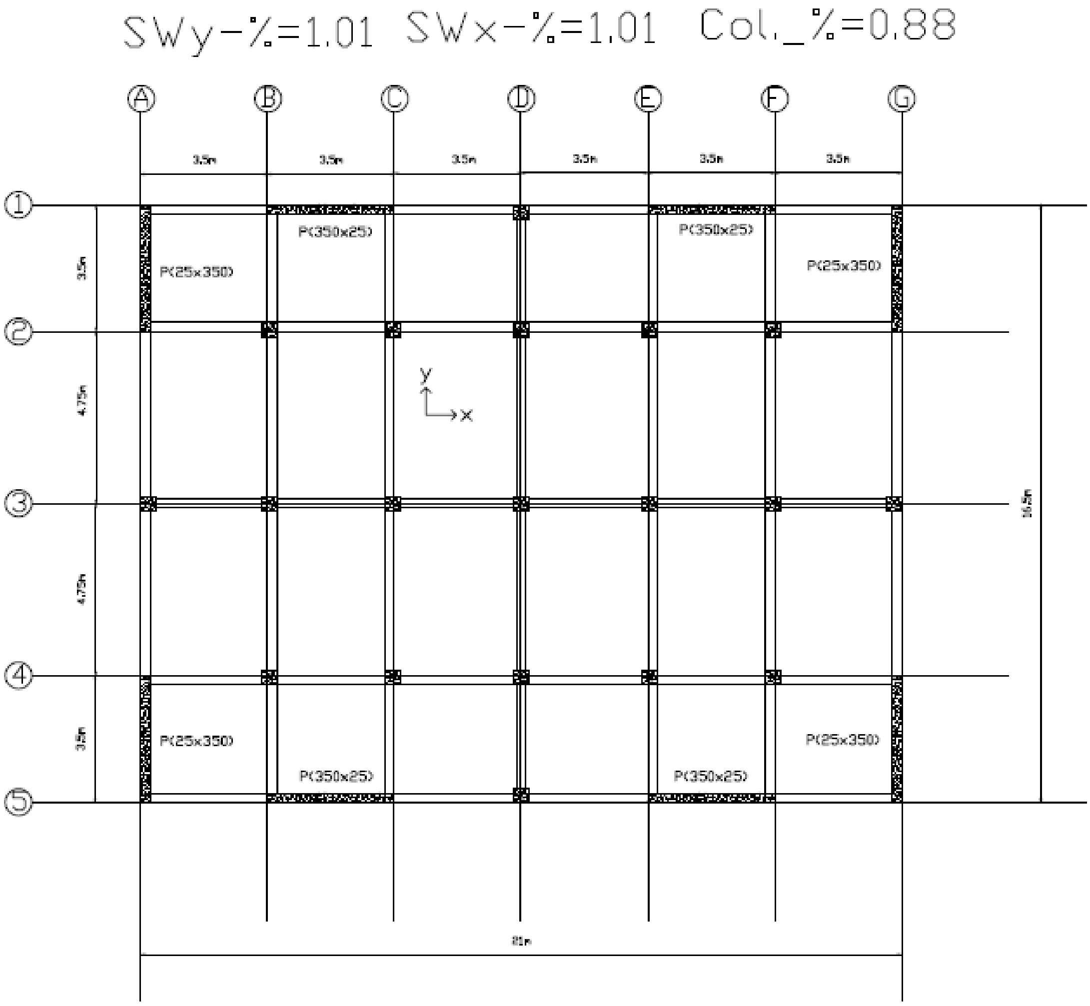

Figure 3.4. Plan of Model 3 with 1.0% Wall Area to Floor Area Ratio in Each Direction

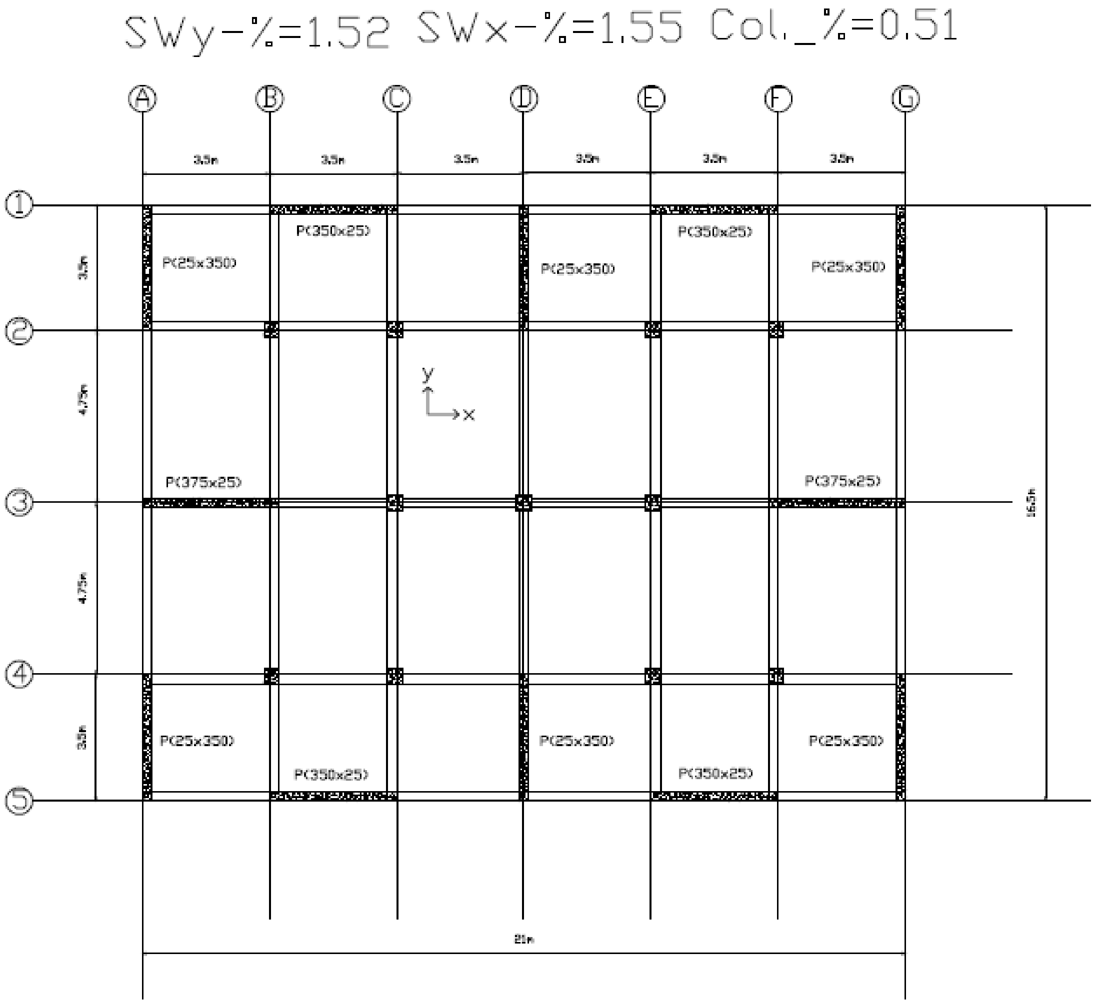

Figure 3.5. Plan of Model 6 with 1.5% Wall Area to Floor Area Ratio in Each Direction

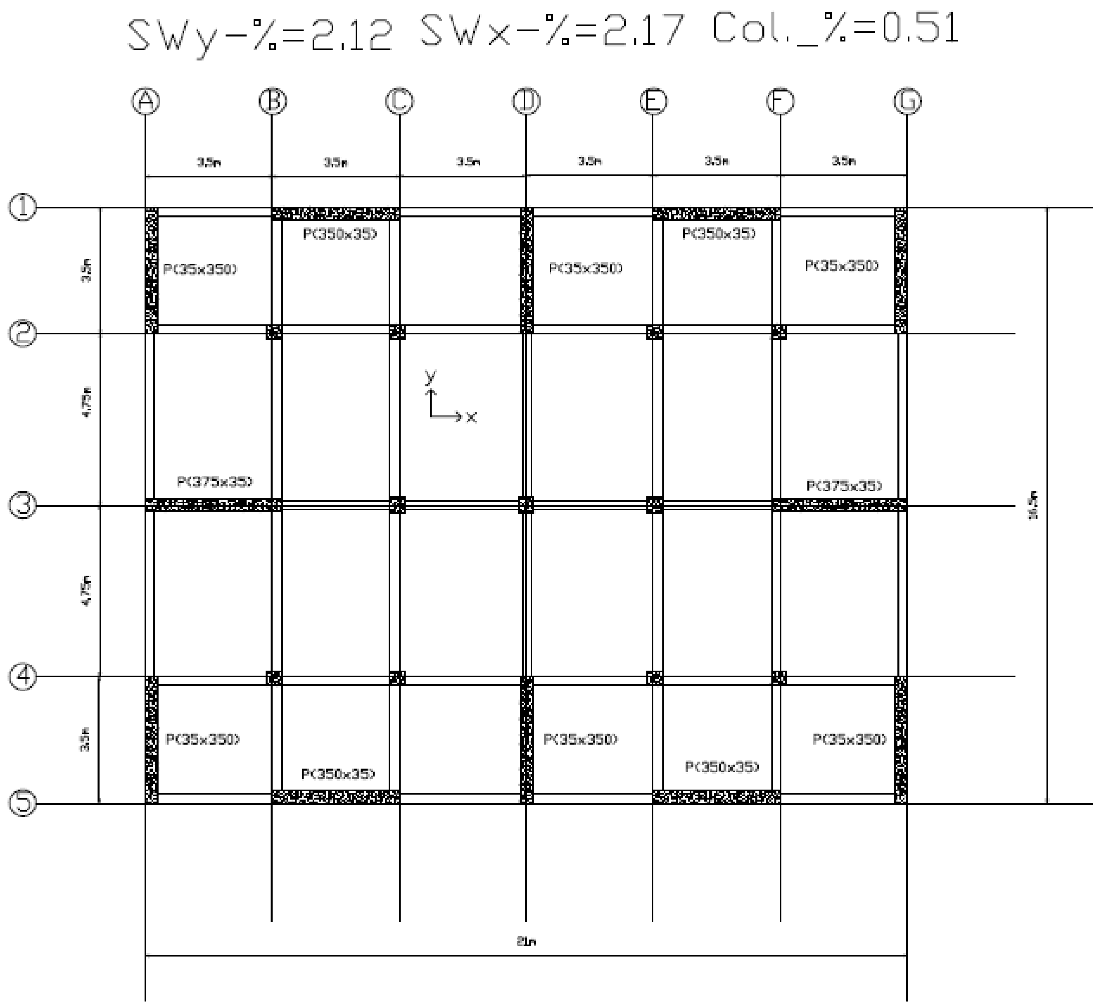

Figure 3.6. Plan of Model 10 with 2.0% Wall Area to Floor Area Ratio in Each Direction

The floor plan is rectangular, 21 m in the X-direction and 16.5 m in the Y-direction with a floor area of 346.5 m^2 in all models. There are 5 frames in the X- direction and 7 frames in the Y-direction. All models are symmetrical in plan with respect to centroidal X and Y axes.

All shear walls have rectangular cross sectional areas with mostly 3.5 m length. To have the specified wall area to floor area ratios, for some models 3.75 m shear walls are used in accordance with the floor plans. The standard shear wall thickness was taken as 250 mm for wall area to floor area ratios below 1.5 %. In order to obtain models with 2.0 percent shear wall ratio, the thickness was modified to 350 mm. This approach was followed to have similar shear wall configurations in between the model buildings.

To simulate the building stock in Turkey; 5 and 8 story buildings that have 3.2 and 2.9 m height for ground and typical stories, respectively, were generated for the study. Total height of the 5 and 8 story building models are 14.8m and 23.5m, respectively.

Except for two models, (Model 21&22), all the columns have square 400x400 mm. cross-sections. In models 21 and 22, in order to study the effect of the total area of the existing columns to floor area ratio on the seismic behavior of the building, the size of the columns was changed to 300x300 mm. and 500x500 mm, respectively. The beams have 250x400 mm. rectangular cross-sections in all models.

Rigid diaphragms were assigned to the slabs by introducing joint constraints at each story level. Design loads on the slabs were determined from TS 498 [29] and design of the slabs was performed by following the requirements of TS500 [30]. Slab thickness is 120 mm. in all models.

All columns, beams and shear walls were designed as highly ductile members based on TEC 2007 [28] requirements. IdeCAD STATIK v 6.0. [31] which is a reinforced concrete design program fully compatible with TEC2007, was used for design.

Concrete class and longitudinal reinforcement were chosen as C20 (f_{ck} = 20 MPa) and S420 (f_{yk} = 420 MPa), respectively. The modulus of elasticity of concrete, E_c was taken as 28-day modulus of elasticity value of C20 concrete (E_c = 28000 MPa) as mentioned in TS 500 [30].

Buildings were assumed to be located in the first seismic zone with Z1 soil type as defined in TEC 2007 [28]. Soil-structure interaction effects were neglected in the model. Therefore, fixed supports were used for the vertical elements at the ground floor level.

In this analytical study, shear wall area to floor area ratios of 0.5, 1.0, 1.5 and 2.0 percent were selected to investigate the dynamic behavior of mid-rise buildings. 2% shear wall ratio was selected as the maximum ratio to be examined in this study, because it is believed that after this value, addition of shear walls to the floor plan will not significantly improve the seismic behavior of a building. Moreover, in the analytical study by Soydaş [23], the behavior of mid-rise shear wall-frame structures was significantly deteriorated, therefore instead of investigating wall ratios beyond 2 percent, this study concentrated on studying a variety of shear wall configurations interacting with each other in two principle directions. Primary direction examined throughout this study was the Y-direction of the buildings models since this direction has more vulnerability to damage due to a lower number of frames as compared to the X-direction. Effect of the change in shear wall ratio in one direction on the dynamic response in the other direction was also investigated for some specific wall ratios. For this purpose, model buildings were designed to provide all possible wall ratios between 0.5 and 2 percent in X-direction for any selected wall ratio in Y-direction. For example, there are four different model buildings for a wall ratio of 2.0 percent in Y-direction, having 0.5%, 1.0%, 1.5% and 2.0% of shear wall ratios in X-direction.

The effects of total column area and shear wall layout on the dynamic performance of the structures were evaluated in two additional models. These models were designed as 5 story buildings having 1.0 percent shear wall to floor area ratio in both directions. 5 story buildings were selected for these models, because observations made by Canbolat et al. [4] indicated that these buildings have more vulnerability when compared to 8 story buildings in overall dynamic behavior. Additionally, in current practice, 1.0 percent shear wall by area in each direction is accepted as a rule of thumb to be sufficient for earthquake resistant design.

The column dimensions of Model 3 which has column size of 400x400 mm, were modified to 300x300 mm and 500x500 cm to create models Model-21 and Model-22, respectively in order to investigate the effect of total column area to floor area ratio on the seismic behavior. The models created to observe the influence of shear wall configuration are also generated from Model-3. To have the same layout, the locations of shear walls in the X-direction were not changed. In Model-23 and Model-24, 3 shear walls with 350 mm thickness were placed in central inner frames of the building in Y-direction. The difference of Model-23 and Model-24 is that shear walls were placed on 3 different frames in the former model, where all the walls lie on a single frame in the latter one. The floor plans for building models with different shear wall configurations are given in Figures 3.7 and 3.8. The plans of the remaining models are represented in Appendix A.1.

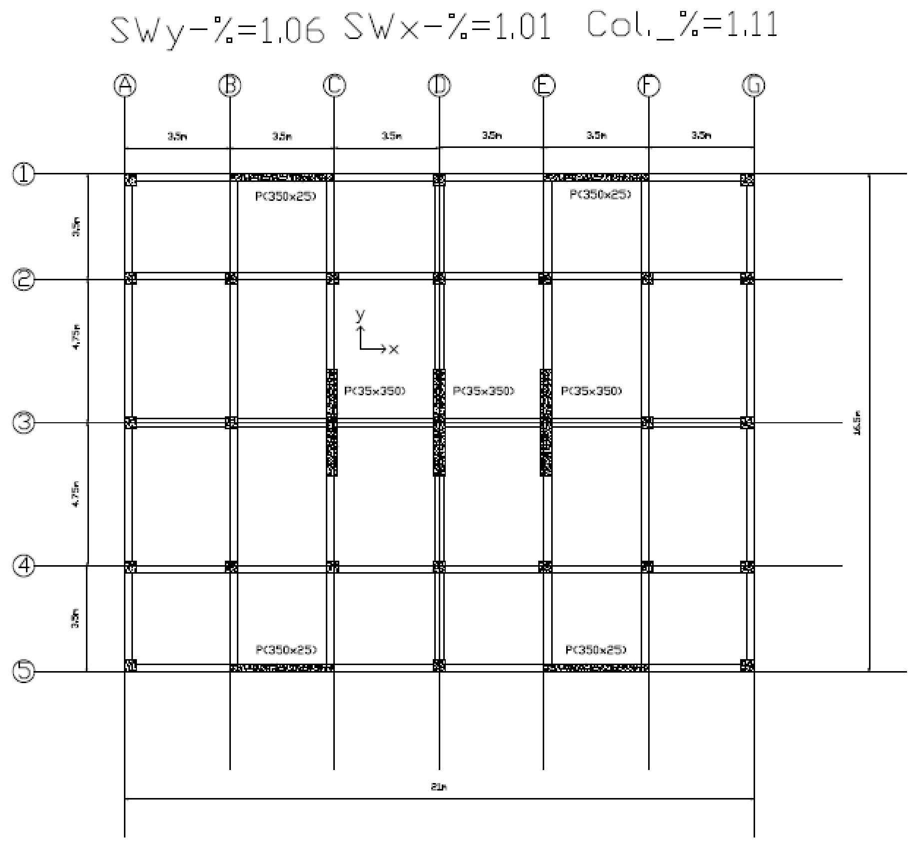

Figure 3.7. Plan of Model 23 with Different Wall Configuration and 1.0% Wall Area to Floor Area Ratio in Each Direction

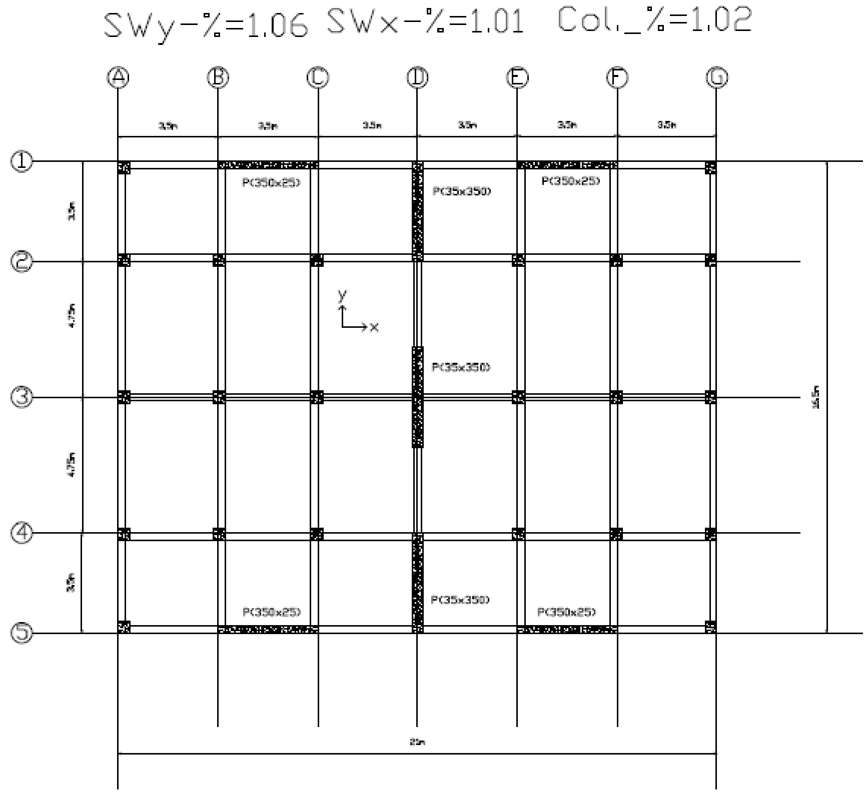

Figure 3.8. Plan of Model 24 with Different Wall Configuration and 1.0% Wall Area to Floor Area Ratio in Each Direction

3.4. ELEMENT MODELING

In the analysis of reinforced concrete shear wall-frame buildings, the structure is defined as a set of various members. In current practice, detailed inelastic beam, column and shear wall members are used in structural analysis and with the help of sophisticated computer aided programs, more accurate analytical models are achieved. In this study, PERFORM 3D v 4.0 [5] which is a nonlinear static and dynamic analysis program is used.

The accuracy of an analytical model is essential for both design and analysis. In order to check the consistency of the analytical model utilized during this study, first the full-scale seven-story reinforced concrete shear wall building tested for U.S.-Japan Cooperative Research Program was modeled. In order to use an efficient analytical model for the shear walls throughout the study, two approaches which are wide column analogy and fiber section method were examined. The details of these two modeling techniques are described in Chapter 2 under the title of “Modeling of Shear Walls”. The comparison of two models shows that wide-column analogy which is easier to implement than fiber section method is more effective to use, since not only it takes less time to implement but also it gives as good results as a much more detailed model which is fiber section method. Details on the verification of the model are given in the following chapter under the title of “Analytical Results for the Seven-Story Full-Scale Test Building”. Therefore, wide column analogy is used in shear wall member modeling. Since the analytical results using the proposed model were consistent with the experimental results of the test building, the building models generated for the parametric study were modeled with the same analytical model under the selected earthquake ground motion records. The analytical results gave information on the overall building response including modal response parameters, roof drifts, interstory drifts and base shear responses and the distribution of inelastic activity throughout the structure.

In this section, the main parameters used in each individual analytical member model for beams, columns and shear walls are explained in detail.

3.4.1. Beam Element

The beam element was modeled as an elastic segment with zero-length moment hinges at the column faces and rigid end zone elements within the column, as illustrated in Figure 3.9. The rigid end zone length was selected as half of the column width.

Figure 3.9. Beam Element

The main parameters that are required to define the elastic beam behavior are section dimensions, moment of inertia, I, modulus of elasticity, E, and Poisson's ratio, ν. The moment of inertia was taken as the cracked moment of inertia and set equal to the 40% of that for gross section as mentioned in TEC 2007 [28]. Effective slab width was also taken into account while computing the gross moment of inertia of the beam section. The modulus of elasticity was obtained from the report by Wight [38] for the seven-story full-scale test building and from TS500 [30] for the models generated for this analytical study. The Poisson's ratio was taken as 0.2 for all building models. The ultimate positive and negative bending moment capacities of the beam was determined by performing moment-curvature analysis using a commercial program called XTRACT [39] taking into account the slab participation. An example moment curvature analysis of a beam in a generated building model is given in Appendix A.3. Additionally, the information on stiffness and ultimate moment capacity of the beams for the seven-story full-scale building are given in Appendix A.4. Rotations corresponding to key moment values (Figure 3.10) were obtained considering the proposed values in Table 6.7 (Table 3.2) of FEMA 356 [8] for the modeling parameters in Figure 3.10. The numerical values of these parameters commonly used for beams are given in Table 3.3. These parameters varied for some of the beams due to the existing tension and compression reinforcement ratio.

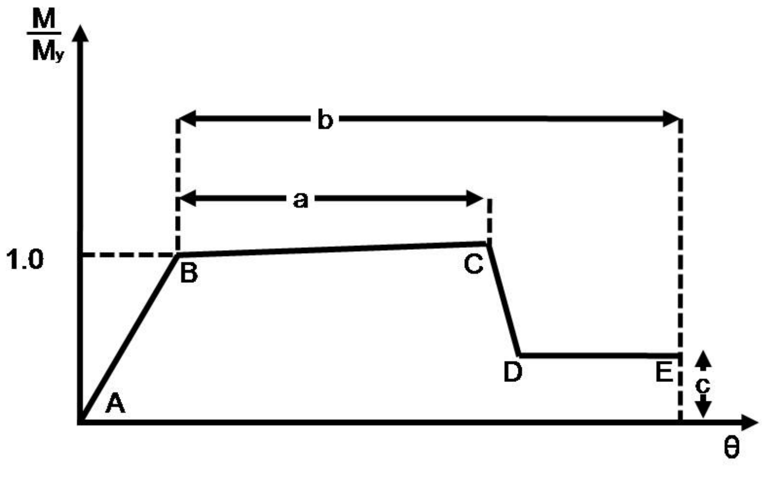

Figure 3.10. Beam Moment versus Rotation Relationship of FEMA 356 [8]

Table 3.2. Modeling Parameters for Reinforced Concrete Beams of FEMA 356 [8] (Table 6.7)

Conditions			Modeling Parameters[3] Plastic Rotation Angle, radians a	b	Residual Strength Ratio c
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
≤ 0.0	C	≥ 6	0.02	0.04	0.2
≥ 0.5	C	≤ 3	0.02	0.03	0.2
≥ 0.5	C	≥ 6	0.015	0.02	0.2
≤ 0.0	NC	≤ 3	0.02	0.03	0.2
≤ 0.0	NC	≥ 6	0.01	0.015	0.2
≥ 0.5	NC	≤ 3	0.01	0.015	0.2
≥ 0.5	NC	≥ 6	0.005	0.01	0.2

Table 3.3. Modeling Parameters Commonly Used for Beams

Element Type	Modeling Parameters Based on FEMA 356		
	a	b	c
Beam	0.025	0.05	0.2

Different energy dissipation coefficients were specified at different critical rotation values to account for stiffness deterioration. Based on the dissipation factors, the software reduces the area within the hysteresis curves proportional to the dissipation factor. For beams, the energy dissipation coefficient was set to 0.3 up to the point where the beam reaches its ultimate moment strength, and 0.2 after the beam started to loose its strength.

3.4.2. Column and Shear Wall Elements

The column element was modeled as an elastic segment with zero-length moment hinges at the beam faces and rigid end zone elements within the beam, as illustrated Figure 3.11. The rigid end zone was selected as half of the beam height for this element. However, in ground story columns, no rigid end zone was defined at the bottom end of the model since no beam exists at this level. The shear wall element was modeled by the same way as the column element; however zero-length moment hinge was defined only at the bottom end of the element for each story (Figure 3.12). Rigid links were used to connect structural walls to the beams.

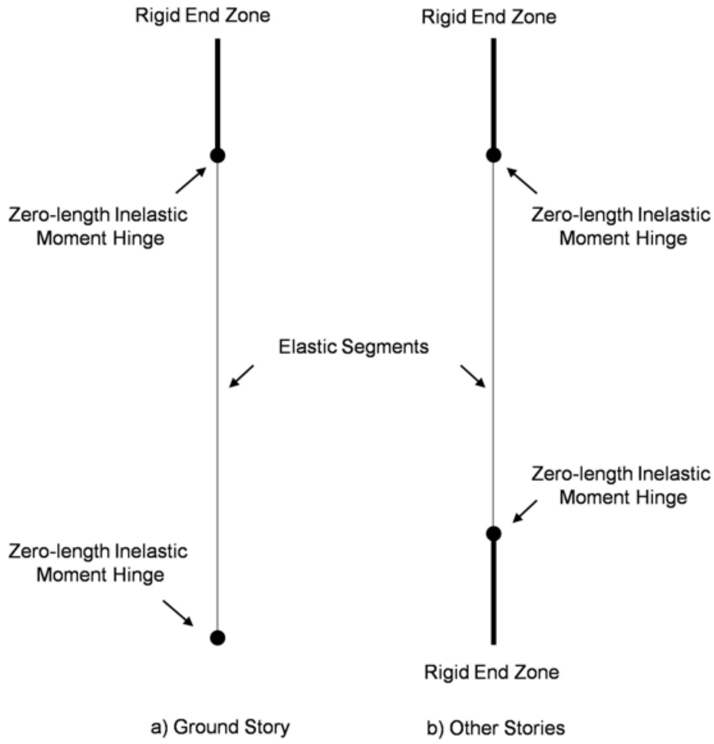

Figure 3.11. Column Element

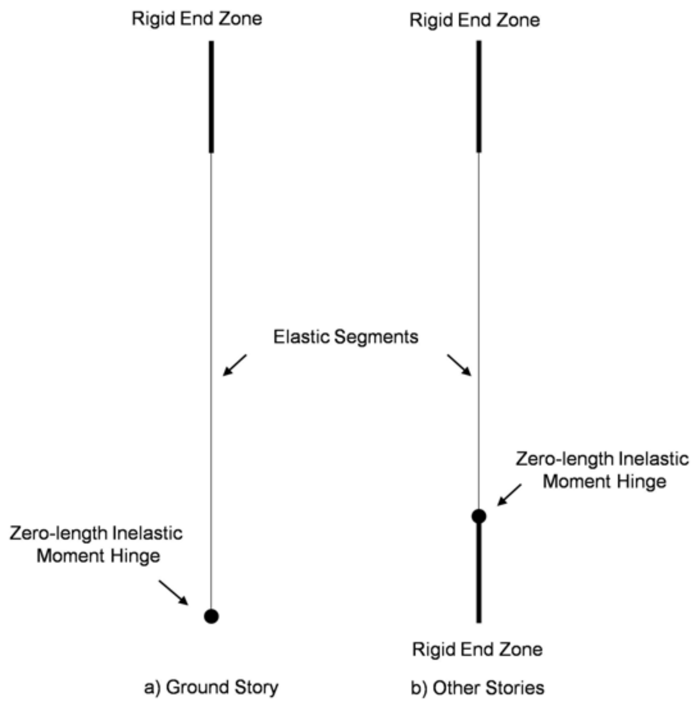

Figure 3.12. Shear Wall Element

The zero-length moment vs. axial load rotation element in PERFORM 3D [5], namely PMM hinge, was selected for modeling the column and shear wall nonlinear hinge behavior.

The main parameters that are required to define the elastic regions of column and shear wall are section dimensions, moment of inertia, I, modulus of elasticity, E, and Poisson's ratio ν. The moment of inertia was taken as the cracked moment of inertia which was calculated according to TEC 2007 [28] considering the amount of axial force carried by the vertical elements and ranged between 40 to 80 percent of the gross value. To define the inelastic properties of PMM hinges, balanced moment capacities of the columns in each principal direction, pure axial compression and concentric axial tension capacities are necessary for each member as shown in Figure 3.13. The axial load capacity values and two other parameters which were used in defining the shape of relationship between moment and axial load were used to define the yield surface. The axial load-moment interaction surface analyses were performed by commercially available program XTRACT [39]. Examples of PMM analyses of a column and a shear wall in a generated building model are represented in Appendix A.5. In addition, the stiffness, balanced moment capacity, pure axial compression and concentric axial tension capacities of columns and shear walls for the seven-story full-scale building are given in Appendix A.6. Rotations corresponding to key moment values (Figure 3.14) were determined from the proposed values in Table 6.8 (Table 3.4) and 6.18 (Table 3.5) of FEMA 356 [8] for columns and shear walls, respectively. The numerical values of modeling parameters commonly used for columns and shear walls are given in Table 3.6. It is worth to mention that due to the increase in axial load levels acting on the members, for example in Model 21 in which the column dimensions were chosen as 30x30 cm, different modeling parameters were used in accordance with the tables proposed by FEMA 356 [8].

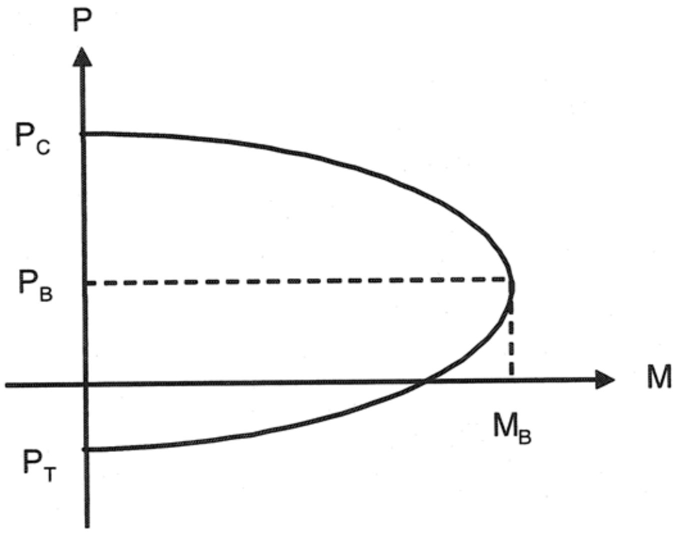

Figure 3.13. Column and Shear Wall Axial Load versus Moment Relationship

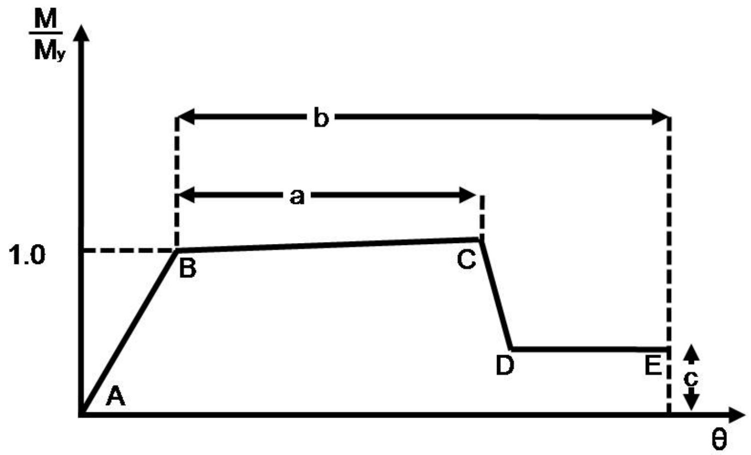

Figure 3.14. Column & Shear Wall Moment versus Rotation Relationship of FEMA356 [8]

Table 3.4. Modeling Parameters for Reinforced Concrete Columns of FEMA 356 [8] (Table 6.8)

Conditions			Modeling Parameters[4]		
			Plastic Rotation Angle, radians		Residual Strength Ratio
			a	b	c
i. Columns controlled by flexure[1]					
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$			
≤ 0.1	C	≤ 3	0.02	0.03	0.2
≤ 0.1	C	≥ 6	0.016	0.024	0.2
≥ 0.4	C	≤ 3	0.015	0.025	0.2
≥ 0.4	C	≥ 6	0.012	0.02	0.2
≤ 0.1	NC	≤ 3	0.006	0.015	0.2
≤ 0.1	NC	≥ 6	0.005	0.012	0.2
≥ 0.4	NC	≤ 3	0.003	0.01	0.2
≥ 0.4	NC	≥ 6	0.002	0.008	0.2

Table 3.5. Modeling Parameters for Reinforced Concrete Shear Walls of FEMA 356 [8] (Table 6.18)

Conditions			Plastic Hinge Rotation (radians)		Residual Strength Ratio
			a	b	c
i. Shear walls and wall segments					
$\frac{(A_s - A'_s) f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]			
≤ 0.1	≤3	Yes	0.015	0.020	0.75
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40
≥ 0.25	≤ 3	Yes	0.009	0.012	0.60
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30
≤ 0.1	≤ 3	No	0.008	0.015	0.60
≤ 0.1	≥ 6	No	0.006	0.010	0.30
≥ 0.25	≤ 3	No	0.003	0.005	0.25
≥ 0.25	≥ 6	No	0.002	0.004	0.20

Table 3.6. Modeling Parameters Commonly Used for Columns and Shear Walls

Element Type	Modeling Parameters Based on FEMA 356		
	a	b	c
Column	0.02	0.03	0.2
Shear Wall	0.015	0.02	0.75

A bilinear relationship was used for moment vs. rotation and a linear one for axial load vs. displacement, with the ultimate values of balanced moment and pure axial compression, respectively. For columns and shear walls, an energy dissipation coefficient of 0.5 was taken at the yield point. This value was reduced to 0.3 at the ultimate point to account for stiffness deterioration and remained constant for larger rotations.

3.5. SELECTION OF EARTHQUAKE RECORDS

3.5.1. Earthquake Record for Seven-Story Full-Scale Test Building

The seven-story full-scale building that does not have non-structural partition walls was subjected to lateral load reversals of inverted triangular distribution. The equivalent single-degree-of-freedom pseudo-dynamic earthquake response test procedure (SPD Test Method) was used to control the roof level displacement. Four test runs (tests SPD-1 through SPD-4) were carried out with varying roof displacements of approximately 1/8630, 1/670, 1/91 and 1/64 of the total height. The details of the test procedure and the observed behavior of the test structure are given by Kaminosono et al. [13] and Yoshimura et al. [40].

In this study, the structural behavior of the test SPD-3 was examined since the obtained data is more detailed in terms of roof displacement and base shear time histories which gives the opportunity to verify the analytical model. In the test SPD-3, a modified version of Theachapi Shock (1952) of Taft Record (EW) having a peak acceleration of 319.80 cm/s^2, the time history of which is given in Figure 3.15, was applied to the building.

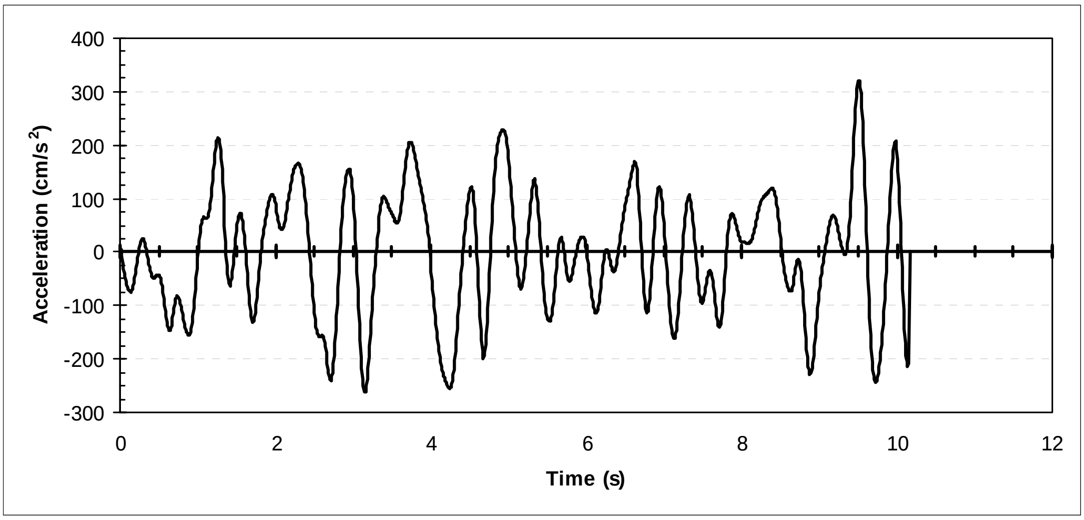

Figure 3.15. Acceleration Time History for the Taft Record

3.5.2. Earthquake Records for Generated Building Models

Seven earthquake ground motions with different characteristics and intensities were used in the inelastic dynamic time history analyses of the generated building models. In Turkish Earthquake Code (TEC2007 [28]), it is stated that at least three earthquake records should be used in time history analysis of buildings provided that the maximum response is taken into consideration. If seven earthquake records are applied to the structure, the average response could be used in the evaluations. In this study, in order to observe the seismic behavior under earthquake records with different characteristics, the latter approach is selected. The time history records were obtained from the database of Pacific Earthquake Engineering Research Center (PEER) [20]. The properties of the selected ground motions are given in Table 3.7 including the earthquake magnitude, distance from the site to the epicenter and the peak ground acceleration values.

Table 3.7. Properties of the Selected Ground Motion Records

Earthquake, Station	Magnitude	Distance	PGA (g)	
			X-Comp	Y-Comp
Kocaeli, Izmit (1999)	7.4	4.8	0.152	0.220
Imperial Valley, El Centro (1940)	6.9	10	0.229	0.336
Erzincan, Erzincan (1992)	6.7	2	0.425	0.476
Duzce, Duzce (1999)	7.1	8.2	0.348	0.535
Chile, Valpariso (1985)	8	42	0.542	0.564
Kobe, Takarazuka (1995)	6.9	3.4	0.693	0.694
Northridge ,Sylmar (1994)	6.7	6.4	0.575	0.825

In this study, the effect of earthquake intensity on the building response was studied. For this purpose, except for one record, the component of the record that has a higher PGA value was applied in the Y-direction. Because in this direction there are lower number of frames which could lead to a potential vulnerability compared to lateral loading in the X-direction. For the Valpariso record of Chile Earthquake, the component with lower peak ground acceleration was applied in the Y-direction of the building model. In this record, a pulse effect was observed in the response spectrum analysis of X-component, besides there is not much difference in PGA values in between two directions.

The corresponding elastic response spectra of the selected ground motions are shown in Figure 3.16.

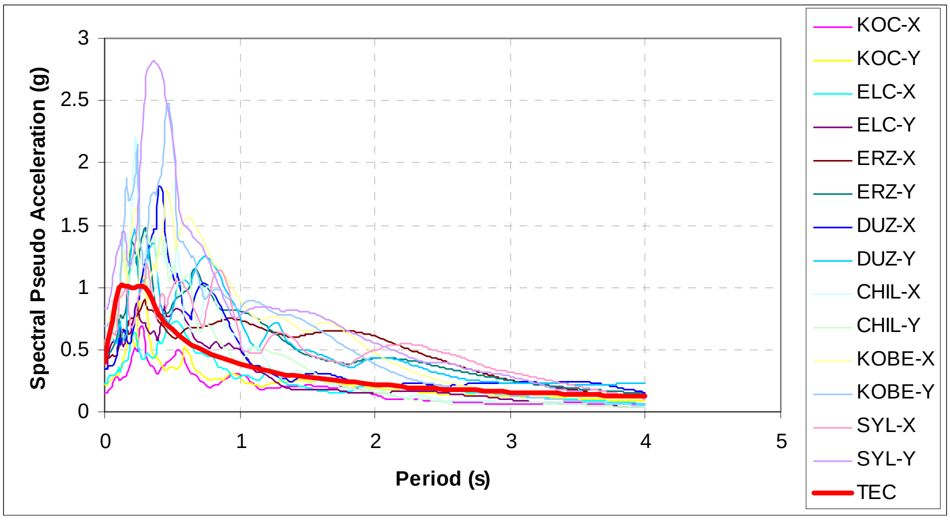

Figure 3.16. Response Spectra of the Selected Earthquake Records

In Figure 3.16, the elastic response spectrum generated for the building models according to TEC2007 [28] is also plotted with a bold line. In the legend, the abbreviations TEC, KOC, ELC, ERZ, DUZ, CHIL, KOBE and SYL stand for the Turkish Earthqauke Code, Kocaeli, El Centro, Erzincan, Duzce, Chile, Kobe and Sylmar records, respectively.

Section 2.9 of the Turkish Earthquake Code (TEC2007 [28]) states that total duration of the earthquake record should not be less than neither 5 times of the natural vibration period of the building nor 15 seconds. In addition, the average of the spectral acceleration value of the records corresponding to the period of zero seconds is restricted with A_0g, where A_0 is the effective acceleration coefficient and g is the gravitational acceleration.

Moreover, the average of the spectral acceleration values of the records considered with 5% damping ratio should not be less than 90 percent of the elastic spectral acceleration values in the interval of 0.2 and 2 times of the first dominant period of the structure in the intended earthquake direction. At this point, only the X and Y components of the Kocaeli record and X component of the El Centro record does not fulfill the consideration mention above. The reason for selecting Kocaeli record was to investigate the seismic behavior in lower intensities. Besides, it is worth to mention that, in this study, these records were utilized neither for design nor seismic evaluation purposes mentioned in the code.

CHAPTER 4

ANALYTICAL RESULTS

4.1. ANALYTICAL RESULTS FOR THE SEVEN-STORY FULL-SCALE TEST BUILDING

In order to select the most accurate analytical model that reflects the real behavior as much as possible and does not require prolonged computational time, two different shear wall models were examined. The first model considered in this study is the wide-column analogy model which is commonly used in design offices as well as in academic studies. The second one is the fiber model of PERFORM 3D [5] in which the features of the model enable users to take advantage of modeling a shear wall with more detailed parameters. However, the use of this model elongates the required run time and the results depend extensively on the inelastic properties defined during modeling.

Top story displacement and base shear time histories of the analytical model, constructed by PERFORM 3D [5], in which shear walls were modeled by wide-column analogy are represented in Figure 4.1 and 4.2, respectively. The deviations from the real behavior until the peak roof displacement and base shear results from the difference in the initial stiffness of the test structure and the building model. Due to the limitations of the analysis program, only constant rigidity can be assigned to the elastic members without any degradation. Since the initial period of the model does not match well with that of the test building, till the peak values are reached the results are not very accurate. However, due to some initial cracking of the test building reported by Kabeyasawa et al. [12] this difference in initial stiffness is expected, since this cracking was not considered in the analytical model. After the fourth second when the inelastic activity starts to dominate the overall behavior, both time histories show fairly good matches in behavior checking the consistency of the proposed analytical model.

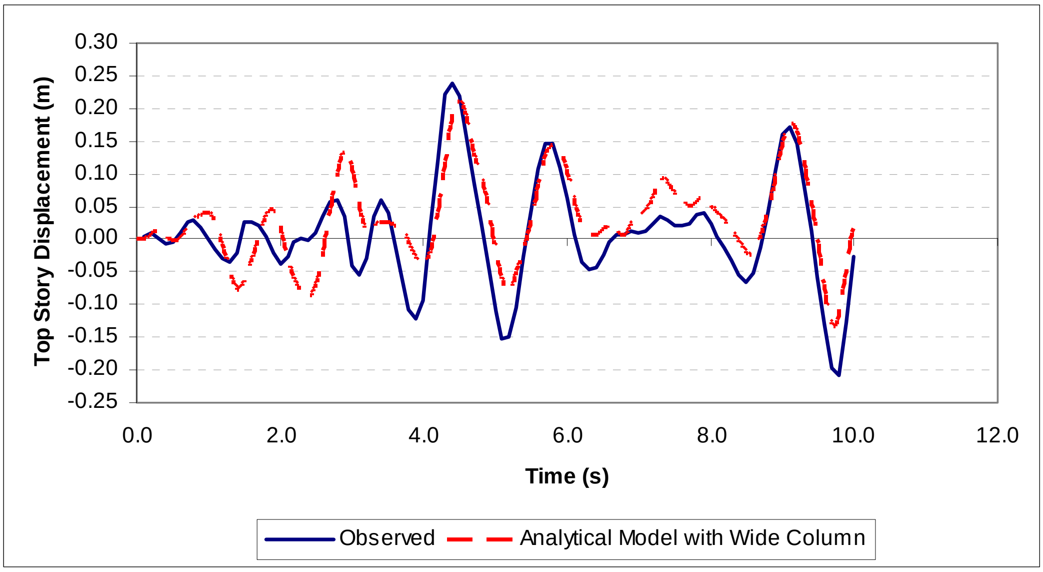

Figure 4.1. Top Story Roof Displacement Time History of Analytical Model with Utilization of Wide-Column Analogy for Shear Walls

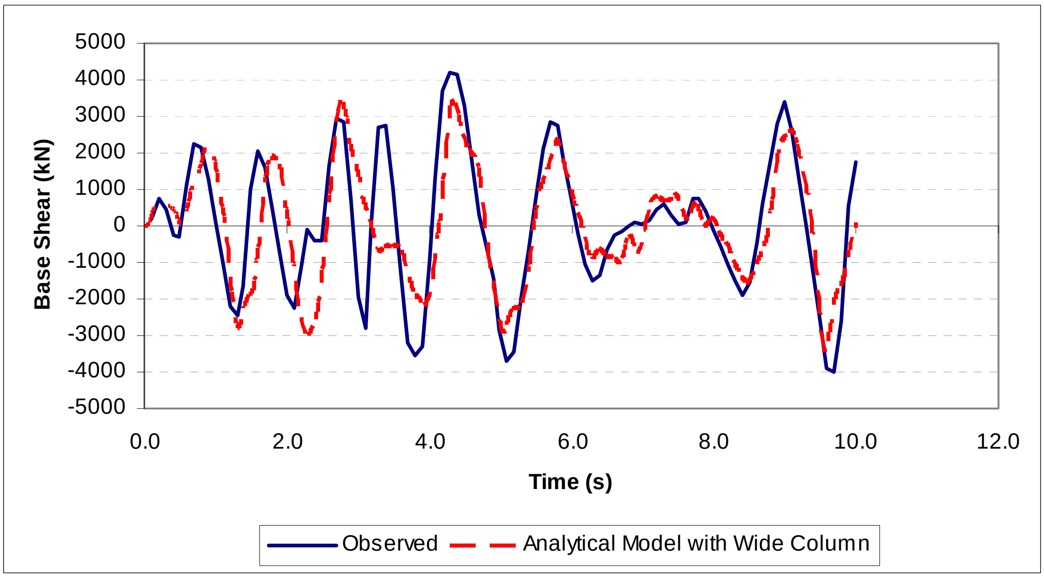

Figure 4.2. Base Shear Time History of Analytical Model with Utilization of Wide-Column Analogy for Shear Walls

The comparison of the top story displacement and base shear response of the two selected analytical models, wide-column and fiber models, are given in Figure 4.3 and 4.4, respectively. Although the initial stiffness of the shear wall member generated by the fiber model was not modified by the cracked section properties, the predicted behavior for the early stages of the time history was similar to the model with wide column. This similarity is due to the low shear wall ratio, around 0.33 percent, which had a negligible effect on the behavior. In addition, the initial stiffness of the beams and columns were taken as the cracked section properties and the overall behavior was dominated by the behavior of these members. The analytical results considering these two analytical models do not show any significant difference in predicting the behavior. The behavior trends as well as the magnitudes are similar for both models except for some short time intervals during which the influence on the overall behavior is negligible.

After the comparison of the analytical results obtained using two different shear wall models with the experimental ones, wide-column analogy model was selected to be utilized in the rest of the analytical study. Because, this model was as effective as the fiber model in predicting the nonlinear behavior and its implementation was much easier.

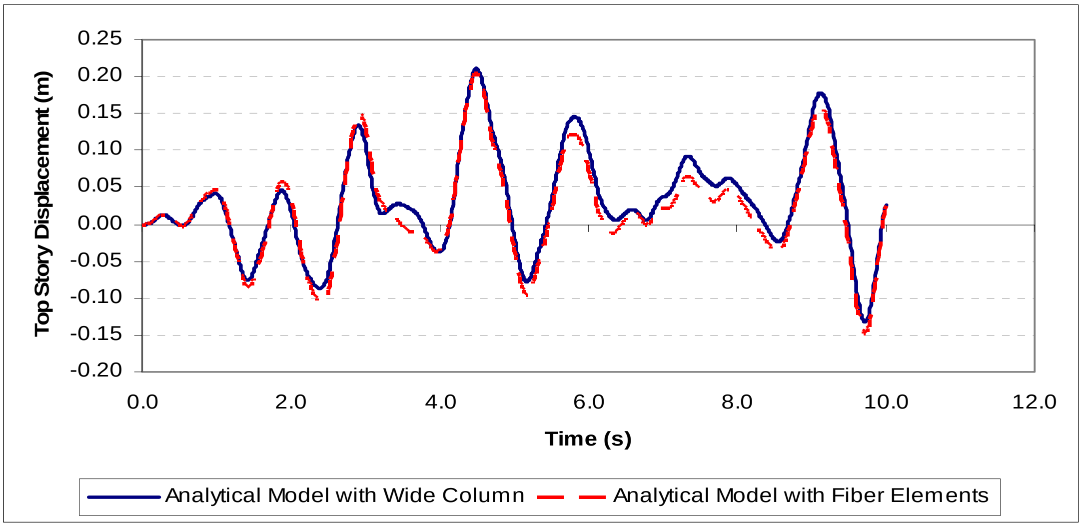

Figure 4.3. Comparison of Top Story Displacement Response of the Analytical Models

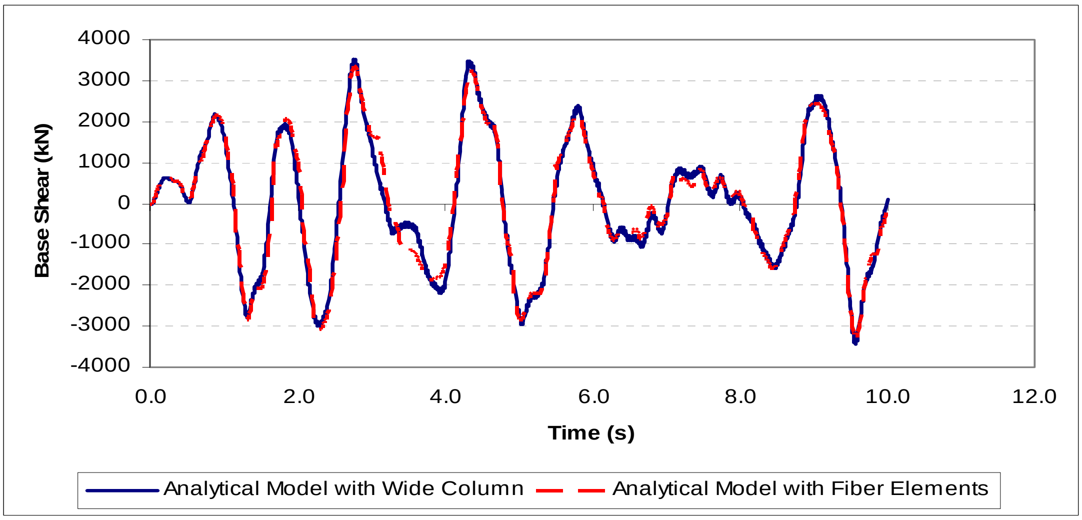

Figure 4.4. Comparison of Base Shear Response of the Analytical Models

4.2. ANALYTICAL RESULTS FOR THE BUILDING MODELS

The nonlinear time history analysis of the building models were performed by using the commercially available software program PERFORM 3D [5]. Each individual model was subjected to 7 different earthquake records and, the average of the data was taken for each control parameter in the evaluation of the results as required by TEC2007 [28]. The orthogonal components of the records were applied to the structure simultaneously to have a realistic overall response.

4.2.1. Variation of Modal Periods

Variation of fundamental building periods with changing shear wall ratios is given in Figure 4.5. The results are approximated by second order polynomial trend lines for each building model with different number of stories and in each direction. The results of the modal analyses reveal that the first two modes are the dominant ones for the building response in both principal directions. As it can be observed from this figure, the variation of building period with changing shear wall area to floor area ratio is significant for lower wall ratios and then its effect degrades. At some selected wall ratios, such as 1.0 %, there is more than one data point for the building period in

Figure 4.5. Because, the effect of the change in shear wall ratio in the X-direction on the dynamic behavior of the building in the Y-direction, which is the primary direction examined in this study, was investigated for the selected wall ratios. For this purpose, building models were designed such that there is a range of wall ratios from 0.5 to 2.0 percent in the X-direction, for a selected wall ratio in the Y-direction. For example, there are four different building models for a shear wall ratio of 2.0 percent in the Y-direction, having 0.5%, 1.0%, 1.5% and 2.0% shear wall ratios in the X-direction leading to four data points.

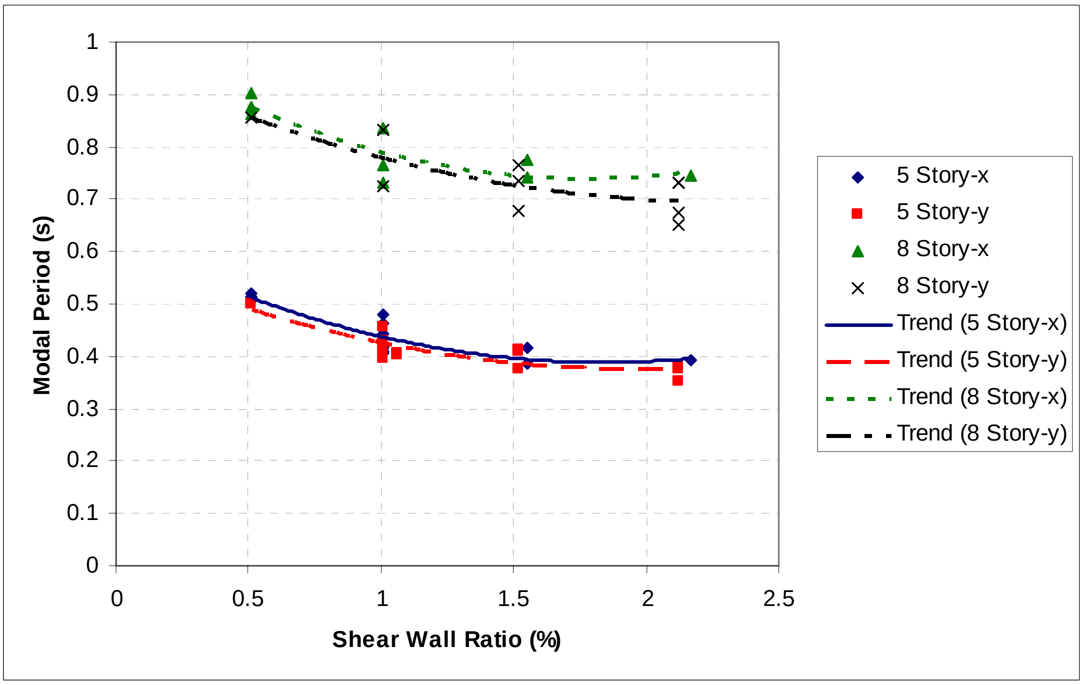

Figure 4.5. Variation of Modal Period with Shear Wall Ratio

4.2.2. Base Shear Carried by Shear Walls

Variation of base shear force ratio with changing shear wall ratios of the building models is given in Figure 4.6. The data points on this graph represent the percentage of the average shear force carried by the ground floor shear walls to the total base shear at the instant when the maximum base shear was observed for the seven selected earthquake records. The results are represented by trend lines passing through the average ordinates of the computed percentage of base shear force carried by the shear walls for different shear wall ratios, for each building model with different number of stories, in each direction. The percentage of the total shear force carried by the walls in the X-direction is slightly higher than that of the Y-direction based on the fitted lines. The number of stories does not significantly effect the contribution of the shear walls. Except for one model, Model-22 with 500x500 cm columns, shear wall contribution in carrying the base shear for the buildings with 1.0 % wall ratio is observed to be higher than 80%. For Model 22 this percentage was 72.5%, due to the intended selection of columns with higher stiffness. More than 90% of the base shear is carried by the walls for buildings with wall ratios of 1.5% and 2.0%. This outcome verifies the implementation in FEMA 356 [8], which states that in design of shear walls, 100% of the shear force is assumed to be carried by the walls in the shear wall-frame structures, although 25% of the total force is also carried by columns.

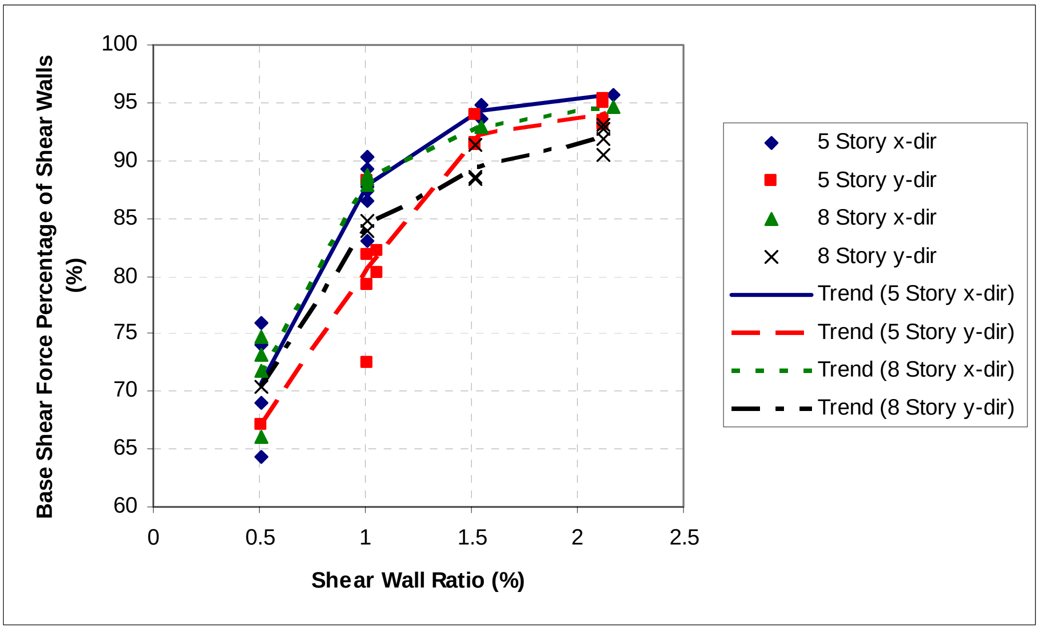

Figure 4.6. Shear Wall Contribution to Base Shear versus Shear Wall Ratio

As expected, the base shear force acting on shear walls decreases when the dimensions of the columns increase. However, the change in base shear force contribution of the shear walls is more significant in Y-direction as compared to X-direction (Figure 4.7). This could be due to the lateral rigidity of the frames in Y-direction. Since the frames in X-direction are more rigid when compared to the ones in the transverse direction, the change in column dimensions in two perpendicular directions does not have a similar effect on the overall behavior.

As explained in Chapter 3, models 3, 23 and 24 were designed to examine the effect of shear wall configuration on the seismic behavior of the building. From the analytical results, it was observed that, the effect of different shear wall configurations for a given floor plan is negligible on the contribution of shear walls in carrying the shear forces at the ground level, if there is no significant floor torsion. However, the level of the base shear percentage carried by the walls differs when two different loading directions are considered (Figure 4.8). This can be explained by the higher lateral rigidity of the frames in X-direction compared to Y-direction, which might have led to carrying higher shear forces in that direction.

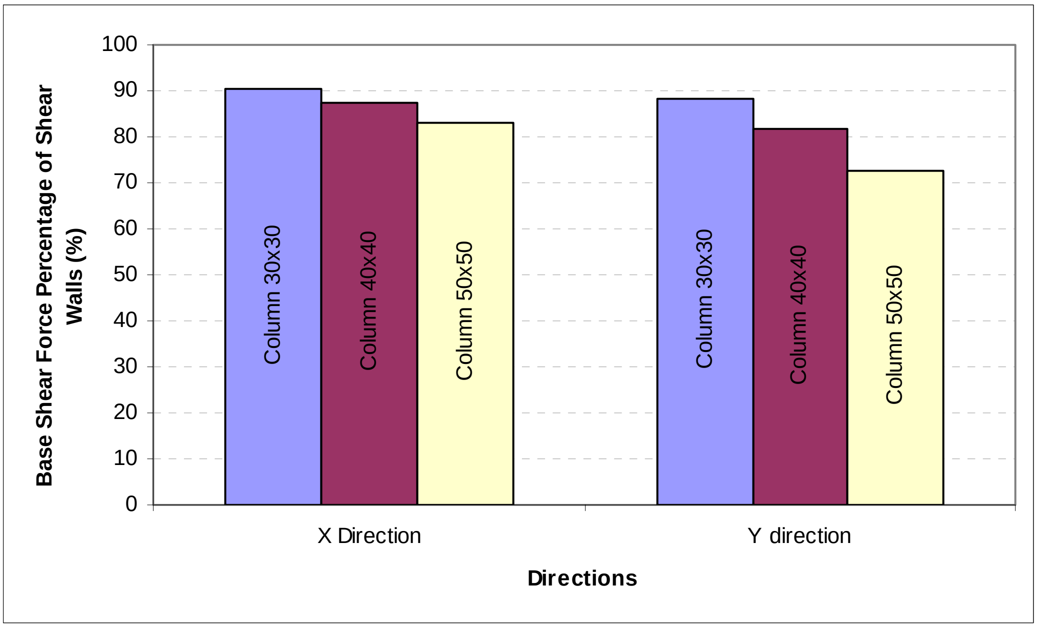

Figure 4.7. Shear Wall Contribution to Base Shear for Varying Column Dimensions

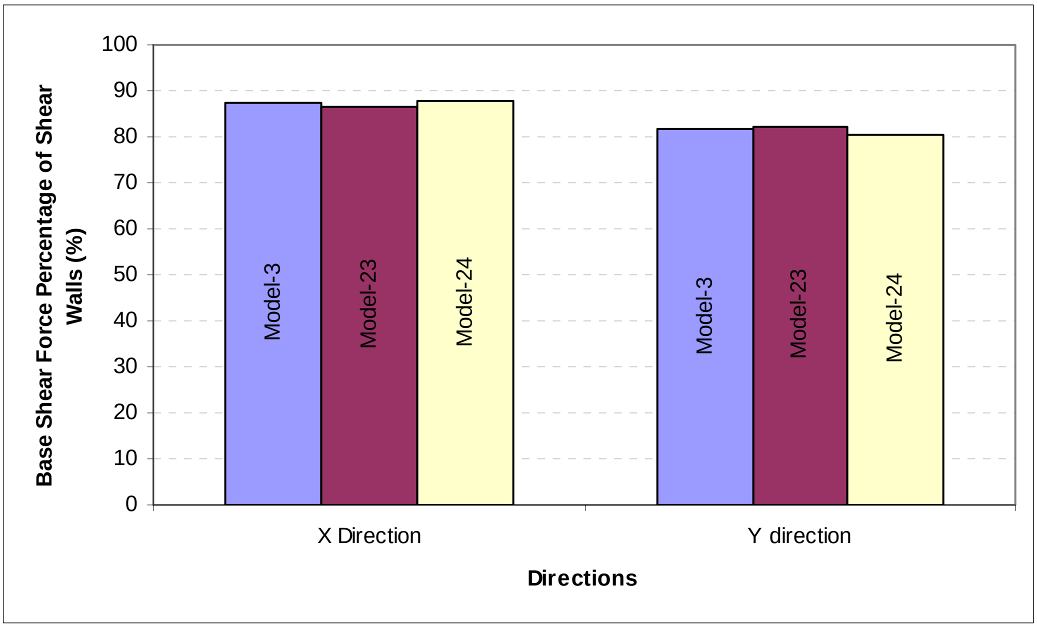

Figure 4.8. Shear Wall Contribution to Base Shear for Different Configurations

4.2.3. Story Shear Distribution

The story shear distribution of Models 3 and 13 representing buildings with 1% shear wall ratio in both of the two principal directions for 5 and 8 stories, respectively, is given in Figures 4.9 to 4.12.

In these graphs, the distribution of shear forces through each story at the instant of maximum base shear for the selected earthquakes is presented. The average shear force values for the seven selected earthquake records at each story level are also shown with a bold line.

Distribution of the average story shear force has a similar trend for both in the X and Y directions for the 5 story buildings. However in 8 story buildings, the story shear distribution trend differs in two principal directions. In the Y-direction of Model 13, the average story shear becomes negative at the sixth story because this was observed in 5 out of 7 earthquake records, except El Centro and Chile records. Whereas only one record, the Chile record has a negative story shear in the X-direction, therefore, the average response has positive story shear forces. At this point it is concluded that different characteristics of the seismic behavior, in terms of shear force distribution of mid-rise buildings with different number of stories should be taken into account for earthquake loadings with different characteristics.

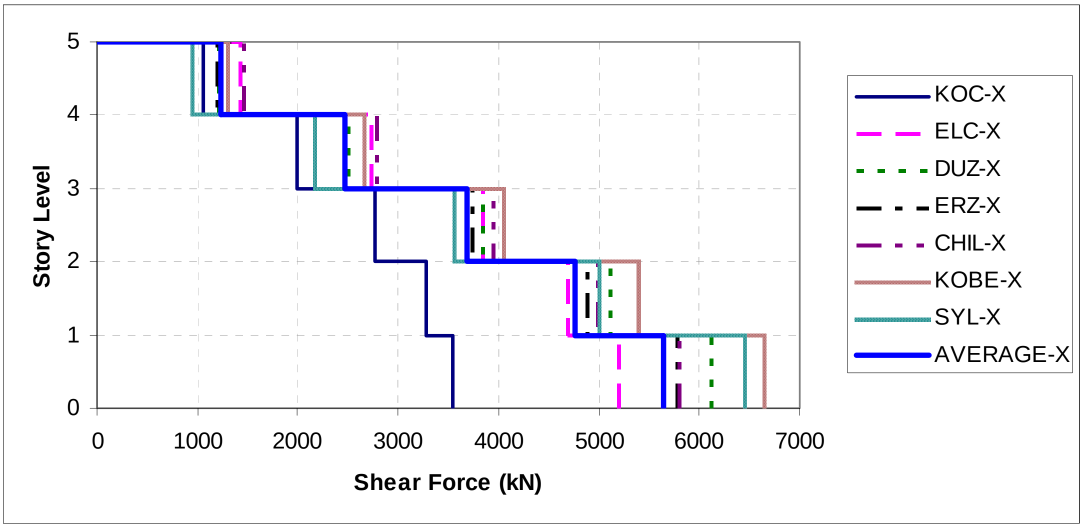

Figure 4.9. Story Shear Distribution of the 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-Direction

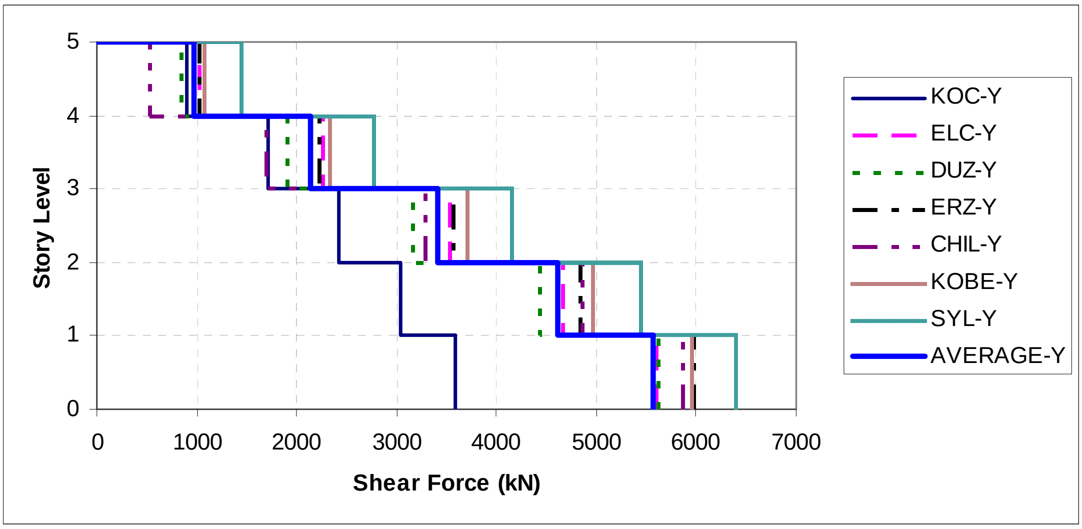

Figure 4.10. Story Shear Distribution of the 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-Direction

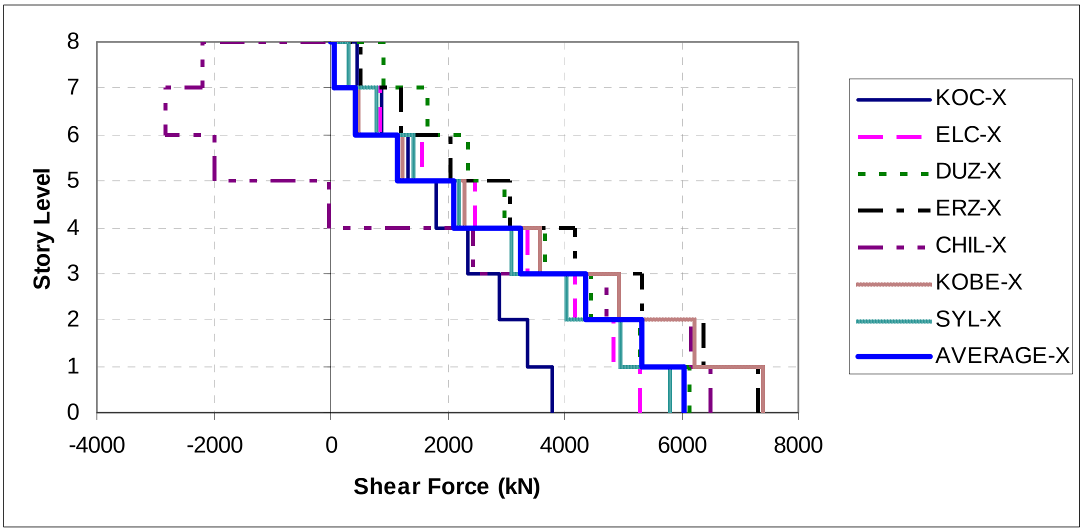

Figure 4.11. Story Shear Distribution of the 8 Story Building with 1.0% Shear Wall Ratio (Model 13) in the X-Direction

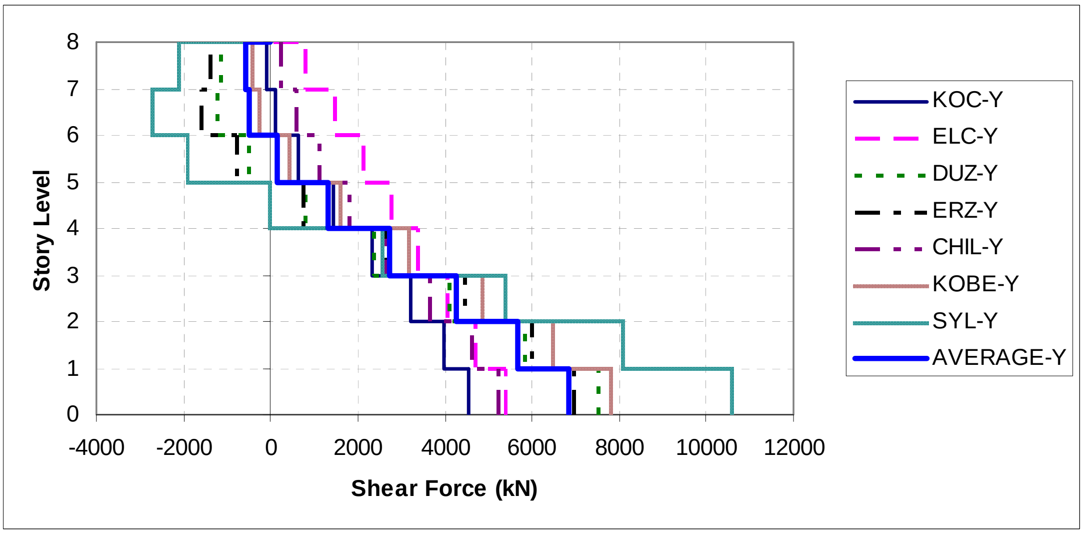

Figure 4.12. Story Shear Distribution of the 8 Story Building with 1.0% Shear Wall Ratio (Model 13) in the Y-Direction

4.2.4. Roof and Interstory Drifts

Roof drift and interstory drift variations with increasing shear wall area to floor area ratio of building models are given in Figures 4.13 and 4.14. Each data point represents average maximum drift obtained by imposing the selected earthquake records. The results are represented by trend lines passing through the average ordinates of the computed roof drift values for different shear wall ratios. The difference of the trend lines representing the drift in X and Y directions is more significant for 5 story buildings when compared to 8 story buildings. The behavior of all buildings in terms of roof drift and interstory drift are very similar for both directions, however the maximum interstory drifts are higher when compared to roof drifts, as expected. The maximum average roof drift is 0.90 %, while the maximum interstory drift is around 1.00 %.

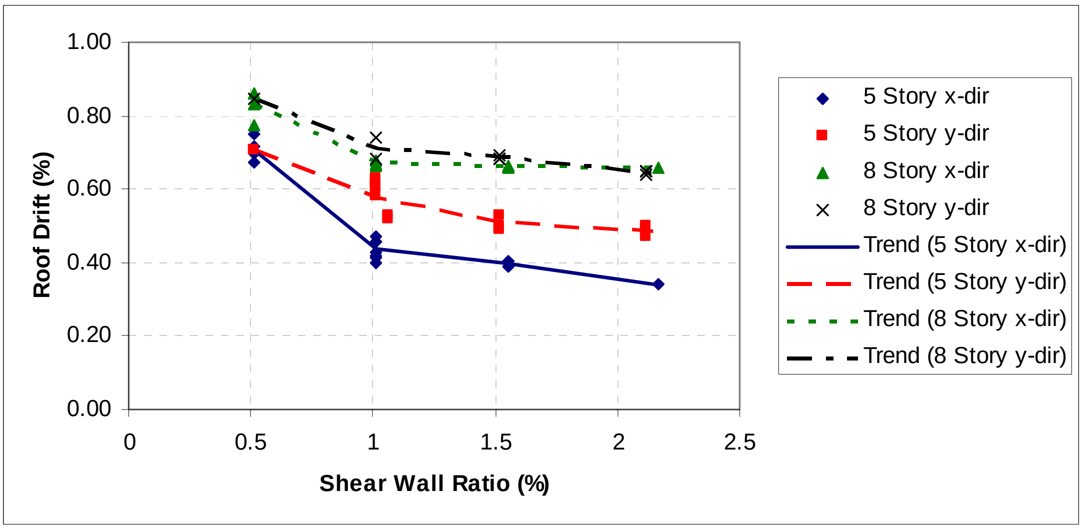

Figure 4.13. Variation of Roof Drift with Increasing Shear Wall Ratio

As it can be seen from these figures, for 8 story buildings, both roof and maximum interstory drift values had similar magnitudes in two orthogonal directions; however, for 5 story buildings the difference is more significant especially for 1.0% and higher shear wall ratios. At this point, it is worth to mention that modal periods of the 5 story buildings were generally within the interval of periods in which the natural periods of the earthquake records were observed and this could be the reason for drift amplifications in the direction of the strong component, which was applied in the Y-direction of the buildings.

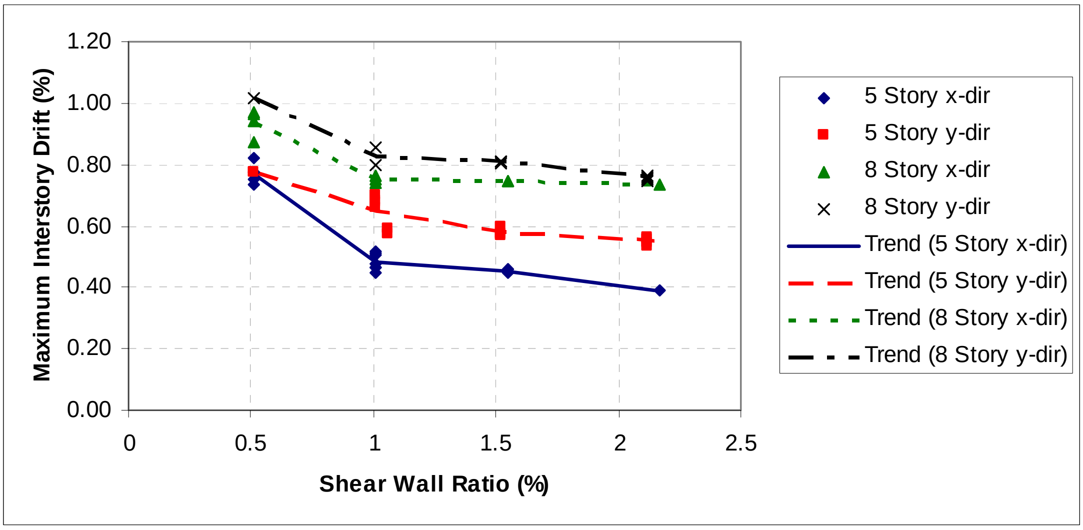

Figure 4.14. Variation of Maximum Interstory Drift with Increasing Shear Wall Ratio

Figures 4.13 and 4.14 indicate significant decrease in drift for increasing shear wall area to floor area ratios between 0.5 % and 1.0 % which indicates that at least 1.0 percent of shear wall ratio should be utilized in design considerations. After this point, decrease in drift is relatively higher between 1.0 % and 1.5 % shear wall ratios when compared to the one between 1.5 % and 2.0 %. This indicates, shear wall ratios up to 1.5 % could significantly improve the seismic performance. However, a shear wall ratio less than 1.0 % is not sufficient to limit the drift values.

In order to examine the change in drift, for shear wall ratios other than 1.0%, two additional graphs were constructed. For this purpose, the average of the data points of roof and interstory drifts for specific wall ratios, i.e. for 0.5%, 1.0%, 1.5% and 2.0%, was calculated and normalized with the drift values corresponding to 1.0% shear wall ratio (Figure 4.15 and 4.16).

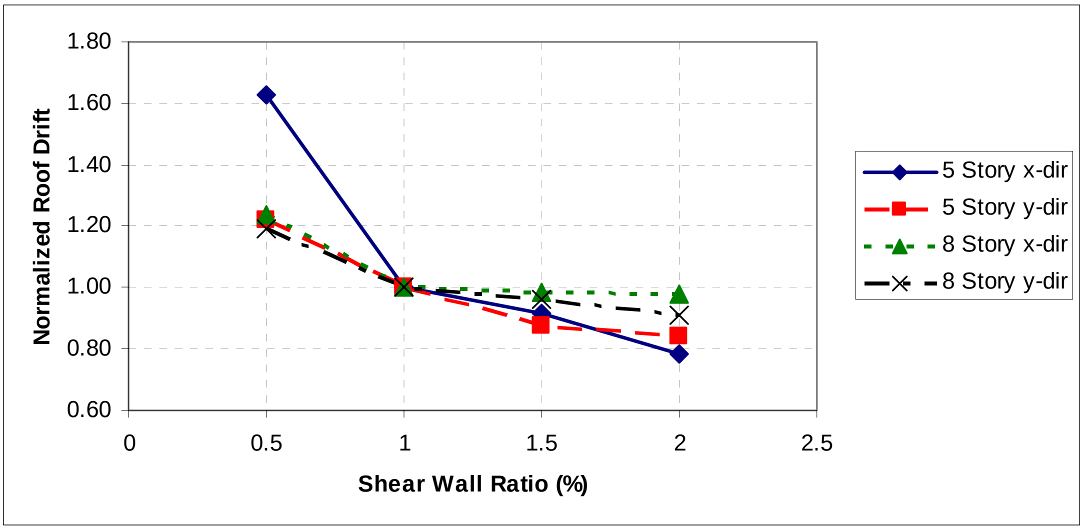

Figure 4.15. Variation of Normalized Roof Drift with Increasing Shear Wall Ratio

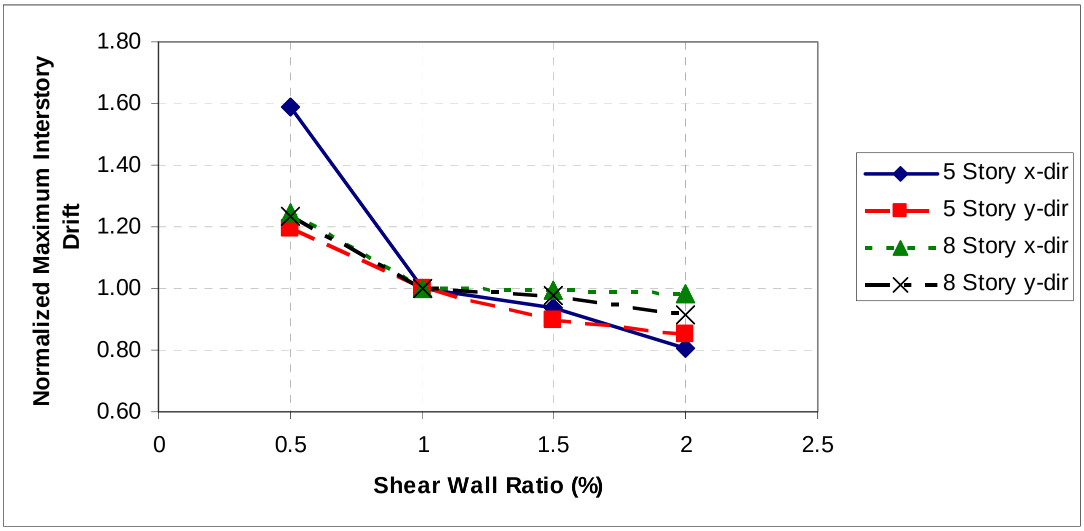

Figure 4.16. Variation of Normalized Maximum Interstory Drift with Increasing Shear Wall Ratio

In both graphs, there is a sharp decrease in drift in the X-direction of 5 story buildings when the shear wall ratio is increased from 0.5% to 1.0%. The normalized roof drift value for 0.5 % shear wall ratio was 1.63. When this ratio is increased further to 1.5% and 2.0%, the normalized roof drifts are 0.91 and 0.78, respectively.

When the Y-direction of the 5 story buildings is considered, the decrease in drift starts to decay significantly after 1.5 % shear wall ratio. The normalized roof drift values of this case for 1.5% and 2.0% of shear wall ratio are 0.88 and 0.84, respectively. In addition, the normalized roof drift value for 0.5% shear wall ratio in Y-direction was 1.22 and the decrease of roof drift in this direction between 0.5 and 1.0 percent shear wall ratios was not as sharp as the one in the X-direction.

The trend in the change of drifts in 8 story buildings differs from 5 story buildings. At first glance, both normalized values of roof drifts and interstory drifts are close to each other. Decrease in roof and interstory drift ratios is observed to be more significant for the shear wall ratios up to 1.0 percent when it is compared to higher wall ratios. After 1.0% wall ratio, the decrease in drift is almost constant in the Y-direction, almost 5% decrease for each 0.5% increase in the shear wall ratio. However, the performance in X-direction differs at this point since decrease in drift for shear wall ratio higher than 1.0 percent is less significant than the one in the transverse direction. In X-direction, the normalized roof drift values for 0.5%, 1.5% and 2.0% of shear wall ratios are 1.24, 0.99 and 0.98, respectively, while for the same shear wall ratios the normalized drift ratios are 1.19, 0.96 and 0.91 in Y-direction.

Figures 4.17 and 4.18 demonstrate the effect of column size on the roof drift and maximum interstory drift. As expected, a decrease in drift is observed in both directions as the column ratio increases; however the change is not significant. For example, the column ratio decreases 45% from Model-3 to Model-21 that have column sizes of 400x400 mm and 300x300 mm, respectively, and the related increase in roof drift ratio is determined as 3.7 % in the Y-direction. In addition, Model-22 with column sizes of 500x500 mm has 56 % more column ratio when compared to Model-3; however this results in a roof drift reduction of 4.2 %. The general trend for both roof drift and maximum interstory drift is similar in both X and Y directions.

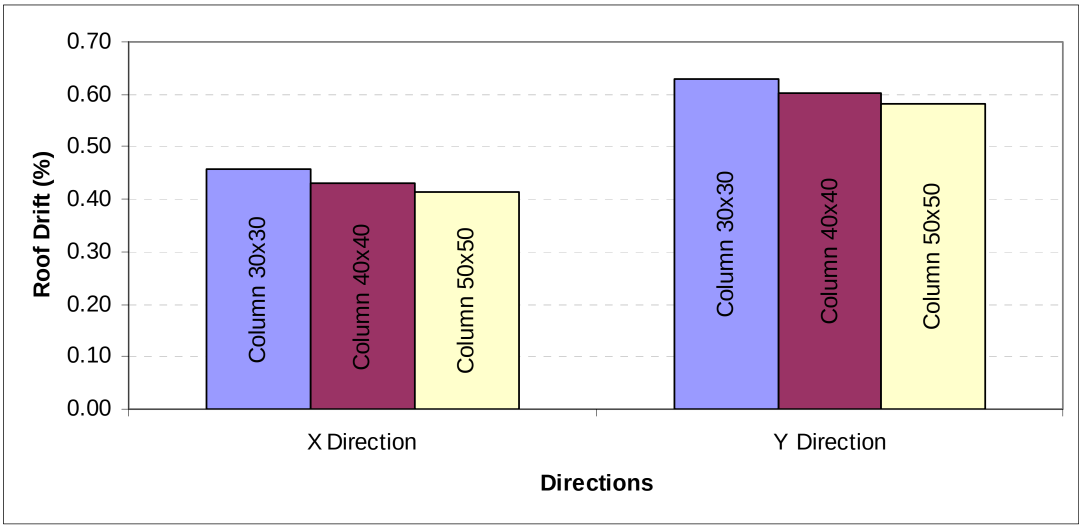

Figure 4.17. Variation of Roof Drift with Increasing Column Dimensions (Models 3, 21 and 22)

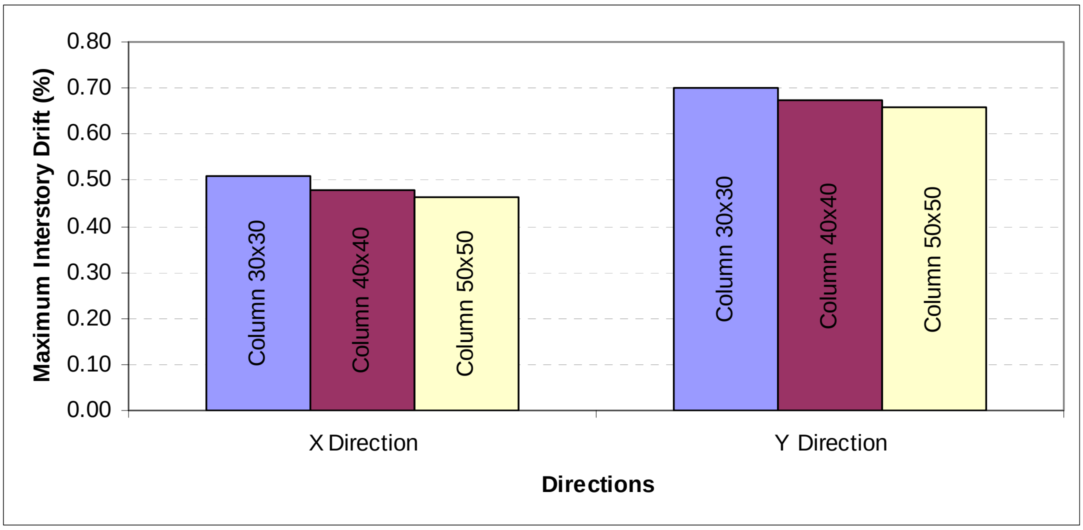

Figure 4.18. Variation of Maximum Interstory Drift with Increasing Column Dimensions (Models 3, 21 and 22)

The effect of the shear wall configuration on drift was examined by comparing the results obtained from Model-3, Model-23 and Model-24. For this purpose, shear wall configuration in the Y-direction was modified and the details of the building plans are presented in Chapter 3. In Model-3, 4 shear walls with 250 mm thickness, adding up to 1.0 % of shear wall ratio, are placed in outer frames of Y-direction. In Model-23 and Model-24, 3 shear walls with 350 mm thickness leading to, again, 1.0 percent of shear wall ratio are placed in central inner frames of the building in Y-direction. The difference of Model-23 and Model-24 is that, shear walls are placed on 3 different frames in the former model, where all three walls lie on the central frame in the latter one.

As it can be observed from Figures 4.19 and 4.20, the general trend of the variation in the roof and interstory drifts are similar. All three models have almost the same roof drift in the X-direction, 0.40 % for Model-23, 0.42 % for Model-24, and 0.43 % for Model-3. In the Y-direction, Model-3 has 14.2 and 12.5 percent more drift when compared to Models 23 and 24, respectively which have shear walls in inner frames. The difference in the total building stiffness is the reason for this result. A constant shear wall area to floor area ratio was maintained in the related models. However, the number of columns and hence the column ratio was higher in models 23 and 24 due to shear wall configurations.

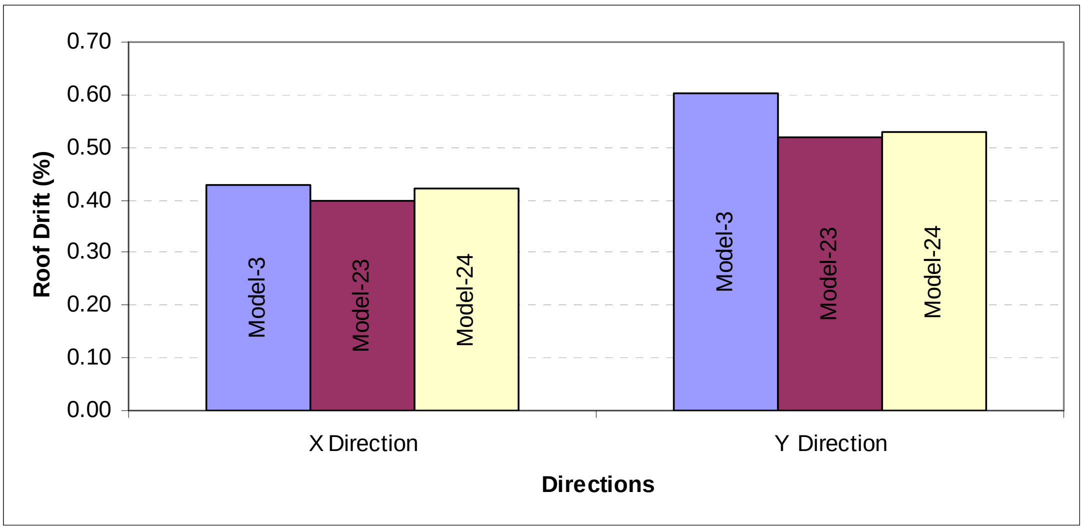

Figure 4.19. Roof Drift Variation for Different Shear Wall Configurations

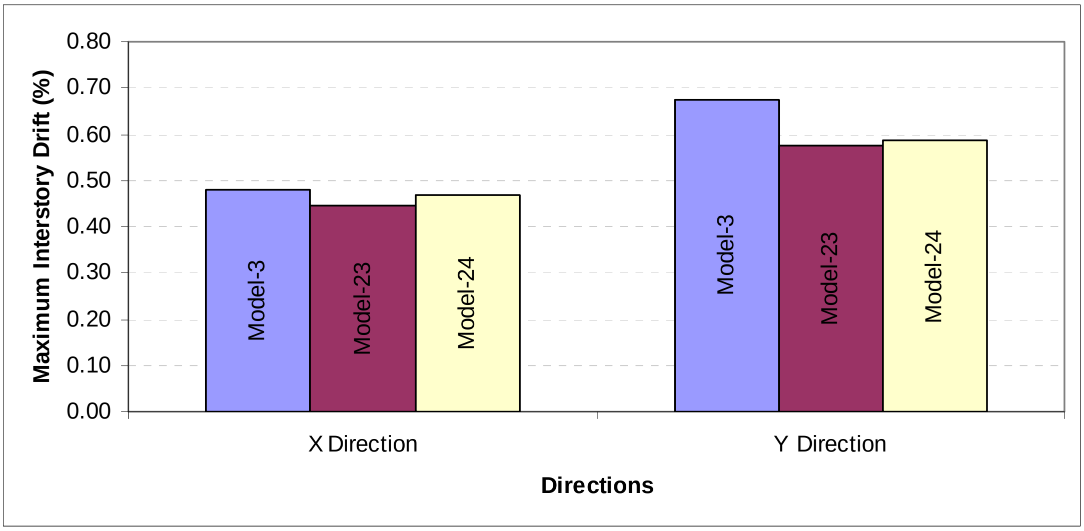

Figure 4.20. Maximum Interstory Drift Variation for Different Shear Wall Configurations

In addition to the effect of shear wall ratio on drift, contribution of column ratio to the seismic performance of the building was also studied. For this purpose, variation of roof and maximum interstory drift with changing shear wall ratio graphs were modified by adding column ratios to shear wall ratios (Figures 4.21 and 4.22). At this point, it is worth to mention that due to the square shape of the columns, column ratios are determined by considering the total area of the columns without any modification for X and Y directions.

The results are represented by idealized trend lines for each building model with different number of stories in each direction. For this purpose, third order polynomial trend lines were approximated for each data set and the corresponding equations were obtained. Then by using these equations, drift values for intermediate "shear wall + column" ratios were computed then these points were linearly connected. The reason for a scattered distribution could be explained by the scattered data which is actually due to the fact that lower shear wall ratios, as being the major parameter affecting the drift, lead to higher drifts which could not be compensated even by high column ratios. It was observed that in some specific models, especially models having a wall ratio of 0.5% in one direction, the summation of column and shear wall ratios is higher even if the shear wall ratio is lower than that of the other models. Such buildings experienced higher drifts leading to scattered data. However, it could roughly be observed that as the summation column and shear wall ratios increase, drift values decrease.

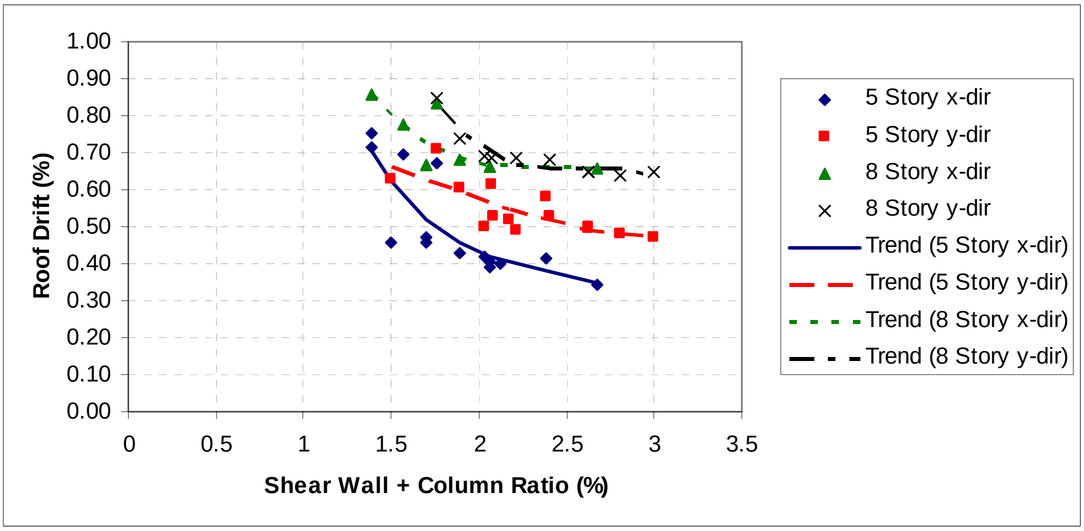

Figure 4.21. Variation of Roof Drift with Increasing Shear Wall + Column Ratio

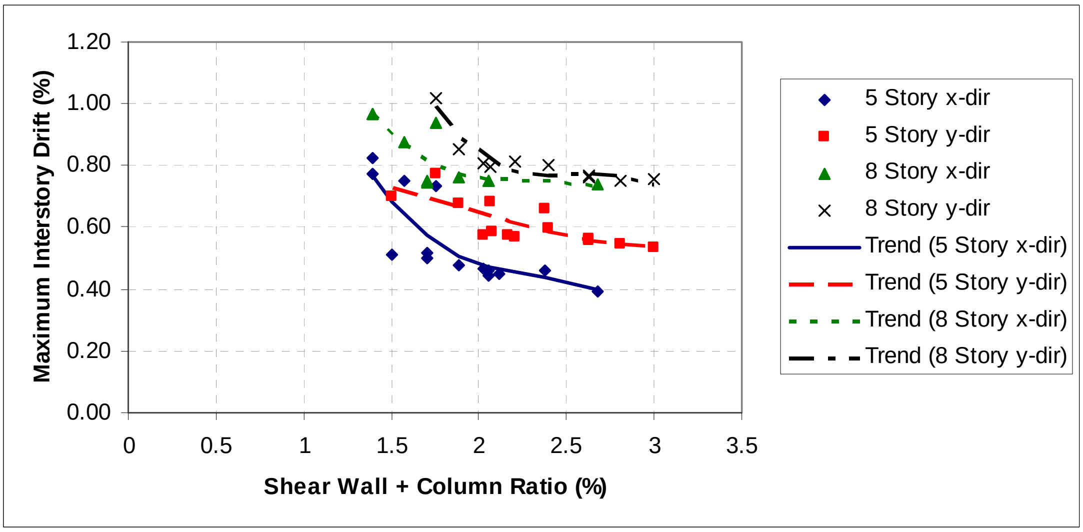

Figure 4.22. Variation of Maximum Interstory Drift with Increasing Shear Wall + Column Ratio

4.2.5. Distribution of Interstory Drift

In this study, the distribution of interstory drift for building models having shear wall ratios of 0.5%, 1.0%, 1.5% and 2.0% in both directions are investigated to observe the overall behavior. Building Models 1, 3, 6 and 10 represent the mentioned shear wall ratios, respectively, for 5 story buildings and Building Models 11, 13, 16 and 20 represent those values for 8 story buildings.

Distribution of maximum interstory drift through each story level is presented in Figures 4.23 to 4.26. These figures verify the outcomes of the preceding section on the roof and interstory drifts and provide some additional information. For 5 story buildings, the interstory drifts were observed to be higher at the upper floors and the 4th story generally has the average maximum interstory drift. The interstory drift at each story level increases with decreasing shear wall area to floor area ratios.

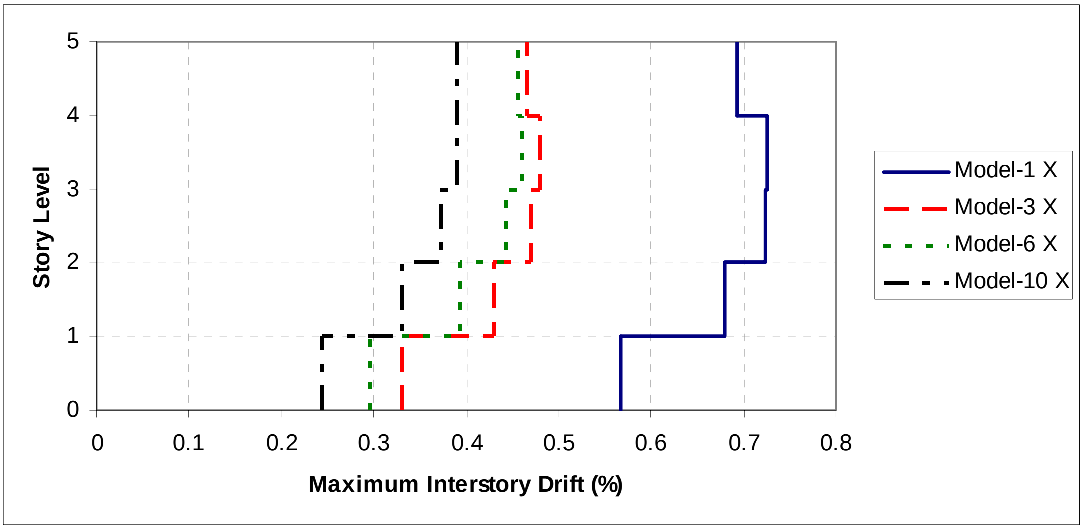

Figure 4.23. Distribution of Maximum Interstory Drift for 5 Story Buildings with Varying Shear Wall Ratios in the X-Direction

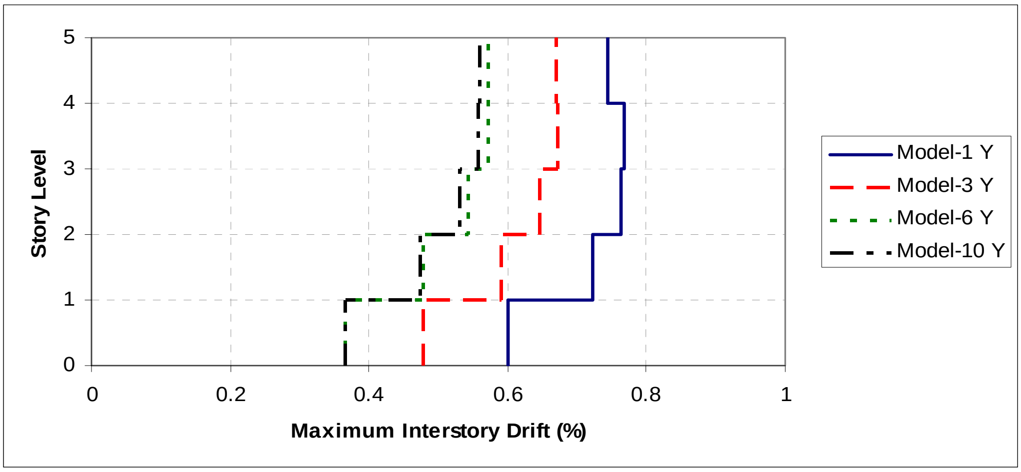

Figure 4.24. Distribution of Maximum Interstory Drift for 5 Story Buildings with Varying Shear Wall Ratios in the Y-Direction

In 8 story buildings, the increase in interstory drift with decreasing shear wall ratio is not as apparent as the case for 5 story ones, especially for the higher floors in the X-direction. For the model with 0.5 % wall ratio, average maximum interstory drifts are observed in 4th and 3rd stories for X and Y-directions, respectively. For models with shear wall ratios higher than 0.5 %, the average maximum interstory drifts are observed between 5th and 7th floors (Figures 4.25 and 4.26).

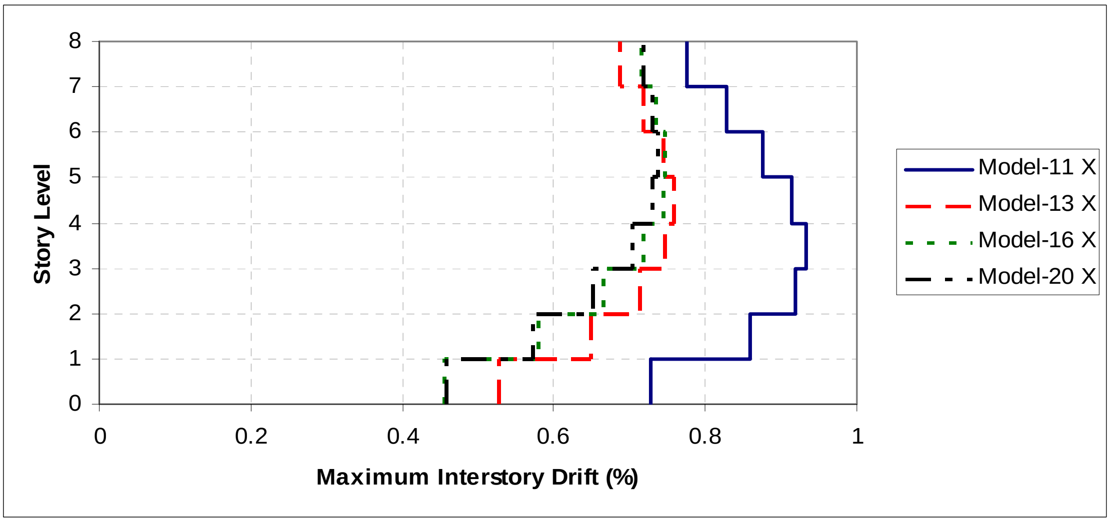

Figure 4.25. Distribution of Maximum Interstory Drift for 8 Story Buildings with Varying Shear Wall Ratios in the X-Direction

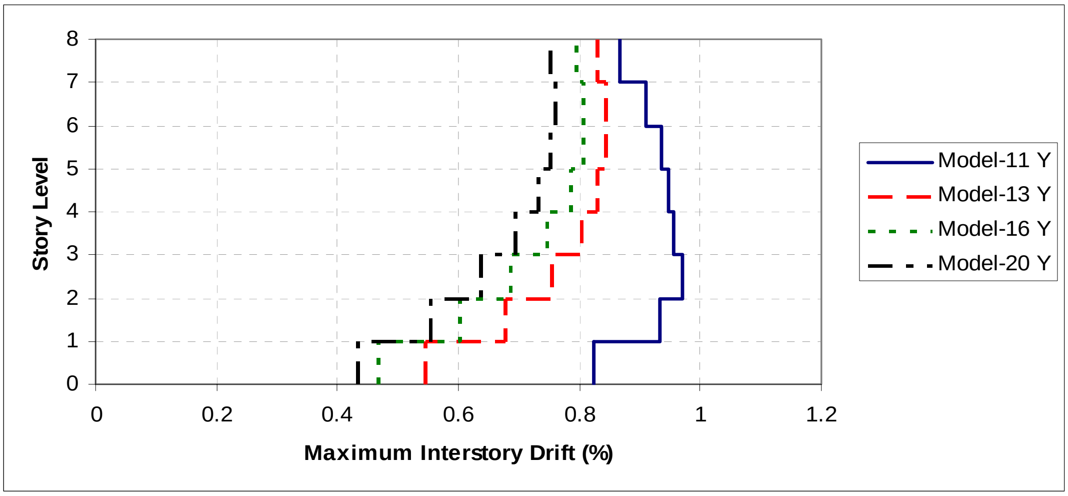

Figure 4.26. Distribution of Maximum Interstory Drift for 8 Story Buildings with Varying Shear Wall Ratios in the Y-Direction

Figures 4.27 and 4.28 show the distribution of interstory drift for the Models 3, 21 and 22 generated to investigate the effect of column size on the dynamic behavior. Story level drift distribution of Models 3, 23 and 24, which were utilized to observe the effect of shear wall layouts are presented in Figures 4.29 and 4.30.

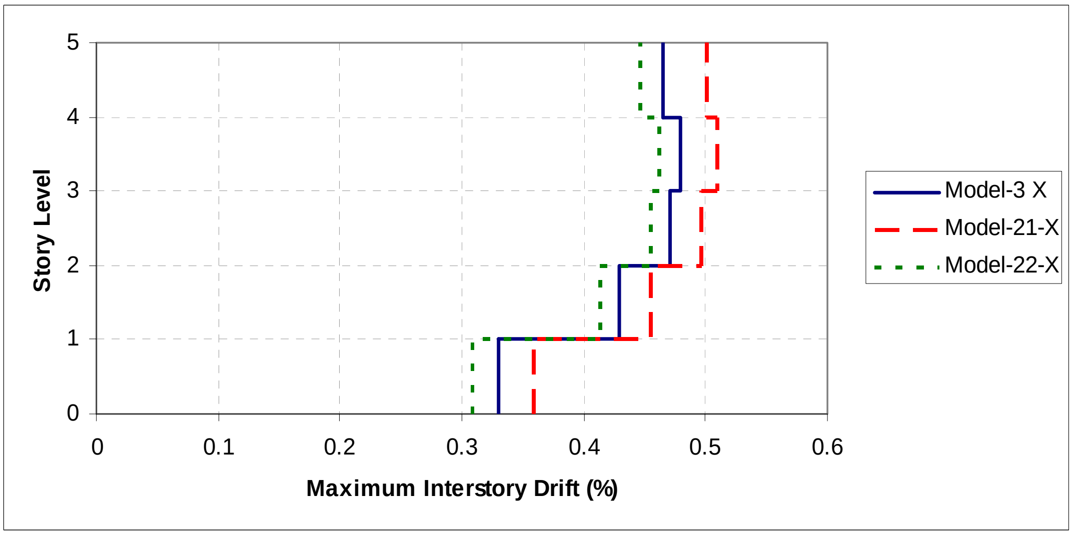

Figure 4.27. Distribution of Maximum Interstory Drift for Varying Column Dimensions in the X-Direction

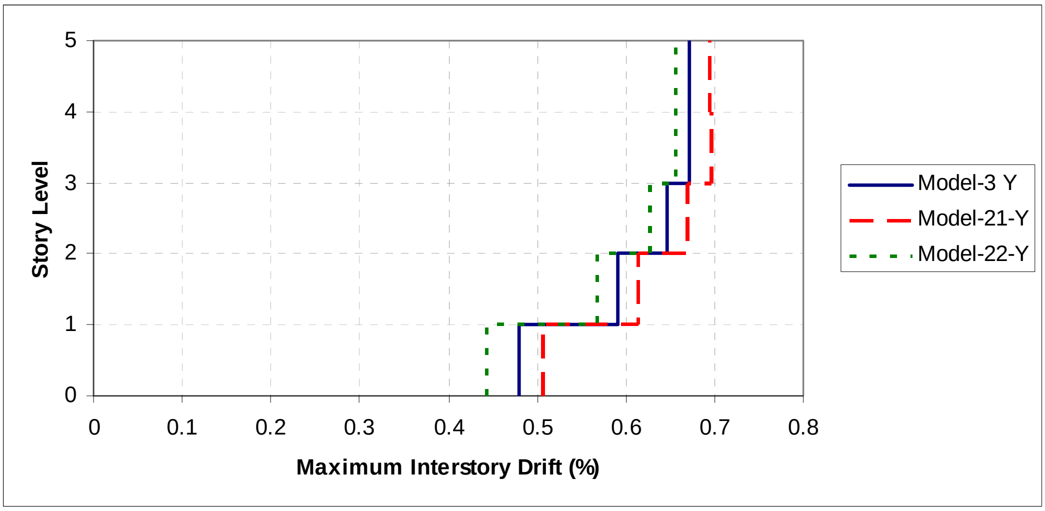

Figure 4.28. Distribution of Maximum Interstory Drift for Varying Column Dimensions in the Y-Direction

The outcomes of the preceding section on the comparison of the behavior for varying column dimensions and shear wall configuration in terms of roof and maximum interstory drifts of the related models are also verified and presented in these figures. According to these graphs, it was observed that as the column dimensions increase, leading to increase in column ratio, both roof and interstory drifts decrease, as expected. Different shear wall configurations in such symmetrical buildings did not result in a significant change in drifts. The distribution of the interstory drifts has the same trend with other 5 story model buildings used in this study.

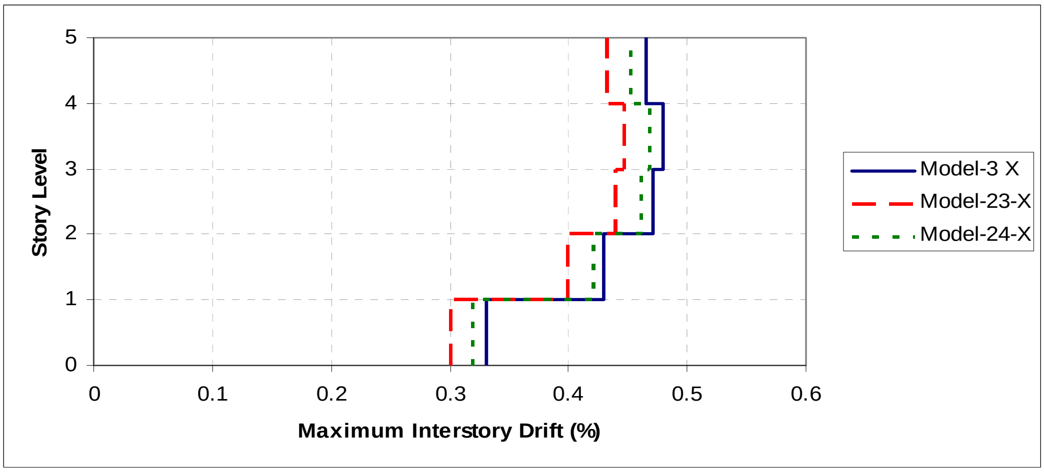

Figure 4.29. Distribution of Maximum Interstory Drift for Different Shear Wall Configurations in the X-Direction

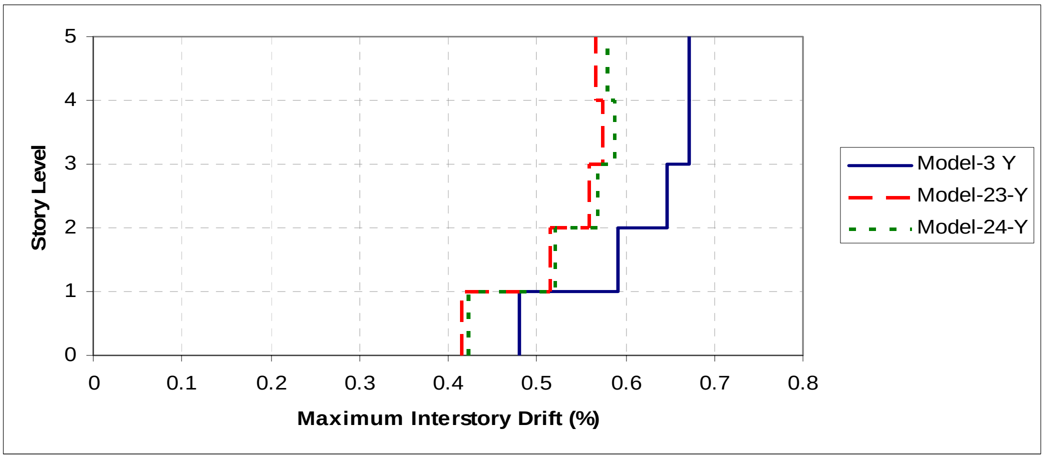

Figure 4.30. Distribution of Maximum Interstory Drift for Different Shear Wall Configurations in the Y-Direction

4.2.6. Base Shear versus Roof Drift Relationship

The comparison of the base shear versus roof drift relationships for Model 3 and Model 13, the 5 and 8 story buildings having 1.0 % shear wall ratio in both directions, is given in Figures 4.31 to 4.38. These figures show the behavior in both principal directions for Kocaeli and Sylmar records. The reason for selecting these two records is to observe the intended relationship under different earthquake characteristics in terms of peak ground accelerations. The peak ground acceleration values for Kocaeli record are 0.152g and 0.220g, whereas these values are 0.575g and 0.825g for Sylmar record in X and Y-directions, respectively. The graphs constructed for the behavior of Model 3 under the rest of the earthquake records are given in Appendix 7.

When the Kocaeli record was applied to the 5 story model, it oscillates mostly in the elastic range in both directions. However, after the peak acceleration of the Sylmar record, the structure experiences significant inelastic behavior. There is a residual drift in both directions; however, the stiffness does not drop significantly. In order to observe the limiting capacity for the Model 3, an additional graph representing the distribution of maximum observed base shear versus the corresponding roof drift was constructed for the selected seven earthquake records (Figure 4.35). The procedure followed is analogous to incremental pushover analysis since the relationship between base shear and roof drift of the building models were plotted for increasing earthquake intensities varying between 0.152 and 0.825 g.

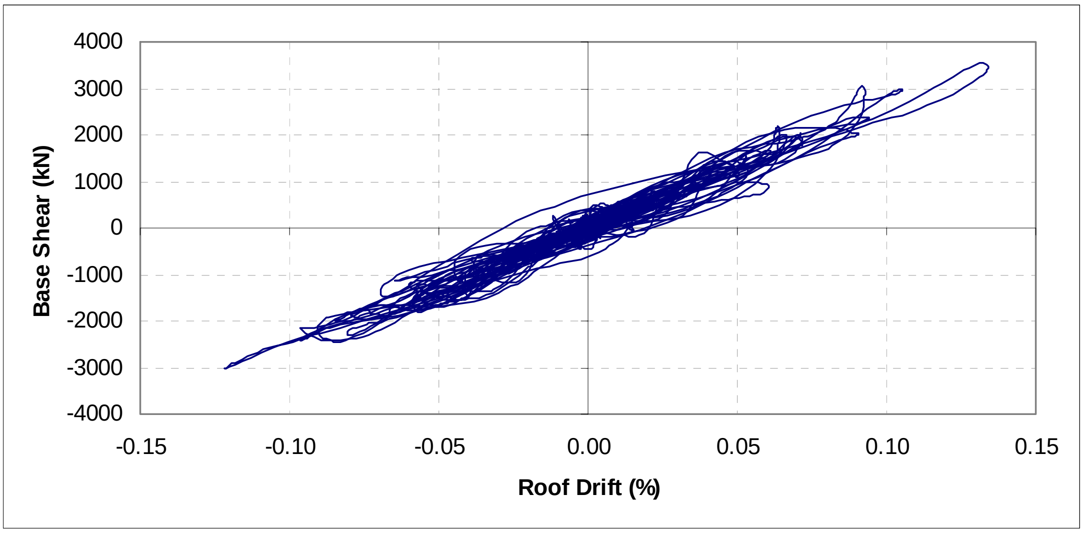

Figure 4.31. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-direction under Kocaeli Record

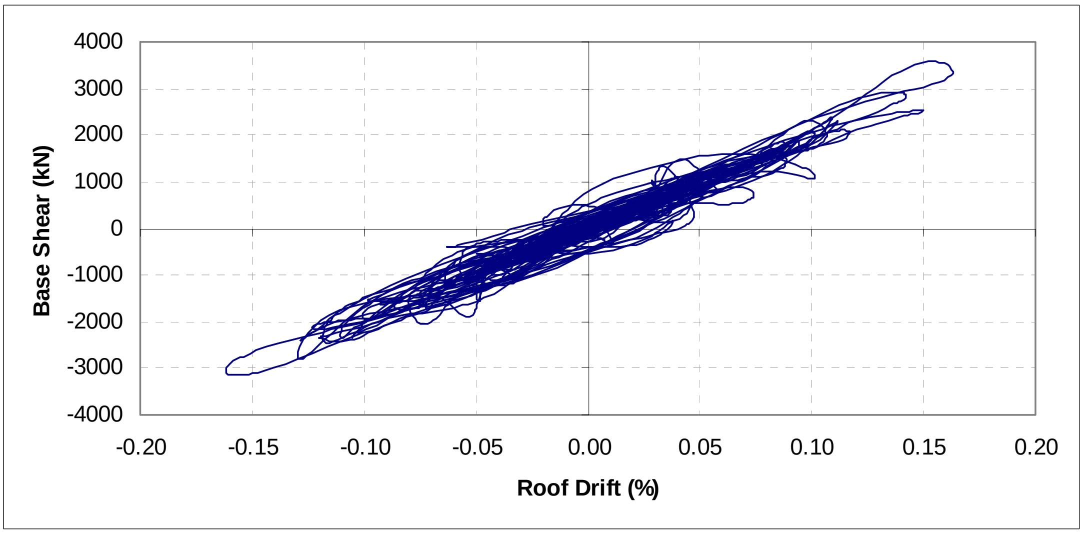

Figure 4.32. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-direction under Kocaeli Record

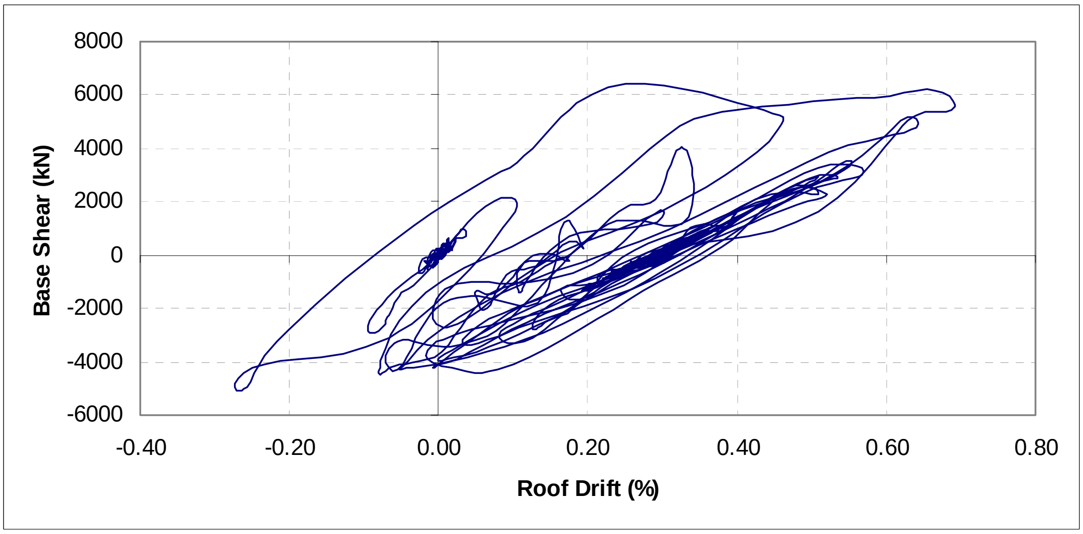

Figure 4.33. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-direction under Sylmar Record

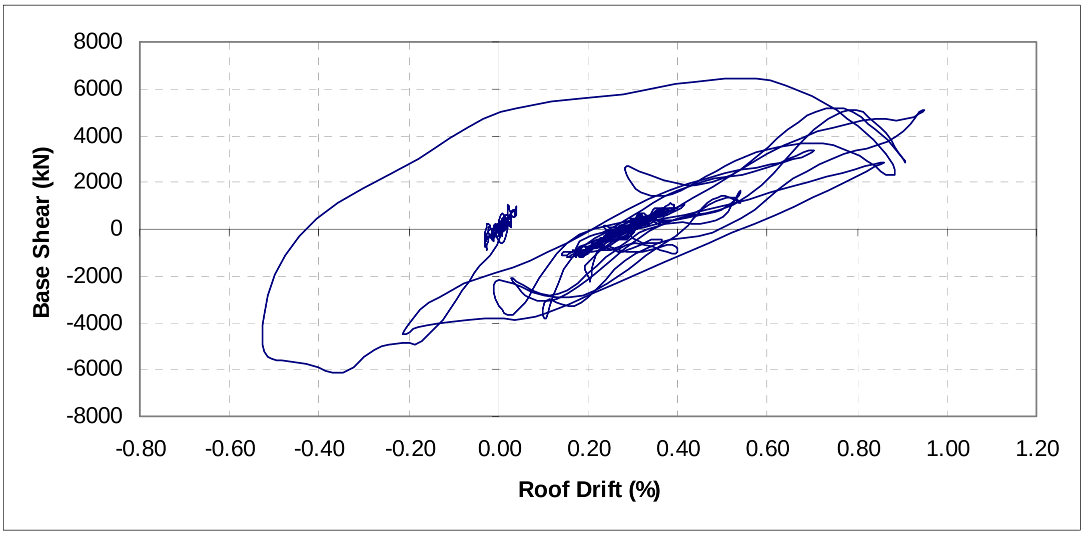

Figure 4.34. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-direction under Sylmar Record

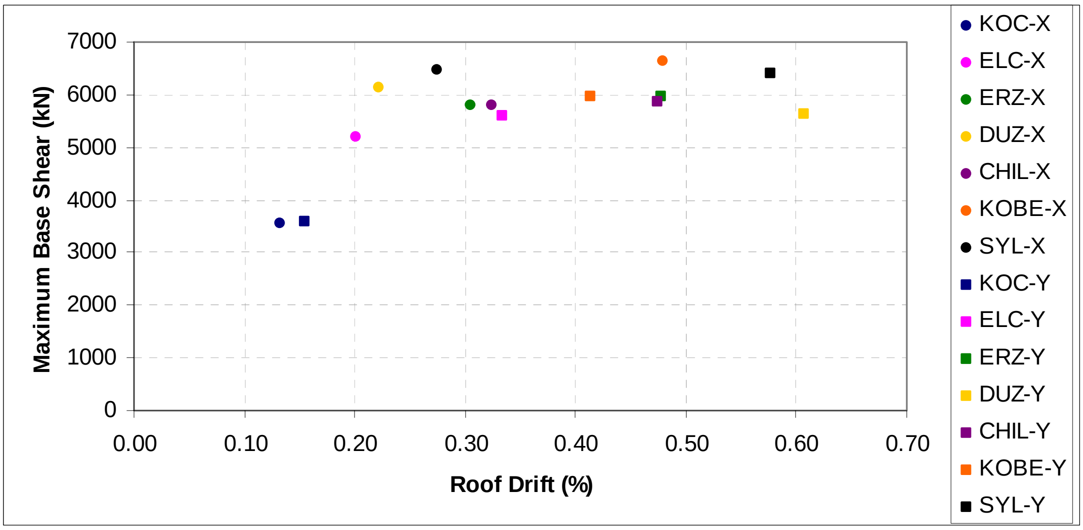

Figure 4.35. Maximum Base Shear vs. Roof Drift Relationship for the 5 Story Building with 1.0 % Shear Wall Ratio (Model 3) under All Selected Earthquake Records

When compared to the behavior of 5 story building model (Model 3), Model 13 with 8 stories experiences larger base shears and roof drifts for both records. When the Kocaeli record was applied, as it can be observed from Figures 4.36 and 4.37, the data points have a wider band width than the data of the 5 story building. The dispersion of the data is mostly due to the difference in building stiffness and mass properties leading to a different response under the same earthquake loading. Under Sylmar record, surprisingly, a residual drift is observed only in the X-direction, even though the peak ground acceleration of the record in the Y-direction is higher (Figures 4.38 and 4.39). Actually, this incident shows that inelastic response of a structure depends on building characteristics and its interaction with the earthquake record leading to changes in hierarchy of the inelastic behavior. Since the first plastic behavior was observed in the X-direction, the building model had more damage in this direction which intervened with the deformations in the Y-direction. In this case, roof drift in the X-direction dominantly affected the overall behavior with a residual drift around 0.63 % at the end of this ground motion. The distribution of maximum observed base shear versus the corresponding roof drift of Model 13 constructed for the selected seven earthquake record is given in Figure 4.40. The distribution of the data does not have an increasing trend for increasing earthquake intensities due to the characteristics of the response of the building especially for the records with pulse and this situation was also observed in some of the other building models. For example, the maximum base shear force was observed in Sylmar record; however the corresponding observed drift was smaller than the drift values for the records with considerably lower intensities.

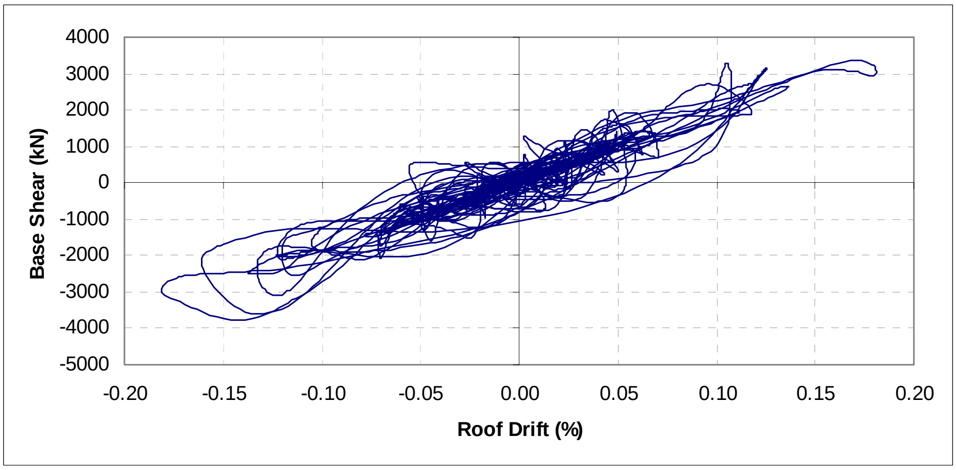

Figure 4.36. Base Shear versus Roof Drift Relationship of 8 Story Building with 1.0% Shear Wall Ratio (Model 13) in the X-direction under Kocaeli Record

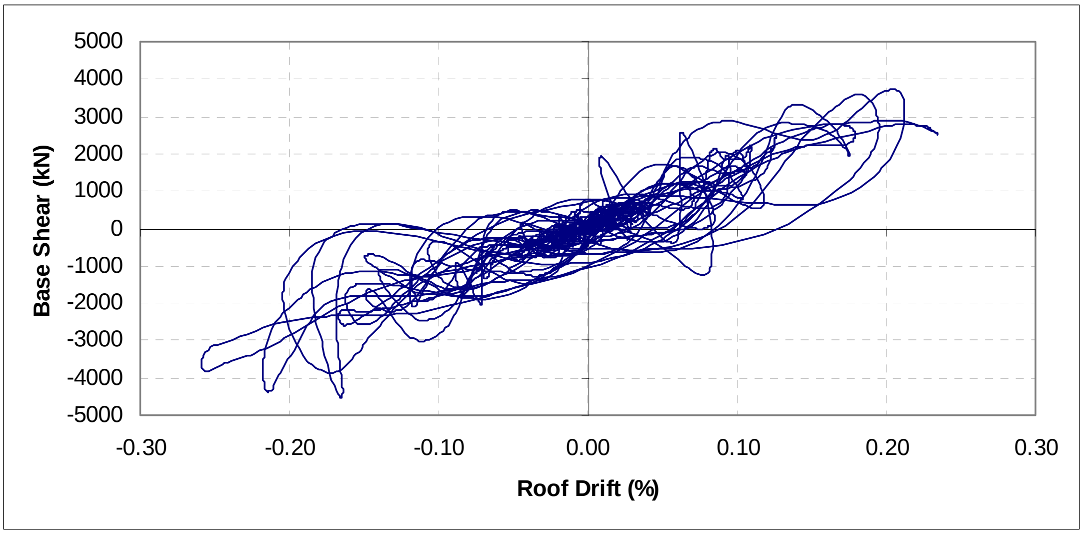

Figure 4.37. Base Shear versus Roof Drift Relationship of 8 Story Building with 1.0% Shear Wall Ratio (Model 13) in the Y-direction under Kocaeli Record

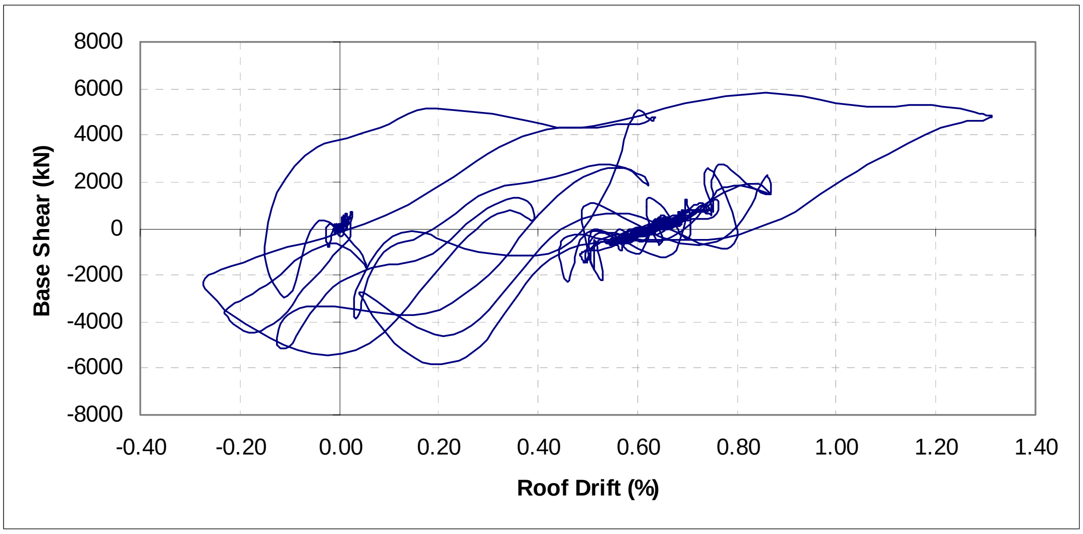

Figure 4.38. Base Shear versus Roof Drift Relationship of 8 Story Building with 1.0% Shear Wall Ratio (Model 13) in the X-direction under Sylmar Record

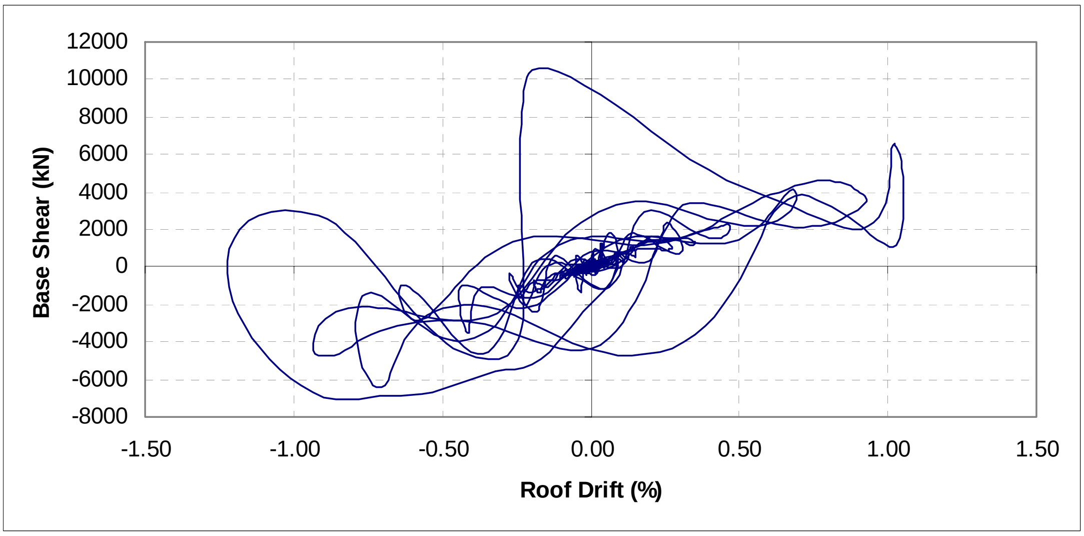

Figure 4.39. Base Shear versus Roof Drift Relationship of 8 Story Building with 1.0% Shear Wall Ratio (Model 13) in the Y-direction under Sylmar Record

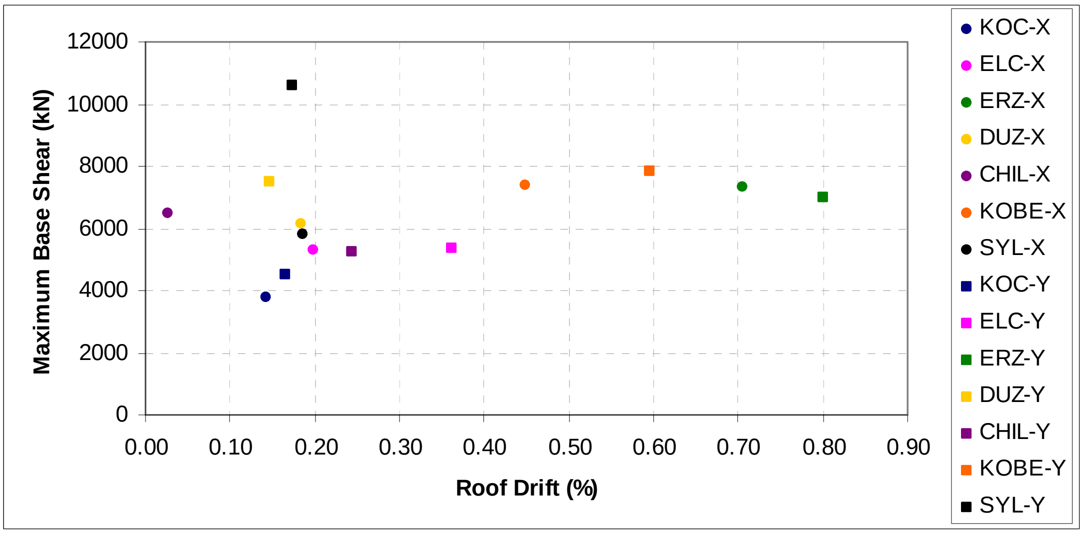

Figure 4.40. Maximum Base Shear vs. Roof Drift Relationship of 8 Story Building with 1.0 % Shear Wall Ratio (Model 13) under All Selected Earthquake Records

Distribution of maximum base shear versus roof drift for models representing different shear wall and column ratios and shear wall configurations are given in Appendix A.8.

The base shear and roof drift relationships for model buildings generated to investigate the effect of shear wall configuration are also examined and the related graphs are given in Appendix A.9. These graphs indicated that, there are some minor differences in behavior of these building models; however, this does not have a significant effect on the roof drift, which could be used as a measure of displacement ductility.

4.2.7. Performance Comparison of the Building Models

In order to examine the performance of the generated building models in terms of inelastic activity, the variation of the time when the first yield of the load carrying members were observed with increasing shear wall ratio was investigated. For this purpose, the Y- direction of Models 1, 3, 6 and 10 with shear wall ratios of 0.5, 1.0, 1.5 and 2.0 percent, respectively, were studied. The records of Chile, Kobe and Sylmar were selected as the time histories due to their higher intensities as compared to the remaining 4 records. Y-direction of these building models were primarily studied because, not only the building has a higher vulnerability potential in this direction than the transverse one but also the higher components of the records were applied in this direction.

The obtained results are summarized in Table 4.1. The difference was negligible in the starting time of the inelastic activity in beams, columns and shear walls of the building models with different shear wall ratios for each record. In the table, additionally, the corresponding base shear and roof drift values are given for the specific time that the inelasticity is first observed for each member group. The response of the structure in terms of drift and base shear force is highly dependent on the characteristics of the record in the time-history analysis. The magnitude of base shear force and roof drift is likely to change drastically within milliseconds under the effect of earthquake records that have pulse effect, especially Kobe and Sylmar, as it can be observed in Table 4.1. The yield pattern did not show a consistent trend in seismic behavior of the building models that can be used to evaluate the efficiency of higher shear wall ratios.

Table 4.1. Comparison of Models 1, 3, 6 and 10 with Shear Wall Ratios of 0.5, 1.0, 1.5 and 2.0 % in Terms of Time of First Yielding of the Members and the Corresponding Base Shear and Roof Drift Values

			Time of First Yield (s)	Base Shear (kN)	Roof Drift (%)
Chile Record	Model -1 (0.5%)	Column	16.9	3220.97	0.3032
		Beam	15.475	271.324	0.1556
		Shear Wall	15.5	850.82	0.17113
	Model -3 (1.0%)	Column	17.65	1765.89	0.25708
		Beam	17.4	2082.45	0.17149
		Shear Wall	17.65	1765.89	0.25708
	Model -6 (1.5%)	Column	17.75	6183.27	0.28716
		Beam	17.375	2755.07	0.15977
		Shear Wall	17.725	5947.36	0.22095
	Model -10 (2.0%)	Column	17.775	7649.35	0.25779
		Beam	17.35	3175	0.11768
		Shear Wall	17.75	7541.67	0.26759
Kobe Record	Model -1 (0.5%)	Column	4.22	3380.8	0.37656
		Beam	4.18	3237.21	0.25654
		Shear Wall	4.02	2462.96	0.22115
	Model -3 (1.0%)	Column	4.24	4581.12	0.34107
		Beam	4.16	4488.95	0.23615
		Shear Wall	4.01	2232.33	0.13014
	Model -6 (1.5%)	Column	4.26	2738.45	0.21486
		Beam	4.2	5977.5	0.2864
		Shear Wall	4.15	6185.77	0.22234
	Model -10 (2.0%)	Column	4.26	1805.87	0.12843
		Beam	4.22	5950.4	0.21805
		Shear Wall	4.16	6847.41	0.22336
Sylmar Record	Model -1 (0.5%)	Column	3.54	4349.00	0.3499
		Beam	3.52	4106.00	0.3067
		Shear Wall	3.52	4106.00	0.3067
	Model -3 (1.0%)	Column	3.54	5481.99	0.2993
		Beam	3.56	5893.07	0.3218
		Shear Wall	3.54	5481.99	0.2993
	Model -6 (1.5%)	Column	3.56	6754.37	0.2333
		Beam	3.54	6645.87	0.2206
		Shear Wall	3.6	7155.20	0.2472
	Model -10 (2.0%)	Column	3.6	8280.82	0.2258
		Beam	3.58	8042.82	0.2237
		Shear Wall	3.62	8272.38	0.2269

When the first yield for each member group did not indicate a clear trend, an additional investigation was carried out in this section. The number of yielded members, in terms of beams, columns and shear walls, at each story level was investigated. Similar to the former investigation, the inelastic activity of Models 1, 3, 6 and 10 under Chile, Kobe and Sylmar records was examined. The results representing the percentage of the yielded members are summarized in Tables 4.2 to 4.4. It was noted that the total number of yielded members varies in building models depending on the shear wall ratio, however the difference is not significant. As the shear wall ratio increases up to 1.5 %, the percentage of yielded members decreases even under strong ground motions. This effect seems to diminish after 1.5% shear wall area to floor area ratio.

In Model 1 with 0.5 percent shear wall ratio, the number of yielded columns in first story is determined as 12 out of 27. Additionally, in the second story of the same model, 3 columns leading to 11.11 % of the total number were observed to be yielded during Kobe and Sylmar records. When the shear wall ratio is higher than 0.5 %, for Models 3, 6 and 10, none of the first story columns experienced inelastic activity under the effect of Chile record. In Models 6 and 10 with 1.5 and 2.0 percent shear wall ratios, respectively, about 10 percent of the first story columns did not reach their yield limits even during Kobe and Sylmar records that have a pulse effect.

Table 4.2. The Percentage of Yielded Columns in Y-direction for Models 1, 3, 6 and 10 with Shear Wall Ratios of 0.5, 1.0, 1.5 and 2.0 %

	Record	Percentage of Yielded Columns (%) (Y-direction)				
		Story-1	Story-2	Story-3	Story-4	Story-5
Model -1 (0.5%)	Chile	44.44	0	0	0	0
	Kobe	100	11.11	0	0	0
	Sylmar	100	11.11	0	0	0
Model -3 (1.0%)	Chile	0	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0
Model -6 (1.5%)	Chile	0	0	0	0	0
	Kobe	90.91	0	0	0	0
	Sylmar	90.91	0	0	0	0
Model -10 (2.0%)	Chile	0	0	0	0	0
	Kobe	90.91	0	0	0	0
	Sylmar	90.91	0	0	0	0

The inelastic activity of the beam members was not considerable affected by the increasing shear wall ratio. In Kobe and Sylmar records which have relatively higher intensities compared to Chile record, most of the beams were yielded. At this point it is worth to mention that a member is defined as yielded even when only one end of the member is observed to exceed its elastic limit.

Table 4.3. The Percentage of Yielded Beams in Y-direction for Models 1, 3, 6 and 10 with Shear Wall Ratios of 0.5, 1.0, 1.5 and 2.0 %

	Record	Percentage of Yielded Beams (%) (Y-Direction)				
		Story-1	Story-2	Story-3	Story-4	Story-5
Model -1 (0.5%)	Chile	92.31	92.31	100.00	100.00	100.00
	Kobe	92.31	100.00	100.00	100.00	100.00
	Sylmar	92.31	100.00	100.00	100.00	100.00
Model -3 (1.0%)	Chile	91.67	91.67	91.67	100.00	100.00
	Kobe	91.67	91.67	100.00	100.00	100.00
	Sylmar	91.67	100.00	100.00	100.00	100.00
Model -6 (1.5%)	Chile	81.82	81.82	81.82	90.91	90.91
	Kobe	90.91	90.91	100.00	100.00	100.00
	Sylmar	90.91	90.91	100.00	100.00	100.00
Model -10 (2.0%)	Chile	81.82	81.82	81.82	90.91	81.82
	Kobe	90.91	90.91	100.00	100.00	100.00
	Sylmar	90.91	90.91	100.00	100.00	100.00

The effect of increase in shear wall ratio on the inelastic performance of the shear walls is observed to be rather significant for wall ratios higher than 0.5 %. As it can be observed from Table 4.4., Model 1 which has 2 shear walls in each direction adding up to 0.5% wall ratio experienced yielding of these shear wall members in all records for the first two stories. However, when the wall ratio exceeds 0.5 percent including the Models 3, 6 and 10, shear walls did not yield at the second story level. This indicates that 0.5 % shear wall ratio is not efficient for the reduction of inelastic activities in the load carrying members when compared to a wall ratio of 1.0 % or higher.

Table 4.4. The Percentage of Yielded Shear Walls in Y-direction for Models 1, 3, 6 and 10 with Shear Wall Ratios of 0.5, 1.0, 1.5 and 2.0 %

	Record	Percentage of Yielded Shear Walls (%) (Y-Direction)				
		Story-1	Story-2	Story-3	Story-4	Story-5
Model -1 (0.5%)	Chile	100	100	0	0	0
	Kobe	100	100	0	0	0
	Sylmar	100	100	0	0	0
Model -3 (1.0%)	Chile	100	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0
Model -6 (1.5%)	Chile	100	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0
Model -10 (2.0%)	Chile	100	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0

Tables summarizing the percentage of yielded members for X-direction are given in Appendix A.10.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1. SUMMARY

This study focuses on the influence of the shear wall area to floor area ratio on the seismic performance of mid-rise reinforced concrete buildings. For this purpose, twenty four mid-rise building models (five and eight stories) with a shear wall area to floor area ratio between 0.51 % and 2.17 % in both principal directions were generated considering the parameters that are included in the database of buildings surveyed after Düzce (1999) and Bingöl (2003) Earthquakes. Their design was carried out following the Turkish Earthquake Code requirements (TEC 2007 [28]). Then, nonlinear time-history analyses were performed to examine their seismic performance by using PERFORM 3D v 4.0 [5], a commercially available program.

The shear wall model that was used throughout this study was selected after checking its accuracy and consistency by comparing the experimental data obtained from the full-scale seven-story reinforced concrete shear wall building tested under PsD loading with the analytical results obtained by applying the same earthquake record in the test to the building model. In the nonlinear analyses, seven different ground motion time histories each having two components, one for each principal direction, were used. The average of the data obtained from each earthquake record was utilized in evaluating the results of the analyses.

The goal of this study is to investigate the structural behavior of mid-rise buildings with different shear wall area to floor area ratios under earthquake loading and study the improvement in overall behavior with increasing wall ratios. To ensure that shear wall area to floor area ratio is the governing parameter that could be used for determining the seismic behavior of reinforced concrete buildings, two other key parameters were also investigated. One of these parameters is the ratio of the summation of total column area and shear wall area to the floor area, which also includes the effect of number of columns and the column dimensions on the overall behavior. The second parameter is selected as the shear wall configuration to examine the effect of the location of the shear wall relative to the center of gravity of the building on the seismic behavior. The results obtained from the nonlinear time-history analyses including roof drift, the magnitude and distribution of interstory drifts and the base shear characteristics and story shear distribution were evaluated to obtain the effect of the aforementioned parameters on the dynamic performance of the buildings.

5.2. CONCLUSIONS

The analytical study on the effect of shear wall area to floor area ratio on the behavior of reinforced concrete buildings under earthquake loading led to the following conclusions restricted to the outcomes of this study:

• It was observed that as the shear wall ratio increased, building behavior was improved in terms of drift, however, when this ratio exceeded 1.5%, the reduction in the values of roof and maximum interstory drift became less pronounced when compared to the reduction in drift levels for a change of the shear wall ratio beyond 0.5%. At this point, it is worth to mention that none of the computed average interstory drifts exceeded 1.0 percent, for both 5 and 8 story buildings investigated in this study, which means that drifts generally were not critical. However, observations on 5 story buildings indicated that at least 1.0 % of shear wall ratio should be provided during design stages in order to control the drift of a shear wall-frame structure.

• When the time of first yield of the load carrying members in 5 story building models with increasing shear wall ratio was examined, it was observed that there is no clear trend between first yielding of the members and the shear wall ratio. However, it was noted that the number of total yielded members varies in different building models depending on the shear wall ratio. As the shear wall ratio increases up to 1.5 %, a decrease in percentage of yielded members is likely even under strong ground motions. This effect seems to diminish after 1.5% shear wall area to floor area ratio.

• In general, when the shear wall area to floor area ratio increased, the roof drift and interstory drift decreased. For 8 story buildings, the drift values for two orthogonal directions had similar magnitudes; however, for 5 story buildings the difference is more significant especially for 1.0% and higher shear wall ratios. This might be due to the fact that natural periods of the 5 story buildings were generally within the interval of periods in which the fundamental period of the earthquake records were observed, leading to drift amplifications in the direction of the strong component, which was applied in the Y-direction. It was observed that roof drifts were generally smaller in magnitude when compared to maximum interstory drifts; however, their general trend was quite similar. For both 5 and 8 story buildings, the maximum interstory drifts were observed in between the top two stories when the shear wall ratio is greater than 0.5%.

• The percentage of the base shear force carried by the ground floor shear walls increased significantly until the shear wall ratio has reached 1.5%. However the rate of increase was considerably higher between 0.5% and 1.0% shear wall ratios than between 1.0% and 1.5%. In buildings having a shear wall ratio of 1.0%, ground floor shear walls carried about 80% of the base shear. The base shear force carried by the walls was observed to be more than 90% for the wall ratio of 1.5% and 2.0%, however the reflection of this increase on the drift performance was not significant.

• As the number of stories increased from 5 to 8, the base shear force percentage carried by the shear walls was not significantly affected. The change in base shear percentages of the walls was also negligible when different shear wall configurations were compared. As expected, when the total area of columns increased, the base shear forces transferred to the shear walls decreased.

• When the column area was added to the shear wall area while computing the ratio of the total vertical member area to the floor area, the results in terms of base shear carried by the structural walls, roof drift and maximum interstory drift were rather scattered. Because, the summation was not able to capture the degradation in the overall behavior of building models with low shear wall ratios when the total column area was relatively high. Therefore, when the total area of the vertical elements in a building is considered in predicting the dynamic behavior, a minimum limit should be set for the shear wall area to floor area ratio.

• An increase in the column ratio, while keeping constant shear wall ratio of 1.0% in each direction, did not significantly influence the drift levels. This result could be supported with the fact that the base shear force was substantially carried by shear walls leading to shear wall dominated behavior throughout the dynamic action.

• Since symmetrical buildings with regular plans were examined during this study, it was observed that the behavior was dominated by the first two modes of the response related to each orthogonal direction. The contribution of the torsional response was not a main concern in dynamic behavior of the buildings, so characteristics of the shear walls such as rigidity and moment capacity were more critical parameters when compared to the location of the shear walls in terms of drift performance.

5.3. RECOMMENDATIONS FOR FUTURE RESEARCH

In this analytical study, only mid-rise buildings with regular plan and shear wall configurations were investigated. The effect of number of stories; different floor plans including geometric irregularities and unsymmetrical wall configurations leading to torsional irregularity could be investigated. In addition, soft and weak story mechanisms could be examined. Moreover, soil-structure interaction can be taken into account.

Rectangular shear wall sections were used in this study. The effect of different wall sections, such as T-shaped, L-shaped or box sections, and different wall aspect ratios on the dynamic performance could be investigated.

Even though the consistency of the equivalent beam approach was verified by comparing it with the fiber model as being a more sophisticated model for shear walls, other models proposed in the literature that have additional features by the use of finite element modeling could be used to enhance the accuracy even further, without a significant change in the computational time.

To obtain more realistic results, nonlinear time history analysis was used throughout the study. Pushover analysis which is more popular in current practice could be utilized in order to investigate the consistency provided by these two different nonlinear analysis approaches and to provide optimum shear wall area to floor area ratios for different limit states.

It was observed that, the nonlinear behavior of the buildings affected significantly from the characteristics of the selected earthquake record. Different earthquake records with different characteristics could be applied to similar buildings to see possible amplifications in different parameters. This could lead to a more improved understanding on the behavior of reinforced concrete shear wall-frame buildings.

Throughout this analytical study, seven ground motions with two principal components were used and the average data was used in evaluation of the results. Earthquake records modified for two probabilities of exceedance, 10% in 50 years and 2% in 50 years, which would provide results that could be used to comment on the performance levels of the buildings could be applied to the structures.

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APPENDIX A

DATA AND ANALYTICAL RESULTS OF BUILDING MODELS

A.1. FLOOR PLANS OF GENERATED BUILDING MODELS

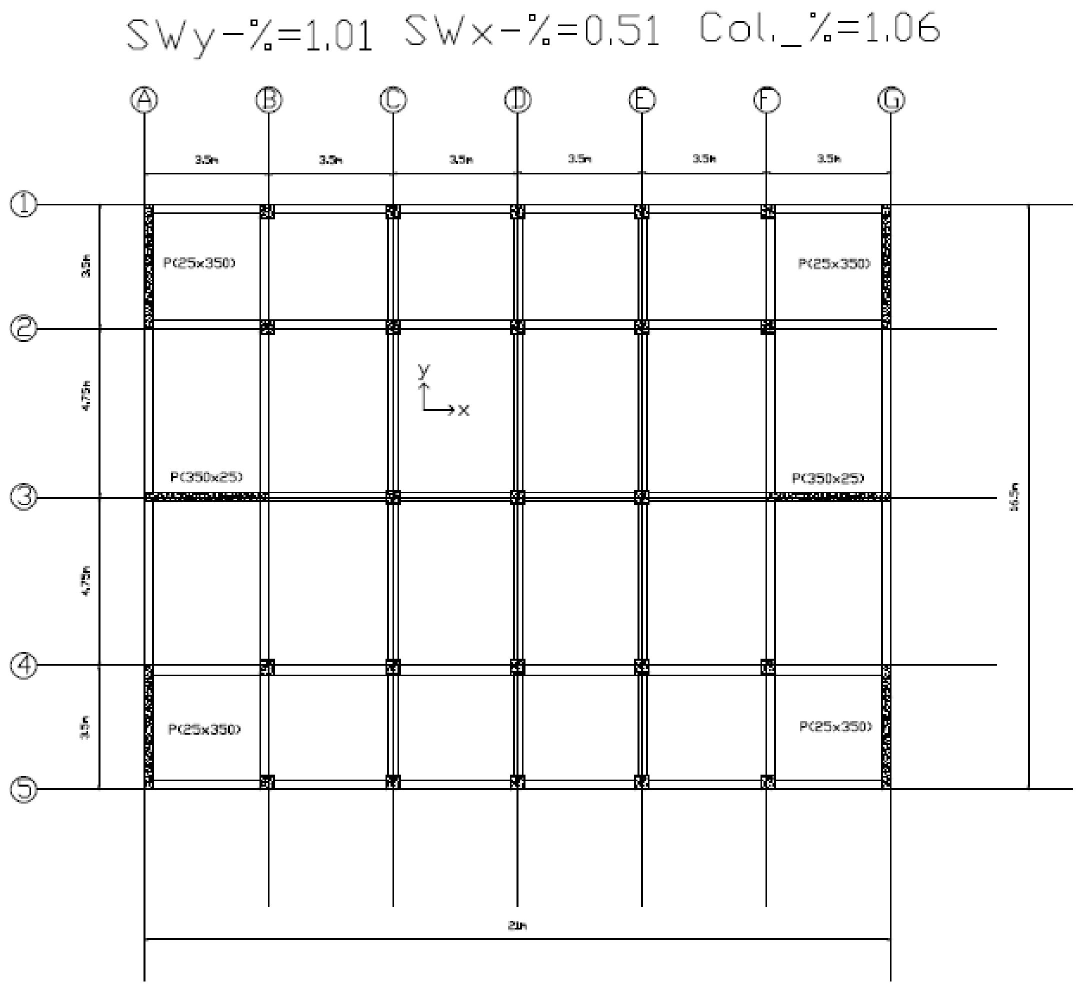


Figure A.1.1. Plan of Model 2 with Wall Area to Floor Area Ratio of 0.5% in X-direction and 1.0% in Y-direction

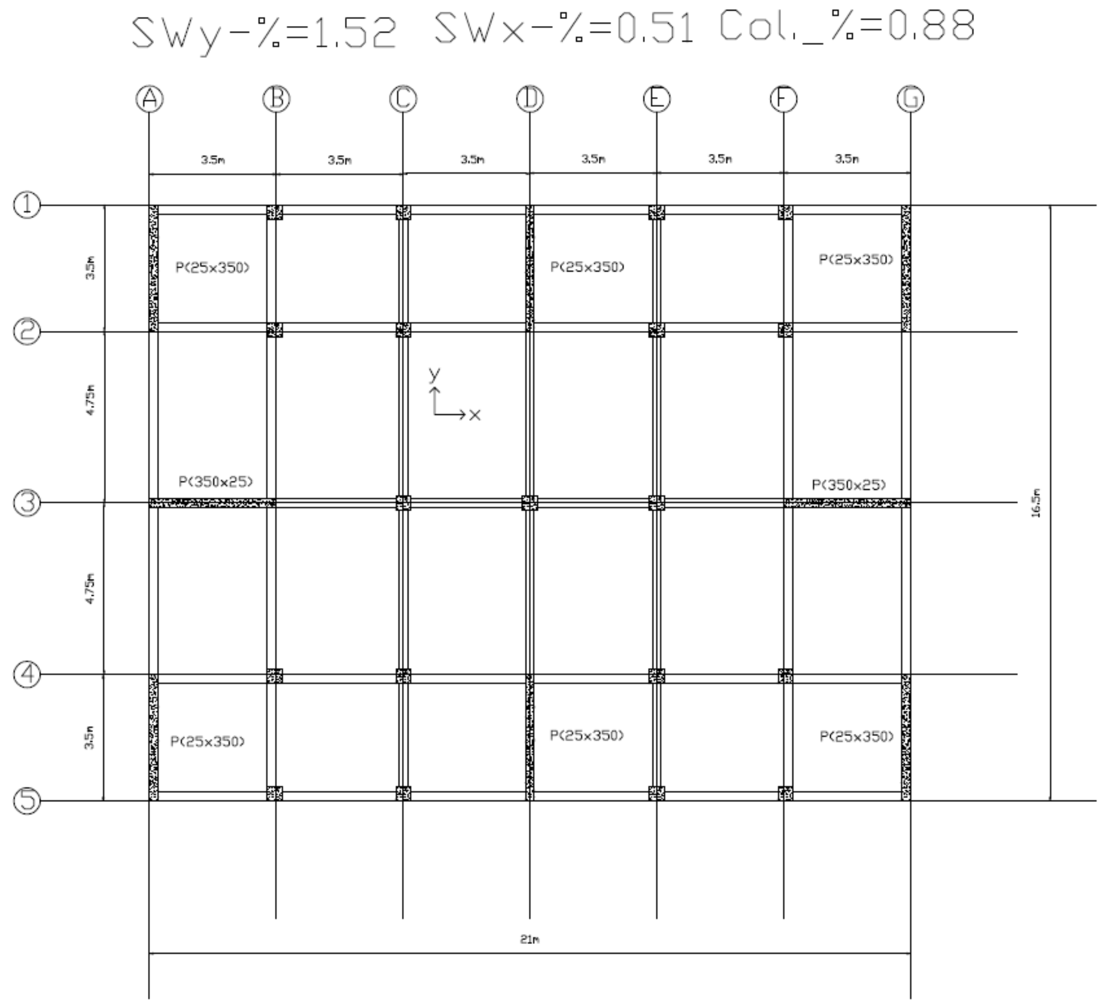


Figure A.1.2. Plan of Model 4 with Wall Area to Floor Area Ratio of 0.5% in X-direction and 1.5% in Y-direction

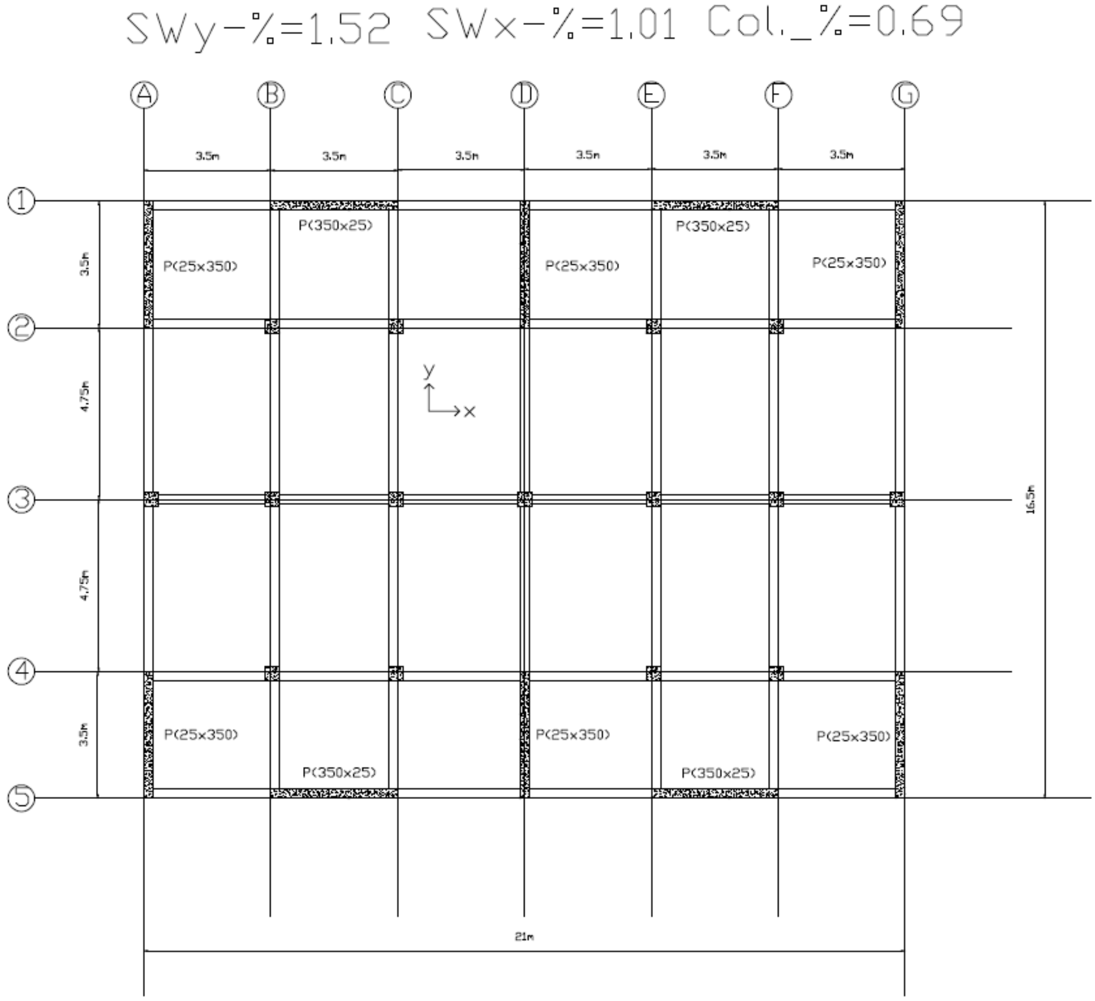


Figure A.1.3. Plan of Model 5 with Wall Area to Floor Area Ratio of 1.0% in X-direction and 1.5% in Y-direction

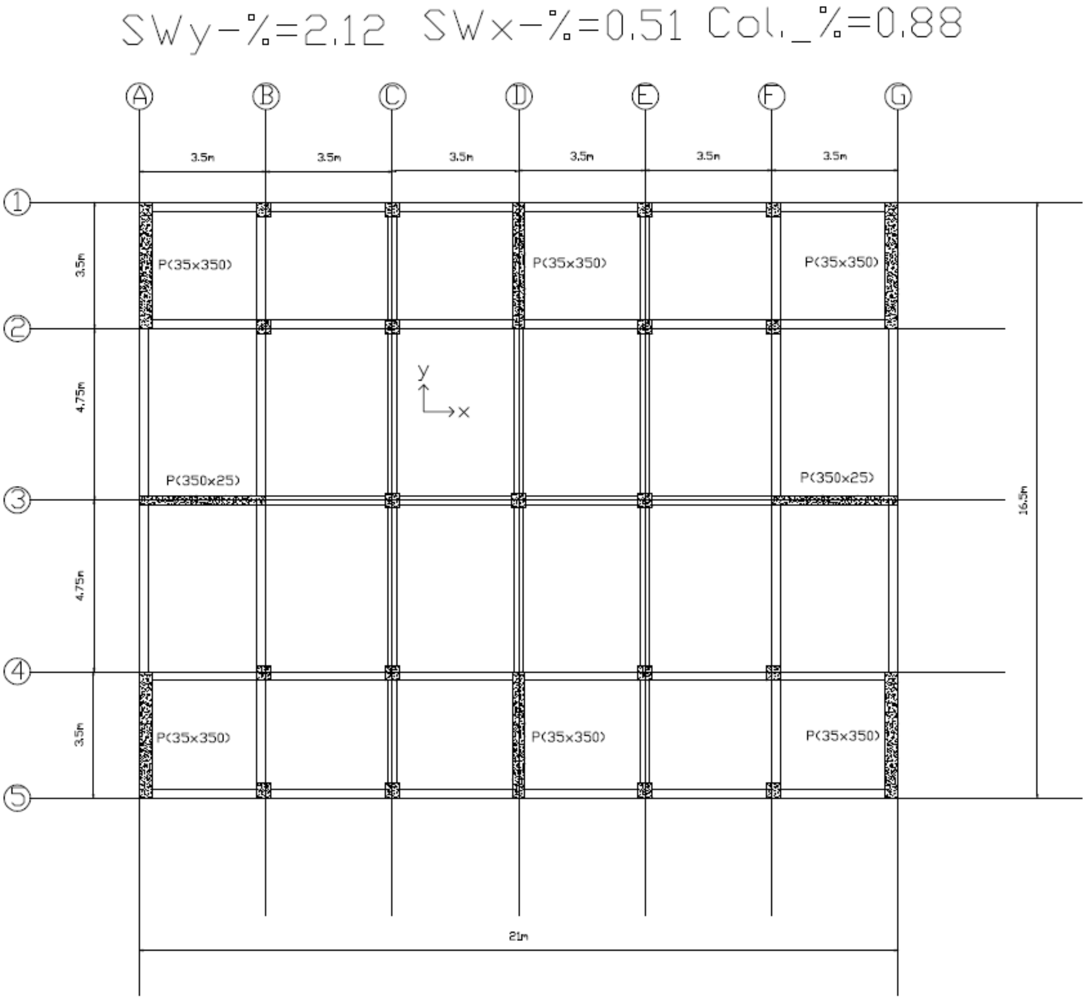


Figure A.1.4. Plan of Model 7 with Wall Area to Floor Area Ratio of 0.5% in X-direction and 2.0% in Y-direction

SWy-%=2.12 SWx-%=1.01 Col._%=0.69

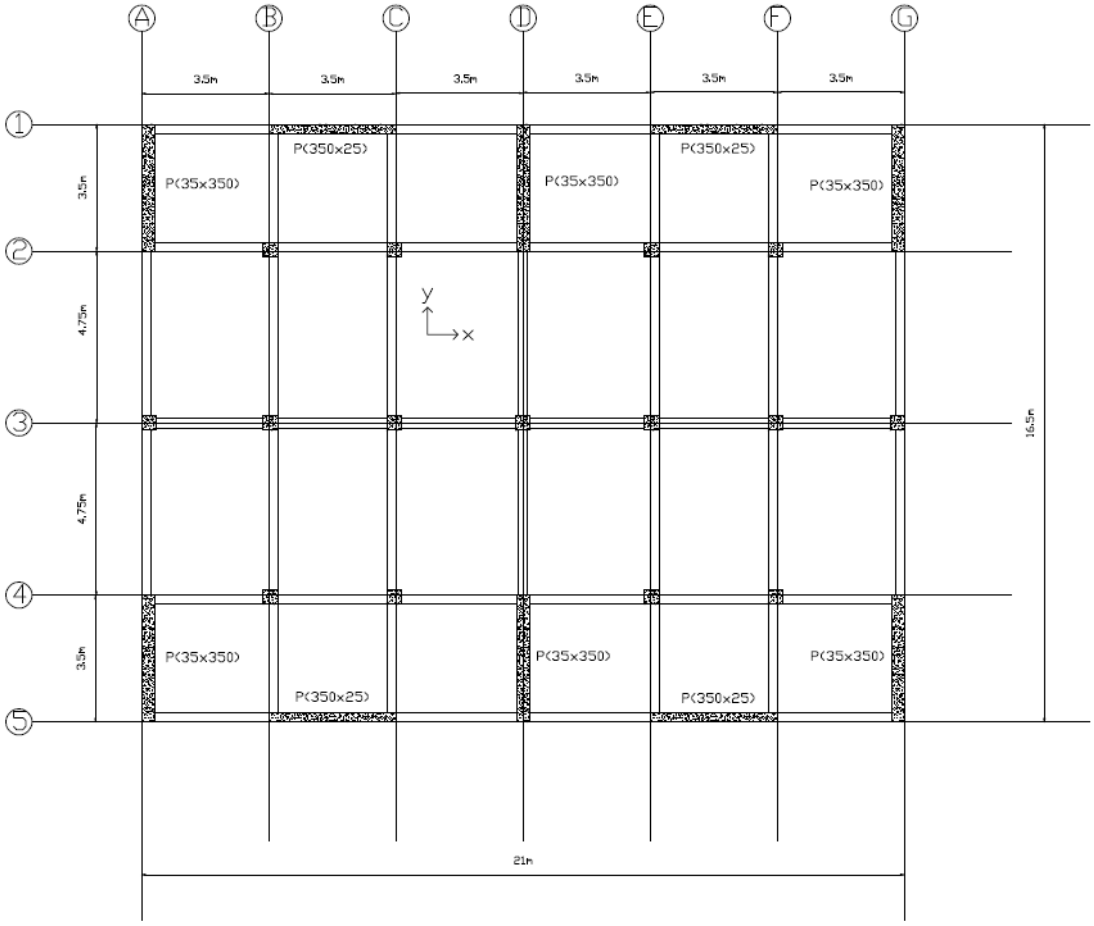


Figure A.1.5. Plan of Model 8 with Wall Area to Floor Area Ratio of 1.0% in X-direction and 2.0% in Y-direction

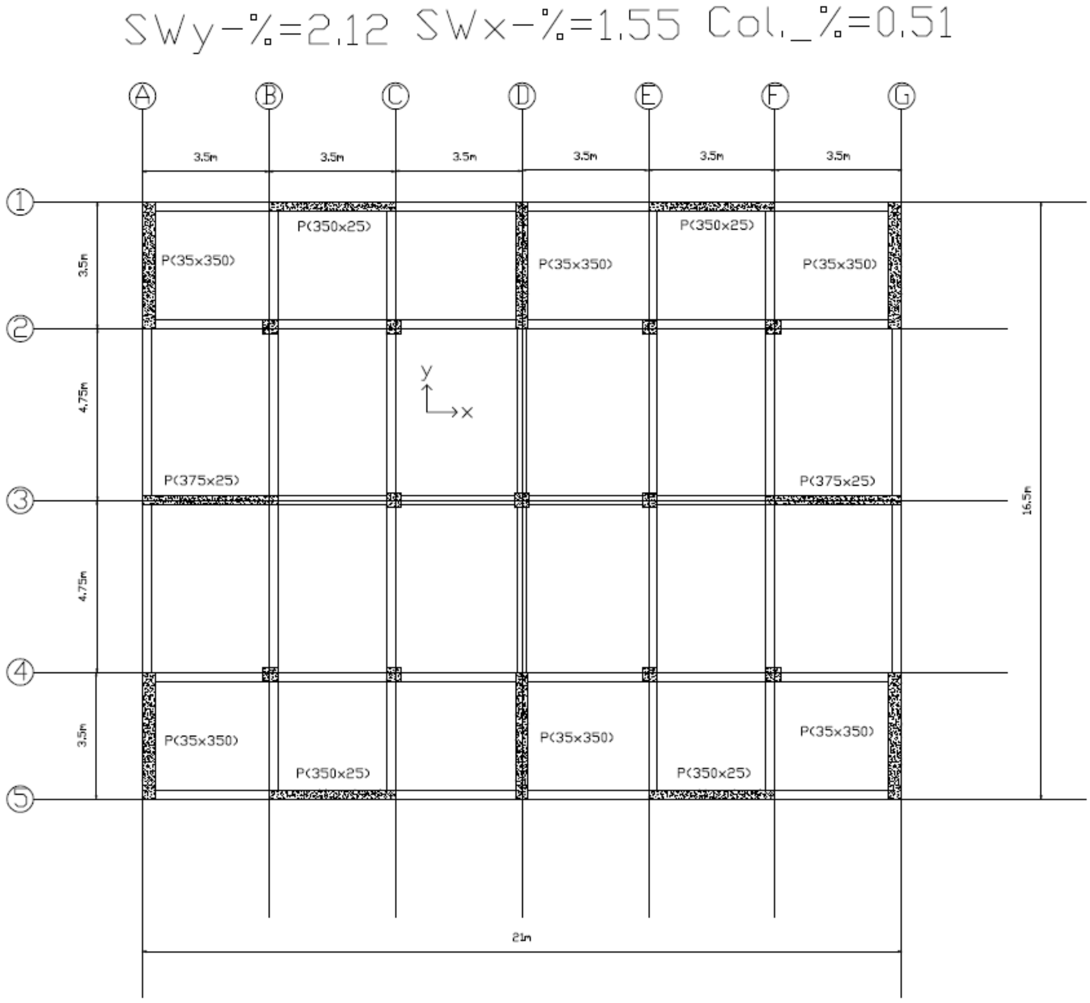


Figure A.1.6. Plan of Model 9 with Wall Area to Floor Area Ratio of 1.5% in X-direction and 2.0% in Y-direction

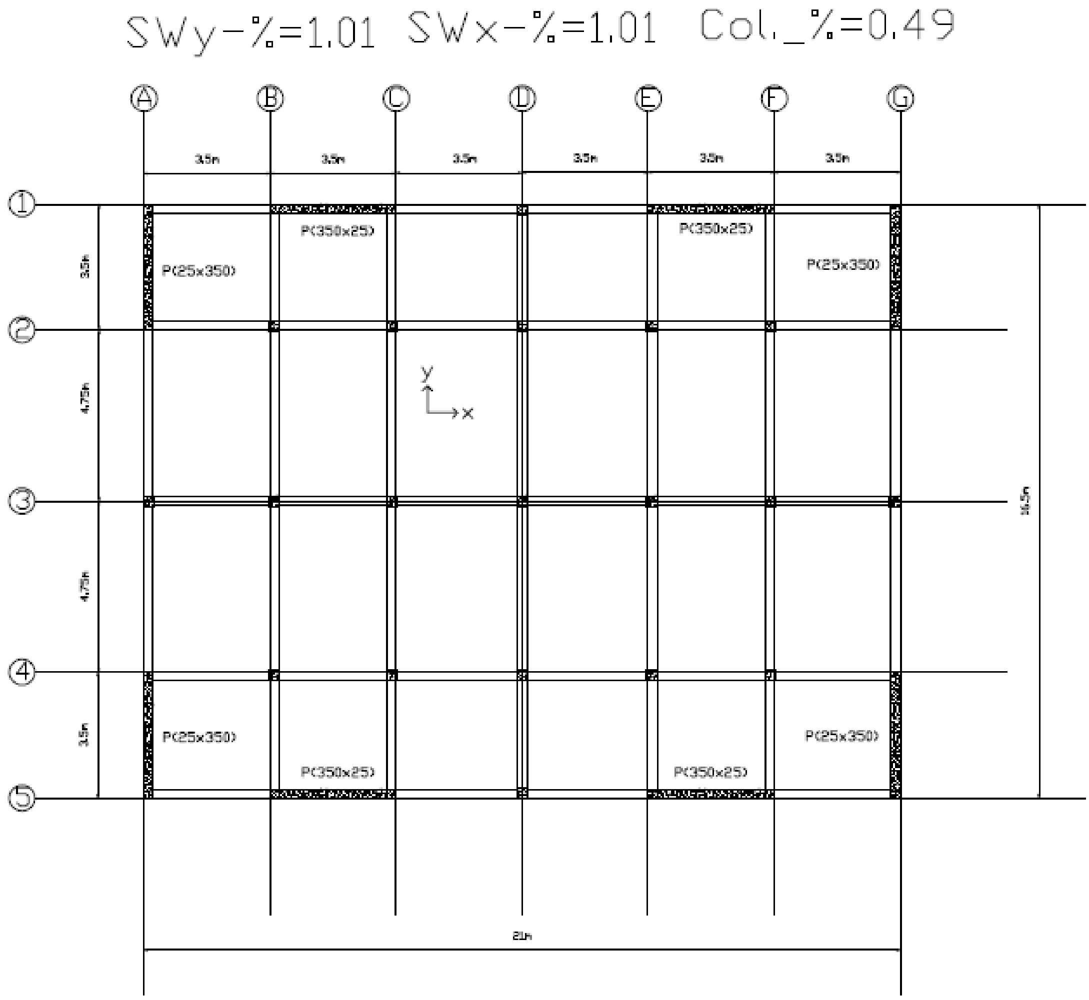


Figure A.1.7. Plan of Model 21 with 30x30 Column Size and Wall Area to Floor Area Ratio of 1.0% in each Direction

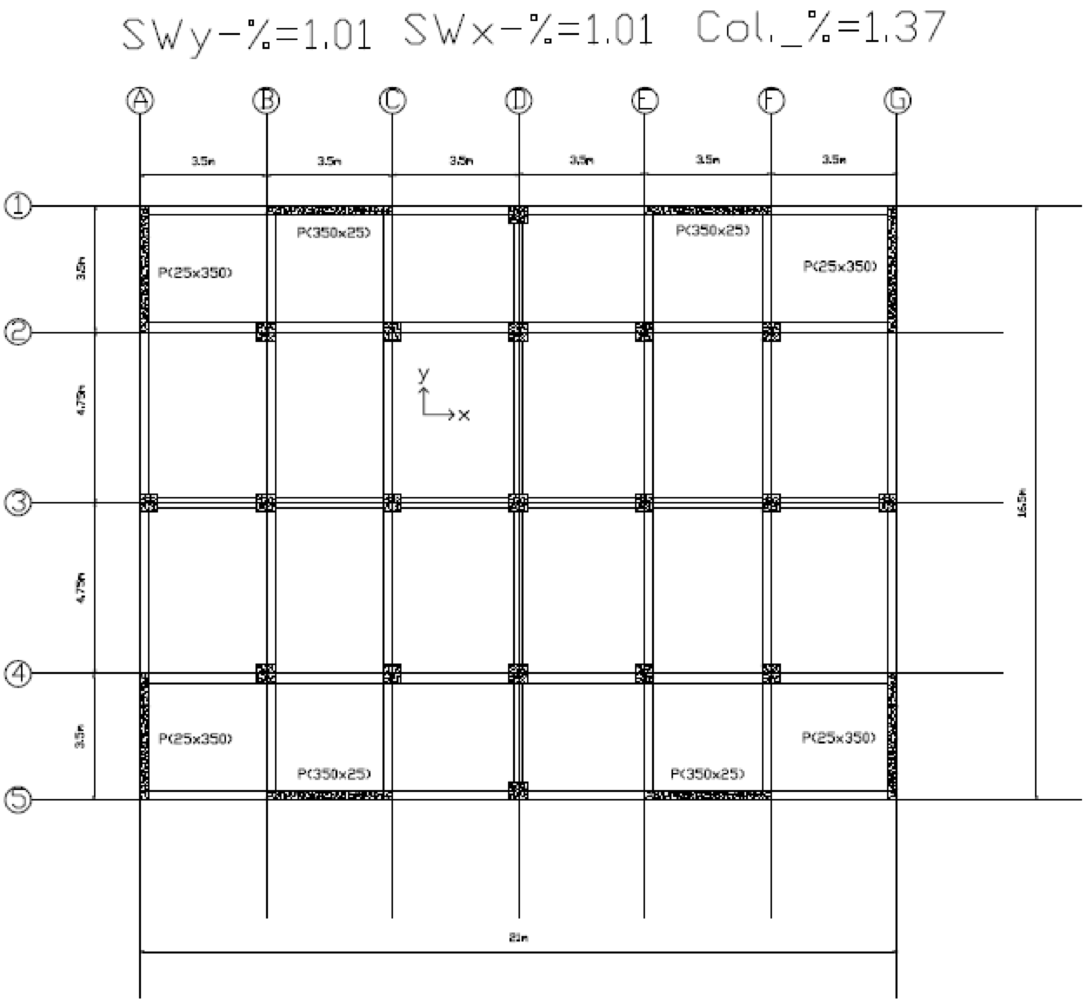


Figure A.1.8. Plan of Model 22 with 50x50 Column Size and Wall Area to Floor Area Ratio of 1.0% in each Direction

A.2. REINFORCEMENT DETAILS OF SEVEN-STORY FULL-SCALE TEST BUILDING

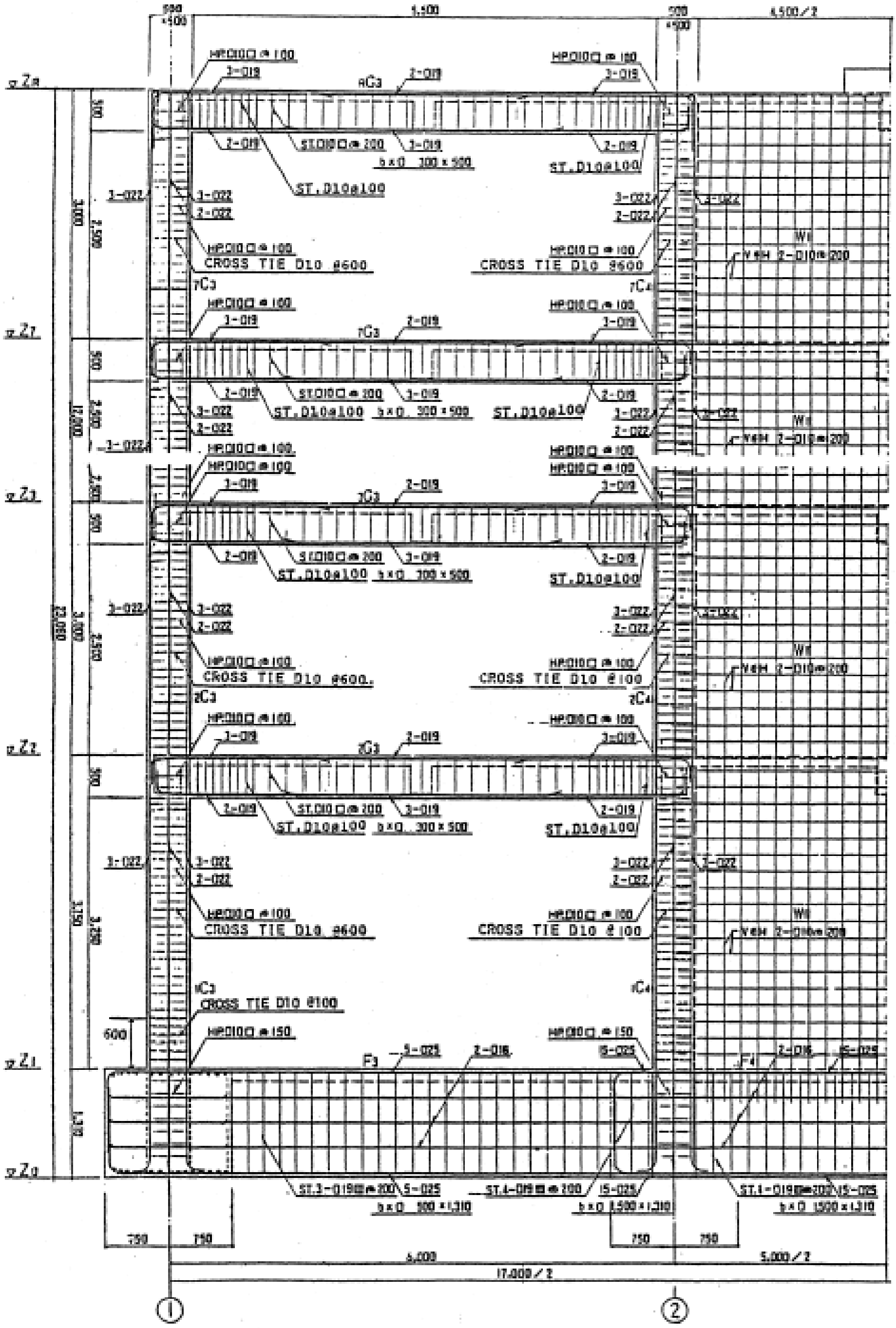


Figure A.2.1. Reinforcement Details of Frame B of Full-Scale Seven-Story Test Building [12]

A.3. EXAMPLES OF MOMENT CURVATURE ANALYSIS OF A BEAM IN GENERATED BUILDING MODELS

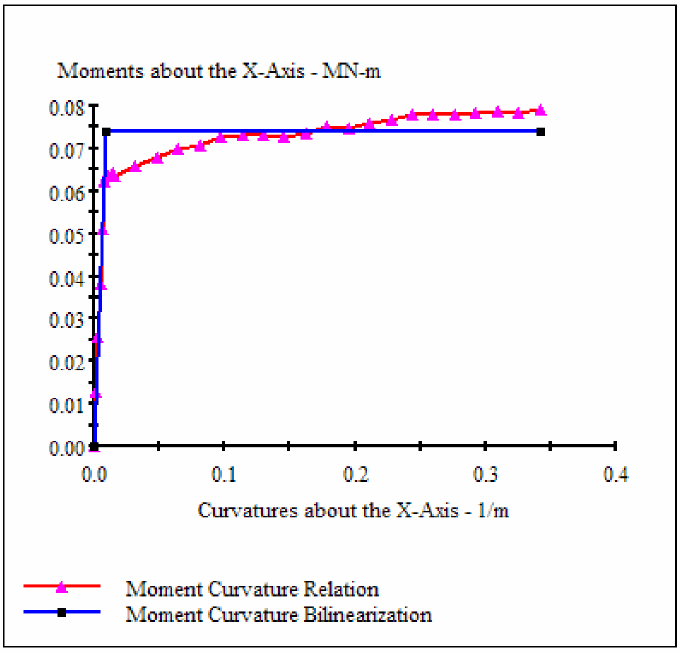


Figure A.3.1. Moment Curvature Analysis of a Beam (25x40cm) (Top in Tension)

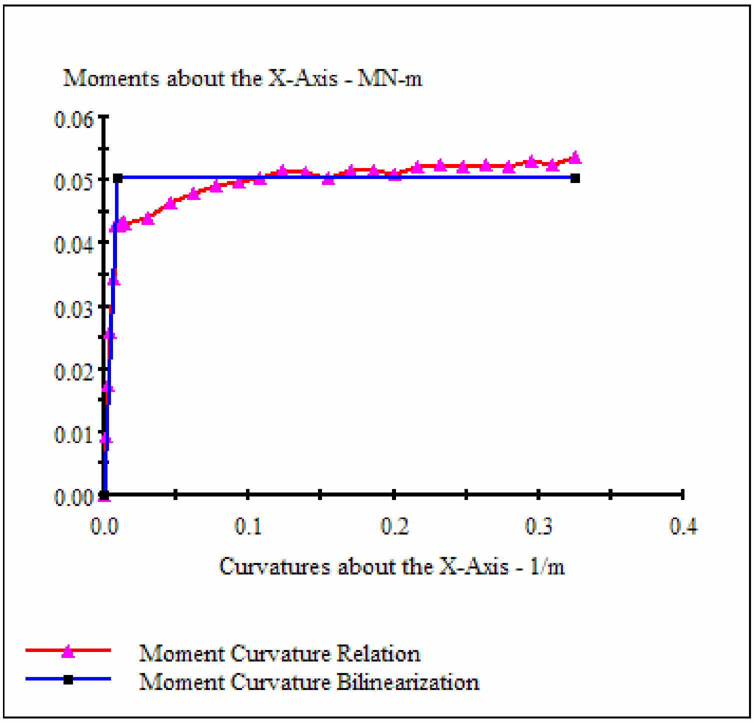


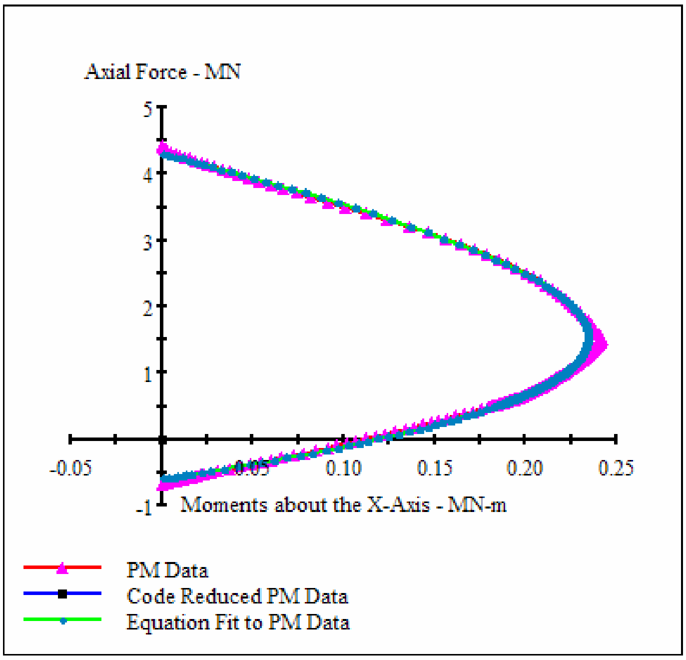
Figure A.3.2. Moment Curvature Analysis of Beam (25x40cm) (Bottom in Tension)

A.4. STIFFNESS AND ULTIMATE MOMENT PROPERTIES OF BEAMS IN SEVEN-STORY FULL-SCALE BUILDING

Table A.4.1. Stiffness and Ultimate Moment Properties of Beams in Seven-Story Full-Scale Building

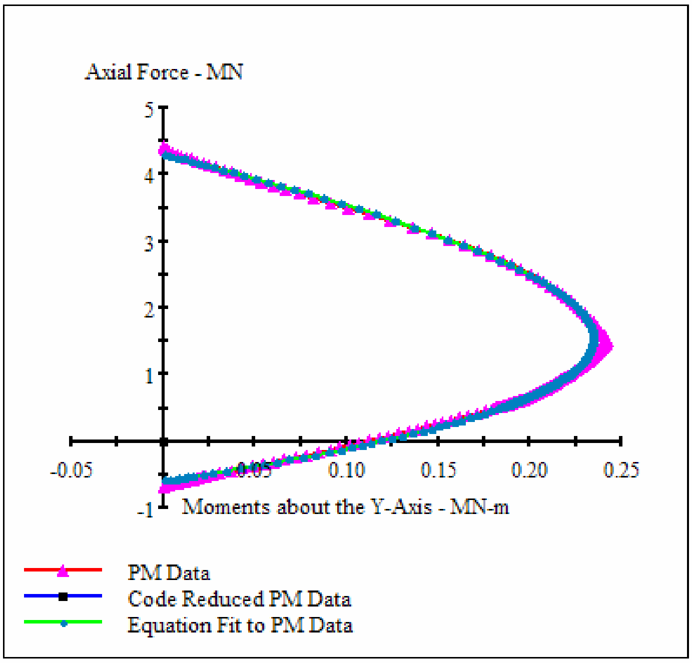
Beam ID	I_g (m^4)	My^+ (kN.m) Story 1-4	My^- (kN.m) Story 1-4	My^+ (kN.m) Story 5	My^- (kN.m) Story 5	My^+ (kN.m) Story 6-7	My^- (kN.m) Story 6-7
G1 & G3	0.0060	118.10	218.20	118.20	218.30	118.10	212.30
G2	0.0056	116.20	218.50	116.20	218.10	116.00	211.60
G4	0.0043	103.00	196.20	103.00	196.30	103.00	184.90
G5	0.0035	97.30	166.00	97.30	166.40	97.10	157.00
B1-i	0.0038	103.90	179.40	103.90	180.30	104.00	172.80
B1-o	0.0038	105.80	152.50	105.90	153.20	104.70	149.00
B2-i	0.0038	133.80	211.20	133.80	211.30	133.70	203.80
B2-o	0.0038	134.50	176.70	134.50	176.70	134.20	172.90

A.5. EXAMPLES OF PMM CURVES OF A COLUMN AND A SHEAR WALL IN GENERATED BUILDING MODELS



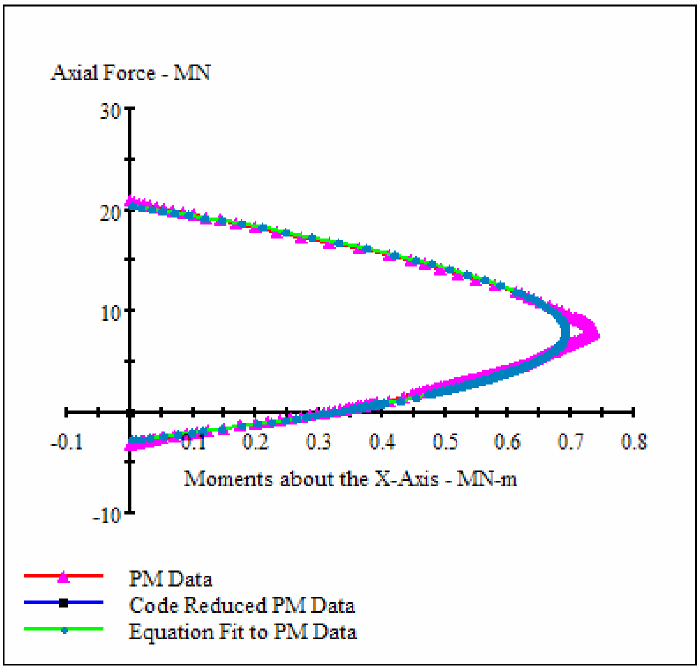
M(P) = .1186 + .7075*(P/Pu) + -1.200*(P/Pu)^2 + .3602*(P/Pu)^3

Figure A.5.1. PMM Curve for a Column (40x40cm) about X-X Axis



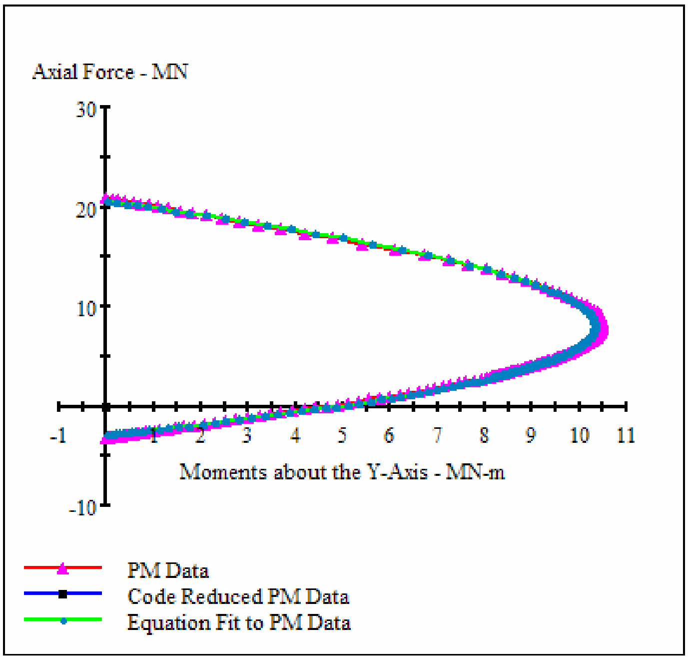
M(P) = .1186 + .7075*(P/Pu) + -1.201*(P/Pu)^2 + .3602*(P/Pu)^3

Figure A.5.2. PMM Curve for a Column (40x40cm) about Y-Y Axis



M(P) = .3220 + 2.006*(P/Pu) + -2.940*(P/Pu)^2 + .5730*(P/Pu)^3

Figure A.5.3. PMM Curve for a Shear Wall (350x25cm) about X-X Axis



M(P) = 4.871 + 29.33*(P/Pu) + -42.36*(P/Pu)^2 + 7.733*(P/Pu)^3

Figure A.5.4. PMM Curve for a Shear Wall (350x25cm) about Y-Y Axis

A.6. STIFFNESS, MOMENT AND AXIAL LOAD CAPACITIES OF COLUMNS AND SHEAR WALLS IN SEVEN-STORY FULL-SCALE BUILDING

Table A.6.1 Stiffness, Moment and Axial Load Capacities of Columns in Seven-Story Full-Scale Building

Story	I_g (m^4)	M_{bal} (kN.m)	P_c (kN)	P_{bal} (kN.m)	P_t (kN)
(1-2)	0.0052	502.50	8018	2515	1095
(3-4)	0.0052	502.50	8018	2515	1095
(-5-)	0.0052	507.80	8176	2552	1095
(6-7)	0.0052	428.50	5870	1963	1095

Table A.6.2 Stiffness, Moment and Axial Load Capacities of Shear Walls in Seven-Story Full-Scale Building

Story	I_{gx} (m^4)	I_{gy} (m^4)	M_{bal-x} (kN.m)	M_{bal-y} (kN.m)	P_c (kN)	P_{bal} (kN.m)	P_t (kN)
(1-2)	4.6542	0.0134	13380	553.6	23880	9613	1433
(3-4)	4.6542	0.0134	13380	553.6	23880	9613	1433
(-5-)	4.6542	0.0134	13640	560.4	24385	9781.5	1433
(6-7)	4.6542	0.0134	9637	451.2	16630	7004	1433

A.7. BASE SHEAR VERSUS ROOF DRIFT RELATIONSHIP FOR MODEL 3

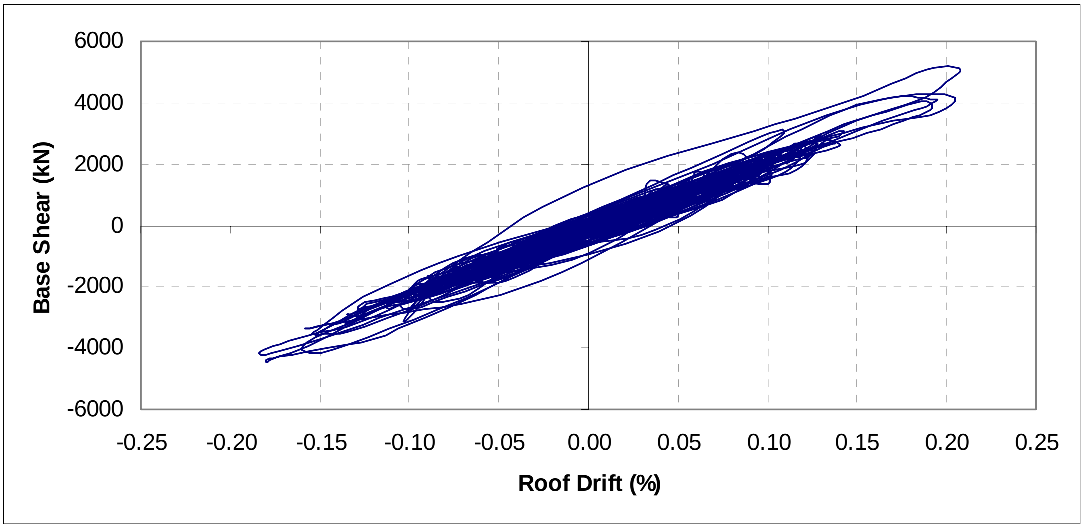


Figure A.7.1. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-direction under El Centro Record

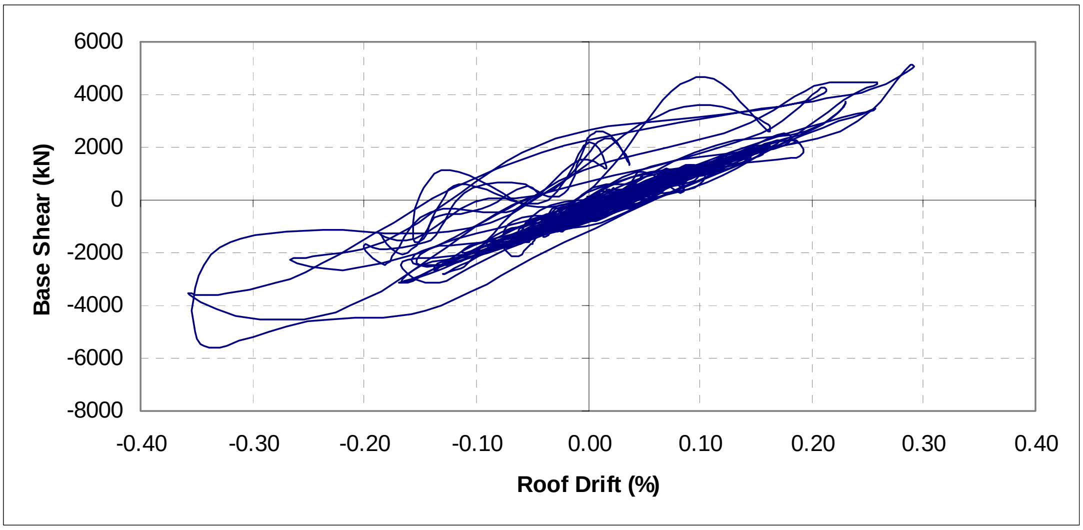


Figure A.7.2. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-direction under El Centro Record

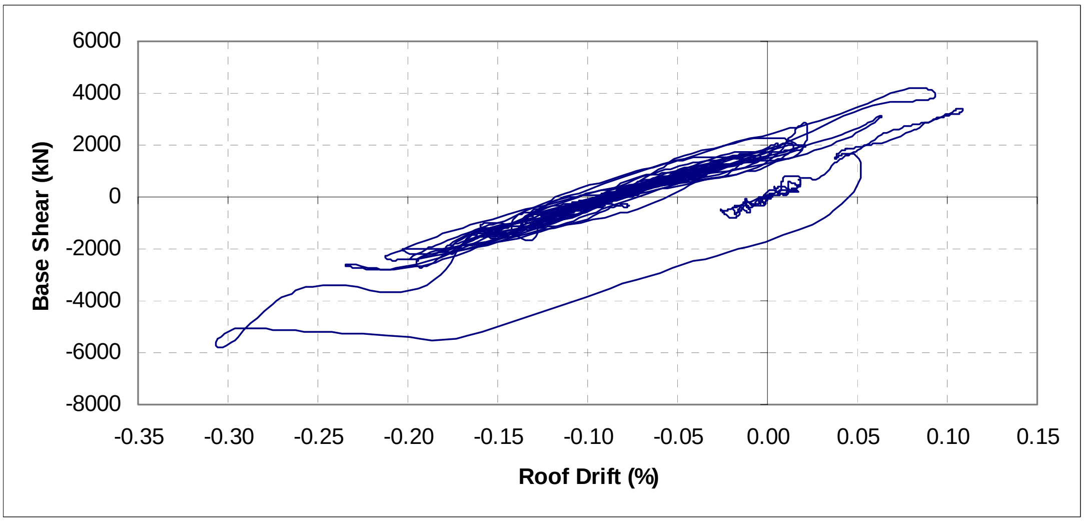


Figure A.7.3. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-direction under Erzincan Record

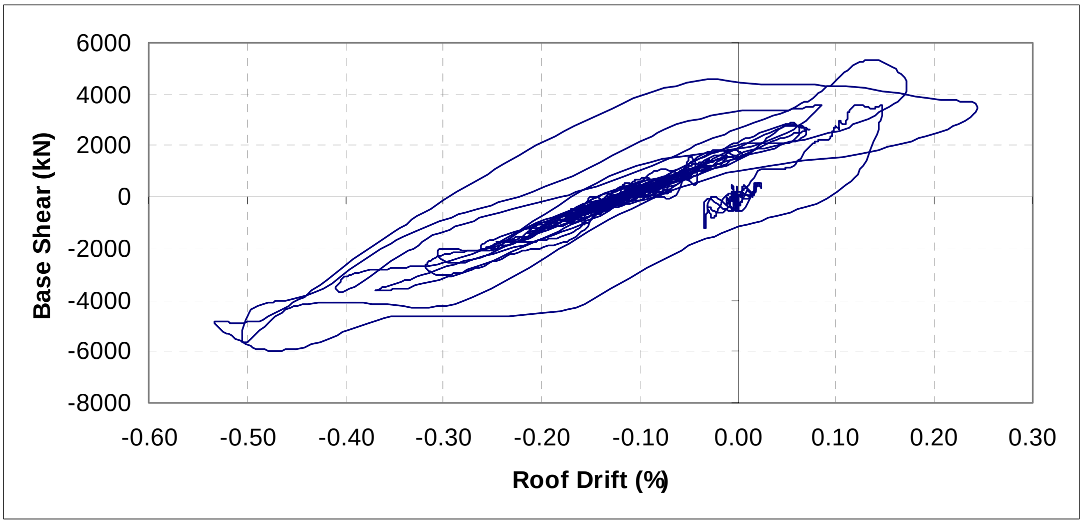


Figure A.7.4. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-direction under Erzincan Record

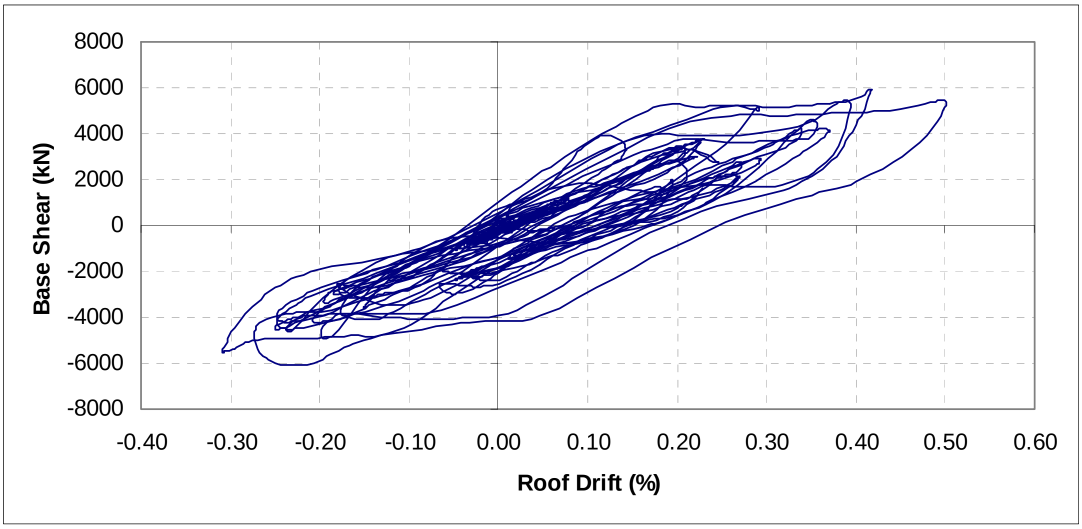


Figure A.7.5. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-direction under Duzce Record

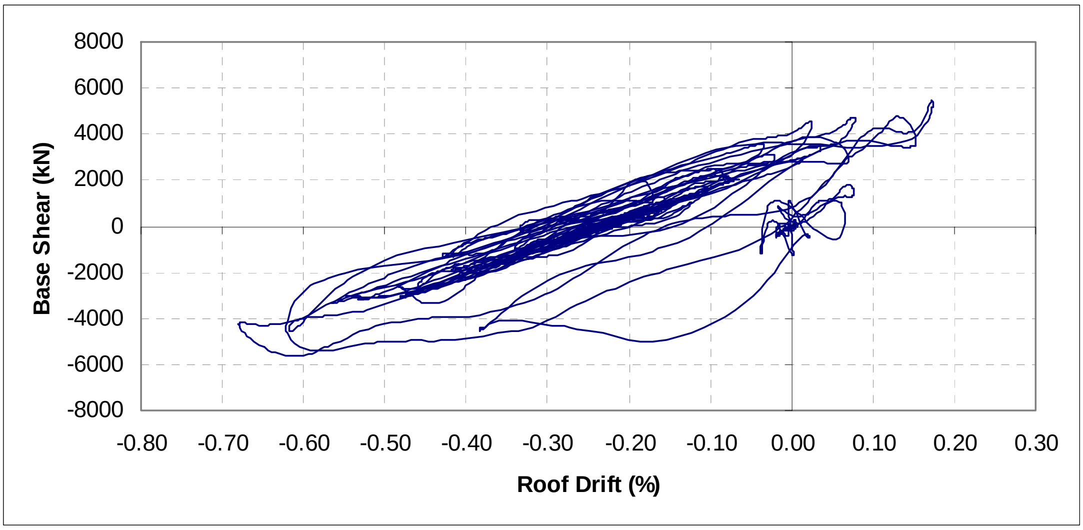


Figure A.7.6. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-direction under Duzce Record

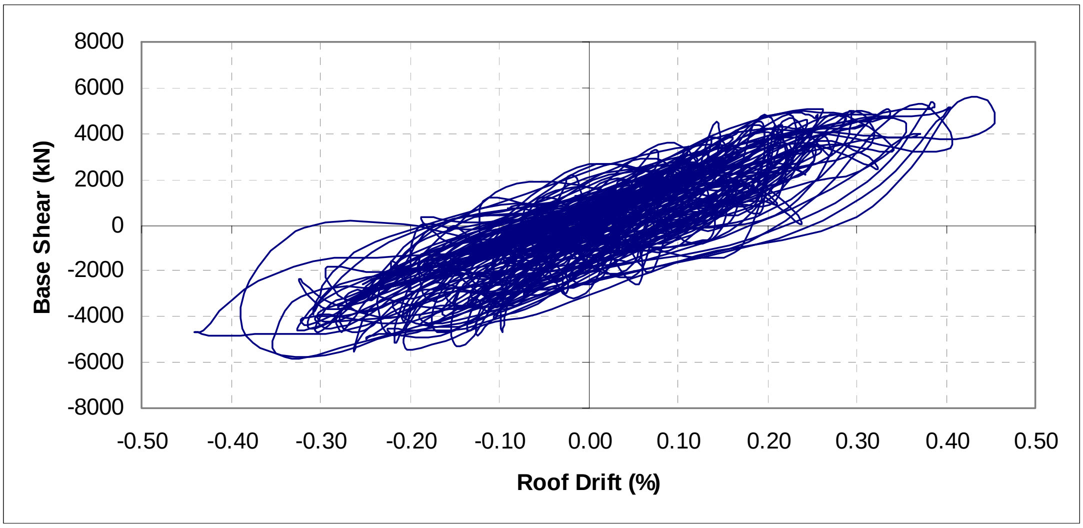


Figure A.7.7. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-direction under Chile Record

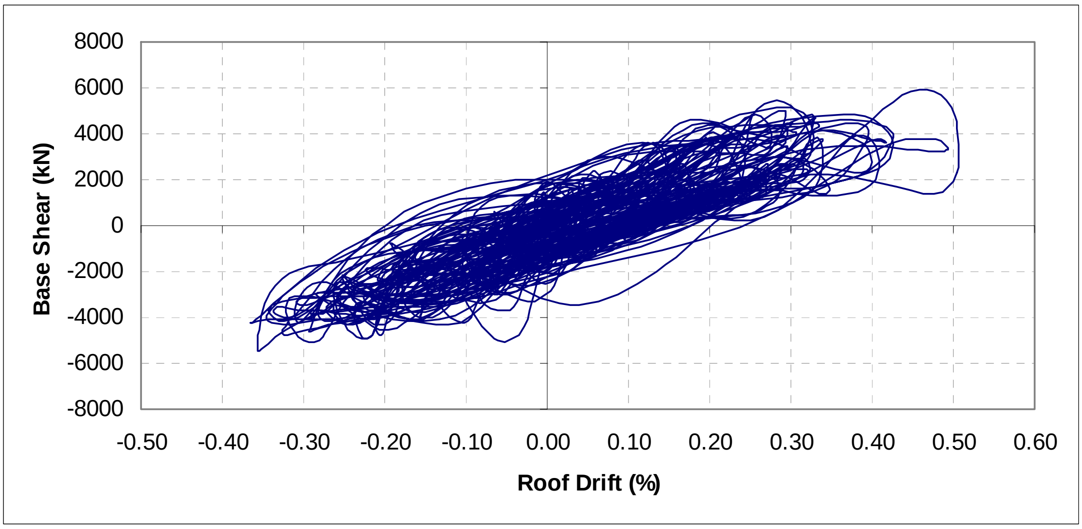


Figure A.7.8. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-direction under Chile Record

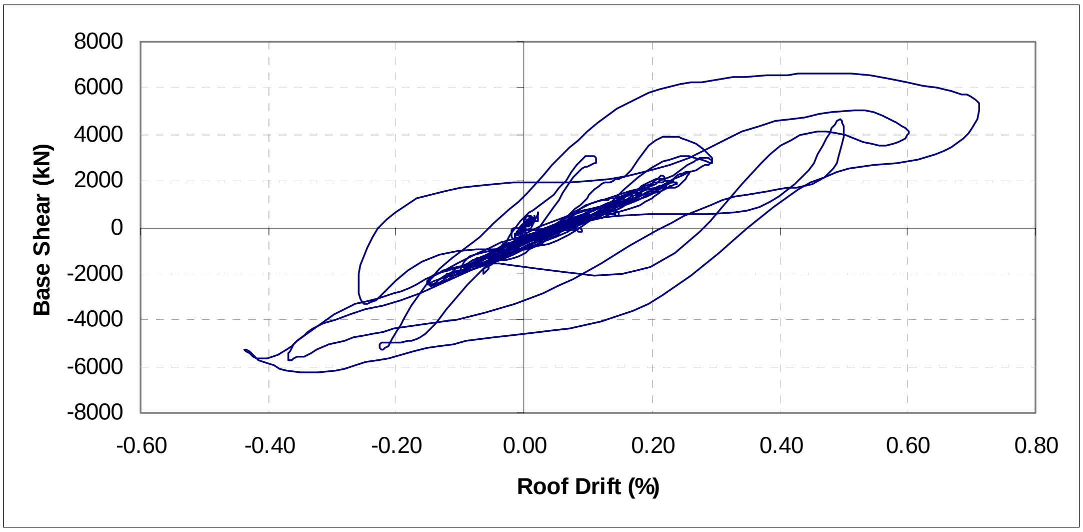


Figure A.7.9. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the X-direction under Kobe Record

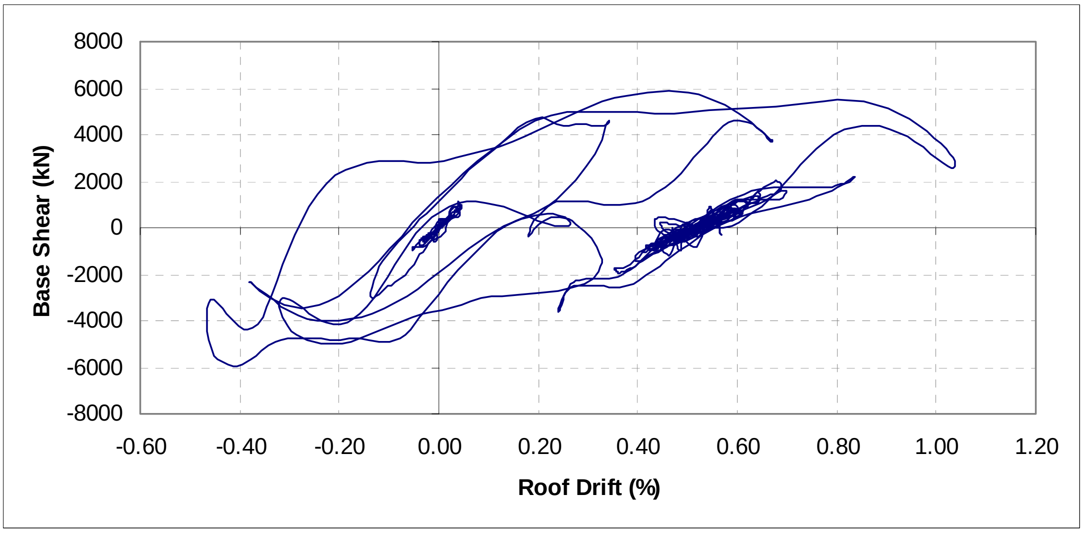


Figure A.7.10. Base Shear versus Roof Drift Relationship of 5 Story Building with 1.0% Shear Wall Ratio (Model 3) in the Y-direction under Kobe Record

A.8. DISTRIBUTION OF MAXIMUM BASE SHEAR AND DRIFT

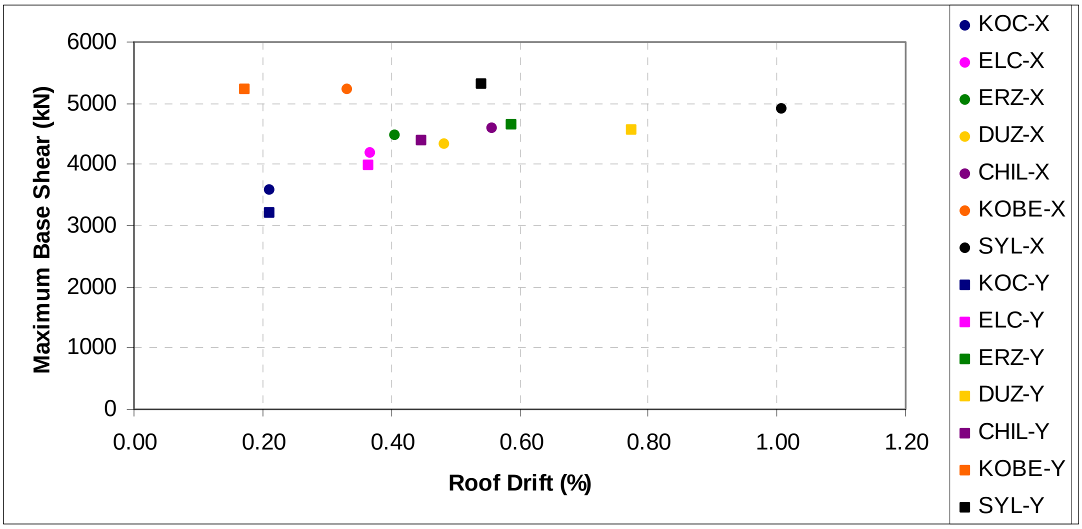


Figure A.8.1. Maximum Base Shear vs. Roof Drift Relationship of 5 Story Building with 0.5 % Shear Wall Ratio (Model 1) under All Selected Earthquake Records

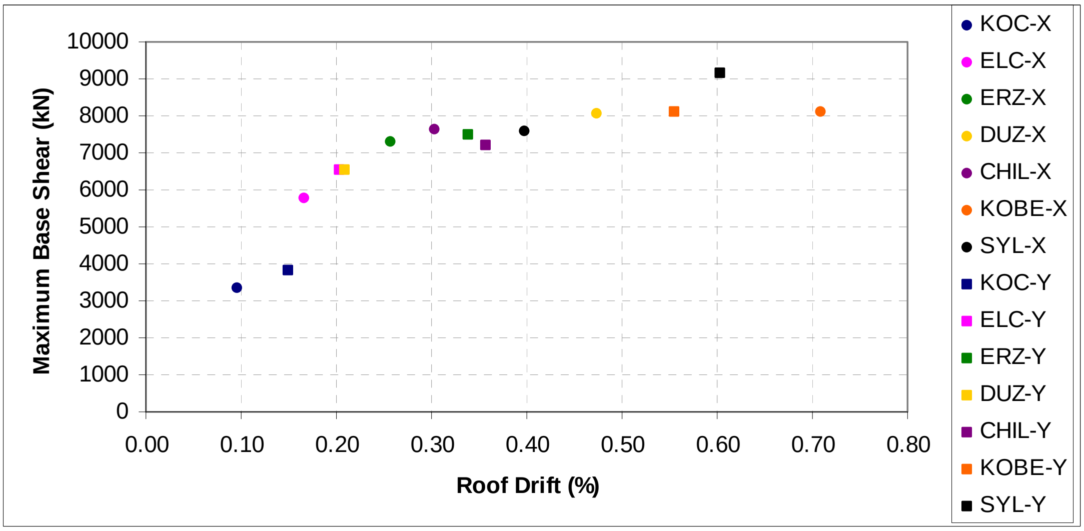


Figure A.8.2. Maximum Base Shear vs. Roof Drift Relationship of 5 Story Building with 1.5 % Shear Wall Ratio (Model 6) under All Selected Earthquake Records

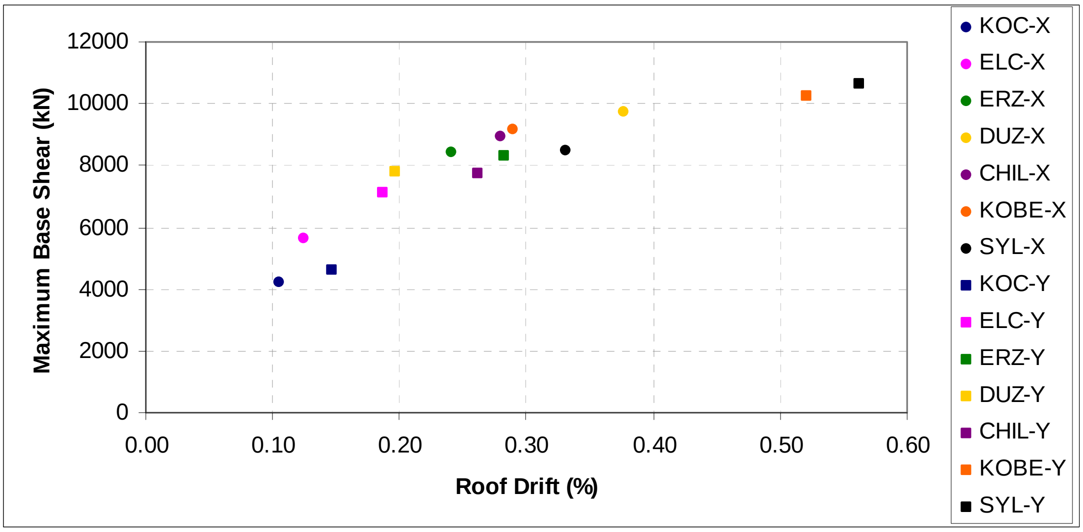


Figure A.8.3. Maximum Base Shear vs. Roof Drift Relationship of 5 Story Building with 2.0 % Shear Wall Ratio (Model 10) under All Selected Earthquake Records

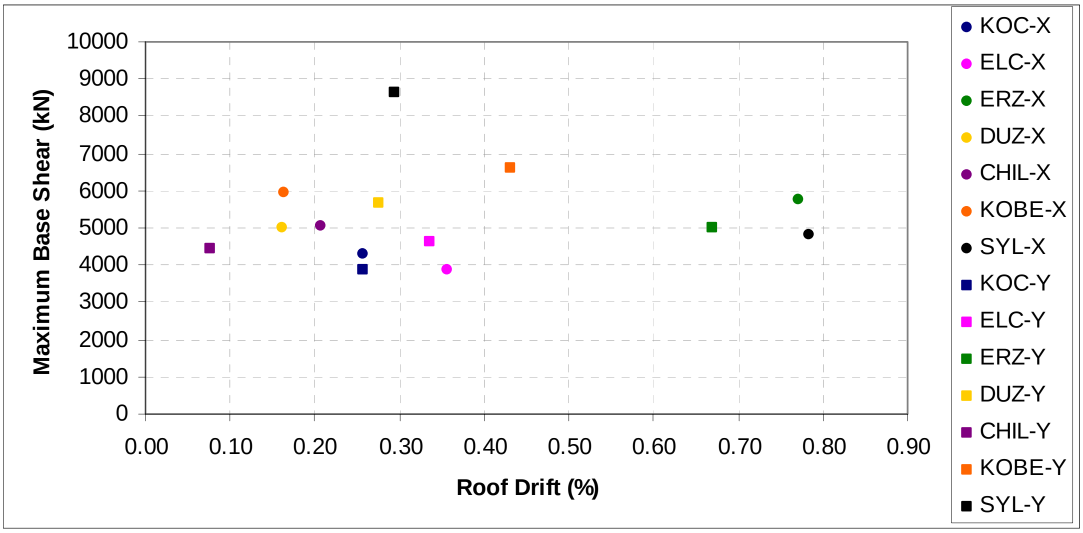


Figure A.8.4. Maximum Base Shear vs. Roof Drift Relationship of 8 Story Building with 0.5 % Shear Wall Ratio (Model 11) under All Selected Earthquake Records

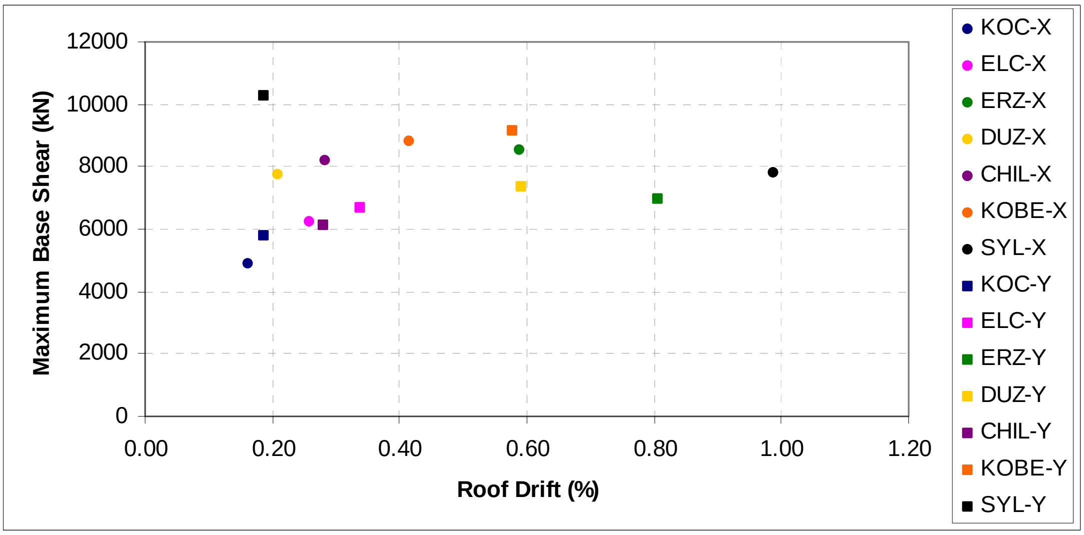


Figure A.8.5. Maximum Base Shear vs. Roof Drift Relationship of 8 Story Building with 1.5 % Shear Wall Ratio (Model 16) under All Selected Earthquake Records

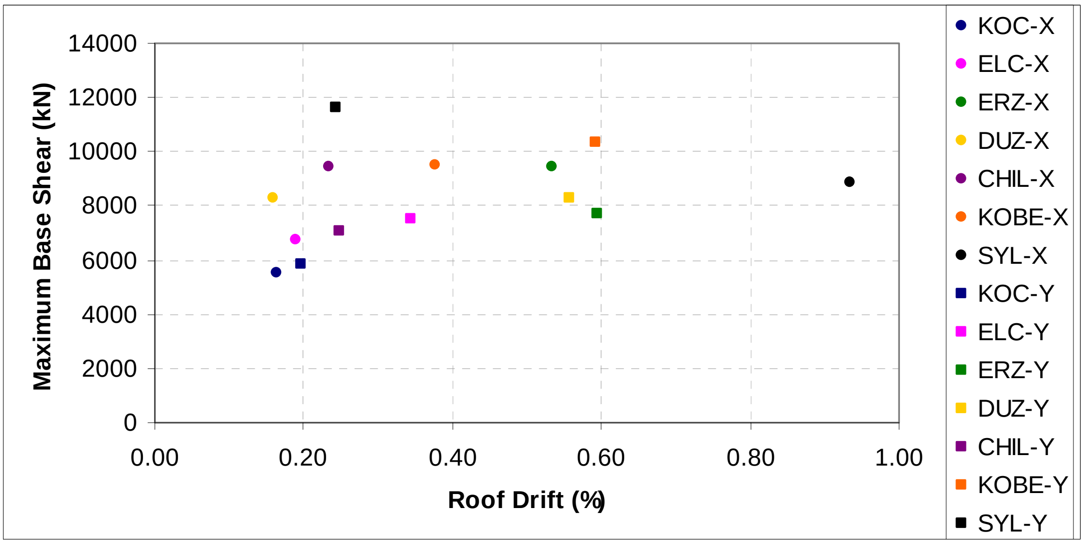


Figure A.8.6. Maximum Base Shear vs. Roof Drift Relationship of 8 Story Building with 2.0 % Shear Wall Ratio (Model 20) under All Selected Earthquake Records

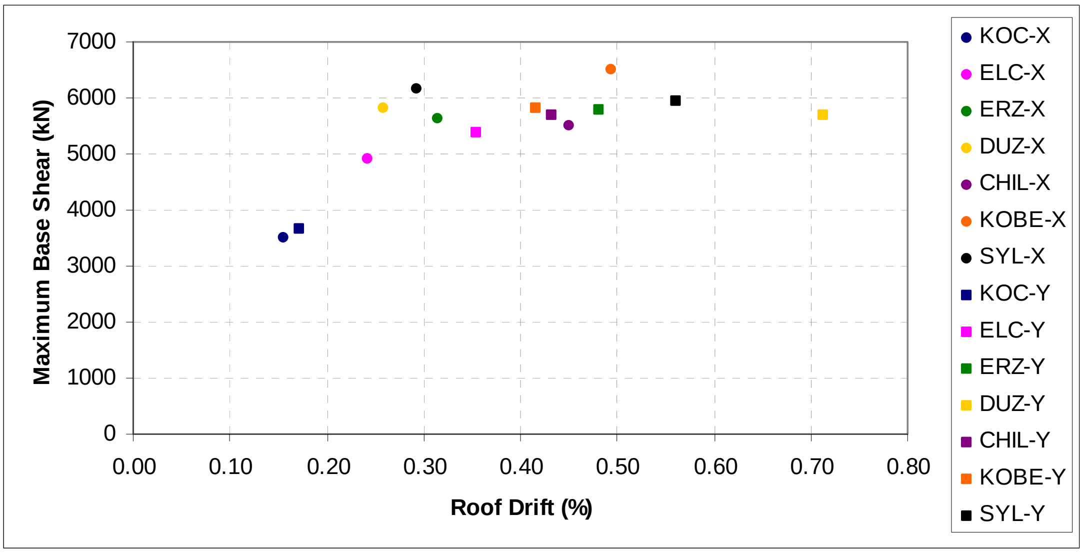


Figure A.8.7. Maximum Base Shear vs. Roof Drift Relationship of 5 Story Building with 1.0 % Shear Wall Ratio (Model 21) under All Selected Earthquake Records

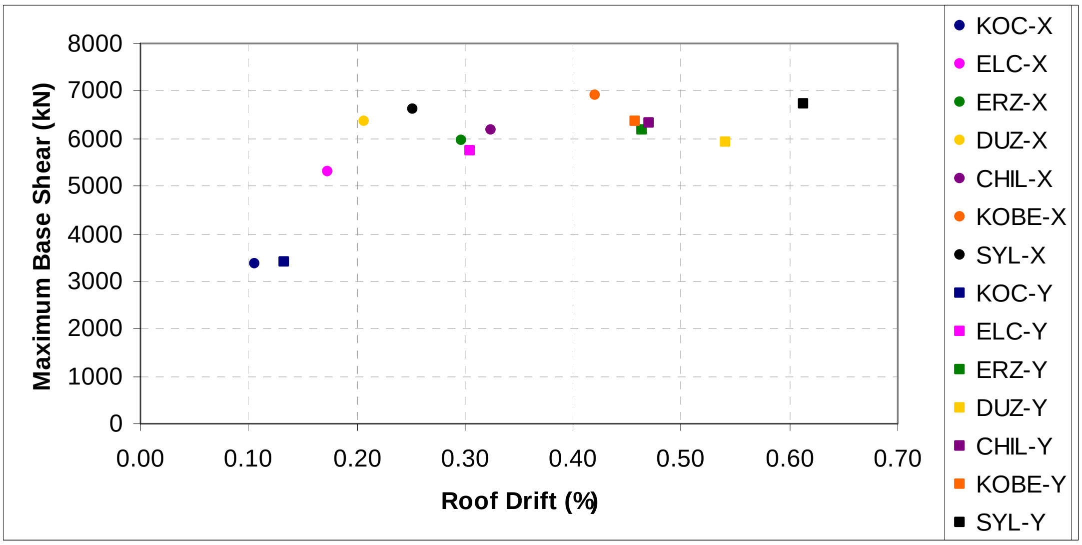


Figure A.8.8. Maximum Base Shear vs. Roof Drift Relationship of 5 Story Building with 1.0 % Shear Wall Ratio (Model 22) under All Selected Earthquake Records

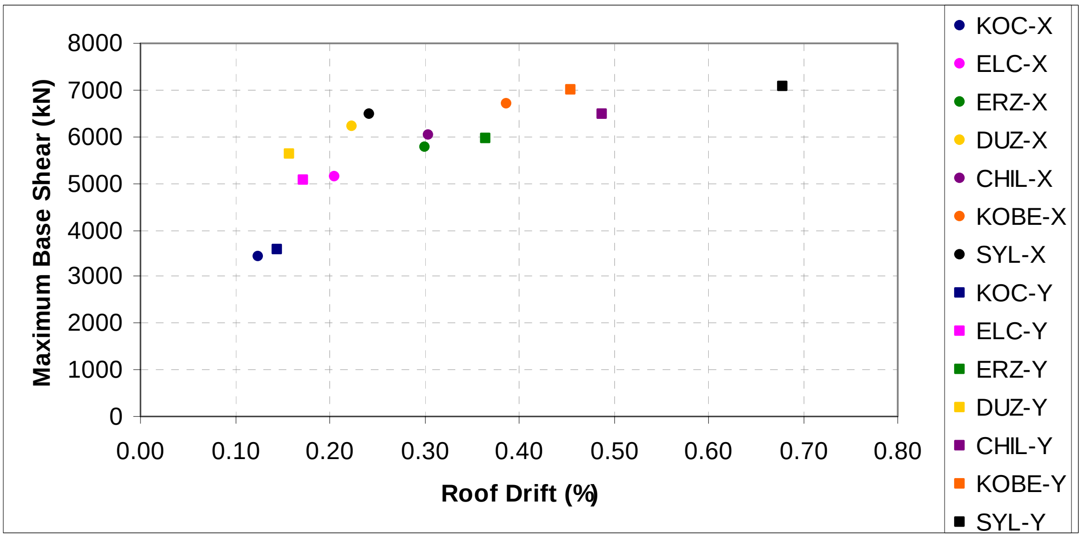


Figure A.8.9. Maximum Base Shear vs. Roof Drift Relationship of 5 Story Building with 1.0 % Shear Wall Ratio (Model 23) under All Selected Earthquake Records

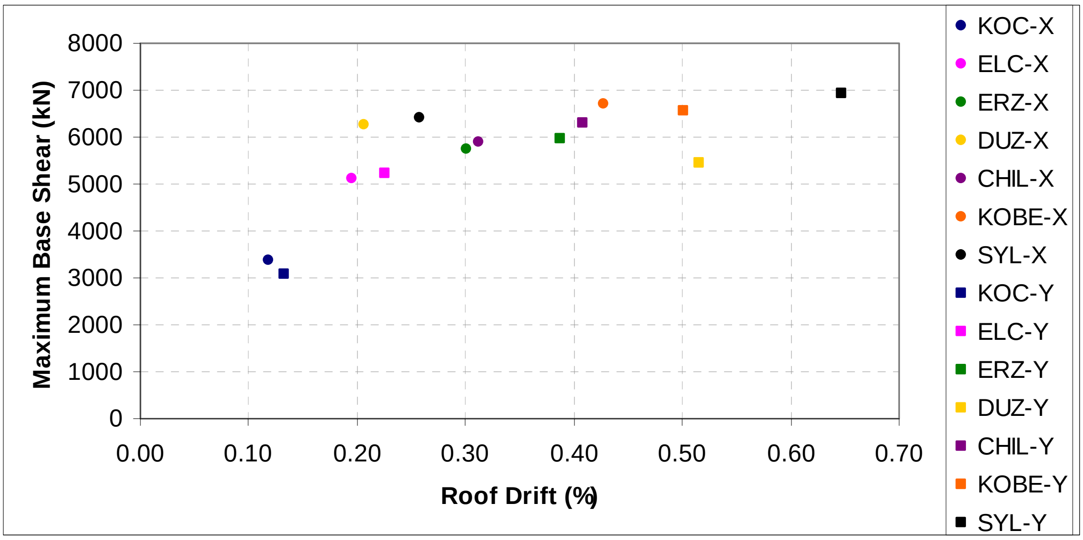


Figure A.8.10. Maximum Base Shear vs. Roof Drift Relationship of 5 Story Building with 1.0 % Shear Wall Ratio (Model 24) under All Selected Earthquake Records

A.9. BASE SHEAR VERSUS ROOF DRIFT RELATIONSHIP FOR MODELS 23&24 GENERATED TO INVESTIGATE THE EFFECT OF SHEAR WALL CONFIGURATION

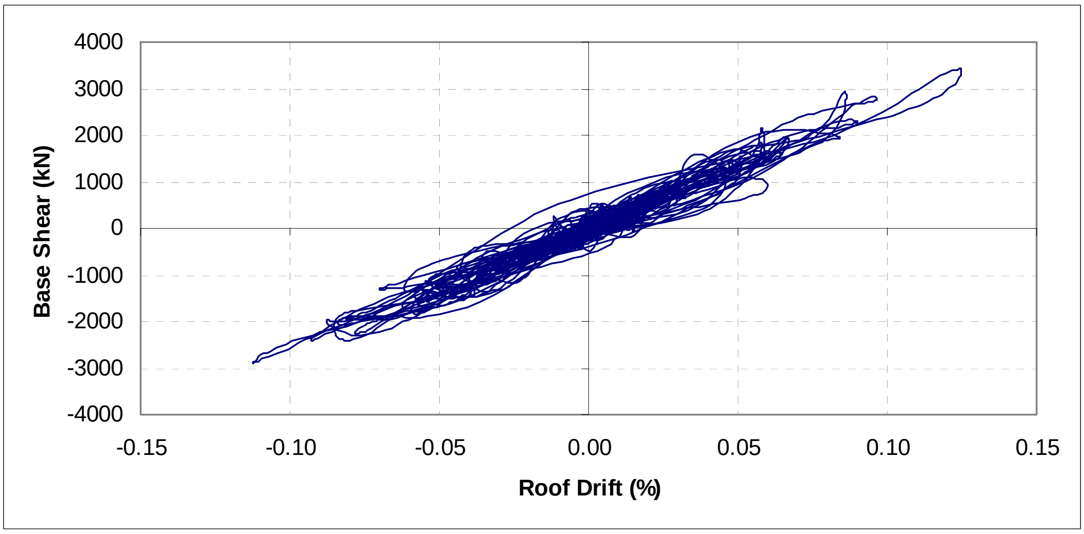


Figure A.9.1. Base Shear versus Roof Drift Relationship of Model 23 in X-direction under Kocaeli Record

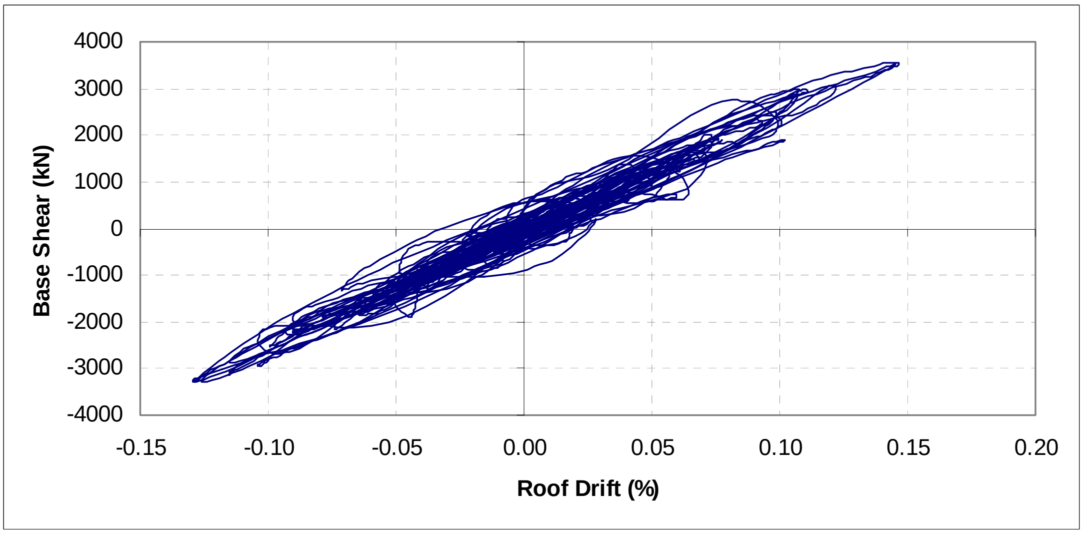


Figure A.9.2. Base Shear versus Roof Drift Relationship of Model 23 in Y-direction under Kocaeli Record

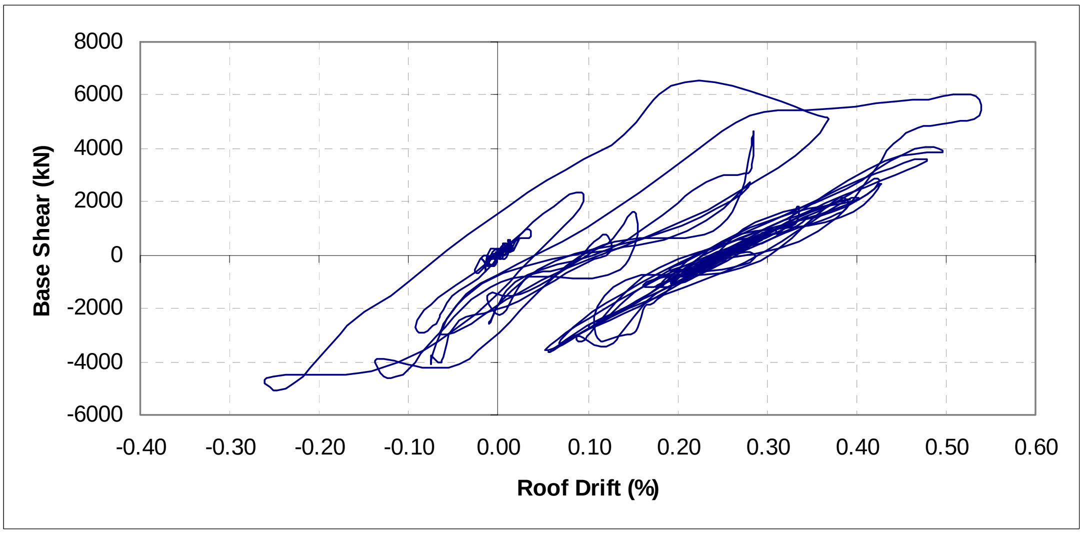


Figure A.9.3. Base Shear versus Roof Drift Relationship of Model 23 in X-direction under Sylmar Record

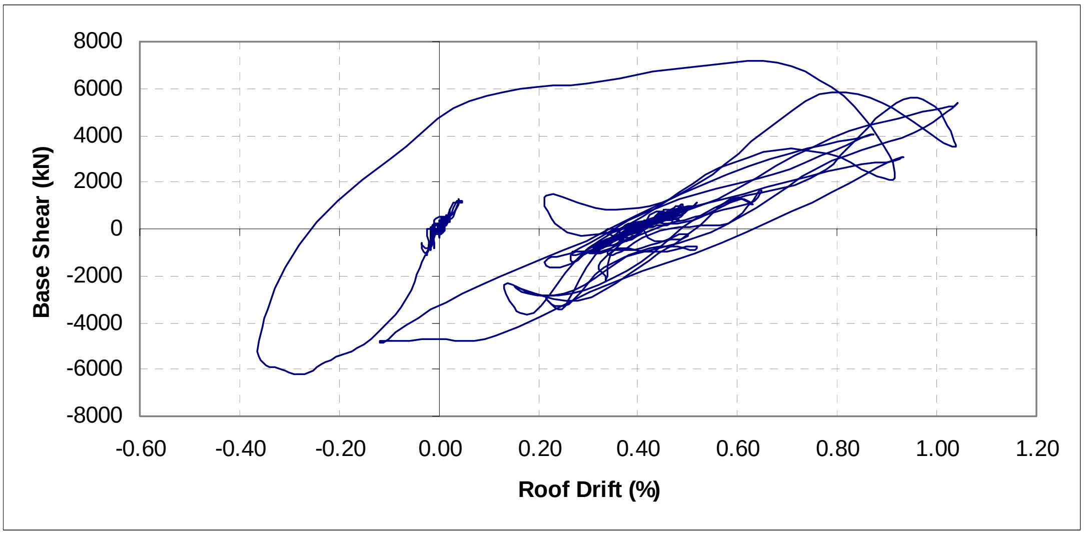


Figure A.9.4. Base Shear versus Roof Drift Relationship of Model 23 in Y-direction under Sylmar Record

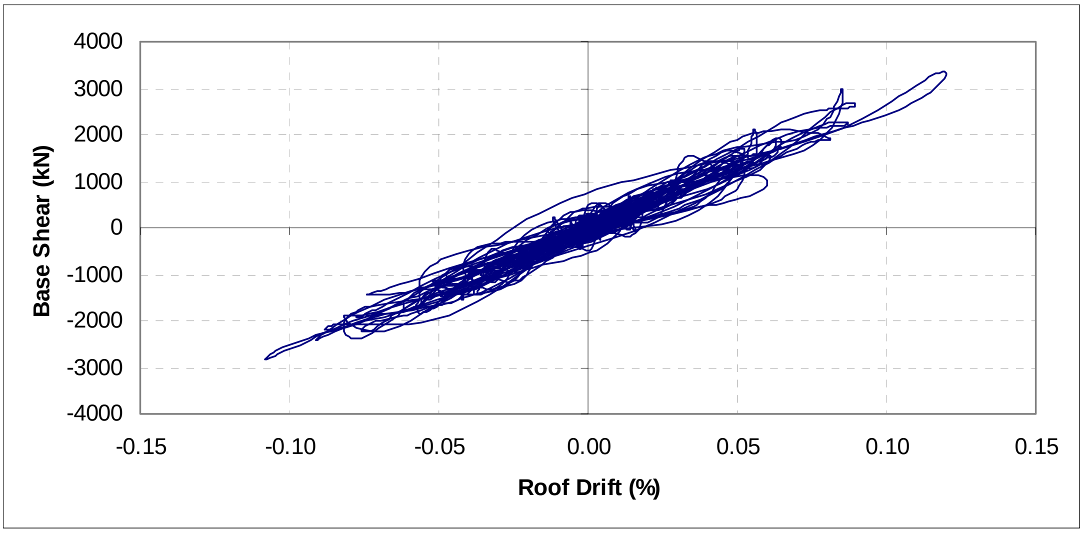


Figure A.9.5. Base Shear versus Roof Drift Relationship of Model 24 in X-direction under Kocaeli Record

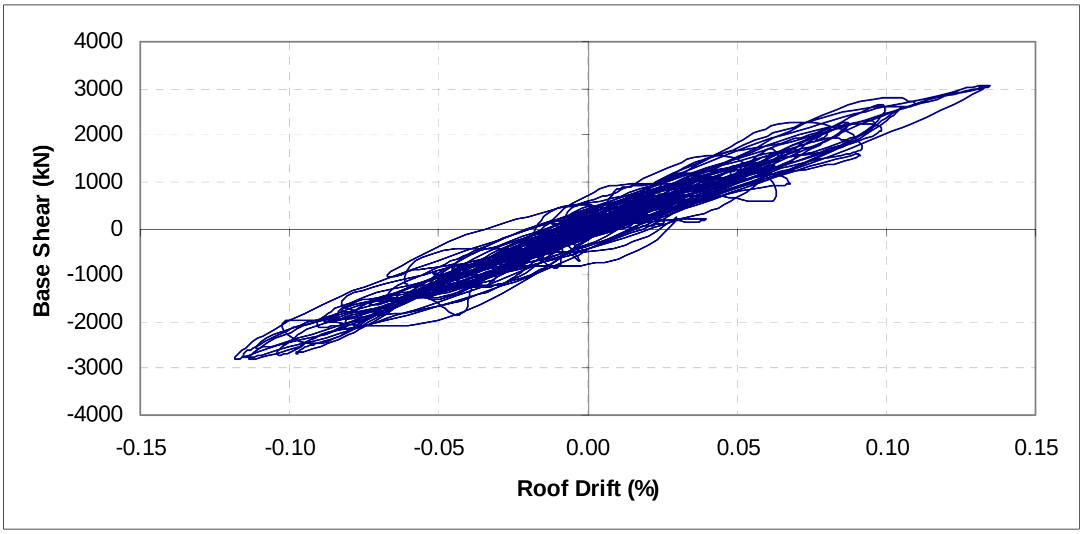


Figure A.9.6. Base Shear versus Roof Drift Relationship of Model 24 in Y-direction under Kocaeli Record

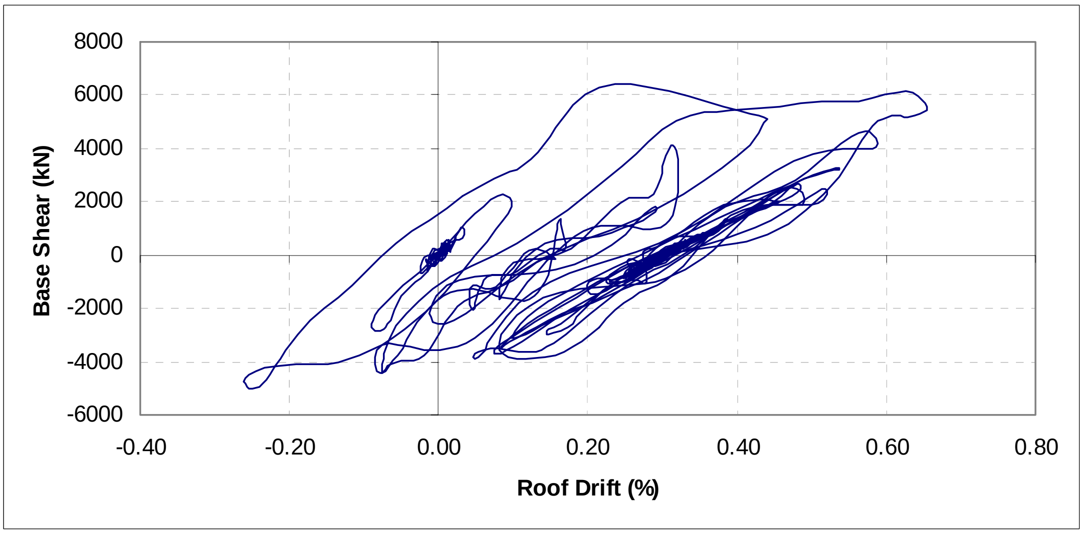


Figure A.9.7. Base Shear versus Roof Drift Relationship of Model 24 in X-direction under Sylmar Record

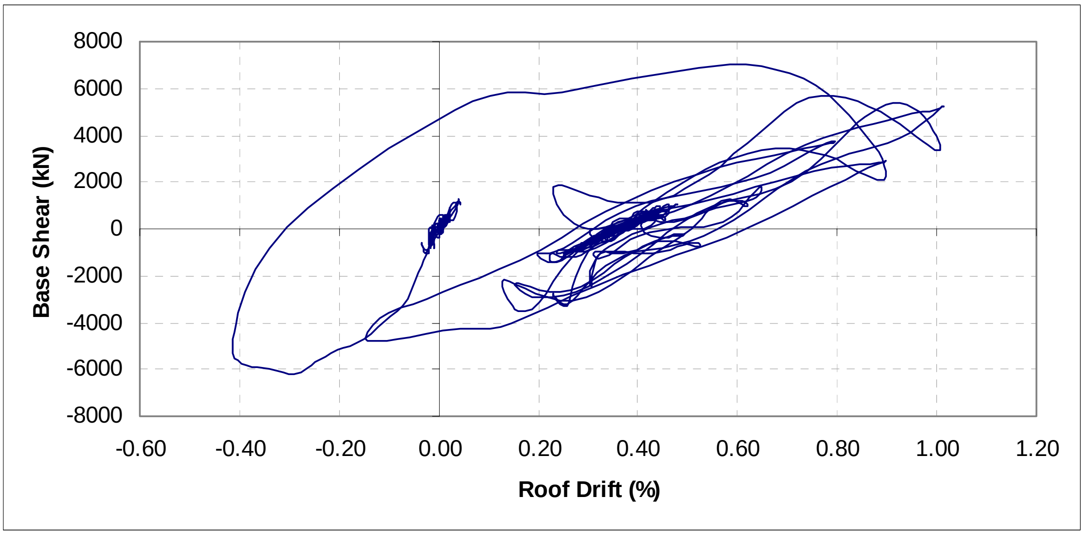


Figure A.9.8. Base Shear versus Roof Drift Relationship of Model 24 in Y-direction under Sylmar Record

A.10. TABLES FOR YIELDED MEMBER PERCENTAGE IN X-DIRECTION

Table A.10.1. The Percentage of Yielded Columns in X-direction for Models 1, 3, 6 and 10 with Shear Wall Ratios of 0.5, 1.0, 1.5 and 2.0 %

	Record	Percentage of Yielded Columns (X-direction)				
		Story-1	Story-2	Story-3	Story-4	Story-5
Model -1 (0.5%)	Chile	44.44	0	0	0	0
	Kobe	100	11.11	0	0	0
	Sylmar	100	11.11	0	0	0
Model -3 (1.0%)	Chile	0	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0
Model -6 (1.5%)	Chile	0	0	0	0	0
	Kobe	90.91	0	0	0	0
	Sylmar	90.91	0	0	0	0
Model -10 (2.0%)	Chile	0	0	0	0	0
	Kobe	90.91	0	0	0	0
	Sylmar	90.91	0	0	0	0

Table A.10.2. The Percentage of Yielded Beams in X-direction for Models 1, 3, 6 and 10 with Shear Wall Ratios of 0.5, 1.0, 1.5 and 2.0 %

	Record	Percentage of Yielded Beams (X-Direction)				
		Story-1	Story-2	Story-3	Story-4	Story-5
Model -1 (0.5%)	Chile	92.86	92.86	92.86	100.00	100.00
	Kobe	92.86	100.00	100.00	100.00	100.00
	Sylmar	92.86	92.86	100.00	100.00	100.00
Model -3 (1.0%)	Chile	91.67	91.67	91.67	100.00	100.00
	Kobe	91.67	91.67	100.00	100.00	100.00
	Sylmar	91.67	91.67	100.00	100.00	100.00
Model -6 (1.5%)	Chile	83.33	83.33	83.33	91.67	91.67
	Kobe	83.33	91.67	100.00	100.00	100.00
	Sylmar	91.67	91.67	91.67	91.67	100.00
Model -10 (2.0%)	Chile	83.33	83.33	83.33	91.67	83.33
	Kobe	91.67	91.67	91.67	100.00	100.00
	Sylmar	91.67	91.67	91.67	91.67	100.00

Table A.10.3. The Percentage of Yielded Shear Walls in X-direction for Models 1, 3, 6 and 10 with Shear Wall Ratios of 0.5, 1.0, 1.5 and 2.0 %

	Record	Percentage of Yielded Shear Walls (X-Direction)				
		Story-1	Story-2	Story-3	Story-4	Story-5
Model -1 (0.5%)	**Chile**	100	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0
Model -3 (1.0%)	**Chile**	100	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0
Model -6 (1.5%)	**Chile**	100	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0
Model -10 (2.0%)	**Chile**	100	0	0	0	0
	Kobe	100	0	0	0	0
	Sylmar	100	0	0	0	0

A.11. TIME HISTORIES OF GENERATED BUILDING MODELS

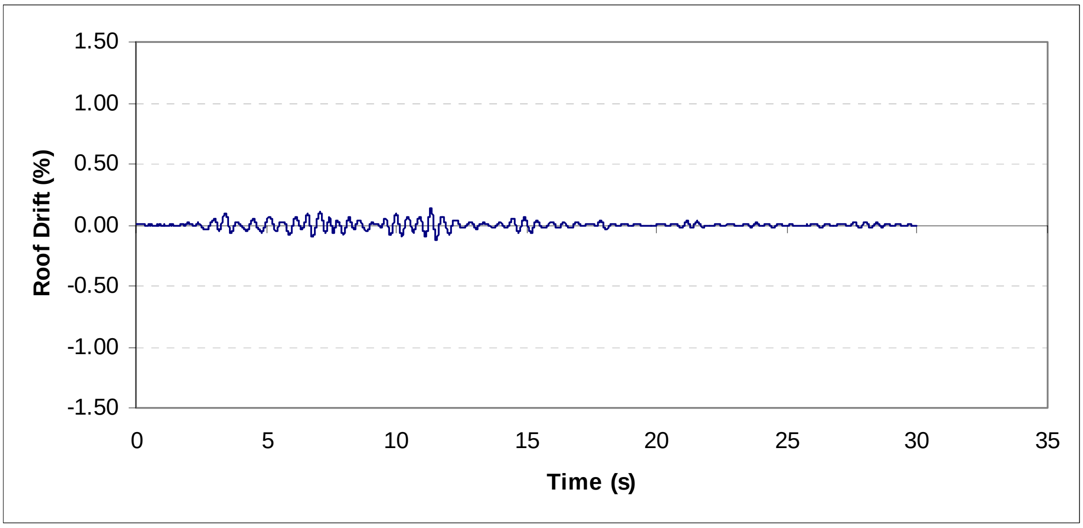


Figure A.11.1. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Kocaeli Record (X-direction)

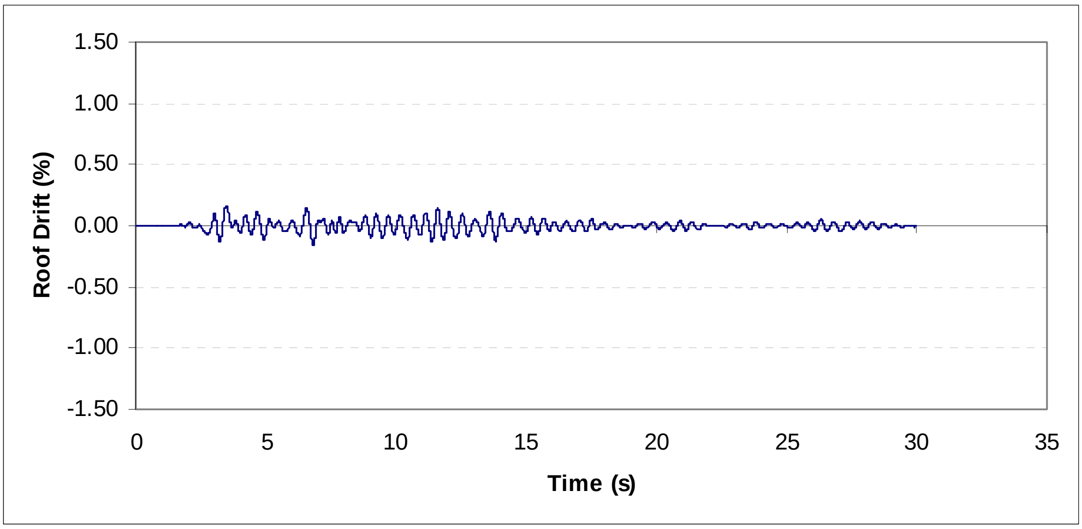


Figure A.11.2. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Kocaeli Record (Y-direction)

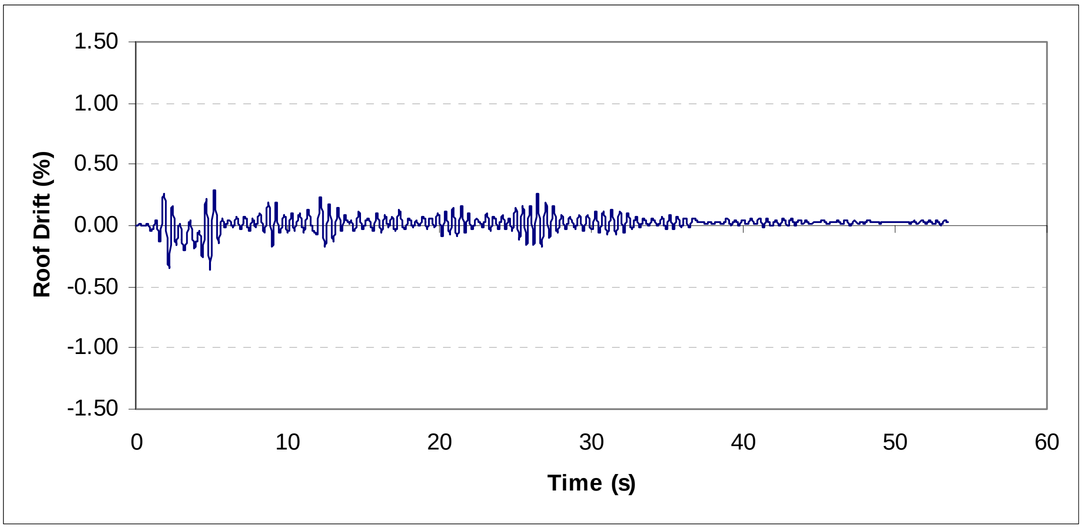


Figure A.11.3. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under El Centro Record (Y-direction)

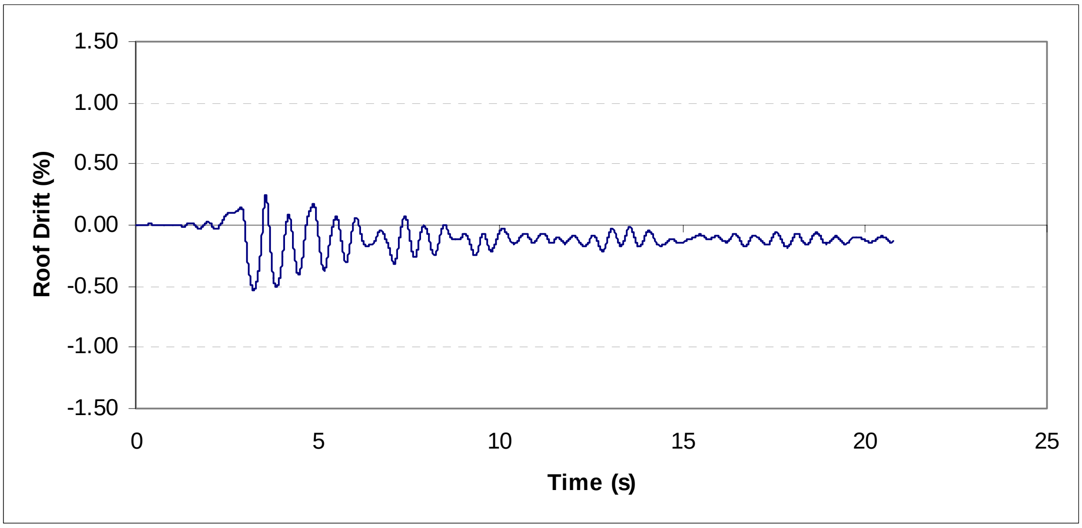


Figure A.11.4. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Erzincan Record (Y-direction)

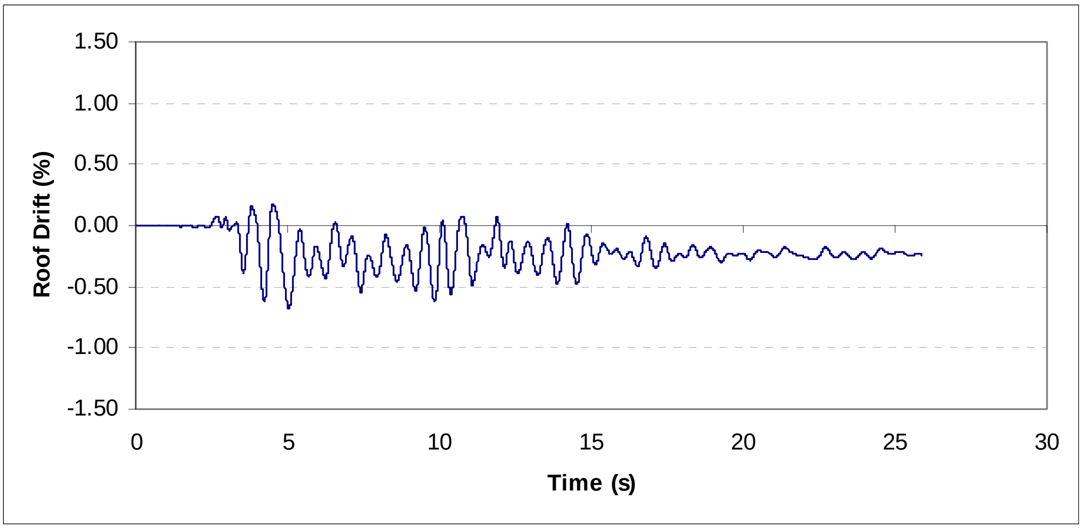


Figure A.11.5. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Duzce Record (Y-direction)

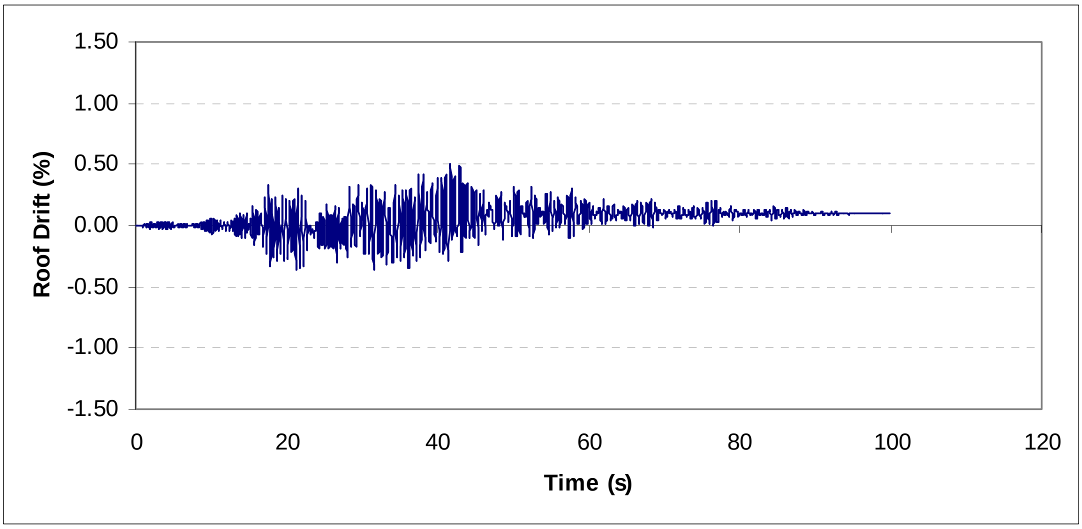


Figure A.11.6. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Chile Record (Y-direction)

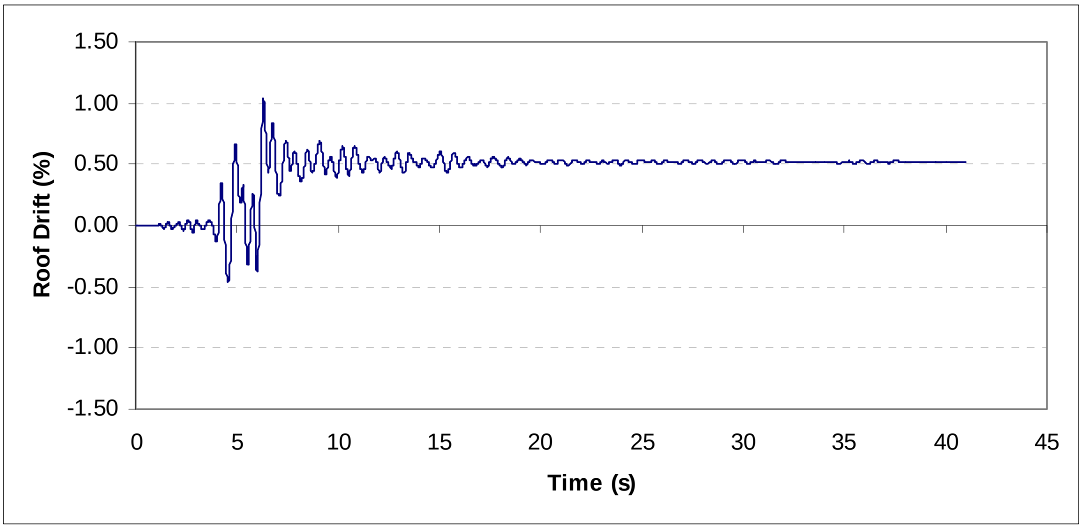


Figure A.11.7. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Kobe Record (Y-direction)

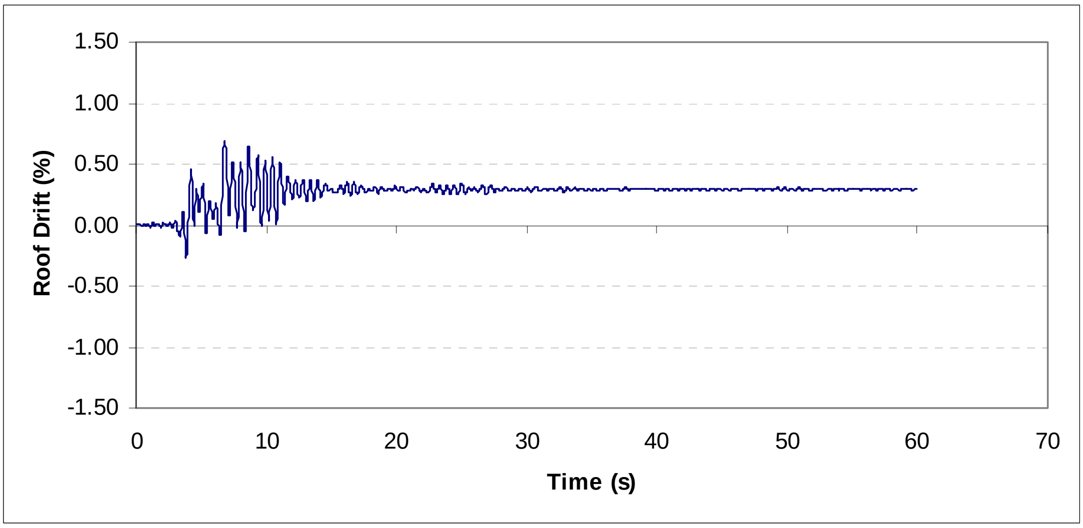


Figure A.11.8. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Sylmar Record (X-direction)

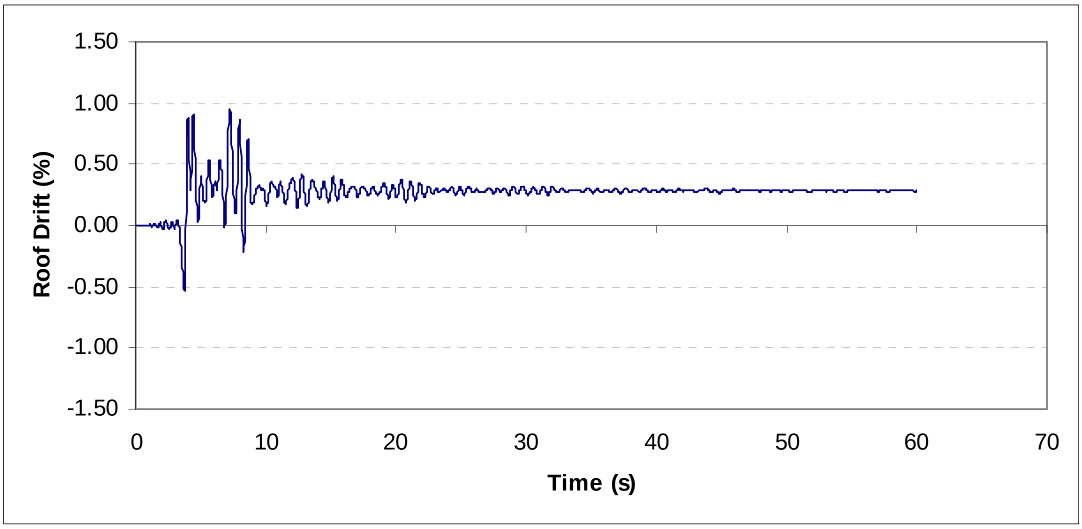


Figure A.11.9. Roof Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Sylmar Record (Y-direction)

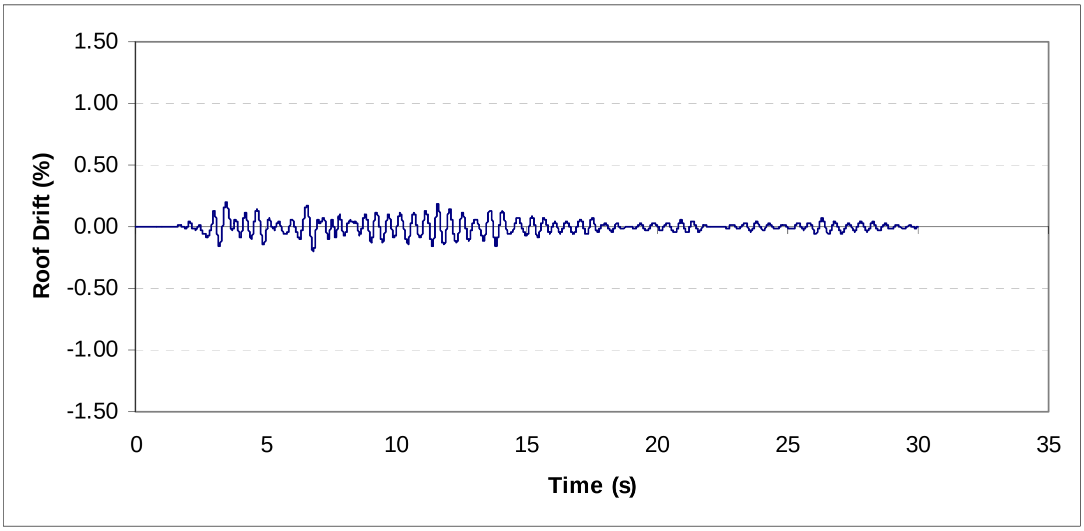


Figure A.11.10. Maximum Average Interstory Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Kocaeli Record (Y-direction)

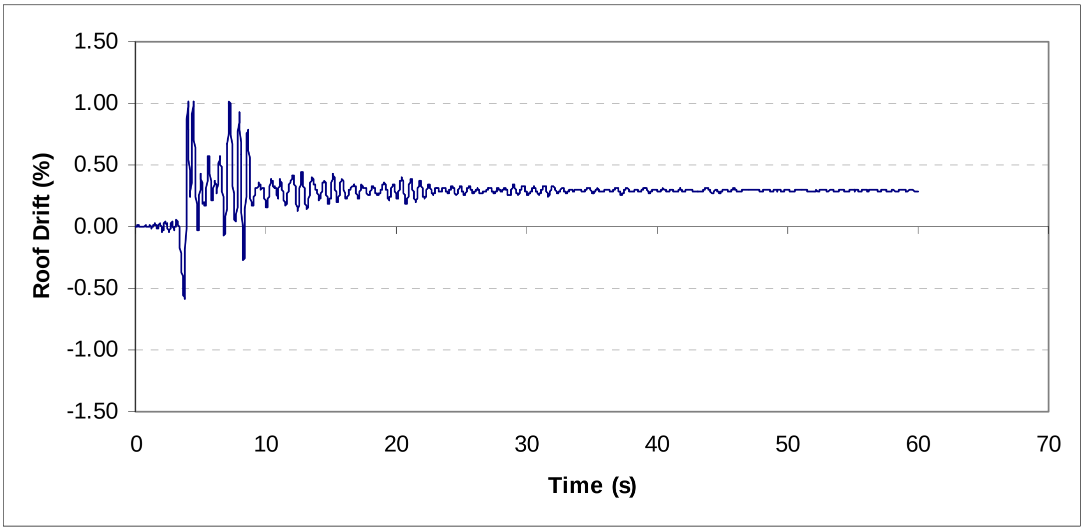


Figure A.11.11. Maximum Average Interstory Drift Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Sylmar Record (Y-direction)

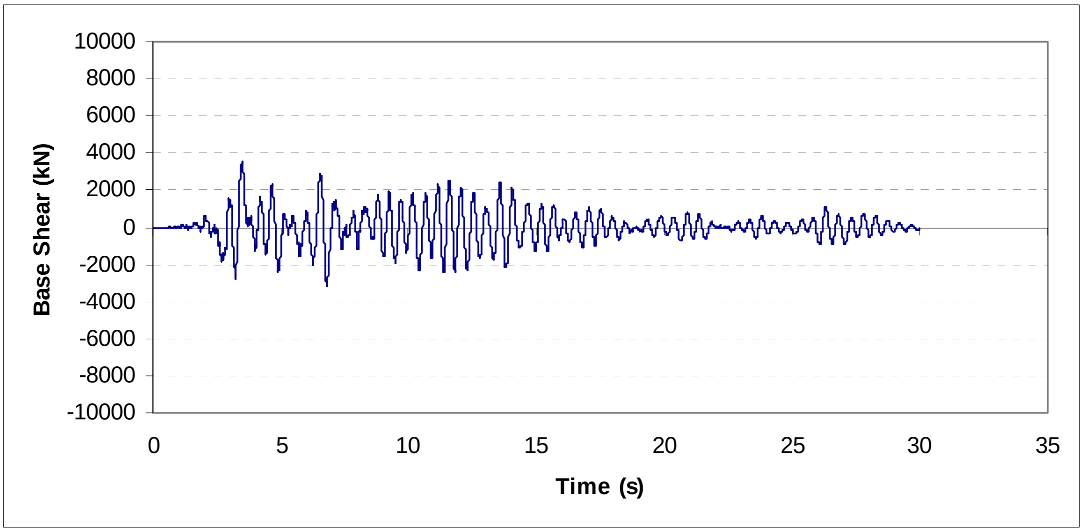


Figure A.11.12. Base Shear Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Kocaeli Record (Y-direction)

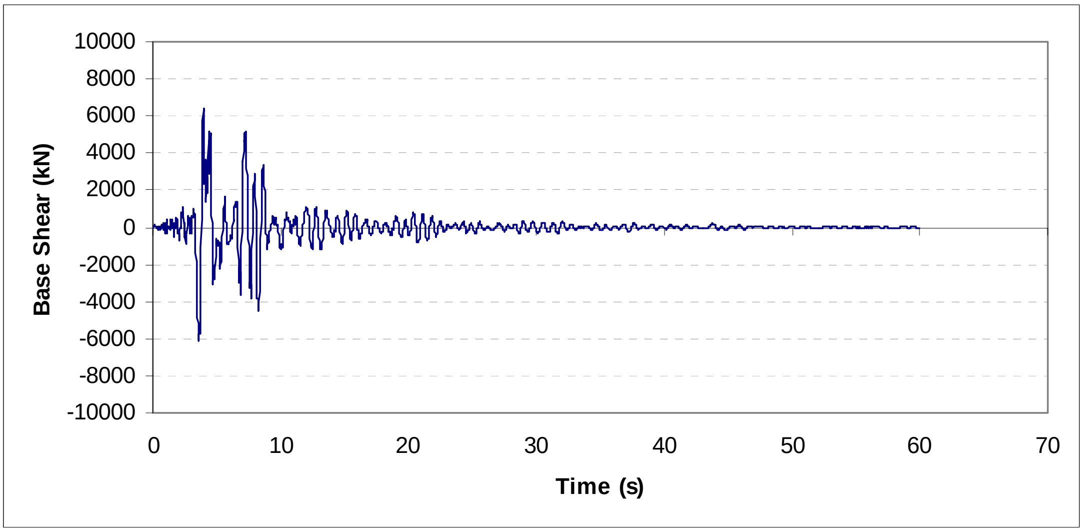


Figure A.11.13. Base Shear Time History of 5 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 3) under Sylmar Record (Y-direction)

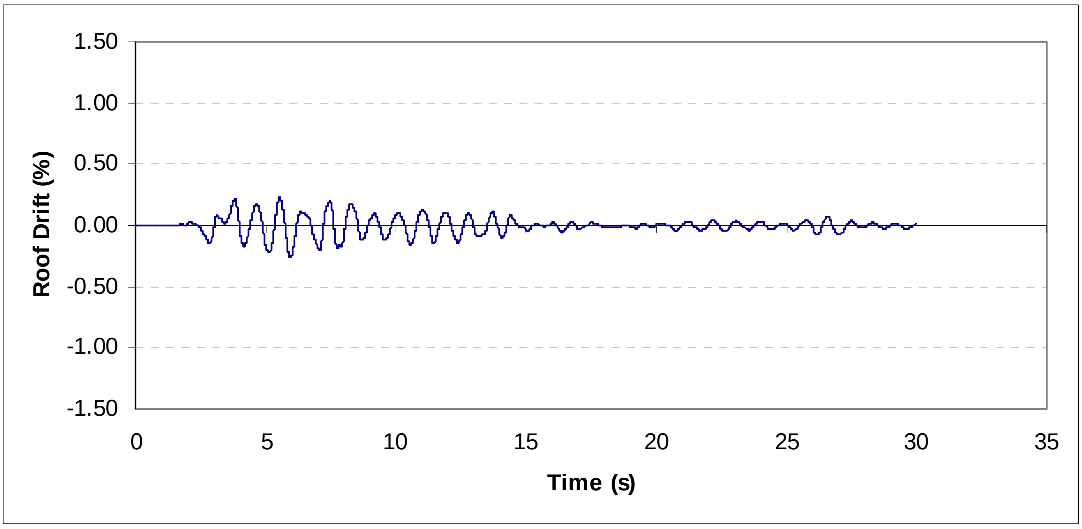


Figure A.11.14. Roof Drift Time History of 8 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 13) under Kocaeli Record (Y-direction)

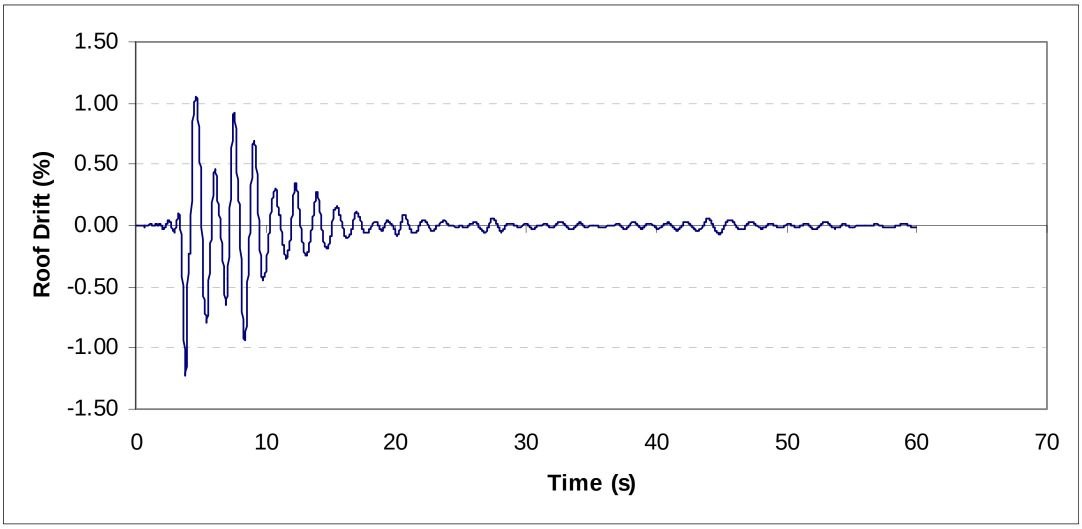


Figure A.11.15. Roof Drift Time History of 8 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 13) under Sylmar Record (Y-direction)

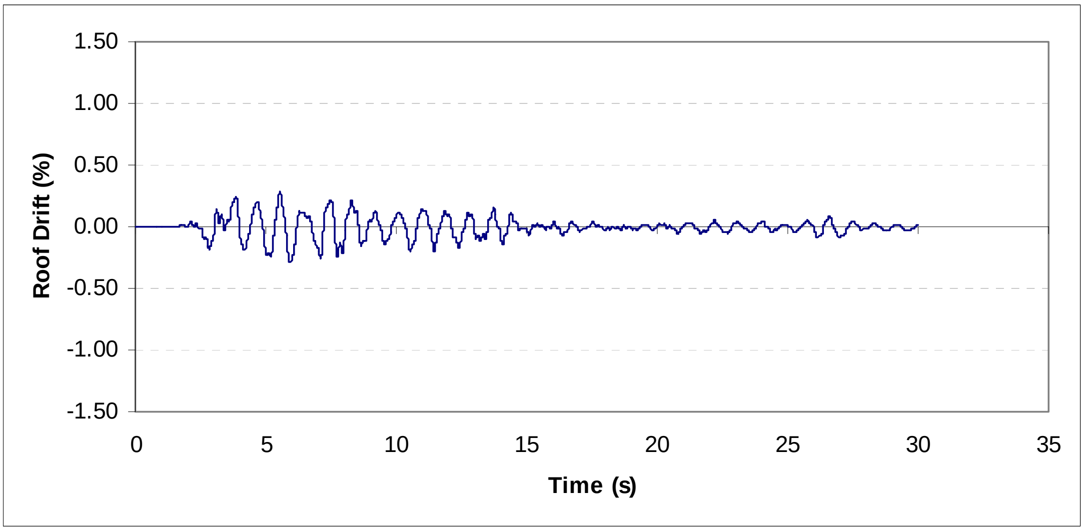


Figure A.11.16. Maximum Average Interstory Drift Time History of 8 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 13) under Kocaeli Record (Y-direction)

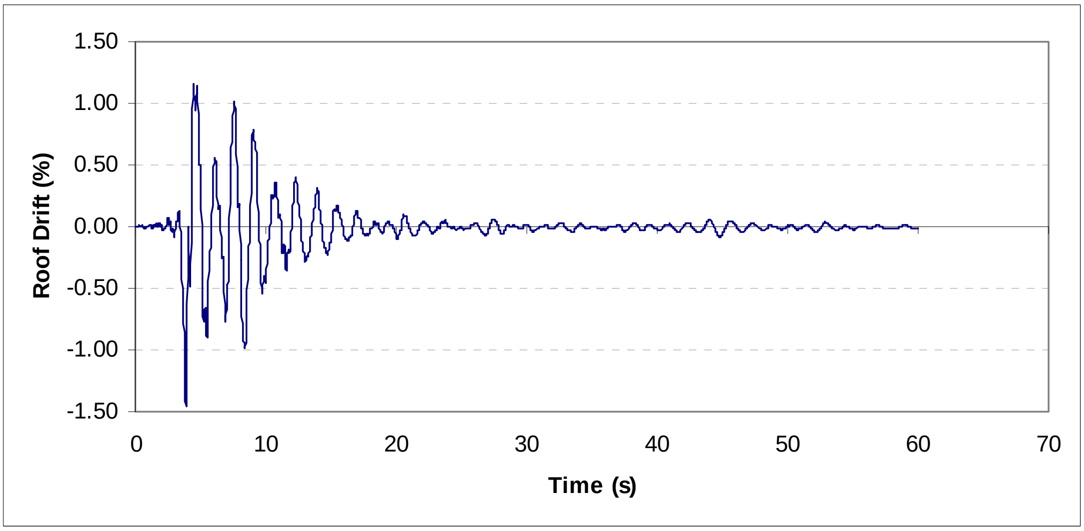


Figure A.11.17. Maximum Average Interstory Drift Time History of 8 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 13) under Sylmar Record (Y-direction)

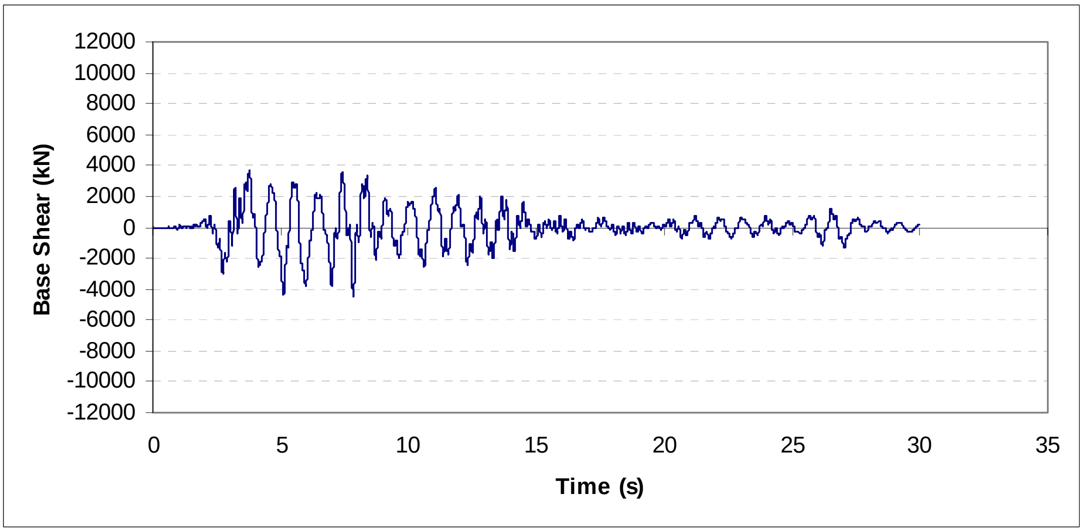


Figure A.11.18. Base Shear Time History of 8 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 13) under Kocaeli Record (Y-direction)

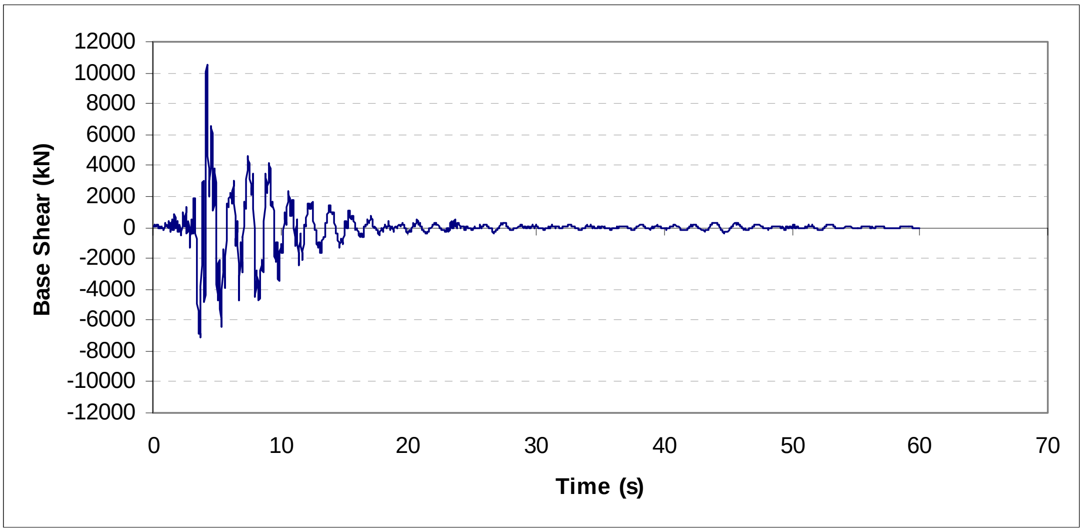


Figure A.11.19. Base Shear Time History of 8 Story Building with Wall Area to Floor Area Ratio of 1.0% in Each Direction (Model 13) under Sylmar Record (Y-direction)