

ACKNOWLEDGEMENTS

I would like to express my deepest gratitude to Assist. Prof. Serdar Selamet for his supervision, guidance and advice, not only throughout this research but also throughout all phases of my graduate study.

Special thanks due to my family and friends who have never deprived me from their support and encouragement during my academic life.

ABSTRACT

THE EFFECT OF SEMI-RIGID STEEL CONNECTIONS TO SEISMIC BEHAVIOR OF HIGH-RISE BUILDINGS

Tall steel structures have always been at risk from fire, explosions and natural disaster, especially earthquakes. It is seen in recent researches, although it is assumed that the seismic performance of steel structures is more efficient than reinforced concrete ones, especially after 1994 Northridge earthquake, considerable deformations and damages have been realized by reason of deficiency of connection details. Therefore, when designing tall buildings, behaviour of steel connections under seismic scenarios have to be investigated more deeply with the purpose of decreasing damages and casualties. In the meantime, it is crucial that attaching importance to nonlinear time history analysis, which is being popular in recent years. Within the scope of this study, the performance of fully rigid and semi-rigid connected tall steel frames are investigated under ground accelerations. Apart from moment connections simple and shear connections are modified with component method to get a better understanding of the behavior in lateral resisting system. 7 real ground motions are used in dynamic analyses after scaling the data according to provided design spectrum which is identified in Turkish earthquake code DBYBHY-2007. As a result of performing analyses, limit controls are checked and performance of the system with modified connections is compared with traditionally connected system. To have an opinion about the efficiency, story drift ratios and base shear of both systems are searched.

ÖZET

YARI RİJİT ÇELİK BAĞLANTILARIN YÜKSEK KATLI YAPILARDA DEPREM DAVRANIŞINA ETKİSİ

Yüksek katlı çelik yapılar; yangın, patlama ve doğal afetlere (özellikle deprem) karşı her zaman büyük bir risk altındadır. Yapılan çeşitli çalışmalar, deprem performansı betonarme yapılara göre daha olumlu olarak değerlendirilen moment dayanımlı çelik yapıların, 1994 Northridge depreminden sonra da görüldüğü gibi, aslında bağlantı detaylarından kaynaklı önemli deformasyonlara ve hasarlara maruz kaldığını göstermektedir. Bu nedenle çok katlı çelik binaların tasarımı esnasında, hasar ve kayıpları en aza indirmek için çelik bağlantıların davranışlarının daha detaylı incelenmesi ve özellikle son yıllarda popülerlik kazanmış zaman tanım alanında yapılan nonlineer deprem analizlerine dizayn aşamasında gerekli önemin verilmesi gerekir. Bu çalışmada, rijit ve yarı rijit birleşimli, süneklik düzeyi yüksek dış merkez çaprazlı ve merkezi çaprazlı çelik çerçevelerle üç boyutlu olarak modellenmiş çok katlı çelik bir yapının zaman tanım alanında yapılan deprem analizleri altındaki performansı değerlendirilmiştir. Önceki çalışmalarda sıklıkla konu edilen moment bağlantılarının aksine, mevcut yapıdaki ikincil kirişlerin mafsalı olarak tanımlanmış bağlantıları yeniden modellenmiş ve daha gerçekçi bir yaklaşımla bağlantı bölgelerinin gerçek davranışlarını temsil etmesi amaçlanarak kısmi rijitlikleri dikkate alınmıştır. İkincil kirişlere monte edilmiş yarı-rijit olarak modellenen yeni bağlantı detaylarının sistemin genel davranışı üzerindeki etkisi incelenmiştir. Dinamik analizlerde 7 adet gerçek yer hareketi ilgili yönetmelikler dikkate alınarak öngörülen tasarım spektrumuna göre ölçeklendirilerek kullanılmıştır. Yapılan analizler sonucunda yeniden matematik modeli hazırlanmış kesme bağlantıları ile mevcut mafsalı birleşim hallerinin performansları karşılaştırılmıştır.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	iii
ABSTRACT	iv
ÖZET	v
LIST OF FIGURES	ix
LIST OF TABLES	xv
LIST OF SYMBOLS	xvi
LIST OF ACRONYMS/ABBREVIATIONS	xviii
1. INTRODUCTION	1
1.1. General	1
1.2. Thesis Layout	5
2. LITERATURE REVIEW	7
2.1. Overview	7
2.1.1. Design and Seismic Analysis Response of Structural Braced Frames	7
2.1.2. Design of Steel Connections	9
2.1.3. Joint Classification Criterias	11
2.1.4. Simple (Shear Resisting) Connections	13
2.1.5. Moment Resisting Connections	14
2.1.6. Semi-Rigid Connections	15
2.1.7. Connection Types with Respect to Rigidity	16
2.1.7.1. Single Web-Angle Connection (Pinned)	17
2.1.7.2. Double We-Angle Connection (Pinned)	17
2.1.7.3. Top and seat angle connections with Double Web-Angle (semi-rigid)	17
2.1.7.4. Top and Seat Angle Connection (Semi-Rigid)	17
2.1.7.5. Extended End Plate Connection (Fixed)	17
2.1.7.6. Flush and Plate Connection (Semi-Rigid)	18
2.1.7.7. Header Plate Connection (Pinned)	18
2.1.8. Single Plate Connections	18
2.1.9. Earthquake Load Estimation	19

2.1.9.1.	Strong Motion Processing	19
2.1.9.2.	Base Line Correction	20
2.1.9.3.	Filters to Reduce Low or Hi-Frequency Noise	22
2.1.9.4.	Strong Motion Records	22
2.1.10.	Earthquake Resistant Design of Steel Structures	23
2.1.10.1.	Seismic Behavior of Steel Structures	24
2.1.10.2.	Seismic Behavior of Bracing Members	25
2.1.10.3.	Seismic Behavior of Beam-Column Members	25
2.1.10.4.	Seismic Behavior of I-sections of Steel	25
2.1.10.5.	Seismic Behavior of Connections	26
2.1.10.6.	Seismic Behavior of Steel Frames	26
3.	CARRIER SYSTEM IDENTIFICATION	28
3.1.	Definition of Steel Carrier System	28
3.2.	Materials and Type of Structural Members	32
3.2.1.	Materials	32
3.2.2.	Structural Members and Sections	34
3.2.3.	Structural Loads	36
3.2.4.	Connections	36
3.3.	Verification Studies	36
3.4.	Building Model in OpenSEES	38
3.4.1.	Nodal Coordinates, Masses and Boundary Conditions	38
3.4.2.	Materials	39
3.4.3.	Element Cross Sections	39
3.4.4.	Elements and Element Connectivity	39
3.4.5.	Gravity and Lateral Loads	39
3.4.6.	Recorders for Outputs	40
3.4.7.	Analysis	40
4.	STRONG GROUND MOTION RECORDS	42
4.1.	Strong Motion Processing	42
4.2.	Selection Conditions of Ground Motions According to DBYBHY (2007)	49
5.	METHODS OF ANALYSIS	50
5.1.	Modal Analysis of Structures	51

5.2. Dynamic Analysis of Multistory Buildings	53
6. CONNECTIONS	59
6.1. Mathematical Model of Connections in Opensees	59
6.2. Material Behaviors of Connections	61
6.3. Modeling Connections in OpenSEES	62
7. RESULTS	64
7.1. Conclusion	64
7.2. Performance Assessment of Semi-Rigid and Rigid Connected Structural System by Comparison With Pinned Connection System	65
7.3. Performance Assessment of the System Semi-Rigid and Rigid Connected System by Comparison With Pinned Connection System with Increased Earthquake Excitation	79
7.4. Results	85
APPENDIX A: OPENSEES MODEL TCL FILES	87
REFERENCES	108

LIST OF FIGURES

Figure 1.1.	Classification of Connections based on Rotation Capacity.	4
Figure 2.1.	Moment Frames.	8
Figure 2.2.	Concentric Braced Frames.	8
Figure 2.3.	Eccentric Braced Frames.	8
Figure 2.4.	Classification of Connections based on resistance and rotation capacity (Eurocode 3).	12
Figure 2.5.	End Plate Beam to Column, Beam to Beam Connections.	12
Figure 2.6.	Fin Plate Beam to Column, Beam to Beam Connections (BS EN 1993).	14
Figure 2.7.	Moment Resisting Connection Types.	14
Figure 2.8.	Standard Fin Plate Connection Details.	15
Figure 2.9.	Experimental Moment-Rotation Relations of Connections.	16
Figure 2.10.	Connection Type Under Consideration in the Study.	19
Figure 2.11.	Acceleration, Velocity and Displacement for an Analog Record.	21
Figure 2.12.	Sequential Baseline Adjustment Applied to the Velocity Time-History (Boore, Bommer, 2005).	21

Figure 2.13.	Displacement Time Histories for a Series of Filters with Different Parameters (Boore, Bommer, 2005).	22
Figure 2.14.	Strong Column - Weak Beam Concept.	24
Figure 3.1.	3-D Model and Typical Plan of the Building.	28
Figure 3.2.	Configuration of Model Structure.	29
Figure 3.3.	Plan of the Building (Lengths are in Meter).	30
Figure 3.4.	Different Types of Material Behavior.	33
Figure 3.5.	Steel01 Material (Mazzoni, McKenna, Scott, Fenves, 2006).	34
Figure 3.6.	Fiber Steel <i>W</i> Section.	35
Figure 3.7.	2-D Basic Frame Model.	37
Figure 4.1.	For Lateral Leap and Reverse-Normal Faults R_{jb} Ve R_{rup} Values, (PEER vd., 2010).	44
Figure 4.2.	Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Cape Earthquake.	45
Figure 4.3.	Effective Durations And Maximum Acceleration Values of Scaled Ground Motion Records of Düzce Earthquake.	46
Figure 4.4.	Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of El-Centro Earthquake.	46

Figure 4.5.	Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Kobe Earthquake.	46
Figure 4.6.	Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Landers Earthquake.	47
Figure 4.7.	Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of San Fernando Earthquake.	47
Figure 4.8.	Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Tabas Earthquake.	47
Figure 4.9.	Elastic Spectral Acceleration Spectrum.	48
Figure 5.1.	Theory of Single Degree of Freedom System.	54
Figure 5.2.	Concept of Newton-Raphson Method.	56
Figure 5.3.	Response Spectra for Different Damping Ratios (Arnold, 2006). . .	57
Figure 5.4.	Effects of Damping.	58
Figure 6.1.	Configuration of Modified Connections Between Secondary Beams and Girders.	59
Figure 6.2.	Spring Model Representing the Connection Details.	60
Figure 6.3.	Single Plate Shear Connection (a) Against Tension and (b) Against Moment Deformations of the Three-Dimensional Finite Element Model.	61

Figure 6.4.	Moment-Rotation Graph for the 3D Single Plate Shear Connection Model.	62
Figure 6.5.	Force-Deformation Graph for the 3D Single Plate Shear Connection Model.	62
Figure 6.6.	The Orientation of Zerolength Elements and EqualDOF in X direction in OpenSees.	63
Figure 7.1.	Comparison of Base Shear of Pinned and Semi Rigid and Rigid Connected Systems in X direction.	66
Figure 7.2.	Comparison of Base Shear of Pinned and Semi Rigid and Rigid Connected Systems in Y direction.	66
Figure 7.3.	Comparison of Top Floor Dipsplacements of Pinned and Semi Rigid and Rigid Connected Systems in X direction.	67
Figure 7.4.	Comparison of Top Floor Dipsplacements of Pinned and Semi Rigid and Rigid Connected Systems in Y direction.	67
Figure 7.5.	Comparison of Drift Ratios of Pinned, SemiRigid and Rigid Connected Systems in X direction.	69
Figure 7.6.	Comparison of Drift Ratios of Pinned, Semi Rigid and Rigid Connected Systems in Y direction.	70
Figure 7.7.	Drift Points and Earthquake Directions in the Analysis.	71
Figure 7.8.	Concept of Modified Connection.	72
Figure 7.9.	Axial Force of Beam End in the Connection Region (y -Direction).	73

Figure 7.10. Axial Force of Secondary Beam End in the Connection Region (y -Direction). 73

Figure 7.11. Rotation of Beam End in the Connection Region (y -Direction). . . 74

Figure 7.12. Rotation of Secondary Beam End in the Connection Region (y -Direction). 74

Figure 7.13. Moment of Beam End in the Connection Region (y -Direction). . . 75

Figure 7.14. Moment of Secondary Beam End in the Connection Region (y -Direction). 75

Figure 7.15. Difference of Axial Force in Zero-Length Element Ends (Rigid Connections). 76

Figure 7.16. Difference of Moments in Zero-Length Element Ends (Rigid Connections). 77

Figure 7.17. Difference of Rotations in Zero-Length Element Ends (Rigid Connections). 77

Figure 7.18. Difference of Axial Force in Zero-Length Element Ends (Semi-Rigid Connection). 78

Figure 7.19. Difference of Moments in Zero-Length Element Ends (Semi-Rigid Connection). 78

Figure 7.20. Difference of Rotation in Zero-Length Element Ends (Semi-Rigid Connection). 79

Figure 7.21. Comparison of Base Shear of Pinned, Semi-Rigid and Rigid Connected Systems in X Direction. 80

Figure 7.22. Comparison of Base Shear of Pinned, Semi-Rigid and Rigid Connected Systems in Y Direction. 80

Figure 7.23. Comparison of Top Floor Displacement of Pinned and Rigid Connected Systems in X direction. 81

Figure 7.24. Comparison of Top Floor Displacement of Pinned and Rigid Connected Systems in Y direction. 81

Figure 7.25. Comparison of Drift Ratios of Pinned, Semi Rigid and Rigid Connected Systems in X Direction. 82

Figure 7.26. Comparison of Drift Ratios of Pinned, Semi Rigid and Rigid Connected Systems in Y Direction. 82

Figure 7.27. Axial Force of Beam in the Connection Region (y -Direction). 83

Figure 7.28. Axial Force of Secondary Beam in the Connection Region (y -Direction). 83

Figure 7.29. Rotation of Beam in the Connection Region (y -Direction). 84

Figure 7.30. Rotation of Secondary Beam in the Connection Region (y -Direction). 84

Figure 7.31. Moment of Beam in the Connection Region (y -Direction). 85

Figure 7.32. Moment of Secondary Beam in the Connection Region (y -Direction). 85

LIST OF TABLES

Table 3.1.	Earthquake Parameters.	29
Table 3.2.	The Members Selected for the 28-Story Building.	31
Table 3.3.	Analysis Results of First Floor Column.	37
Table 4.1.	Selected and Scaled Earthquake Records for the Analysis.	44
Table 4.2.	Physical Properties of Selected and Scaled Earthquake Records for the Analysis.	44
Table 4.3.	Acceleration, Velocity and Displacement Values for Scaled Records.	48
Table 5.1.	First Six Periods (T) of Models Prepared With OpenSees and Etabs.	53
Table 7.1.	The first 3 Periods (T) of Pinned, Semi-Rigid and Rigid Connection Structural System.	65

LIST OF SYMBOLS

A_0	Effective ground acceleration coefficient
a_{max}, a_{min}	Maximum and minimum acceleration
b	strain hardening
b_p	Width of plate
c	damping coefficient
d	Site class
dU	displacement increment
dt	time increment
E, E_0	Modulus of Elasticity
e_1	End distance
e_2	Edge distance
$F_{r1}, F_{r2}, F_{r3}, F_c$	Forces
F_i	Mode shapes
F_y	Yield stress
f	Scaling factor
g	Gravitational acceleration
I	Building importance factor
I	Moment of inertia
k	stiffness
L	Length of elements
M	Moment
M_b	Bending moment resistance of connected beam
M_{pl}	Moment of partial strength connection
M_{Rd}	Moment of full strength connection
m	mass
$p(t)$	External force
R	Behavior factor
R_{jb}	Distance to the fault plane of Joyner-Boore
R_{rup}	Closest distance to the fault plane

St-44, St-52, St-64	Steel material types
Sae(T)	Elastic response spectra
T_A, T_B	Spectrum characteristic periods
Ti	Time periods
t_p	Thickness of plate
t_{eff}	Effective earthquake time
Vs30	Shear wave speed at 30 meters depth
w(Ti)	Weight function
W_D	Natural circular frequency
Z_1, Z_2, Z_3, Z_4	Soil profile types, according to Turkish Earthquake Standarts
δ	Displacement
ε	Strain
ξ	Fraction of critical damping coefficient
ρ_1	Vertical bolt spacing for end plates
ρ_3	Transverse distance between bolt centerlines (gauge)
σ	Stress
ψ	Rotation
Φ_j	jth mode participation factor
φ	Rotation

LIST OF ACRONYMS/ABBREVIATIONS

AISC	American Institute of Steel constructions
ASCE	American Society of Civil Engineers
BCSA	British Constructional Steelwork Association
BS	British Standards
DBYBHY	Turkish Regulations on Building in Seismic Zones
EUROCODE	European Committee for Standardization
EW	East-West
FEMA	Federal Emergency Management Agency
GUI	Graphical User Interface
LS	Life Safety
MDOF	Multiple Degree of Freedom
MRF	Moment Resisting Frames
MSE	Mean Square Error
NS	North-South
PC	Preventing Collapse
PEER	Pacific Earthquake Engineering Research Center
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
RCM	Reverse Cuthill Mckee
SCI	Steel Construction Institute
SDOF	Single Degree of Freedom
SS	Strike Slip
UK	United Kingdom
USGS	United States Geological Survey

1. INTRODUCTION

1.1. General

Tall steel structures have always been at risk from fire, explosions and natural disasters, especially earthquakes. Case studies such as Muto and Krishnan investigated the performances of high-rise steel structures under a 7.8 magnitude earthquake scenario. In this study, notable cracks, damages and torsional irregularities on the system, contains welded steel frames which are durable against horizontal forces and fixed connections, was observed. Also, many structures showed high story drifts beyond the limits and occurrence of failure has been observed in older design buildings. Similarly, in case study of Krishnan, (2006), existing 18-story steel building was again modeled and the seismic performance under California San Andreas earthquake excitation was investigated. Considered structure was designed with moment frames. Relative storey drifts are determined beyond the limits and significant damage assessments have been recognized due to many connection details. It is because to integrate structural analysis of earthquake scenarios with design of tall buildings is necessary to minimize damage and casualties. In consequence of similar studies remarkable deformations were observed that caused by deficiency of connection details, especially after the (Northridge, 1994) and (Kobe, 1995) earthquake, and it is noticed that the performance of structures could be increased by paying attention to the connection details more than normal. After this awareness, braces and steel frames with semi-rigid connections, beyond traditional modeling, were researched for preventing buckling as studied in Dutta and (Hamburger, 2010), (Awkar and Lui, 1999), (Asgarian and Moradi, 2011), (Degertekin and Hayalioğlu, 2004), (Aksoylar *et al.*, 2013). Especially processed in detail in (Awkar and Lui, 1999) and (Aksoylar *et al.*, 2013), it was shown that how general performance of the system was affected from modeling fixed moment connection to semi-rigid connection and quite satisfying results were obtained in term of resisting horizontal displacement. In contrast, the impact of pinned connections on structures was not overemphasized.

The difference of this research from studies done in the past is the examination of

performance level of simple (pinned) and shear connections, despite moment connections, in the structure in terms of resisting the lateral forces and displacements. Simple connections out of moment joints are handled as pinned connection traditionally and they are not considered in resisting of lateral forces. However, a difference will occur in the horizontal force performance of the system as a result of turning these connections into semi-rigid formation. The primary objective of this study is to examine the behavior of shear and simple (pinned) connections under earthquake motions. Also, the effect of semi-rigid connections to behavior of high-rise steel structures will be investigated. (Lui and Astaneh-Asl, 2000) made a research similar to this approach and foresaw that shear connections work with floor systems. In experiments carried out on different bolted and welded designs, it is observed that simple connections exhibit more effective behavior than the traditional method.

Connections being one of the interests of this research have a crucial importance into the whole system. It is seen in recent researches, although it is assumed that the seismic performance of steel structures is more efficient than reinforced concrete ones, especially after (Northridge, 1994) earthquake, considerable deformations and damages have been realized by reason of deficiency of connection details. Design codes (Eurocode) contain some parts of explanation for designing connections under variety of assumptions including simple (pinned), rigid or semi-rigid moment rotation behavior. Simple joints should be capable of transmitting the internal forces and accepting resultant rotations under the design loads but in this type joints moment resistance is ignored whilst rigid joints need to have sufficient rotational stiffness based on full continuity. On the other hand, semi-rigid joints should provide a foreseeable degree of interaction between members based on the design moment rotation behavior of the joints as shown in Figure 1.1. They could transmit internal forces and moment simultaneously. Flexible (semi-rigid) connected frames often experience larger interstory drifts but smaller base shears when subjected to dynamic loadings, although this behavior has been demonstrated to be frequency dependent (Awker and Lui, 1997).

Here is the application rules given in Eurocode 3 Part 1-8 “Joints” satisfy this design assumptions:

Joints shall be designed on the basis of a realistic assumption of the distribution of internal forces and moments. The following assumptions shall be used to determine the distribution of forces:

- the internal forces and moments assumed in the analysis are in equilibrium with the forces and moments applied to the joints,
- each element in the joint is capable of resisting the internal forces and moments,
- the deformations implied by this distribution do not exceed the deformation capacity of the fasteners or welds and the connected parts,
- the assumed distribution of internal forces shall be realistic with regard to relative stiffness within the joint,
- the deformations assumed in any design model based on elastic-plastic analysis are based on rigid body rotations and/or in-plane deformations which are physically possible, and
- any model used is in compliance with the evaluation of test results EN 1990 (Eurocode 3 Part 1-8)

On the other hand, one of the considerable scope of this thesis is the calculation of the nonlinear response of structural steel beam to column and beam to beam joints. For every load level, since the actual moment and relative deformation between connected members are considered, the moment M versus rotation φ curve of structural joints stands for an important characteristic of their response. Because of designing joints mostly as rigid or pinned in the past, evaluating entire estimation of the joint nonlinear behavior was limited. Being in need of procuring realistic behavior of connections, well establish methodologies have been started to use for the evaluation of joint strength and initial stiffness in order to describe joint characteristic such as rotational capacity or the $M - \varphi$ curve. The analytical studies (Sivakumaran, 1988), (Lui, Lopes, 1997) have demonstrated that connection flexibility tends to reduce frame stiffness and thus increase the vibration period of the frame. If these characteristics could be reliably estimated, then a series of advances in the design of structures could be realized.

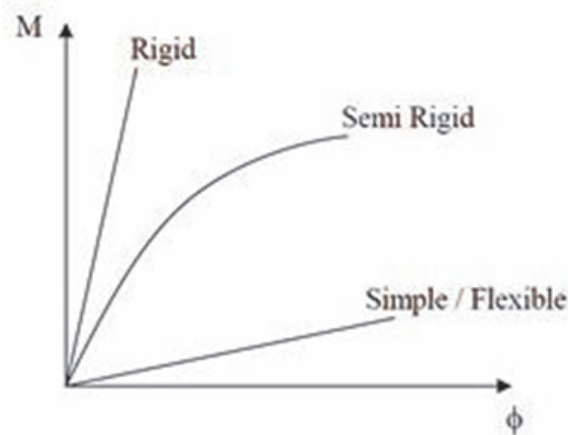


Figure 1.1. Classification of Connections based on Rotation Capacity.

For the estimation of the joint response, apart from experimental testing, three modeling options are available. These are empirical or analytical models, finite element models and mechanical models. Analytical and empirical models usually feature simple analytical expressions for calculation of the response and thus they offer the advantage of ease of use, at the cost however of reduced reliability, which is not easily scalable to cases of joint designs other than those used for their calibration (Lemonis, Gantes, 2009)). Conversely, advanced finite models can be regarded considerably reliable for the simulation of the nonlinear joint response. At the same time because of the need of high computational proficiency and complicated preparation process it could be seen as a deficiency of this method. Mechanical models, which are regulated as an assembly of springs and rigid elements and are based on the characterization of individual components, include all the features of the two previous modeling type. Because of the utilities, both finite element method and mechanical modeling were selected to use in this thesis.

This thesis is principally concerned with the impact of steel connections to entire system behavior; however there is also a need to model the forementioned 28-floor steel building and understand the lateral load resistance of the structure. Therefore this thesis will first illustrate modeling details of the structural steel building before focusing on the specific use of modeling of connections. After these all, the effect of modified connection detailing to the structure behavior under lateral loads will be investigated.

The results of a numerical study on the responses of multistory flexibly and rigidly connected frames subjected to earthquake excitations will be presented. In addition to explain connection flexibility, the analysis considers geometrical and material nonlinearities. For geometrical nonlinearities in the form of P- Δ effects are investigated and for material nonlinearities in the form of column inelasticity is considered. A modal analysis and earthquake excitation procedure will be used for the study and response of the frames will be assessed by investigating inter-story drifts, strength of the elements and modal responses. The aim of this thesis is also to investigate the effect selected earthquakes have on the behavior of entire system, steel connections and model them in such a way that they can be in structural analysis. For these purposes, in the frame of this research, three dimensional structure model of the existing multi-storey steel building will prepared and analyzed by OpenSees, widely used open-source finite element program. The connection details will be examined using with mathematical spring model methods by means of utility of the program OpenSees. Non-linear time history analysis have been made and second order effects were also considered in this analysis. Real earthquake motion records used in the analysis and are scaled according to the seismic design spectrum which is determined according to seismic parameters. At the stage of evaluating, relative story drifts, base shears, rotations in specific connections were investigated and interpreted within the framework of limits. Analysis was repeated with the changes made in the connection regions and compared with normal results. Within the scope of the thesis, the Turkish Regulation on Buildings in Seismic Zones (DBYBHY, 2007), the American Institute of Steel Construction (AISC, 2005 AISC 341s1-05), European Committee for Standardization (Eurocode, 2004), the Federal Emergency Management Agency (FEMA, 2000 FEMA 356), American Society of Civil Engineers (ASCE, 2006-ASCE 7-05) regulations were used in necessary parts.

1.2. Thesis Layout

This thesis consists of seven chapters. Chapter 2 contains a review of design of structural braced frames and connection types which are considered, such as single plate connections, in this thesis. This also includes a review of earthquake load estimation and earthquake resistant design of steel structures including international codes and

standards as well as previous research into joint classification and prediction of seismic behavior of stability members and connections details. On the other hand, widely-used finite element software program OpenSees is introduced with relevant parts with this study.

Chapter 3 describes the carrier system identification of considered existing high-rise steel braced frame structure. This also gives the material types, properties and types of structural members of the building. Structural loads, connection details which are used in the system, building the model with Opensees are presented including verification studies in this chapter.

Chapter 4 reveals the information about strong ground motion records which are used in the scope of this thesis. This includes strong motion processing, determination of earthquake excitations, rules of selection of the record data and performance criteria for earthquakes.

In Chapter 5, analysis methods are investigated. This contains modal and non-linear dynamic analysis with detailed review. Analysis procedures and parameters are examined deeply in this chapter.

The first section of Chapter 6 incorporates the mathematical model of connections into material behaviors described in chapter 3. This involves theoretical spring model of connections with explanation of the force-deformations and moment-rotation behaviors of each relevant degrees of freedom. The second section looks at modeling the considered single plate connection with finite element method by using ABAQUS (Simulia, 2009).

Chapter 7 includes discussion and general conclusions of the research project as well as recommendations for future work.

2. LITERATURE REVIEW

2.1. Overview

2.1.1. Design and Seismic Analysis Response of Structural Braced Frames

In practice, steel structures are strengthened by making very stable with different connection types and fasteners. Different stability connections are used depending on building geometry or other construction features. The shapes, cross sections, joint properties of the stability members directly affect the seismic performance. Because, these stability members are affecting plastic deformations which occur in structures as a result of plastic hinges. This naturally affects ductility of the structure, the natural period of the system and ultimately the structural behavior factor. In some studies, such as (Gioncu, Mazzolani, 2002), it is stated that the seismic performance of the structure does not increase by the stability of used elements, even it is affected negatively in some circumstances. Notably, the performance of structures is suddenly decreased by buckling on elements which are used for stability in the nonlinear behavior of the structures subjected to strong ground motions. In this case, the structure ductility is significantly reduced and consequently the energy absorption capacity of the structure is also getting lower.

Steel braced frames are divided into two groups as concentric and eccentric braced frames Figure 2.3 and Figure 2.1 . Both of two types of braced frames are in this thesis scope. In concentric braced frames, ductility is taken into account lower than the ductility of moment resisting frames. The characteristic of eccentric braced frames is the ability to combine the strength of steel centric braced frame rigidity and high ductility of steel moment resisting frames Figure 2.2. Actually, the basic parameter of evaluating the behavior of a braced system is concentric and eccentric braced members no fewer than connections. Nevertheless, in the belief that connections can even affect the behavior of braced system, it is investigated that the impact of connection details to general behavior of systems in seismic regions.

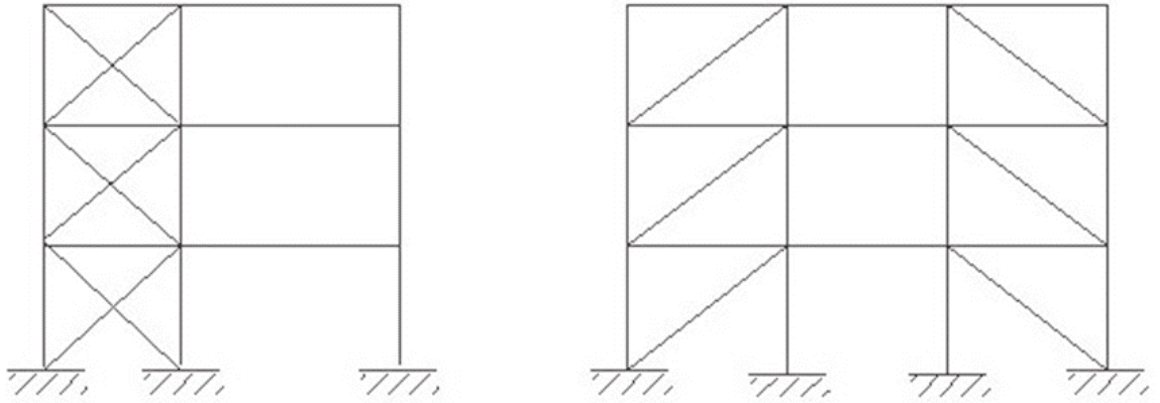


Figure 2.1. Moment Frames.

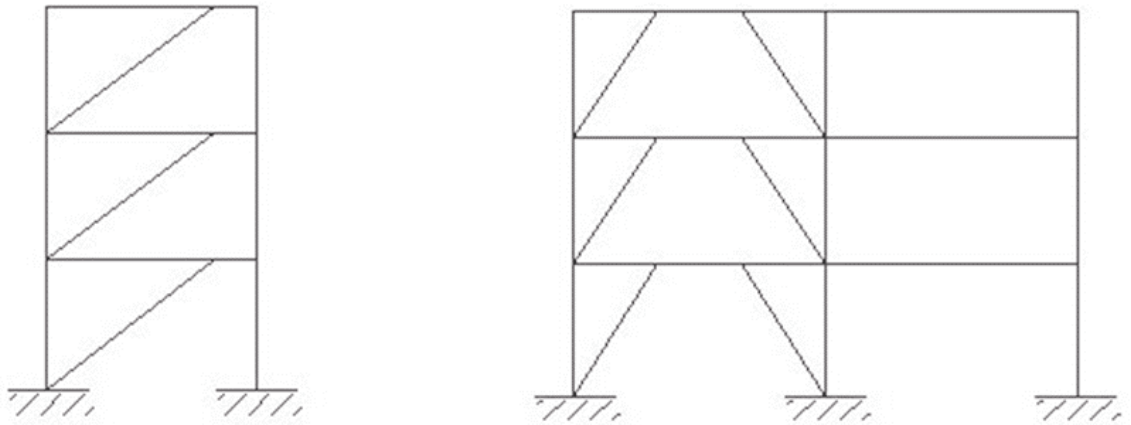


Figure 2.2. Concentric Braced Frames.

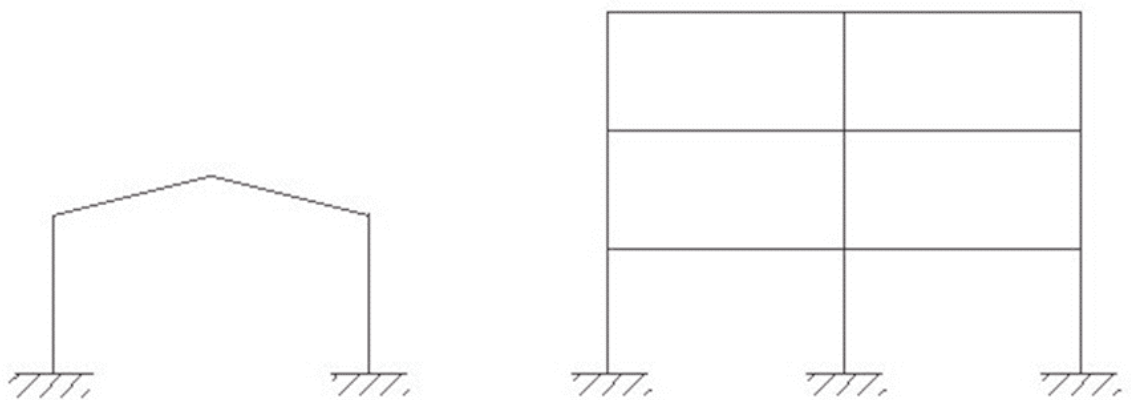


Figure 2.3. Eccentric Braced Frames.

Some of the earliest works studied on multistory steel structures and seismic

performances to achieve ductile behavior of structural braced frames with strength design. (Asgarian and Moradi, 2010); investigated seismic performance of steel frames with some special brace type. To do so, buildings with various stories and different bracing configurations were considered. Steel braced frames subjected to three ground motion records and nonlinear time history analysis have been performed using finite element software program (Opensees). The computational model of the structures was developed using the modeling capability of the software framework Opensees. Needed assumed assumptions were released in the paper. Yang and Leon and (DesRoches, 2007); designed three braced models which were 3-9-20 story buildings. Pushover analysis of the model were performed using Opensees to estimate the over strength, inelastic strength and deformation capacities for the entire structures.

Pursuant to experimental studies, Krawinkler, 1995); indicated that after Northridge earthquake, the exposed problems about welded connections of moment resisting frame and the standard, providing unreliable performance, has been used in most steel structures show us there is a need to improve system behavior by better detailing around connections. This can be achieved in several ways. If welded connections are used, the quality of welding must be improved and the stress level around the welds must be kept at a safe level. This requires consideration of the expected rather than nominal strength of the elements (including cyclic hardening), and the development of configuration in which overstressing of welds in severe earthquake is avoided (Krawinkler 1995).

2.1.2. Design of Steel Connections

Because of the ability to absorb the energy of dynamic actions owing to large inelastic deformation capacity, steel frames are very efficient in withstanding to forces generated from ground motions since frames are designed aptly. In consideration of this reason steel is generally preferred for structures in seismically active regions. Within the scope of researches relevant to dynamic behavior of multistory steel frames under-ground excitations, it is seen that traditionally used full-strength high stiffness rigid connections may not be the best option in seismic applications. This is also proved

in earlier earthquakes (Northridge etc.) witnessing a weak performance of steel frames related to geometric nonlinearities and brittle connection failure at or near the tension flange region. It is no longer clear that if the flexibility effects of the connections are taken into consideration, dynamic behavior of frames will be different from that of rigid frames.

Connection is a structure which is only as strong as its weakest link. If they are not designed in a proper manner, the connections joining the members may be weaker than the members being joined. To obtain an economical design, it is usually crucial that the connections enhance the strength of the members. Despite the failure of members is ductile, connection failure is usually brittle. That is because to avoid connection failure before member failure is desirable. Within a frame, joint behavior affects the distribution of internal forces and moments and the overall deformation of the structure. In many cases, however, the effect of modeling a continuous joint as fully rigid, or a simple joint as perfectly pinned, compared to modeling the real behavior, is sufficiently small to be neglected. Joints are defined as follows: Simple, continuous, semi-continuous (Figure 2.5).

- Simple: In this kind of joint, bending moments are not transmitted. It is sometimes referred to pinned connections. The UK National Annex BS EN 1993-1-8 indicates that connections designed in accordance with the principles given in SCI P358 may be classified as nominally pinned.
- Continuous: This kind of joints are stiff enough for the effect of its flexibility on the frame bending moment diagram to be neglected. It is sometimes referred to as rigid connections. The UK National Annex refers the designer to SCI P398 in which, for many cases, but importantly not all cases, connections designed for strength alone can be considered as rigid.
- Semi-continuous: This kind of joints are too flexible to qualify as continuous, but is not a pin. The behavior of this type of joint must be taken into account in the frame analysis. These are describe as “ductile connections“ in UK practice and are used in plastically designed semi-continuous frames. The UK NA indicates that these may be designed using the principles given in SCI P18.

2.1.3. Joint Classification Criterias

According to BS EN 1993-1-8, nominally pinned connections should be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members or the structure as a whole. Also, they should be capable of accepting the resulting rotations under the design loads. In addition the joint must provide the directional restraint to members which has been assumed in the member design (Eurocode 3). Regarding the connection properties three classification criteria can be adapted, namely:

- Based on stiffness
- Based on strength
- Based on both stiffness and strength

Stiffness is appropriate for elastic global analysis while strength is appropriate for rigid plastic analysis. With a combination of two, stiffness and strength is appropriate for elastic-plastic global analysis.

BS EN 1993-1-8 requires that joints be classified according to their stiffness for analysis and should also have sufficient strength to transmit the forces and moments acting the joint that results from the analysis. As far as strength is concerned, connections are classified as either full strength, or partial strength depending on whether they are capable of transferring the full plastic moment of the framing beams. As far as stiffness is concerned, connections are classified as full restraint, partial restraint or simple connections depending on the service load stiffness. (James A. Swanson, 2000)

In the case of elastic design, the classification system requires only the rotational stiffness criterion leading to three categories: nominally pinned, rigid and semi-rigid connections. On the other hand, flexural resistance is taken into consideration in rigid plastic analysis. In such kind of case, joints can be classified as full strength and partial strength. Full strength connections have a design resistance at least equal to that of the connected member. In such a case, yielding can occur at both the beam end and in

the fastening elements. Moreover, the resistance of partial strength connections is less than that of the connected members. A plastic hinge will be form in the connection. For this reason sufficient rotation capacity is required Figure 2.4.

On the other hand, the Eurocode 3 defines joints as three classes with respect to its moment resistance: Full strength, partial strength and pinned.

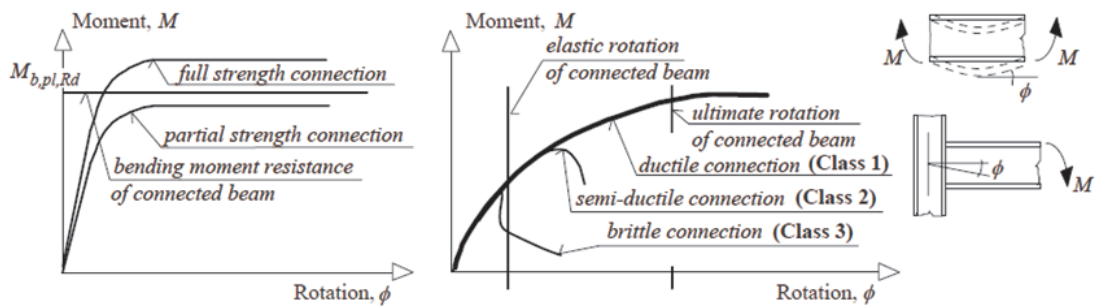


Figure 2.4. Classification of Connections based on resistance and rotation capacity (Eurocode 3).

As illustrated in Figure 2.5, in the M-Rotation graph, fixed connections fall into the region with a high slope, named as initial stiffness, whereas pinned connections exhibit a very low initial stiffness, possessing an ultimate moment close to null. The encased region in between is defined as the semi-rigid behavior.

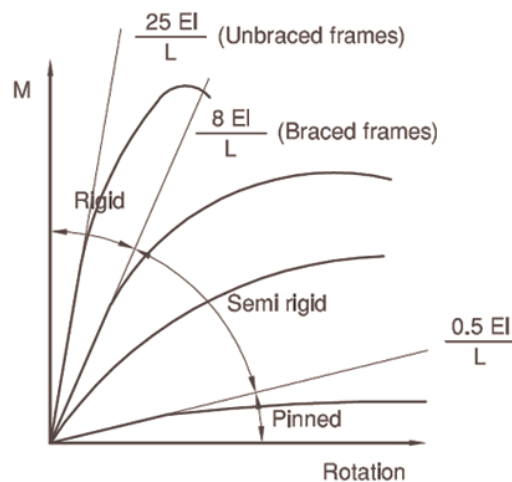


Figure 2.5. End Plate Beam to Column, Beam to Beam Connections.

In practice the joints are neither ideally hinged nor ideally rigid. In fact all the joints exhibit some relative rotation between members being joined. This is due to the deformation of elements in the joint. The moment versus relative joint rotation of different types of connections is shown in Figure 2.5. Any joint developing more than 90% of the ideal rigid joint moment is classified as rigid and similarly any joint exhibiting less than 10% of the ideal rigid joint moment is classified as pinned joint; and the joint developing moments and rotations in between are referred as semi-rigid. Based on test results and theoretical studies, moment rotation relationship for different standard connections exhibiting semi-rigid behavior has been presented in literature.

Structural steel frames consist of universal steel beams and columns connected together at various locations in the structure. These joints can be list as:

- Beam to Column
- Beam to Beam
- Column to Column
- Column to Foundation

In this thesis, beam to column and beam to beam connection types will be concerned.

2.1.4. Simple (Shear Resisting) Connections

Simple connections are defined as those connections that transmit end shear only and have negligible resistance to rotation and therefore do not transfer significant moments at the ultimate Limit State (BCSA 1996). Simple connections include partial depth end plates, full depth end plates Figure 2.6 and fin plates Figure 2.7.

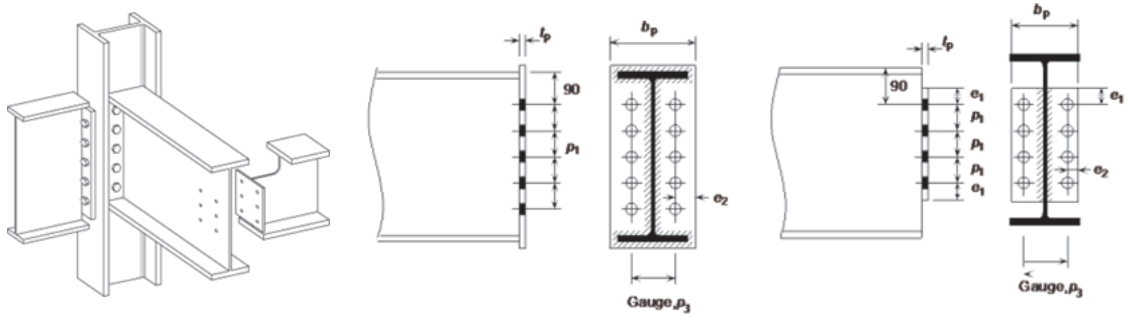


Figure 2.6. Fin Plate Beam to Column, Beam to Beam Connections (BS EN 1993).

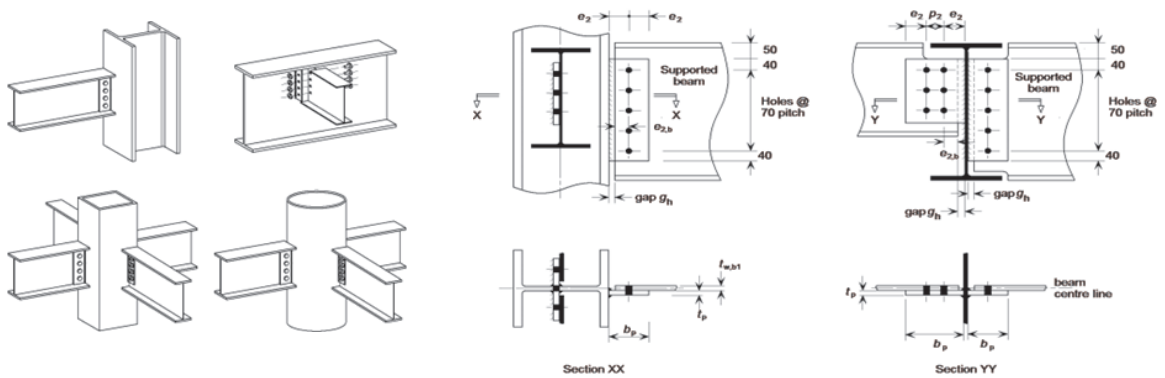


Figure 2.7. Moment Resisting Connection Types.

2.1.5. Moment Resisting Connections

Moment resisting connections between beams and columns in multistoried buildings are very common. They can be classified as; full depth end plate connections, extended end plate connections, haunched connection, bolted end plate and welded beam to column connections Figure 2.8. These kind of connections have a moment capacity greater than the supported member, that is because they are assumed to behave as a rigid support. Depending upon the type of joining method and elements used to make the joint, the flexibility of the joint may vary from hinged to rigid joint condition. Moment connections are used in multi-storey un-braced buildings and single storey portal frame buildings.

Moment connections are designed to transfer bending moments, shear forces and sometimes normal forces. The design strength and stiffness of a moment connection are defined in relation to the strength and stiffness of the connected members. The design

strength of a moment connection may be full-strength (i.e. the moment capacity of the connection is equal to or large than the capacity of the connected member) or partial-strength (i.e. the moment capacity of the connection is less than that of the connected member). Similarly the stiffness of a moment connection can be rigid or semi-rigid compared to the stiffness of the connected member (Eurocode 3 (prEN 1993-1-8: 2003)).

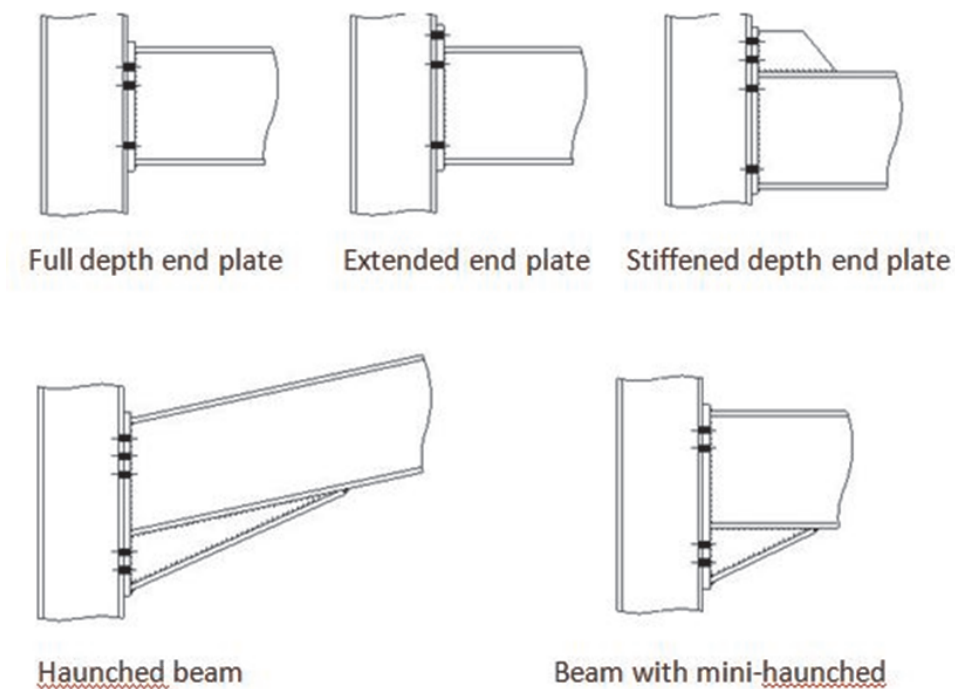


Figure 2.8. Standard Fin Plate Connection Details.

2.1.6. Semi-Rigid Connections

Semi-rigid connections are designed to provide a predictable degree of interaction between members, based on the design moment-rotation behavior of the joints. These kind of connections are usually considered in the estimation model by means of a rotational spring, which is characterized by the elastic constant K_{φ} . The typical values of K_{φ} are commonly expressed in nondimensional form through the ratio K between the rotational stiffness K_{φ} of the connection and the flexural stiffness $E \times I_b/L$ of the connected beam. In Figure 2.9, it is given that some kind of connection types which are obtained from experimental studies.

The structures that involve semi-rigidity turn out less expensive in many cases, apart from the fact that a realistic joint is neither pinned nor fixed, that is, all connections possess semi-rigidity to some extent. Besides, the amount of joint rotation originating from the semi-rigid response may have significant effects in a second order analysis.

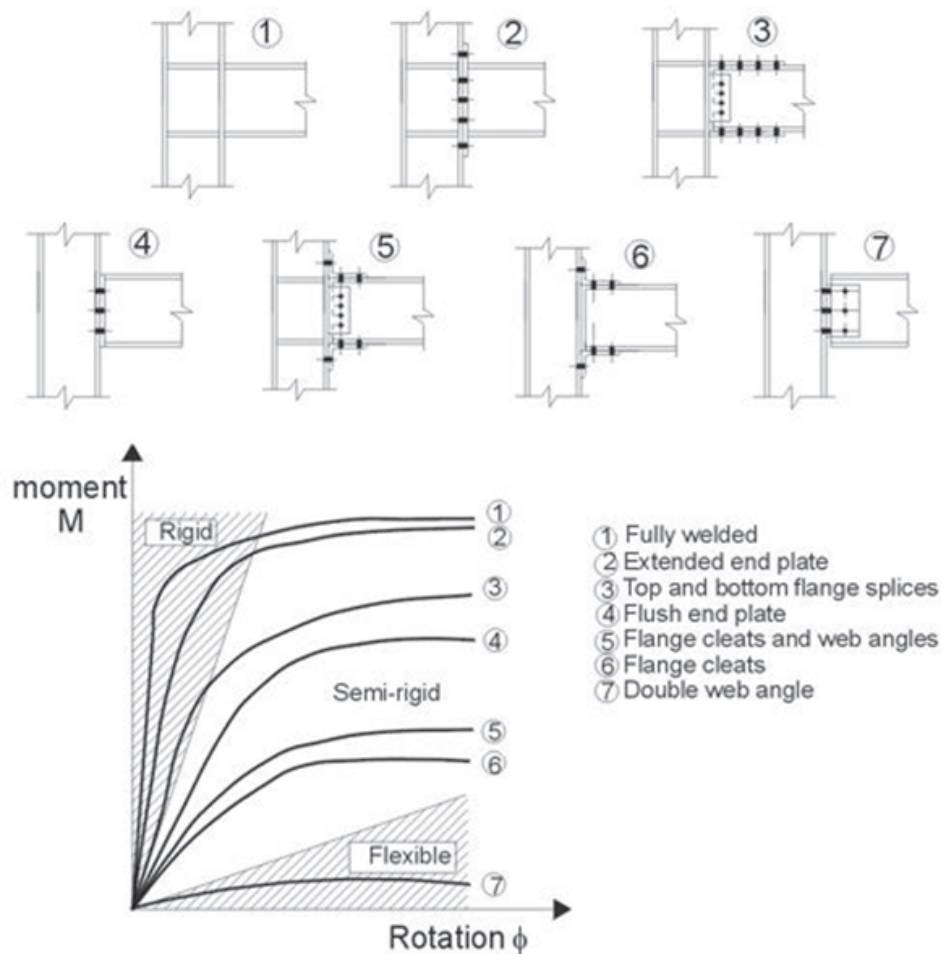


Figure 2.9. Experimental Moment-Rotation Relations of Connections.

2.1.7. Connection Types with Respect to Rigidity

As mentioned above, connections can be classified as pinned, semi-rigid and rigid according to rigidity. In general bolting is associated with pinned and semi-rigid joints while welding is considered as a means to define a rigid connection. Some of the major connection types are classified by Kishi and Chen as:

2.1.7.1. Single Web-Angle Connection (Pinned). Single web-angle connections consist of an angle either bolted or welded to both the column flange and the beam web, while single plate connections are made up with a rectangular plate bolted or fillet welded to the beam web and butt welded to the column flange.

2.1.7.2. Double We-Angle Connection (Pinned). Double web-angle connections consist of two angles fastened to the beam and the column. They display a rigidity, approximately two times that of single web-angle connections although they are considered as within the scope of the classical steel design. So as to examine their behavior under flexure, a number of researchers conducted experiments on double web-angle connections.

2.1.7.3. Top and seat angle connections with Double Web-Angle (semi-rigid). Top and seat angle connections with double web-angle consist of two angles connecting beam flanges to the column and two angles connecting the beam web to the column. Top and seat angles are used to transfer the moment, while web angles serve mainly to transfer the shear.

2.1.7.4. Top and Seat Angle Connection (Semi-Rigid). Top and seat-angle connections differ from the preceding connection type by the lack of the two web-angles that were welded therein to transfer the beam shear. Here, it is assumed that the seat angle carries the beam shear force as well as the beam moment.

2.1.7.5. Extended End Plate Connection (Fixed). Extended end-plate connections are constituted first by welding a steel plate to the beam end and then connecting this end-plate to the column flange by means of fasteners (i.e., rivets or bolts). Extended end-plates can be classified further into two sub-categories on tension side or extended on both sides. The former is the one that is widely used as rigid connections in steel structures. Often, the column demands flange stiffeners confronting beam flanges in order not to cripple under high moment values of the rigid end-plate connections.

2.1.7.6. Flush and Plate Connection (Semi-Rigid). In flush end-plate connections the end-plate extends just to the edges of the beam flanges. Owing to this reduction in the moment arm as compared to the extended end-plate connection, flush end-plate connections exhibit semi-rigid behavior.

2.1.7.7. Header Plate Connection (Pinned). Header plate connections are also made up of an end-plate welded to the beam web and riveted or bolted to the column flange, but this time, the length of the end plate is less than the beam depth. Thus, displaying similar characteristics to double web-angle connections, header plate connections are considered as hinged joints.

Within the scope of this thesis, single plate connections will be considered and examined deeply.

2.1.8. Single Plate Connections

Shear connections are used in almost all modern steel building structures to transfer the reaction of a beam to its support. The supporting member is usually a flange or web of a column or web of a girder. One of the most common types of shear connections is the single plate shear connection which is under consideration in this study. Typical use of this connection is shown in Figure 2.10. This thesis examines the effect of real behavior of single plate shear connections to entire system under gravity and lateral seismic loads. In moment connections, moment is dominant and moment-rotation curves usually are the most important characteristic behavior. It is unlike in shear connections where shear force is dominant although rotation capacity of connection has a major role in its performance.

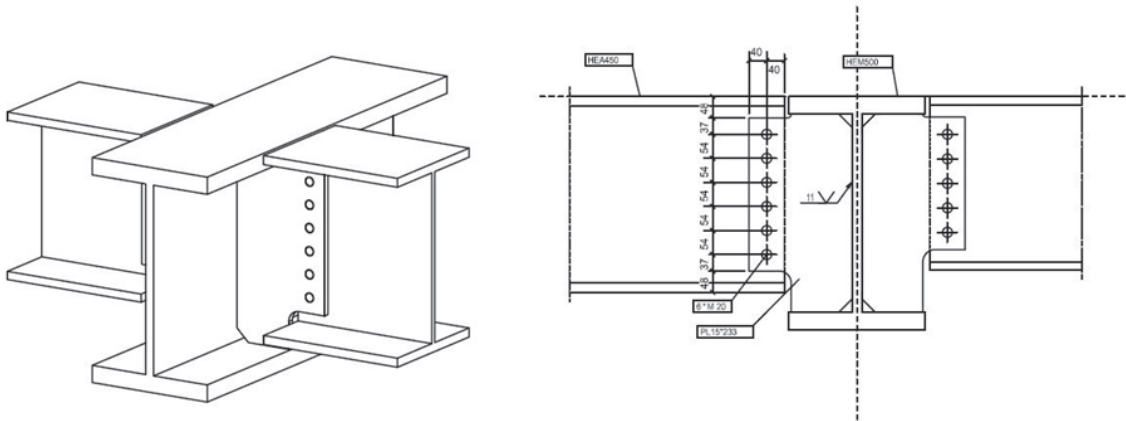


Figure 2.10. Connection Type Under Consideration in the Study.

The behavior of welded steel moment frame buildings in the Northridge Earthquake suggested that the contribution of the shear connections, such as shear tabs, in the frame designed solely for carrying the gravity loads, contributed to the overall stability of these structures. Furthermore, it was hypothesized that the presence of the floor slab had a significant impact on the moment-rotation behavior of the simple connections (Astaneh-Asl *et al.*, 2002). In an effort to quantify the contribution of the simple connections to the lateral resistance of steel structures there are some experimental studies were undertaken (Lui, Astaneh-Asl, 1999, Lui, Astaneh-Asl, 2000).

2.1.9. Earthquake Load Estimation

2.1.9.1. Strong Motion Processing. In many records, errors are found that are not from the characteristics of the instrument. These are non-standard errors and should be removed prior to routine processing. It can never be claimed that a complete and accurate description of the ground motion can be obtained from accelerograms. It is crucial to be able to guess the level of noise exist in each accelerograms for engineering uses. The degree of noise can affect different parameters. The main parameters of interests for engineering are the ordinates of response spectra, both of acceleration and displacement. The peak ground acceleration (PGA) has been traditionally widely used by earthquake engineers to characterize the severity of ground motion. In practice, however, the importance of PGA alone to quantify the damage potential of the recorded

ground motion may have been overemphasized. On the other hand, peak ground velocity (PGV) is obtained by direct integration of the acceleration trace, which makes it sensitive to the noise content of the accelerogram. For structures with fundamental frequency in the intermediate range, e.g., medium height buildings, PGV is a better indicator of damage potential than PGA.

2.1.9.2. Base Line Correction. A major problem encountered with accelerograms is distortions and shifts of the reference baseline, which result in unphysical velocities and displacements. One approach to compensating for these problems is to use base line adjustments, whereby one or more baselines, which may be straight lines or low-order polynomials, are subtracted from the acceleration trace (D.M. Boore, J.J. Bommer (2005)).

Displacements derived from many of the accelerogram recordings Figure 2.11 show drifts when only a simple baseline derived from the pre-event portion of the record is removed from the records Figure 2.12. Changes in the zero level of the acceleration are responsible for these drifts. These changes due to: hysteresis in the sensor, static built-up in A/D converter, tilting of the ground etc. Even though the offset is small in acceleration it can produce completely unrealistic ground displacements. One physical constraint on the ground motion is that the ground velocity be zero, on average, at a sufficiently long time after an earth-quake; for this reason the best way to determine whether there are baseline problems in the recorded acceleration is to look at the velocity obtained by integrating the acceleration.

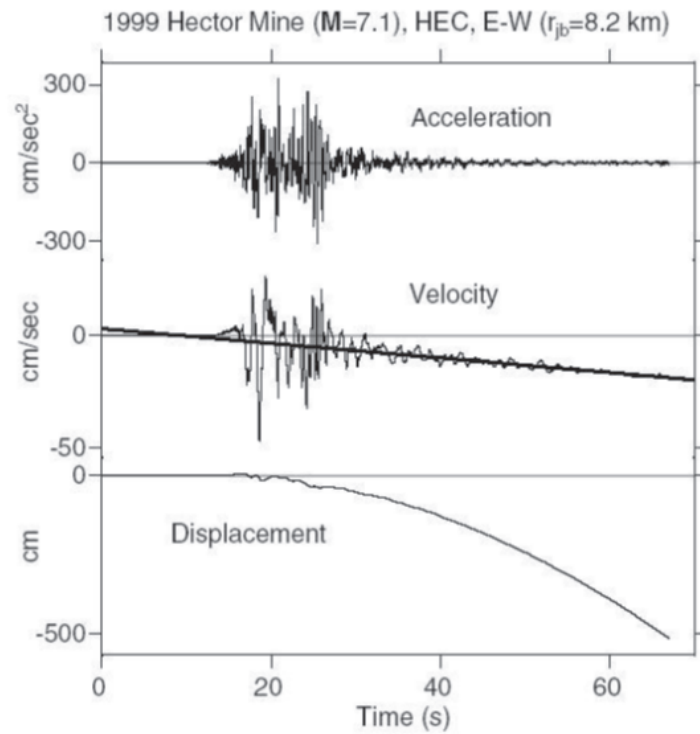


Figure 2.11. Acceleration, Velocity and Displacement for an Analog Record.

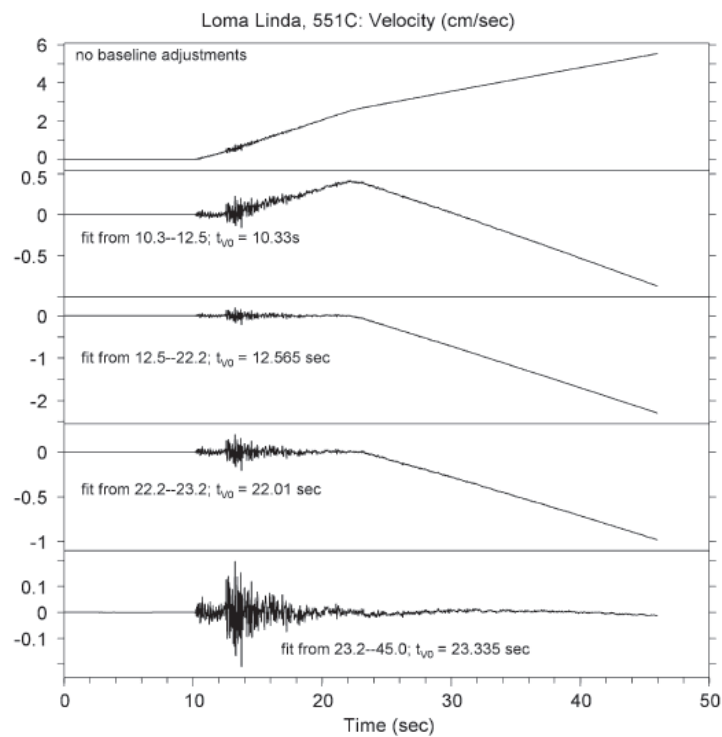


Figure 2.12. Sequential Baseline Adjustment Applied to the Velocity Time-History (Boore, Bommer, 2005).

2.1.9.3. Filters to Reduce Low or Hi-Frequency Noise. The most widely used - and also the most effective and least subjective- tool for reducing the long-period noise in accelerograms is the low or hi-cut filter Figure 2.13. Although the benefits of applying filters are clear, it is important to be aware of sensitivity of the results obtained to the actual parameters selected for the filter (Boore, Bommer, 2005). The selection of these parameters is therefore a critical issue.

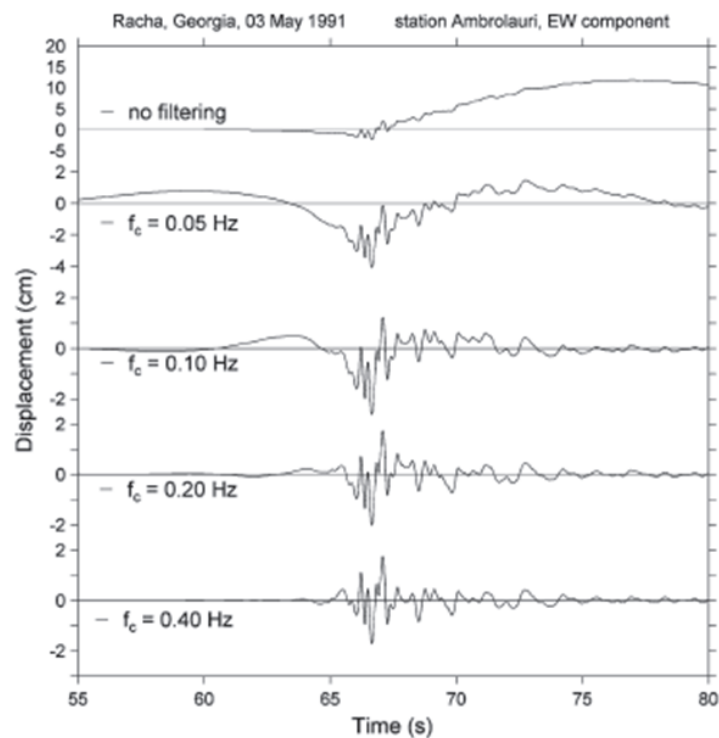


Figure 2.13. Displacement Time Histories for a Series of Filters with Different Parameters (Boore, Bommer, 2005).

2.1.9.4. Strong Motion Records. Ground motions recorded at different sites and in different earthquakes will vary significantly due to several factors, including, but not limited to, earthquake magnitude, faulting mechanism, distance from the recording site to the earthquake source, local site conditions, depth of sediments, basin and other wave-focusing effects and source directivity effects. Although a fully accurate prediction of site ground motion parameters is not possible, the art of ground motion prediction has progressed in recent years such that probable intensity criteria have been established in areas where historical earthquake records and geological information are

available.

Seven different acceleration records for three component directions, available from PEER Ground Motion Database (Pacific Earthquake Engineering Research Center), were used in the analysis. At the deciding stage, the distance of earthquake from the station recording data, site classifications, magnitude and mechanism of the ground motions are taken into consideration. These seven ground motion data were chosen with respect to these features and base-line corrections were made for all of the directions of the data. In this thesis, the decided ground motion magnitudes changes between 6 and 8. Mechanism of the earthquakes is strike slip (SS). The distance from the station changes between 10 and 50 km for each earthquake data. Site class is D<180 according to USGS (United States Geological Survey).

2.1.10. Earthquake Resistant Design of Steel Structures

Steel frames are structures that resist horizontal seismic actions mainly through bending of their members. They have a large number of energy dissipative zones located near to the beam-to-column connections. The energy is dissipated through cyclic bending behavior. During seismic design, it is assumed that the frame as a whole satisfies the basic criterion of avoiding the creation of a soft storey. Under this criterion, the aim is to form plastic hinges in the beams and not in the columns in a global failure mechanism, except at the bases of the columns. This mechanism is the so-called “strong columns-weak beams” concept Figure 2.14. When the design is such that plastic hinges form in the beams rather than in the columns, these hinges have the role of spreading yield through the structure. Moreover, the P- Δ effect is reduced and interaction between axial force and biaxial bending moments in the columns is avoided. The concept of “strong columns-weak beams” is not applied to single storey frames, to the top floor of multi-storey frames and at the base of columns where they are connected to foundations.

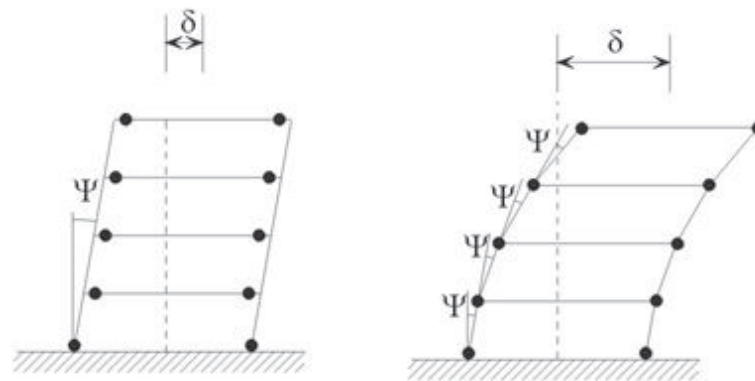


Figure 2.14. Strong Column - Weak Beam Concept.

2.1.10.1. Seismic Behavior of Steel Structures. Experience shows that steel structures subjected to earthquakes behave well. Global failures and a great numbers of casualties are mostly associated with structures made from other materials. This may be explained by some of the specific features of steel structures. Two options can be taken into consideration for evaluating the resisting performance of steel structures. First, it is possible to make structure with very large sections that they are subject to only elastic stresses. Second is that to design structures with smaller sections on the purpose to get numerous plastic zones. A structure designed to the first option will be heavier and may not provide a safety margin to cover earthquake actions that are higher than expected, as element failure is not ductile. In this case the structure's global behavior is "brittle". In a structure designed to the second option selected parts of the structure are intentionally designed to undergo cyclic plastic deformations without failure, and the structure as a whole is designed such that only those selected zones will be plastically deformed. The structure's global behavior is "ductile". A ductile behavior, which provides extended deformation capacity, is generally the better way to resist earthquakes. One reason for this is that because of the many uncertainties which characterise our knowledge of real seismic actions and of the analyses we make, it may be that the earthquake action and/ or its effects are greater than expected. By ensuring ductile behaviour, any such excesses are easily absorbed simply by greater energy dissipation due to plastic deformations of structural components.

Steel structures have been known to perform well under earthquake loads pro-

vided certain guidelines are followed in design. Some of these guidelines are discussed qualitatively at the material, member and structural levels.

Steel being a ductile material, equally strong in compression and tension, is ideally suited for earthquake resistant structures. The common grades of mild steel have adequate ductility and perform well under cyclic reversal of stresses. High strength steels provide higher elastic limits but have less ductility. Another disadvantage in using high strength steels is that they require less areas of cross-section as compared to mild steels and thereby get more prone to instability effects.

2.1.10.2. Seismic Behavior of Bracing Members. Bracing members are used either as part of a lateral load resisting system or to increase the stiffness of a frame in the lateral direction. They may be either pin-ended or fixed ended. Pin-ended braces will be subjected to only axial forces and usually fail by global buckling under compressive load. After the initial buckling, the buckling strength gets reduced in subsequent cycles due to non-straightness of the brace. However, the maximum tensile strength remains relatively unchanged during cycling and presents a ductile behavior. Another advantage of using braces is that it becomes possible to dissipate energy without damaging the main structure and it is easy and economical to replace the braces after an earthquake.

2.1.10.3. Seismic Behavior of Beam-Column Members. Frame members such as beams and columns are normally expected to develop ductile plastic hinges at critical sections. Therefore such members should have plastic cross sections. However, since plastic cross-sections are not always very efficient there is a tendency to use compact and sometimes semi-compact sections in which case adequate ultimate moment capacity, ductility or rotation capacity and also the hysteretic energy dissipation capacity should be ascertained.

2.1.10.4. Seismic Behavior of I-sections of Steel. Steel I-sections have good ductility and energy dissipation capacities and usually fail by local buckling of the flanges, which is sometimes followed by initiation of cracks at the flange-web junction or in the

case of built-up sections, at the weldments.

2.1.10.5. Seismic Behavior of Connections. Connections are the most vulnerable part in steel structures. Bolted connections using black bolts tend to slip, which reduces their energy dissipation capacity under cyclic loading and so are to be avoided. All bolted connections exhibit pinching of the hysteretic loops, which reduces their energy dissipation capacities. Brittle welding failures are common due to low-cycle fatigue and so special care needs to be taken to reduce stress concentrations at welds. The cost of various types of connections dictates the lateral load resisting system used in steel framed structures.

Simple connections are not expected to carry any moments and so only rigid and semi-rigid connections will be discussed. Rigid connections are usually strengthened to an extent that their rotations / deformations are negligible compared to that of the members being connected. This is because in conformity with the capacity design philosophy, it is advantageous to ensure the development of plastic hinges in beams away from the beam-column connection.

Observations on steel moment resisting connections damaged during the Northridge and Kobe earthquakes indicate that such connections develop brittle fractures under high strain rates (Bruneau et al 1998) and so it may be advantageous to provide some degree of flexibility in the connections and go for semi-rigid connections. Semi-rigid connections for composite beams may be obtained by providing top reinforcement.

2.1.10.6. Seismic Behavior of Steel Frames . Steel Frames can be classified as sway frames and non-sway frames depending upon their sensitivity to second-order effects in the elastic range. Both types of frames can be either braced or unbraced even though braced frames normally fall under non-sway category. Braced frames can be classified as concentrically braced or eccentrically braced.

Concentric bracing may be designed to resist either the entire seismic load or as

a supplementary system in a moment resisting frame. In the former case the bracing is used in combination with simple beam-to-column connections (shear connections).

Moment resisting frames (MRF) rely on the ability of the frame to act as a partially or fully rigid jointed frame while resisting the lateral loads. Due to their flexibility, moment resisting frames experience a large drift especially in multi-storeyed buildings. The frames can be designed either to dissipate energy by the formation of plastic hinges at the beam-ends or to dissipate energy in the connections. The former is preferred over the latter due to the complexities associated with connection analysis and design. However, in both cases it is necessary to ensure a strong and ductile connection. Measures taken to improve the performance of connections include the use of column-web stiffeners in I-beam to I-column connections to stiffen the panel zones. Beams ends may also be haunched to ensure the formation of the plastic hinge away from the connection and thereby obtain better performance.

3. CARRIER SYSTEM IDENTIFICATION

3.1. Definition of Steel Carrier System

The structure which is subjected to this thesis has almost a symmetric layout. Lateral forces are resisted by mostly braced frames in north-south (NS) and east-west (EW) directions. There are also moment and shear connections in the system to resist typical forces for each direction. In this study, behavior of moment and pinned connections and impact of connections to global system are investigated under earthquake excitations. To figure out the behavior of connections, they will be remodeled as a flexible connection. All of the structural members and loads are taken from the existing reference building. Beside this, new connection types are designed and implemented. Inspected 28-story steel building has 3 spans for each direction and the values of these spans are 6.0 m, 3.8 m, 6.0 m in X direction and 9.5 m in Y direction (Figure 3.3). From the very first to sixth story, story-height is 4.5 m however height of other stories are 3.5 m. Also, columns are considered fixed in the foundation at the bottom of the building. Plan and elongation sections of reference building are shown in Figure 3.1. The members selected for the 28-story building are shown on Table 3.1.

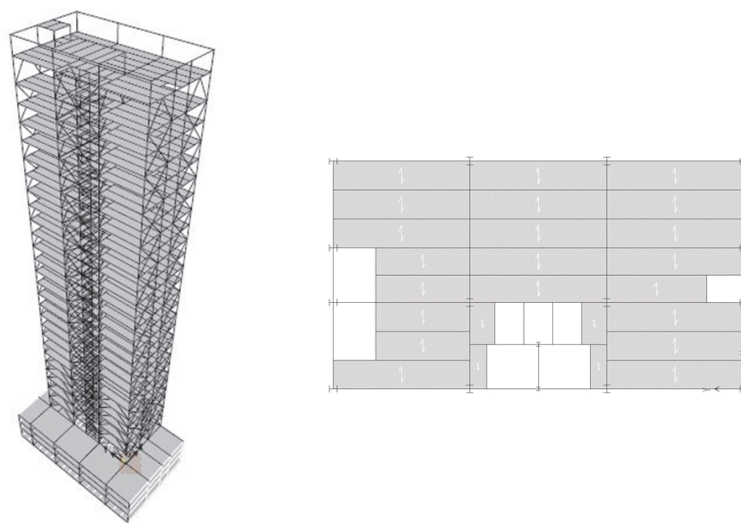


Figure 3.1. 3-D Model and Typical Plan of the Building.

The structural effect of the light concrete slab is not taken into account in the numerical model. The building is modeled as a frame system which contains columns, beams and braces Figure 3.2 and Figure 3.3. Dead and live loads coming from floors are acted on beams as gravity with tributary area method. Beside that, self weight of structural members is acted on related member as distributed load locally downward direction. The dead and live loads on the floors are procured from the original model of the building without multiplying with any coefficient. Masses of each story are distributed to the nodes which lie in the same plane individually. When the location and ground conditions of the building is taken into consideration, it is found out that the building is in first-degree seismic zone and soil class is Z2 with regard to Turkish Earthquake Standarts (DBYBHY 2007). Earthquake parameters are chosen as in Table 3.1 according to information mentioned above. Rigid diaphragm is not considered in the system with purpose of free acting of nodes which lie in a same plane.

Table 3.1. Earthquake Parameters.

Effective Coefficient of Ground Acceleration, A_0	0.4
Building Importance Coefficient, I	1
Spectrum Coefficient	TA = 0.15 sn TB = 0.40 sn
Behavior Factor, R	5

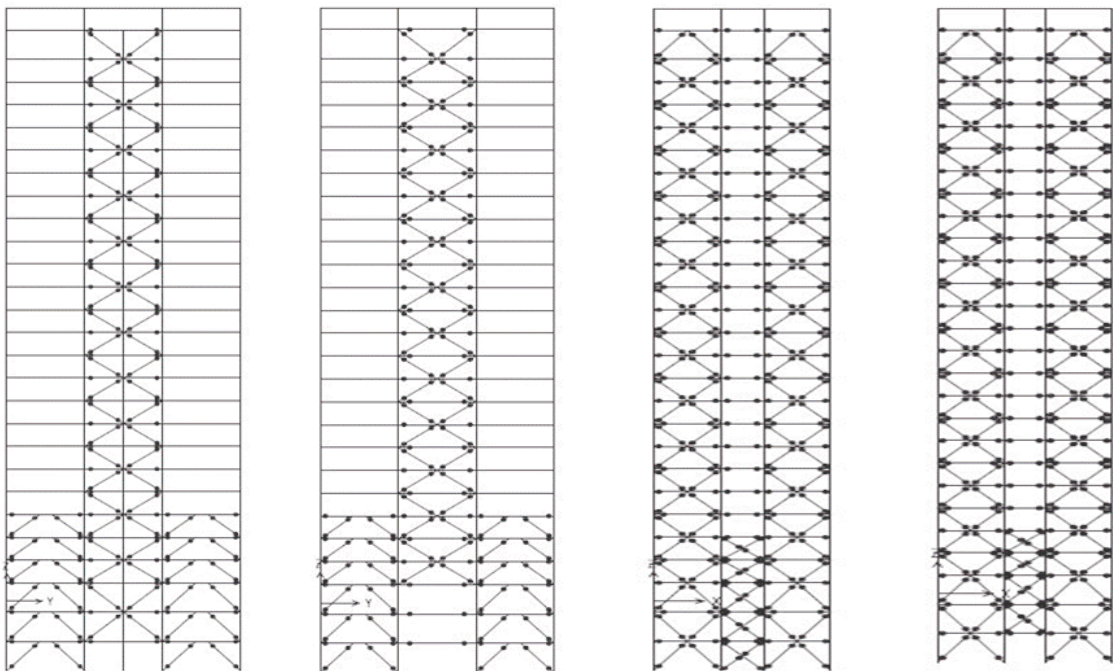


Figure 3.2. Configuration of Model Structure.

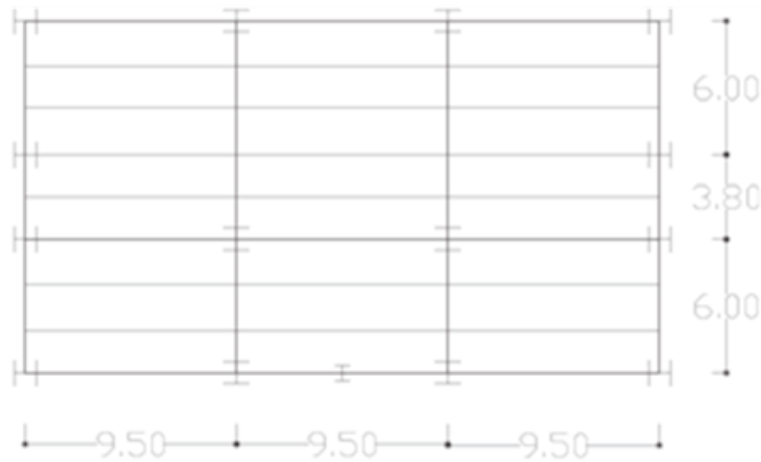


Figure 3.3. Plan of the Building (Lengths are in Meter).

The model described above were used to investigate the behavior and performance of the semi-rigid connection models by means of both gravity and non-linear dynamic analyses using the OpenSEES program. During the analysis, fiber section approach is used to procure non-linear material behavior not only concentrated at one point but also along the member length and height. On the other hand, it was possible to consider $P-\Delta$ and $P-\delta$ effects as a result of the performing capability of nonlinear analysis of the software framework program. $P-\Delta$ effects and column inelasticity are incorporated in the column stiffness matrix. For column inelasticity, the tangent modulus concept is used besides using stability functions for member-frame instability.

Although all columns and some of the beams which are connected with a moment connection are considered as fix-ended, some of the beam elements and braces are regarded as pin-ended. This is because beam elements are exposed to gravity loads and behave elastically. They will have no function in the lateral resisting system. For this reason, lateral earthquake forces were resisted by braces and moment connections. Braces were modeled as moment released elements at both ends aiming to make braces behave as axial members. It was also assumed that the braces are exposed to compressive loads without buckling.

Table 3.2. The Members Selected for the 28-Story Building.

Story	Column	Beam	Brace
28	H400X347X10	HE400A - HE300A	
27	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
26	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
25	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
24	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
23	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
22	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
21	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
20	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
19	H400X347	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
18	H400X383	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
17	H400X383	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
16	H400X383	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
15	H400X422	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
14	H400X422	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
13	H400X422	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
12	H400X422	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
11	H400X463	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
10	H400X463	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
9	H400X463	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
8	H400X463	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
7	H400X900	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
6	H400X900	HE500A - HE400A HE300A - H400X187BL	HE300A - HE240A
5	H400X900	HE500A - HE400A HE300A - H400X187BL	HE300A - HE260A
4	H400X900	HE500A - HE400A HE300A - H400X187BL	HE300A - HE260A
3	H400X900	HE500A - HE400A HE300A - H400X187BL	HE300A - HE260A
2	H400X900	HE500A-HE400A-HE300A-H400X187BL	HE300A - HE260A
1	H400X900	HE500A - HE400A HE300A - H400X187BL	HE300A - HE260A

For beams, columns and braces a displacement based beam-column element was

used. Fiber section was also implemented to this type of element. Combining the section area, total stress, forces and deformations (considering all integration points), force-deformation and moment curvature curves are calculated.

One of the parameters of the assessment of analysis is to investigate the yielding and buckling capacities of columns under severe earthquakes. Therefore, both material and geometric non-linearity were taken into consideration at the modeling stage. For advanced modeling and analysis of the building, the finite element method based on the uniaxial elements was used. For material non-linearity in the form of column inelasticity, distributed plasticity was put to use through the use of the tangent modulus concept. Prediction of linear and non-linear buckling, taking the element's general stiffness matrix and geometric matrix on board, is made by considering a fiber cross section to plastification of the element over the member's length and cross section. On the other hand, "corotational method" was used bearing in mind that the geometric non-linearity in columns. Geometrical non-linearity in the form of P- Δ effects are accounted for by the use of stability functions in a stiffness formulation (Awkar, Lui, 1999).

3.2. Materials and Type of Structural Members

3.2.1. Materials

For modeling members in the non-linear scope of deformation, the following assumptions were assumed:

To present the behavior of St-44, St-52 and St-64 steel materials, "steel01" bi-linear kinematic stress-strain curve was assigned to the elements from the library of materials introduced in OpenSEES. This command is used to construct a uniaxial bi-linear steel material object with kinematic hardening and optional isotropic hardening described by a non-linear evolution equation (Filip Filippou, UC Berkeley). Taking into consideration the idealized elasto-plastic behavior of steel materials, compressive and tensional yield stresses were accepted equal to steel's yield stress. The used section

of each member is the fiber section and the strain hardening of % 0.5 was considered for the member behavior. In the plastic region, the true stress increases continuously i.e when a metal is strained beyond the yield point, more and more stress is required to produce additional plastic deformation and the metal seems to have become more stronger and more difficult to deform. This implies that the metal is becoming stronger as the strain increases. Hence, it is called name “Strain Hardening”. A material that does not show any strain hardening ($n=0$) is classed as perfectly plastic. Such a material would show a constant flow stress irrespective of strain. Strain hardening reduces ductility and increases brittleness. It is given in Figure 3.4 that different type of material behavior to indicate the effect of strain hardening.

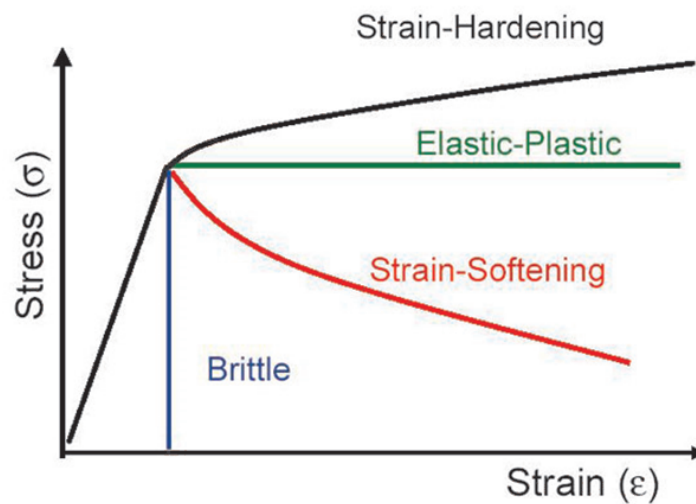


Figure 3.4. Different Types of Material Behavior.

The types of “uniaxial material” is used to describe the behavior of structural elements. For beam-column elements “Steel01” material is defined from the library of OpenSees. This command is used to construct a uniaxial bilinear steel material object with kinematic hardening and optional isotropic hardening described by a non-linear evolution equation (Mazzoni, McKenna, Scott, Fenves, 2006). Steel01 material stress-strain curve is shown in Figure 3.5. In here, b , E_0 and F_y represents strain hardening ratio, modulus of elasticity and yield stress of material respectively.

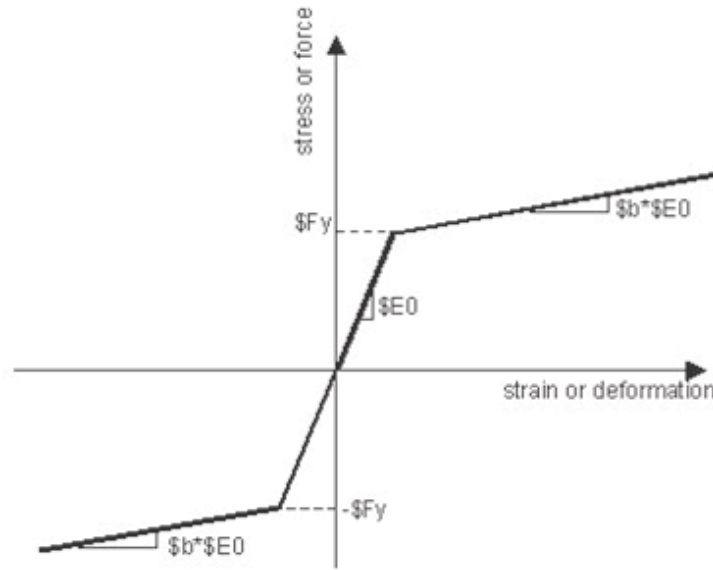


Figure 3.5. Steel01 Material (Mazzoni, McKenna, Scott, Fenves, 2006).

On the other hand, the material type which is used for connection behavior for modified connections is “uniaxial elastic Multilinear” material. In this type of material, the nonlinear stress-strain relationship is given by a multi linear curve which is defined by a set of points. The behavior is nonlinear but it is elastic. This means that the material loads and unloads along the same curve, and no energy is dissipated. The slope has a damage point on the positive strain and a yielding value equal to the beam yielding stress on the negative strain.

3.2.2. Structural Members and Sections

Because of aiming to get geometrical nonlinearities, the elements used in the model are all nonlinear. For columns, beams and braces “displacement beam column” element is used in this study. This kind of elements provide a distributed-plasticity and displacement based prismatic elements. The elements are divided into five integration points to have more accuracy in a finite element analysis. Each parts in an element has same properties.

When examining the connection regions, “zero-length element” is used to define

a new behavior for modified connection details. By means of defining two different nodes at the same location, a zero-length element can be constructed. The nodes are connected by multiple material objects to represent the force-deformation or moment-rotation relationship for the element.

On the other hand, “section” command was used to construct an object which represents force-deformation (or resultant stress-strain) relationships at beam-column elements. A section defines the stress resultant force-deformation response at a cross section of a beam column or plate element (Mazzoni, McKenna, Scott, Fenves, 2006). There are three types of sections available in the library of OpenSees:

- Elastic : defined by materials by material and geometric constants
- Resultant : general nonlinear description of force-deformation response, e.g. mome-nt-curvature
- Fiber : section is discretized into smaller regions for which the material stress-strain response is integrated to give resultant behavior

Within the scope of the analysis in this thesis, fiber section is selected to have a better accuracy. A fiber section has a general geometric configuration formed by subregions of simpler, regular shapes (e.g. quadrilateral, circular and triangular regions) called *patches*. It is shown that the fiber section of steel W shape used in the study in Figure 3.6.

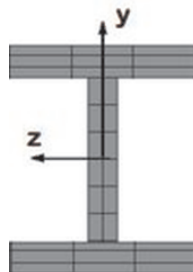


Figure 3.6. Fiber Steel *W* Section.

3.2.3. Structural Loads

Within the scope of the investigated high-building, approximately, 2 KN/m² of dead loads and 3 KN/m² of live loads are taken into consideration. Gravity loads are applied to the beams as distributed load along the elements in each floor. Self-weight of sections considered as gravity distributed loads and applied to related elements. Differently from this, lateral loads are procured from a set of earthquake excitations. Ground motions are applied to the system in both x and y directions.

3.2.4. Connections

Even in well-designed steel frame structure, inelastic deformations occurring at stability members under various seismic loads are limiting the beam plastic hinge occurred in the beam-column joints. The energy absorption of the structure is increased by distribution of plastic deformations and rotation capacity of plastic hinges.

There are two types of connections used in the model system which is called fixed and pinned connections. Fixed connections refer moments and latter ones refer simple and shear connections. Fixed or moment connections take charge in resisting lateral loads.

3.3. Verification Studies

It is also proved that OpenSEES program works accurately during the analysis procedure by other academic studies. Moreover, 28- story building is model in both OpenSees and Etabs finite element programs then performed modal and dynamic analysis to check the consistency of results. To get better understanding about differences of calculation methods between two programs, dynamic analysis, section and material design parameters etc. are examined. To do this, some basic analysis are performed on some small 2-D and 3-D frames and models were prepared by both OpenSEES and Sap2000 finite element software program. In Figure 2.2, a studied 2-D frame model is shown. The model consists of sixteen nodes, 21 elastic beam-column elements and

constraints totally fixing the nodes at the base of the building. The loads of the frame are implemented as uniform distributed along the length of members. Also, lateral load are used for lateral analysis for each story. Geometric transformations, such as P-Δ transformation for columns and linear transformation for beams, are taken into consideration. In this probe, it is obtained that both base reactions and other decisive results are considerable close within the frame of gravity and lateral analysis Table 3.3.



Figure 3.7. 2-D Basic Frame Model.

Table 3.3. Analysis Results of First Floor Column.

Program	Column 1			Column 2			Column 3			Column 4		
	P	V	M	P	V	M	P	V	M	P	V	M
End 1												
OpenSEES	-8.83	11.07	22.22	-69.13	18.27	31.97	-65.47	17.69	31.1	-51.57	12.98	23.4
Sap2000	-9	11.16	23.66	-69.34	18.02	32.62	-65.36	17.55	31.93	-51.29	13.26	24.59
Program	Column 1			Column 2			Column 3			Column 4		
End 2	P	V	M	P	V	M	P	V	M	P	V	M
OpenSEES	-8.82	11.07	10.99	-69.13	18.27	22.82	-65.47	17.69	21.98	-51.57	12.98	15.54
Sap2000	-9	11.16	-9.81	-69.34	18.02	-21.45	-65.36	17.55	-20.73	-51.29	13.26	-15.19
Periods	1	2	3	4	5							
OpenSEES	0.072	0.024	0.014	0.013	0.011							
Sap2000	0.077	0.026	0.015	0.012	0.011							

3.4. Building Model in OpenSEES

The following tasks are performed when building the model:

- Defining units
- Defining model
- Defining recorders for output
- Defining and applying gravity
- Defining modal and dynamic analysis

The model which is the topic of this thesis is a 3D frame building containing six degrees of freedom at each node, the three translations in the planes and the three rotations about the normal of planes. The model type of this building is defined using the “Model” command. In this investigation, meter m and newton N is used for length and force respectively. Seconds is used for time.

3.4.1. Nodal Coordinates, Masses and Boundary Conditions

The nodal coordinates are defined using the “node” command. The number of parameters associated with this command are referenced to the model command. Meanwhile, nodal masses is defined at the same time as the coordinates. Taken three-dimensional problem into consideration here, x , y and z coordinates of each node are defined but only x and y directions are used for defining masses because of interest of only laterally movements. The nodal mass is used to calculate the eigenvalues and to perform the dynamic analysis.

On the other hand, the boundary conditions are defined using the “fix” command. For the structure under consideration, the bases of columns are completely fixed.

3.4.2. Materials

The type of material which is used within the scope of this thesis is “uniaxial material” which represents uniaxial stress-strain (or force-deformation) relationships.

3.4.3. Element Cross Sections

As requiring that element cross section be defined for some kind of element types, it is needed to use “section” command to represent force-deformation relationships at beam-column elements in this study. Among many types of sections, the “fiber” section is used in the structure under consideration. In using of fiber section, simpler quadrilateral patches which give an opportunity to use different type of materials with each patch, are taken into account for defining the surface of elements.

3.4.4. Elements and Element Connectivity

In the scope of this study, “nonlinear BeamColumn” element is used to have a geometrical nonlinear behavior. The section type properties are correlated with beam-column element command to enforce the behavior of materials. Also, the geometric “transformation” is used to relate the local element and section coordinates to the global system coordinates. The OpenSees “Geometric Transformation” command defines how the element coordinates correlate to the global model coordinates.

Elements are defined between two nodes which are identified before. While some of the beam-to-column and all of the column-to-column connections are fixed, braces, secondary beams and some of the main beams have pinned connection.

3.4.5. Gravity and Lateral Loads

Gravity loads in the system are dead and live loads come from slabs and self-weight of the elements. These loads are applied to the beams and each element in the structure as uniformly distributed.

Lateral loads in the analysis are coming from earthquake excitations. Earthquakes are applied only x and y directions to the structure in the analysis. Loads are defined using the OpenSees “pattern” command. Although “plain pattern” is used for gravity loads, “uniform excitation pattern” is used for lateral forces.

3.4.6. Recorders for Outputs

Recorders are used to monitor the state of the model at each step. In this study, recorder of nodes, elements, sections, fibers and connections are used to get better understanding of the behavior of structure under consideration. As nodal displacements and forces are procured from node recorders, moment and axial forces are obtained from element recorder.

3.4.7. Analysis

Both modal and dynamic analysis are operated in this study. Here are the parameters used for defining analysis procedure:

- constraints

The OpenSees “constraints” command is used to construct the constraint handler object. Constraints enforce a relationship between degrees-of-freedom. The constraint handler object determines how the constraint equations are enforced in the analysis.

- numberer

The OpenSees numberer Command is used to construct the DOF Numberer object. The DOF Numberer object determines the mapping between equation numbers and degrees-of-freedom, how degrees-of-freedom are numbered.

- system

The OpenSees system Command is used to construct the LinearSOE and LinearSolver objects to store and solve the system of equations in the analysis.

- test

The OpenSees test Command is used to construct a ConvergenceTest object. Certain SolutionAlgorithm objects require a ConvergenceTest object to determine if convergence has been achieved at the end of an iteration step.

- algorithm

The OpenSees algorithm Command is used to construct a SolutionAlgorithm object, which determines the sequence of steps taken to solve the non-linear equation.

- integrator

The OpenSees integrator Command is used to construct the Integrator object. The Integrator object determines the meaning of the terms in the system of equation object. The Integrator object is used for the following:

- i) determine the predictive step for time $t + dt$
- ii) specify the tangent matrix and residual vector at any iteration
- iii) determine the corrective step based on the displacement increment dU

- analysis

The OpenSees analysis Command is used to construct the Analysis object. This analysis object is constructed with the component objects previously created by the analyst. All currently-available analysis objects employ incremental solution strategies. Here are two types of analysis currently available which are used in this study:

- i) Static - for static analysis
- ii) Transient - for transient analysis with constant time step

4. STRONG GROUND MOTION RECORDS

4.1. Strong Motion Processing

Ground motions recorded at different sites and in different earthquakes will vary significantly due to several factors, including, but not limited to, earthquake magnitude, faulting mechanism, distance from the recording site to the earthquake source, local site conditions, depth of sediments, basin and other wave-focusing effects and source directivity effects.

Seven different acceleration records for three component directions, available from PEER Ground Motion Database (Pacific Earthquake Engineering Research Center), were used in the analysis. At the deciding stage, the distance of earthquake from the station recording data, site classifications, magnitude and mechanism of the ground motions are taken into consideration. These seven ground motion data were chosen with respect to these features and base-line corrections were made for all of the directions of the data. In this thesis, the decided ground motion magnitudes changes between 6 and 8. Mechanism of the earthquakes is strike slip (SS). The distance from the station changes between 10 and 50 km for each earthquake data. Site class is D<180 according to USGS (United States Geological Survey).

Records to be used in the research framework are obtained from the most recent archived database of Pacific Earthquake Engineering Research Center (PEER). The soil type of the region which model structure located is decided *Z2* according to DBYBHY Turkish Earthquake Standards. The average size of the design parameters is taken over by the corresponding articles of Turkish Earthquake Standards with seven real ground motion records which are chosen by limitations mentioned above. Ground motion records differ from the point of their own properties such as maximum ground acceleration, effective duration, frequency content, etc. Therefore, they must be scaled to reflect a similar earthquake level (Aksoylar vd., 2013). For the selected seven real ground motion records, depending on the availability given by PEER which is rec-

ognized competence in international cross platforms, scaling coefficients are procured based on the design spectrum of soil type Z2 according to DBYBHY. During the selection phases of needed suitable raw records for the scaling process, many parameters such as the location properties of the structure, soil type parameters, the importance of status and seismic parameters like earthquakes are decided by the user. On the other hand, while selecting the appropriate database records, weight function which provides significant advantages in scaling and great flexibility can be specified. Weight function is a function that enables assigning relative weights from outside to various period ranges (PEER, 2010).

The scaling process defined by the user according to the target spectrum, the suitability of geometric mean of the selected and scaled seven ground motion records' spectrum curves to the target spectrum is investigated by PEER database system with calculating mean square error values (MSE) which is given in the Equation 4.1. The information about selected and scaled seven earthquake data, within the scope of this thesis, are given in Table 4.1. Furthermore, information about physical properties of selected records are given in Table 4.2.

$$MSE = \frac{\sum_i w(T_i) \left\{ \ln [SA^{hedef}(T_i)] - \ln [fxSA^{kayit}(T_i)] \right\}^2}{\sum_i \mathbf{w}(\mathbf{T}_i)} \quad (4.1)$$

In this equation, $w(T_i)$ and f show weight function and scaling factor respectively. In addition, the related two components in horizontal direction are used as normal and parallel component to the fault line on condition that rotation of these. In the scaling process according to target spectrum defined by user, the convenience of geometric mean of the spectral curves which belong to the selected and scaled seven ground motions is investigated by PEER database system with calculating the values of MSE. As shown in Figure 4.1, R_{jb} means the distance to the fault plane of Joyner-Boore, R_{rup} indicates the closest distance to the fault plane. V_{s30} shows the shear wave speed at 30 meters depth (PEER vd., 2010).

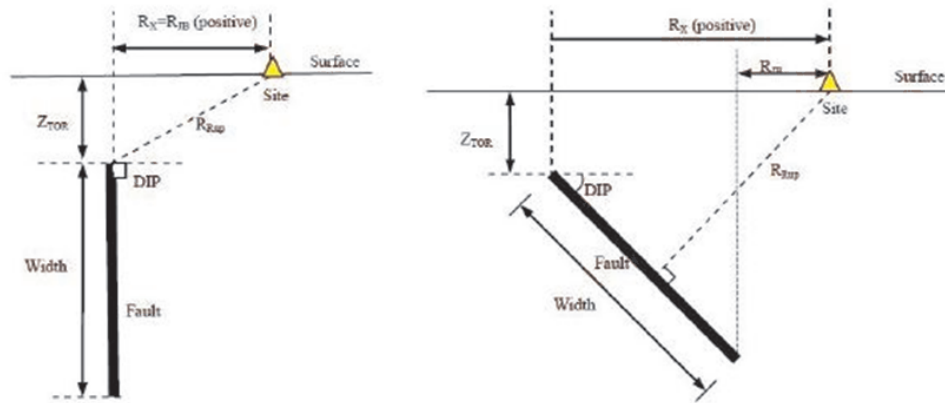


Figure 4.1. For Lateral Leap and Reverse-Normal Faults R_{jb} Ve R_{rup} Values, (PEER vd., 2010).

Table 4.1. Selected and Scaled Earthquake Records for the Analysis.

Eartquake	Year	Station	Squence Number	MSE	f
San Fernando	1971	LA-Holloywood Stor FF	68	0.0201	2.7997
Tabas	1978	Boshrooyeh	138	0.0379	3.4231
Cape Mendocino	1992	Loleta Fire Station	3750	0.0257	1.8413
Landers	1992	Mission Creek Fault	880	0.0382	4.7955
Düzce	1999	Lamont 1058	1611	0.0508	3.7665
ElCentro	1940	Imperial Valley-02	6	0.1529	1.9052
Kobe	1995	Abeno	1100	0.0318	2.5979

Table 4.2. Physical Properties of Selected and Scaled Earthquake Records for the Analysis.

Eartquake	Magnitude	Fault Mechanism	R_{jb} (km)	R_{rup} (km)	V_{s30} (m/s)
San Fernando	6.61	Reverse-slip	22.77	22.77	316.46
Tabas	7.35	Reverse-slip	24.07	28.79	324.57
Cape Mendocino	7.01	Reverse-slip	23.46	25.91	515.65
Landers	7.28	Strike-slip	26.96	26.96	355.42
Düzce	7.14	Strike-slip	0.21	0.21	529.18
ElCentro	6.95	Strike-slip	6.09	6.09	213.44
Kobe	6.9	Strike-slip	24.85	24.85	256

Before checking whether the selected recordings are suitable for conditions of DBYBHY, filtering and corrective actions have been implemented for the scaled cases of these records. Then, it is examined whether these ground motion records provide the requirements mentioned in DBYBHY or not. In Table 4.3, effective durations, maximum acceleration, velocity and displacement values of the selected earthquake data are shown. In addition, duration of the scaled strong ground motion records and maximum acceleration values are given in Figure 4.2. The effective time of the earthquake correlate with the duration for expansion of the fault lines along surface. To determine the effective time of the seismic records in this research, the needed calculation is performed with “Bracket Duration” method. The scaled acceleration values are used to figure the Bracket Duration out. “Bracket Duration” is identified as a duration between the first and the last sequence of exceeding a certain threshold acceleration value (typically 0.05) (Fajhan, 2008).

According to DBYBHY, for the related periods, the average of the spectral acceleration values which are found again for 5% damping ratio of each scaled ground motion must not be less than 90% of the elastic response spectra. Considering the qualifications to be among 0.2T1 with 2T1 as a requirement in the Turkish Code, this condition is provided for all periods between the first effective period, 0.686 and 5.145 second for the considered high rise steel structure Figure 4.9.

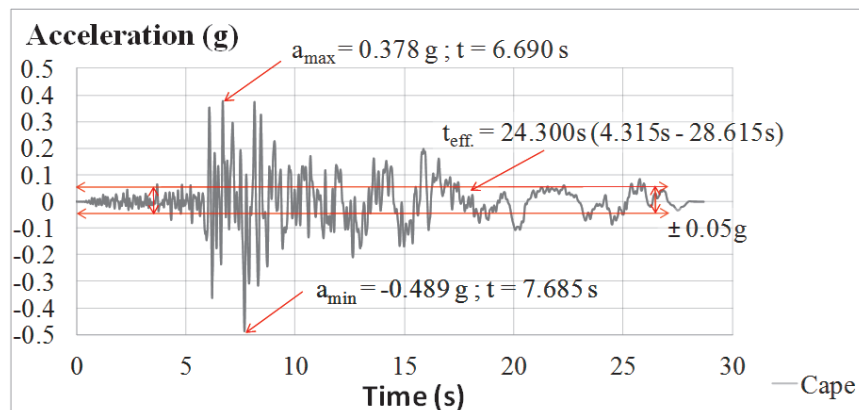


Figure 4.2. Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Cape Earthquake.

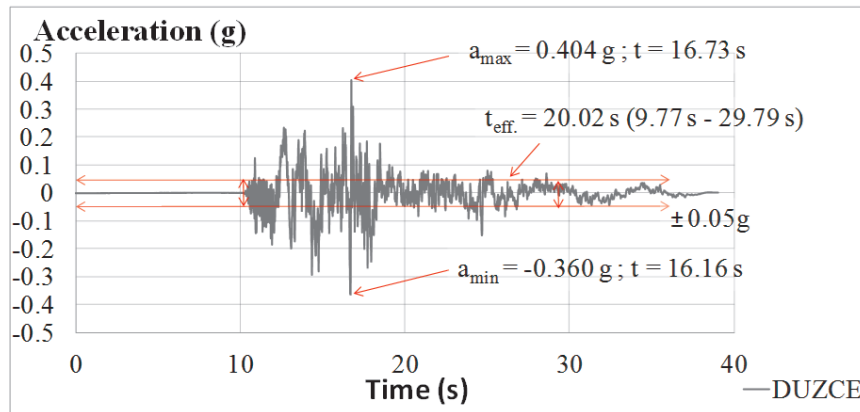


Figure 4.3. Effective Durations And Maximum Acceleration Values of Scaled Ground Motion Records of Düzce Earthquake.

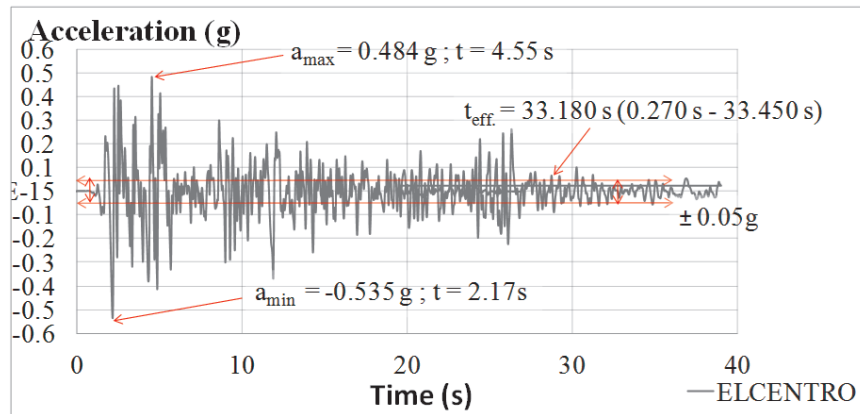


Figure 4.4. Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of El-Centro Earthquake.

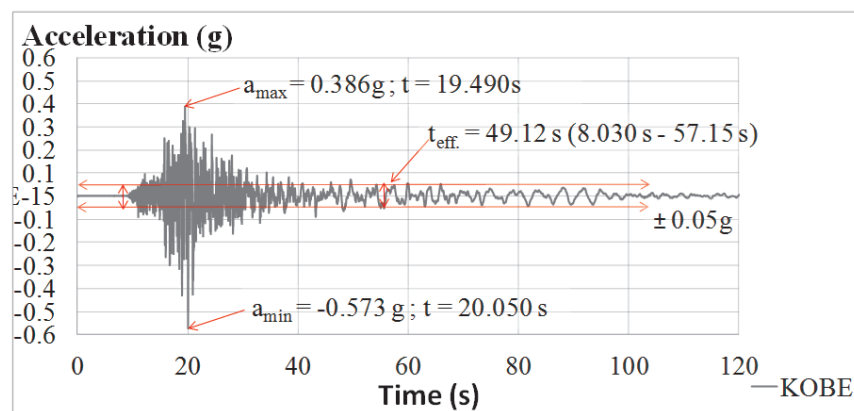


Figure 4.5. Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Kobe Earthquake.

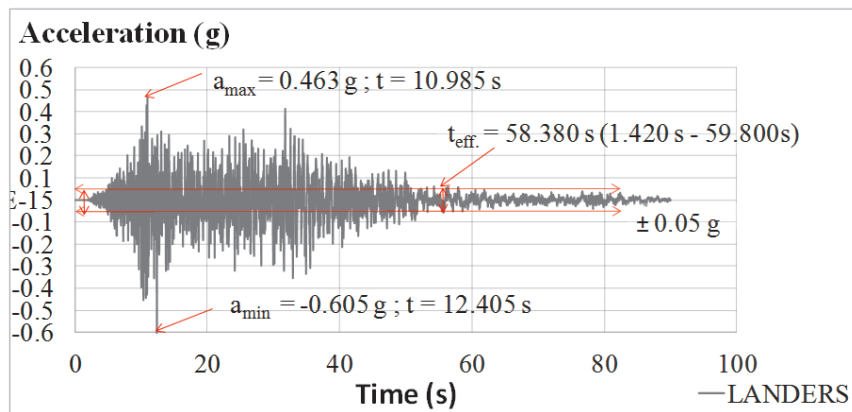


Figure 4.6. Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Landers Earthquake.

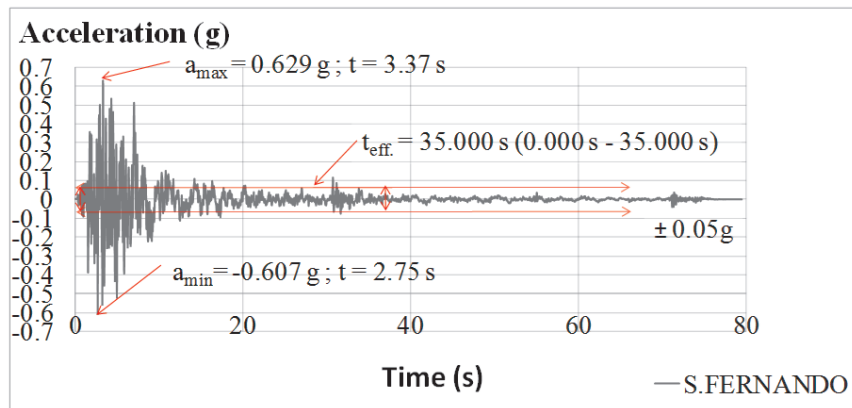


Figure 4.7. Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of San Fernando Earthquake.

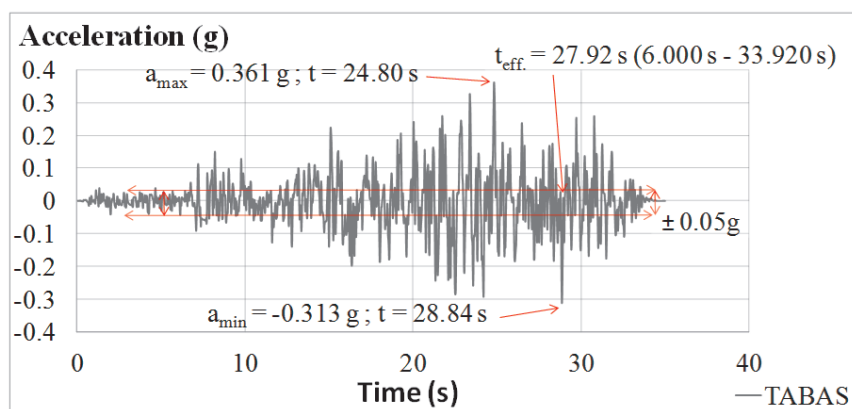


Figure 4.8. Effective Durations and Maximum Acceleration Values of Scaled Ground Motion Records of Tabas Earthquake.

As expected in Table 4.3, the time of strong ground motion movements is greater than $5T_1$ and 15 seconds prescribed in DBYBHY, 2007 for all of the selected records. The mean of the maximum ground motion acceleration values corresponding zero periods is calculated as 0.444. Within the scope of this thesis, the reference effective peak ground acceleration coefficient is obtained as 0.4 for the first degree seismic regions according to DBYBHY. Another condition that should be procured by selected records according to regulations is confirmed with $0.444 > 0.4$.

Table 4.3. Acceleration, Velocity and Displacement Values for Scaled Records.

Earthquake	Effective Duration (t)	Max Acceleration (g)	Max Velocity (cm/s)	Max Displacement
San Fernando	35	0.6293	60.8253	35.8216
Tabas	27.92	0.3612	45.5089	25.8006
Cape Mendocino	24.3	0.3781	52.5158	53.6974
Landers	58.38	0.4628	29.6756	9.8305
Düzce	20.02	0.4039	59.5522	32.7758
ElCentro	33.18	0.4841	47.9439	11.8388
Kobe	49.12	0.3862	55.1723	20.5126

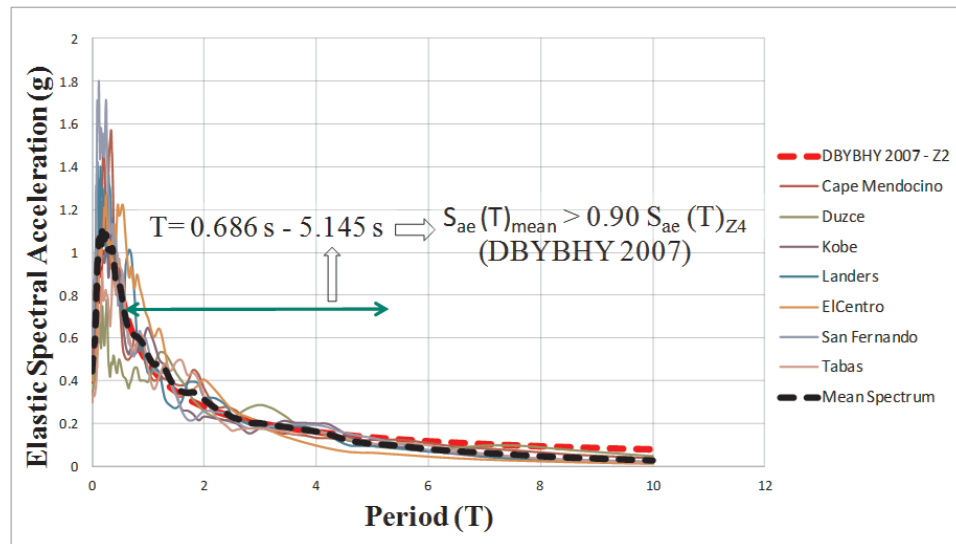


Figure 4.9. Elastic Spectral Acceleration Spectrum.

Aiming that doing nonlinear elastic analysis in this study, the carrier system behavior coefficient R , is recognized as 5 with taking eccentric and concentric braces and normal ductility into account in steel buildings. This value has been recorded in the form of I/R to the ground motions as acceleration multiplier effect. In here,

I is determined as the building importance factor by DBYBHY. According to codes, building importance factor of the considered existing structure in the study has been chosen as 1.

4.2. Selection Conditions of Ground Motions According to DBYBHY (2007)

In Turkish Earthquake Codes, in the case of using of artificial ground motions, it is required to produce at least three ground motions with the following properties.

- The duration of the part of strong motion will not be less than 5 times of the first natural vibration period of the building and 15 seconds.
- The mean of the zero period spectral acceleration values corresponding to the generated ground motions will be less than A_{og} .
- The average of the spectral acceleration values which are found again for 5% damping ratio of each scaled ground motion must not be less than 90% of the elastic response spectra $S_{ae}(T)$.

5. METHODS OF ANALYSIS

The OpenSees takes the input data of structure from text file written in *Tcl/Tk* programming language format. All needed data such as materials specifications, elastic elements data, nonlinear elements data, masses, loads, support conditions, records, and static and dynamic load patterns should be defined appropriately in the text input file. While it may be rather convenient to define two dimensional structures via a line-by-line written input text file, it seems that preparation of such input data file for three-dimensional structure is quite time-consuming (Mani'ie, Moghadam, 2012). The problem would be overcome by either using an appropriate graphical user interface (GUI) or using Loop commands in the *Tcl/Tk* input file. The authors have taken benefits from the second technique by defining all the needed input data in lists and then calling these lists (via the lindex command in *Tcl/Tk*) in loop commands to make the complete input file. In this way, the time needed to model even large structures would be reduced quite effectively.

As we know, performing nonlinear analyses, especially time-history procedures, are quite time consuming. This, mainly, comes from complicated iterations needed in solving the equilibrium equations and updating the state of the structure in the current step. It is obvious that the speed of the analysis, in nonlinear three-dimensional structures, depends on various parameters such as numbers of active degrees of freedom in the structure, hysteretic rules adopted for a specific material or element model, computer hardware specifications and capabilities of the solution algorithms and the computer program used as the analysis engine. Analysis engine of OpenSees is quite robust and powerful. It takes benefits from various algorithms for solving large equilibrium systems of equations, various convergence tests (based on energy, displacement, etc), various integrators and constraints. Thus, this is one of the reasons for using OpenSees in recent study for earthquake analysis.

5.1. Modal Analysis of Structures

One of the aims of the present study is to investigate the seismic responses of flexibly connected frame through their modal responses. By examining the modal responses, important insight can be gained in to the response characteristics of frame. Modal analysis method is based on the dynamic analysis of structure. First, a free vibration analysis (eigen value analysis) is performed to determine the time periods, T_i and mode shapes ϕ_i of the structure in different modes. Then the floor loads in j^{th} mode can be obtained as:

$$\{Q_i\}_j = \Gamma_j [M] \{\phi\}_j A_j \quad (5.1)$$

Where, Γ_j is the j^{th} mode participation factor, given as

$$\Gamma_i = \frac{\{\phi\}_j^T [M] \{i\}}{\{\phi\}_j^T [M] \{\phi\}_j} = \frac{\sum_{i=1}^N M_i (\phi_i)_j}{\sum_{i=1}^N M_i (\phi_i)_j^2} \quad (5.2)$$

And mass matrix is;

$$[M] = \begin{bmatrix} M_1 & & \\ & \dots & \\ & & M_n \end{bmatrix} \quad (5.3)$$

The base shear in j^{th} mode can be obtained as

$$(V_b)_j = \sum_{i=1}^N (Q_i)_j = M_j^* A_j \quad (5.4)$$

where M_j^* is the effective mass participating in j^{th} mode, given as

$$M_j^* = \frac{(\sum_{i=1}^N M_i (\phi_i)_j)^2}{\sum_{i=1}^N M_i (\phi_i)_j^2} \quad (5.5)$$

and

$$A_j = \frac{Z}{2} \frac{I}{R} \left(\frac{S_a}{g} \right)_j \quad (5.6)$$

On the other hand, modal analysis has an importance to prepare a setting and design techniques for seismic problems. To perform accurate modal analysis masses of floors has an important role. Modeling mass properties of the model is another issue of importance. In general, mass can be defined either by defining the total mass at each floor at the calculated center of mass or by defining the nodal masses separately. It seems that for modeling structures with mass irregularities in plan, it would be better to define masses via the latter approach; since the former needs defining polar mass moment of inertia, which again depends on the distribution of nodal masses in the plan. Unbalancing nodal masses for introducing mass eccentricity in plan should be based on a rational distribution. An interesting subject which needs more study, is investigating the effects of nodal mass distribution on nonlinear response of structures. At best, this can be a sensitivity analysis (Mani'ie, Moghadam, 2012).

For estimation modal responses of the structure, 3D analytical model is used. This model is prepared both for OpenSEES and ETABS finite element software program and natural periods are obtained for both programs (Table 5.1). This result show us there are some kind of differences of assumptions in modeling and analysis steps between these two programs. But in general, they proved that the stiffness and mass distribution of structure show consistency. To calculate natural periods, modal analysis is performed. Modal analysis has an importance on results to evaluate the general behavior of whole system. Also, at deciding stage of design techniques and spectral analysis, needed information is obtained from modal analysis.

On the other hand, the modal analysis is valid only for linear behavior. Even we specify nonlinear elements, they are treated as linear. For example, if we include contact elements, their stiffness are calculated based on their initial status and throughout the analysis theory never change. Also, material properties can be linear, isotropic or orthotropic, and constant or temperature-dependent. We defined both stiffness and

mass for a modal analysis. Nonlinear properties are ignored.

Table 5.1. First Six Periods (T) of Models Prepared With OpenSees and Etabs.

	OPENSEES	ETABS
T1 (x direction)	4.9056	4.2452
T2 (y direction)	4.3397	3.8147
T3 (torsion)	1.7695	2.6894
T4	1.3342	1.1823
T5	1.3158	1.1177
T6	1.2018	0.7639

5.2. Dynamic Analysis of Multistory Buildings

Within the scope of seismic problems, earthquake results in horizontal and vertical forces on structures due to inertia effects. Out of these, horizontal forces are generally more critical for the stability of the structure. The earthquake motion consists of a series accelerations and decelerations, which result in inertial forces, continuously changing its direction. To estimate the behavior of the structure under a real earthquake motion, a non-linear dynamic analysis is required. Such an analysis is very complex and requires good understanding of the non-linear behavior of structure under cyclic loads. It is assumed that the columns supporting and interconnecting the floor systems are massless and the entire mass of the structure is concentrated at the floor levels; the floor systems and beams are rigid whereas the columns are flexible to lateral deformation but rigid in the vertical direction. The structure is assumed to be supported on rigid ground. This so-called shear building model is useful in developing the basic concepts of multistory building dynamics (Anil, Chopra, University of California, Berkeley).

In a single-degree-of-freedom (SDOF) system, as seen in Figure 5.1, the mass m

can only move along the vertical x-axis. It is described by the following equation:

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = p(t) \quad (5.7)$$

where m , c , and k means the mass, the damping coefficient and the stiffness respectively. The sum of the inertial force ($m\ddot{u}(t)$), damping force ($c\dot{u}(t)$) and reactions produced by the stiffness k ($ku(t)$) should be equal to external force $p(t)$.

We can show taking into account the second order effects caused by the vertical load, panel shear deformations in the column-beam connections, elasticity of semi-rigid connections and shear deformations of beam column elements as a reason among the main causes segregation of multi-storey steel structures than normal ones. As a part of this study, with considering specified effects non-linear dynamic analysis is made by matrix method approach. The matrix equation discussed in the dynamic solution of high-rise steel frame is summarized in Equation 5.5.

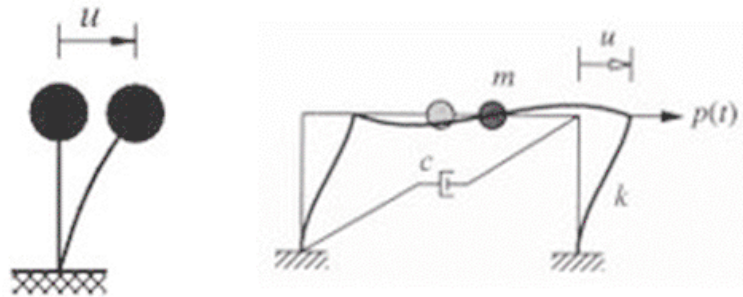


Figure 5.1. Theory of Single Degree of Freedom System.

A multiple-degree-of-freedom (MDOF) system is described by the following equation;

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = p(t) \quad (5.8)$$

where M is $\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}$, C is $\begin{bmatrix} c_1 + c_2 & -c_1 \\ -c_1 & c_2 \end{bmatrix}$ and K is $\begin{bmatrix} k_1 + k_2 & -k_1 \\ -k_1 & k_1 \end{bmatrix}$. Also $u(t)$ is displacement which depends on the history of the deformation.

In this study, non-linear dynamic analysis is made by using real seismic datas to perform seismic evaluation of the model structure. Displacements of top point of the structure, lateral drifts between floors, base shear force diagraphs and amount of rotation of the connections are investigated in the evaluation of these analysis. Beside this, the evaluation of the performance differences between normal and modified form of the existing connections were made. Dynamic analysis was performed using the finite element program OpenSEES which is developed by PEER in Berkeley University. Nonlinear behavior of the elements defined in this program with fiber elements. For the steel material (*St-44*, *St-52*, *St-64*), Steel 01 material model in OpenSEES library, is selected, which has two-directional kinematic load displacement curve behavior was used in the analysis. Each element in the model was defined with fiber model and hardening was taken as 0.5%. On the other hand, “Rayleigh” damping method which is generally used in nonlinear dynamic analysis for considering as a linear combination of stiffness and mass participation ratios. When using rayleigh damping in OpenSees, the damping matrix for an element or node, D is specified as a combination of stiffness and mass-proportional damping matrices Equation 5.9. “Rayleigh” command in OpenSees is shown in Equation 5.10.

$$D = \alpha M + \beta K_{current} + \beta K_{init} + \beta K_{comm} \quad (5.9)$$

$$\text{Rayleigh } \alpha M \ \beta K \ \beta K_{init} \ \beta K_{comm} \quad (5.10)$$

Where, αM , βK , βK_{init} and βK_{comm} are factors applied to elements or nodes mass matrix, applied to elements current stiffness matrix, applied to elements initial stiffness matrix and applied to elements committed stiffness matrix respectively. Within the frame of dynamic analysis, “Transformation” method is used for applying the equations which are used for transferring the degrees of freedom of defined nodes to the analysis. With this method, stiffness matrix is generated by concentrating on restricted degrees of freedom. By the way, the operation size could be decreased. Besides that, numbering of the matrices and equations is potentially complicated because of

the size and detail of matrices and equations. RCM (Reverse Cuthill-McKee) method is used for numbering automatically (Cuthill and Tschemmerneegg, 1969). Also, to control the tolerance of iterations which is generated during the analysis, “Energy Increment” method is considered.

“Modified Newton” method which is updated version of “Newton-Raphson” method common-used in nonlinear solutions is used for the analysis. In Newton algorithm tangent stiffness, which are used in iterations of equations that examined force-displacement relationship, are regenerated in each step, whereas tangents stay constant for each step in “Modified Newton” algorithm. By this way, it enables to reduce the size of equations matrices. Also, to solve differential equations “Newmark” method which is generally used for dynamic analysis, is considered. Damping matrices in which in the scope of “Newmark” method, is obtained from combination of stiffness and mass matrices.

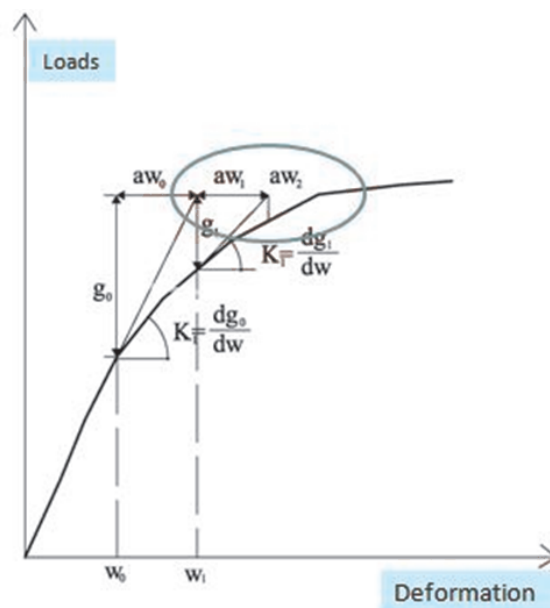


Figure 5.2. Concept of Newton-Raphson Method.

According to the analysis, seven real earthquake data used and are scaled with regard to the design spectrum predicted in DBYBHY. Scaling operations are described in detail in related chapter. On the other hand, one of the factors effects earthquake forces is “Damping”. Damping of the structures is the property which causes dissipation of the energy and results in reduction of effective earthquake force on the structure. It is

expressed as damping ratio or percentage. It depends on the material of construction and level of strains developed in the material. It is generally taken as 2%, 5% and 10% for steel structures. Damping ratio, a parameter to be considered substantial among the dynamic analysis parameters, is taken 5% in this study. If a structure damping value begins to decrease, the movement of the ground acceleration can be increased rapidly. So, damping ratio is crucial in determination of the seismic behavior of the structure. In the response spectra in Figure 5.2, it is seen that peak acceleration value for 0% damping ratio is 3.2g, for 2% it is 0.8g and for 10% it is 0.65g (Arnold, 2006). Engineers generally use 5% damping values for building type structures.

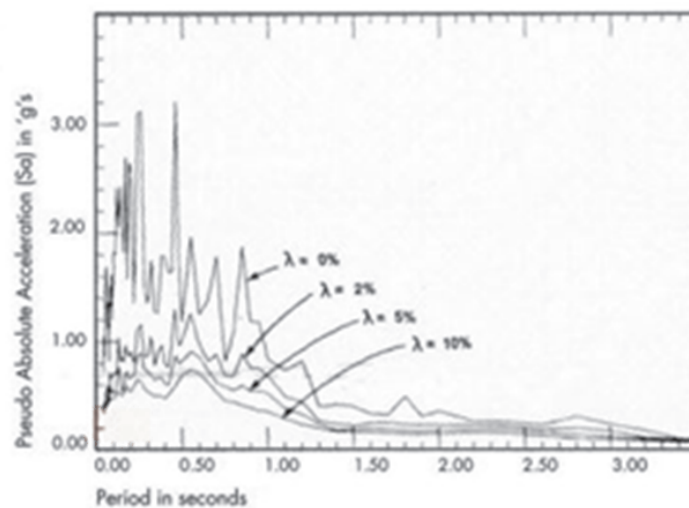


Figure 5.3. Response Spectra for Different Damping Ratios (Arnold, 2006).

Damping has the effect of decreasing the natural circular frequency of vibration and increasing the natural period of vibration. Figure 5.3 shows the effects of damping. For damping ratios less than 0.2, a range which includes most structures, these effects are negligible (Anil, Chopra, Berkeley).

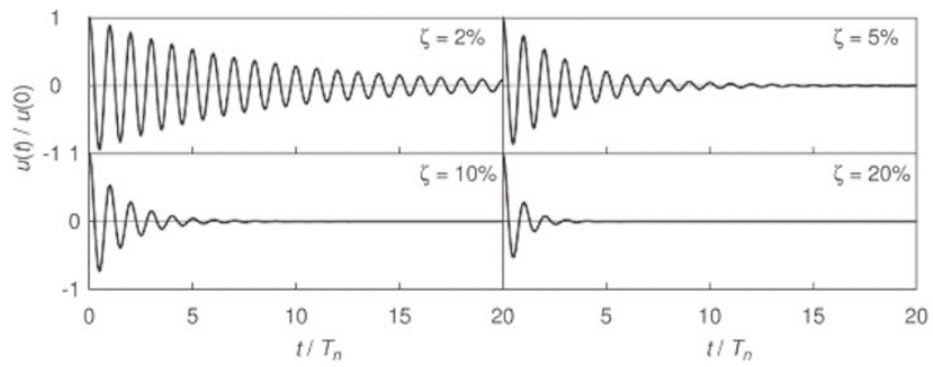


Figure 5.4. Effects of Damping.

The natural circular frequency ω , and period of vibration T are affected by damping as follows:

$$\omega_D = \omega \sqrt{1 - \xi^2} \quad (5.11)$$

$$T_D = T / \sqrt{1 - \xi^2} \quad (5.12)$$

where ξ is the fraction of critical damping coefficient.

6. CONNECTIONS

All of the secondary beams in the structure are in global Y direction. They have pinned connection in both sides connected to girders. Here is the typical pinned connection model taking no moments in axial direction Figure 6.1. Normally these connections do not work against lateral forces although they carry loads and transfer them from slabs to main girders. All of the connections belong secondary beams have same configuration and same capacity. Therefore only one type of bolted shear connection is investigated in the scope of this thesis. So, it is just able to carry shear loads but no moments. After modification, we will have chances to understand the real behavior of the connection that transfer moment to girders when the connection starts to rotate under loads and the bottom flange of the beam reaches the girder then starts to transfer moment.

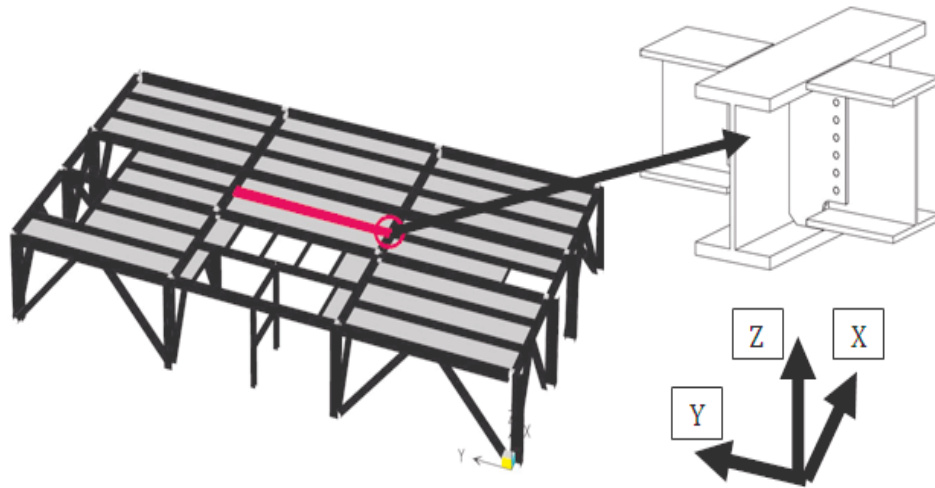


Figure 6.1. Configuration of Modified Connections Between Secondary Beams and Girders.

6.1. Mathematical Model of Connections in Opensees

Traditionally used in practical usage in the analysis, connections are assumed as fully rigid and pinned, but actually, connection details are never fully rigid or pinned.

Studies performed on the basis elements in steel structures generally show ductile deformations, conversely, this condition occurs usually in the form of brittle fracture for connection regions. Deformations occurred in the connection regions affect the global behavior of the system significantly. Semi-rigid connections in OpenSees model are applied as simplified rotational spring model shown in Figure 6.2 in the analysis.

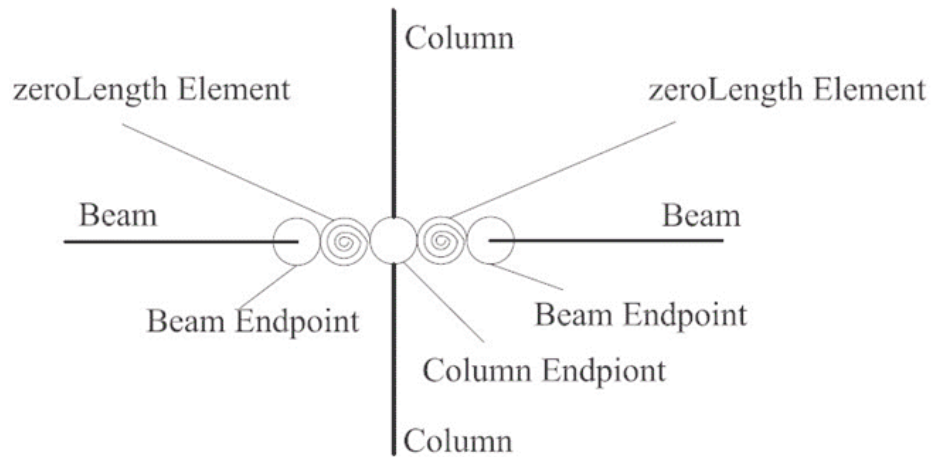


Figure 6.2. Spring Model Representing the Connection Details.

“Zerolength Element” from the database of OpenSees, was assigned between the ends of column and beam elements. All of the degrees of freedom of the connections which are previously defined as pinned and relevant degrees of freedom of joint are left free are taken into consideration with new approach. Thus, partial translational and rotational stiffness is given to the shear connections. As a result, the real behavior of connections and their effect on the global response of the structure is considered.

The stiffness of the semi-rigid connection type (shear) used in representative model structure, is determined by three dimensional finite element model given in Figure 6.2. This model was subjected to both tensile forces and moments separately. Then, the strength of the connection was calculated. Strength calculations of connection model were carried out using finite element program ABAQUS. As seen in Figure 6.3, after a while, bottom flange of beam element contacts the girder column due to large rotation and the connection begins to transfer moment. According to capacities of connection elements, the system shows permanent deformation in the connection

region at large tensile force and moment.

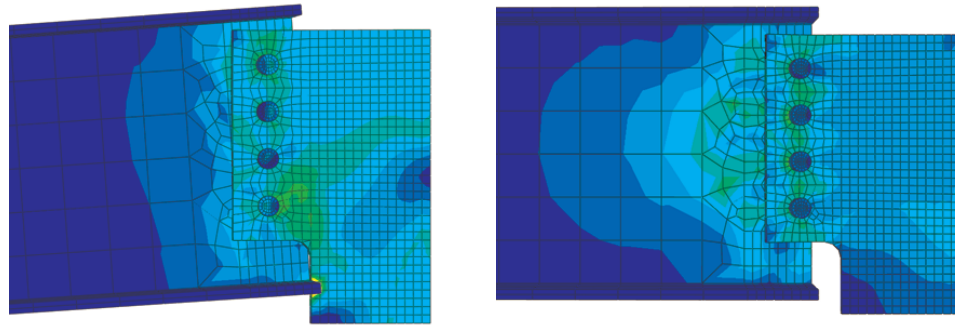


Figure 6.3. Single Plate Shear Connection (a) Against Tension and (b) Against Moment Deformations of the Three-Dimensional Finite Element Model.

Consequently, as examined in detail in the studies, (Minas, Charis, 2009), (Silva, Coelho, 2001), (Huber, Tschemmerneegg, 1998), (Block *et al.*, 2007), (Warmka, Truman, 2000), (Tagawa, Gurel, 2005), (Zhu *et al.*, 1995), it is expected that having positive influence to respond under seismic movements by giving partially stiffness to cyclic behavior of the shear connections through converting pinned connections into semi-rigid connections.

6.2. Material Behaviors of Connections

The nonlinear behavior of the connection in both tension and moment is simulated using “UniaxialMaterial ElasticMultilinear” command. In this section, non-linear load-displacement relationships are discussed. The behavior of model structure is elastically non-linear. This means, during the load and unload stages, material shows behavior on the same curve and will not be any loss of energy. By this way, yielding and damage of connection details are not taken into account. Strength-deformation graphs represents the non-linear behavior of the springs are shown in Figure 6.4 and Figure 6.5.

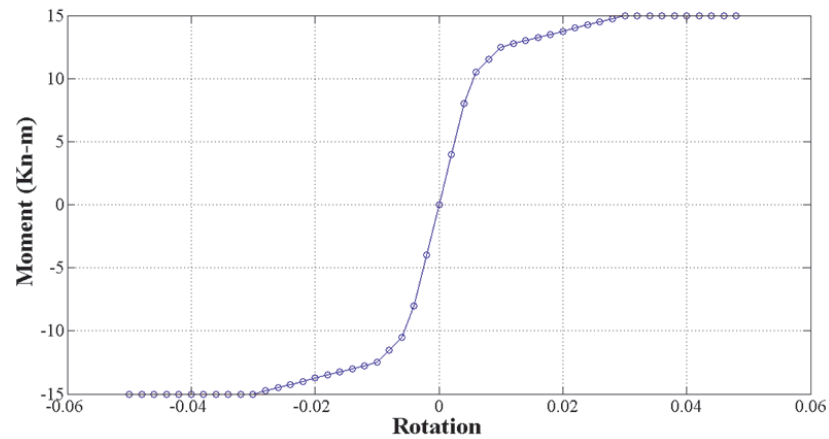


Figure 6.4. Moment-Rotation Graph for the 3D Single Plate Shear Connection Model.

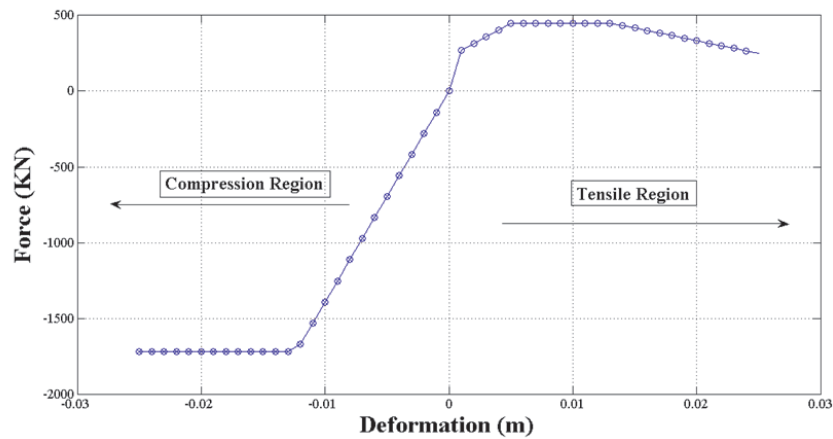


Figure 6.5. Force-Deformation Graph for the 3D Single Plate Shear Connection Model.

6.3. Modeling Connections in OpenSEES

In OpenSEES, equalDOF command was used to define the behavior of each degree of freedom (DOF) in a connection. As shown in Figure 6.6, there is a master node (10) which is the point that beam and column connected, and a slave node (20) in the connection. All the nodes in a connection are located in same coordinates. As a result of modeling 3D system, there are six degrees of freedom in a connection. To decide the behavior of connection each DOF is taken into consideration. For making pinned connection along X direction, M_y is omitted from the command. This is because there is no moment in X direction in pinned connection. On the other hand, for making

semi-rigid connection, all six degrees of freedom are used to define behavior for each of them. OpenSEES enables users to describe each DOF individually with springs. With the help of zero-length element, it is possible to enforce moment-rotation or load-deformation behavior to the connections with a new material that is defined separately.

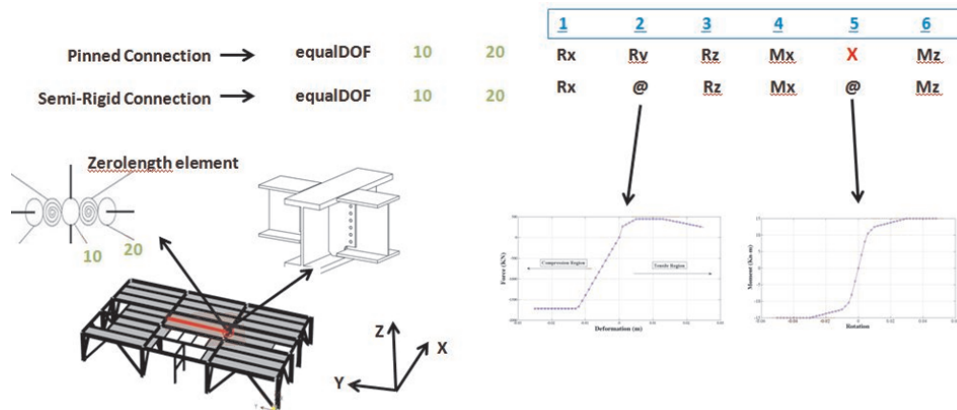


Figure 6.6. The Orientation of Zerolength Elements and EqualDOF in X direction in OpenSees.

7. RESULTS

7.1. Conclusion

In the scope of the recent study, a nonlinear dynamic earthquake analysis is conducted. As it is mentioned in Chapter 4, real earthquake ground motion data is used for the analysis. The earthquake motion is scaled with a certain coefficient obtained according to the site and location of the building. The most hazardous direction of earthquake data is taken for both $-x$ and $-y$ directions as explained in (Aksoylar, Elnashai Amr, Öztürk, 2013). Earthquake forces are inertia forces which act on each mass particle of the structure and act throughout the structure. It is proportional to the mass and acceleration. The acceleration increases along the height of the building and different floors have different acceleration. Therefore, it is convenient to assume that the mass of the building is lumped at the centre of its floors and it can be assumed that the earthquake forces are acting at these masses. However, the masses are applied to each node in a floor, not at the center of floors, because of absence of diaphragm approach.

Secondary beams used in the 28 -story tall steel structure are connected to the structural system with simple shear connections. These secondary beams are designed only to carry the gravity, and they do not resist to lateral forces. In a typical structural analysis program, these shear connections are idealized as pinned connections, which are assumed to resist no moment and assumed to transfer the beam axial force without any deformation (connection is as strong as the beam in tension and in compression). In this study, semi-rigid connection approach has been applied to simulate a more realistic behavior of simple shear connection.

With a semi-rigid connection approach, the structural system is assumed to transfer more moment and even absorb more energy if the connection overloads and has plastic deformations. To assess the performance of the structural system with and without semi rigid connections, three measures were used. First, the inter-story drift

ratios were calculated. Second, the base shear of the building is investigated and ultimately, the moment capacities of specific members in the system are investigated. Besides, changes in the values obtained from the modal analysis periods are examined through modification of the connection details. Natural periods and vibration modes are obtained from the eigenvalue analysis. When interpreting the results obtained in the scope of these parameters, the first 3 periods of rigid, pinned and partially rigid modeled system are compared in Table 7.1. Significant changes are not observed from the modal analysis results. Since the semi-rigid connections parallel to the y -direction have now limited rotational stiffness, the semi-rigid structural system has slightly less (1%) natural period in y -direction (T_2) as expected. The natural periods of the building cannot change significantly because the zero-length elements have very small stiffness compared to the building size.

Table 7.1. The first 3 Periods (T) of Pinned, Semi-Rigid and Rigid Connection Structural System.

	Pinned	Semi-Rigid	Rigid
T1 (x direction)	4.9056	4.9047	4.8962
T2 (y direction)	4.3397	4.3002	3.9773
T3 (torsion)	1.7695	1.7692	1.766

7.2. Performance Assessment of Semi-Rigid and Rigid Connected Structural System by Comparison With Pinned Connection System

Base shear is important parameter in seismic design and it related to the building's overall stiffness in the direction of an earthquake. If the building is excited in x -direction, the semi-rigid connections, which are parallel to the y -direction should have no affect on the building. When the base shear force is compared, it is observed that there is not a significant difference between the structural system with pinned connections and with semi-rigid connections Figure 7.1. In the y -direction, however, visible differences are obtained as seen in Figure 7.2. This means that modifying the pinned connections of the secondary beams as semi rigid connection has an impact on resisting the earthquake loads in the parallel direction with secondary beams. When

the magnitude of the excitation force gets higher, the rotations at the secondary beam ends get larger and the semi-rigid connection transfers larger moments; hence the base shear gets larger with the increased building stiffness.

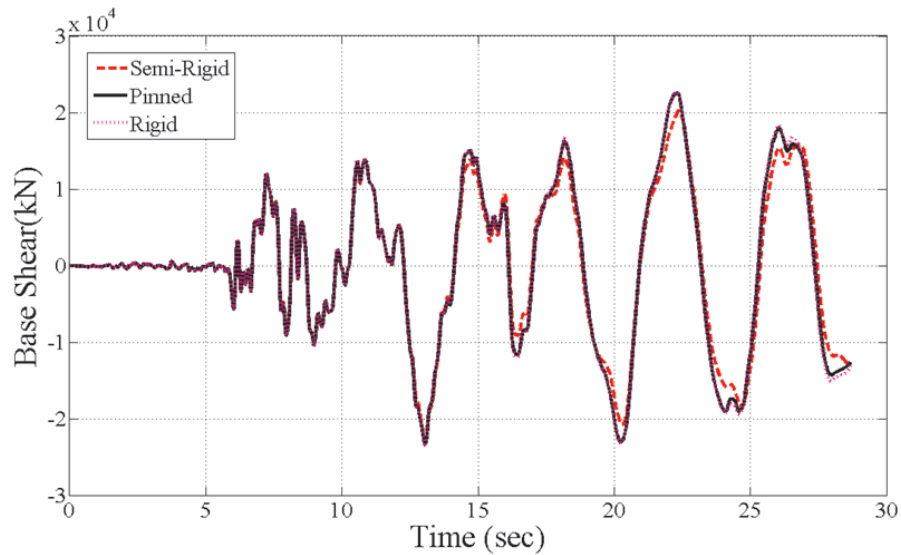


Figure 7.1. Comparison of Base Shear of Pinned and Semi Rigid and Rigid Connected Systems in X direction.

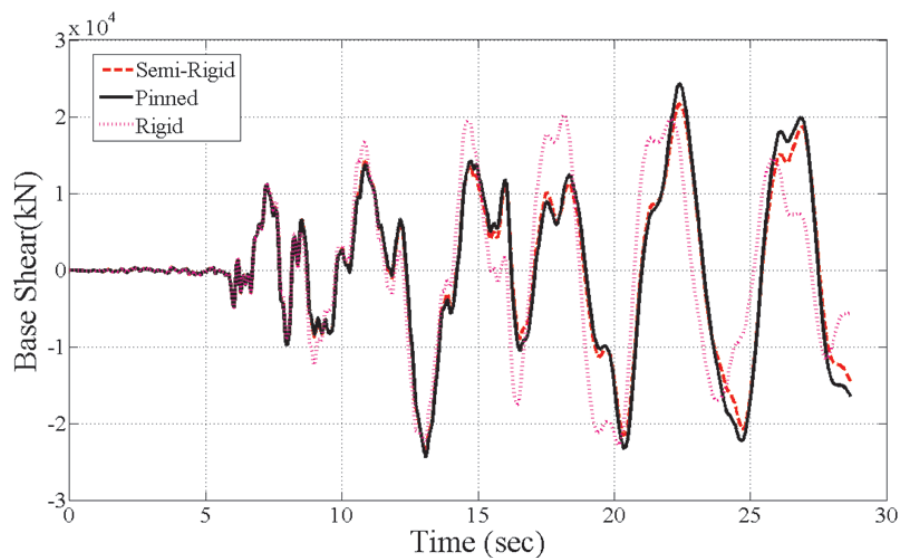


Figure 7.2. Comparison of Base Shear of Pinned and Semi Rigid and Rigid Connected Systems in Y direction.

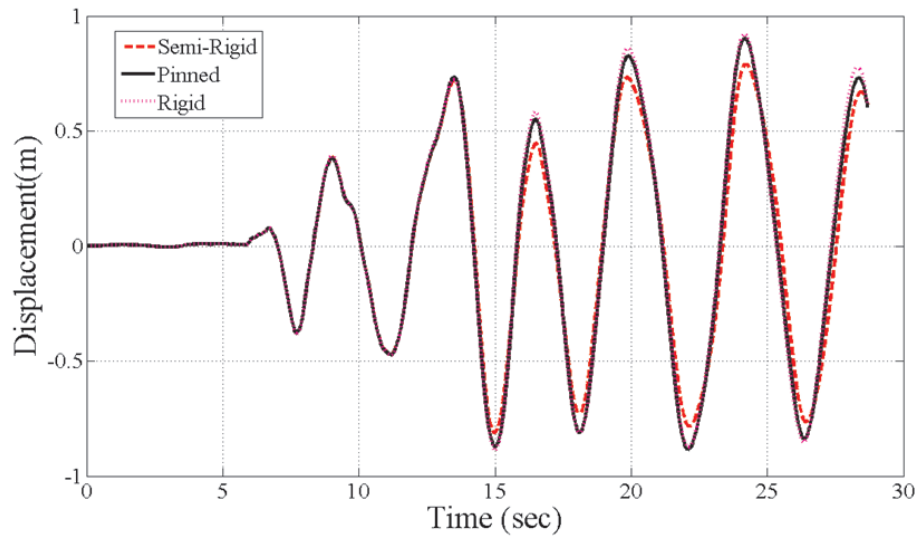


Figure 7.3. Comparison of Top Floor Displacements of Pinned and Semi Rigid and Rigid Connected Systems in X direction.

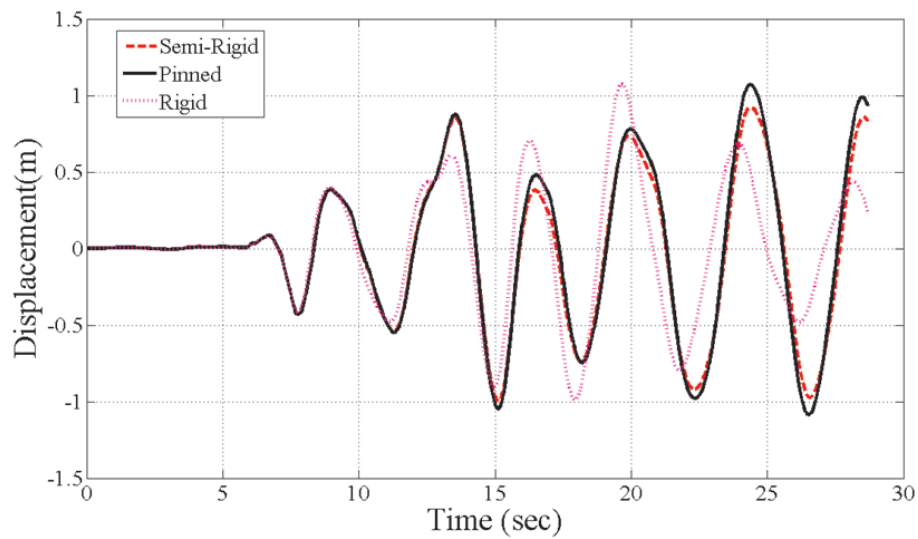


Figure 7.4. Comparison of Top Floor Displacements of Pinned and Semi Rigid and Rigid Connected Systems in Y direction.

Figure 7.3 and Figure 7.4 show the top floor displacements on the 28th floor in X - and Y - directional earthquake excitation, respectively. As clearly observed, the drift on the top floor is very similar between the structural system with semi-rigid connections and the system with idealized pinned connections. However, rigid connections give us notably different results. It proves that making connections more

stiff causes different behaviors.

Figure 7.5 show the inter-story drift ratios in 28 story tall building. The drift ratios in the tall building do not exceed 5% and the largest drift is seen on the 6th - 8th floors. When the building is excited in y - direction, it experiences larger drift ratios. The reason of this is the use of the braces in the first floors more than upper floors. Also earthquake loads are increased in the upper floors. The roof top of the building is not a complete floor, hence the drift ratio on the last story could be neglected in the observations Figure 7.6.

Using semi-rigid connections instead of pinned connections enables the secondary beams to carry moments; hence, the structural system becomes more resistant to horizontal displacements, which should in principle reduce the drifts in building stories. However, the inter-story drift ratio does not seem to be affected by the semi-rigid connections in both x - and y - directions. Depending on the frame height, connection type and earthquake, connection flexibility may or may not increase the inter-story drifts for lower stories. Former studies such as (Awkar and Lui, 1997) show that flexibly connected frames experience more inter-story drifts than rigid counterparts. This study gives us a different conclusion; the differences in drift ratios are negligible as seen in Figure 7.5 and Figure 7.6. One of main reasons for similar drift ratios could be attributed to significant number of lateral bracings in the structural system. The tall building has a dual lateral force resistant system: it has moment connections as well as bracings. The bracings keep the drift ratios at a reasonable level. It is also observed that the drift ratios are virtually not changed in x - directional earthquake excitation. This could be attributed to more lateral brace elements in x - direction and then in y - direction. The structure is more stiffer against x - directional earthquake excitation.

Unlike a static problem in which deflection is inversely proportional to the structural stiffness, the relationship between the deflection and the stiffness is more complex for a dynamic problem. In a dynamic problem, the excitation force is resisted not only by force associated with structural stiffness, but forces associated with damping and inertia as well. A response may be in phase or out of phase with the excitation depend-

ing on the specific combination of mass, damping, stiffness of the system and frequency of the excitation force (Awkar, Lui, 1997).

Within the framework of this study, at the stage of evaluating the performance of the structure, life safety (LS) (design earthquake), preventing collapse (PC) (maximum earthquake) is used as criterion for sparse and rarely seismic ground motions respectively. These criteria were taken as 2.5% for LS and 5% for PC under consideration of drift limits (FEMA 356).

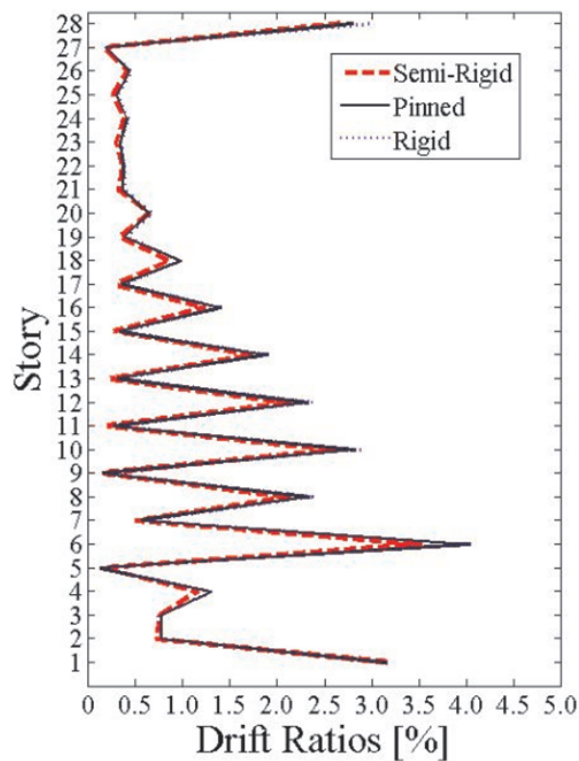


Figure 7.5. Comparison of Drift Ratios of Pinned, SemiRigid and Rigid Connected Systems in X direction.

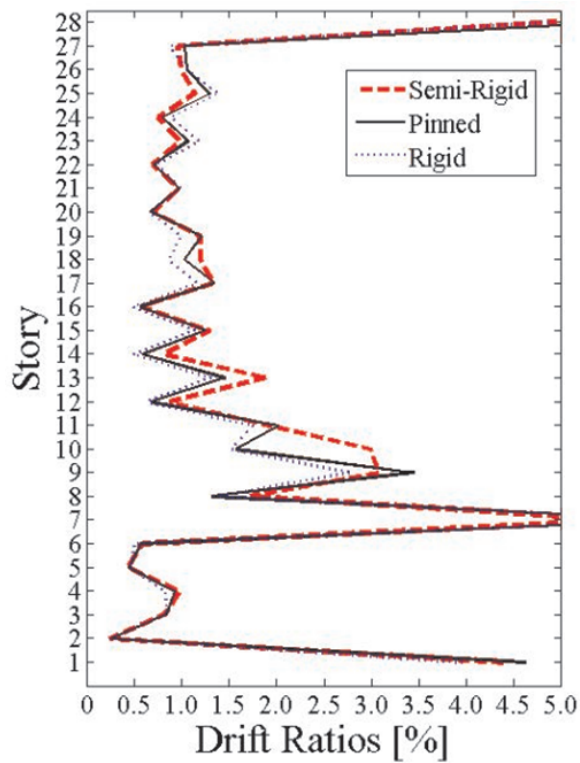


Figure 7.6. Comparison of Drift Ratios of Pinned, Semi Rigid and Rigid Connected Systems in Y direction.

In Figure 7.7, the points which considered for drift evaluation are shown. Although there is no rigid diaphragm in each story, all the points in a story have same displacement ratio. It is because that outer points in the system are chosen for drift investigation. As seen in Figure 7.7, number of brace elements in X direction are more than in Y direction. So, this fact make the system more stiff and rigid in X direction by lateral forces.

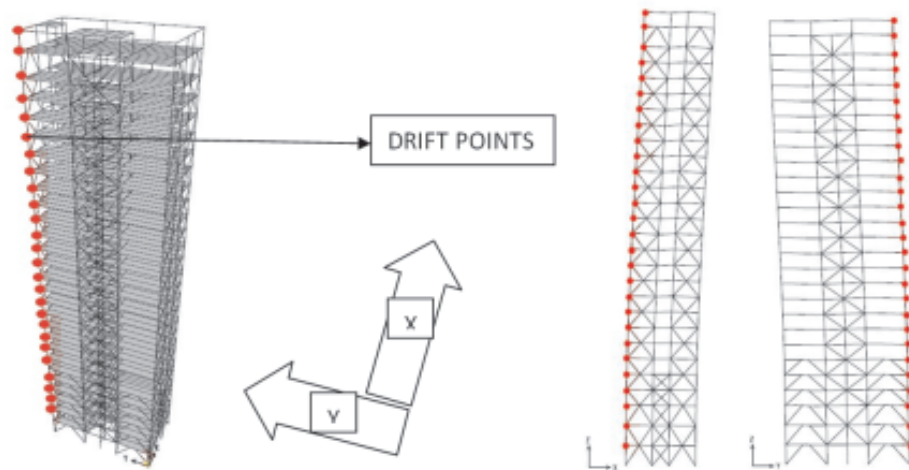


Figure 7.7. Drift Points and Earthquake Directions in the Analysis.

When observing the global performance on the basis of elements of the structure, permanent deformations on the elements are not seen in the normal seismic conditions. Columns, beams and connection elements of the tall building are examined at each step in analysis and it is obtained that there is no loss of capacity through the system.

It is generally accepted that a ductile connection is beneficial for the earthquake performance in a tall building since it dissipates energy in the form of plastic strains and stresses. When observing the moments and rotations in the semi-rigid connections (namely, zero-length elements), they stay within the failure limits in both the tension and moment. Therefore, no dissipation is observed. However, in later sections it will be clear that these connections dissipate significant energy if the earthquake motion is scaled up.

The connection of the secondary beam in the middle of the system is examined Figure 7.8. Connection type is pinned connection in earliest type of the model. We made the connection both semi-rigid and rigid separately. Then we ran two different analyses to see the difference in the behavior of connection type.

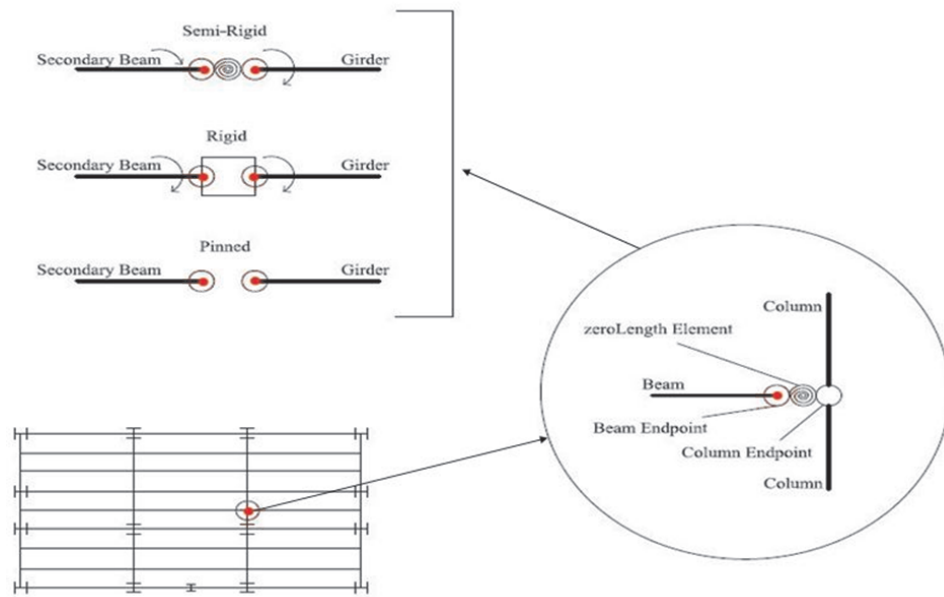


Figure 7.8. Concept of Modified Connection.

The beam bending stiffness in a sway frame is estimated as $6EI/L$, which is approximately 14.87×10^3 kN.m/rad for the IPE330 secondary beam. Here, E is the modulus of elasticity, I is the moment of inertia and L is the length of the member. The IPE330 beam is repetitively used in the building. The rotational stiffness (in the elastic region) of the single plate connection is 1.43×10^3 kN.m, which is about 1/10 for IPE 330 of the beam bending stiffness. On the other hand, the axial stiffness is 100000 kN/m as opposed to 132000 kN/m for semi-rigid connection. This confirms that moment and axial stiffness in semi-rigid connection is smaller than traditionally pinned model.

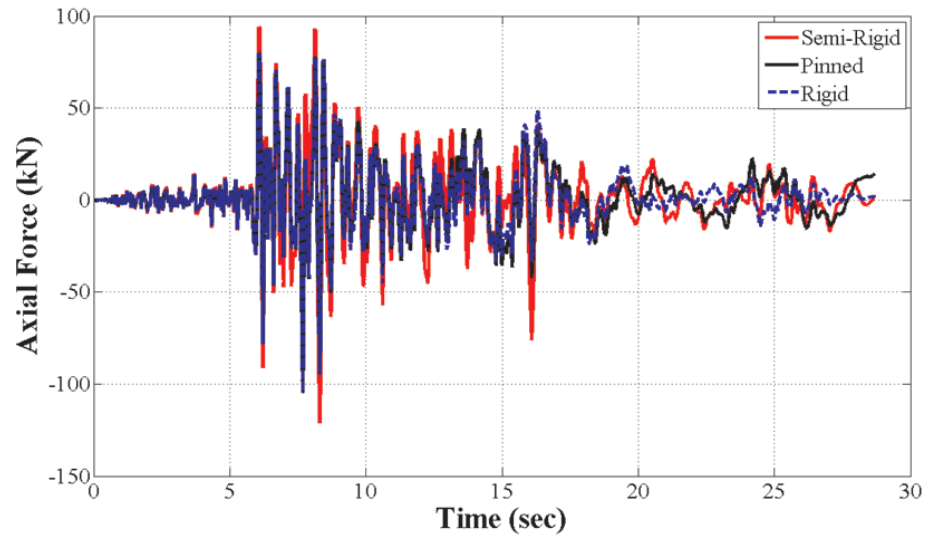


Figure 7.9. Axial Force of Beam End in the Connection Region (y -Direction).

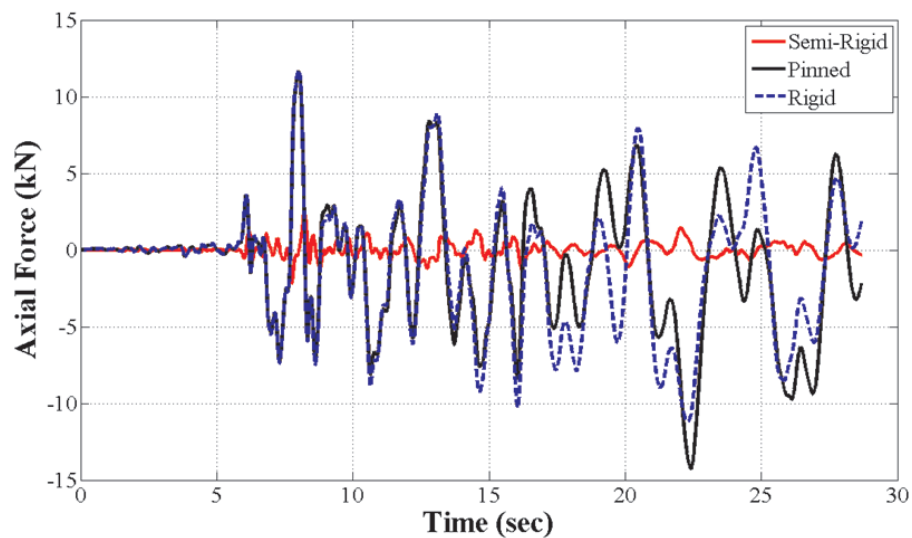


Figure 7.10. Axial Force of Secondary Beam End in the Connection Region (y -Direction).

As seen in Figure 7.9 and Figure 7.10, axial force in the end point of the main beam is higher for semi-rigid connection than rigid and pinned one. Whereas, in the end point of secondary beam, axial force became so decreased for semi-rigid type, although pinned and rigid ones stayed at same level. With semi-rigid connection (zero-length element), the floors carry lowed axial load. However, if the concrete slab on each floor was considered in the model, the result might have been different.

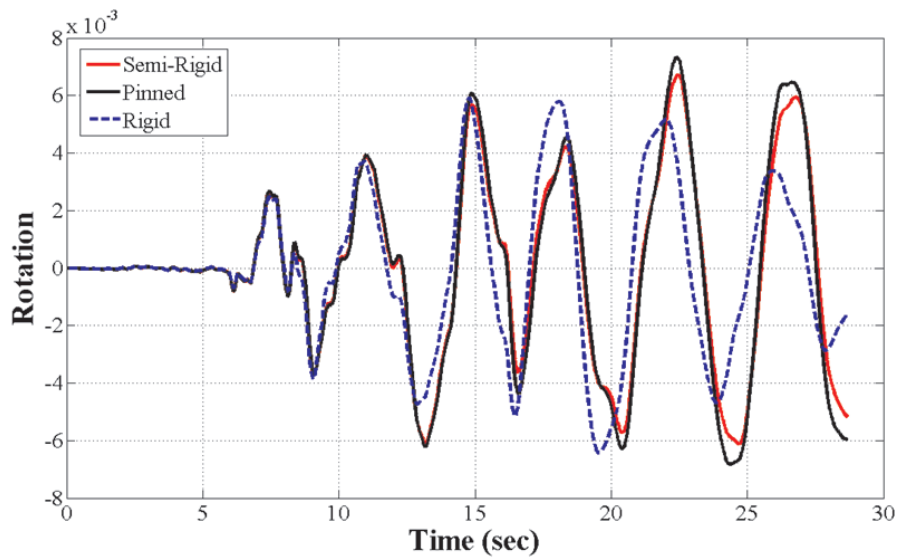


Figure 7.11. Rotation of Beam End in the Connection Region (y -Direction).

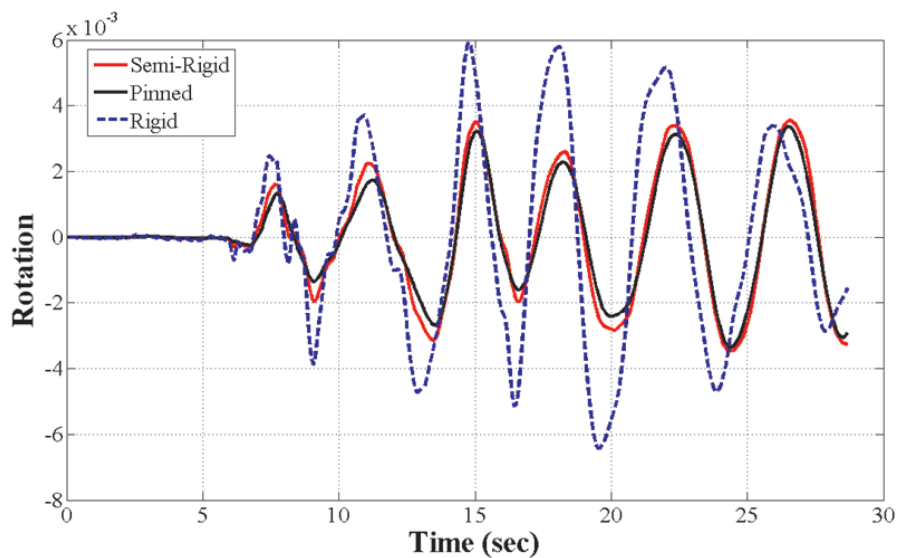


Figure 7.12. Rotation of Secondary Beam End in the Connection Region (y -Direction).

On the other hand, comparison of rotation capacities and rotational behavior of the connection types is given in Figure 7.11 and Figure 7.12. It is clearly seen from the figures that rotation of rigid connection on both sides of zero length elements are same. However, secondary beam end of the connection has lower rotation values than other point of zero length element. The rotation at this end are related to the gravity loads on secondary beams, not to lateral translation of the building. Also, pinned

connection type shows less rotation by comparison with semi-rigid. So, it is proved that it is possible to reduce rotation of a connection with more rotational stiffness.

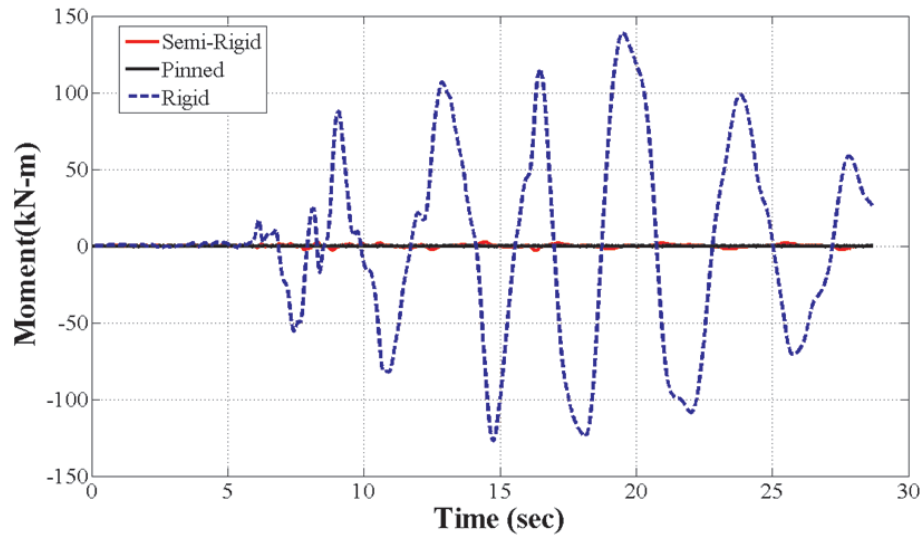


Figure 7.13. Moment of Beam End in the Connection Region (y -Direction).

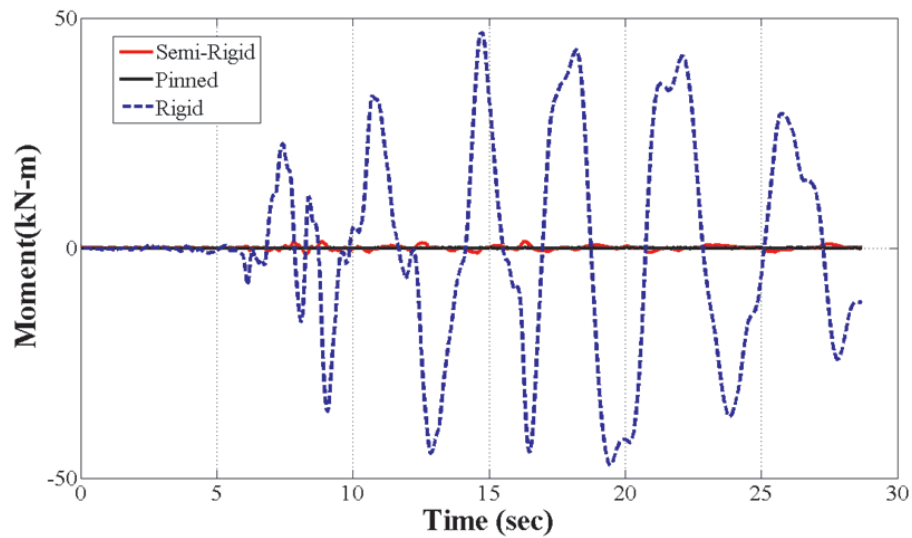


Figure 7.14. Moment of Secondary Beam End in the Connection Region (y -Direction).

Needless to say that rigid connections have more moment capacity. So, it is seen that in Figure 7.13 and Figure 7.14, rigid type of zero length element region shows more moment values. The reason of moment value difference between two ends of zero

length element in rigid connection is the different moment of inertias of the column and secondary beams. To have a better understanding about the behavior of connection types, axial force, moment and rotation diagrams are given in Figure 7.15, Figure 7.16, Figure 7.17, Figure 7.18, Figure 7.19 and Figure 7.20 individually. Lateral loads in semi-rigid connections are absorbed more than rigid connection design. Also due to moment and rotations capacity of semi-rigid connections, it is seen that lower moment and rotation values in comparison rigid and pinned connections.

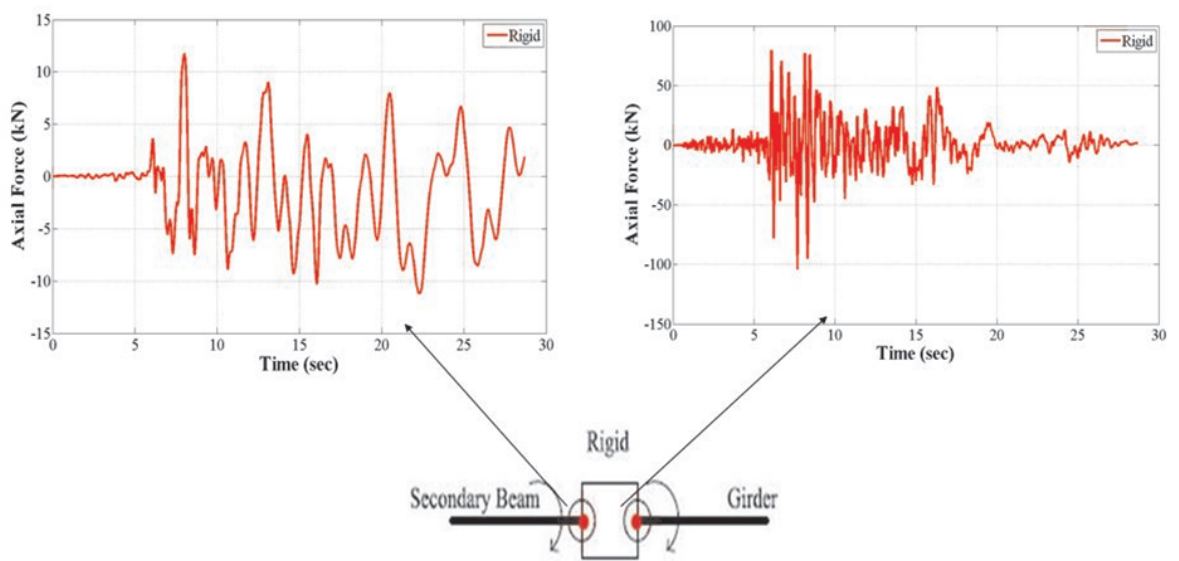


Figure 7.15. Difference of Axial Force in Zero-Length Element Ends (Rigid Connections).

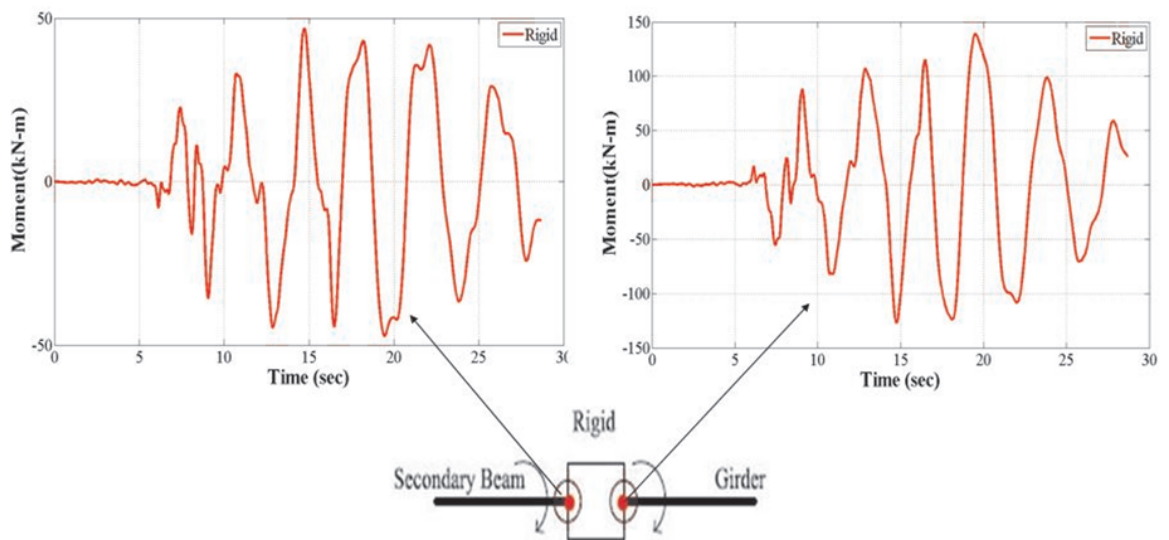


Figure 7.16. Difference of Moments in Zero-Length Element Ends (Rigid Connections).

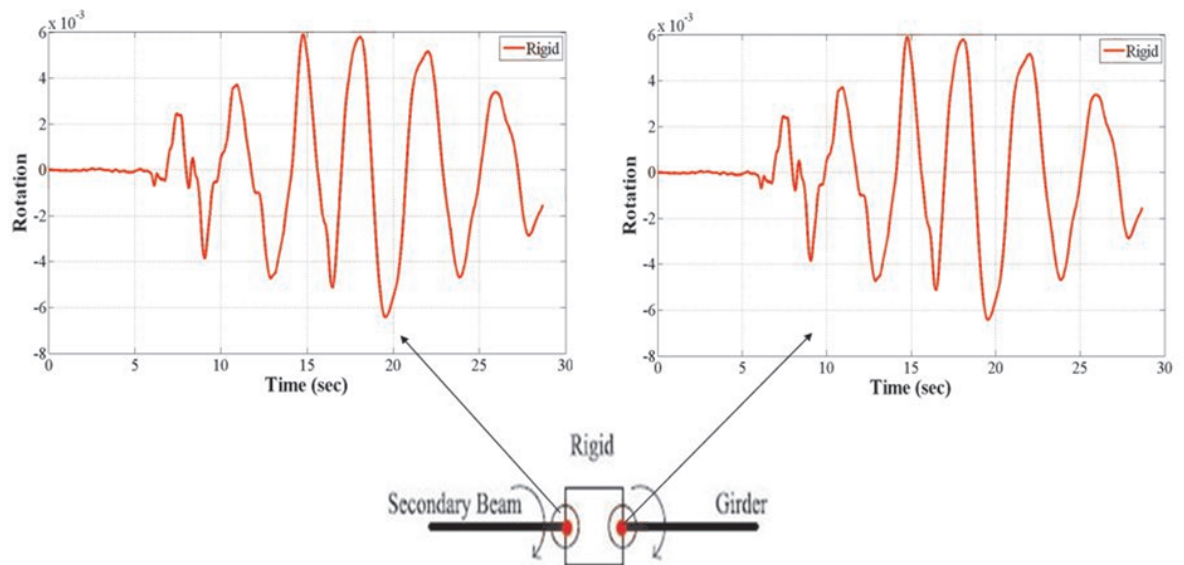


Figure 7.17. Difference of Rotations in Zero-Length Element Ends (Rigid Connections).

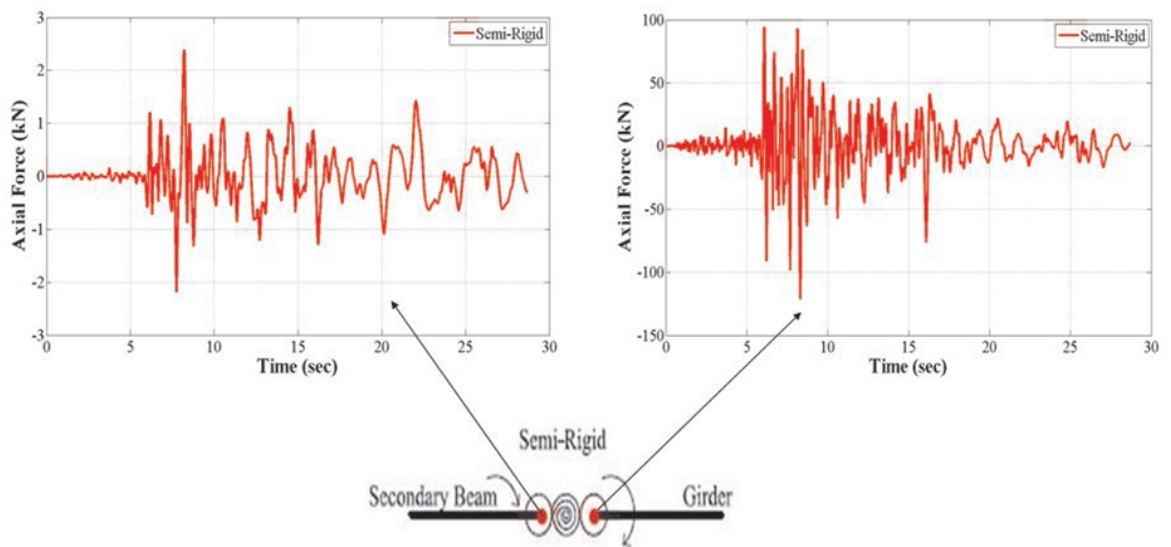


Figure 7.18. Difference of Axial Force in Zero-Length Element Ends (Semi-Rigid Connection).

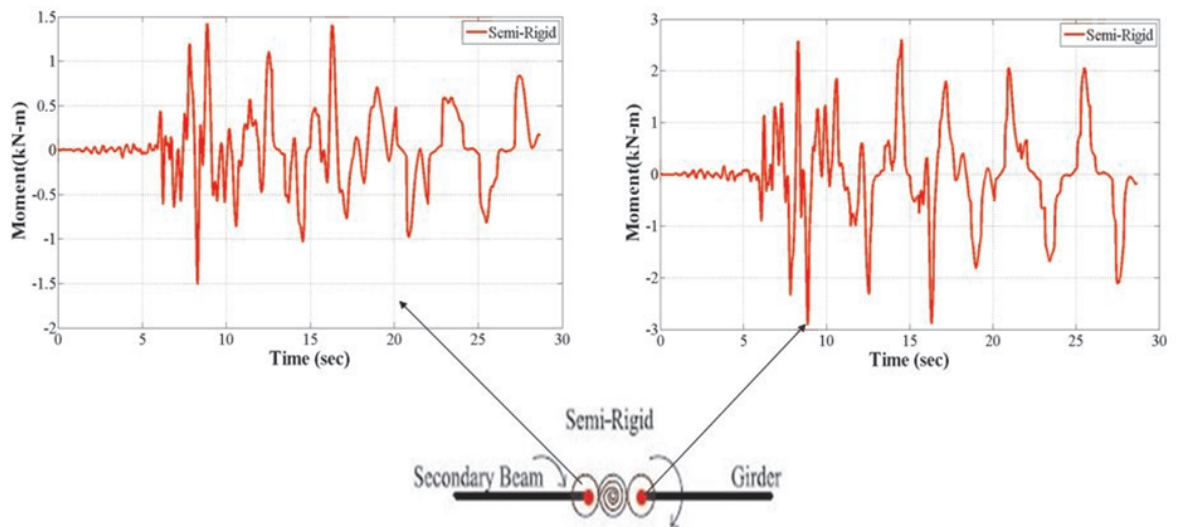


Figure 7.19. Difference of Moments in Zero-Length Element Ends (Semi-Rigid Connection).

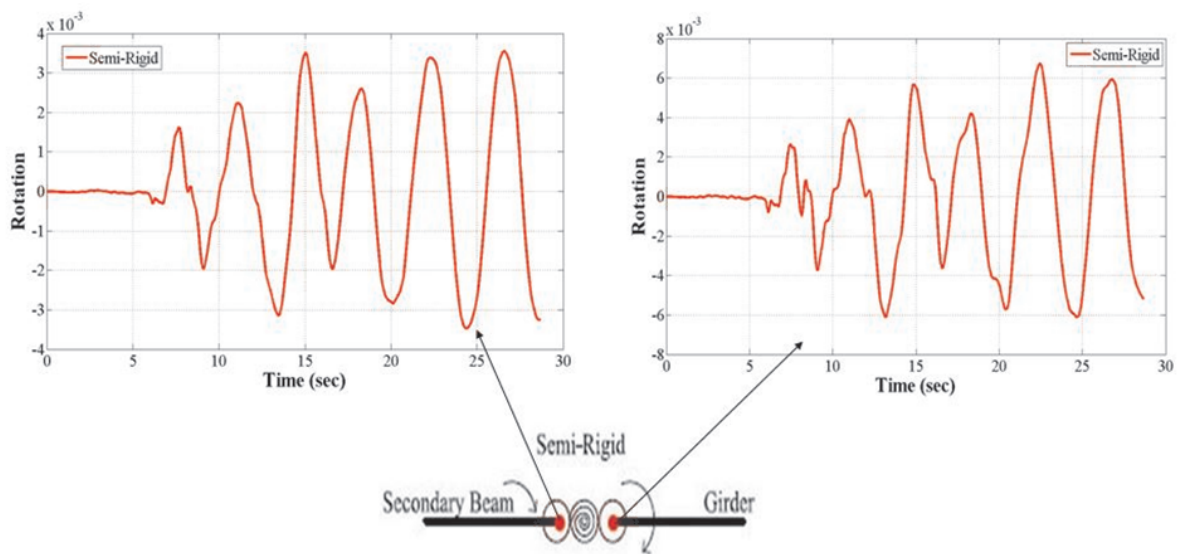


Figure 7.20. Difference of Rotation in Zero-Length Element Ends (Semi-Rigid Connection).

7.3. Performance Assessment of the System Semi-Rigid and Rigid Connected System by Comparison With Pinned Connection System with Increased Earthquake Excitation

The differences in building response get significantly larger when the earthquake excitation is increased by two or three- fold. Under more devastating earthquake conditions, the semi-rigid connections start to yield and such behavior changes the overall building response. Here, the base shear, the top floor displacements, the drift ratios and the connection forces and rotations are shown with increasing earthquake response.

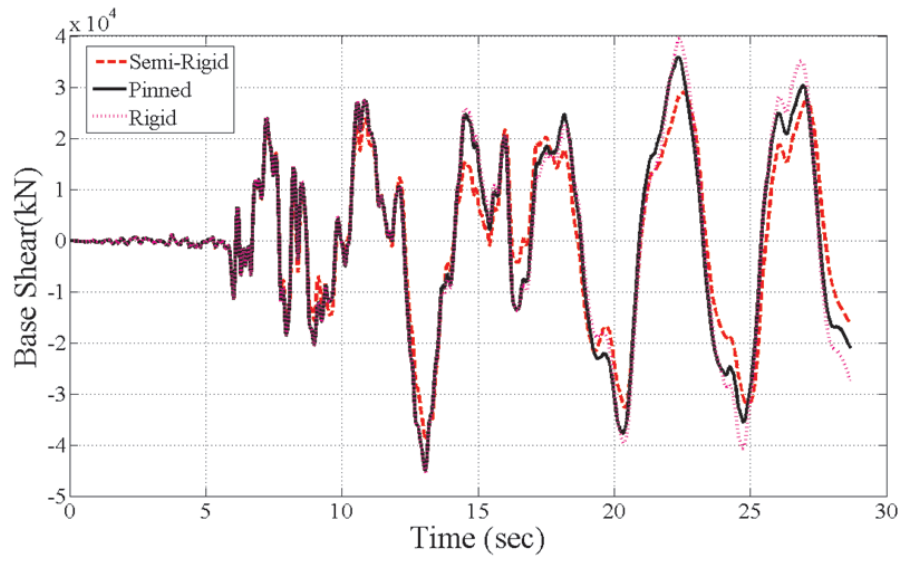


Figure 7.21. Comparison of Base Shear of Pinned, Semi-Rigid and Rigid Connected Systems in X Direction.

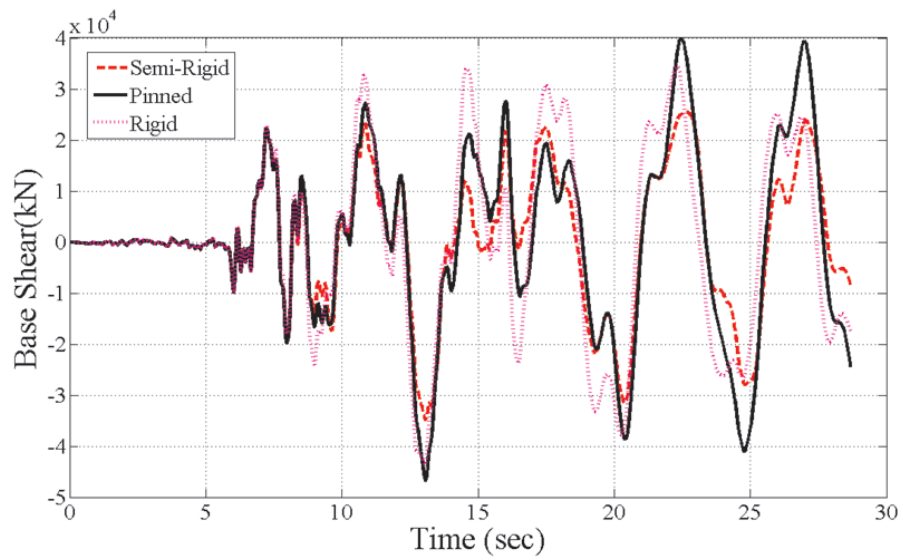


Figure 7.22. Comparison of Base Shear of Pinned, Semi-Rigid and Rigid Connected Systems in Y Direction.

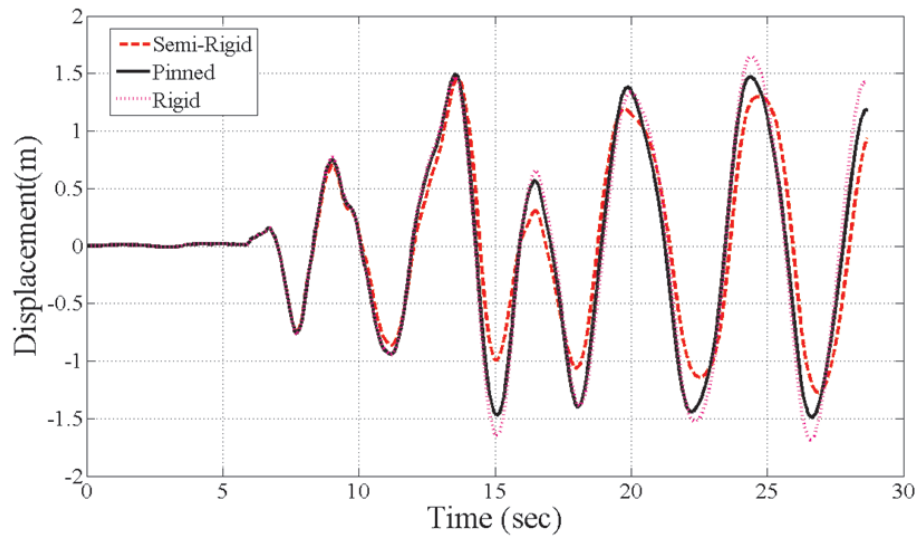


Figure 7.23. Comparison of Top Floor Displacement of Pinned and Rigid Connected Systems in X direction.

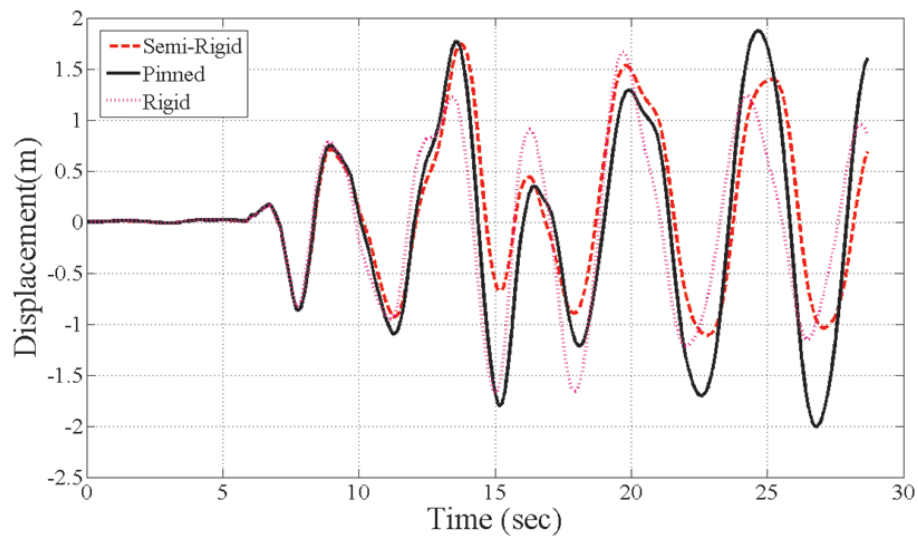


Figure 7.24. Comparison of Top Floor Displacement of Pinned and Rigid Connected Systems in Y direction.

As clearly seen in Figure 7.21 and Figure 7.22 base shears in both directions show tendency to reduce although displacements of top floor trend to increase Figure 7.23 and Figure 7.24. On the other hand, drift ratios reduced in X direction, conversely in Y direction which is seen increasing of drift ratio of semi rigid types.

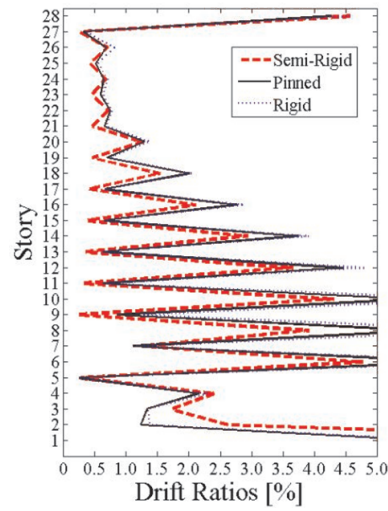


Figure 7.25. Comparison of Drift Ratios of Pinned, Semi Rigid and Rigid Connected Systems in X Direction.

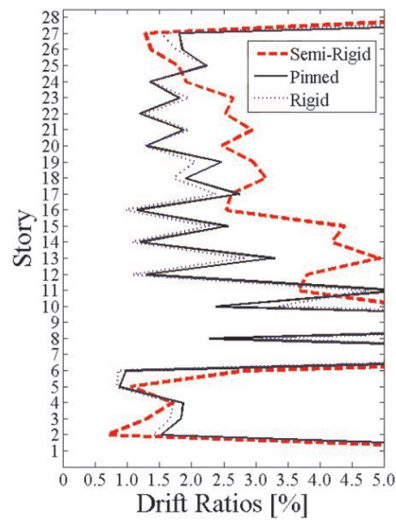


Figure 7.26. Comparison of Drift Ratios of Pinned, Semi Rigid and Rigid Connected Systems in Y Direction.

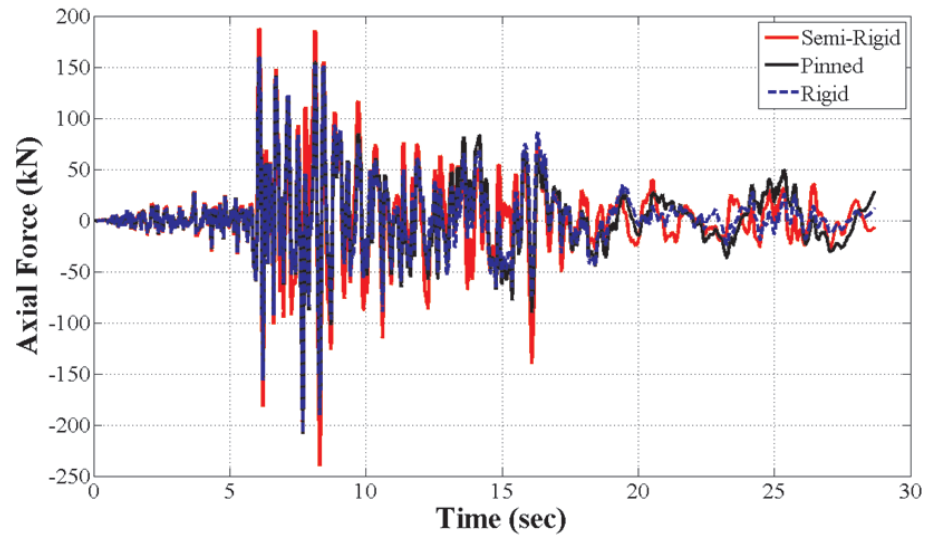


Figure 7.27. Axial Force of Beam in the Connection Region (y -Direction).

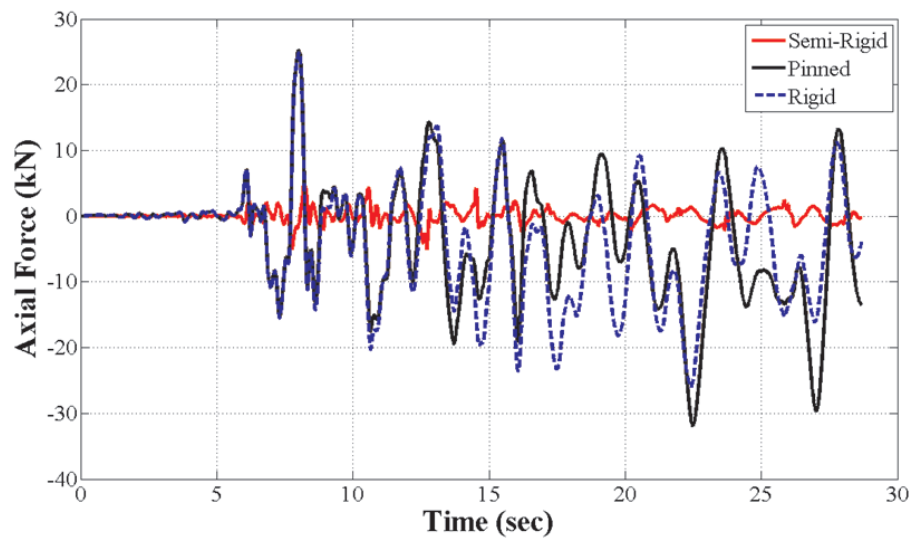


Figure 7.28. Axial Force of Secondary Beam in the Connection Region (y -Direction).

The interesting point is rigid and pinned connection types show same behavior although semi rigid ones trend to increase. Some kind of deformations and yielding of elements could cause that kind of difference. This topic is still under investigation.

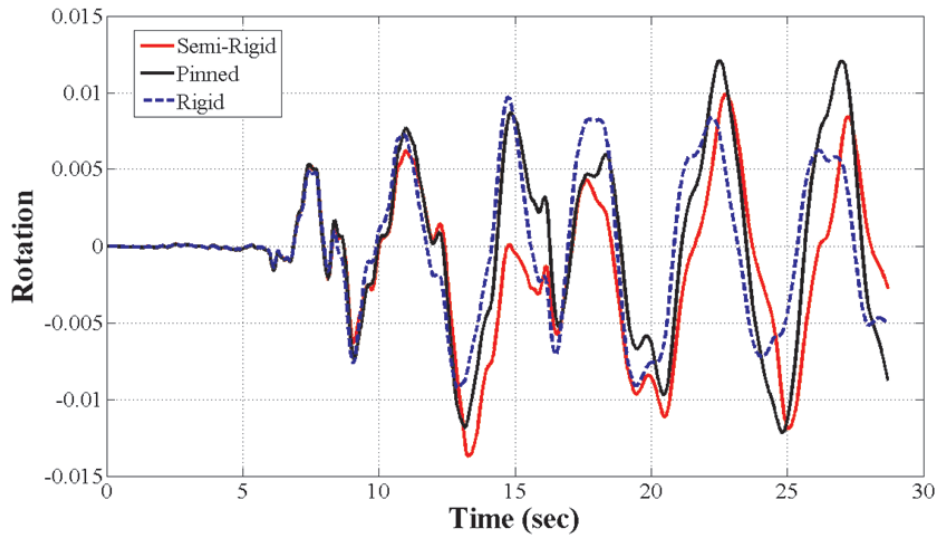


Figure 7.29. Rotation of Beam in the Connection Region (y -Direction).

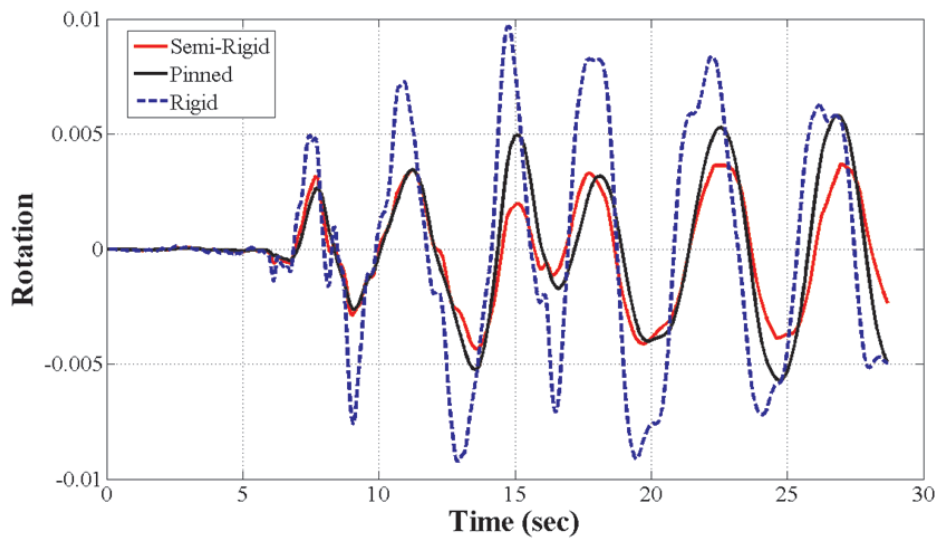


Figure 7.30. Rotation of Secondary Beam in the Connection Region (y -Direction).

Rotation values are really smaller in semi-rigid connections than rigid ones Figure 7.29 and Figure 7.30. Because of the energy absorption, moments, rotations and axial forces show difference between two endpoints of zero length elements.

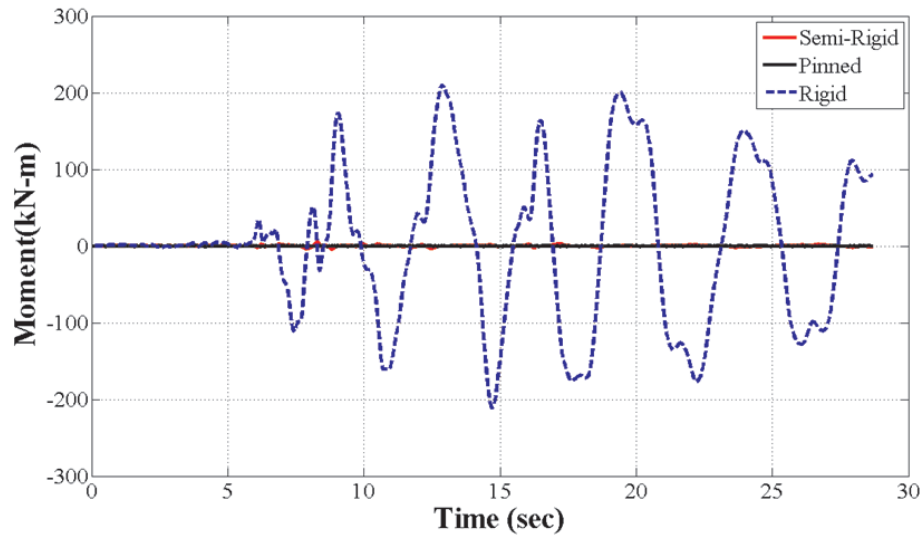


Figure 7.31. Moment of Beam in the Connection Region (y -Direction).

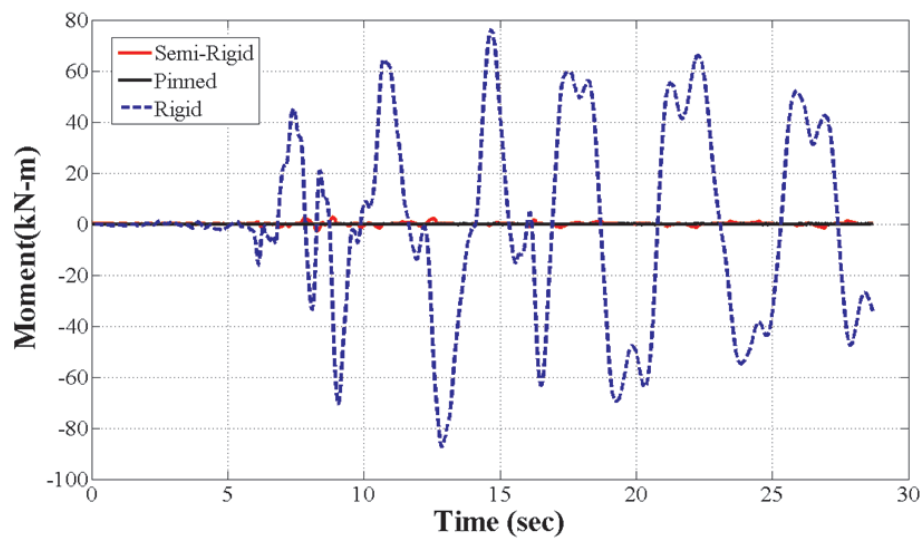


Figure 7.32. Moment of Secondary Beam in the Connection Region (y -Direction).

7.4. Results

- (i) To change of rigidity of connection region do not cause any difference in periods for semi-rigid connections which are obtained from modal analysis. But the more stiffness increase the higher differences are obtained in periods.
- (ii) Changing the rigidity of the connection region of secondary beams does not significantly change the overall lateral response of the building. This is due to

large number of bracings used in the building. In global Y direction, it is seen that there is an increase in base shears although there is a decrease in top floor displacements.

- (iii) Drift ratios do not change in X direction, but they increased in Y direction, especially for semi-rigid connections.
- (iv) The axial forces in the floors with semi-rigid connections are smaller compared to floors with pinned and rigid ones.
- (v) By simulating the semi-rigid behavior of connections with zero length elements, it is possible to trigger additional lateral stiffness in secondary beam regions which initially do not contribute to the lateral resistance of building. In this way, it is possible to avoid overuse of structural elements at design stage.

APPENDIX A: OPENSEES MODEL TCL FILES

SOURCE IN PROCEDURES

```

>>> model
source THESIS.tcl
>>> procedure for reading GM file and converting it to proper format
source ReadSMDfile.tcl
‘‘Uniform arthquake ground motion (uniform acceleration input at all
support nodes)’’
>>> ground-motion direction
set GMdirection “1 1”
>>> ground-motion filenames
set GMfile “Cape-270 Cape-270”
>>> ground-motion scaling factor
set GMfact “1 1”
set Tol 1e-3
set iSupportNode “1 2 3 4 5 6 7 8 9 10 11 12 13 14 15”
Define DISPLAY
>>> the deformed shape is defined in the build file
DisplayModel3D DeformedShape 10 10 10 1200 800
recorder Node -file Tepe-noktasi/node594disp.out -time -node
594 -dof 1 2 3 disp
>>> a window to plot the nodal displacements versus time
recorder plot Tepe-noktasi/node594disp.out Displ-X 800 10 500
500 -columns 1 2
>>> a window to plot the nodal displacements versus time
recorder plot Tepe-noktasi/node594disp.out Displ-Z 800 150 500
500 -columns 1 3
>>> a window to plot the nodal displacements versus time
recorder plot Record/node2265disp.out Displ-Z 500 310 500
500 -columns 1 4

```

```

set up ground-motion-analysis parameters
>>> time-step Dt for lateral analysis
set DtAnalysis [expr 0.005 * $sec]
>>> maximum duration of ground-motion analysis
- should be 50*$sec
set TmaxAnalysis [expr 28.68 * $sec] set up analysis parameters
>>> constraints Handler, DOFnumberer, system-of equations,
convergence Test, solution Algorithm, integrator
source LibAnalysisDynamicParameters.tcl
define & apply damping
>>> RAYLEIGH damping parameters, Where to put
M/K-prop damping, switches
(http://opensees.berkeley.edu/OpenSees/manuals/usermanual/1099.htm)
>>>  $D = \alpha M * M + \beta_{curr} * K_{current} +$ 
 $\beta_{comm} * K_{lastCommit} + \beta_{init} * K_{initial}$ 
>>> damping ratio
set xDamp 0.05
set MpropSwitch 1.0
set KcurrSwitch 0.0
set KcommSwitch 1.0
set KinitSwitch 0.0
>>> mode 1 set nEigenI 1
>>> mode 3 set nEigenJ 3
>>> eigenvalue analysis for nEigen J modes
puts $lambdaI
puts $lambdaJ
set omegaI [expr pow($lambdaI,0.5)]
set omegaJ [expr pow($lambdaJ,0.5)]
>>> M-prop. damping D = alphaM*M
set alphaM [expr $MpropSwitch* $xDamp*
(2*$omegaI* $omegaJ)/ ($omegaI+$omegaJ)] >>> current-K
+beat Kcurr * KCurrent

```

```

set betaKcurr [expr $KcurrSwitch*2.*$xDamp/
($omegaI+$omegaJ)]
>>> last-committed K +beta Kcomm * KlastCommitt
set betaKcomm [expr $KcommSwitch*2.*$xDamp/
($omegaI+$omegaJ)]
>>> initial-K + beatKinit*Kini
set betaKinit [expr $KinitSwitch*2 . *$xDamp /
($omegaI+$omegaJ)]
>>> RAYLEIGH damping
rayleigh $alphaM $betaKcurr $betaKinit $betaKcomm
>>> perform Dynamic Ground-Motion Analysis
>>> The following commands are unique to the
Uniform Earthquake excitation
>>> for uniformSupport excitation
set IDloadTag 400
>>> Uniform EXCITATION: acceleration input
foreach GMdirection $GMdirection GMfile $GMfile
GMfact $GMfact {
incr IDloadTag
set inFile $GMdir/$GMfile.at2
>>> set variable holding new filename
(PEER files have .at2/dt2 extension)
set outFile $GMdir/$GMfile.g3
ReadSMD File $inFile $outFile dt
>>> data in input file is in g Unifts -
ACCELERATION TH
set GMfatt [expr $g*$GMfact]
>>> time series information
set AccelSeries "Series -dt $dt -filePath
$outFile -factor $GMfatt"
>>> create Unifform excitation}
pattern UniformExcitation $IDloadTag

```

```

$GMdirection -accel $AccelSeries
set Nsteps [expr int($TmaxAnalysis/$DtAnalysis)]
>>> actually perform analysis returns ok=0
if analysis was successful set ok [analyze $Nsteps $DtAnalysis]
>>> analysis was not successful.
if {$ok != 0} {
>>> change some analysis parameters to
achieve convergence
>>> performance is slower inside this loop
>>> Time-controlled analysis
set ok 0
set controlTime [getTime]
while {$controlTime < $TmaxAnalysis && $ok == 0} {
set controlTime [getTime]
set ok [analyze 1 $DtAnalysis]
if {$ok != 0} {
puts "Trying Newton with Initial Tangent .."
test NormDispIncr $Tol 1000 0
algorithm Newton -initial
set ok [analyze 1 $DtAnalysis]
test $testTypeDynamic $TolDynamic $maxNumIterDynamic 0
algorithm $algorithmTypeDynamic}
if {$ok != 0} {
puts "Trying Broyden .."
algorithm Broyden 8
set ok [analyze 1 $DtAnalysis]
algorithm $algorithmTypeDynamic}
if {$ok != 0} {
puts "Trying NewtonWithLineSearch .."
algorithm NewtonLineSearch .8
set ok [analyze 1 $DtAnalysis]
algorithm $algorithmTypeDynamic

```

```
}}}  
>>> end if ok !0  
puts "Ground Motion Done. End Time: [getTime]"  
THEESIS.TCL  
>>> Create ModelBuilder  
model BasicBuilder -ndm 3 -ndf 6  
set dataDir Data  
file mkdir $dataDir  
set GMdir "GMfiles/"  
source LibUnits.tcl  
source DisplayPlane.tcl  
source DisplayModel3D.tcl  
source Wsection.tcl  
source NodeCoord.tcl  
>>> Define geometry  
source SPConstraint.tcl  
>>> Define Single Point Constraints  
source NodeMass.tcl  
>>> Define nodal masses  
source MPConstraint.tcl  
>>> Define Multi Point Constraints  
source Materials.tcl  
>>> Define material(s)  
source kadir-Sections.tcl  
>>> Define section(s)  
source GeoTran.tcl  
>>> Define geometric transformation(s)  
source Elements.tcl  
>>> Define element(s)  
source TimeSeries.tcl  
>>> Define time series  
initialize
```

```

>>> Get Initial Stiffness
source LoadPattern-1.tcl
>>> Define load pattern
source LoadPattern-2.tcl
>>> Define load pattern
source Recorder-nodeReactions.tcl
>>> Define recorder(s)
source Recorder-Drift.tcl
>>> Define recorder(s)
source Recorder-StressStrain.tcl
>>> Define recorder(s)
source Recorder-zeroLength.tcl
>>> Define recorder(s) loadConst -time 0.0
wipeAnalysis
puts "Model Built"
DISPLAY MODEL 3D.TCL
proc DisplayModel3D { {ShapeType nill}
{dAmp 5} {xLoc 0} {yLoc 0} {xPixels 0} {yPixels 0}
{nEigen 1} } {
>>> load global unit variable
global TunitTXT
>>> read global values for screen resolution
global ScreenResolutionX ScreenResolutionY
>>> set blank if it has not been
defined previously.
if { [info exists TunitTXT] != 1 } {set TunitTXT ""}
>>> set default if it has not been defined previously.
if { [info exists ScreenResolutionX] != 1 }
{set ScreenResolutionX 1024}
>>> set default if it has not been defined previously.
if { [info exists ScreenResolutionY] != 1 }
{set ScreenResolutionY 768}

```

```

if { $xPixels == 0 } {
set xPixels [expr int($ScreenResolutionX/2)]
set yPixels [expr int($ScreenResolutionY/2)]
set xLoc 10
set yLoc 10}
if { $ShapeType == "nil" } {
puts "" puts "" puts "—————"
puts "View the Model? (N)odes, (D)eformedShape,
anyMode(1),(2),
(>>>). Press enter for NO."
gets stdin answer
if {[length $answer] > 0 } {
if { $answer != "N" & $answer != "n" } {
puts "Modify View Scaling Factor=$dAmp? Type factor,
or press enter for NO."
gets stdin answerdAmp
if {[length $answerdAmp] > 0 } {
set dAmp $answerdAmp}}
} elseif {[string index $answer 0] "D"}
set ShapeType DeformedShape}
else {
set ShapeType ModeShape
set nEigen $answer}}
else {
return}}
if { $ShapeType == "ModeShape" } {
>>> define constant
set PI 3.14
>>> period
set Tperiod [expr 2*$PI/$omega]
set fmt1 "Mode Shape, Mode=%.1i Period=%.3f %s "
set windowTitle [format $fmt1 $nEigen $Tperiod

```

```

$TunitTXT ]
} elseif {$ShapeType == "NodeNumbers" } {
set windowTitle "Node Numbers"
} elseif {$ShapeType == "DeformedShape" } {
set windowTitle0 "Deformed Shape "
if {$ShapeType == "DeformedShape" } {
set xPixels [expr int($xPixels/2)]
set yPixels [expr int($yPixels/2)]
set xLoc1 [expr $xLoc+$xPixels]
set yLoc1 [expr $yLoc+$yPixels]
set planeTXT "-Plane"
set viewPlane XY
set windowTitle $windowTitle0$viewPlane$planeTXT
recorder display $windowTitle $xLoc1 $yLoc
$xPixels $yPixels -wipe
DisplayPlane $ShapeType $dAmp $viewPlane
set viewPlane ZY
set windowTitle $windowTitle0$viewPlane$planeTXT
recorder display $windowTitle $xLoc $yLoc
$xPixels $yPixels -wipe
DisplayPlane $ShapeType $dAmp $viewPlane
set viewPlane ZX
set windowTitle $windowTitle0$viewPlane$planeTXT
recorder display $windowTitle $xLoc $yLoc1
$xPixels $yPixels -wipe
DisplayPlane $ShapeType $dAmp $viewPlane
set viewPlane 3D
set windowTitle $windowTitle0$viewPlane
recorder display $windowTitle $xLoc1 $yLoc1
$xPixels $yPixels -wipe DisplayPlane $ShapeType
$dAmp $viewPlane
} else {

```

```

recorder display $windowTitle $xLoc $yLoc
$xPixels $yPixels nowipe
set viewPlane XY
DisplayPlane $ShapeType $dAmp $viewPlane $nEigen 1
set viewPlane ZY
DisplayPlane $ShapeType $dAmp $viewPlane $nEigen 2
set viewPlane ZX
DisplayPlane $ShapeType $dAmp $viewPlane $nEigen 3
set viewPlane 3D
DisplayPlane $ShapeType $dAmp $viewPlane $nEigen 4}}
DISPLAY PLANE.TCL
proc DisplayPlane {ShapeType dAmp viewPlane
{nEigen 0} {quadrant 0}} {
  >>> view bounds in global coords -
  will add padding on the sides
  set Xo 0
  set Yo 0
  set Zo 0
  >>> viewPlane local-x axis in global coordinates
  set uLocal [string index $viewPlane 0]
  >>> viewPlane local-y axis in global coordinates
  set vLocal [string index $viewPlane 1]
  if {$viewPlane == "3D" } {
    set uMin $Zmin+$Xmin
    set uMax $Zmax+$Xmax
    set vMin $Ymin
    set vMax $Ymax
    set wMin -10000
    set wMax 10000
    >>> dirn defining up direction of view plane
    vup 0 1 0
  } else {

```

```

set keyAxisMin "X $Xmin Y $Ymin Z $Zmin"
set keyAxisMax "X $Xmax Y $Ymax Z $Zmax"
set axisU [string index $viewPlane 0]
set axisV [string index $viewPlane 1]
set uMin [string map $keyAxisMin $axisU]
set uMax [string map $keyAxisMax $axisU]
set vMin [string map $keyAxisMin $axisV]
set vMax [string map $keyAxisMax $axisV]
if {$viewPlane == "YZ" —— $viewPlane == "ZY" } {
set wMin $Xmin
set wMax $Xmax
} elseif {$viewPlane == "XY" —— $viewPlane
== "YX" } {
set wMin $Zmin
set wMax $Zmax
} elseif {$viewPlane == "XZ" —— $viewPlane
== "ZX" } {
set wMin $Ymin
set wMax $Ymax
} else {>>> define constant
return -1}}
>>> make windows width or height not zero
when the Max and Min values of a coordinate are same
set epsilon 1e-3
set uWide [expr $uMax - $uMin+$epsilon]
set vWide [expr $vMax - $vMin+$epsilon]
set uSide [expr 0.25*$uWide]
set vSide [expr 0.25*$vWide]
set uMin [expr $uMin - $uSide]
set uMax [expr $uMax + $uSide]
set vMin [expr $vMin - $vSide]
>>> pad a little more on top, because of

```

window title

set vMax [expr \$vMax + 2*\$vSide]

set uWide [expr \$uMax - \$uMin+\$epsilon]

set vWide [expr \$vMax - \$vMin+\$epsilon]

set uMid [expr (\$uMin+\$uMax)/2]

set vMid [expr (\$vMin+\$vMax)/2]

>>> keep the following general, as change

the X and Y and Z for each view plane >>> next three

commmands define viewing system, all values in global coords

>>> point on the view plane in global coord,

center of local viewing system

vrp \$Xo \$Yo \$Zo

if {\$vLocal == "X"} {

>>> dirn defining up direction of view plane

vup 1 0 0

} elseif {\$vLocal == "Y"} {

>>> dirn defining up direction of view plane

vup 0 1 0

} elseif {\$vLocal == "Z"} {

>>> dirn defining up direction of view plane

vup 0 0 1 }

if {\$viewPlane == "YZ"} {

>>> direction of outward normal to view plane

vpn 1 0 0

>>> eye location in local coord sys defined

by viewing system

prp 10000. \$uMid \$vMid

>>> distance to front and back clipping planes

from eye

plane 10000 -10000

} elseif {\$viewPlane == "ZY"} {

>>> direction of outward normal to view plane

vpn -1 0 0

*>>> eye location in local coord sys defined by
viewing system*

prp -10000. \$vMid \$uMid

>>> distance to front and back clipping planes from eye

plane 10000 -10000

} elseif {\$viewPlane == "XY" } {

>>> direction of outward normal to view plane

vpn 0 0 1

*>>> eye location in local coord sys defined by
viewing system*

prp \$uMid \$vMid 10000

>>> distance to front and back clipping planes from eye

plane 10000 -10000

} elseif {\$viewPlane == "YX" } {

>>> direction of outward normal to view plane

vpn 0 0 -1

*>>> eye location in local coord sys defined by
viewing system*

prp \$uMid \$vMid -10000

>>> distance to front and back clipping planes from eye

plane 10000 -10000

} elseif {\$viewPlane == "XZ" } {

>>> direction of outward normal to view plane

vpn 0 -1 0

*>>> eye location in local coord sys defined by
viewing system*

prp \$uMid -10000 \$vMid

>>> distance to front and back clipping planes from eye

plane 10000 -10000

} elseif {\$viewPlane == "ZX" } {

>>> direction of outward normal to view plane

```

vpn 0 1 0
>>> eye location in local coord sys defined by
viewing system

prp $uMid 10000 $vMid
>>> distance to front and back clipping planes from eye
plane 10000 -10000
} elseif {$viewPlane == "3D" } {
>>> direction of outward normal to view plane

vpn 1 0.25 1.25
>>> eye location in local coord sys defined by
viewing system

prp -100 $vMid 10000
>>> distance to front and back clipping planes from eye
plane 10000 -10000
} else {
return -1}
>>> next three commands define view, all values
in local coord system

if {$viewPlane == "3D" } {
viewWindow [expr $uMin-$uWide/4] [expr $vMax]
} else {
viewWindow $uMin $uMax $vMin $vMax}
>>> projection mode, 0:perspective, 1: parallel
projection 1
>>> fill mode needed only for solid elements
fill 1
if {$quadrant == 0} {
>>> area of window that will be drawn into
(uMin, uMax, vMin, vMax)
port -1 1 -1 1
} elseif {$quadrant == 1} {
>>> area of window that will be drawn into

```

```

(uMin, uMax, vMin, vMax)
port 0 1 0 1
} elseif {$Squadrant == 2} {
>>> area of window that will be drawn into
(uMin, uMax, vMin, vMax)
port -1 0 0 1
} elseif {$Squadrant == 3} {
>>> area of window that will be drawn into
(uMin, uMax, vMin, vMax)
port -1 0 -1 0
} elseif {$Squadrant == 4} {
>>> area of window that will be drawn into
(uMin, uMax, vMin, vMax)
port 0 1 -1 0 }
if {$ShapeType == "ModeShape" } {
>>> display mode shape for mode $nEigen
display -$nEigen 0 [expr 5.*$dAmp]
} elseif {$ShapeType == "NodeNumbers" } {
>>> display node numbers
display 1 -1 0
} elseif {$ShapeType == "DeformedShape" } {
>>> display deformed shape the 2 makes the nodes small
display 1 2 $dAmp }

```

DYNAMIC ANALYSIS PARAMETERS.TCL

```

>>> CONSTRAINTS handler
>>> Plain Constraints – Removes constrained
degrees of freedom from the system of equations
>>> Lagrange Multipliers – Uses the
method of Lagrange multipliers to enforce constraints
>>> Penalty Method – Uses penalty
numbers to enforce constraints
>>> Transformation Method – Performs

```

a condensation of constrained degrees of freedom

variable constraints TypeDynamic Transformation

constraints \$constraintsTypeDynamic

»»» *DOF NUMBERER*

»»» *Plain* – Uses the numbering provided

by the user

»»» *RCM* – Renumbers the DOF to minimize

the matrix band-width using the Reverse Cuthill-McKee algorithm

variable numbererTypeDynamic RCM

numberer \$numbererTypeDynamic

»»» *SYSTEM*

»»» *ProfileSPD* – Direct profile

solver for symmetric positive definite matrices

»»» *BandGeneral* – Direct solver

for banded unsymmetric matrices

»»» *BandSPD* – Direct solver for

banded symmetric positive definite matrices

»»» *SparseGeneral* – Direct solver for unsymmetric sparse
matrices (-piv option)

»»» *SparseSPD* – Direct solver

for symmetric sparse matrices

»»» *UmfPack* – Direct UmfPack

solver for unsymmetric matrices

variable system TypeDynamic UmfPack

system \$systemTypeDynamic

»»» *TEST*

»»» *NormUnbalance* – Specifies a

tolerance on the norm of the unbalanced load at the current iteration

»»» *NormDispIncr* – Specifies a

tolerance on the norm of the displacement increments at the current iteration

»»» *EnergyIncr* – Specifies a tolerance

on the inner product of the unbalanced load and displacement

increments at the current iteration

»»» *RelativeNormUnbalance*

»»» *RelativeNormDispIncr*

»»» *RelativeEnergyIncr*

»»» *Convergence Test: tolerance*

variable TolDynamic 1.e-3

»»» *Convergence Test: maximum number of*

iterations that will be performed before “failure to converge” is returned

variable maxNumIterDynamic 100

»»» *Convergence Test: flag used to print*

information on

variable printFlagDynamic 1

»»» *1: print information on each step*

convergence (optional)

»»» *Convergence-test type*

variable testTypeDynamic EnergyIncr

test \$testTypeDynamic \$TolDynamic

\$maxNumIterDynamic \$printFlagDynamic

»»» *for improved-convergence procedure:*

variable maxNumIterConvergeDynamic 2000

variable printFlagConvergeDynamic 0

»»» *Solution ALGORITHM*

»»» *Linear – Uses the solution at the*

first iteration and continues

»»» *Newton – Uses the tangent at the*

current iteration to iterate to convergence

»»» *ModifiedNewton – Uses the tangent*

at the first iteration to iterate to convergence

»»» *NewtonLineSearch –*

»»» *KrylovNewton –*

»»» *BFGS –*

»»» *Broyden –*

variable algorithmTypeDynamic ModifiedNewton

algorithm \$algorithmTypeDynamic

»»» *Static INTEGRATOR*

»»» *LoadControl* – Specifies the incremental load factor to be applied to the loads in the domain

»»» *DisplacementControl* – Specifies the incremental displacement at a specified DOF in the domain

»»» *Minimum Unbalanced Displacement*

Norm – Specifies the incremental load factor such that the residual displacement norm is minimized

»»» *Arc Length* – Specifies the incremental arc-length of the load-displacement path

»»» *Transient INTEGRATOR:* – determine the next time step for an analysis including inertial effects

»»» *Newmark* – The two parameter time-stepping method developed by Newmark

»»» *HHT* – The three parameter Hilbert-Hughes-Taylor time-stepping method

»»» *Central Difference* – Approximates velocity and acceleration by centered finite differences of displacement

»»» *Newmark-integrator gamma parameter (also HHT)*

variable NewmarkGamma 0.5

»»» *Newmark-integrator beta parameter*

variable NewmarkBeta 0.25

variable integratorTypeDynamic Newmark

integrator \$integratorTypeDynamic

\$NewmarkGamma \$NewmarkBeta

»»» *ANALYSIS*

»»» *Static Analysis* – solves the $KU=R$ problem, without the mass or damping matrices.

»»» *Transient Analysis* – solves the

time-dependent analysis. The time step in this type of analysis is constant. The time step in the output is also constant.

»»» variableTransient Analysis – performs the same analysis type as the Transient Analysis object. The time step, however, is variable. This method is used when »»» there are convergence problems with the Transient Analysis object at a peak or when the time step is too small. The time step in the output is also variable.

variable analysisTypeDynamic Transient

analysis \$analysisTypeDynamic

MATERIALS.TCL

»»» Material “1-st44”: matTag Fy E

b a1 a2 a3 a4

uniaxialMaterial Steel01 1 +2.750000E+008

+2.000000E+011 +5.000000E-002

+0.000000E+000

+1.000000E+000 +0.000000E+000 +1.000000E+000

»»» Material “2-st52”: matTag Fy E

b a1 a2 a3 a4

uniaxialMaterial Steel01 2 +3.550000E+009

+2.000000E+011 +5.000000E-002

+0.000000E+000

+1.000000E+000 +0.000000E+000 +1.000000E+000

»»» Material “3-st64”: matTag Fy E

b a1 a2 a3 a4

uniaxialMaterial Steel01 3 +4.600000E+008

+2.000000E+011 +5.000000E-002

+0.000000E+000

+1.000000E+000 +0.000000E+000 +1.000000E+000

READ SMD FILE.TCL

»»» read gm input format

»»» A procedure which parses a ground motion

record from the PEER

»»» *strong motion database by finding dt in the record header, then*

»»» *echoing data values to the output file.*

»»» *Formal arguments*

»»» *inFilename – file which contains PEER strong motion record*

»»» *outFilename – file to be written in format G3 can read*

»»» *dt – time step determined from file header*

```

proc ReadSMDFile {inFilename outFilename dt} {
  upvar $dt DT
  if [catch {open $inFilename r} inFileID] {
    puts stderr "Cannot open $inFilename for reading"
  } else {
    set outFileID [open $outFilename w]
    set flag 0
    if {[llength $line] == 0} {
      »»» Blank line -j do nothing
      continue
    } elseif {$flag == 1} {
      »»» Echo ground motion values to output file
      puts $outFileID $line
    } else {
      »»» Search header lines for dt
      foreach word [split $line] {
        »»» Read in the time step
        if {$flag == 1} {
          set DT $word
          break}
        »»» Find the desired token and set the flag
        if {[string match $word "DT="] == 1} {set flag 1}}}}

```

```

close $outFileID
close $inFileID}}
MODAL ANALYSIS.TCL
>>> Written by "Kadir Topcu"
>>> set num $
>>> Plaese set your number of Frequency needs
source LibAnalysisDynamicParameters.tcl
>>> Plaese set your number of Frequency needs
set numModes 90
file mkdir ModalAnalysis
for {set i 1} {$i=$numModes} {incr i} {
>>> recorder Node -file ModalAnalysis/mode$i.out
-nodeRange xxxx -dof 1 "eigen $i"
recorder Node -file Modal Analysis/ Dir1mode$i.out -
node 5 120 235 352 470 591 695
770 848 924 997 1073 1146 1222 1295 1371
1444 1520 1593 1669 1742 1818 1891
1967 2040 2116 2189 2265
-dof 1 "eigen $i"
recorder Node -file Modal Analysis /Dir2mode$i. out -node
5 120 235 352 470 591
695 770 848 924 997 1073 1146 1222 1295 1371
1444 1520 1593 1669 1742 1818 1891 1967 2040 2116 2189
2265 -dof 2 "eigen $i" }
set OMEGA [eigen $numModes]
set pi 3.14159265359
foreach OMEGA $OMEGA {
lappend omega [expr pow($OMEGA,0.5)]
lappend f [expr pow($OMEGA,0.5)/(2*$pi)]
lappend T [expr (2*$pi)/pow($OMEGA,0.5)]}
set s "PeriodsOfStructure.txt"
set PeriodsOfStructure [open ModalAnalysis/$s "w"]

```

```
foreach t $T {  
  puts $PeriodsOfStructure "Period=$t"  
  close $PeriodsOfStructure  
  set DtAnalysis [expr 0.01*$sec]  
  set TmaxAnalysis [expr 10.*$sec]  
  source LibAnalysisDynamicParameters.tcl  
  set Nsteps [expr int($TmaxAnalysis/$DtAnalysis)]  
  set ok [analyze $Nsteps $DtAnalysis]
```

REFERENCES

1. Aksoylar, N.D., S. Elnashai Amr, A.Z. Öztürk, 2013, “Yarı Rijit Birleşimli Çelik Çerçevelerin Kat Öteleme ve Birleşim Dönmelerinin İncelenmesi”, *İnşaat Mühendisler Odası Teknik Dergi*, Vol. 398, pp. 6351-6377.
2. Anil, K.C., 1995, *Dynamics of Structures- the Theory and Applications to Earthquake Engineering*, University of California at Berkeley.
3. Arnold, C., 2006, “Depremi Yapılar Üzerindeki Etkisi”, *Fema 454 Designing for Earthquakes A Manual for Architects*, pp. 111-137.
4. Asgarian, B., S. Moradi, 2011, “Seismic Response of Steel Braced Frames with Shape Memory Alloy Braces”, *Journal of Construction Steel Research*, Vol. 67, pp. 65-74.
5. *American Institute of Steel Construction, Inc.*, 2005, Seismic Provisions for Structural Steel Buildings, 341S1-05.
6. American Society of Civil Engineers, 2006, *Minimum Design Loads for Building and Other Structures*, ASCE SEI 7-05.
7. Awkar, J.C., E.M. Lui, 1999, “Seismic Analysis and Response of Multistory Semi-rigid Frames”, *Engineering Structures*, Vol. 21, pp. 425-441.
8. Block, F.M., I.W. Burgess, J.B. Davison, R.J. Plank, 2007, “Development of a Component-Based Connection Element for Endplate Connections in Fire”, *Fire Safety Journal*, Vol. 42, pp. 498-506.
9. Berkeley, W., 2005, SAP2000, *Structural Analysis Program*, Computers and Structures Inc., California.
10. Caltrans, C.G.S., 2010, *Technical Report for the Peer Ground Motion Database*

- Web Application*, Beta Version.
11. Caltrans, C.G.S., 2010, *User Manual for the Peer Ground Motion Database Web Application*, Beta Version.
 12. Chen, W.F., and N. Kishi, 1989, "Semi-Rigid Steel Beam to Column Connections; Database and Modeling", *Journal of Structural Engineering*, American Society of Civil Engineers , Vol.115, No. 1, pp. 105-119.
 13. Cuthill, E., J. Mckee, 1969, "Reducing the Bandwidth of Sparse Symmetric Matrices", *ACM'69 Proceeding of the 1969 24th National Conference*, pp.157-172.
 14. Degertekin, S.O., M.S. Hayalioglu, 2004, *Design of Non-linear Semi Rigid Steel Frames with Semi Rigid Column Bases*, *Electronic Journal of Structural Engineering*, 4.
 15. DBYBHY 2007, *Deprem Belgelerinde Yapılacak Binalar Hakkında Yönetmelik*, 2007, Resmi Gazete No: 26511.
 16. Dutta, A., R.O. Hamburger, 2010, "Case Study of a 40-Storey Buckling-Restrained Braced Frame Building Located in Los Angeles", *The Structural Design of Tall and Special Buildings*, Wiley Interscience, Vol. 19, pp. 77-93.
 17. EN 1991-1-1 : Eurocode3: *Design of Steel Structures- Part 1-8: Design of Joints*, 2005, CEN, European Committee for Standardization, Brussels, Belgium.
 18. EN 1991-1-1 : Eurocode3: *Design of Structures for Earthquake Resistance-Part 1: General Rules, Seismic Actions and Rules for Buildings*, 2004, CEN, European Committee for Standardization, Brussels, Belgium.
 19. Fahjan, Y.M., 2008, "Türkiye Deprem Yönetmeliği (DBYBHY,2007) Tasarım İvme Spektrumuna Uygun Gerçek Deprem Kayıtlarının Seçilmesi ve Ölçeklenmesi", *İnşaat Mühendisler Odası, Teknik Dergi*, Vol. 292, pp. 4423-4444.

20. FEMA 356, 2000, *Federal Emergency Management Agency), Prestandart and Commentary for the Seismic Rehabilitation of Buildings*, Washington D.C.
21. Huber, G., F. Tschemmerneegg, 1998, "Modeling of Beam to Column Joints", *Journal of Construction Steel Research*, Vol.45, No.2, pp. 199-216.
22. Krishnan, S., J. Chen, D. Komatitsch, J. Tromp, 2006, "Case Study of Damage to Tall Steel Moment Frame Buildings in Southern California During Large San Andreas Earthquakes", *Bulletin of the Seismological Society of America*, Vol. 96, No. 4A, pp. 1523-1537.
23. Liu, J., A. Astaneh-Asl, 2000, *Seismic Behavior and Desing of Steel Shear Connections with Floor Slabs*, 12th, World Conference on Earthquake Engineering, Auckland, New Zealand.
24. Mazzoni, S., F. McKenna, M.H. Scott, G.L. Fenves, 2006, *Open System for Earthquake Engineering Simulation (OpenSEES) User Command Language Manual*, Pacific Earthquake Engineering Research Center, Uni. Calif. Berkeley, CA, (<http://opensees.berkeley.edu>).
25. Minas, E.L., J.G. Charis, 2009, "Mechanical Modeling of the Nonlinear Response of Beam to Column Joints", *Journal of Construction Steel Research*, Vol. 65, pp. 879-890.
26. Muto, M., S. Krishnan, 2010, *Response of Tall Steel Buildings in Southern California to the Magnitude 7.8 Shakeout Scenario Earthquake*, Inaugural International Conference of the Engineering Mechanics Institute.
27. Silva, L.S., A.G. Coelho, 2001, "A Ductility Model for Steel Connections", *Journal of Constructional Steel Research* Vol. 57, pp. 45-70.
28. Strong Motion Database, 2007, *Pacific Earthquake Engineering Research Center California* (<http://peer.berkeley.edu/smcat>).

29. Tagawa, H., S. Gurel, 2005, "Application of Steel Channels as Stiffeners in Bolted Moment Connections", *Journal of Constructional Steel Research*, Vol. 61, pp. 1650-1671.
30. Warmka, G.R., K.Z. Truman, 2000, *Improving the Performance of Steel Beam-Column Moment Resistant Connections*, 12th World Conference on Earthquake Engineering, Auckland, New Zealand.
31. Zhu, K., F.G.A. Al-Bermani, S. Kitipornchai, B. Li, 1995, "Dynamic Response of Flexibly Jointed Frames", *Engineering Structures*, Vol. 17, No. 8, pp. 575-580.