EPOKA UNIVERSITY
FACULTY OF ARCHITECTURE AND ENGINEERING
CIVIL ENGINEERING DEPARTMENT

SEISMIC RESPONSE OF HIGH RISE BUILDINGS USING FRICTION DAMPER

By

Endrita MULLETI

Tirana, February 2014

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-CE 500-

MASTER THESIS

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To my beloved father and mother

ABSTRACT

Earthquakes present the biggest natural hazard faced in the zones with high seismicity. During recent years, historical seismicity data of Albania, shows a high number of small earthquakes, sparse medium – sized earthquakes (magnitude M 5,4 – 5.9) and rare large earthquakes (magnitude M > 6.5); and hence no seismic provisions have been incorporated into the existing buildings.

The purpose of this study is to evaluate the seismic response of high rise buildings by using friction dampers. The finite element method (ETABS) is used to evaluate the behavior of the structure equipped by these devices. Two different methods of analyzing (Free Vibration and Time History analysis) have been done to accomplish this objective. In general, this study shows that friction dampers help to reduce story drifts, reduce accidental torsional motions of tall buildings, reduce axial loads of columns and beams, shear load and bending moment of beams, and base shear; reduce vibration amplitude and increase damping ration (the input energy dissipation).

In this study, the seismic performance of the "Sinani tower" building is taken into consideration. This is a 9 story poly-functional building, 1 underground story, 29.6 m in height. The seismic survey is compiled by the Seismology Engineering Department at the Academy of Science. The object is located in an area with the expected seismic intensity with magnitude 7.5 MSK-64. The core focus purpose in this study, is to show how the implementation of friction dampers will improve the seismic response of this object. After the free vibration analysis and time history, the results show that the period, displacement of story, axial load of columns, shear load of beams and bending moment of beams of the damped structure is considerably reduced.

Keywords: seismic, high rise building, friction damper

ABSTRAKT

Termetet paraqesin rrezikun me te larte natyror, te cilet me se shumti hasen ne zonat me seizmicitet te larte. Gjate viteve te fundit, te dhenat historike sizmike te Shqiperise, tregojne nje numer te madh te termeteve te vegjel, numer me te ulet te termeteve te mesem (magnitude M 5,4-5,9) dhe numer me te rralle te termeteve te fuqishem (me magnitude M>6,5); dhe perseri asnje mase mbrojtese nuk eshte ndermarre per ndertesat ekzistuese.

Qellimi i ketij studimi eshte vleresimi i ndikimit sizmik te ndertesave te larta duke perdorur shuaresit sizmik. Per te vleresuar ndikimin te ketyre pajisjeve ne keto ndertesa eshte perdorur metoda e elementeve te fundem (ETABS). Jane perdorur dy metoda te ndryshme analize per kete qellim (Lekundjet e lira dhe Historia kohore). Ne pergjithesi, ky studim tregon se shuaresit sizmik ndihmojne ne zvogelimin e levizjes se nderkateve, reduktimin e levizjeve perdredhese aksidentale ne ndertesat e larta, reduktimin e forcave aksiale ne kolllona e trare, forcat prerese dhe momenti perkules i trareve, forca prerese ne baze; redukton aplituden e vibracionit dhe rrit raportin shuares (shuarjen e energjise hyrese).

Ne kete studim eshte marre ne konsiderate performanca sizmike e kulles “Sinani’. Kjo eshte nje ndertese polifunksionaleme nente kate, 1 kat nentoke, lartesi 29,6 m. Raporti sizmik eshte perpiluar nga Instituti sizmik ne Akademine e Shkencave. Objekti eshte vendosur ne nje zone me intensitet te pritur 7.5 MSK-64. Qellimi kryesor i ketij studimi eshte per te treguar se si implementimi i shuaresve sizmik do te permiresoje reagimin sizmik te ketij objekti. Pas analizes se lekundjeve te lira dhe historise kohore, rezultatet tregojne se perioda, zhvendosja e nderkateve, forca aksiale ne kollona forca prerese ne trare dhe momenti perkules ne trare ne strukuren pa shuares sizmik reduktohen ne menyre te konsiderueshme.

Fjale kyce :sizmike, ndertesa te larta, shuares sizmik

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A special thanks go to my family and friends. Words cannot express how grateful I am for all of the sacrifices that you've made on my behalf.
Finally I thank God, for letting me through all the difficulties.

Endrita Mulleti

APPROVAL SHEET

I certify that an Examination Committee has met on date of viva to conduct the final examination of **Endrita MULLETI** on her MSc in Civil Engineering Term Project entitled "***SEISMIC RESPONSE OF HIGH RISE BUILDINGS USING FRICTION DAMPER***".

Members of the Examination Committee are as follows:

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Date: 07/03/2014

DECLARATION

I hereby declare that the project is based on my original work except for quotations and citations which have been duly acknowledged. I also declare that it has not been previously or concurrently submitted for any other degree at Epoka University or other institutions.

ENDRITA MULLETI

Date: 07/03/2014

TABLE OF CONTENTS

LIST OF TABLES

Table **Page**

LIST OF FIGURES

Figure **Page**

LIST OF SYMBOLS

m	-	mass
w	-	weight
C	-	damping coeffincient
$\dot{u}$	-	velocity of structure
$\ddot{u}$	-	acceleration of structure
K	-	stiffness of structure
u	-	relative displacement
U_o	-	amplitude of vibration
ξ	-	damping ratio
Ω	-	frequency
ω	-	angular frequency
$\bar{\omega}$	-	frequency of earthquake
f_{ck}	-	strength of concrete
C_r	-	critical damping
E_C	-	modulus of elasticity of concrete

E_S	-	modulus of elasticity of steel
ν	-	Poisson's Ratio
$P_{eff}(t)$	-	effective force time history
P_s	-	slip force
P_y	-	yield force
E_D	-	dissipated energy
E_I	-	Input Energy
f_y	-	yield stress
f_u	-	ultimate stress

CHAPTER 1

INTRODUCTION

1.1 GENERAL

The world population is growing so rapidly in recent years that there has been a resurgence of high – rise constructions in the major cities. High rise buildings have become a trend and, moreover, they have paved the way to world competition in constructing tall buildings to exhibit the symbol of power and technology possessed by its population. Also a competition around the whole world to construct high rise buildings is occurring to indicate the symbol of power and technology owned by its population. However, high rise buildings are subjected to vibrations. These vibrations can be due to wind loads, earthquakes, machinery vibrations and other sources of vibration. These vibrations can cause structural damage or even collapse of the structure.

In particular, during recent years earthquakes have been the main cause of structure collapse. An earthquake is a natural phenomenon of ground shaking caused by the sudden energy release inside earth and thus, it causes breaking and movement of the tectonic plates. This may contribute to create other phenomena such as tsunamis, landslides, rock falls, ground settling and liquefaction. As a matter of fact, high rise buildings are not suitable to be constructed in areas with high seismicity, if not studied and analyzed accordingly. However, due to human choice high rise buildings in seismic areas are inevitably built.

1.2 BACKGROUND

Earthquakes are the most serious phenomena that engineers are extremely concerned about. The place and time of occurrence of an earthquake are unpredictable and therefore, this categorizes them as a disaster phenomenon. During an earthquake, a large amount of energy is pumped into the structure. The damage degree of the structure is determined by the way that this energy is consumed. The criteria specified in common building codes is to design structures that can resist to moderate earthquakes without any significant damage and avoid collapse during major earthquakes. The most important emphasis is on life safety. Recent earthquakes have clearly demonstrated that even in developed countries, typical constructions, are not immune to destruction.

The most fearful effects of earthquakes are collapse of structures, especially high rise buildings due to high displacement of stories. The key problem is to reduce the structural response by decreasing the dissipation of input energy due to earthquake. The main objective is to control a major portion of energy that is getting into the structure, so that the seismic response of the structure and damage control potential could be improved. Moreover, these objectives can be accomplished by adopting new techniques of base isolation and energy dissipation devices. Damper devices are the most popular instruments of increasing the dissipation of input energy.

The main goals of any structural design are safety, serviceability, and economy. Achieving these tasks is the most difficult task, due to the uncertainties and unpredictability of when, where and how an earthquake will happen, especially in regions with high seismicity. The main goal in this study is to analyze the seismic behavior of high rise buildings using friction dampers. Different nonlinear computer programs are now capable of modeling friction dampers. Some of these programs are SAP2000, ETABS, ANSA, ANSYS, LUSAS, etc.

In this study , the ETABS is used, as one of the most precise and practicable software in industry and university researches. It is used for dynamic analysis such as earthquake and water wave loading on structures. (Computers & Structures, CSI America)

1.3 PROBLEM STATEMENT

By far, earthquakes pose the highest event natural hazard faced in the zones with high seismicity. They may affect the socioeconomic aspect of the communities involved, and they can cause injury, loss of life, cause huge damages to structures. The damages caused by earthquakes varies from place to place, dependent upon the regional and local geology. Albania is part of the Adriatic-Ionic seismic zone, listed as a moderate seismic region, including Shkodra region, Western Albania -Ohrid-Korca area Peshkopi, with the active line in Lushnje, Elbasan , Dibra and especially Vlora- Tepelena- Erseka-axis. The highest amount of earthquakes occurs in these zones.(Albanian Institute of Seismicity)

During recent years a numerous low to medium intensity earthquakes reached Albania region. Therefore, due to the impact of earthquakes, buildings in Albania may be at risk and may preserve a repeating low cycle failure that can lead to damage of the structures if no structural retrofitting scheme is implemented to prevent losses of life and collapse of buildings. This study is important because implementing Pall Friction Dampers in not common in Albania. There are no previous studies in this area and Pall FD are not used in any structure.

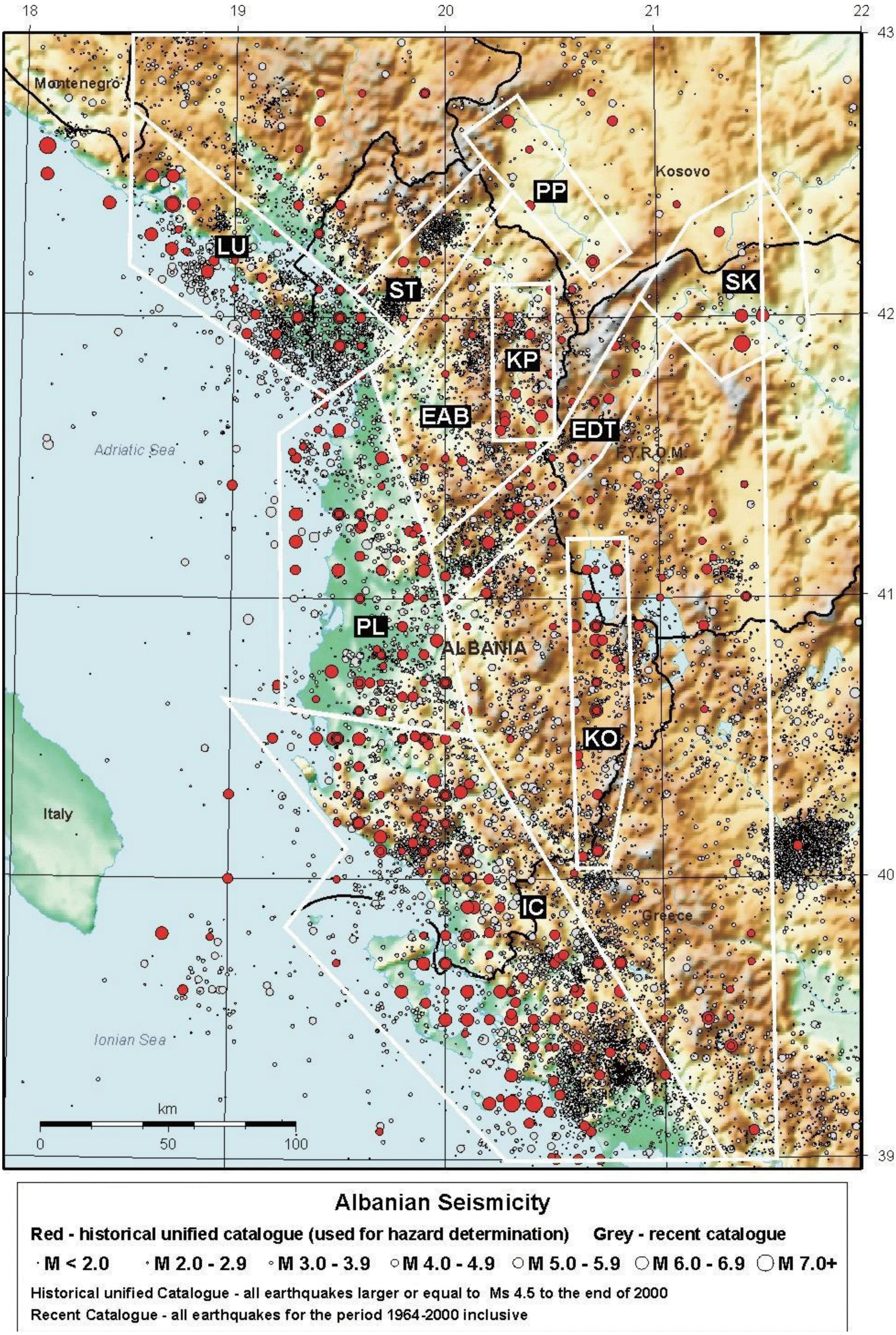


Figure 1 Historical earthquakes around Albania (Aliaj, Adams, Halchuk, Sulstarova, Peci, Muco., 2004)

1.4 OBJECTIVES

The objectives of this study can be listed as following:

a) To remodel a high rise building structure (Sinani Tower) by using Pall friction damper.

b) To locate the suitable types and positioning of friction damping system.

c) To assess the building performances in terms of overall deflection, inter – story drift, and joint acceleration.

1.5 SCOPE OF STUDY

- Damper characteristics (Type of damper devices)
- Seismic evaluation response of high rise building structures equipped by friction damper.

1.6 ORGANIZATION OF STUDY

The preparing of the objectives and scopes of the study are explained below:

Stage 1: Explaining the project objectives and scopes of the study.

It is to verify the feasibility of the study outcomes and planning of methodology in efficient thesis of input and output.

Stage 2: Literatures collecting data and modeling of structure.

Initial study should be done to understand the behavior of tall building structures and best solution to retrofit it. Knowing the performance of tall building structures is essential to assume the structure behave according to literature findings. Obtaining the information of model before head and spearhead the modeling technique is part of the requirement in successful overall analysis.

Stage 3: Verification of retrofitting devices and modeling.

The purpose of this stage is to identify appropriate and application of retrofitting devices, which are the friction dampers devices. In addition, the theoretical background of the frame equipped by friction damper device is also included to verify the concept of work on the device. Material properties and analysis methods have been determined to obtain correct mode shapes. The structure with and without damper has been modeled by ETABS software to verify the response of structure with appropriate earthquake signals. In other words, the models are proposed with (damped) and without (UN damped) friction damper for comparison purposes.

Stage 4: Vulnerability assessment of modeling and response analysis.

Time history analysis have been done to find responses of the two models.

Stage 5: Discussion and Conclusion

Summary of the project according to the different analysis methods and the proposed retrofitting device will be presented. Comments on the further improvement to the study are to be enumerated.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

During an earthquake, a building must exhibit adequate strength, redundancy and ductility. When severe earthquakes happen, they cause the ground to shake and, consequently, they vibrate the structure. The behavior of a reinforced concrete structure depends on the detailing of its component elements, in the inelastic deformation regions.

The structures that are constructed before the development of seismic design exhibit low ductility. During strong earthquakes it is observed that non-ductile concrete framed structures are often severely damaged or also collapse. The seismic response of the structure can be improved if a major portion of the energy can be consumed during the motion of the structure. The manner in which this energy is consumed in the structure determines the level of damage.

Repairing a damaged concrete structure, it is required a detailed evaluation of the type and extent of damage. The repair technique should be selected in accordance with the location of damage and its influence in the response. Generally, all the present methods of a seismic design and analysis place credence on the ductility of the structural elements, i.e., ability to dissipate energy while undergoing inelastic deformations causing bending, twisting, and cracking. This method assumes some permanent damage that may be as economically significant as the total collapse of the structure. The response of the structure can be controlled without structural damage if a major portion of the seismic energy can be dissipated mechanically.

2.2 EARTHQUAKE CHARACTERISTICS

An earthquake is a trembling of the ground that results from the sudden shifting of rock beneath the earth's crust. Earthquakes may cause landslides and rupture dams. Severe earthquakes destroy power and telephone lines, gas, sewer, or water mains, which, in turn, may set off fires and/or hinder firefighting or rescue efforts. Earthquakes also may cause buildings and bridges to collapse. (Quizlet Dictionary, 2002)

Ground movement during an earthquake is seldom the direct cause of death or injury. Most earthquake-related injuries result from collapsing walls, flying glass, and falling objects as a result of the ground shaking, or people trying to move more than a few feet during the shaking. (FEMA 2004)

2.3. RESPONSE OF HIGH RISE BUILDING

2.3.1 Definition

High Rise building is a building in which tallness influence planning, design and use, or a building whose height creates different condition in the design, construction, and use than those that exist in common buildings of a certain region. A high rise building is a complex system that consists of structural elements, beams and columns, and also non-structural elements, cladding, floors and partitions that are assembled together by different connection types, (Marsono. A., 2009)

The principal load resisting elements:

The two primary types of vertical load resisting elements of tall buildings are columns and walls.

– Walls may act either independently as shear walls, or in assemblies as shear wall cores, around stairwells and elevators.

– Columns will be provided in otherwise unsupported regions to transmit gravity loads, and in some types of structures, lateral loads (wind and seismic).

– Since the gravity loading on different floors tends to be similar, the weight of the floor system per unit floor area is constant, regardless of building height.

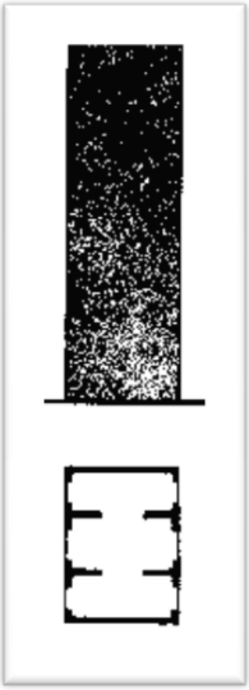
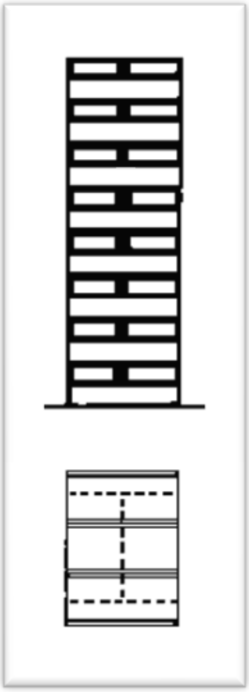
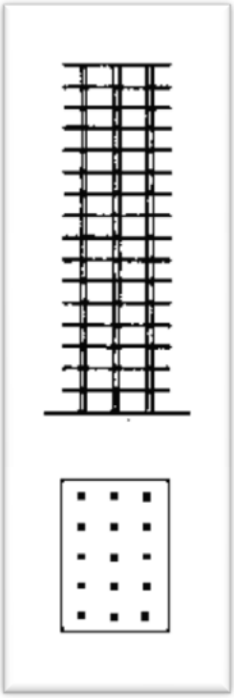
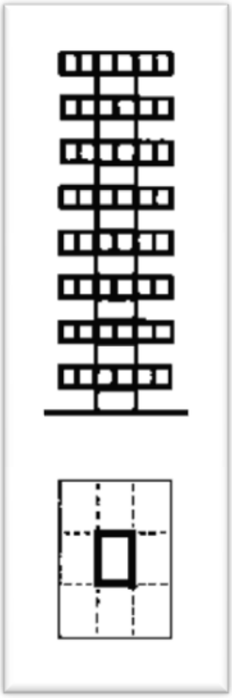
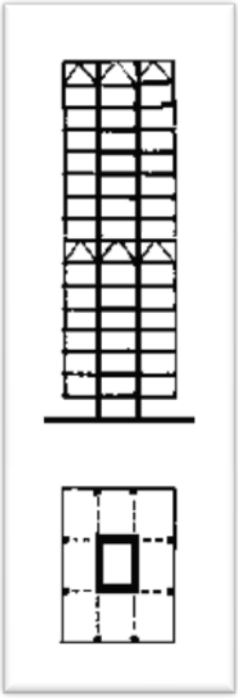
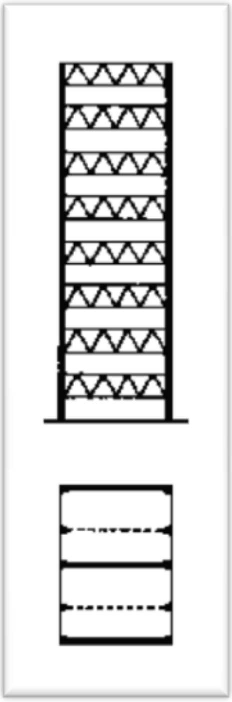
– Since the load on a column is cumulative of the floors above it, the weight of column per unit area increases linearly with the building height.

– The bending moments caused by lateral loads increase with at least the square of building height, becoming more important as building height increases.

2.3.2 Structural Systems for High Rise Buildings

In recent years, the structural system for high rise buildings has undergone a high evolution. According to the experiences lateral loads such as wind and earthquake loads, affect the type of structural systems, especially for high rise buildings. Two fundamental lateral load resisting systems are the braced frame (also known as shear truss or vertical frame) and the moment resisting frame (moment frame or rigid frame), (Council on tall buildings and urban habitat, 1995). Nowadays, there are different types of lateral load resistance systems. Some of these different types of lateral load resistance systems are listed in the table below:

Table 1 Different types of lateral load resistance systems (Luebkeman, 1996)

Bearing Wall System	Bearing Walls with Cores	Self-Supporting Boxes	Core with Cantilevers
			
Flat Plate Systems	**Interstitial Frames**	**Suspension**	**Staggered Truss**
			

2.3.3 Structural Response Characteristics

Response of tall building structure depends only on stiffness, strength and ductility. Stiffness is the ability of a component or an assembly of components to resist deformations when subjected to actions. Strength is the capacity of a component or an assembly of components for load resistance at a given response station. Ductility is the ability of a component or an assembly of components to deform beyond the elastic limit, (Amr. S. E., and Luigi. D. S., 2008).

The lateral deformability of structural system is measured through the horizontal drift. In buildings, story drifts Δ are the absolute displacement of any floor relative to the base, while inter-story drift δ defines the relative lateral displacement between two respective floors. The overall lateral displacement is affected by the structural system that is used.

2.4 DAMPING

Damping is a phenomenon by which mechanical energy is dissipated in dynamic systems; generally it is converted into thermal energy. Damping reduces the build-up of the strain energy and the system response, especially for near resonance conditions, where damping controls the response. In other words, damping is utilized to characterize the ability of structures to dissipate energy during dynamic response.

Damping values depends on several factors such as, vibration amplitude, material of construction, fundamental periods of vibration, mode shapes and structural configurations, (Amr. S. E. and Luigi. D. S. 2008).

Types of damping

Three primary mechanisms of damping

- Internal Damping - of material

- Structural Damping - at joints and interface
- Fluid Damping – through fluid-structure interactions

Two types of external dampers can be added to a mechanical system to improve its energy dissipation characteristics:

- Active Dampers; require external source of power
- Passive Dampers; does not require external source of power

The main functions of dampers can be listed as follows:

a) Reducing story drifts of tall building structures,
b) Reducing accidental torsional motions of tall building structures,
c) Increasing damping ratio; increasing dissipation of input energy due to earthquakes,
d) Reducing vibration amplitude of tall structures.

2.4.1 Passive Energy Dissipation Devices

Passive energy dissipations have been under development for a number of years with a rapid increase in implementation starting in the midd-1990s. The principal function of a passive energy dissipation system is to reduce the inelastic energy dissipation demand on the framing system of a structure (Constatinou and Symans 1993; Whittaker et al. 1993).

The result is to reduce damage to the structure system. A high number of passive energy dissipation devices are available and others are under development. Devices that have most commonly been used for seismic protection of structures include viscous fluid dampers, viscoelastic solid dampers, friction dampers, and metallic dampers. Other devices that could be classified as passive energy dissipation devices (or, more generally, passive control devices) include tuned mass and tuned liquid dampers, both of which are primarily applicable to wind vibration control, recentering dampers, and phase transformation dampers. In addition, there is a class of dampers, known as semi active dampers, which may be regarded as controllable passive devices in the sense that they passively resist the relative motion between their ends but have controllable mechanical properties. Examples of such dampers include variable-orifice dampers, magneto rheological dampers, and electro rheological dampers (Symans and Constantinou 1999).

Semi active dampers have been used for seismic response control in many countries, notably Japan (Soong and Spencer 2002). The growth in application and development of passive energy dissipation devices has led to a number of publications that present detailed discussions on the principles of operation and mathematical modeling of such devices, analysis of structures incorporating such devices, and applications of the devices to various structural systems (e.g., Constatinou et al. 1998; Soong and Dargush 1997; Hanson and Soong 2001).

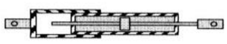
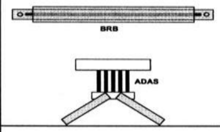
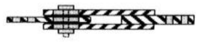
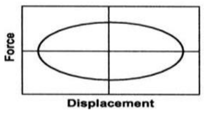
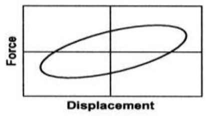
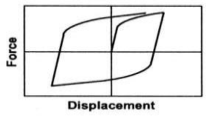
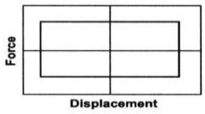
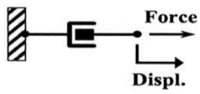
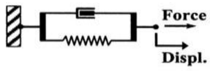
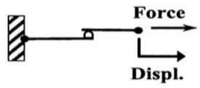
2.4.2 Some Basic Types of Dampers

The damage of a structural system could be reduced if part of the input energy caused by the earthquake, could be dissipated through special devices which can be replaced if necessary after an earthquake.

A variety of passive energy dissipation devices are available and have been implemented worldwide for seismic protection of structures. These devices can be classified into three categories: viscous and viscoelastic dampers, metallic dampers, and friction dampers.

Table 2 shows the classification of dampers and their basic characteristics:

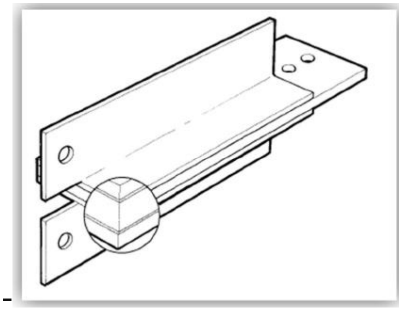
Table 2 Classification of dampers and their basic characteristics(M.D Symans, 2008)

	Viscous Fluid Damper	Viscoelastic Solid Damper	Metallic Damper	Friction Damper
Basic Construction				
Idealized Hysteretic Behavior				
Idealized Physical Model			Idealized Model Not Available	
Advantages	- Activated at low displacements - Minimal restoring force - For linear damper, modeling of damper is simplified. - Properties largely frequency and temperature-independent - Proven record of performance in military applications	- Activated at low displacements - Provides restoring force - Linear behavior, therefore simplified modeling of damper	- Stable hysteretic behavior - Long-term reliability - Insensitivity to ambient temperature - Materials and behavior familiar to practicing engineers	- Large energy dissipation per cycle - Insensitivity to ambient temperature
Disadvantages	- Possible fluid seal leakage (reliability concern)	- Limited deformation capacity - Properties are frequency and temperature-dependent - Possible debonding and tearing of VE material (reliability concern)	- Device damaged after earthquake; may require replacement - Nonlinear behavior; may require nonlinear analysis	- Sliding interface conditions may change with time (reliability concern) - Strongly nonlinear behavior; may excite higher modes and require nonlinear analysis - Permanent displacements if no restoring force mechanism provided

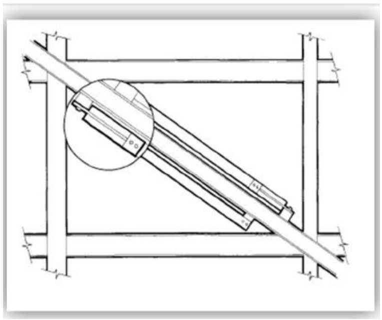
Viscoelastic Dampers

Viscoelastic (VE) damper is one of the best appropriate dissipation devices. This type of damper dissipates the building's mechanical energy by converting it into heat. Several factors such as ambient temperature and the loading frequency will affect the performance as well as the effectiveness of the damper system. VE dampers have been able to increase the overall damping of the structure significantly, therefore, improving the overall performance of dynamically sensitive structures.

In addition, the viscoelastic (VE) dampers are considered to be the most promising and have been installed in several buildings all over the world. It consists of layers of VE material (copolymers or glassy substances) bonded with steel plates. Vibration energy is dissipated through shear deformation of VE material sandwiched between steel plates, (Nishant. K. R. et al, 2009).



(a)



(b)

Figure 2 (a) Viscoelastic shear damper; (b) diagonal bracing configuration with viscoelastic damper, (Nishant Kishore Rai et al, 2009).

Metallic Dampers

Metallic dampers dissipate energy through hysteretic behavior of metals when deformed into their inelastic range. A wide variety of devices have been developed and tested that dissipate energy in flexural, shear, or extensional deformation modes. Figure 3 shows the triangular added damping and stiffness (TADAS) device that uses triangular steel plates.

As suggested by its name, this device increases both stiffness and damping of the structure, (Chopra, 2002).

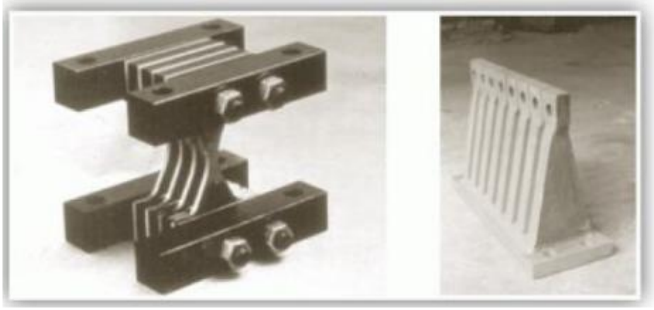


Figure 3 TADAS device, (Chopra, 2002)

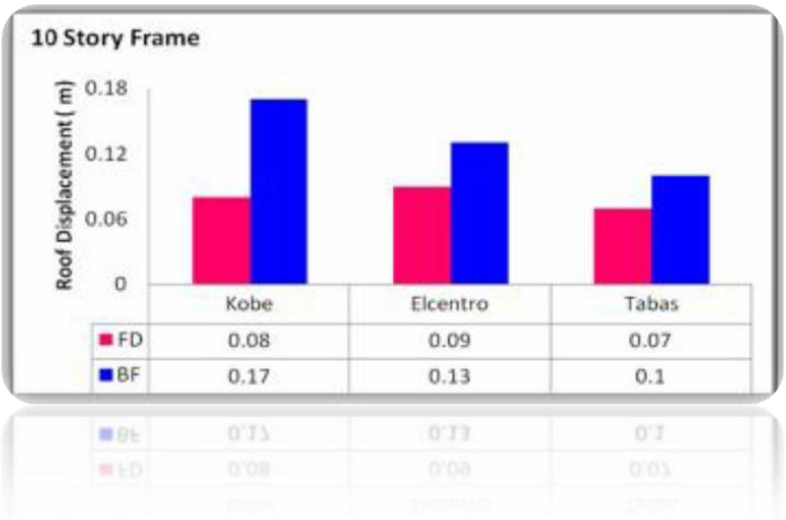
Friction Dampers

This device depend on the resistance created between two solid interfaces sliding relative to one another, during severe seismic excitations, the device slips at a predetermined load, providing the desired energy dissipation by friction while at the same time shifting the structural fundamental mode away from the earthquake resonant frequency. One of these devices is slotted bolted connection (SBC), (Chopra, 2002).

Figure 4 Slotted Bolted Connection (SBC)

2.5 SESMIC PERFORMANCE OF FRICTION DAMPER

(Vaseghi. J. et al. 2009), in this study, the behavior of eccentric braced frame (EBF) is studied with replacing friction damper (FD) in confluence of these braces, in 5 story and 10 story steel frame. Two have been chosen as reference buildings for this study: 5 story and 10 story frame structures. The two buildings have an identical 3 bay layout in plan, 6 m span and 3 m store height. The methodology proposed in this study is based on performing a numerical parametric analysis of building structures occupied with FD system. In this study, the nonlinear dynamic analyses were performed using three earthquake records. These records include El – Centro (1940), Tabas (IRAN, 1978 and Kobe (1995) earthquakes. Results of this study show that, roof displacement, base shear and axial loads of columns of two buildings have been decreased by using friction dampers (Figures 5, 6 and 7).



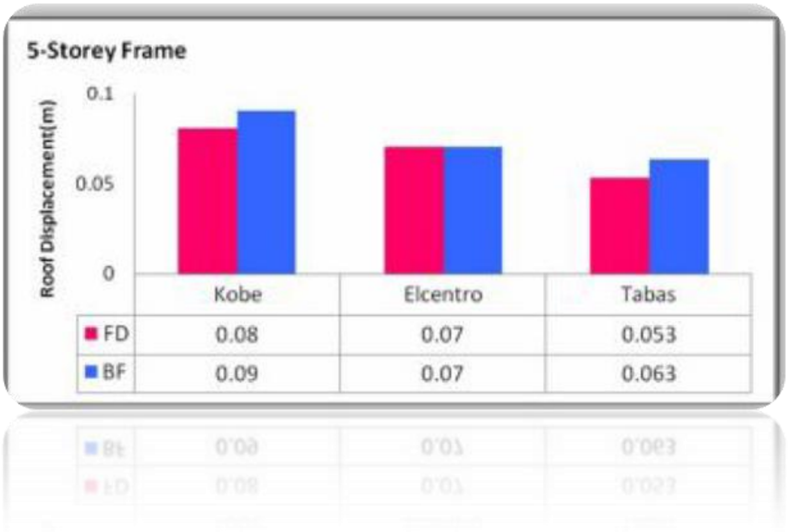
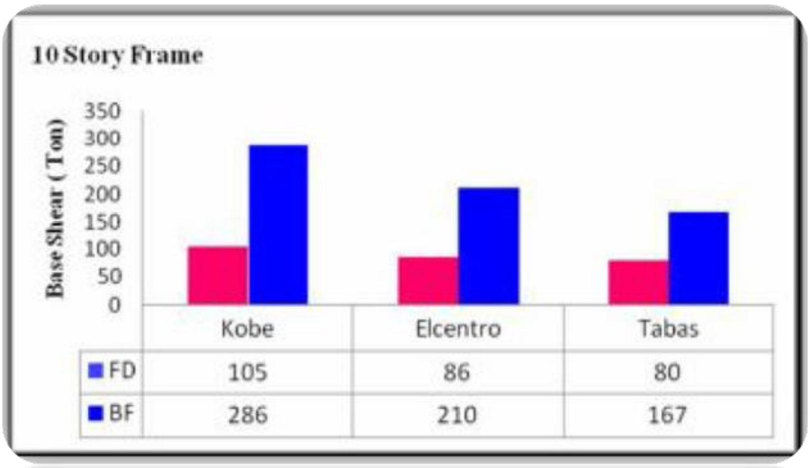


Figure 5 Envelope of Maximum Roof Displacement (Vaseghi. J. et al. 2009)



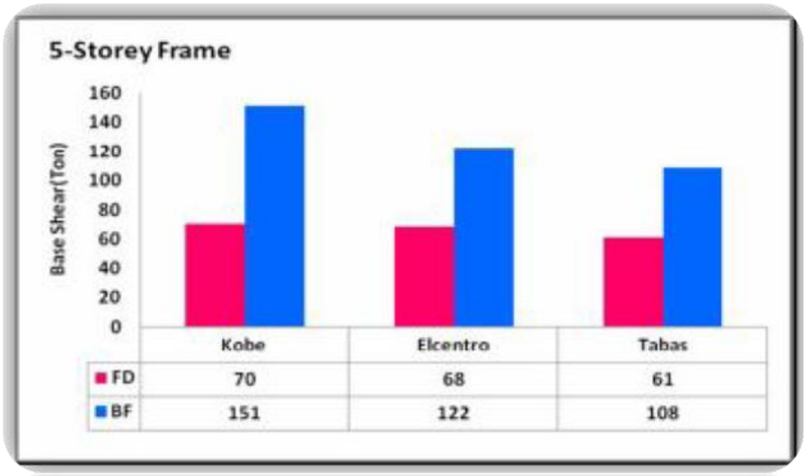


Figure 6 Envelope of Maximum Base shear, (Vaseghi. J. et al. 2009).

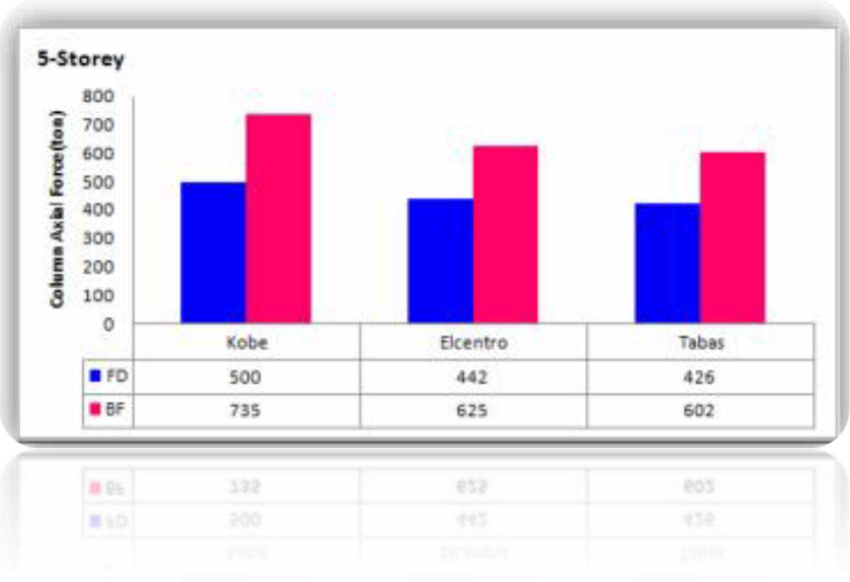


Figure 7 Envelope of Column axial force, (Vaseghi. J. et al. 2009)

Filiatrault et al. (1987), in this study, a three – story frame equipped with friction dampers was tested on a shake table at the University of British Columbia, Vancouver. Even an earthquake record with a peak acceleration of 0.9g did not cause any damage to friction damped braced frame, while the conventional frames were severely damaged at lower levels.

X- Braced friction damper

Figure 8 Friction dampers as X – brace and chevron brace, (Moe Cheung et al, 2000)

(A.Malhotra, D. Carson, P. Gopal, A. Braimah, G. Di Giovanni, and R. Pall, 2004), in this study the upgrade of St. Vincent Hospital in Ottawa is performed using friction dampers. St. Vincent Hospital comprises of 5-storey concrete frame construction. One of the blocks is a new construction and the other four blocks were built between 1890 and the early 1950's. The earthquake resistance of the existing structures was significantly less than that required by the current building codes. Since hospitals are of post disaster importance, the engineers recommended that the existing structures be upgraded along with the new expansion.

The traditional methods of seismic rehabilitation were not considered suitable as concrete shear walls or rigid steel bracing because they require expensive and time consuming foundation work. Supplemental damping in conjunction with appropriate stiffness offered an innovative and attractive solution for the seismic rehabilitation of this hospital. This was achieved by using Pall Friction Dampers in steel bracing. In addition, analyses were carried out for earthquake motions in combined x and y-axes. A total of 183 friction dampers were used in all blocks. The design slip load of friction damper was 300 kN.

In addition, researches about friction damper, in particular those have been done by Canadian Society for Civil Engineering, illustrates that as soon as the structure undergoes small deformations, the friction dampers are activated and start dissipating energy. Since the dampers dissipate a major portion of the seismic energy, the forces acting on the structure are considerably reduced. In addition, higher energy dissipation capacity of friction dampers makes up for the lack of ductility and mitigates damage to other nonstructural components. They have also found out that, the energy dissipation of friction damper is the largest compared to other damping devices such as viscous or viscoelastic devices.

As a result of above properties of friction damper, the required amount of supplemental damping is provided by using. Unlike other devices, the maximum force in a friction damper is pre – defined and remains the same for any future ground motion. Therefore, the design of bracing and connections is simple and economical. There is nothing to yield and damage, or leak. Thus, they do not need regular inspection, maintenance, repair or replacement before and after the earthquake.

2.6 SEISMIC PERFORMANCE OF HIGH-RISE BUILDINGS

(Azlan Adnan, Tan Chee Wei, 2001), have found out from the analysis that the structures of high-rise building with different heights behave differently under earthquake loadings. They have realized that the maximum axial load, shear and bending moments occur at ground columns. Also the results show that the effect of earthquake decreases as intensity of earthquake is reduced. The effects of earthquake are not proportional to the building height, for example, the axial load increases as the number of floors is reduced. Therefore, different building systems will behave differently and the response of the buildings are more depending on their natural period.

2.7 STRUCTURAL DESIGN AND CONTROL

Harsh ground shaking induces lateral inertial forces on buildings, causing them to sway back and forth with amplitude proportional to the energy fed in. If a major portion of this energy can be consumed during building motion, the seismic response can be noticeably improved. The manner in which this energy is consumed in the structure determines the level of damage. (Avtar S. Pall, 1982)

Normal design of buildings includes satisfying strength and serviceability limit states. The suitable approach is to correlate the components to satisfy the strength limit states and then check for serviceability limit state. However, in the case of high-rise buildings, where sensitivity to structural motion caused by environmental effects, such as wind and earthquake loads, the structure is first proportioned to satisfy the serviceability limit state and then will check for strength limit state. As the height of building increases, the influence of lateral loads becomes more essential. To have an economical solution for the problem, supplemental control techniques need to be applied for decreasing the structural response. Nowadays, these methods of structural control are commonly considered: passive, semi-active, and active control systems.

2.8 SEISMIC RETROFITTING DESIGN

Passive energy dissipation devices such as viscoelastic, metallic, and friction dampers have been widely used to decrease the response of civil engineering structures that are subjected to seismic loads. These devices by absorbing the structural vibratory as well as dissipating the input energy through their natural hysteresis behavior minimize the structural damages. Since the hysteretic behavior of friction damper devices could be kept unwavering for cyclic loads and also their simple energy dissipation mechanism and easy manufacturing, installation and maintenances, have been caused to develop and applied them for the seismic protection of building structures.

For the relevance of these friction dampers for the seismic design of structures, their slip loads as well as the stiffness of supporting braces should be determined. Filiatrault and Cherry (1987) have presented the design procedure for friction dampers that minimizes the sum of the displacement and dissipating energy by carrying out the parameter studies such as the structural fundamental period, frequency components of excitation load and the slip load of friction damper. Garcia and Soong (2002) have presented the method that obtains the optimal viscosity in terms of a story distribution by iterating the process that the inter-story drift or the inter-story velocity is defined as a controllable index and then installs the viscous damper at the location of maximum controllable. Viti, et al. (2006)

presented both the weakening retrofit to reduce maximum acceleration and the supplemental damping devices to control structural deformations.

2.9 SUMMARY OF LITERATURE REVIEW

The first part of the literature review is about the earthquake characteristics as well as the causes of earthquake, and followed by the structural systems of high rise structures. After that, according to the previous studies and researches, the application of different kinds of dissipation devices especially friction dampers have been discussed and presented. In other words, the literature review presented the technological development and how to practice these devices to control the level of damage during high seismic loads.

CHAPTER 3

THEORETICAL BACKGROUND

3.1 GENERAL

Structural safety is one of the most important tasks for a structural engineer. In recent years, structural safeties against natural and man-made hazards have been in various stages of development and research. Table 3.1 shows three different types of structural protective systems: 1) Base Isolation; 2) Passive Energy Dissipation; 3) Semi – Active and Active Control.

Table 3 Structural Protective Systems, (Soong et al, 1997)

Base Isolation	Passive Energy Dissipation (PED)	Semi – Active and Active Control
• Elastomeric Bearing • Lead Rubber Bearing • Sliding Friction Pendulum	• Metallic Damper • Friction Damper • Viscoelastic Damper • Viscous-Fluid Damper • Tuned-Mass Damper • Tuned-Liquid Damper	• Active Bracing System • Active Mass Damper • Variable stiffness or Damping system • Smart Materials

Of the three groups, friction dampers can now be considered more mature technology with wider applications as compared with the other two groups (Soong et al, 1997).

3.2 EQUATION OF MOTION

The principal problem of structural dynamics that concerns structural engineers is the behavior of structures that are subjected to earthquake – induced motion of the base of structure. The structure's linear response can be explained by the equation of motion as is given below:

$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g(t)$ Eq. 3.1

Where:

m=mass

$\ddot{u}$=acceleration of structure

c=damping coefficient

$\dot{u}$=velocity of structure

k=stiffness of structure

u=relative displacement with time

$\ddot{u}_g$ (t)=ground acceleration that varies with time

The right side of above equation can be replaced by; P_{eff}(t), effective force time history. In fact, effective force can be considered as an equivalent static force to the ground motion, (Chopra, 2002).

3.3 DAMPER CHARACTERISTICS

As it was mentioned earlier, the process that free vibration steadily diminishes in amplitude is called damping. In damping, the energy of the vibrating system will be dissipated by various mechanisms. In brief, under seismic excitations that have relatively long durations, a structure undergoes several cycles during the forced vibration, therefore, its response depends more on the amount of energy that is dissipated during each cycle (area under the force – displacement loop) than on the nature of the dissipative force that develops (viscous, friction, elastoplastic or viscoplastic). Because of this, the dissipation properties of structures have been traditionally averaged over a cycle of motion and expressed in term of dimensionless ratios which originates from the linear theory of structural dynamics (Chopra 2002).

It is worth emphasizing that, the response of the structure is much more sensitive to the nature of the dissipation mechanism, than to the amount of energy dissipated per cycle. Under cyclic forces or deformations, this behavior implies formation of a force – deformation hysteresis loop, (Chopra 2002). The damping energy that will be dissipated during one deformation cycle, between deformations limits $\pm u_o$, is given by the area within the hysteresis loop.

In order to represent the ability of a structure or a structural component to dissipate energy, various dimensionless quantities have been proposed to express damping. According to studies, one of the most popular dimensionless quantities for measuring the damping property of a structure is damping ratio (ξ). Damping ratio can be obtained by following equation:

$$C = 2\xi m\omega_o = E_D / \pi\, \Omega U_o^2 \qquad \text{Eq. 3.2}$$

The damping coefficient (C) is a measure of the energy dissipated in a cycle of free vibration that will appear shortly. In addition, the damping ratio (ξ), which is a dimensionless measure of damping, is a property of the structure that also depends on mass and stiffness of structure. In above equation, U_o is the amplitude of vibration and Ω, is the frequency and E_D is the dissipated energy. Therefore, the damping ratio can be obtained by the following equation:

$$\xi = 1\omega_o E_D / 4\pi\,\Omega E_S \qquad \text{Eq. 3.3}$$

Where:

$E_S = 1/2\ m\omega_0^2 U_0^2$ Eq. 3.4

Is the stored strain energy. (Chopra 2002)

The energy dissipated (E_D) by process of damping in one cycle of harmonic vibration is:

$$E_D = \int f_D du = \int_0^{2\pi/\omega} (c\dot{u})\dot{u}dt = \int_0^{2\pi/\omega} c\dot{u}^2 dt = c\int_0^{2\pi/\omega} [\omega u_0 \cos(\omega t - \phi)]^2 dt = \pi c\omega {u_0}^2 = 2\pi\xi \frac{\omega}{\omega_n} k{u_0}^2 \quad \text{Eq. 3.5}$$

The energy dissipated is proportional to the square of the amplitude of motion. It is not a constant value for any given amount of damping and amplitude since the energy dissipated increases linearly with excitation frequency. Therefore, the input energy (E_I) to the system due to the applied force will be dissipated by the damping process. The external force p(t), inputs energy to the system, which for each cycle of vibration is:

$$E_I = \int p(t)du = \int_0^{2\pi/\omega} p(t)\dot{u}dt = \int_0^{2\pi/\omega} [p_0 sin\omega t][\omega u_0 \cos(\omega t - \phi)]dt = \pi p_0 u_0 sin\phi \quad \text{Eq. 3.6}$$

The input energy varies linearly with the displacement amplitude. In contrast, the dissipated energy varies quadratically with the displacement amplitude. According to studies, before steady state, the input energy per cycle exceeds the energy dissipated during the cycle by damping, which leads to larger amplitude of displacement in the next cycle. With growing displacement amplitude, the dissipated energy increases more rapidly than the input energy According to the researches that have been done, during the early stages of response, there is a rapid build – up of the input energy (E_I), similar to an impulsive loading. And also, it has been found that damping dissipates energy over the response cycle. For low damping ratio, the energy dissipated (E_D) per cycle is small, and therefore many cycles are required before the input energy is eventually dissipated.

Thus, as damping ratio (ξ) is being increased, the energy dissipated per cycle increases, and therefore the stored energy (E_S) build – up is reduced. If the part of this energy could be dissipated through special devices which can be replaced, as necessary, after an earthquake, the structural damage could be reduced. Such devices may be cost – effective in the design of new buildings or added to existing structures that are deficient in their earthquake resistance, (Chopra 2002).

According to researches, damping ratio (ξ) of a structure increases by the increment of vibration amplitude of the structure. For example, in the vibration test of Millikan Library building on the Campus Institute of Technology in Pasadena, the damping ratio (ξ) in the fundamental east – west mode varied between 0.7 to 1.5%, increasing with the amplitude of response. And also in the north – south direction, the natural period of the fundamental mode was 0.51 sec, increasing roughly 4% over the resonant amplitude range of testing; acceleration of 0.005g to 0.02g at the roof. The damping ratio in this mode varied between 1.2 and 1.8%, again increasing with the amplitude of response, (Chopra 2002). It is worth to mention, that most building codes do not recognize the variation of damping ratio with structural materials, and therefore typically a 5% damping ratio is being used in the code – specified earthquake forces and design spectrum.

3.4 PALL FRICTION DAMPER

Pall Friction Dampers are simple and foolproof in construction. Basically, these consist of series of steel plates, which are specially treated to develop very reliable friction. These plates are clamped together and allowed to slip at a predetermined load. Decades of research and testing have led to perfecting the art of friction. Their performance is reliable, repeatable and they possess large rectangular hysteresis loops with negligible fade. Pall Friction Dampers are passive energy dissipation devices and, therefore, need no energy source other than earthquakes to operate it.

Pall Friction Dampers are available for long slender tension-only cross bracing, single diagonal tension-compression bracing and chevron bracing. The damper for cross bracing is a unique mechanism. When the brace in tension forces the damper to slip, the damper mechanism forces the other brace to shorten and, thus avoid buckling. In this manner, the other brace is immediately ready to slip the damper on reversal of cycle. These dampers have been used in 22 meter (65 feet) long slender bracing. To avoid pounding at the expansion joints, Pall Friction Connectors are custom made to accommodate bi-directional movements.

The damping of typical structures is about 1- 5% of critical, while, the damping of structures that have been built by friction damper devices is easily about 20-30% of critical. Pall Friction Dampers are available for long slender tension – only cross bracing, single diagonal tension – compression bracing and chevron bracing (Figure 9).

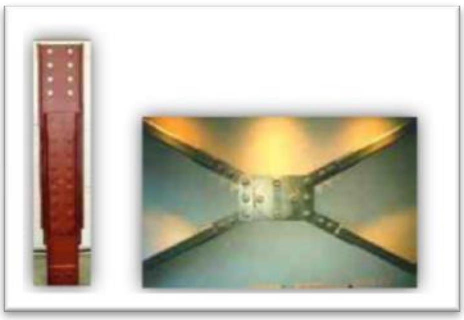


Figure 9 Pall Friction Dampers, diagonal and X – bracing, (Pasquin, 2002)

Another attractive feature of friction dampers is that they offer stiffness in conjunction with supplemental damping. While supplemental damping is beneficial in reducing the earthquake forces and amplitudes of vibration, added stiffness is beneficial for stability.

For a given force and displacement in a damper, the energy dissipation of friction damper is the largest compared to other damping devices. Therefore, fewer friction dampers are required to provide a given amount of supplemental damping. Unlike other devices the maximum force in a friction damper is pre – defined and remains the same for any future ground motion.

3.4.1 Slip Load

The friction dampers are designed not to slip during wind. During a major earthquake, they slip prior to yielding of structural members. In general, the lower bound is about 130% of wind shear and the upper bound is 75% of the shear at which the members will yield. As seen in the diagram for Response versus Slip Load, if the slip load is very low or very high, the response is very high. (Pall Dynamics)

Several parametric studies have shown that the slip load of the friction damper is the principal variable with the appropriate selection of which it is possible to tune the response of structure to an optimum value. Optimum slip load gives minimum response. Selection of slip load should also ensure that after an earthquake, the building returns to its near original alignment under the spring action of an elastic structure. Studies have also shown that variations up to ±25% of the optimum slip load do not affect the response significantly. Therefore, small variations in slip load (8-10%) over life of the building do not warrant any adjustments or replacement of friction damper.

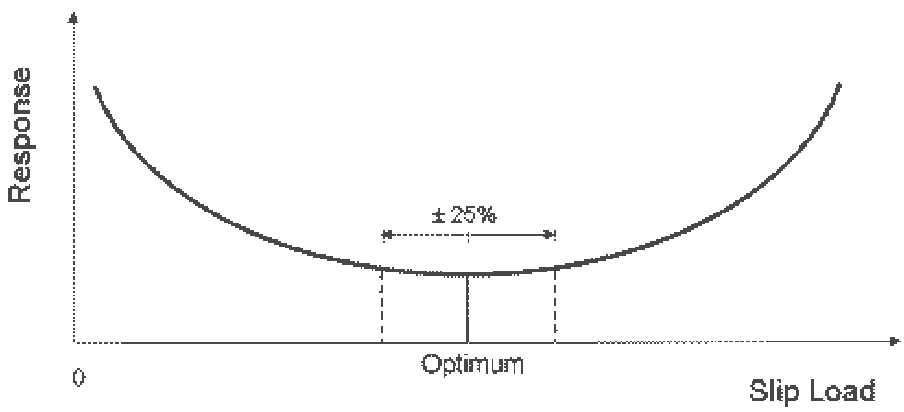


Figure 10 Response versus Slip Load

3.4.2 Characteristics of Pall Friction Damper

- They are simple and foolproof in construction.
- Offer reliable and repeatable performance at low cost.
- Possess large rectangular hysteresis loops.
- Greater energy dissipation for a given force. Hence, fewer Pall Friction Dampers are needed. Conversely, exert lesser force for a given damping.
- Provide supplemental damping and stiffness for added stability.
- Performance is independent of velocity and temperature.
- Constant force for all future earthquakes. Therefore, design of connections and members is economical.
- They are not active during service loads and wind. Hence, no possibility of failure due to fatigue before an earthquake.
- Need no repair or replacement before and after earthquake. There is nothing to damage or leak.
- Energy dissipation is through friction and not through the damaging process of yielding.
- After an earthquake, the building returns to its near original alignment due to spring action of an elastic structure.
- Compact and narrow enough to be hidden in partitions.
- They can accommodate foundation settlements.
- Available for all types of bracing, including tension cross bracing, and expansion joints.
- Custom made. Easily adaptable to any site condition. Can be welded or bolted

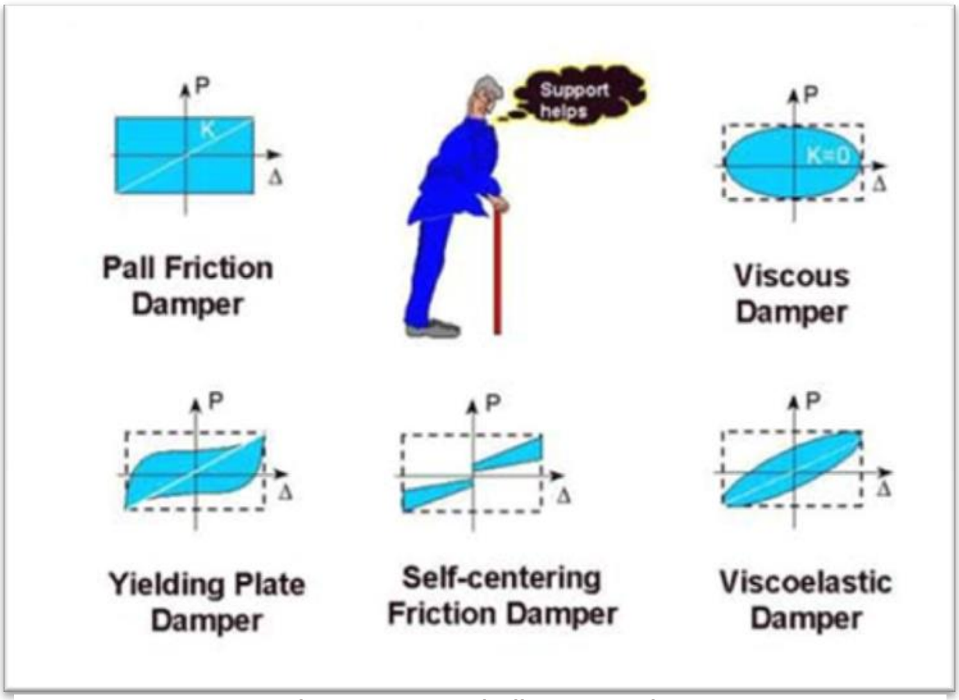


Figure 11 Comparison of hysteresis loops of different kinds of dampers (Pall Dynamics)

3.4.3 Design Criteria

The quasi-static design procedures given in most building codes are ductility based and do not explicitly apply to buildings with supplemental damping. In the past few years, several guidelines on the analysis and design procedure of passive energy dissipation devices have been developed in the U.S. The latest and most comprehensive document is the "NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 356 / 357, issued in 2000". It is the damped braced natural frequency, k_f and k_a are the stiffness of the frame and the added friction damper brace, respectively, and $f(t) = f_f(t) + f_a(t)$ is the combined system restoring forces, as shown in Figure 12. The frame restoring force – displacement relationship $f_f(t)$ vs. u_t depends on the frame stiffness k_f and the frame yield force p_y, while the added restoring force – displacement relationship $f_a(t)$ vs. u_t depends on the added stiffness k_a and the friction damper slip force p_s.

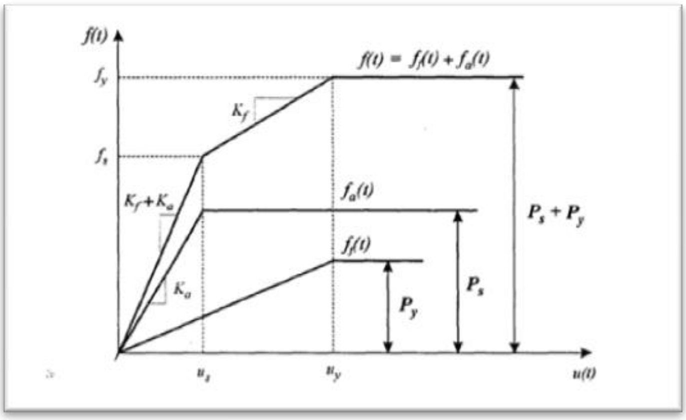


Figure 12 Force relationship of a friction damped model, (Aliyeh Jowrkesh Safai, 2001)

The total system restoring force f (t) is nonlinear due to the friction unit slipping and the frame member yielding. As displacement u (t) increases, the system restoring force increases with stiffness $k_f + k_a$. When u (t) exceeds u_s (slip displacement) the added friction damper slips, and the system restoring force is defined by stiffness k_f alone. Further, when u(t) exceeds u_y the frame members yield and assuming a perfect elasto – plastic behavior, the combined system restoring force is limited to $f_y = p_s + p_y$. When the displacement decreases, the strain energy stored in the frame members and the added component is recovered. Thus, the force – displacement hysteresis loops can be plotted as shown in Figure 3.7, (Aliyeh Jowrkesh Safai, 2001).

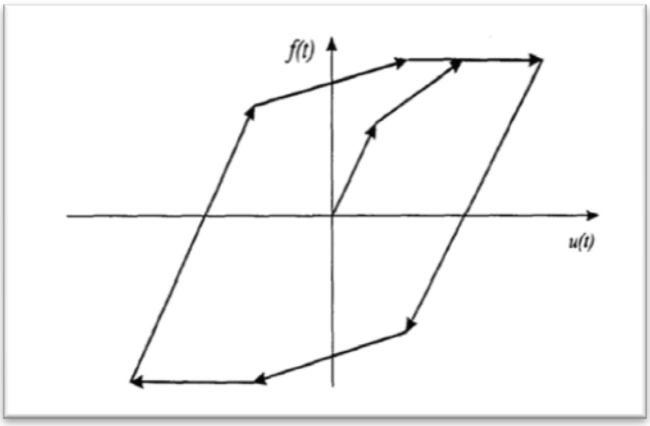


Figure 13 Force-displacement hysteresis loop of a friction damped model, (Aliyeh Jowrkesh Safai, 2001)

3.5 CLASSIFICATION OF SEISMIC ANALYSIS METHODS

3.5.1 Free Vibration Analysis

Free vibration analysis is needed to understand the character of the structures for dynamic impact. The natural periods and mode shapes are the most important factors to determine the dynamic characteristics. The factors are calculated by finding the ratio between the periods of earthquakes and the natural periods of the structures. The formulas for damping ratio ξ, frequency ratio β and dynamic amplification factors D (Chopra, 2002) are shown as below:

$$D = \frac{1}{\sqrt{(1-\beta^2)^2 + (2\xi\beta)^2}} \qquad \textbf{Eq. 3.7}$$

Where $\xi = C/Cr$, (C is structure damping and Cr is critical damping), and $\beta = \bar{\omega}|/\omega$ where $\bar{\omega}$ is the frequency of earthquake excitation and ω is the angular frequency of structure. The plot of dynamic amplification equation is shown in the Figure 14. It shows that the resonance effects occur at β = 1. If D is equal to 1 the response of structure in dynamics is equivalent to the static response. However if D is less than 1 there is no structural response to the earthquake load.

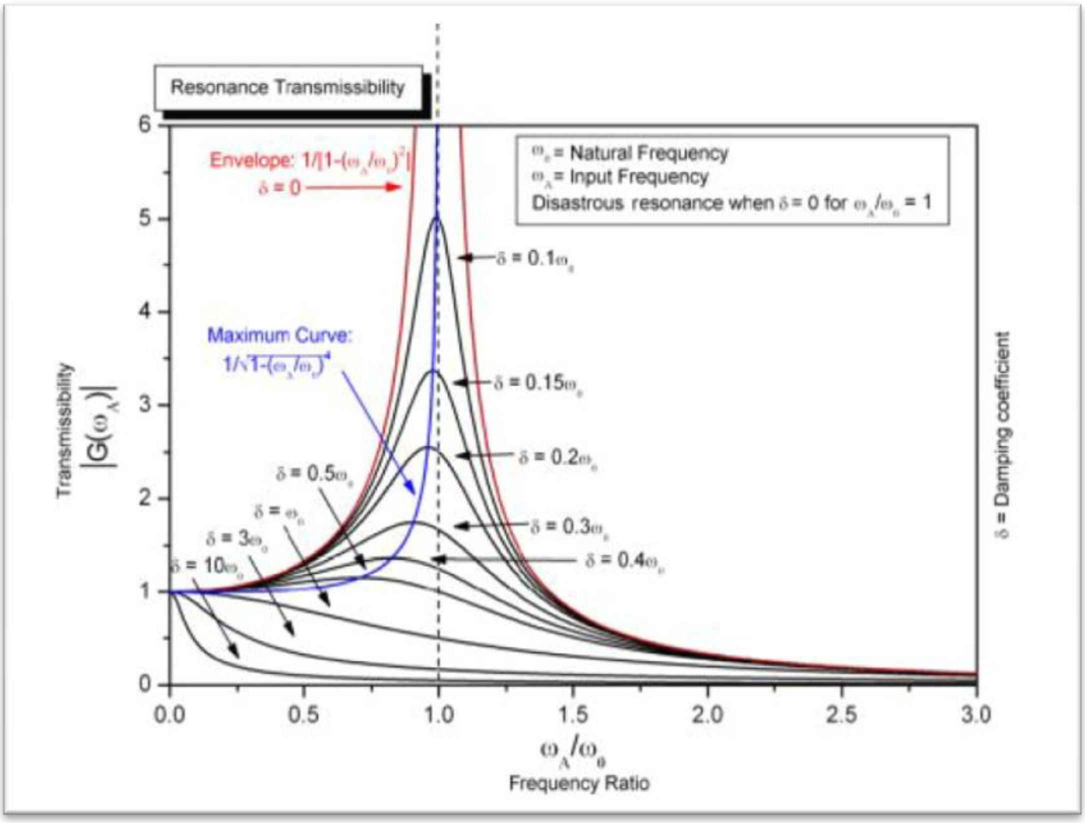


Figure 14 The plot of dynamic amplification equation, (Fabriciuss. I. 2010)

It is noticeable that, free vibration analysis is carried out to obtain natural frequencies while time history and response spectrum analysis that will appear shortly, are carried out to obtain response of structure.

3.5.2 Time History Analysis

The calculation of structural response as a function of time when the system is subjected to a given acceleration $(\ddot{u}_g(t))$, is called Time History Analysis or response history analysis (THA).The equation of motion for multistory buildings with rigid floor diaphragms and plans having two orthogonal axes of symmetry subjected to horizontal ground motion along one of those axes is presented as follow.

$$m\ddot{u} + c\dot{u} + ku = -m1\ddot{u}_g(t) \qquad \textbf{Eq. 3.8}$$

Where **u** and $\dot{u}$ and $\ddot{u}$ are the vector of lateral floor displacements relative to the ground, velocities and accelerations, respectively. **m** is a diagonal mass matrix, **c** is a damping matrix of the building and k is the lateral stiffness matrix of the building, (Chopra 2002).

3.6 ETABS Program

ETABS is an engineering software product that caters to multi-story building analysis and design. Modeling tools and templates, code-based load prescriptions, analysis methods and solution techniques, all coordinate with the grid-like geometry unique to this class of structure. Basic or advanced systems under static or dynamic conditions may be evaluated using ETABS. For a sophisticated assessment of seismic performance, modal and direct-integration time-history analyses may couple with P-Delta and Large Displacement effects (Computers & Structures, CSI America)

3.7 SUMMARY OF THEORETICAL BACKGROUND

Idealization of the modeling analysis must satisfy argument of response components arrangement that has been discussed above. The above discussion has been focused about the linear and non-linear behavior of friction damper to the overall system.

CHAPTER 4

METHODOLOGY

4.1 GENERAL

Structural collapse occurs if vertical load – carrying elements fail in compression and if shear transfer is lost between horizontal and vertical elements, such as shear failure between flat slabs and columns. In addition, collapse may also be caused by global instability. Individual stories may exhibit excessive lateral displacements and second – order P-Δ effects significantly increase overturning moments, especially in columns at lower stories. Therefore it can be concluded, that by decreasing the story drifts of the structure, the main reason of collapse can be solved. As it was mentioned before, devices like friction dampers can play a significant role to reduce response of structures.

4.2 PLANNING OF STUDY

The purpose of this study is to evaluate the seismic behavior of high rise buildings by using friction dampers. A case study approach is chosen for modeling and analyzing that is the “Sinani Tower” in Tirana. As it was mentioned earlier, the design and analysis is performed by using ETABS software. The building it is modeled with and without friction dampers, and then, the response spectrum is compared within the two models. In order to understand the seismic behavior of the building, free vibration, response spectrum and time history analysis are considered to be performed. For the purpose of analysis, for Time History analysis, El Centro earthquake (18/05/1940) that has been recorded is considered.

For more understanding are listed the stages of this study.

Stage 1: Explaining of study

Stage 2: Literatures, collecting data and modeling

Stage 3: Verification of retrofitting devices and modeling

Stage 4: Seismic analysis

Stage 5: Discussion and conclusion

4.3 GATHERING OF INFORMATION AND DATA

For this study, data collection works for the structure have been done in order to obtain the significant information for modeling and analysis. The information is as follow:

- ✓ Background of building
- ✓ Location of building
- ✓ Configuration of building
- ✓ Materials that are used for the building and their properties such as strength, modulus of elasticity and etc.
- ✓ The limitations such as displacements and stresses

4.3.1 Description of building

As it was mentioned earlier, the building that has been considered for modeling with friction damper devices is “Sinani Tower” building. The “Sinani Tower” building is a 9 story building in Tirana. One level is underground and it is used as a parking garage, and other levels are residential spaces. The construction of this building has been finished at 2006. The height of the building from the street is 29.6 m.

Figure 15 View from the south (Scale 1:100)

4.4 MODELING BY ETABS

Modeling of this building structure has been done by ETABS (Figure 15). For the generation of the structure model for analysis, the model contains the following features:

- ✓ The geometry, connectivity, member types and sized for the structure are correctly and accurately defined in the computer model
- ✓ All other relevant members, sections and group properties and characteristics are accurately defined
- ✓ All restrains are accurately defined so that the stiffness of structural members correctly transfers the loads such as vertical and lateral

In addition, during the modeling and studying some basic assumptions are made. These assumptions are listed as follow:

i. Center of rigidity and center of gravity match the same point as the geometry of the structure is almost symmetric.
ii. All restrains that have been modeled are assumed to be fixed
iii. Only ground acceleration of X-direction is taken into account

iv. Since the stiffness of floor slabs, partition walls and other secondary members are very small in relative to beams and columns of the frame, their contribution to the overall structure is not taken into consideration.

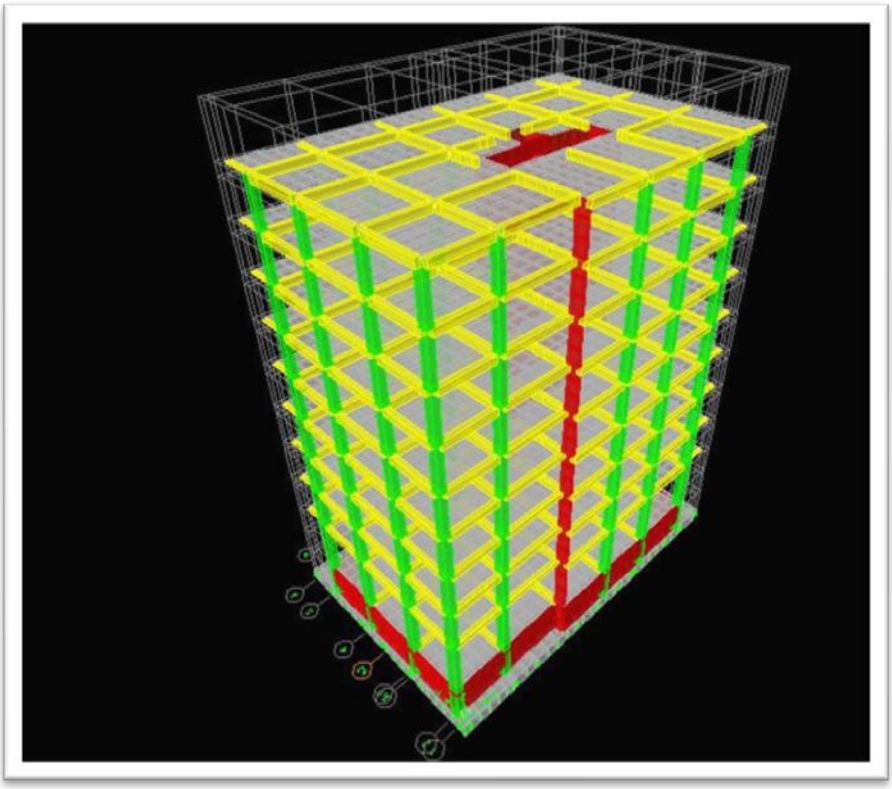


Figure 16 Modeled structure in ETABS

4.4.1 Material Properties

The following material properties have been used for the structure modeling and analysis:

Table 4 Properties of materials

No.	Material Properties	Values
1	Mass per unit volume of concrete	2400 kg/m^3
2	Weight per unit volume of concrete	23.5616 Mpa
3	Strength of concrete (f_{ck})	30 Mpa
4	Modulus of Elasticity of concrete (E_c)	32000000 kN/m^2
5	Mass per unit volume of steel	7.8271 kg/m3
6	Weight per unit volume of steel	76.8195 Mpa

7	Modulus of Elasticity of steel (E_s)	1.999E+08 kN/m^2
8	Yield stress of bracing steel (F_y), ASTM 99 F_y50	344.737 Mpa
9	Ultimate stress of bracing steel (F_u), ASTM 99 F_y50	448.159 Mpa
10	Poisson's Ratio of concrete (ν_c)	0.2
11	Poisson's Ratio of steel (ν_s)	0.3

4.5 FRICTION DAMPER

Conventional concrete shear walls are very rigid and attract higher ground accelerations during an earthquake, causing higher inertial forces on the structure. Supplemental damping in conjunction with appropriate stiffness offered an innovative solution for the seismic control of this building. This was achieved by incorporating Pall Friction Dampers in steel bracing. As soon as the structure undergoes small deformations, the friction dampers are activated and start dissipating energy. Since the dampers dissipate a major portion of the seismic energy, the forces exerted on the structure are considerably reduced. In contrast to shear walls, the friction – damped bracing need not to be vertically continuous. This aspect was particularly appealing to the project architects as it offered great flexibility in space planning.

Since the damped bracing do not carry gravity load, they do not need to go down through the basement to the foundation. This allows more open spaces for car parking in the basement. At the ground floor level, the lateral shear from the bracing is transferred through the rigid floor diaphragm to the perimeter retaining wall of the basement. The computer modeling of Pall Friction Dampers is very easy. Since the hysteretic loop of the friction dampers is perfectly rectangular, similar to perfectly elasto-plastic material, the friction dampers can be modeled as fictitious plasticity element having yield force equal to slip load. (Avtar Pall).

ETABS / SAP2000, v8 Nonlinear

- The single diagonal tension/compression brace with friction damper (Figure 17) can be modeled as a damped brace using the following link properties as shown in Table 5:

Table 5 Link properties of the single diagonal tension/compression brace with friction damper

Name	PFD1
Type	Plastic 1
M	Mass of damped brace
W	Weight of damped brace
Rotational Inertia 1,2,3	Name
Direction U1	Check nonlinear box
Ke (Effective Stiffness)	AE/L
Ce (Effective damping)	0
K (stiffness)	Ke
Yield Strength	Slip load of friction damper
Post Yield Stiffness Ratio	0.0001
Yielding Exponent	10

- The **chevron friction damper** (Figure 19) can be modeled using the following link properties (Table 6):

Table 6 Link properties of the chevron friction damper

Name	PFD1
Type	Plastic 1
M	0.0013(Kip-inch) or 0.225 (kN-m)
W	0.5 (Kip-inch) or 2.22 (kN-m)
Rotational Inertia 1,2,3	0
Direction U1	Chech nonlinear box
Ke (Effective Stiffness)	25 x damper slip load (Kip-inch) 1000 x damper slip load (kN-m)
Ce (Effective damping)	0
K (stiffness)	Ke
Yield Strength	Slip load of friction damper
Post Yield Stiffness Ratio	0.0001
Yielding Exponent	10

In this case, the brace plus friction damper (damped brace) is modeled as link element.

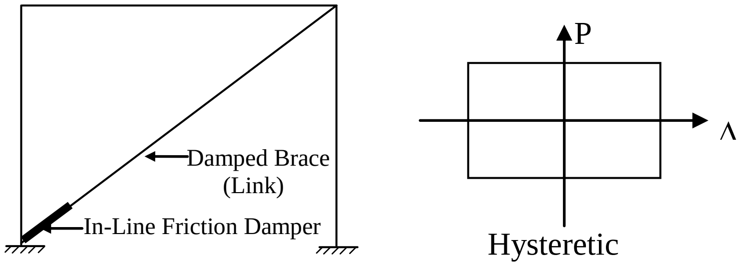


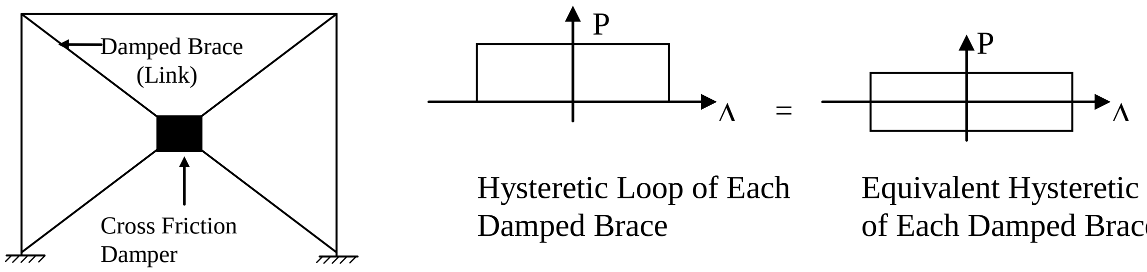
Figure 17 Single Diagonal Tension-Compression Brace with Pall Friction Damper

Hysteretic

A = Area of Brace

L = Length of Damper + Brace

The hysteretic loop of each tension brace is equal to hysteretic loop of one single diagonal tension-compression brace having half the slip load. However, the brace and the connections should be designed considering the full slip load.



Hysteretic Loop of Each Damped Brace

Equivalent Hysteretic Loop of Each Damped Brace

Figure 18 Tension Only Cross Brace with Pall Friction Damper

The brace is modeled as frame element. Braces are from joints A and E and joints B and E. The beams at top are from joints C and D and joints D and F. The friction damper is modeled as a nonlinear axial link element between joints D and E. Joint E can be, say, 0.25 mm lower and 150 mm away from joint D. Joint E is disconnected from the diaphragm otherwise the damper will not work or move. (C. Pasquin, 2012)

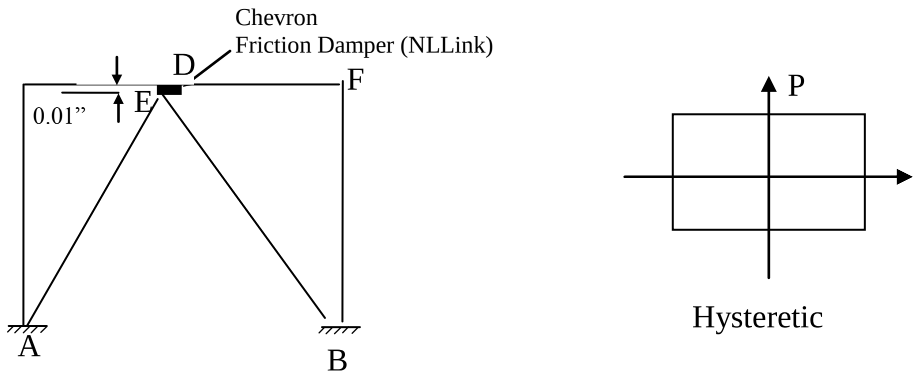


Figure 19 Chevron Brace with Pall Friction Damper

In this study the Single Diagonal Tension-Compression Brace is used to model the damped structure.

4.6 ANALYSIS

The free vibration analysis and time history earthquake analysis is performed by using ETABS software. The natural and mode shapes of the building are obtained from the free vibration analysis. From the time history analysis, the time dependent dynamic responses of the building for the whole duration of the earthquake excitation have been determined. The analysis is checked when initially performed and then checked again at the end of the project. The results of analysis of building with and without friction damper devices are presented in the next chapter.

4.7 VERIFICATION OF FINITE ELEMENT TECHNIQUE

The ETABS software is used to confirm the structural performance and analysis techniques. To achieve the objectives of this study, three dimensional analyzing has been done. The finite element approaching and analytical modeling techniques are efficient tools for the study to make sure that a proper model is being established, thus, it is able to represent the overall structural system. Nowadays, the computer engineering programs have the ability of analysis of seismic loads more effectively.

4.8 SUMMARY OF METHODOLOGY

The understanding of seismic behavior of tall building structure by friction damper devices has been done by two analysis methods, such as, free vibration and Time History analysis. The structure was modeled using ETABS software. El – Centro earthquake data (08/May/1940) that have been recorded is selected for Time History analysis to understand the seismic performance of the case study.

CHAPTER 5

RESULTS AND ANALYSIS

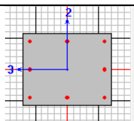
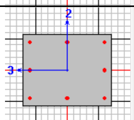
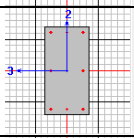
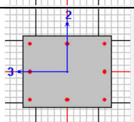
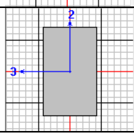
5.1 INTRODUCTION

Generally this chapter concerns about seismic behavior of tall building structures using friction damper. Thus, to achieve this purpose "Sinani Tower" building has been selected as a case study of tall building. Free vibration and Time History analysis have been done to evaluate the seismic behavior of the building with and without friction damper.

5.2 SECTION PROPERTIES

The following tables show the section properties of all the structure's members.

Table 7 Section properties

No.	Frame Element	Section	Material	Area of section
1	**Column 50x40** Depth: 0.4m Width: 0.5m		Concrete C30	0.2 m^2
2	**Column 55x45** Depth: 0.45m Width: 0.55m		Concrete C30	0.2475 m^2
3	**Column 60x30** Depth: 0.6m Width: 0.3		Concrete C30	0.18 m^2
4	**Column 60x50** Depth: 0.5m Width: 0.6m		Concrete C30	0.3 m^2
5	**Beam 30x50** Depth: 0.5m Width: 0.3m		Concrete C30	0.15 m^2

No.	Element	Thickness
1	Shear wall	0.3 m
2	Plate	1 m
3	Slab	0.15 m
4	Console	0.15 m

5.3 FREE VIBRATION ANALYSIS

Free vibration analysis is needed to understand the character of the structures for dynamic impact. The natural periods and mode shapes are the most important factors to determine the dynamic characteristics. Figures on Appendix A show the main mode shapes of the two structures.

The table below shows the summary of the mode shapes based on periods (sec) and frequencies (1/sec). The last column of the table shows the reduction in percentage of the period.

Table 8 Comparison of mode shapes based on period and frequency

Mode	Un Damped Period	Damped Period	Un Damped Frequency	Damped Frequency	Reduction (%)
1	1.0080	0.7104	0.992	1.408	±29.5 %
2	0.9253	0.6302	1.080	1.587	±31.89 %
3	0.7296	0.4683	1.371	2.135	±35.81 %
4	0.3292	0.2113	3.038	4.733	±35.81 %
5	0.2853	0.1767	3.505	5.659	±38.06 %
6	0.2193	0.1205	4.559	8.299	±45.05 %
7	0.1846	0.1151	5.417	8.688	±37.65 %
8	0.1461	0.0824	6.845	12.136	±43.60 %
9	0.1240	0.0818	8.065	12.225	±34.03 %
10	0.1141	0.0615	8.764	16.260	±46.10 %
Average					**±37.75 %**

5.4 TIME HISTORY

As mentioned previously, El – Centro earthquake data (08/May/1940) that have been recorded with PGA of 0,3109g is selected for Time History analysis to understand the seismic performance of the case study.

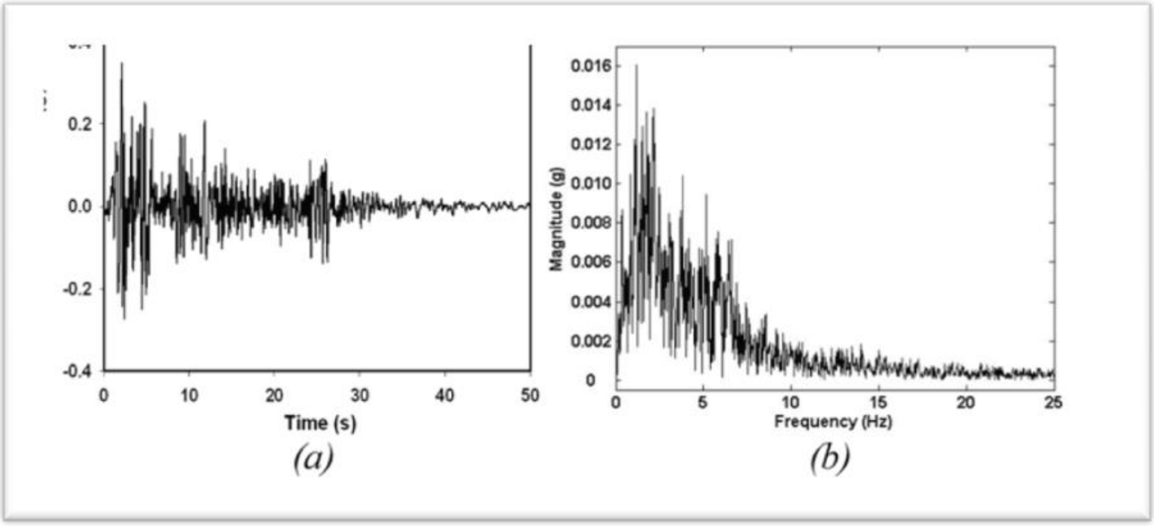
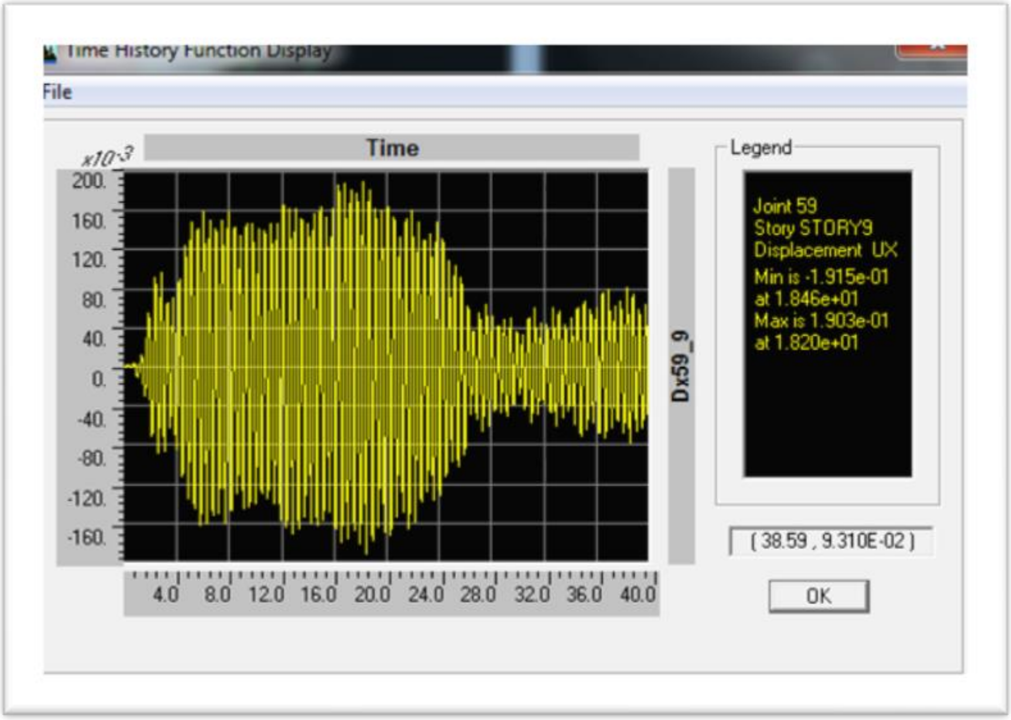
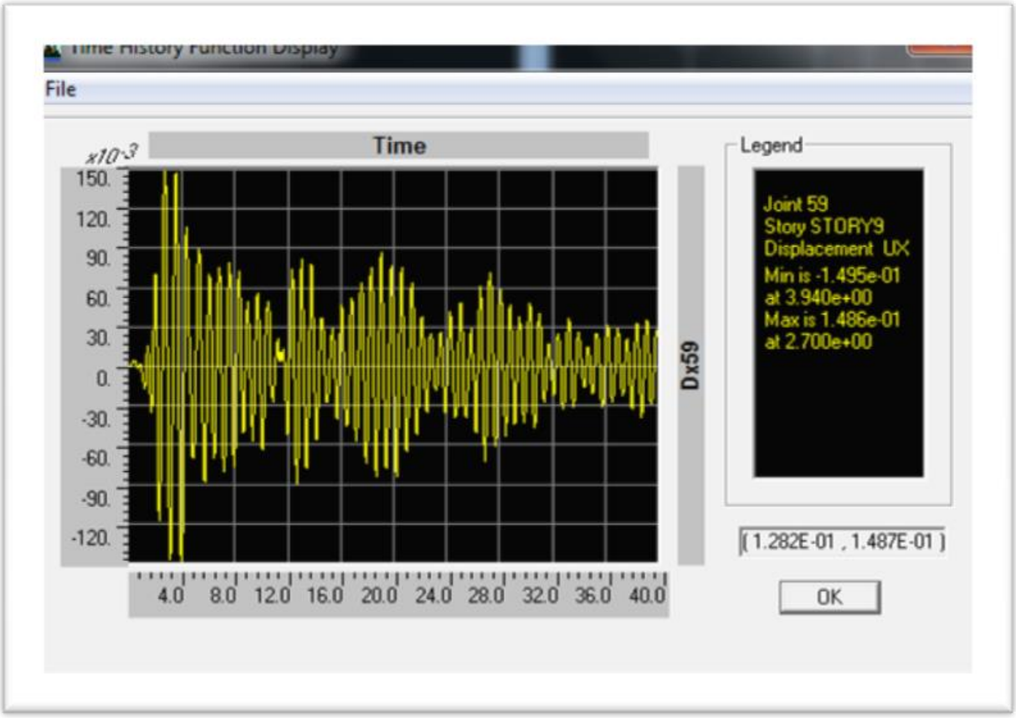


Figure 20 Ground acceleration data corresponding to the 1940 El Centro earthquake and (b) spectrum of the ground acceleration signal (Courtesy US Geological Survey)

5.4.1 Displacement of Story

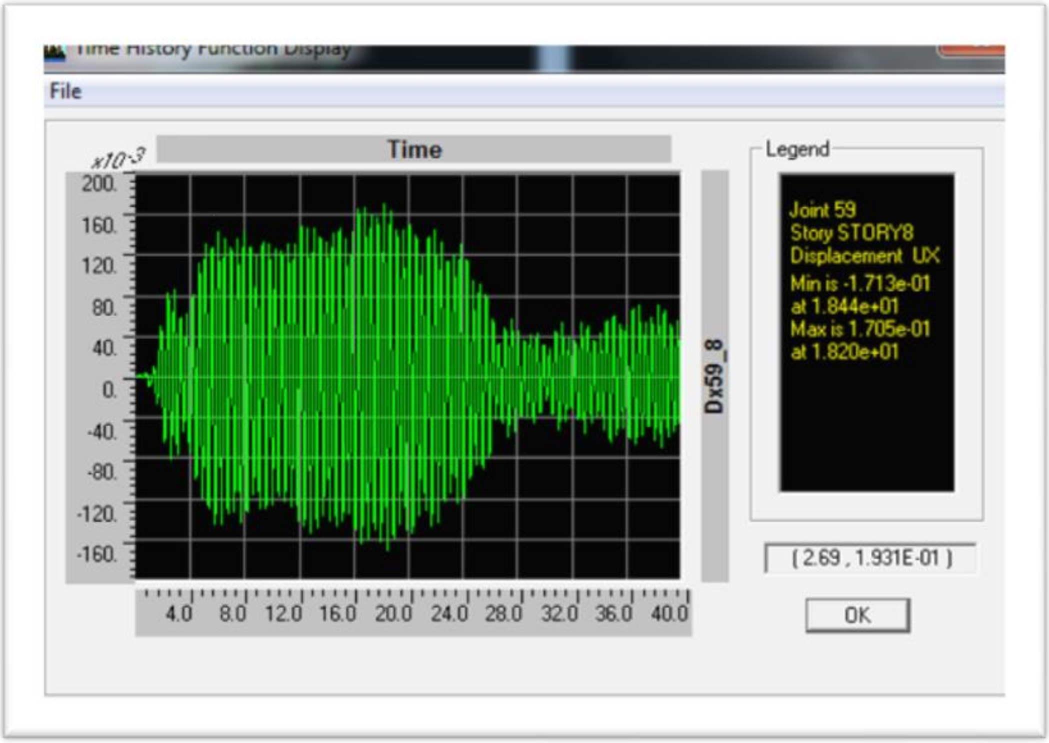


(a)

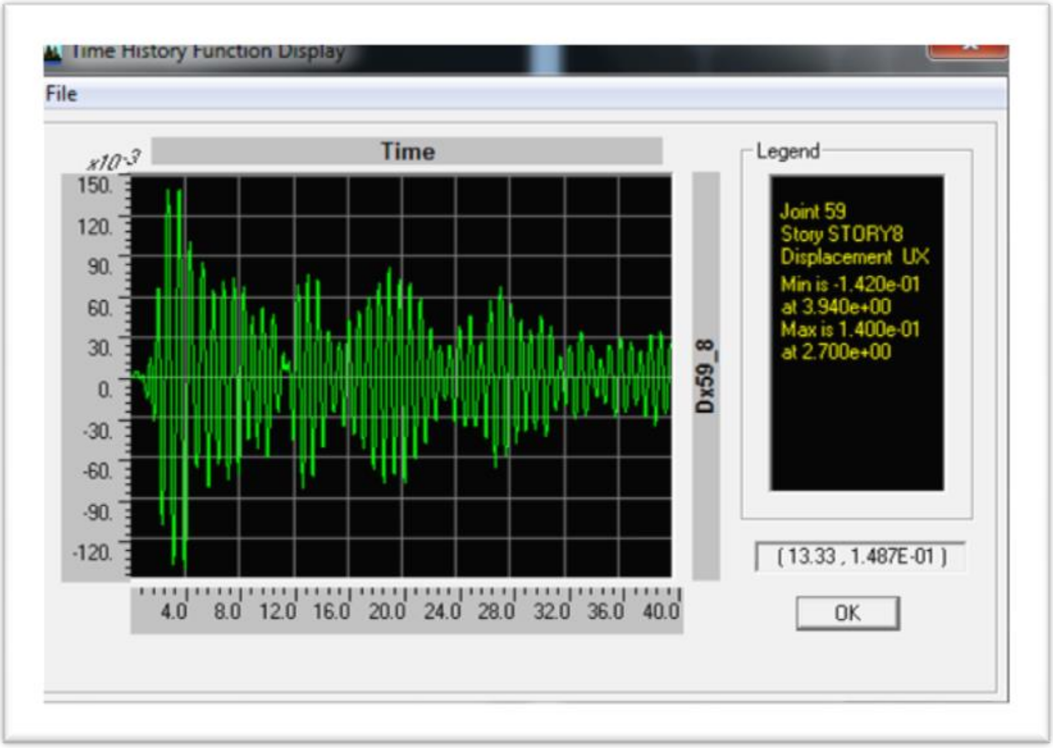


(b)

Figure 21 Displacement of the 9th story (a) Un Damped structure (b) Damped structure

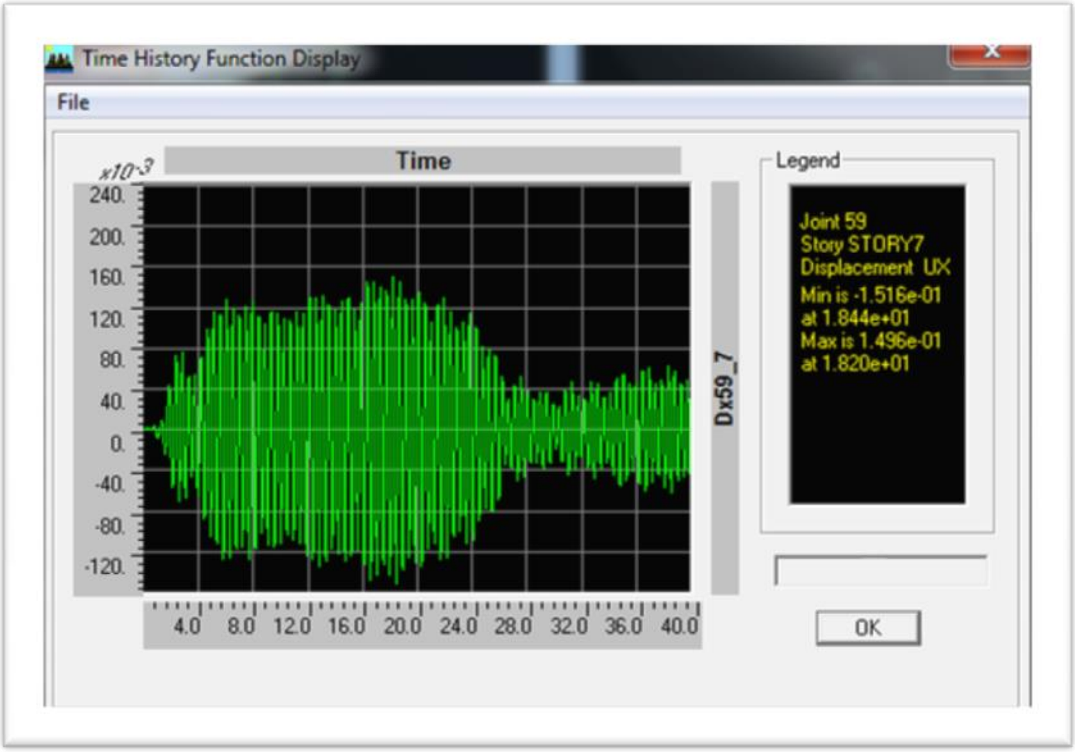


(a)

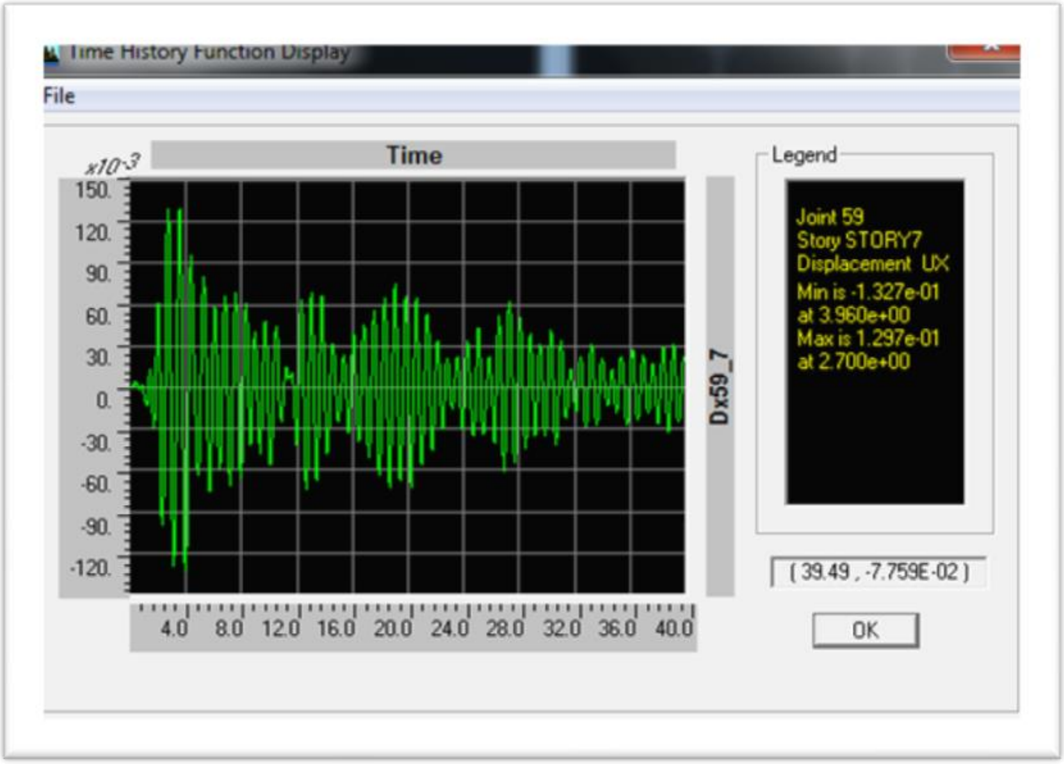


(b)

Figure 22 Displacement of 8th story (a) Un Damped structure (b) Damped structure

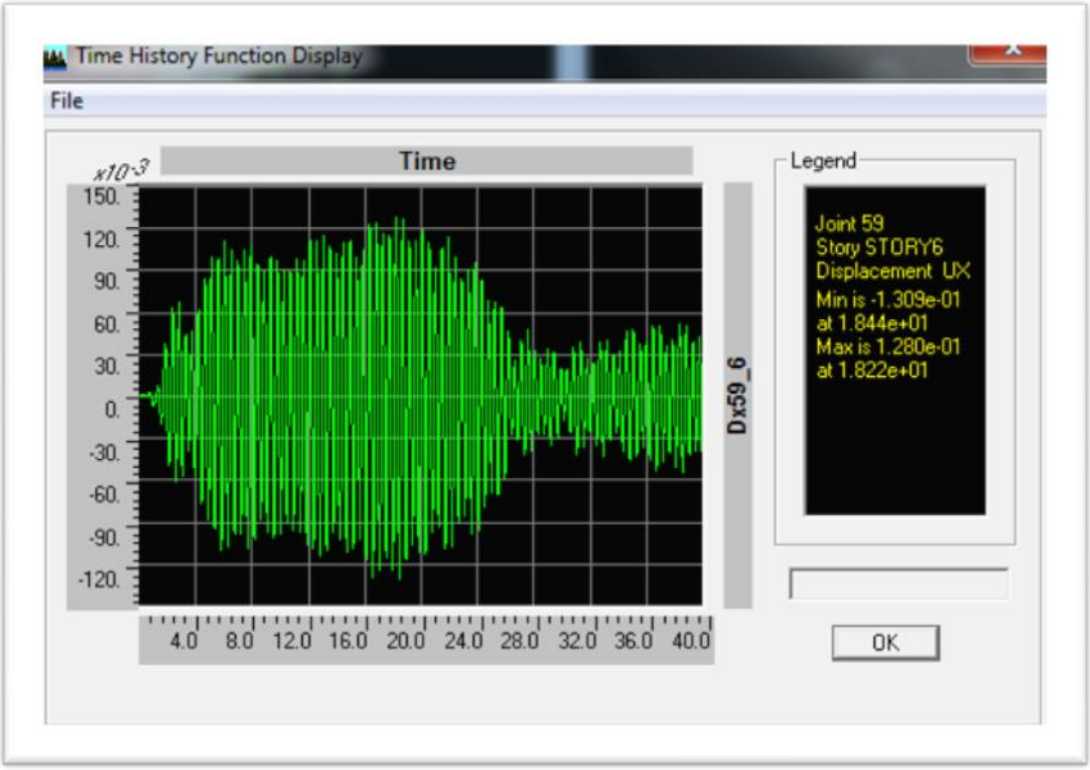


(a)

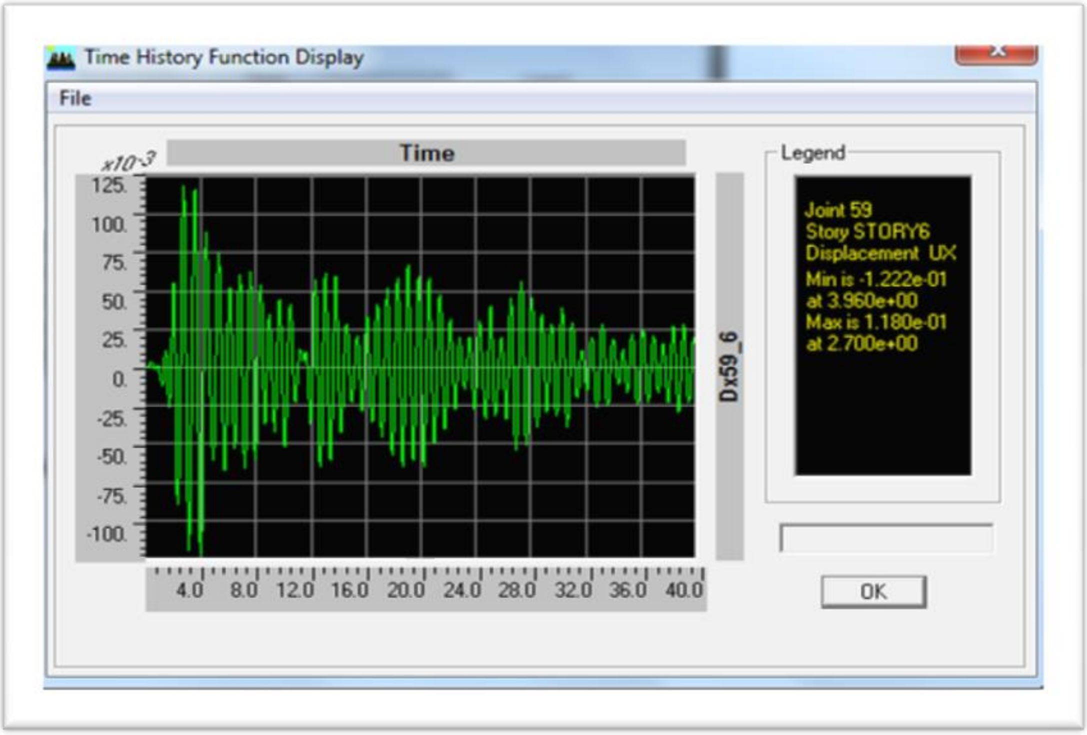


(b)

Figure 23 Displacement of 7th story (a) Un Damped structure (b) Damped structure

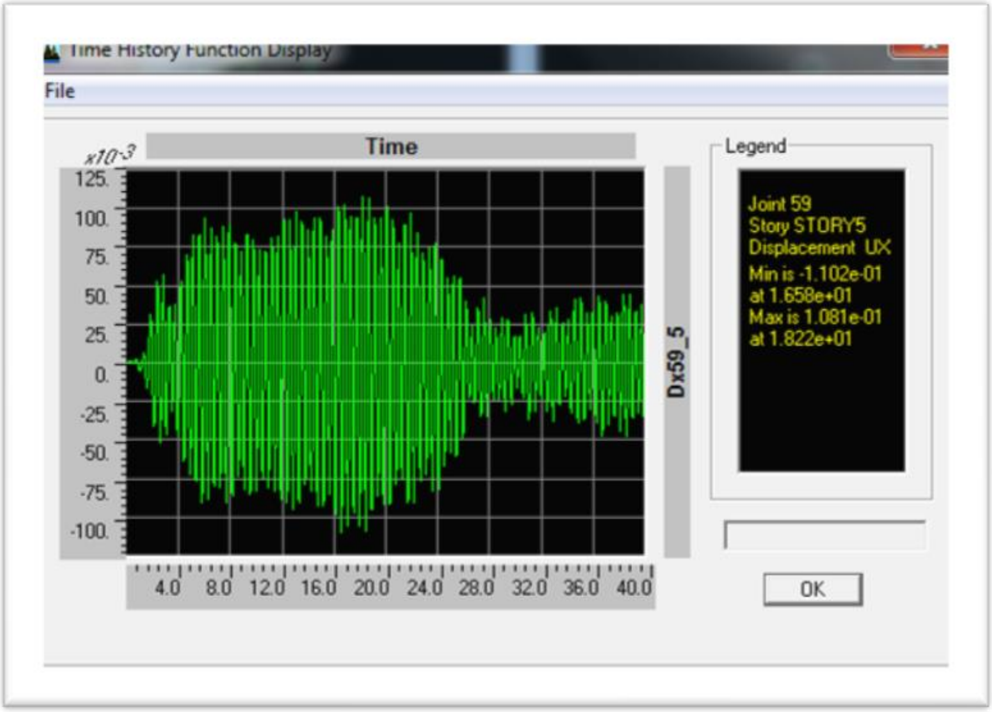


(a)

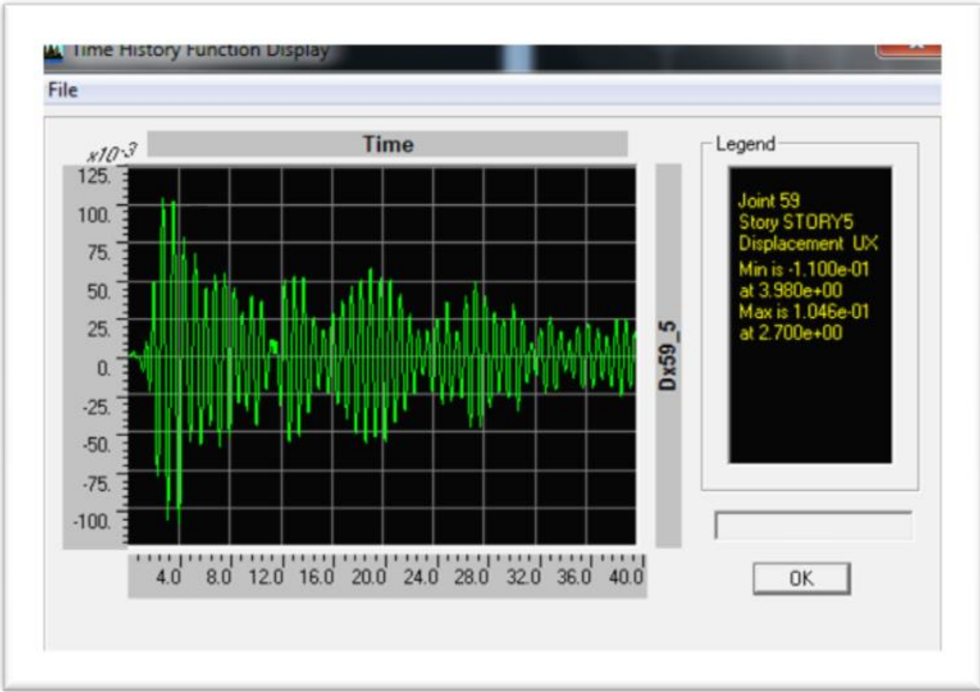


(b)

Figure 24 Displacement of 6th story (a) Un Damped structure (b) Damped structure



(a)



(b)

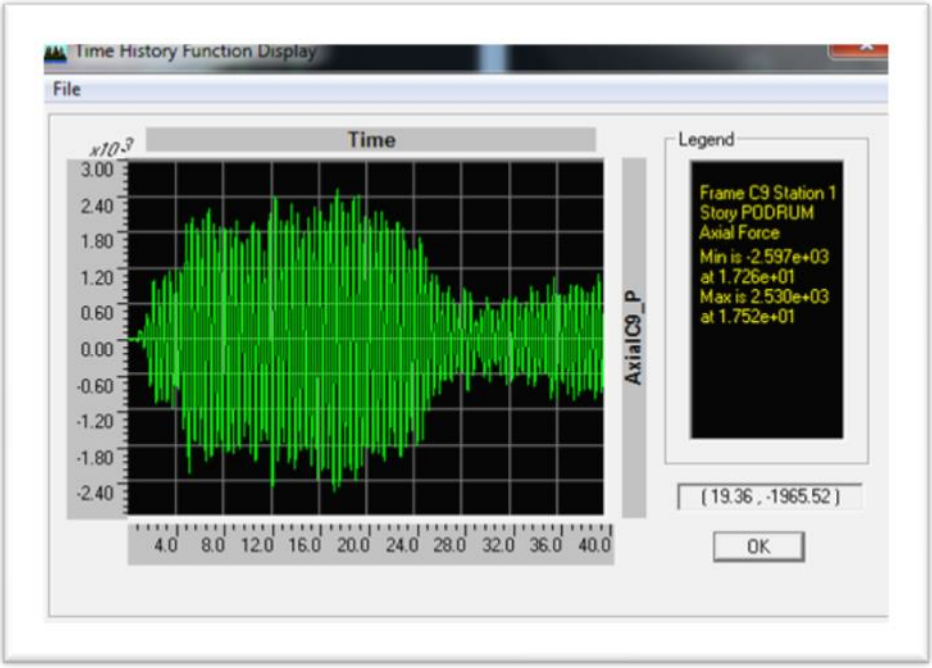
Figure 25 Displacement of 5th story (a) Un Damped structure (b) Damped structure

This procedure was conducted for the same Joint (Point 59) in each floor. The table below shows the maximum and minimum displacement (in m) of this point in each floor. The last column shows the reduction in percentage.

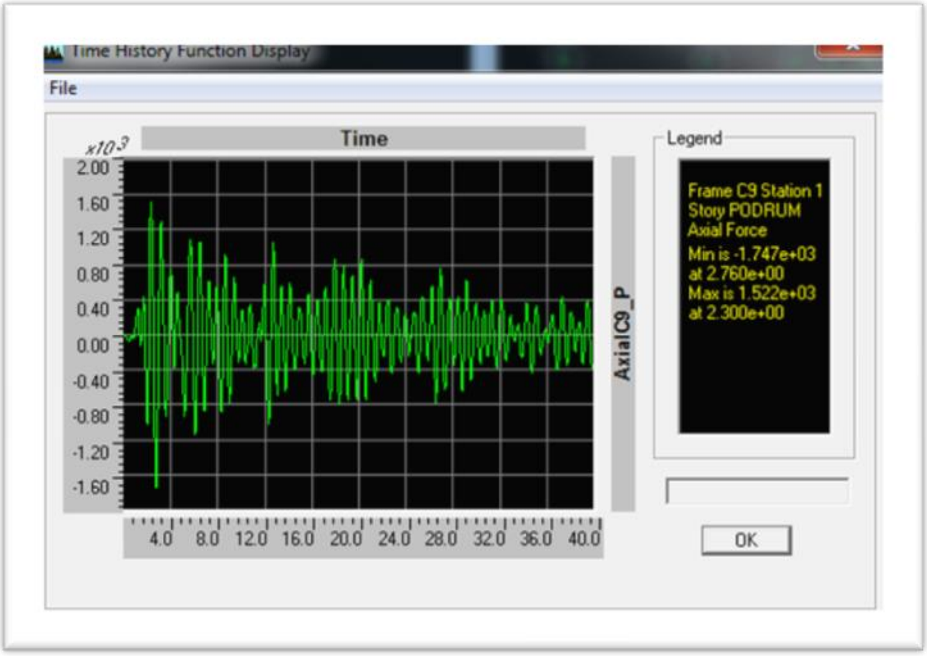
Table 9 Comparison of results of displacement

Story	Point	Un Damped	Damped	Reduction
9	59	Min: -0.1903	Min: -0.1495	±23.9%
		Max: 0.19	Max: 0.14	
8	59	Min: -0.1713	Min: -0.142	±17.35%
		Max: 0.17	Max: 0.14	
7	59	Min: -0.1516	Min: -0.1327	±12.9%
		Max:0.1496	Max: 0.1297	
6	59	Min: -0.1309	Min: -0.1222	±7.2%
		Max: 0.128	Max:0.118	
5	59	Min: -0.1102	Min: -0.11	±0.27%
		Max: 0.1081	Max: 0.1046	
4	59	Min: -0.0964	Min: -0.09	±4.6%
		Max: 0.09	Max: 0.08761	
3	59	Min: -0.08102	Min: -0.06945	±12.05%
		Max: 0.07475	Max: 0.06728	
2	59	Min: -0.06339	Min: -0.04924	±21.4%
		Max: 0.0578	Max: 0.04754	
1	59	Min: -0.04371	Min: -0.03054	±27.95%
		Max: 0.03961	Max: 0.02938	
Base	59	Min: -0.01084	Min: -0.00734	±31.8%
		Max: 0.01028	Max: 0.00705	
Average				**±15.9%**

5.4.2 Axial Load of Columns

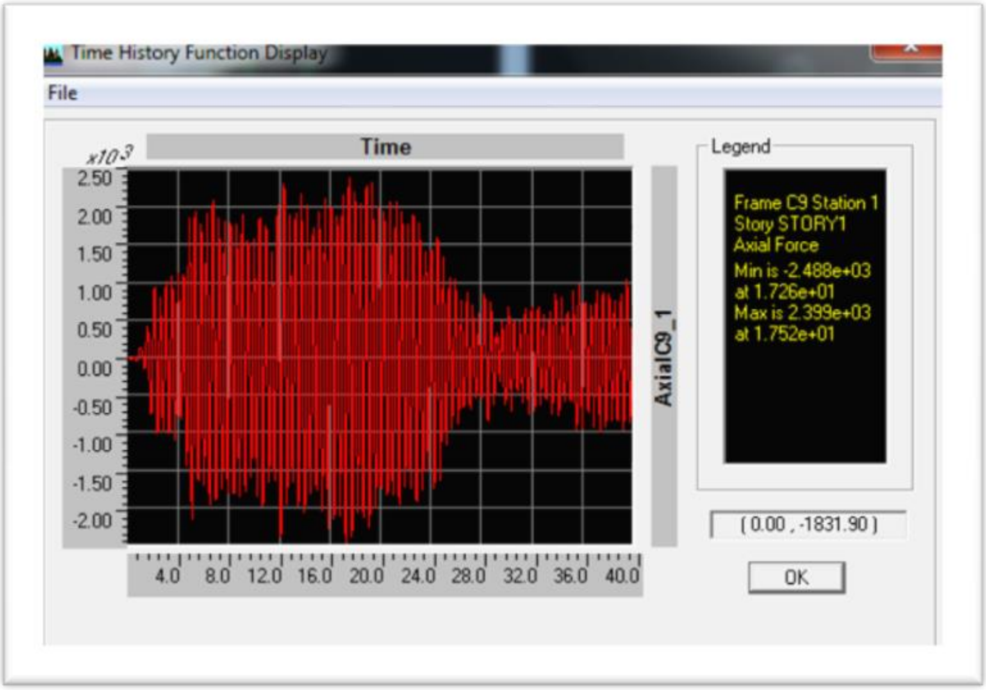


(a)

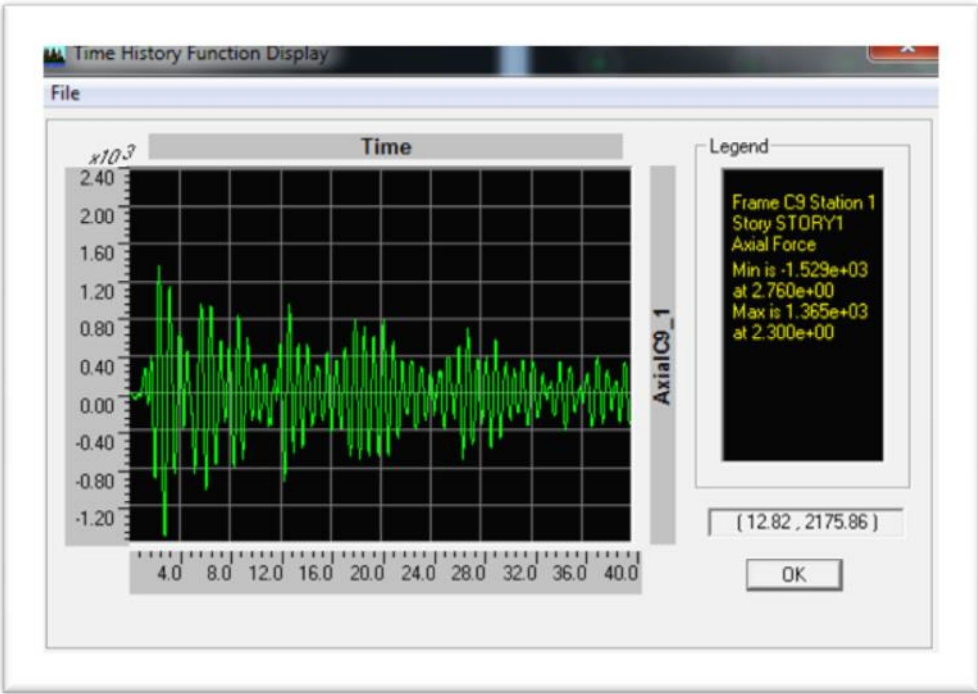


(b)

Figure 26 Axial load of the base story column (a) Un Damped structure (b) Damped structure

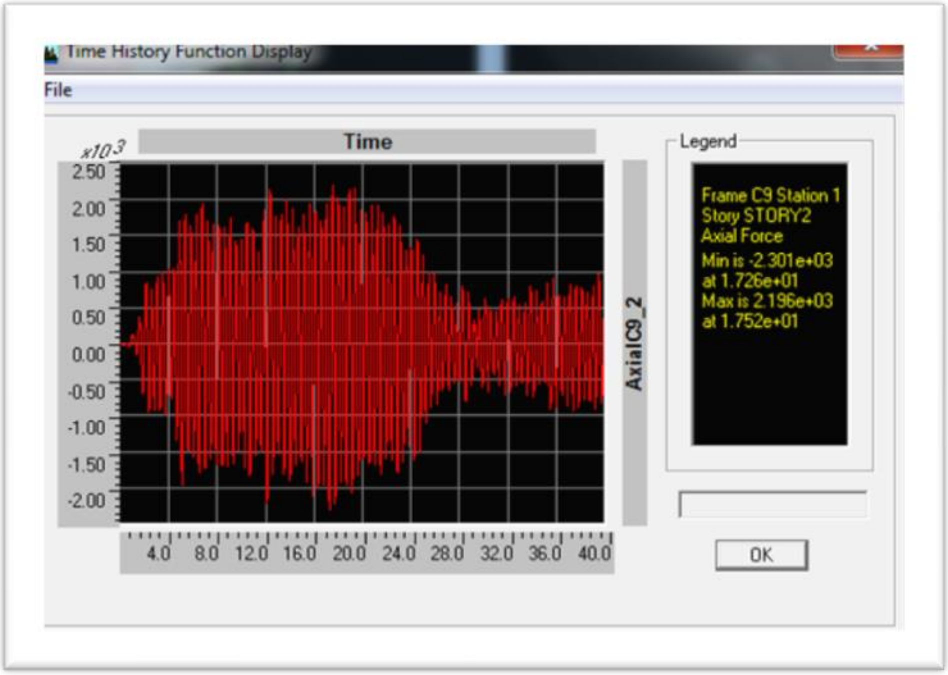


(a)

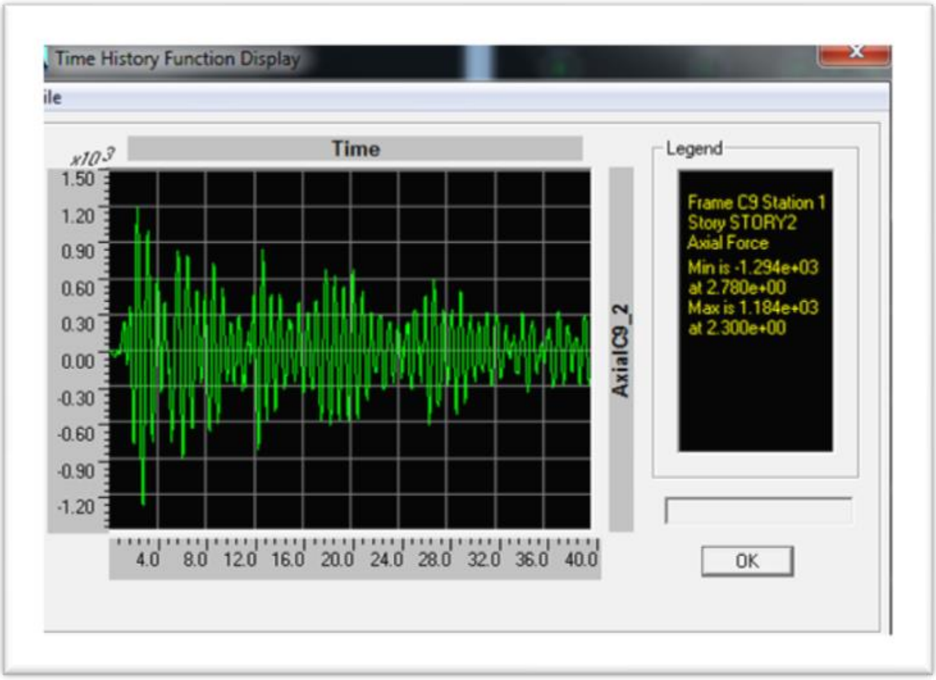


(b)

Figure 27 Axial load of the 1st story column (a) Un Damped structure (b) Damped structure



(a)



(b)

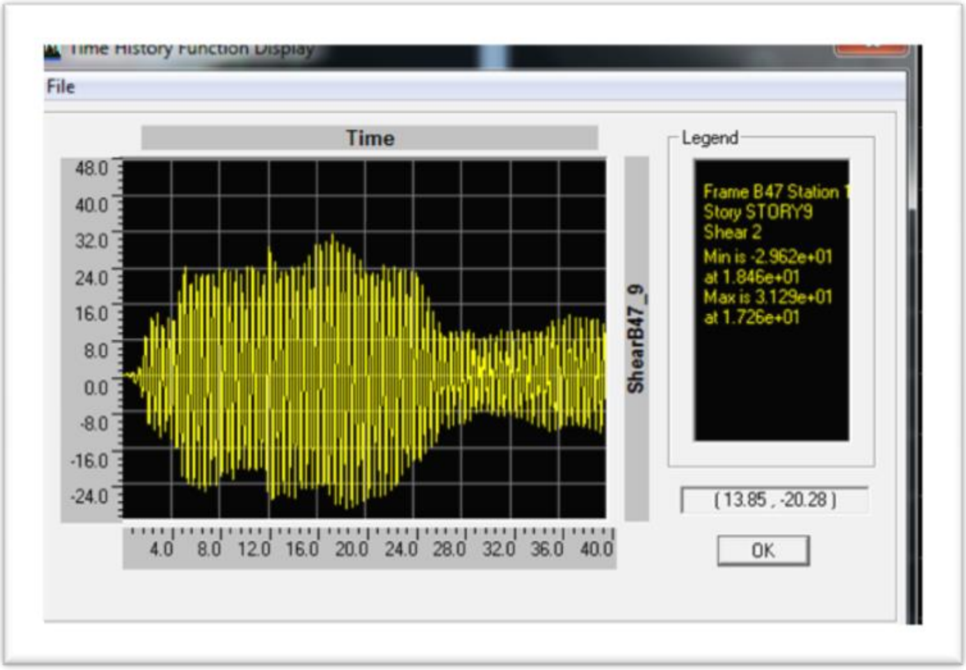
Figure 28 Axial load of the 2nd story column (a) Un Damped structure (b) Damped structure

This procedure was conducted for the same Column (C9) in each floor. The table below shows the maximum and minimum axial load (in kN) of this column in each floor. The last column shows the reduction in percentage.

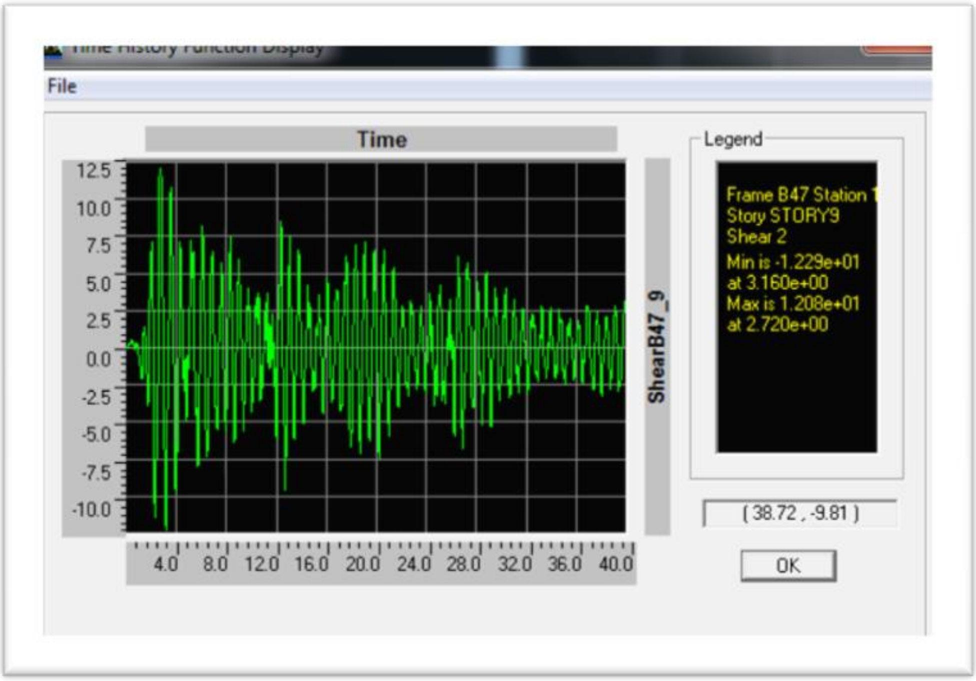
Table 10 Comparison of results for axial load

Story	Point	Un Damped	Damped	Reduction
9	C9	Min: -189.9	Min: -71.78	±61.2%
		Max: 178.7	Max: 70.99	
8	C9	Min: -482.7	Min: -190.3	±59.5%
		Max: 454.3	Max: 188.7	
7	C9	Min: -793.6	Min: -329.9	±57.35%
		Max: 747.5	Max: 326.6	
6	C9	Min: -1126	Min: -490.1	±55.45%
		Max: 1061	Max: 483.3	
5	C9	Min: -1442	Min: -663.1	±53.25%
		Max: 1358	Max: 645.4	
4	C9	Min: -1755	Min: -856.2	±50.75%
		Max: 1650	Max: 819.9	
3	C9	Min: -2050	Min: -1073	±47.9%
		Max: 1934	Max: 1003	
2	C9	Min: -2301	Min: -1294	±44.9%
		Max: 2196	Max: 1184	
1	C9	Min: -2488	Min: -1529	±40.8%
		Max: 2399	Max: 1365	
Base	C9	Min: -2597	Min: -1747	±36.38%
		Max: 2530	Max: 1522	
Average				**±50.75%**

5.4.3 Shear Load of beams

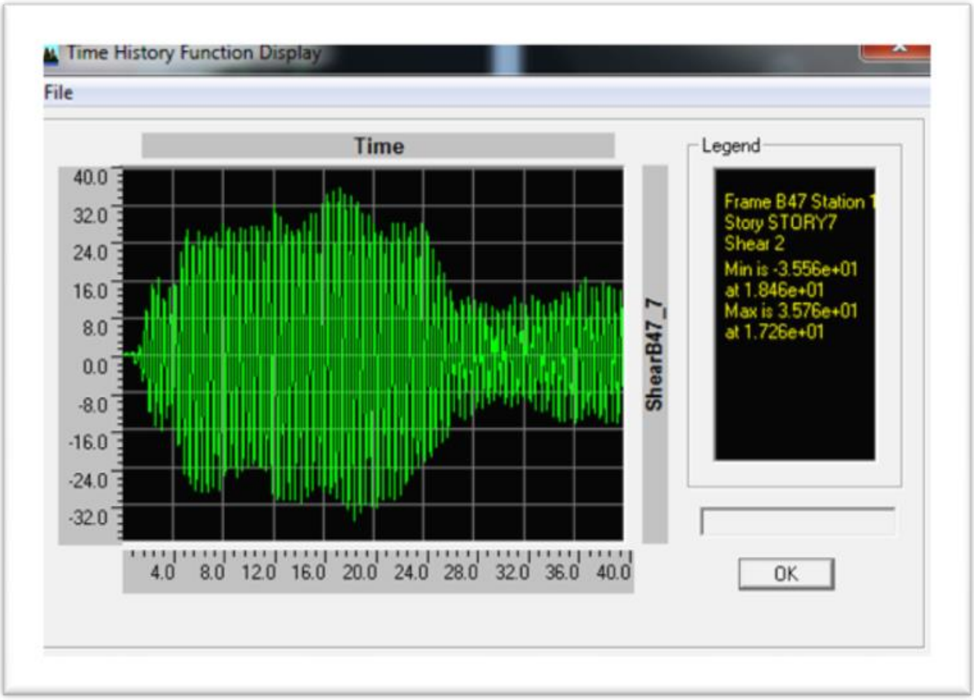


(a)

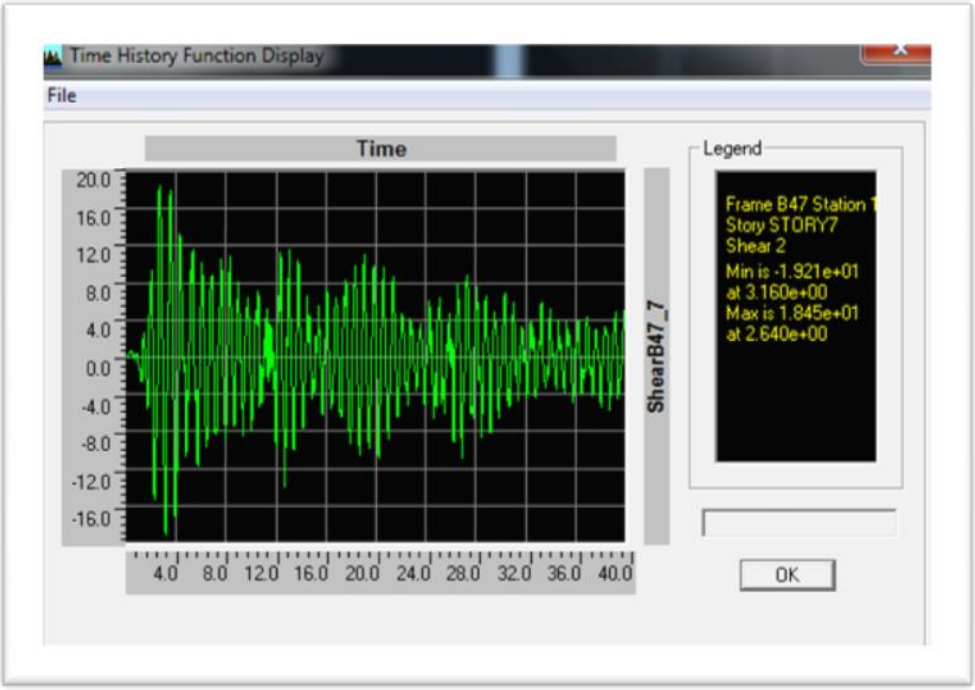


(b)

Figure 29 Shear load of the 9th story beam (a) Un Damped structure (b) Damped structure

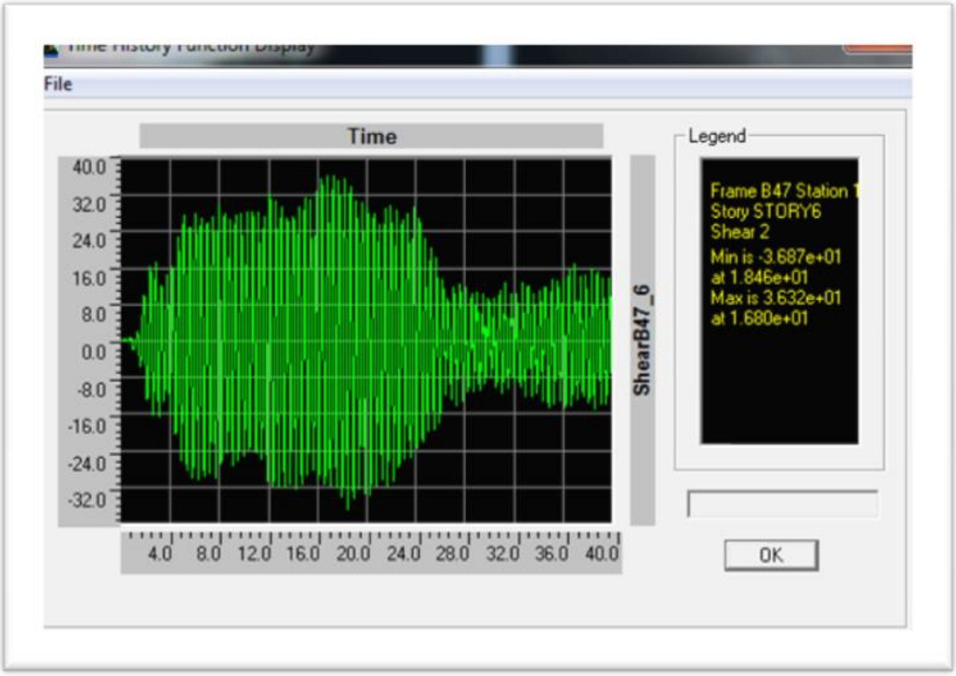


(a)

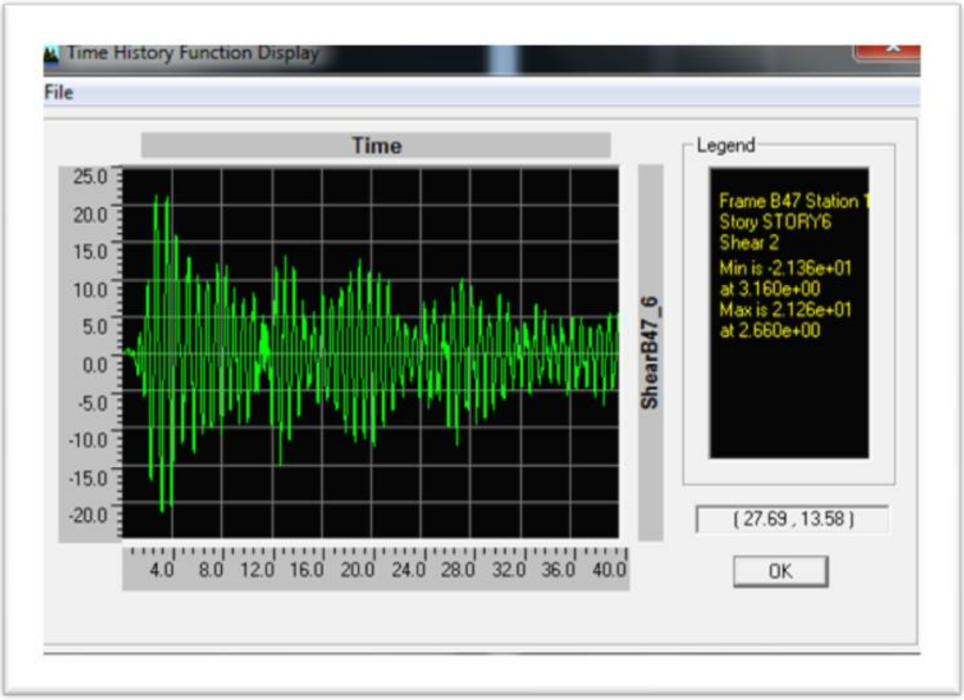


(b)

Figure 30 Shear load of the 7th story beam (a) Un Damped structure (b) Damped structure



(a)



(b)

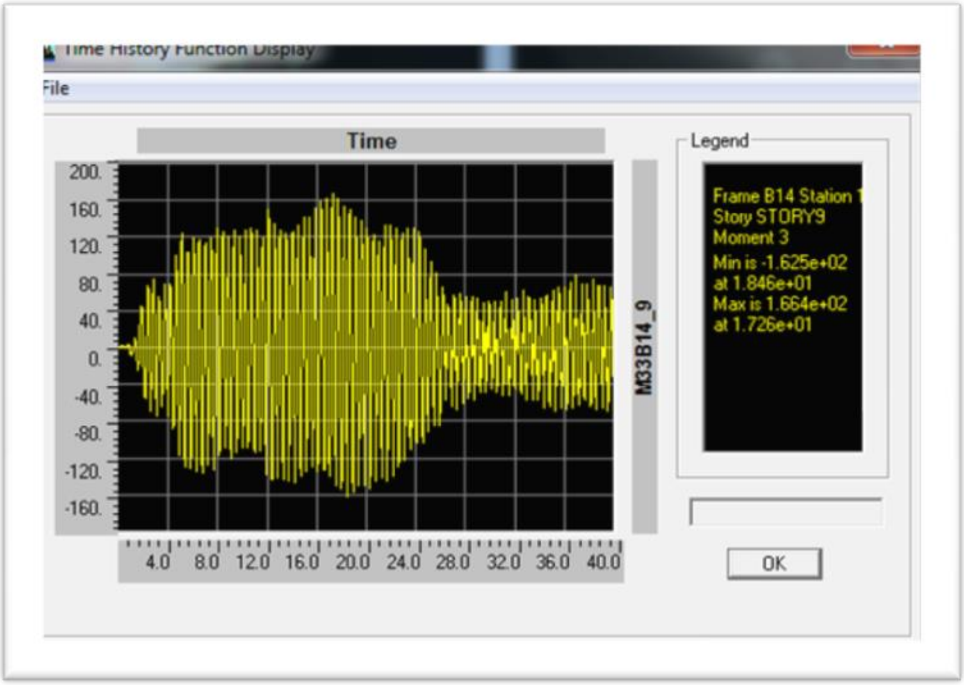
Figure 31 Shear load of the 6th story beam (a) Un Damped structure (b) Damped structure

This procedure was conducted for the same Beam (B47) in each floor. The table below shows the maximum and minimum shear load (in kN) of this beam in each floor. The last column shows the reduction in percentage.

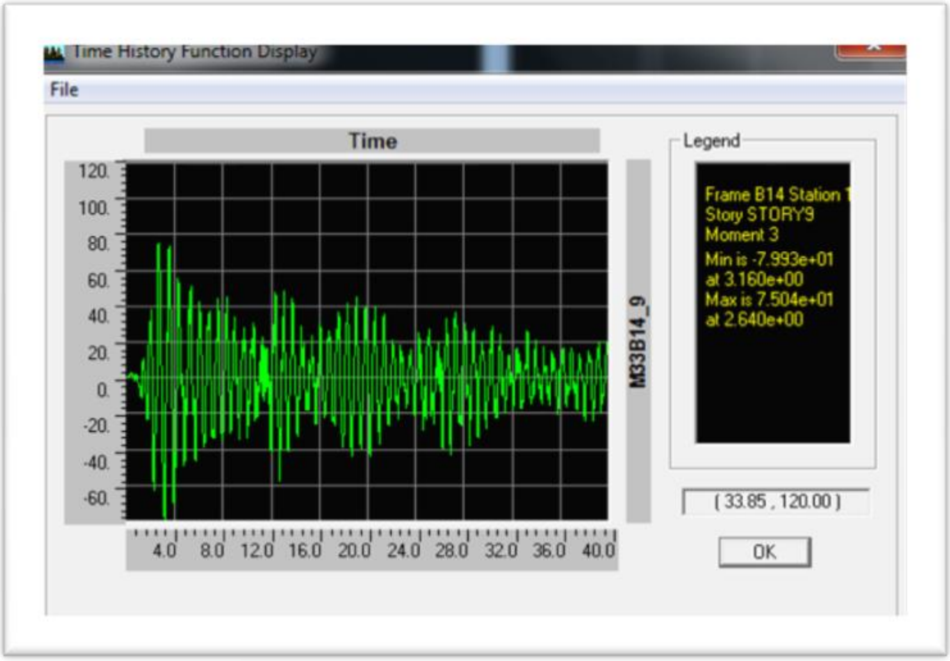
Table 11 Comparison of results for shear load

Story	Point	Un Damped	Damped	Reduction
9	C9	Min: -29.62	Min: -12.29	±59.95%
		Max: 31.29	Max: 12.08	
8	C9	Min: -33.42	Min: -16.01	±53.85%
		Max: 34.4	Max: 15.27	
7	C9	Min: -35.56	Min: -19.21	±47.19%
		Max: 35.76	Max: 18.45	
6	C9	Min: -36.87	Min: -21.36	±41.8%
		Max: 36.32	Max: 21.26	
5	C9	Min: -36.52	Min: -23.5	±34.47%
		Max: 35.67	Max: 23.8	
4	C9	Min: -35.77	Min: -26.63	±25.87%
		Max: 35.42	Max: 26.23	
3	C9	Min: -35.03	Min: -29.09	±17.52%
		Max: 34.03	Max: 27.88	
2	C9	Min: -33.06	Min: -31.69	±6.5%
		Max: 32.06	Max: 29.2	
1	C9	Min: -35.83	Min: -29.25	±14.76%
		Max: 31.92	Max: 28.36	
Base	C9	Min: -31.15	Min: -20.59	±30.74%
		Max: 27.27	Max: 19.75	
Average				**±30.74%**

5.4.4 Bending moment of beams

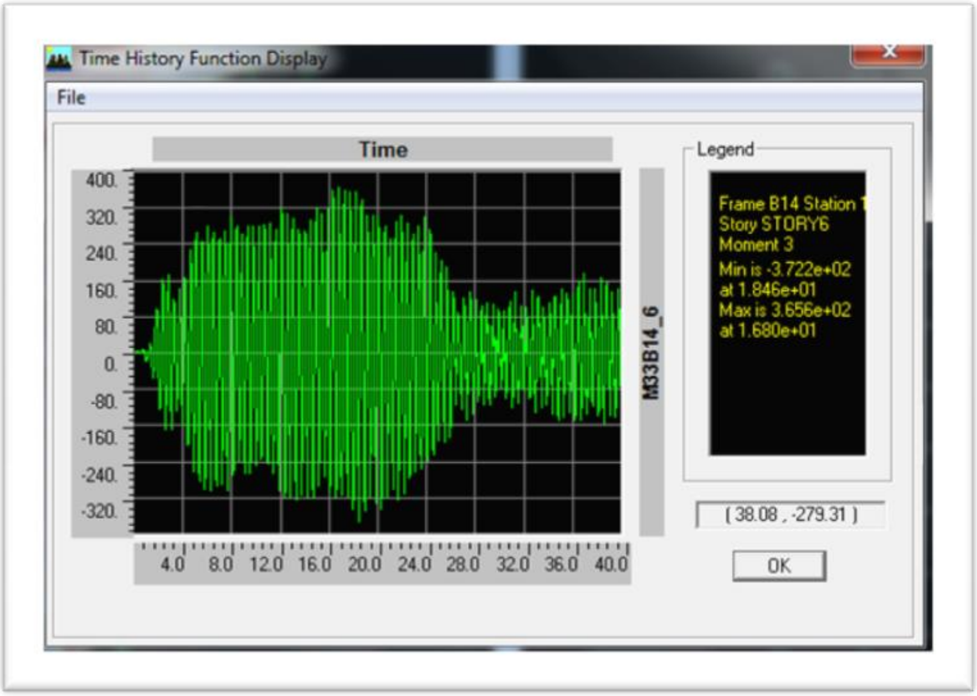


(a)

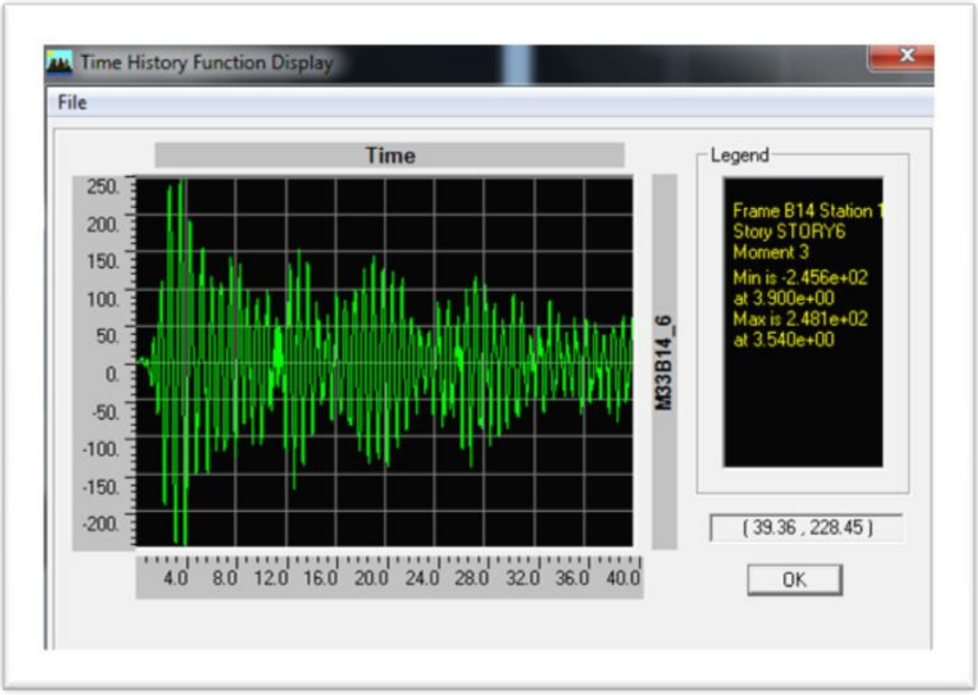


(b)

Figure 32 Bending moment of the 9th story beam (a) Un Damped structure (b) Damped structure

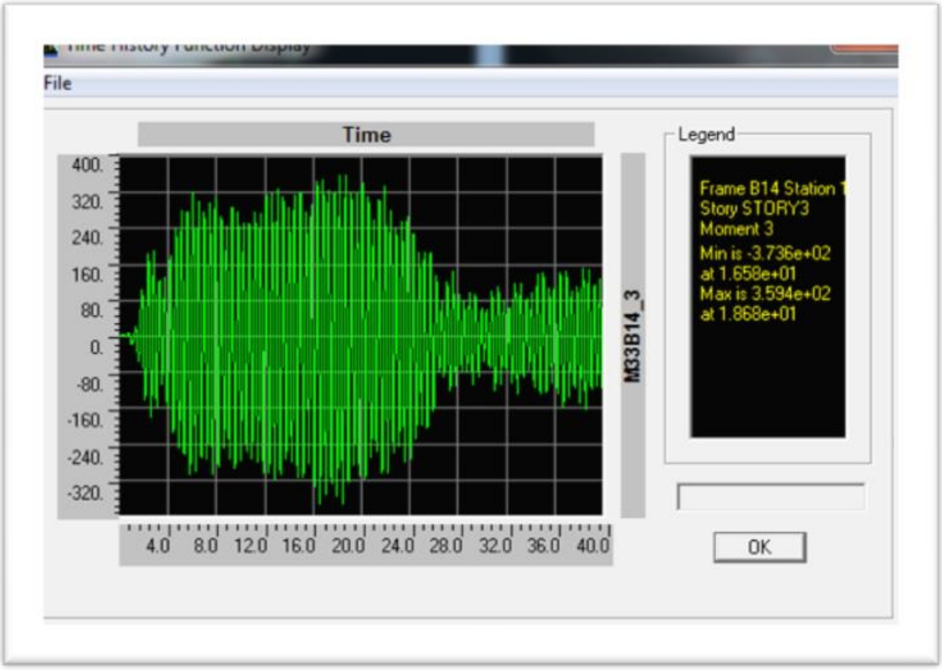


(a)

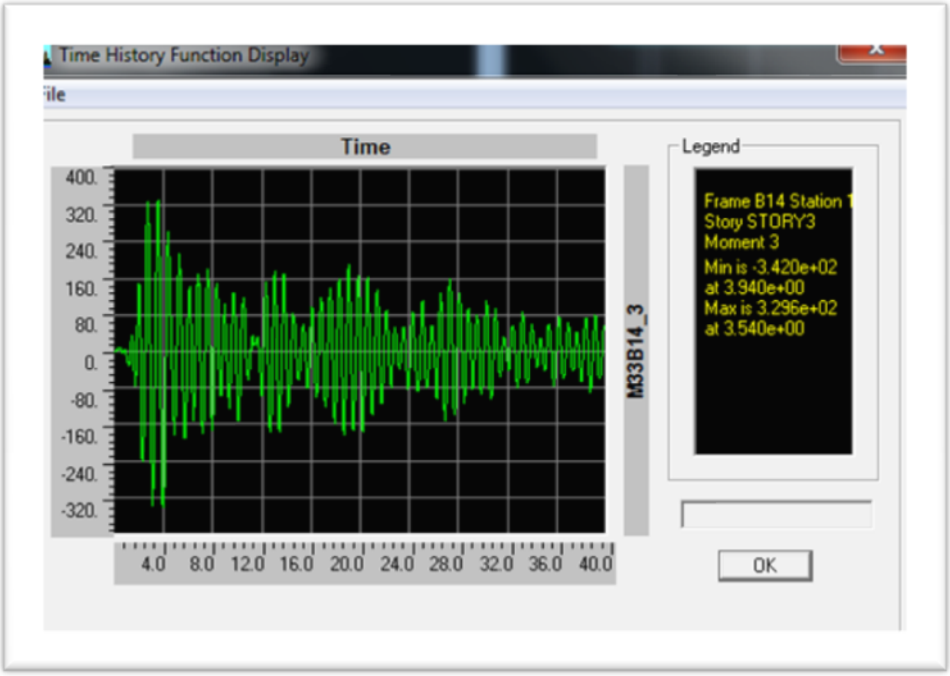


(b)

Figure 33 Bending moment of the 6th story beam (a) Un Damped structure (b) Damped structure



(a)



(b)

Figure 34 Bending moment of the 3rd story beam (a) Un Damped structure (b) Damped structure

This procedure was conducted for the same Beam (B14) in each floor. The table below shows the maximum and minimum bending moment (in kNm) of this beam in each floor. The last column shows the reduction in percentage.

Table 12 Comparison of results for bending moment

Story	Point	Un Damped	Damped	Reduction
9	C9	Min: -162.5	Min: -79.93	±52.85%
		Max: 166.4	Max: 75.04	
8	C9	Min: -329.8	Min: -180	±47.15%
		Max: 332.3	Max: 169.6	
7	C9	Min: -351.1	Min: -216.4	±39.15%
		Max: 347.4	Max: 208.1	
6	C9	Min: -372.2	Min: -245.6	±33%
		Max: 365.6	Max: 248.1	
5	C9	Min: -375.5	Min: -284.8	±23.05%
		Max: 365.4	Max: 284.9	
4	C9	Min: -373.1	Min: -316.4	±15%
		Max: 366.5	Max: 311.7	
3	C9	Min: -373.6	Min: -342	±8.3%
		Max: 359.4	Max: 329.6	
2	C9	Min: -382.9	Min: -367.9	±2%
		Max: 350.6	Max: 351.3	
1	C9	Min: -394	Min: -303.7	±19.6%
		Max: 346.2	Max: 289.7	
Base	C9	Min: -346.2	Min: -226.2	±32.35%
		Max: 309.1	Max: 215.9	
Average				**±27.24%**

5.5 DISCUSSING FINDINGS

The present study was design to determine the effect of friction damper in tall buildings structure. It is necessary to be mentioned that, all of the obtained results are based on applied arrangements of friction dampers as well as their types (single diagonal tension – compression brace). And also the results are based on this type of building and seismic region. Generally, this finding supports previous research into the brain area which, response of structure can be dramatically reduced by using friction damper.

As mentioned in the literature review, the percentage of reductions of displacement, axial load of columns and base shear for low to medium high – rise buildings is more than 60% for base shear, 50% for displacement, and 30% for axial loads. Although, this study produced results which corroborate the findings of a great deal of the previous work in this field, but the results of this study did not show such a significantly decrease. A possible explanation for this might be that, the high performance of friction damper devices is when they use in low high-rise buildings. Another possible explanation for this is that, friction dampers can be used economically for the high seismic activity areas with severe earthquake loads.

CHAPTER 6

RECOMMENDATION AND CONCLUSION

6.1 OVERVIEW

The present study was designed to determine the seismic behavior of high-rise building structures by friction dampers. The main objectives of the study are stated in chapter one. The purpose of the study was to investigate friction damper devices that not only provides adequate energy dissipation by different models, but also are easy to install and inspect. In addition, there are not like cross – bracings which may be undesirable in the field of aesthetically and architecturally.

6.2 CONCLUSION

The following conclusions can be drawn from the present study:

1. The results of this investigation show that generally, the response of the structure can be dramatically reduced by using friction damper.

2. According to free vibration analysis the percentage of reduction is 37.75% for period.

3. According to time history analysis the percentage of reduction for the story displacement is 15.9%; for the axial load of columns is 50.75%; for shear load is 30.75% and for the bending moment is 27.24%.

6.3 LIMITATION OF THE CURRENT STUDY

The current study was not specifically designed to evaluate the seismic vulnerability of the specific high-rise buildings in Albania. In other words, the current study has only seismically analyzed the tall building to determine generally the seismic behavior of tall building structures by friction damper.

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APPENDIX A

a) Main mode shapes of the undamped and damped structure

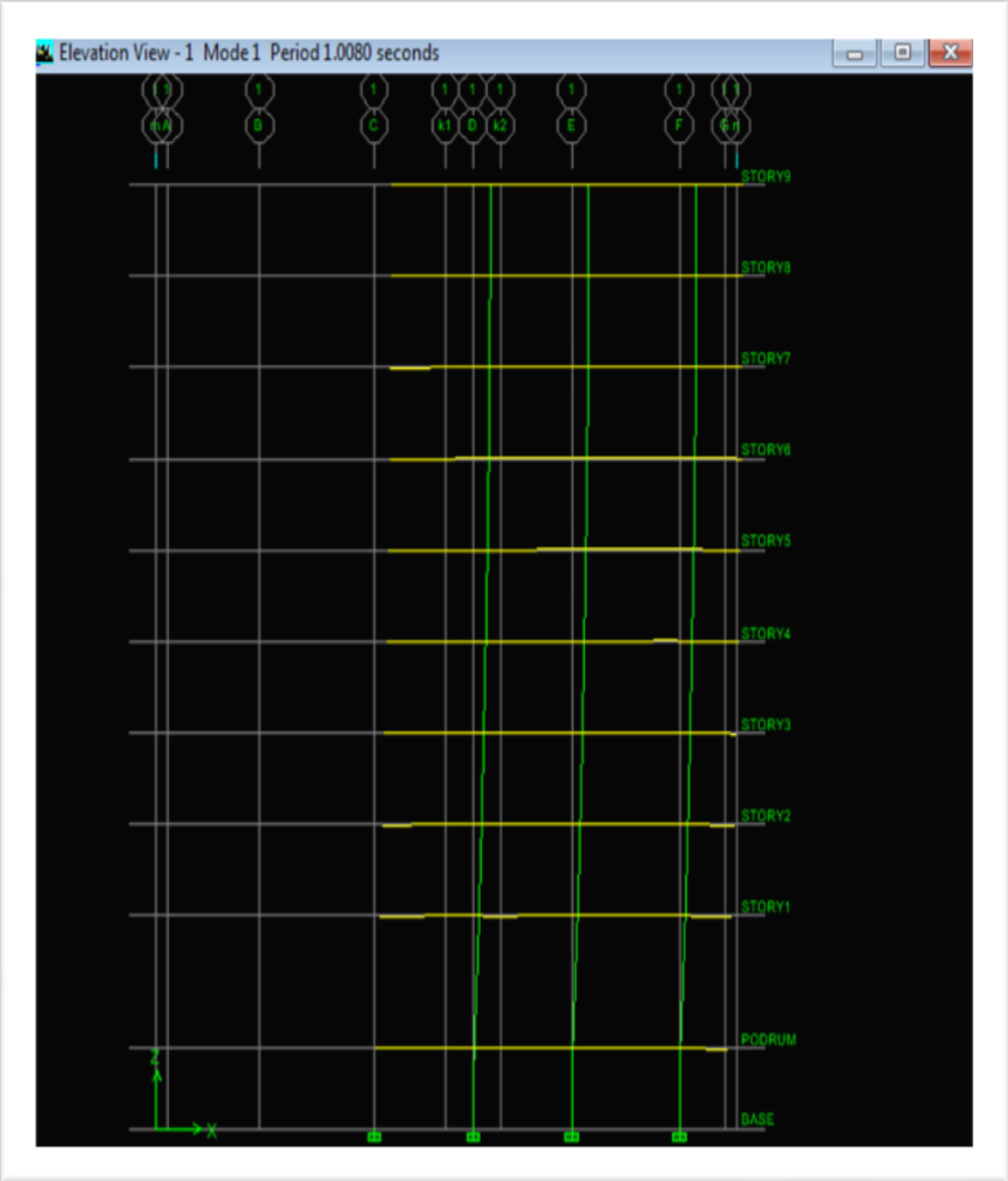

Figure 35 Un Damped structure (Mode 1, T=1.008 sec)

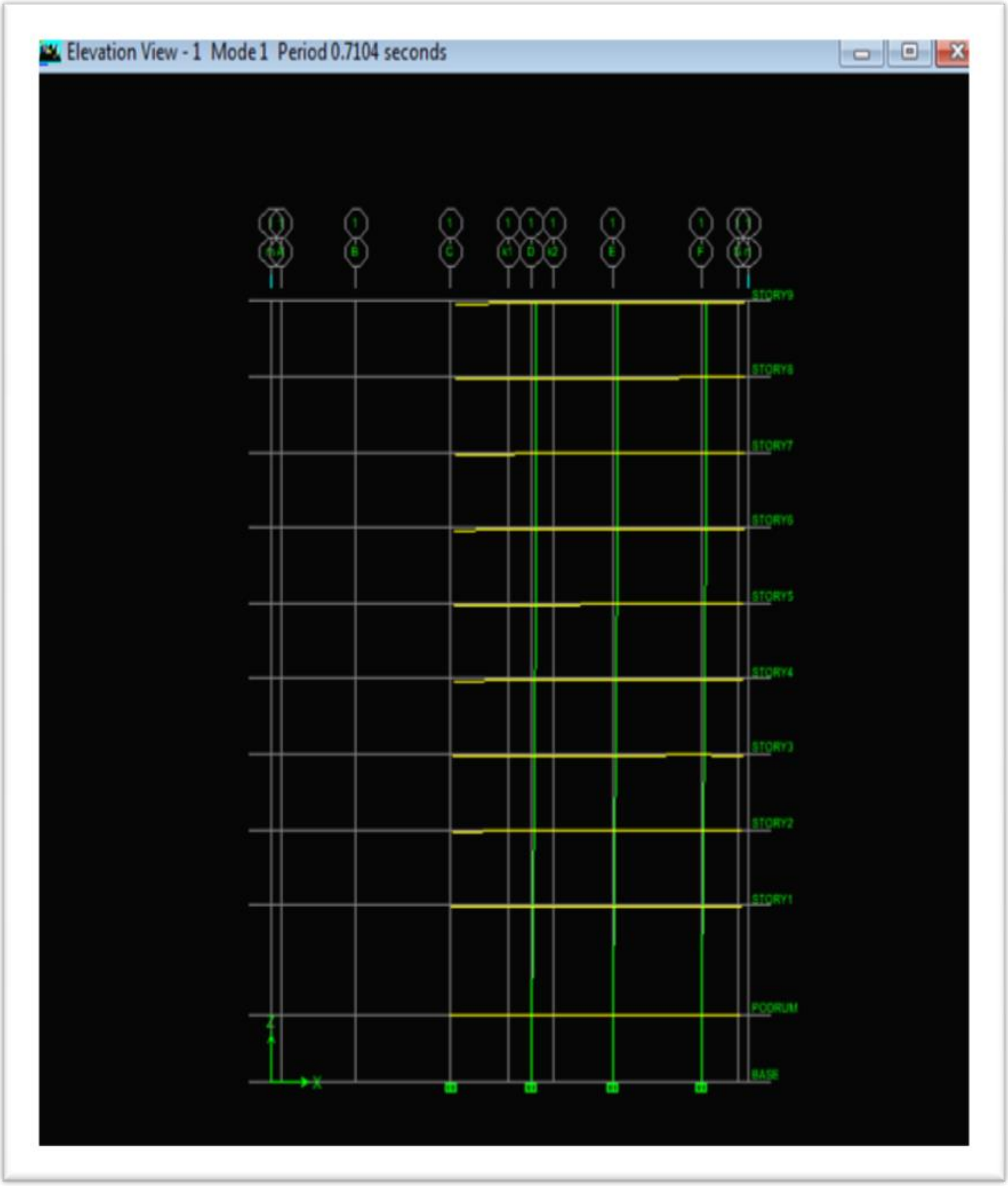

Figure 36 Damped structure (Mode 1, T=0.7104 sec)

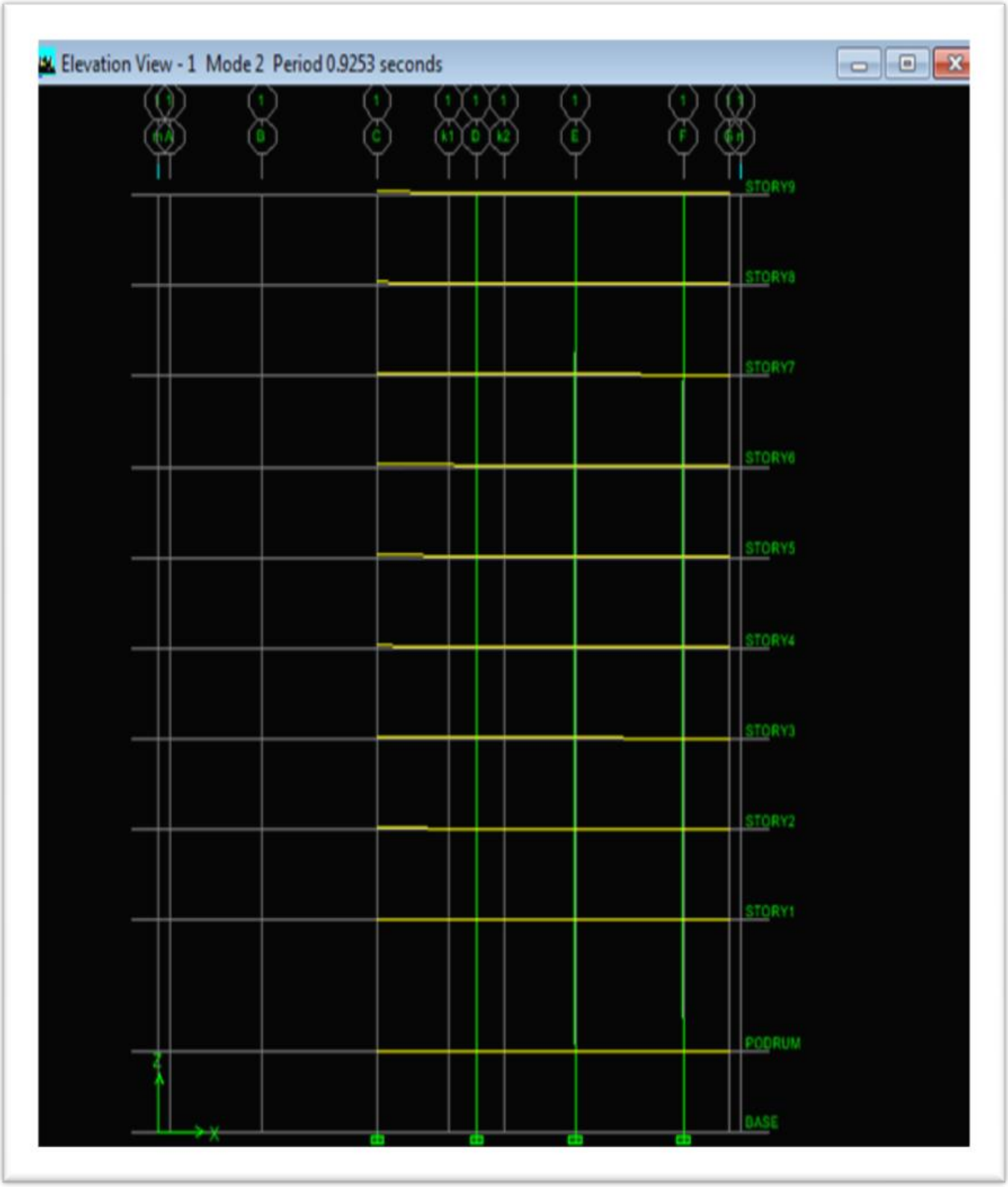

Figure 37 Un Damped structure (Mode 2, T=0.9253 sec)

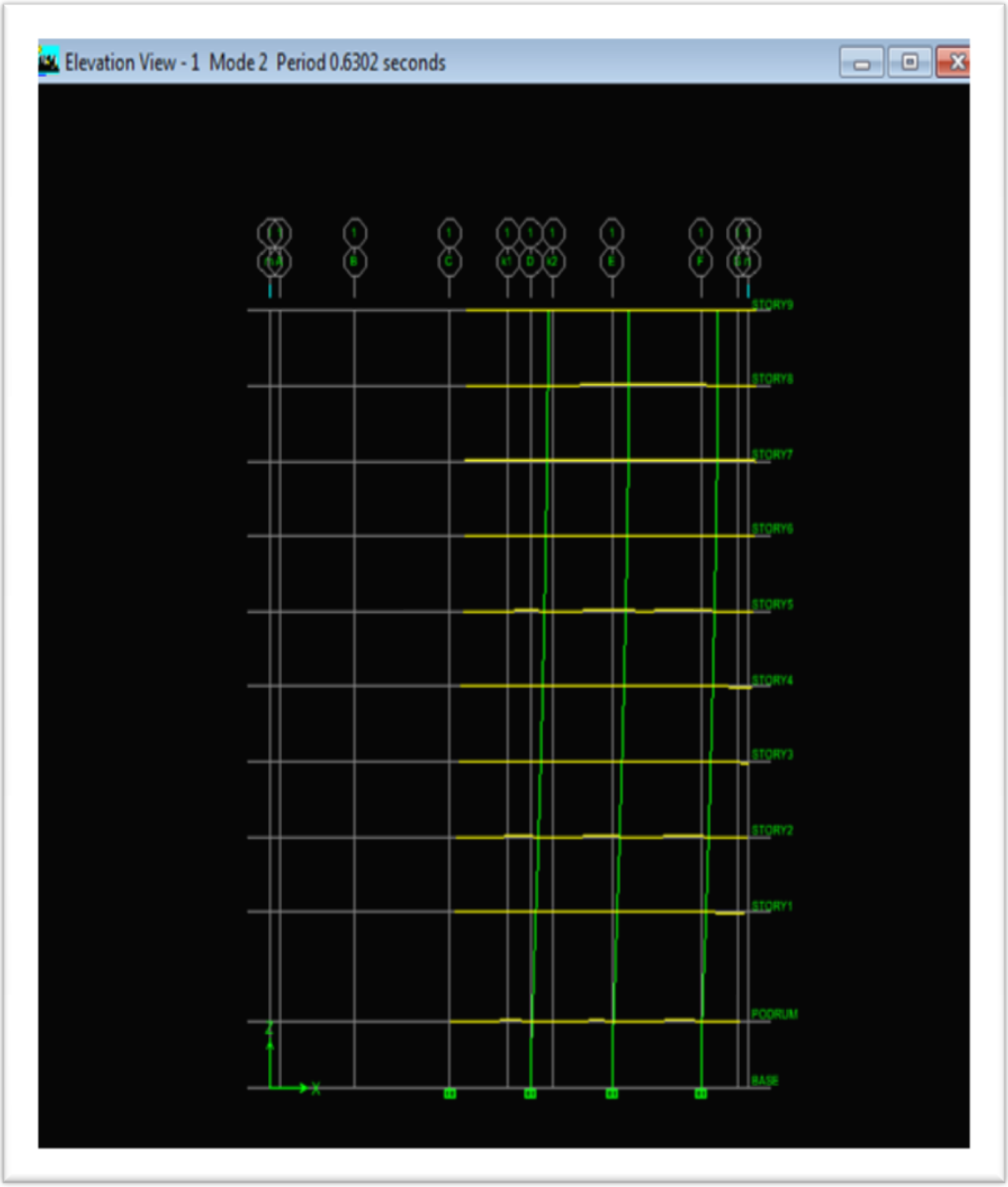

Figure 38 Damped structure (Mode 0.6302)

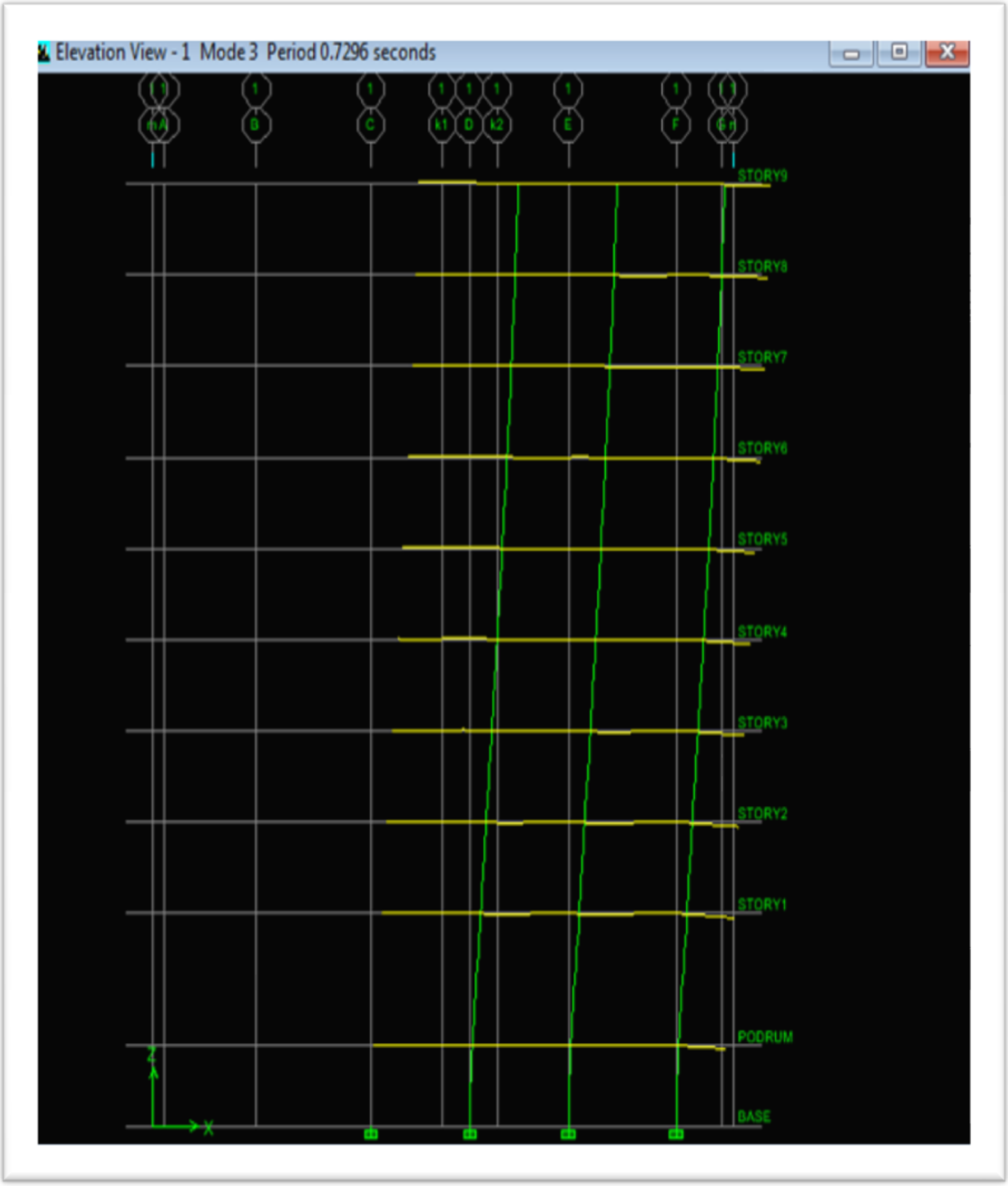

Figure 39 Un Damped structure (Mode 3, T=0.7296)

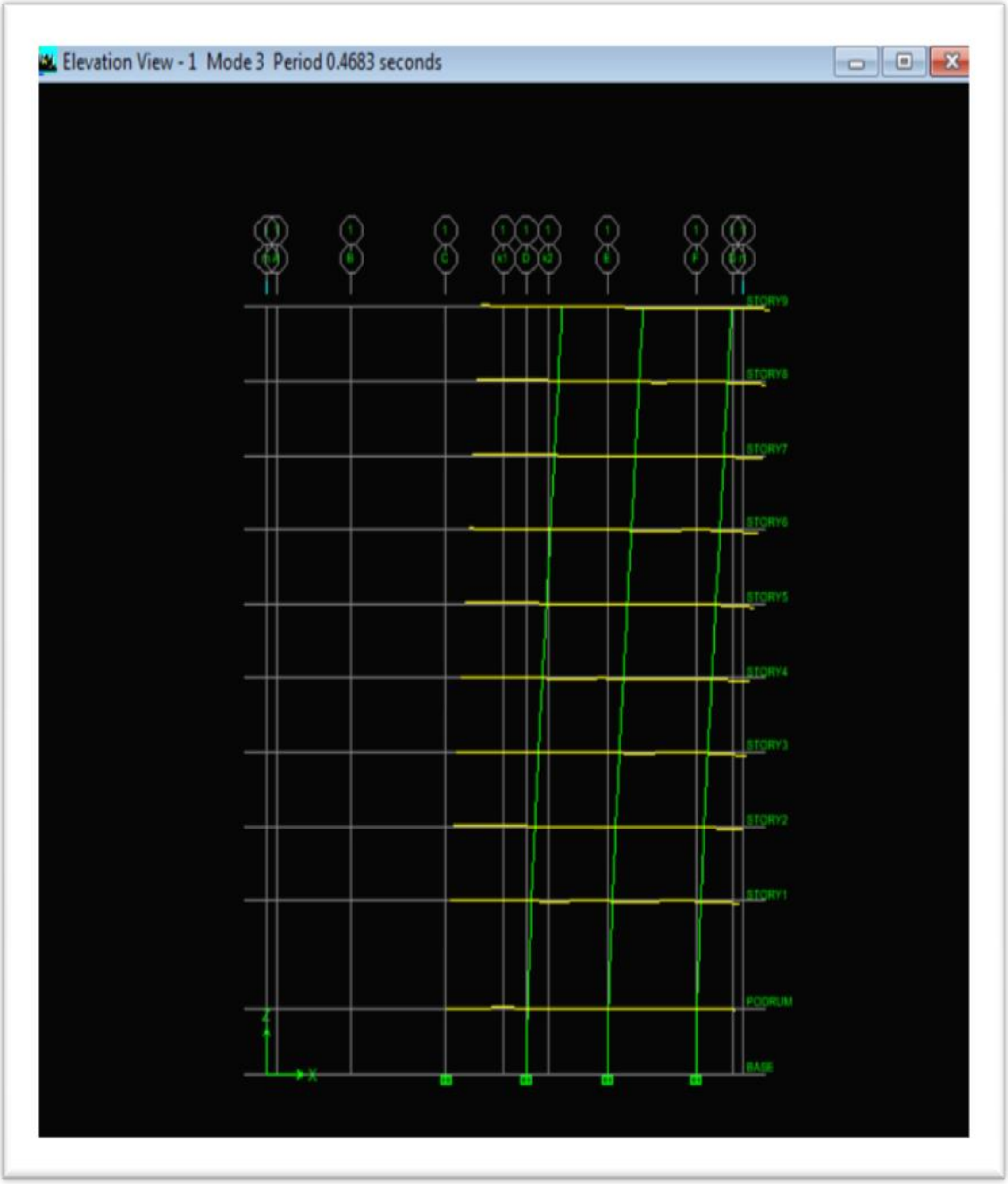

Figure 40 Damped structure (Mode 3, T=0.4683)

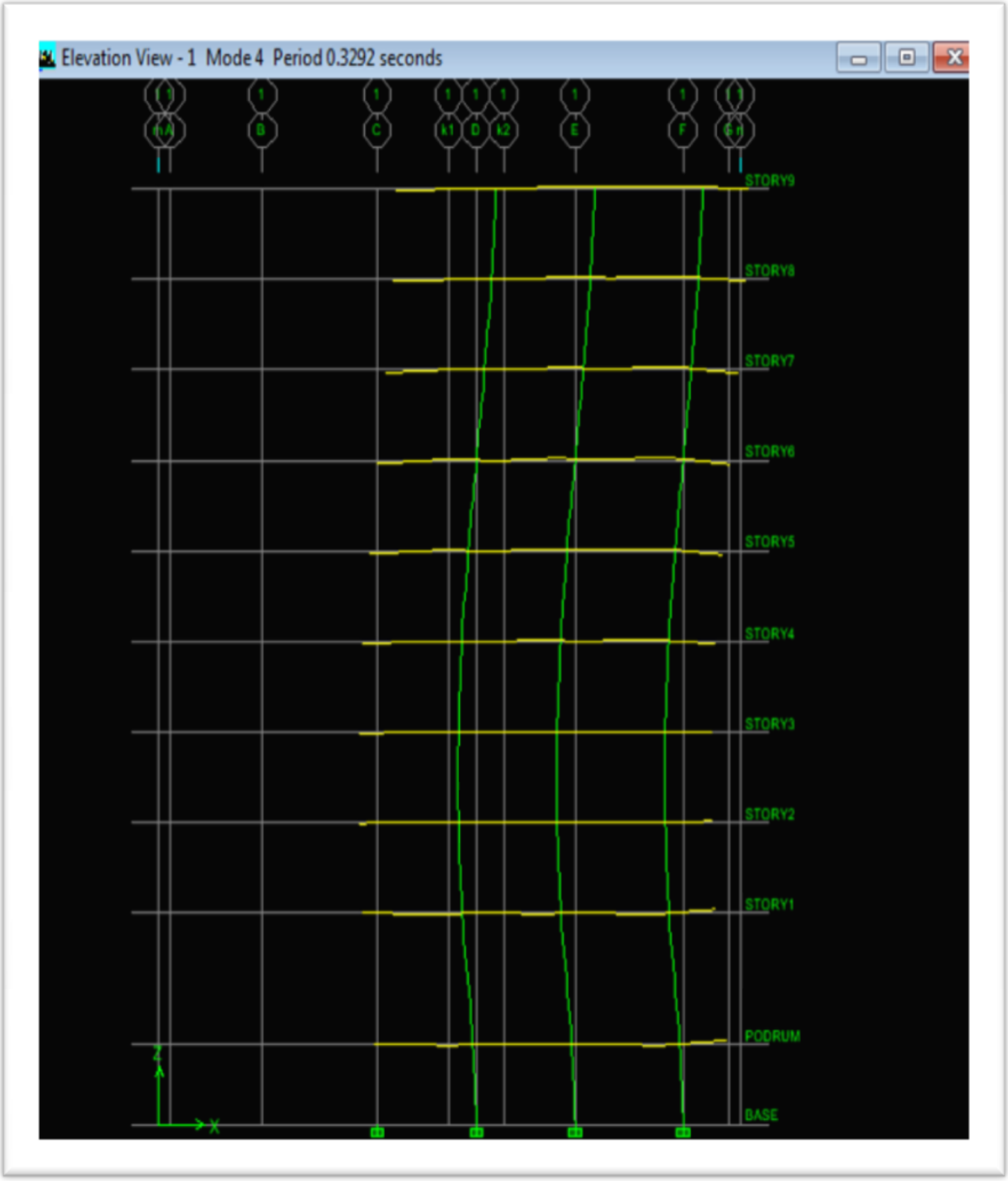

Figure 41 Un Damped structure (Mode 4, T=0.3292)

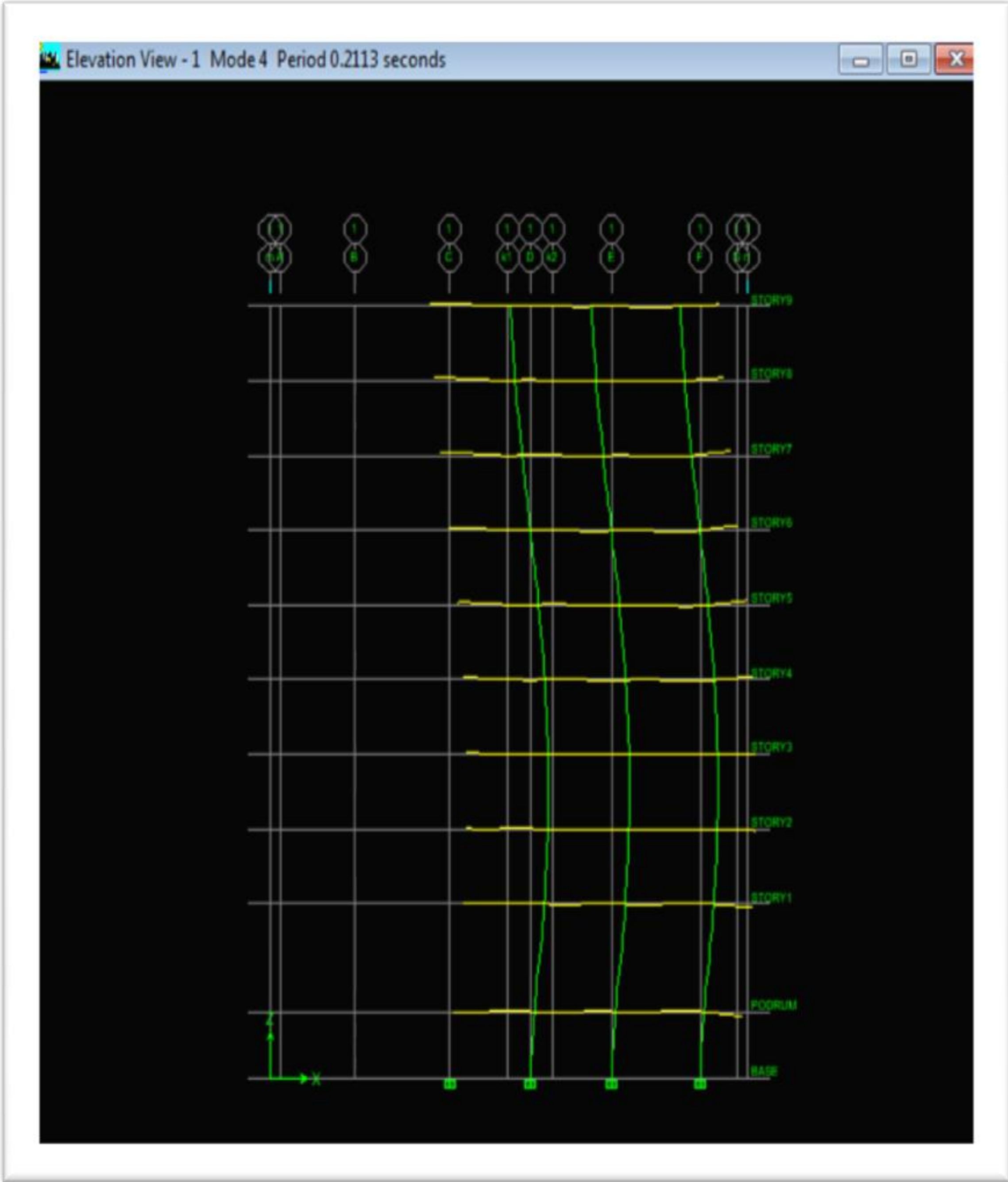

Figure 42 Damped structure (Mode 4, T=0.2113)

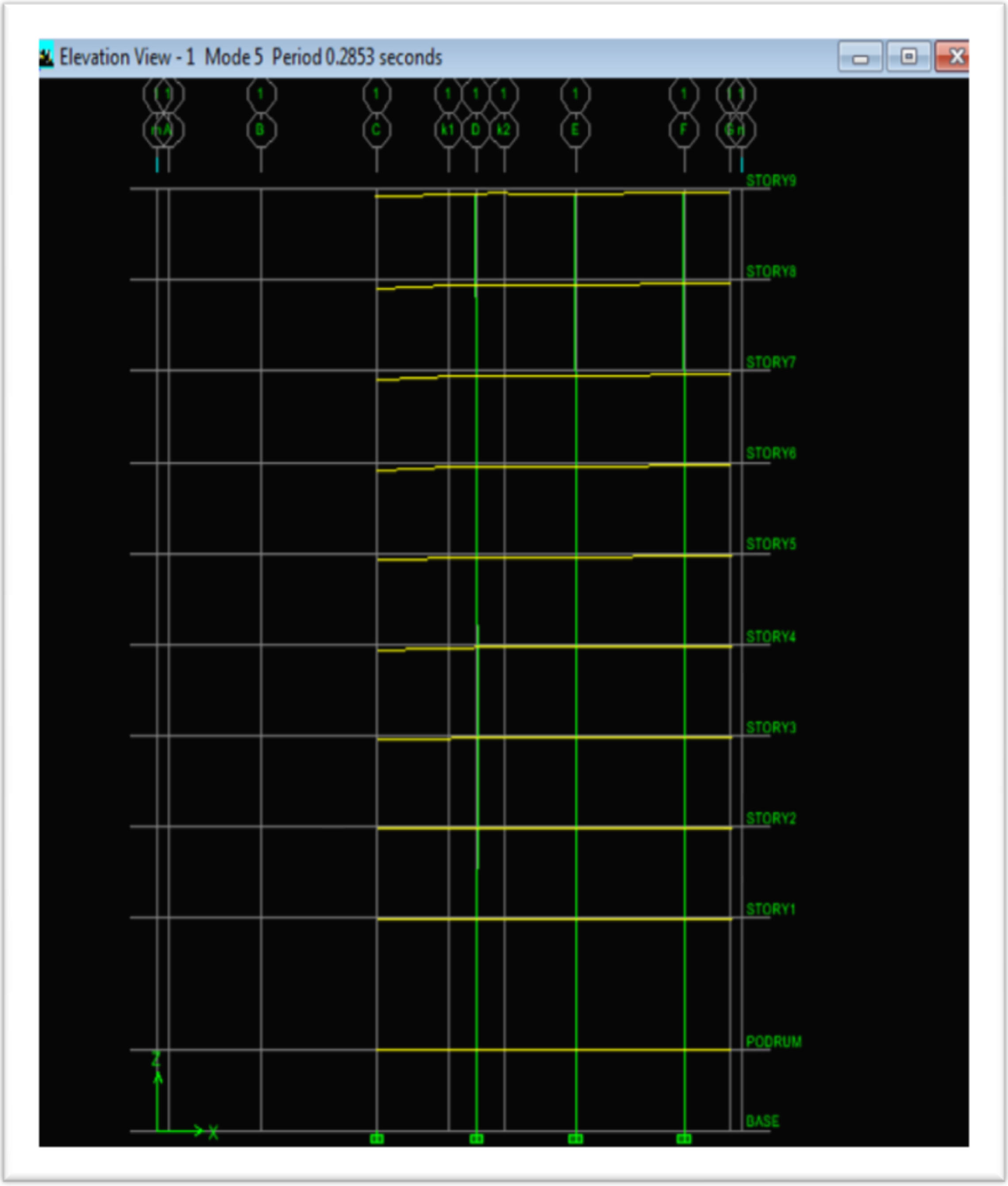

Figure 43 Un Damped structure (Mode 5, T=0.2853)

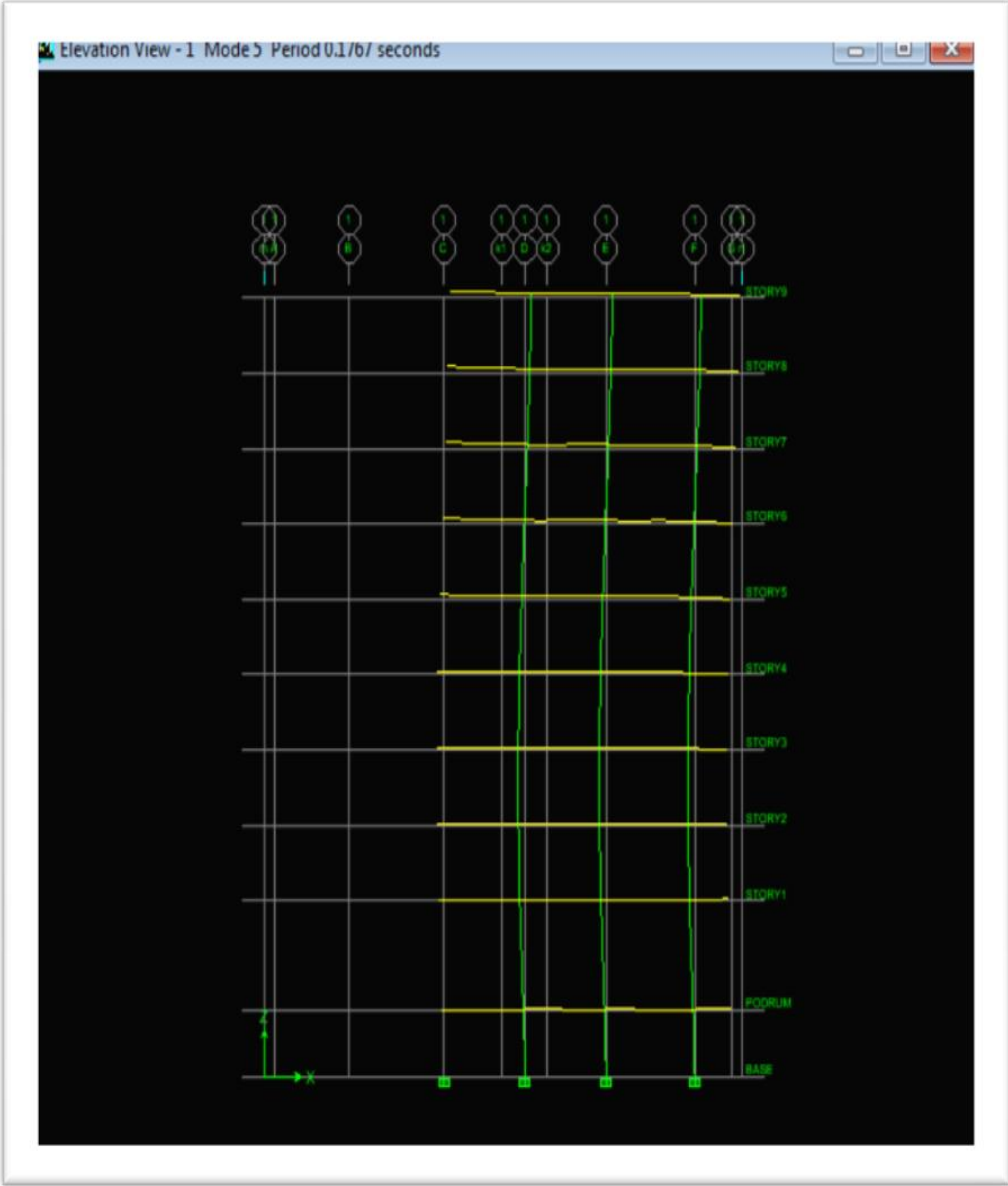

Figure 44 Damped structure (Mode 5, T=0.1767 sec)

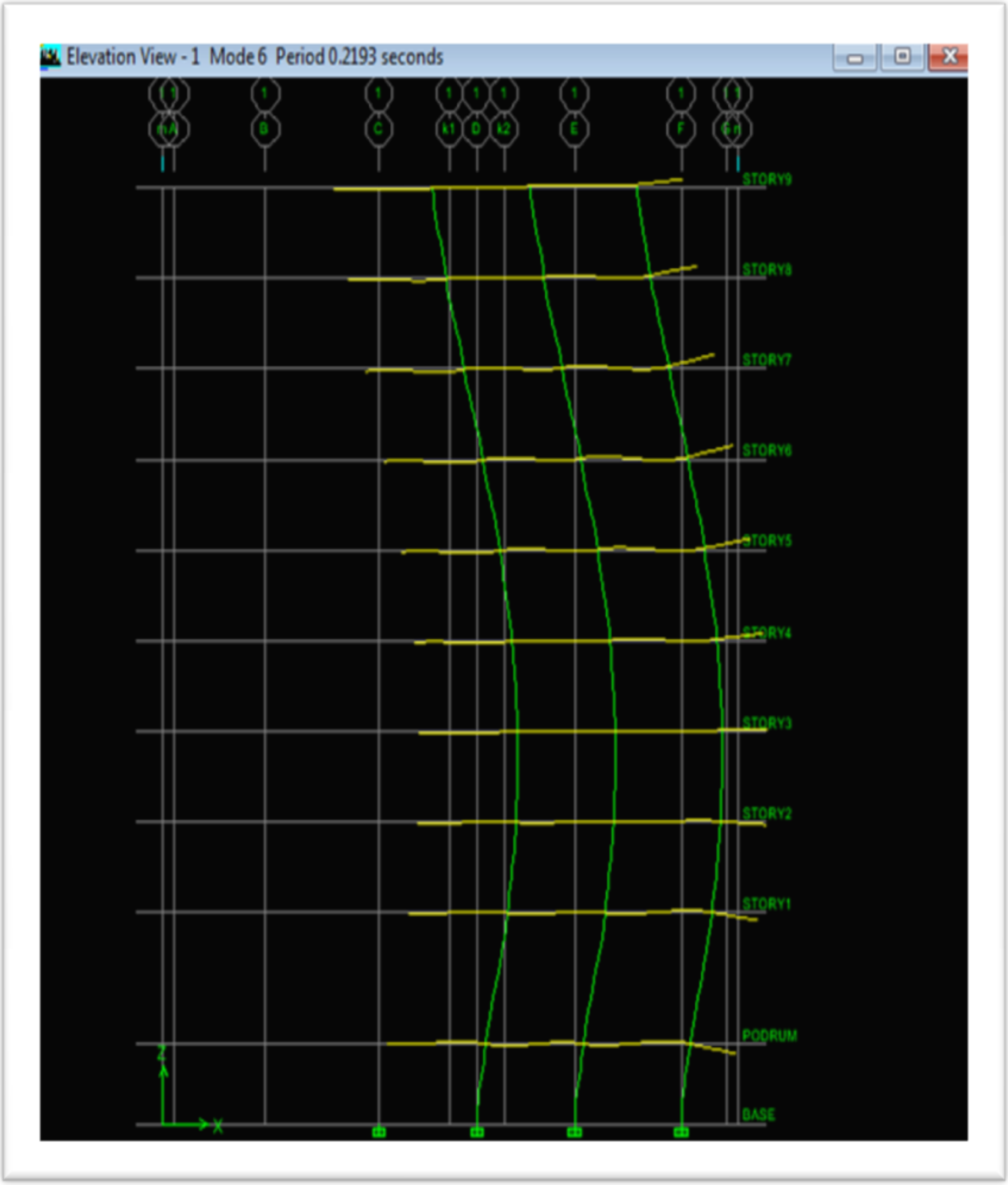

Figure 45 Un Damped structure (Mode 6, T=0.2193 sec)

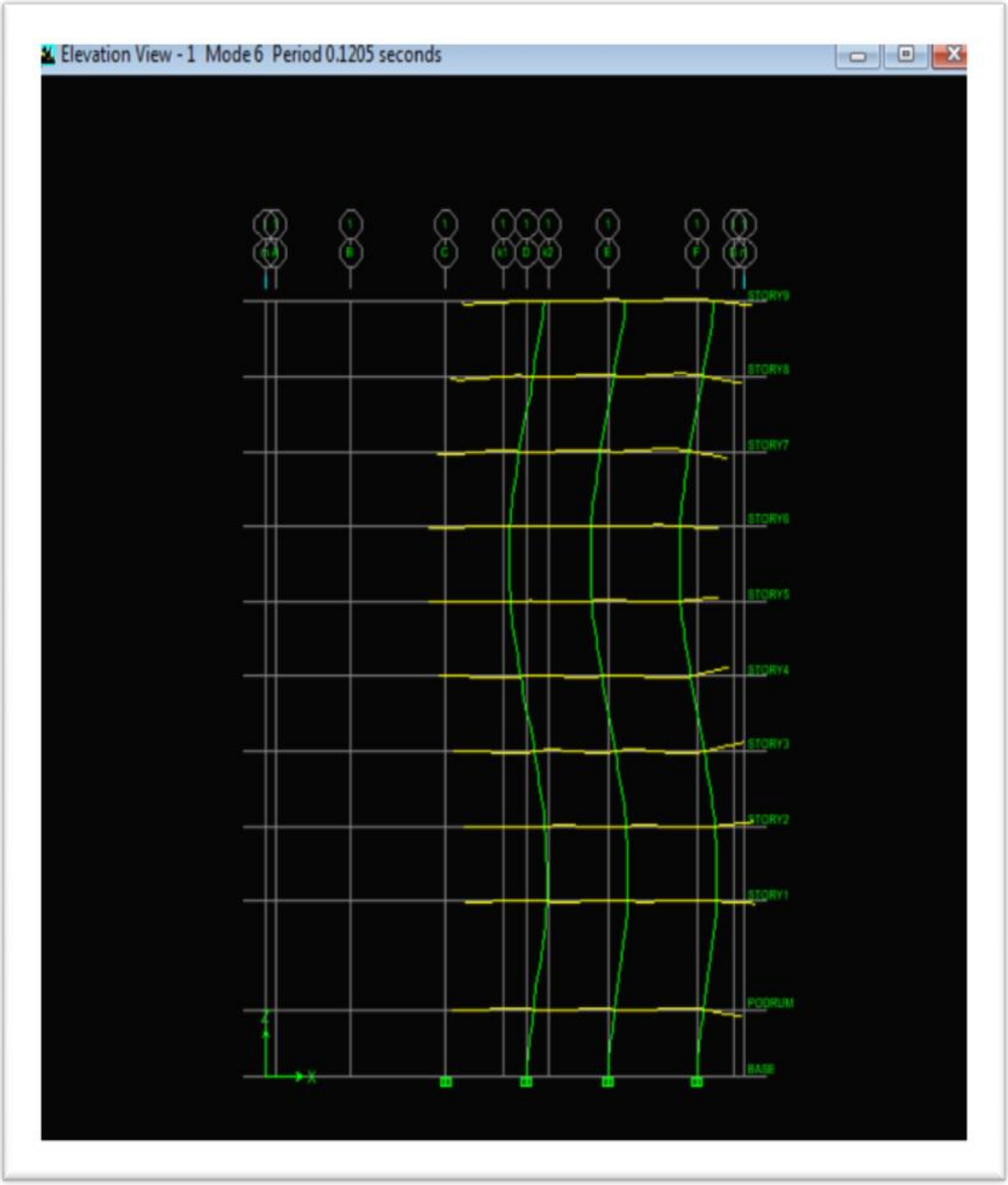

Figure 46 Damped structure (Mode 6, T=0.1205 sec)

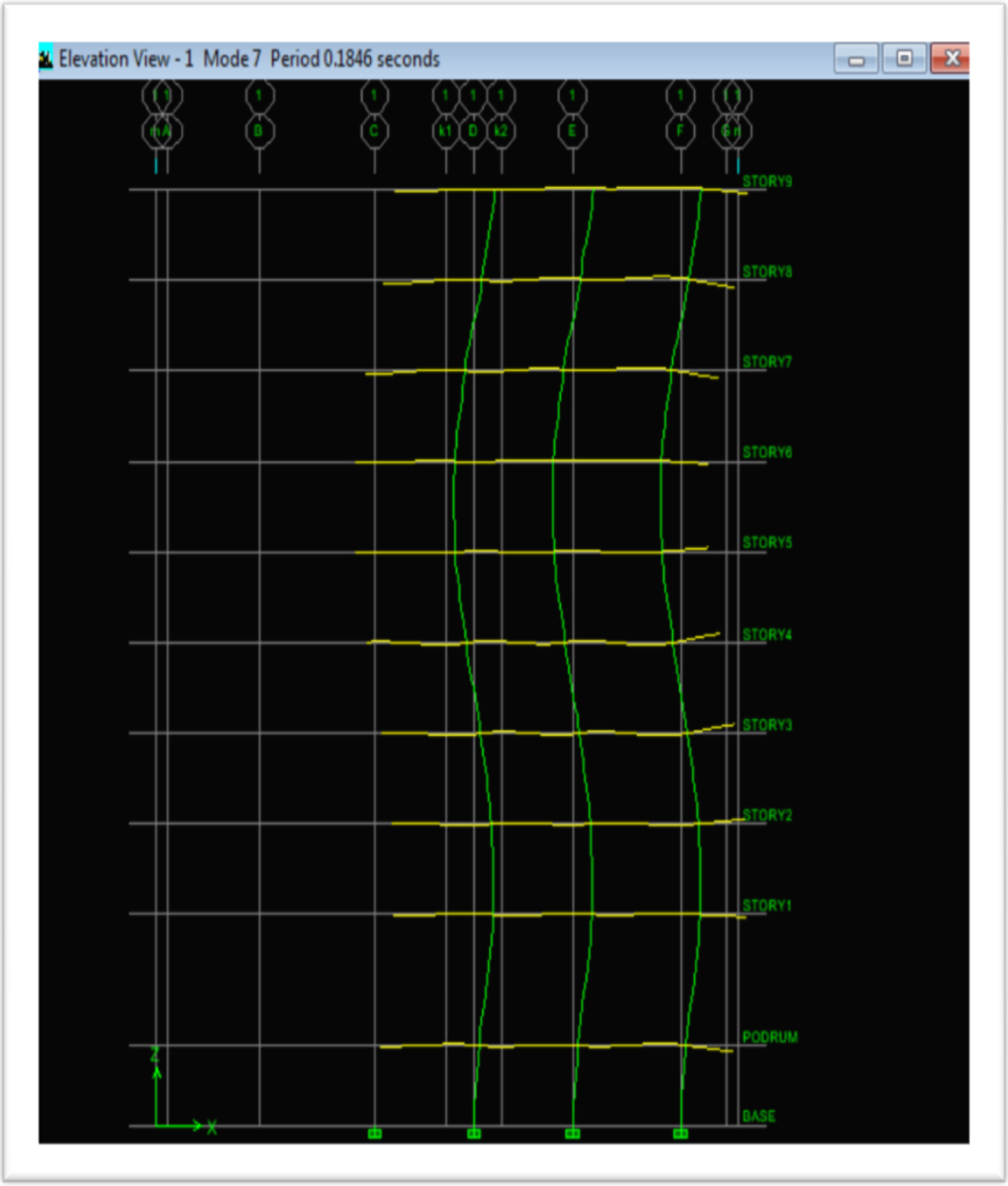

Figure 47 Un Damped structure (Mode 7, T=0.1846 sec)

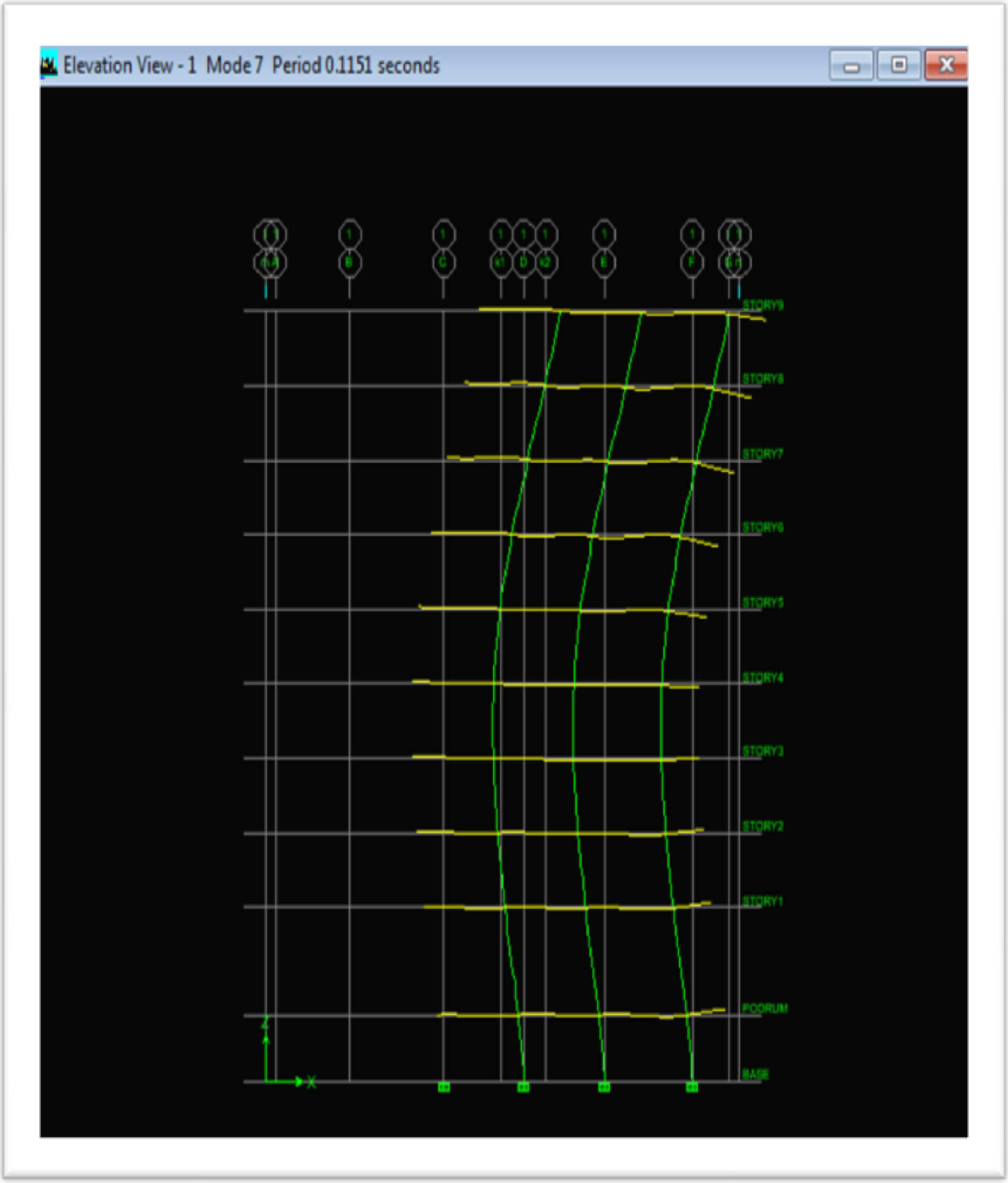

Figure 48 Damped structure (Mode 7, T=0.1151 sec)

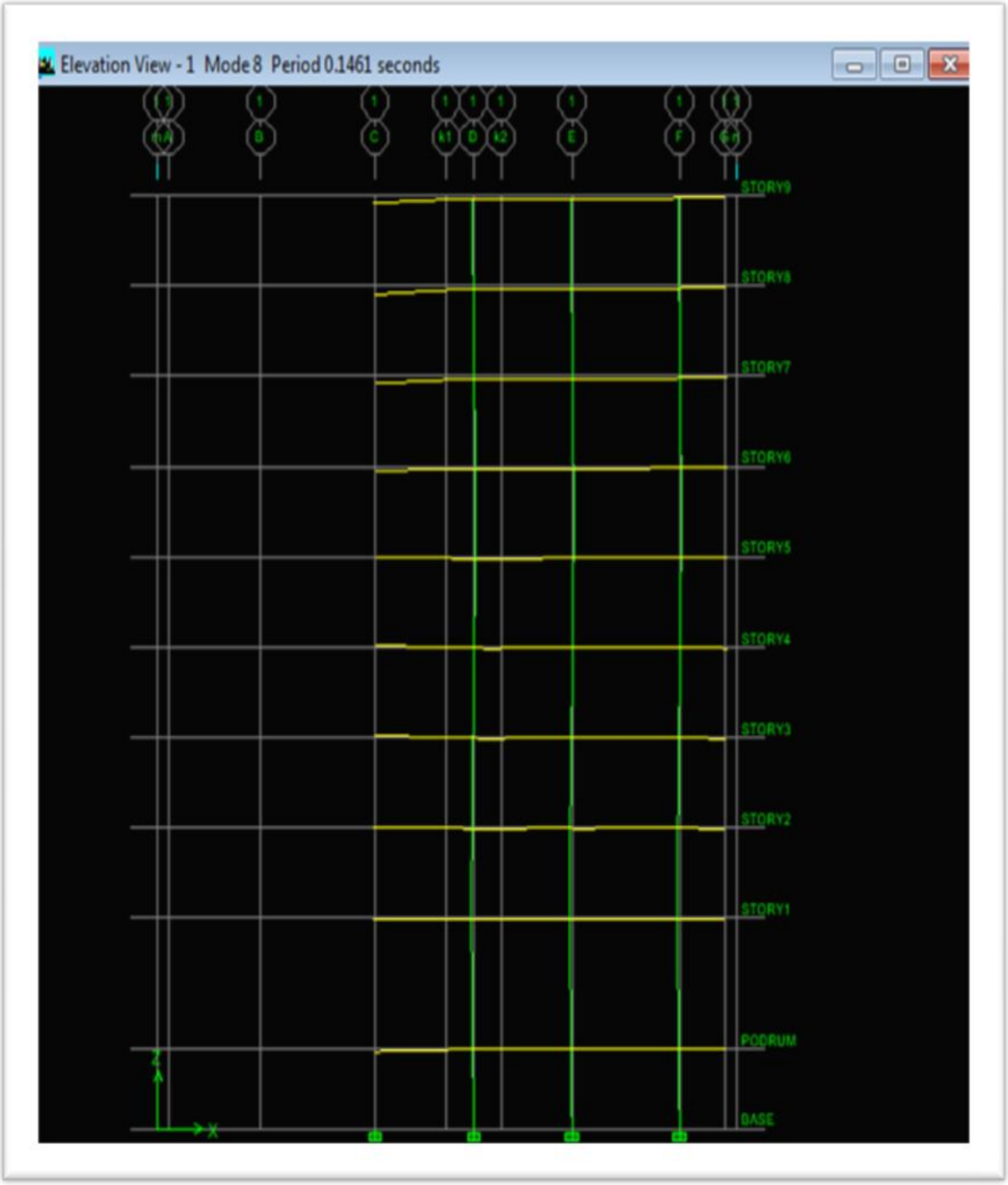

Figure 49 Un Damped structure (Mode 8, T=0.1461)

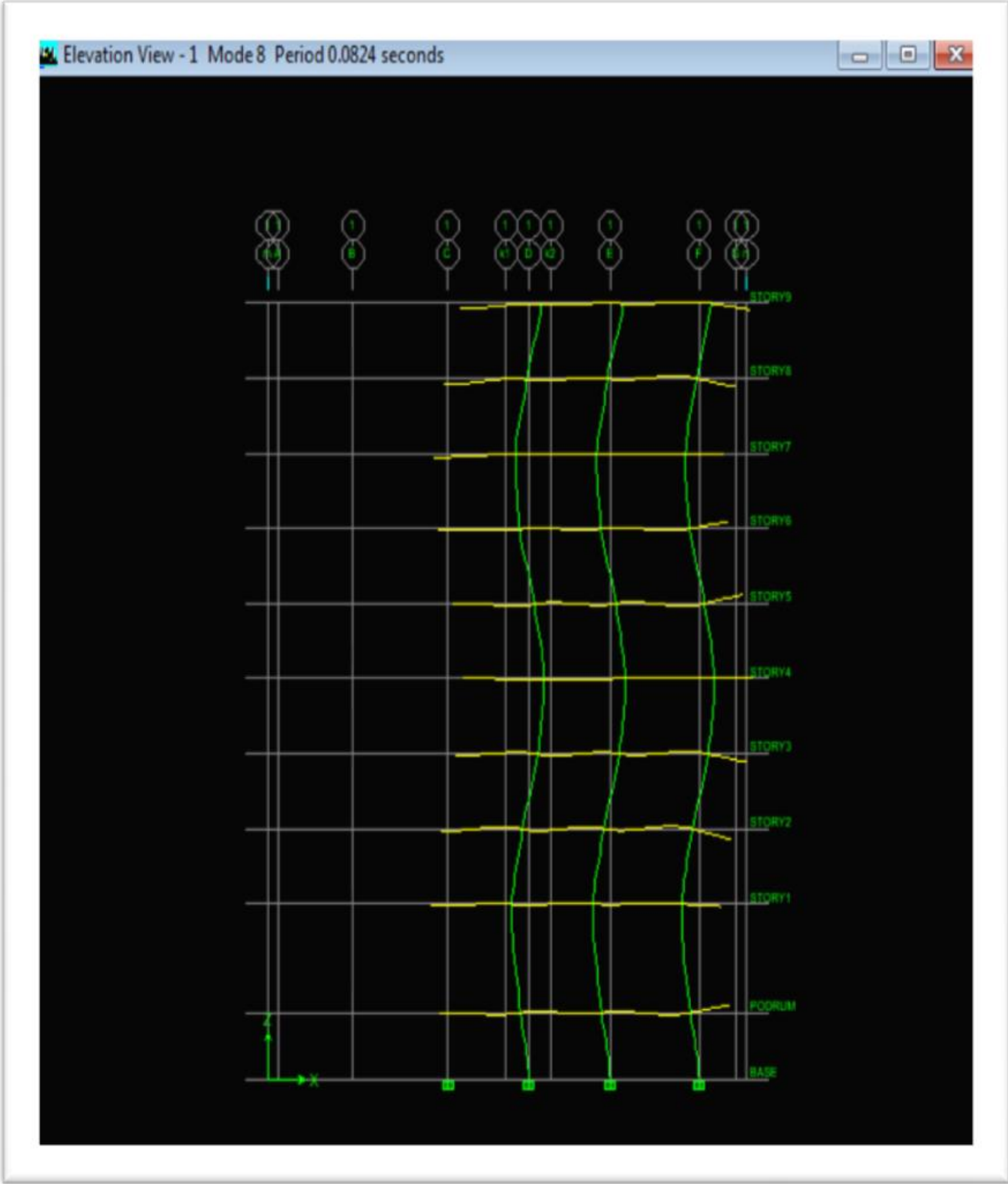

Figure 50 Damped structure (Mode 8, T=0.0824 sec)

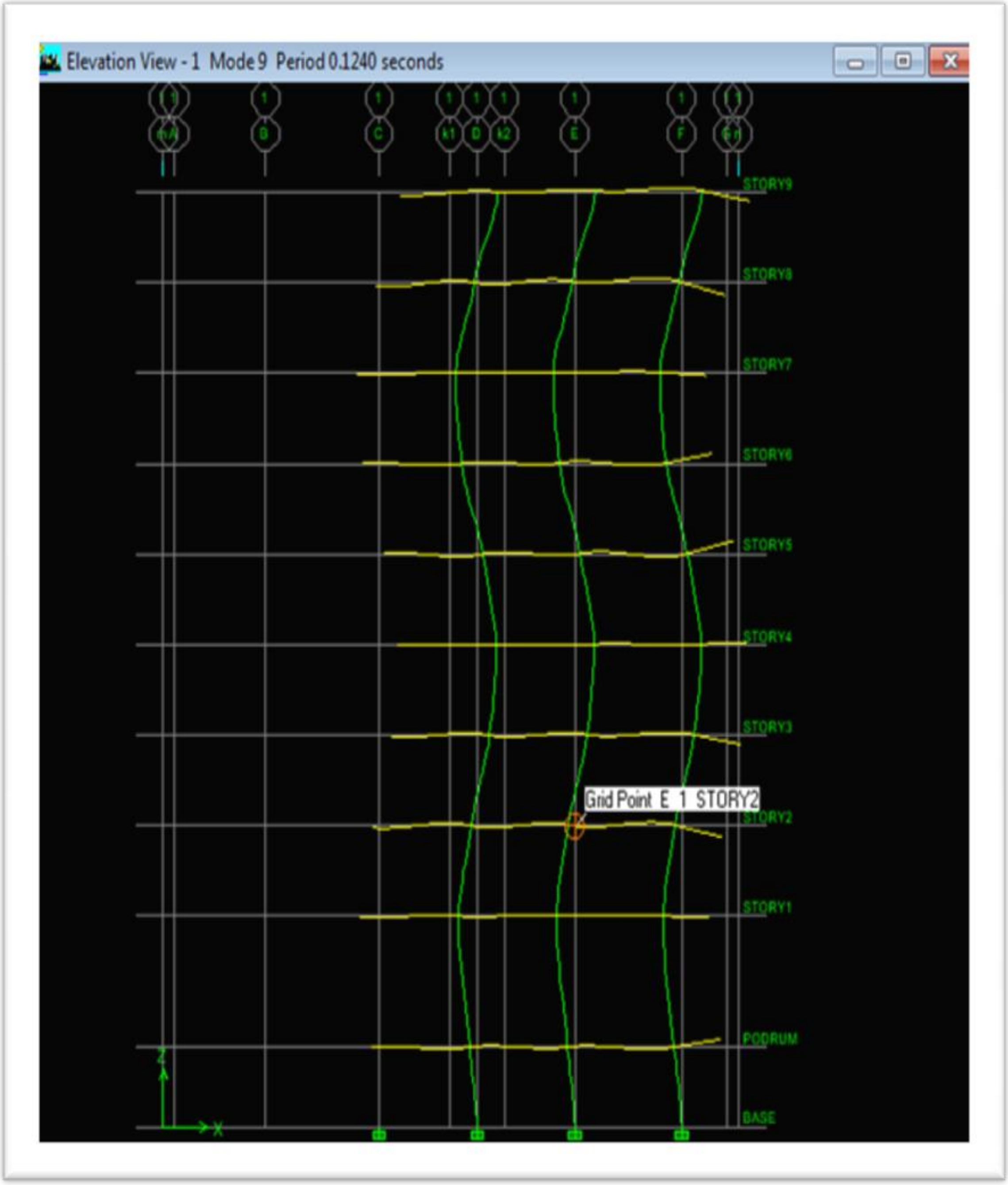

Figure 51 Un Damped structure (Mode 9, T=0.124 sec)

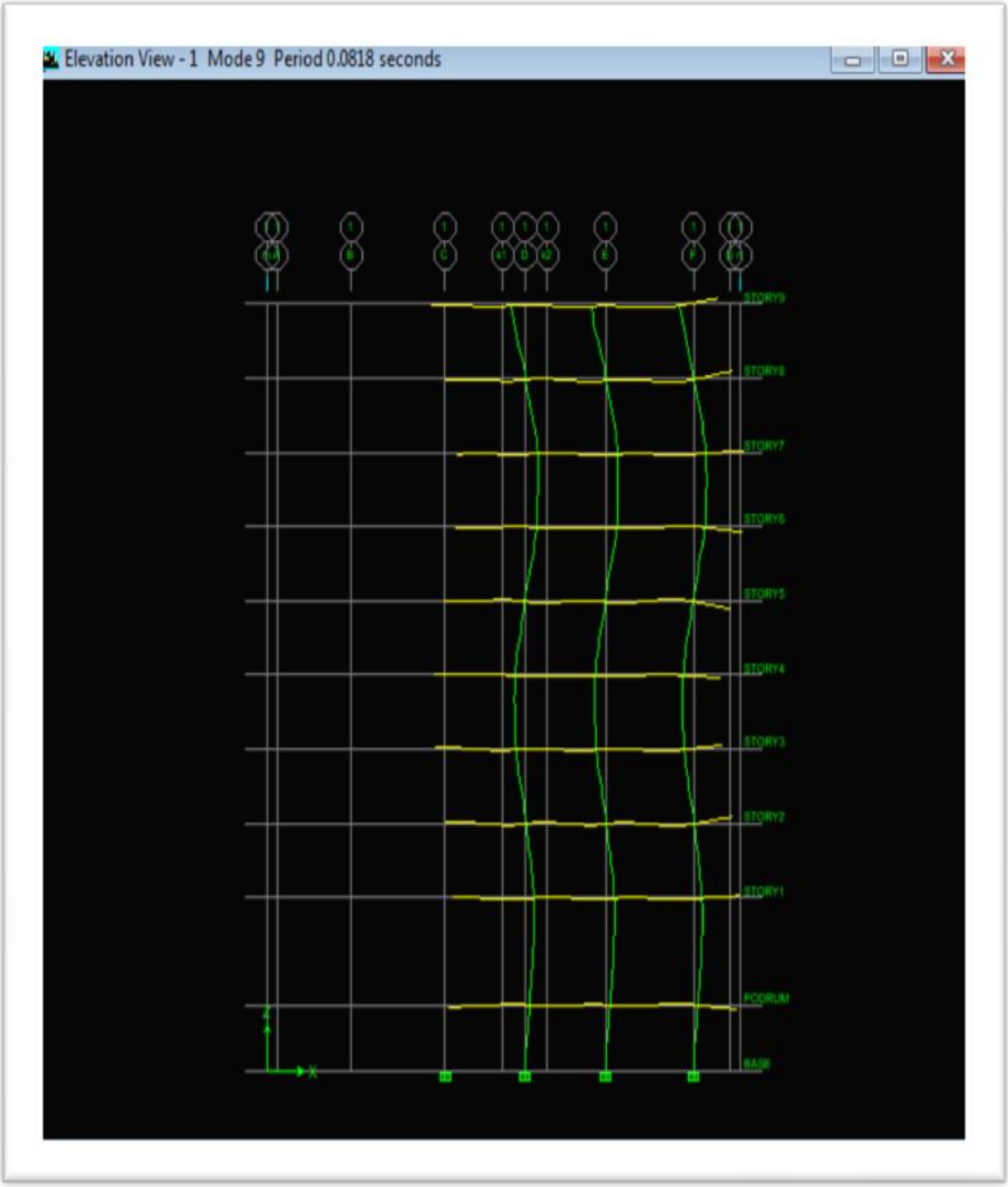

Figure 52 Damped structure (Mode 9, T=0.0818 sec)

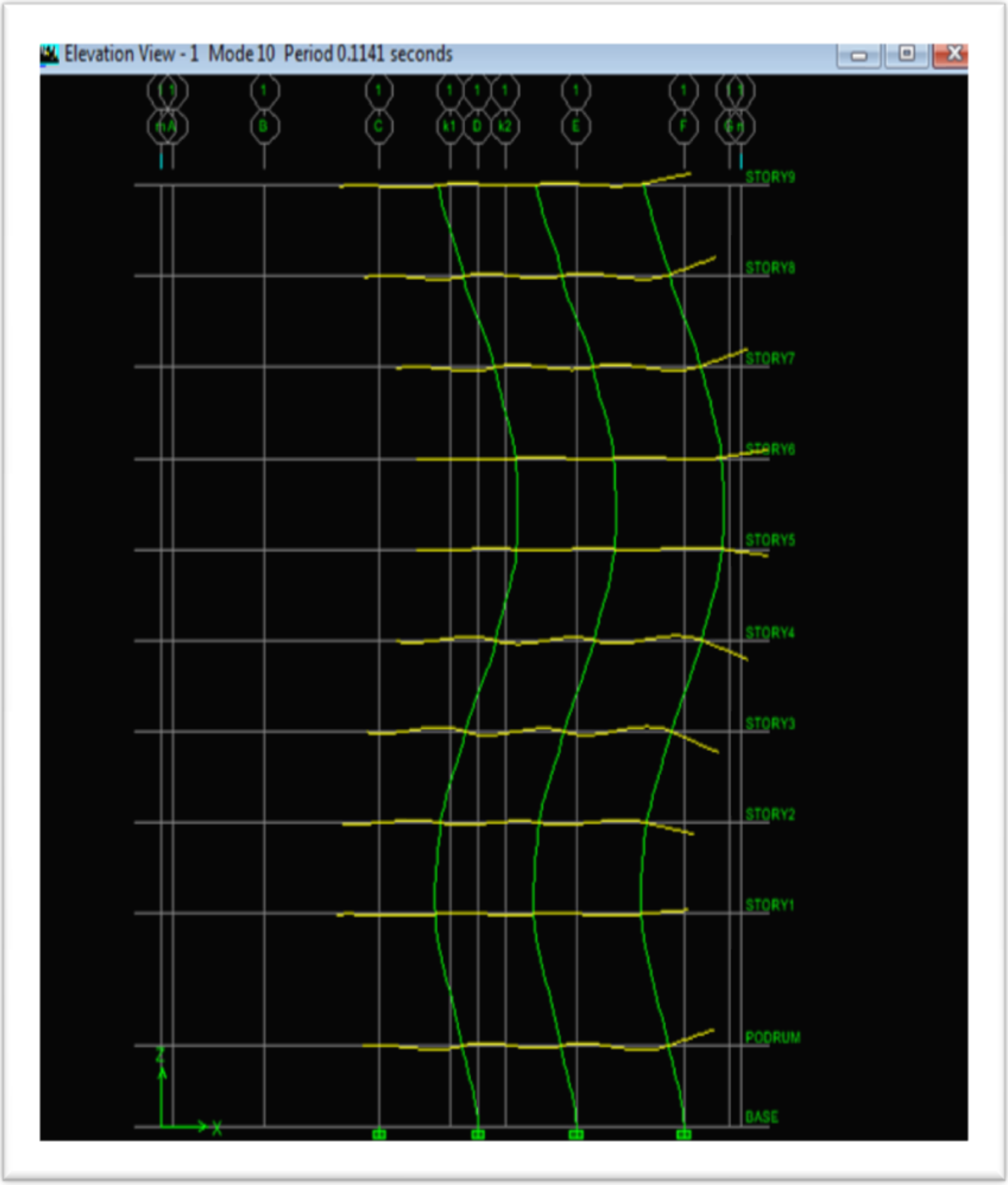

Figure 53 Un Damped structure (Mode 10, T=0.1141)

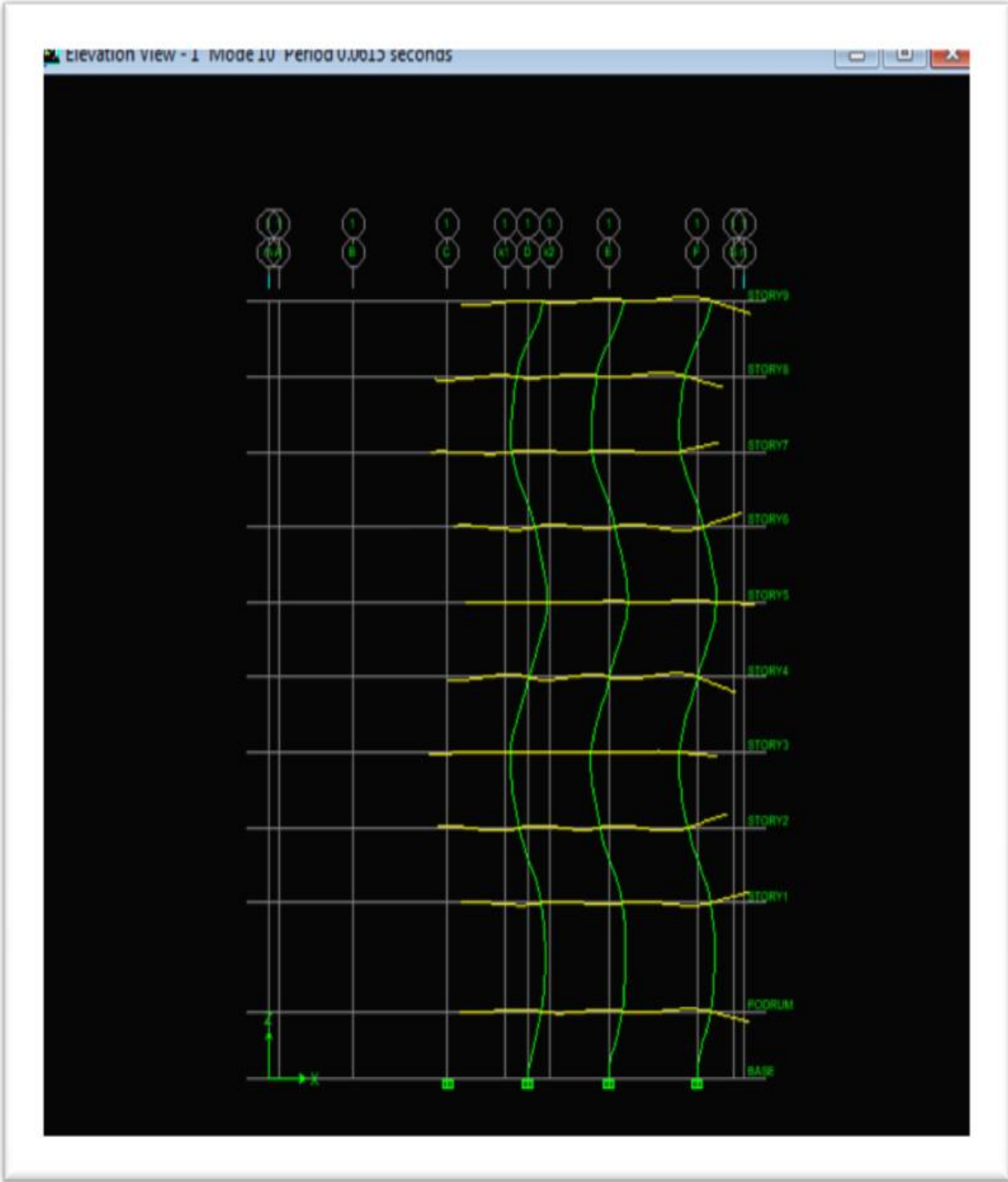

Figure 54 Damped structure (Mode 10, T=0.0615)