UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES

MODELLING AND SEISMIC RESPONSE OF STRUCTURES EQUIPPED WITH VISCOELASTIC DAMPERS

M. Sc. THESIS
IN
CIVIL ENGINEERING

BY
TUĞBA GÜREL
APRİL 2011

Modelling and Seismic Response of Structures Equipped with Viscoelastic Dampers

M.Sc. Thesis
in
Civil Engineering
University of Gaziantep

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April 2011

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UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES
CIVIL ENGINEERING DEPARTMENT

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Exam date:	April 07, 2011

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ABSTRACT

MODELLING AND SEISMIC RESPONSE OF STRUCTURES EQUIPPED WITH VISCOELASTIC DAMPERS

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April 2011, 61 pages

In recent years, there have been studies on innovative approaches additional to the conventional design approaches in order to better protect the structures from the external dynamic forces. These innovative approaches focus on the materials and systems that react to the dynamic forces and dissipate the energy in itself. These innovative systems that generally dissipate the energy through friction or plastic deformation are called as passive energy dissipation systems. This study is concerned with the analytical modelling and seismic response analysis of structures with passive energy dissipation devices, such as viscoelastic dampers. Viscoelastic dampers were used in structures in order to mitigate dynamic effects. For this purpose, five and twelve storey four bay steel moment resisting frames with and without viscoelastic dampers were designed. The reference frames (without damper) and the frames with viscoelastic dampers were analyzed through nonlinear time history analysis by using natural ground motions. In the dynamic analyses, 1991 Alkion, 1992 Erzincan, and 1999 İzmit earthquakes were utilized as a ground motion. As a result of these analyses, roof displacement time history, inter-storey drift ratios, maximum plastic rotation of beams and columns, and energy time history plots were evaluated. From the results of this study, it was pointed out that the viscoelastic passive energy dissipation systems had substantial positive effects on the seismic behavior of steel framed structures.

Keywords: Dynamic analysis, Framed structures, Passive energy dissipation systems, Seismic behavior, Viscoelastic damper

ÖZET

VİSKOELASTİK SÖNÜMLEYİCİLİ YAPILARIN MODELLENMESİ VE DEPREM ETKİSİ ALTINDAKİ DAVRANIŞLARI

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April 2011, 61 sayfa

Son yıllarda yapıları dinamik dış etkilere karşı daha iyi korumak için mevcut klasik tasarıma ek olarak onu tamamlayıcı yeni yaklaşımlar üzerinde çalışılmaktadır. Bu yaklaşımlar, yapıya gelen etkilere karşı kuvvetler uygulayacak veya etkiyi kendi içinde sönümleyecek malzemeler ve sistemler üzerine yoğunlaşmaktadır. Pasif enerji sönümleme sistemleri dinamik etkilere maruz yapılardaki titreşim enerjisini iç sürtünme ve plastik deformasyonlar ile gidermektedir. Bu çalışmada, pasif kontrol sistemlerinden viskoelastik sönümleyicili yapıların analitik modeli ve deprem etkisini altındaki davranışları araştırılmıştır. Bu amaçla, referans çerçeve olarak beş katlı ve oniki katlı dört açıklıklı iki çelik çerçeve sistemi seçilmiş, moment taşıyan çerçeveler sönümleyicisiz ve viskoelastik sönümleyicili olarak tasarlanmıştır. Referans çerçevelerin ve viskoelastik sönümleyicili çelik çerçevelerin doğal deprem ivmeleri altındaki davranışları zaman tanım alanında dinamik analizleri yapılarak incelenmiştir. Dinamik analizlerde yer hareketi olarak 1991 Alkion, 1992 Erzincan ve 1999 İzmit depremleri kullanılmıştır. Böylece, elde edilen tepe yerdeğiştirmeleri, göreli kat öteleme oranları, kolon ve kirişlerdeki maksimum plastik dönme değerleri ve yapılardaki enerji dağılımları karşılaştırılmalı olarak değerlendirilmiştir. Bu analitik çalışma sonucunda elde edilen verilere göre, viskoelastik sönümleyicili sistemlerin çelik çerçeveli yapıların deprem davranışları üzerinde önemli etkilerinin olduğunu göstermiştir.

Anahtar kelimeler: Çerçeve yapı, Deprem performansı, Dinamik analiz, Pasif enerji sönümleme sistemleri, Viskoelastik sönümleyici

ACKNOWLEDGEMENT

I would like to express my sincere gratitude to my supervisor, Assist. Prof. Dr. Esra METE GÜNEYİSİ for her helpful suggestions, enormous guidance, advices and technical support that she has provided during the preparation of this thesis.

My deep appreciations and thanks to Assoc. Prof. Dr. Erhan GÜNEYİSİ for his helps and valuable suggestions.

I would like to special thanks to Assoc. Prof. Dr. Mehmet GESOĞLU and Assist. Prof. Dr. M. Erdoğan ÖZBAY for serving on the committee.

Finally, I would also thanks to my family for their support and encouragement during my study.

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LIST OF SYMBOLS/ABBREVIATIONS

A_j	Cross-sectional area of viscoelastic device j
α	Bilinear stiffness ratio
C_d	Damping coefficient of viscoelastic damper
c_{cr}	Critical damping coefficient
Δ_u	Target roof displacement
Δ_y	Yield roof displacement
$(\Delta_u)_{eq}$	Equivalent target displacement
$G''(w)$	Shear loss modulus
$G'(w)$	Shear storage modulus
G'_j	Shear storage modulus
K_d	Stiffness of viscoelastic damper
K_{eq}	Equivalent stiffness
K_{VED}	Effective stiffness of viscoelastic damper
k'_j	Storage stiffness of viscoelastic damper
ξ_h	Hysteretic damping ratio
ξ_i	Inherent damping ratio
ξ_{eq}	Total equivalent damping ratio
ξ_{VED}	Viscous damping ratio provided by viscoelastic damper
m	Mass
M_w	Magnitude
M_y	Yield moment
μ	Ductility
η	Loss factor
S_a	Spectral acceleration
t_j	Thickness of the viscoelastic device *j*
T_{eq}	Equivalent period
u_o	Amplitude a cycle of periodic motion
V_d	Design force
V_u	Ultimate force capacity
V_y	Yield force
W	Energy dissipated by linear damper device
W_s	Maximum strain energy of the system
ω	Natural circular frequency

ϖ	Frequency of a cycle of periodic motion
RF	Reference frame
VE	Viscoelastic
VF	Viscous fluid
AMD	Active mass damper
HMD	Hybrid mass damper
PGA	Peak ground acceleration
PGV	Peak ground velocity
TLD	Tuned liquid damper
TMD	Tuned mass damper
VED	Viscoelastic damper
ADAS	Added damping and stiffness
MDOF	Multi degree of freedom
SDOF	Single degree of freedom
VEDF	Viscoelastically damped frame

CHAPTER 1

1. INTRODUCTION

Conventional structural seismic resistant systems, such as reinforced concrete frames (Chung et al., 1990; Pujol et al., 2000), steel moment resisting frames (Mazzolani, 2003; Karavasilis et al., 2008a), concentric braced frames (Youssef et al., 2007; Karavasilis et al., 2008b) or eccentric braced frames (Ghobarah and Abou-Elfath, 2001; Özel and Güneyisi, 2011) are currently designed to experience significant inelastic deformations and form a global plastic mechanism under moderate to strong earthquakes. Such a design philosophy which results in inelastic deformations, has several advantages including economy and reduced forces developed in structural members and foundation due to inelastic softening. However, inelastic deformations result in damage, residual drifts, and economical losses such as repair costs, costly downtime while the building is repaired and cannot be used or occupied, and perhaps, building demolition due to the complications associated with repairing and straightening large residual drifts (Ramirez and Miranda, 2009).

With recent development and implementation of modern structural protective systems, the entire structural engineering discipline is now undergoing a major change. The traditional idealization of a building or bridge as a static entity is no longer adequate. Instead, structures that incorporate structural protective systems must be analyzed and designed by considering their dynamic behavior. Modern structural systems able to achieve high performance, i.e., no damage under small and moderate earthquakes, and little damage which can be repaired without loss of building operation under strong earthquakes (Wada, 2010). Performance based seismic design is expected to focus on modern energy dissipation systems such as passive dampers and self-centering devices (Christopoulos and Filiatrault, 2006; Wada, 2010). If carefully designed, these systems will slightly increase the initial building design cost and significantly reduce the great life-cycle cost related to earthquake damage (Wada, 2010).

Recently, considerable attention has been paid to research and development of structural control devices, with particular emphasis on alleviation of wind and seismic response of buildings and bridges. In both areas, serious efforts have been undertaken to develop the structural control concept into a workable technology. In general, the modern protective control systems are divided into three classes, namely base isolation, passive energy dissipation, and active control systems (Soong and Spencer-Jr, 2002). Such systems are characterized by their capability to enhance energy dissipation in the structural systems in which they are installed. These devices generally operate on principles such as frictional sliding, yielding of metals, phase transformation in metals, deformation of viscoelastic (VE) solids or fluids and fluid orificing. Among them, base isolation system can now be thought as a more mature technology with extensive applications as compared with the other two (ATC 17-1, 1993). However, the other two possess more recent emerging technology and under progressing. An active control device is defined as a system which typically requires a large power source for operation of actuators which supply control forces to the structure. A semi-active control system is similar to the active control systems but the external energy requirements are orders of magnitude smaller than typical active control systems. A passive control system is defined as a system which does not requires an external power source for vibration response (Symans and Constantinou, 1999; Housner et al., 1999; Symans et al., 2008).

Passive energy dissipation systems are a viable alternative to upgrading the existing structures which are vulnerable to wind and earthquake damage. In the case of new buildings, supplemental damping could be used to reduce the member sizes (by designing for the same ductility levels as in conventional design) in order to have an economical structural system. Adding dampers in new buildings, in order to reduce the risk of damage, could itself prove to be economical in the long run (Munshi, 1997). Passive energy dissipation systems for seismic applications have been under development for a number of years with a rapid increase in implementations starting in the mid-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

(Constantinou and Symans, 1993; Whittaker et al., 1993). The result is reduced damage to the framing system. A number of passive energy dissipation devices are either commercially available or under development. Device 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 (Symans et al., 2008).

Viscoelastic dampers are widely used in many different fields. For example, viscoelastic dampers have often been employed in controlling the vibrations of aircrafts, aerospace, and machine structures to dampen the deployment process and vibrations from other sources (Bilbao et al., 2006; Lewandowski and Pawlak, 2011). In civil engineering, for a few decades, viscoelastic dampers have been used successfully in high-rise buildings to minimize wind effects. However, the application of viscoelastic dampers to reduce seismic response in buildings is relatively new in comparison to the use of metallic and friction devices (Craig et al., 2002). Moreover, in the literature, more recent studies included experimental investigations by Vulcano and Mazza (2000) and Asano et al. (2000), and an analytical investigation by Soda and Takahashi (2000), Tezcan and Uluca (2003), and Singh and Chang (2009) were available, and these studies also suggest that there is a potential for the use of viscoelastic dampers for the seismic protection of building structures.

1.1. Objectives of the Thesis

The main purpose of this thesis is to further examine the seismic behavior of the structures equipped with and without the passive energy dissipation device. Among the passive energy dissipation device, viscoelastic dampers were utilized and the application of the viscoelastic dampers to mid-rise and high-rise buildings was investigated. Analytical modeling of the steel moment resisting frames with and without viscoelastic dampers was achieved by using the nonlinear dynamic analysis program of DRAIN-2DX (Prakash et al., 1993). To simulate the time history response of the structures subjected to the recorded earthquakes, the non-linear time history analysis was carried out for all cases studies: a) a 5-storey steel frame and b) a 12 storey steel frame. In the analyses, 1991 Alkion, 1992 Erzincan, and 1999 İzmit earthquakes were employed as a ground motion. The results of analyses carried out on the frames are presented in terms of the roof displacement time history, the inter-storey drift ratios, the maximum plastic rotation of beams and columns as well as the energy time history plots, and then critically discussed.

1.2. Outline of the Thesis

Chapter 1-Introduction: Aim and objectives of the thesis are introduced.

Chapter 2-Literature review and background: A literature survey based on this thesis is given. For this, firstly, the research and development on structural control devices as well as their classification in the literature are explained. Secondly, the utilization of passive energy dissipation systems in the structural system is given. Thereafter, the properties and use of viscoelastic dampers in relation with the structural applications and available studies on this issue in the literature are summarized.

Chapter 3-Case study: This chapter provides a description of analytical models of the selected buildings. Additionally, the methodology used in the analysis and design of the structures equipped with viscoelastic dampers is summarized and details of every step is given in this chapter. Moreover, the properties of the ground motion records used in the nonlinear time history analysis are described in this chapter.

Chapter 4-Discussion of the results: Results obtained from the nonlinear time history analysis of the structures with and without viscoelastic dampers are presented. Discussion on the results of the analysis is given in this chapter.

Chapter 5-Conclusion: Conclusions based on results of this comparative studies are presented.

CHAPTER 2

2. LITERATURE REVIEW

2.1 Structural Control Systems

Conventional seismic design philosophy is based on the concept of balancing demand with capacity. The structures are designed to ensure that the ductility demands developed in structural members are balanced by the ductility capacities of those members. The structural members dissipate the seismic energy by undergoing large inelastic deformation, which leads to damage in those structural members. This design philosophy concentrates on preventing structural collapse, but higher levels of performance (e.g. Immediate Occupancy) may be required of modern buildings. For example, the interstory drifts required to achieve significant energy dissipation in the ductile structural members are often large and may result in severe damage to nonstructural components. Substantial damage to nonstructural components could affect the overall performance of the structures and preclude achieving Life Safety or Immediate Occupancy levels of performance (Craig et al., 2002).

In recent years, considerable attention has been paid to research and development of structural control devices, with particular emphasis on alleviation of wind and seismic response of buildings and bridges. In both areas, serious efforts have been undertaken to develop the structural control concept into a workable technology. As shown in Table 2.1, the modern protective control systems studied by the structural control are classified into three broad areas, namely base isolation, passive energy dissipation, and active control (Soong and Spencer-Jr, 2002). After a brief summary of these control systems, the next section will be focused on studies regarding passive energy dissipation systems, in particularly viscoelastic dampers.

A seismic isolation system is typically placed at the foundation of a structure. By means of its flexibility and energy absorption capability, the isolation system partially reflects and partially absorbs some of the earthquake input energy before this energy can be transmitted to the structure. However, it cannot be used either to reduce the individual displacement of one structure or to connect two neighboring structures. Passive energy dissipation devices for structural applications are similar to seismic isolation technology. Their basic function is to absorb or consume a portion of the input energy, thereby reducing energy dissipation demand on primary structural members and minimizing possible structural damage. Contrary to semi-active or active systems, there is no need for an external supply of power. Semi-active and active structural control is an area of structural protection in which the motion of a structure is controlled or modified by means of the action of a control system through some external energy supply. However, semi-active systems require only nominal amounts of energy to adjust their mechanical properties, and unlike fully active systems, they cannot add energy to the structure (Warnotte et al., 2007).

Among the structural control system, base isolation system can now be thought as a more mature technology with extensive applications as compared with the other two (ATC 17-1, 1993). On the other hand, passive energy dissipation systems cover a range of materials and devices for improving damping, stiffness, and strength, and can be utilized both for seismic hazard mitigation and for rehabilitation of aging or deficient structures (Constantinou et al., 1998; Hanson and Soong, 2001). By and large, such systems are characterized by their capability to enhance energy dissipation in the structural systems in which they are installed. These devices generally operate on principles such as frictional sliding, yielding of metals, phase transformation in metals, deformation of viscoelastic (VE) solids or fluids and fluid orificing.

Active and semi-active structural control systems are a natural evolution of passive control technologies. The possible use of active control systems and some combinations of passive and active systems (hybrid systems) as a means of structural protection against seismic loads has received considerable attention in recent years. In deed, the active and semi-active control systems are force delivery devices integrated with real-time processing evaluators/controllers and sensors within the structure. They act simultaneously with the hazardous excitation to provide enhanced structural behavior for improved service and safety (Soong and Spencer-Jr, 2002).

Table 2.1 Structural protective systems (Soong and Spencer-Jr, 2002)

Seismic isolation	Passive energy dissipation	Semi-active and active control
-Elastomeric bearings -Lead rubber bearings -Sliding friction pendulum	-Metallic dampers -Friction dampers -Viscoelastic dampers -Viscous fluid dampers -Tuned mass dampers -Tuned liquid dampers	-Active bracing systems -Active mass dampers -Variable stiffness or damping systems -Smart materials

2.2 Passive Energy Dissipation Systems

Warnotte et al. (2007) reported that passive control techniques are based on the artificial increase of the dissipation capacity, obtained by means of the insertion, in proper positions, of special devices of which both the stiffness and strength have to be defined in order to achieve: i) a limitation of the relative move of buildings one toward the other and ii) energy dissipation. On the other hand, the primary function of supplemental dampers is to reduce the structural response by dissipating the energy in the dampers instead of in the structure so that the potential damage in the framing system will decrease significantly during an earthquake (Symans et al., 2008).

Many different types of passive energy dissipation devices have been considered for seismic protection applications in buildings. Metallic energy dissipation devices depend upon plastic deformation of metallic materials, such as mild steel or lead. Friction dampers dissipate energy through the friction that develops between two solid bodies sliding relative to one another. Viscoelastic dampers use polymeric materials which dissipate energy when subjected to shear deformations, while viscous fluid dampers dissipate energy when subjected to a velocity input. Tuned mass dampers and tuned liquid dampers can also be treated as passive energy dissipation devices (Towashiraporn et al., 2002).

2.2.1 Metallic Dampers

One of the most effective mechanisms for dissipating input seismic energy in a structure is through the inelastic deformation of metallic materials under time-dependent cyclic loading. Mild structural steel is a popular (and inexpensive) choice for a metallic energy dissipation device because of its relatively high elastic stiffness, good ductility, and its high potential for dissipating energy in the post-yield region. Alternative materials include lead and shape-memory alloys. The latter offer very high ductility, but present a more complicated and temperature-sensitive multi-phase behavior (Towashiraporn et al., 2002).

The idea of utilizing metallic hysteretic dampers to dissipate a portion of the input energy in structures first appeared in the early 1970s. Kelly et al. (1972), Skinner et al. (1973), and Skinner et al. (1975) reported on the conceptual and experimental development of thin U-shaped flexure strips that could be deformed by rolling and unrolling. Tyler (1978) proposed the concept of the tapered flexure as an energy dissipation device. This is a flexural device in which the taper is designed so that the stress is more uniformly distributed over the whole length of the device. This device, which consists of a number of identical triangular structural steel plates layered in parallel, clamped along one edge and loaded at the opposite vertex, is typically installed within a frame bay between chevron braces and the bottom flange of the beam above. This damper primarily resists horizontal forces associated with interstorey drift by uniform flexural deformation of the individual plates. Following the triangular plate and U-shaped flexural strip energy dissipaters, a wide variety of such devices has been studied or tested (Bergman and Goel, 1987; Whittaker et al., 1991). Many of these devices use mild steel plates with X-shapes so that yielding is spread almost uniformly throughout the material. Figure 2.1 illustrates a typical X-shaped plate damper or added damping and stiffness (ADAS) device. Other materials, such as lead and shape-memory alloys, have also been studied by Aiken and Kelly (1992). Some particularly desirable features of these devices are their stable hysteretic behavior, low-cycle fatigue property, long term reliability, and relative insensitivity to environmental temperature. Hence, numerous analytical and experimental investigations have been conducted to determine these characteristics of individual devices (Soong and Spencer-Jr, 2002).

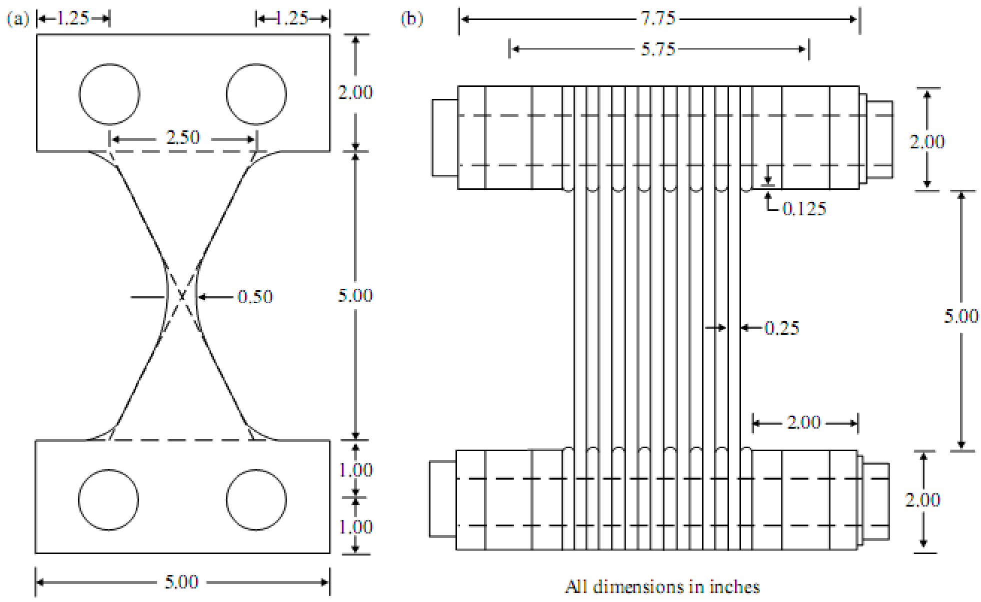

Figure 2.1. A typical X-shaped plate damper or added damping and stiffness (ADAS) device (Whittaker et al., 1991)

After getting confidence in their performance based primarily on experimental evidence of such devices, researchers and practitioners turned their interest to possible implementation of metallic dampers in real structures. The first structural application of metallic energy dissipation devices took place in New Zealand and Japan. A number of these interesting applications were reported in the study of Skinner et al. (1980) and Fujita (1991). More applications include the use of ADAS dampers in the seismic upgrade of existing buildings in Mexico (Martinez-Romero, 1993) and in the USA (Perry et al., 1993). Another application can be given by Goodno et al. (1998). They investigated the application of cladding connectors using tapered energy dissipation devices developed by Pinelli (1992). A 20-story steel frame building located in Oakland was selected to carry out computational analyses and check the performance of the tapered connectors. DRAIN-2DX, which can perform nonlinear time-history dynamic analysis, was used. Over a thousand tapered energy dissipative cladding connectors were added to the building model.

2.2.2 Friction Dampers

Friction dampers utilize the mechanism of solid friction that develops between two solid bodies sliding relative to one another to provide the desired energy dissipation. Several types of friction dampers have been developed for the purpose of improving seismic response of structures (Soong and Spencer-Jr, 2002). An example of X braced friction damper studied by Pall and Marsh (1982) is given in Figure 2.2. During cyclic loading, the mechanism enforces slippage in both tensile and compressive directions. Generally, friction devices generate rectangular hysteretic loops similar to the characteristics of Coulomb friction. After a hysteretic restoring force model has been validated for a particular device, it can be readily incorporated into an overall structural analysis.

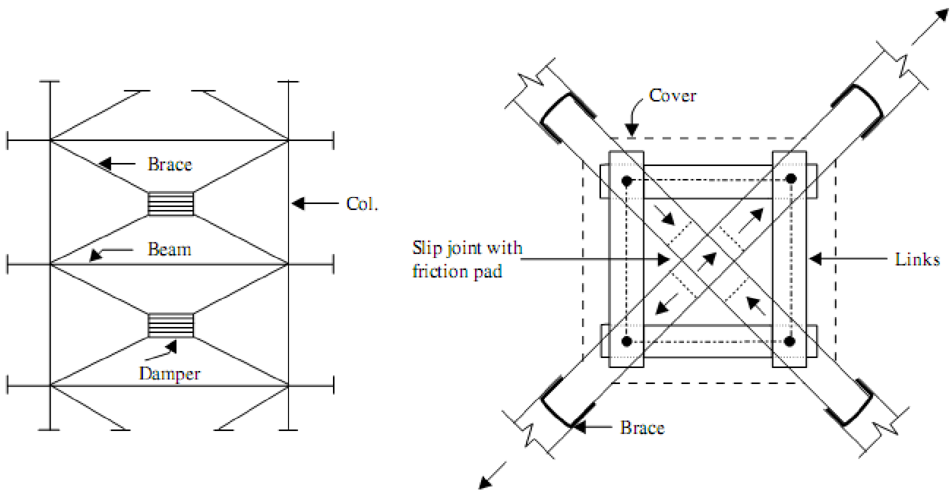

Figure 2.2. An example of X braced friction damper (Pall and Marsh 1982)

There have been a number of structural applications of friction dampers aimed at providing enhanced seismic protection of new and retrofitted structures. This activity in North America is primarily associated with the use of Pall friction devices in Canada and the USA (Pall and Marsh, 1982) and slotted-bolted connection in the USA (Grigorian et al., 1993). For example, the applications of friction dampers to the McConnel Library of the Concordia University in Montreal, Canada are discussed in the study of Pall and Pall (1993).

2.2.3 Viscous Fluid Dampers

Viscous fluid (VF) dampers include a piston head with orifices contained in a hollow cylinder filled with fluid which is mostly a compound of silicone or similar type of oil. Energy is dissipated in the damper as the piston rod moves through the fluid and forces the fluid to flow through the orifices in the piston head (Lee and Taylor, 2001; Hanson and Soong, 2001). Since the fluid flows at high velocities, it causes friction (between fluid particles and the piston head) which produces energy dissipation in the form of heat (Makris, 1998).

Figure 2.3 demonstrates a typical viscous damper. A central piston strokes through a fluid-filled chamber. As the piston moves it pushes fluid through orifices around and through the piston head. Fluid velocity is very high in this region so the upstream pressure energy converts almost entirely to kinetic energy. When the fluid subsequently expands into the full volume on the other side of the piston head it slows down and loses its kinetic energy into turbulence. There is very little pressure on the downstream side of the piston head compared with the full pressure on the upstream side of the piston head. This difference in pressures produces a large force that resists the motion of the damper (Lee and Taylor, 2001).

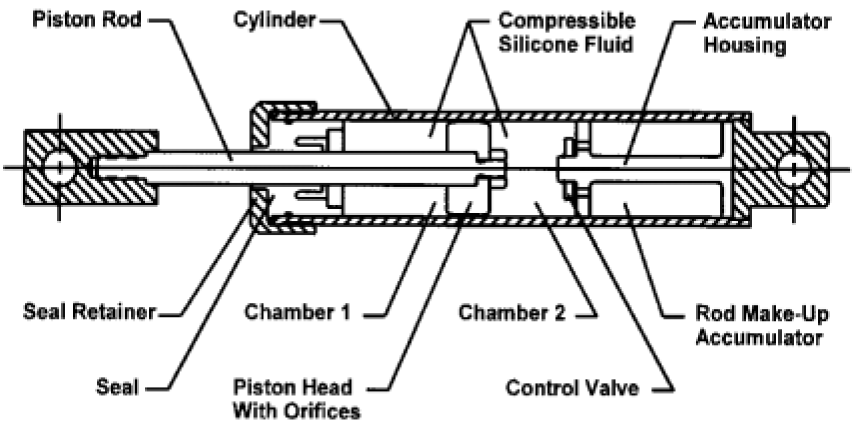

Figure 2.3 Typical viscous damper (Lee and Taylor, 2001)

Viscous fluid dampers have in recent years been incorporated into a large number of civil engineering structures. In several applications, they were used in combination with seismic isolation systems. For example, in 1995, viscous fluid dampers were incorporated into base isolation systems for five buildings of the San Bernardino County Medical Center, located close to two major fault lines. The five buildings required a total of 233 dampers, each having an output force capacity of 320 000 lb and generating an energy dissipation level of 3000 horse-power at a speed of 60 in/s (Soong and Spencer-Jr, 2002).

2.2.4 Tuned Mass Dampers

The objectives of incorporating a tuned mass damper (TMD) into a structure is basically the same as those associated with metallic dampers and other energy dissipation devices, namely to reduce energy dissipation demand on the primary structural members under the action of external forces. This reduction, in this case, is accomplished by transferring some of the structural vibrational energy to the tuned mass damper which, in its simplest form, consists of an auxiliary mass-spring dashpot system anchored or attached to the main structure. In tuned mass dampers, typically a solid concrete or metal block acts as the secondary mass. Additional spring and dampers are used to attach this secondary mass to the primary structure, and to provide the restoring and dissipate mechanisms needed to tune the system for near-optimal response under various types of dynamic excitations. It is noted that a passive tune mass dampers can only be tuned to a single structural frequency. While the first-mode response of a multi degree of freedom (MDOF) structure with TMD can be substantially reduced, the higher mode response may in fact increase as the number of stories increases. For earthquake-type excitations, the response reduction is large for resonant ground motions and diminishes as the dominant frequency of the ground motions gets further away from the structure's natural frequency to which the TMD is tuned (Warnotte et al., 2007).

It is also noted that the interest in using tuned mass dampers for vibration control of structures under earthquake loads has resulted in some innovative developments. An interesting approach is the use of a tuned mass damper with active capability, the so called active mass damper (AMD) or hybrid mass damper (HMD). Systems of this type have been implemented in a number of tall buildings in recent years in Japan (Soong and Spencer-Jr, 2002).

2.2.5 Tuned Liquid Dampers

The basic principles involved in applying a tuned liquid damper (TLD) to reduce the dynamic response of structures is quite similar to that discussed for the tuned mass damper. In effect, a secondary mass in the form of a body of liquid is introduced into the structural system and tuned to act as a dynamic vibration absorber. However, in the case of TLDs, the damper response is highly non-linear due either to liquid sloshing or the presence of orifices. Tuned liquid dampers have also been used for suppressing wind-induced vibrations of tall structures. In comparison with tuned mass dampers, the advantages associated with TLDs include low initial cost, virtually free of maintenance and ease of frequency tuning. The TLD applications have taken place primarily in Japan for controlling wind-induced vibration. Examples of TLD-controlled structures include airport towers and tall buildings (Soong and Spencer-Jr, 2002).

2.2.6 Viscoelastic Dampers

Viscoelastic (VE) dampers are widely used in many different fields. For example, VE dampers have often been employed in controlling the vibrations of aircrafts, aerospace, and machine structures to dampen the deployment process and vibrations from other sources (Bilbao et al., 2006; Lewandowski and Pawlak, 2011). In civil engineering, for a few decades, viscoelastic dampers have been used successfully in high-rise buildings to minimize wind effects. However, the application of VE dampers to reduce seismic response in buildings is relatively new in comparison to the use of metallic and friction devices (Craig et al., 2002). Moreover, in the literature, more recent studies included experimental investigations by Vulcano and Mazza (2000) and Asano et al. (2000), and an analytical investigation by Soda and Takahashi (2000), Tezcan and Uluca (2003), and Singh and Chang (2009) were available, and these studies also suggest that there is a potential for the use of viscoelastic dampers for the seismic protection of building structures.

Viscoelastic dampers are devices that behave in a manner that both viscous damping and elastic spring characteristics. The elastic component has a linear relationship with deformation, whereas the viscous force has a phase difference. The corresponding stress-strain relationship is shown in Figure 2.4 (Conner, 2003). The viscoelastic damper behaviour can be idealized by very simple viscoelastic linear models constituted by an elastic spring and a dashpot acting in parallel (Kelvin model) or in series (Maxwell model). The calibration of either Kelvin model or Maxwell model may be done tuning the corresponding stiffness and damping constants to the fundamental frequency of the entire damped structure. However, the description of the variations in the viscoelastic damper properties, depending on the frequency and temperature variations, may be inaccurate. In particular, Kelvin and Maxwell models can be calibrated exactly to match the experimental results corresponding only to a given value of the frequency for an assigned temperature. Nevertheless, both the models can be useful to obtain an initial proportioning of the viscoelastic dampers. Figure 2.5 reveals models for the description of the viscoelastic damper response (Mazza and Vulcano, 2010).

Viscoelastic materials used in structural applications are usually copolymers or glassy substances that dissipate energy through shear deformation. A typical VE damper, which consists of VE layers bonded with steel plates, is shown in Figure 2.6. The seismic energy is dissipated when the structural dynamic response induces relative motion between the outer steel flanges and the center plate (Soong and Spencer-Jr, 2002). Significant advances in research and development of viscoelastic dampers, particularly for seismic applications, have been made in recent years through analyses and experimental tests.

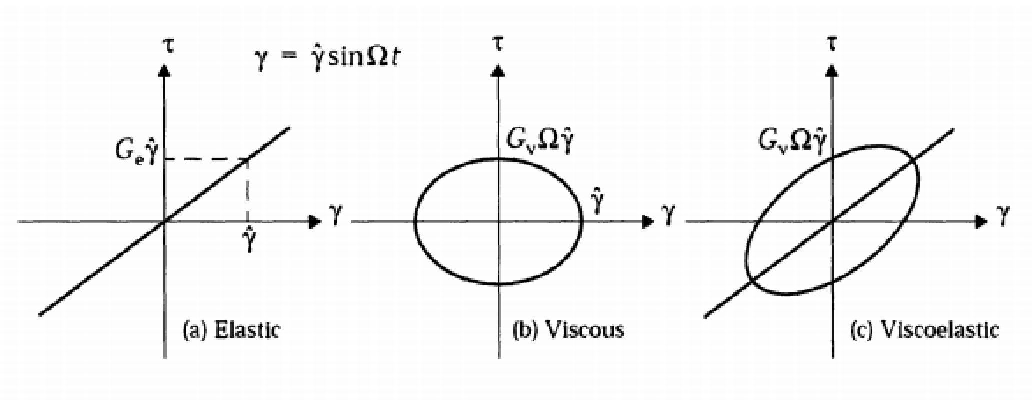

Figure 2.4 Stress-strain relationship for elastic, viscous, and viscoelastic materials (Conner, 2003)

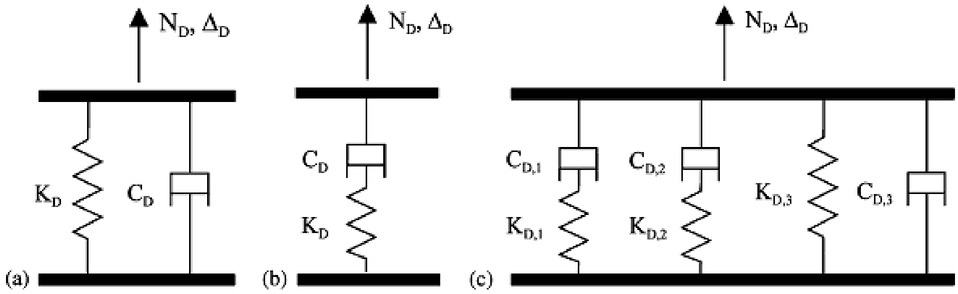

Figure 2.5. Models for the description of the viscoelastic damper response: (a) Kelvin model, (b) Maxwell model, and (c) Generalized model (Mazza and Vulcano, 2010)

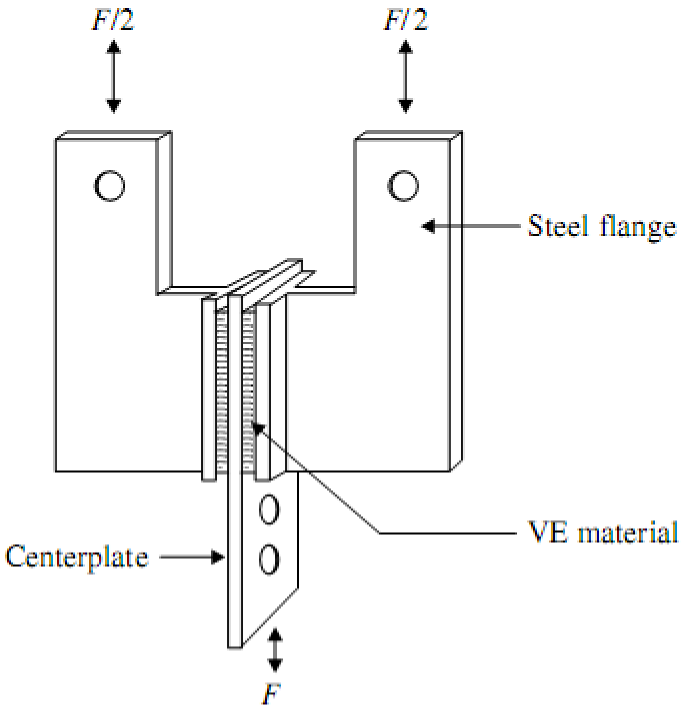

Figure 2.6. Typical VE damper configuration (Soong and Spencer-Jr, 2002)

In the work of Chang et al. (1992), the seismic response characteristics of a 2/5 scale steel frame structure with added viscoelastic dampers were investigated experimentally. Damper property tests were carried out using an MTS axial-torsional testing system. A lumped mass system simulating the dynamic properties of the prototype structure was accomplished by adding steel plates at each floor level. The ends of the first floor columns were welded to base plates which were bolted to a large concrete boat-type foundation secured to a shaking table. The diagonal bracing members with added viscoelastic dampers were connected by bolts to the gusset plates welded to the girder. The major emphasis was placed on the ambient temperature effect. Results indicated that, even at high temperatures, the viscoelastically damped structure could achieve a significant reduction of structural response as compared to the case without dampers.

In the study of Aiken et al. (1993), a shake table experiment on a scale model of a 9-story moment resisting steel frame was performed. The viscoelastic dampers were inserted to the structural frames through single diagonal braces. The viscoelastic material utilized in this experiment was an acrylic copolymer developed by 3M Co. A total of fourteen different ground motions were applied to the model. Both experimental and analytical results indicated the superior performance of the viscoelastic damped steel frames. Response comparisons for selected ground motions showed that interstory drifts were decreased by 10 to 60 percent, while floor level accelerations were reduced by 25 to 60 percent over those of the unmodified steel frames.

Chang et al. (1993) investigated the relationship between the performance of the viscoelastic dampers and factors affecting their properties such as an ambient temperature. The test structure was a 2/5-scale 5-story steel frame sitting on a shake table. It was found that as the ambient temperature increased, the viscoelastic materials became softer and their efficiency decreased. Despite the reduction in energy dissipation capacity due to the increase in temperature (as high as 42°C), the performance of the viscoelastic dampers was still remarkable. Reduction in interstory drift and floor acceleration responses of as high as 73 and 78 percent, respectively, were obtained.

An experimental study regarding the effect of viscoelastic dampers on inelastic response of reinforced concrete structures was conducted by Lobo et al. (1993). A one-third scaled model of a 3-story lightly reinforced concrete framed building was tested under simulated ground motions using a shake table. The reinforced concrete building model was retrofitted with viscoelastic damped diagonal braces in the interior bay of each frame. The test results revealed that retrofitting reinforced concrete frames with viscoelastic dampers can diminish the overall response, but more prominently, can reduce the risk of a collapse mechanism in the structure.

Asano et al. (2000) carried out an experimental study on viscoelastic material dampers inserted in the frame systems. The mechanical properties of viscoelastic damper as well as the mechanical model of the damper were examined comparatively. The mechanical properties of four kinds of materials, which were currently produced, were studied based on dynamic loading experiment of full-scale dampers. The two mechanical models were constructed. One considered the dependency on the number of repeated cycle and amplitude, and another considered the dependency on frequency. By comparing with the experimental results, the constructed models agreed accurately. The earthquake response analyses of a building with the dampers were carried out. Results indicated that the responses were reduced in the analyses remarkably.

Choi et al. (2003) generated a non-linear response spectrum using a non-linear oscillator model to simulate a building with viscoelastic dampers installed. The parameters used in the non-linear damper model were obtained experimentally from dynamic loading tests. Figure 2.7 compares the experimental and modeled hysteresis cycles of the viscoelastic damper at several displacements. The results showed that viscoelastic dampers effectively reduced the seismic displacement response of a structure, but transmitted more seismic force to the structure, which essentially increased its seismic acceleration response.

Min et al. (2004) conducted vibration tests on 5-storey single bay steel structure with added viscoelastic dampers. The mechanical properties of viscoelastic dampers and the dynamic characteristics of the model structure were obtained from experiments using harmonic excitation, and the results were used in the design process. Figure 2.8 shows the test specimen made of two layers of viscoelastic materials while Figure 2.9 demonstrates the location of viscoelastic devise and accelerometers in the frame system. In their study, the additional damping ratios required to reduce the maximum response of the structure to a desired level were obtained. Then, the size of dampers to realize the required damping ratio was determined using the modal strain energy method by observing the change in modal damping ratio due to the change in damper stiffness. The designed viscoelastic dampers were installed in the first and the second stories of the model structure. The results from experiments using harmonic and band limited random noise indicated that after the dampers were installed the dynamic response of the full-scale model structure reduced as desired in the design process.

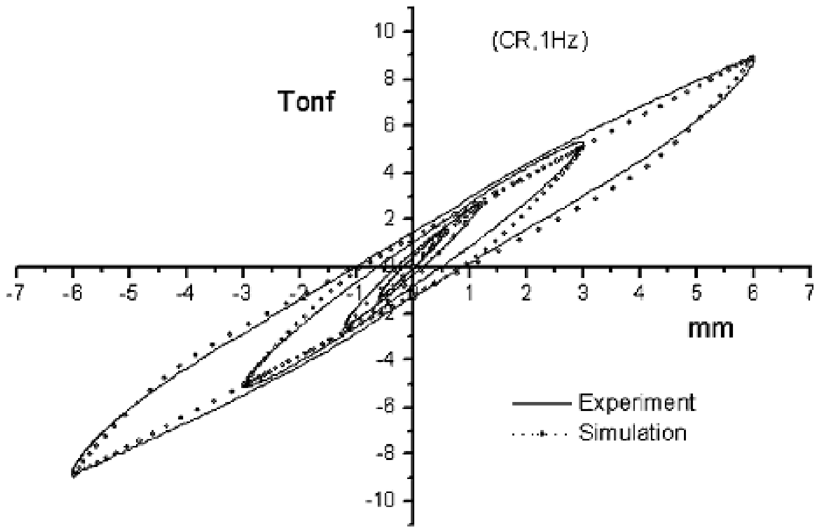

Figure 2.7 Hysteresis cycles of a viscoelastic damper obtained experimentally and using the non-linear model (Choi et al., 2003)

In the study of Chang and Lin (2004), the seismic response of full-scale structure with added viscoelastic dampers were investigated experimentally. The ISD110 viscoelastic dampers made by 3M Company were installed to the long (East–West) direction of the structure. The two single-bay full-scale five-story structures are shown in Figure 2.10. Both structures were identical. The center to center distances between columns in the East–West and North–Southern short directions are 6.0 and 4.0 m, respectively. Each story had a height of 2.6 m. In addition to carrying out ambient and free vibration tests, the dynamic responses of the structure under four moderate earthquakes were recorded and analyzed. The ambient vibration tests showed that the increase of damping in the full-scale structure was not obvious because of the small input energy. Nevertheless, in the free vibration tests and under earthquake ground motions, the increase of damping ratio was very significant. In addition to providing important database for verifying the effectiveness of applying the viscoelastic dampers to full-scale structures, their study also obtained important experiences concerning the implementation and detailed design of damper connections for practical applications.

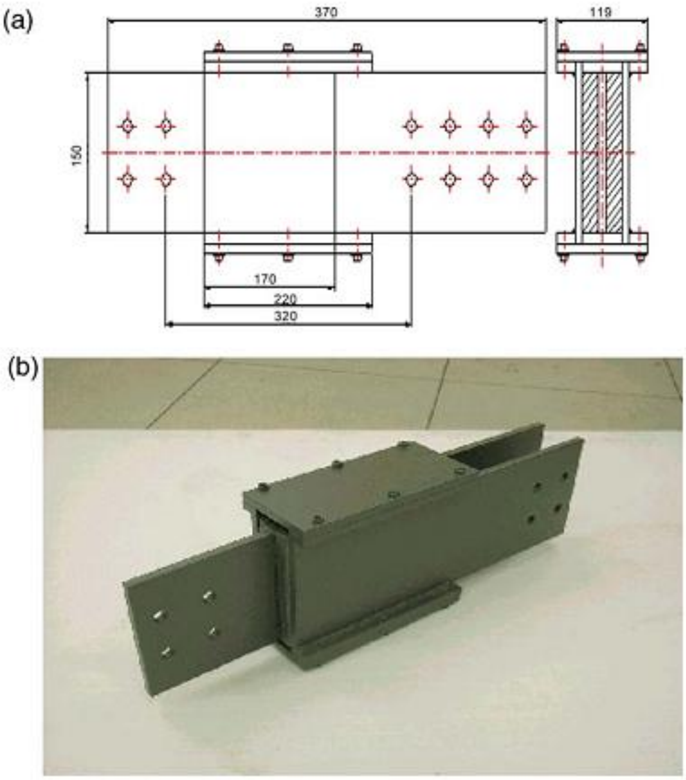

Figure 2.8 Viscoelastic dampers used in the experiment: (a) Dimension of the designed viscoelastic device; (b) Photograph of the viscoelastic device (Min et al., 2004)

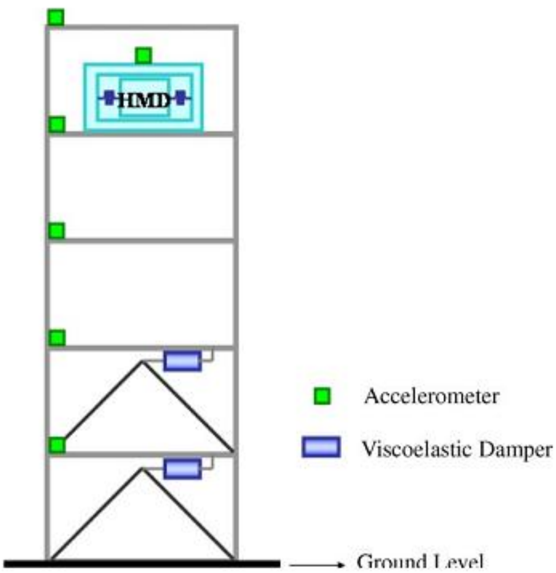

Figure 2.9 Locations of the viscoelastic devise and accelerometers in the frame system (Min et al., 2004)

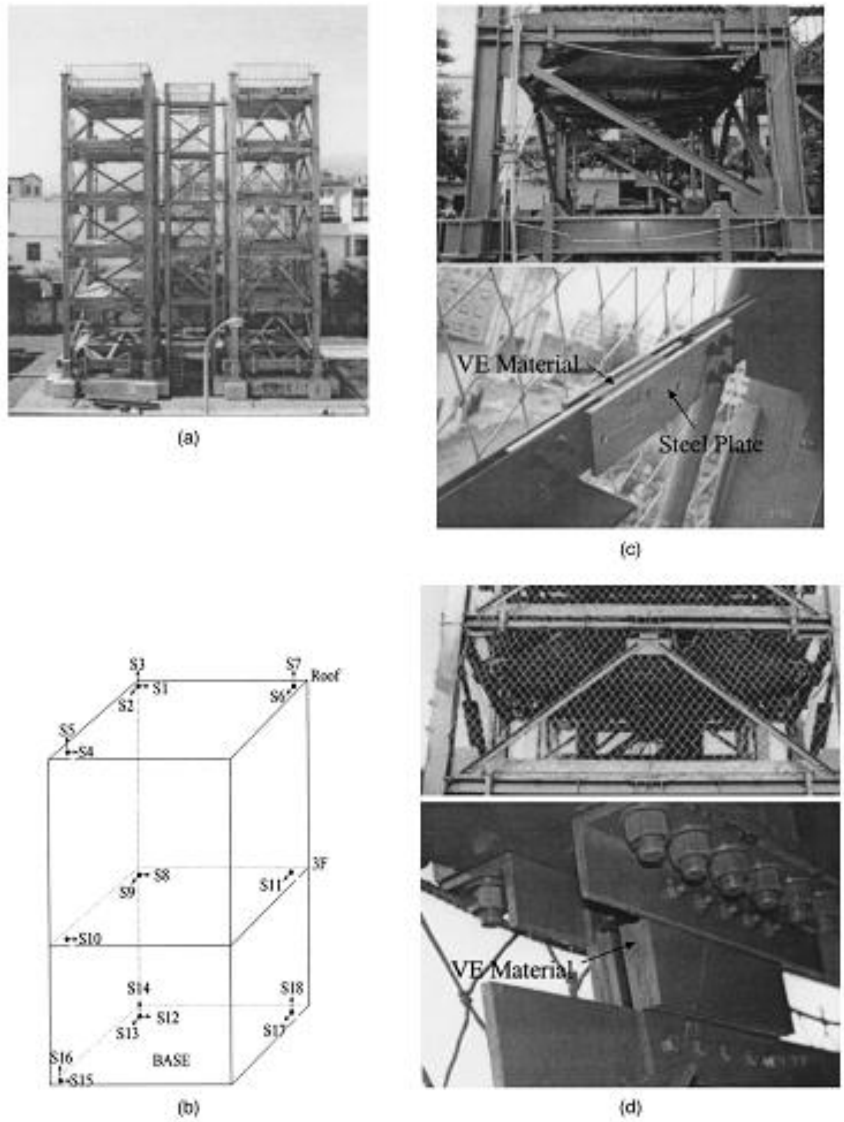

Figure 2.10 Details of the full-scale five-story structures to be tested a) Layout of full-scale five story structures; b) Layout of instruments; c) Viscoelastic frame with diagonal–damper–brace assembly, and d) Viscoelastic frame with floor–damper–brace assembly (Chang and Lin, 2004)

Vulcano and Mazza (2000) studied the seismic performance of frames using different dissipative braces. For this, a typical five-storey reinforced concrete frame, which was designed according to Eurocode 8, was employed. To protect the frame, the following dissipative braces were supposed inserted into the frame itself: cross-braces with hysteretic (friction or yielding) dampers, chevron braces with viscoelastic damper, diagonal brace with viscous damper. The effects produced by the dissipative braces on the response of the framed structure were evaluated assuming different properties of the frame members, braces and dissipative devices. Aspects concerning the behaviour and modeling of the dissipative braces were discussed.

In the study of Soda and Takahashi (2000), the performance based seismic design of building structures with viscoelastic dampers was investigated. The first part of the study outlined mechanical properties of a viscoelastic damper which were temperature and frequency dependent. The second part included an analytical method for quantifying damping capacity of a building with the viscoelastic dampers whose mechanical properties were represented by generalized Maxwell models. It was shown that the damping effect depended on the stiffness of members to support the damper, and that there was an optimum amount of the damper to provide the structure with the maximum damping factor. It was also proved that the viscoelastic dampers were available to assure the ultimate stability of multi-story buildings for even quite intense ground motions.

Lee et al. (2002) used conventional analysis methods for building structures with added viscoelastic dampers. Direct integration, complex mode superposition, and modal strain energy method were compared, and a procedure based on rigid diaphragm assumption and matrix condensation technique was proposed for application in the preliminary analysis and design stages. The results from the various analysis methods with and without the matrix condensation were compared, in view of both accuracy and efficiency. According to the eigenvalue analysis the major vibration modes were mostly preserved after the matrix condensation. It was also found that the matrix condensation technique applied to dynamic analysis of a structure with added viscoelastic dampers provided quite accurate results in significantly reduced time, regardless of the plan shape and the location of the viscoelastic dampers.

Guo et al. (2002) studied the seismic reliability analysis of hysteretic structure-viscoelastic dampers with and without parameter uncertainties. The dynamic response of a hysteretic shear beam type structure with viscoelastic dampers under random seismic excitation was first evaluated in the state space utilizing stochastic response analysis and equivalent linearization technique. By taking the maximum story drift as a measure of structure limit state and the maximum deformation of viscoelastic material as a measure of damper limit state, the failure probabilities of the structure and viscoelastic dampers either for a given earthquake event or during the entire service time were then estimated using the first-order reliability method and the response surface approach for the system with and without uncertainties, respectively. Finally, the framework was applied to a ten-story building with and without viscoelastic dampers and parameter uncertainties. It was found that the existence of uncertainties reduced the reliability of the building but the installation of viscoelastic dampers of proper parameters significantly enhanced the reliability of the building.

Tezcan and Uluca (2003) investigated the reduction of earthquake response of plane frame buildings by viscoelastic dampers. The study focused on the viscoelastic dampers to be used as energy-absorbing devices in buildings. Their advantages and disadvantages as well as their application on three model structures were described. The analytical studies of the model structures exhibiting the structural response reduction due to these viscoelastic devices were presented. In order to exhibit the benefits of viscoelastic dampers, a nonlinear time history analysis is carried out for all case studies: (a) a 7-storey steel frame, (b) a 10-storey reinforced concrete frame, and (c) a 20-storey reinforced concrete frame. The top storey relative displacements as well as the top storey absolute accelerations and also the base shear values obtained indicated that these viscoelastic dampers when incorporated into the super-structure behave like a break pedal and reduce the earthquake response significantly in proportion to the amount of damping supplied in these devices.

In the study of Lin et al. (2003), a seismic displacement based design method for new and regular buildings equipped with passive energy dissipation systems were investigated. In their study, viscous, friction, yielding, and viscoelastic dampers as passive energy dissipation systems were studied. Using the substitute structure approach for the building structure and simulating the mechanical properties of the passive energy dissipation devices by the effective stiffness and effective viscous damping ratio, a rational linear iteration method was proposed. A target displacement was at first specified and then the corresponding design force, strength, and stiffness were obtained. Comprehensive procedures for displacement based design of several buildings with passive energy dissipation systems were presented. The results were verified by dynamic inelastic time history analysis. Based on the study, it was found that the proposed displacement based design method was straightforward and could accurately predict the nonlinear behavior of buildings equipped with passive energy dissipation systems

Palmeri and Ricciardelli (2006) studied the problem of estimating the fatigue life of structural components of tall buildings provided with viscoelastic dampers. First, a dynamic model of the building in the modal space was established, which accounted for the viscoelastic memory of the devices, as opposed to the classical modal strain energy method, which brought an equivalent Kelvin–Voigt model. A cycle counting method was then adopted, based on the separation of the building dynamic response into quasi-static and resonant parts. Doing so the total number of cycles was evaluated as the sum of the low-frequency (quasi-static) and high-frequency (resonant) cycles. Finally, the fatigue life was evaluated using the well-known Palmgren–Miner rule of linear damage accumulation. An application to a 15-storey building was included in their study. It was found that the memory of the devices significantly affected the results. In particular, inaccuracies were found in the frequency response function, as well as in the standard deviation of the response and in its apparent oscillation frequency, when the memory was neglected. Based on the results, it was also concluded that appropriate dynamic models including memory had to be used when estimating the fatigue life of a building provided with viscoelastic dampers, in case a reasonable level of accuracy was required.

Ou et al. (2007) examined the seismic response analysis of structures with velocity dependent passive energy dissipation devices such as viscous and viscoelastic dampers. The modeling of a damper–brace component composed of a viscous or viscoelastic damper connecting with braces in series was presented. Figure 2.11 demonstrates the analytical model of the structures with velocity dependent dampers. Several key parameters influencing the energy dissipation efficiency of such dampers in the damper–brace component were investigated and the relationships of the parameters and efficiency of the dampers were established. An equivalent model for the passive energy dissipation system was developed, which could significantly simplify the dynamic analysis of structures with the velocity-dependent dampers. The seismic responses of the structure with the viscous and viscoelastic dampers were analyzed to verify the effectiveness of the passive energy dissipation devices for suppression of dynamic responses of structures and the reliability of the proposed simplified computational methods. It was concluded that the seismic responses of the structure with the viscous and viscoelastic dampers, calculated based on the equivalent model, were found to be in good agreement with those obtained by the exact model. This illustrated that the equivalent model was able to accurately predict the seismic responses of structures with the viscous and viscoelastic dampers.

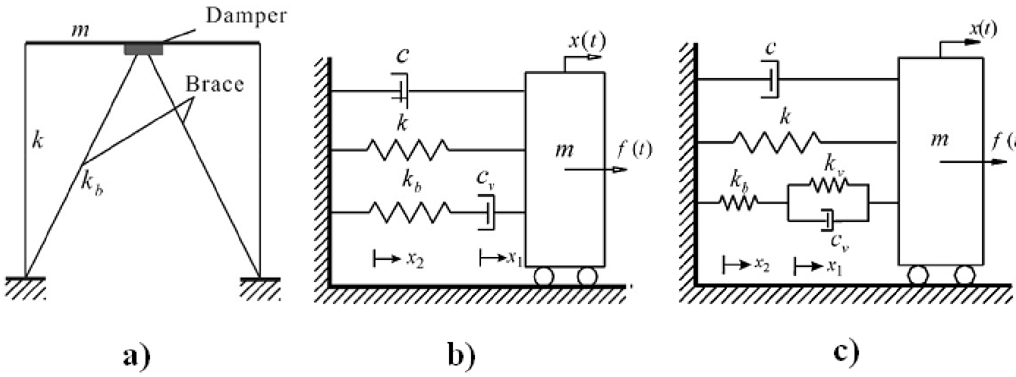

Figure 2.11 Analytical models of the structure with velocity dependent dampers a) A single-story structure with a damper, b) The structure with a viscous damper, and c) The structure with a viscoelastic damper (Ou et al., 2007)

In the study of Singh and Chang (2009), viscoelastic dampers were used in structures to mitigate dynamic effects. In their study, the two classical mechanical models consisting of Kelvin chains and Maxwell ladder were employed. They proposed to use equivalent mechanical analog models such as Maxwell ladder or Kelvin or Voigt chains shown in Figure 2.12. The models of varying complexities from simple Maxwell element to differential models with fractional and complex order derivatives were utilized to represent their frequency-dependent force deformation characteristics. More complex models were able to capture the frequency dependence of the material properties better, but were difficult to use in analyses. However, the classical models consisting of assemblies of Kelvin and/or Maxwell elements with an adequate number of parameters could be formed to capture the frequency dependence as accurately as the more sophisticated fractional derivative models could do. The main advantage in adopting these classical models was a simpler and smaller system of equations, which could be conveniently analyzed for nonlinear and linear systems. It was pointed out that these mechanical models were as effective as the fractional derivative model in capturing the effect of the frequency dependence of the material properties in response calculations and were more convenient to use in dynamic response analyses.

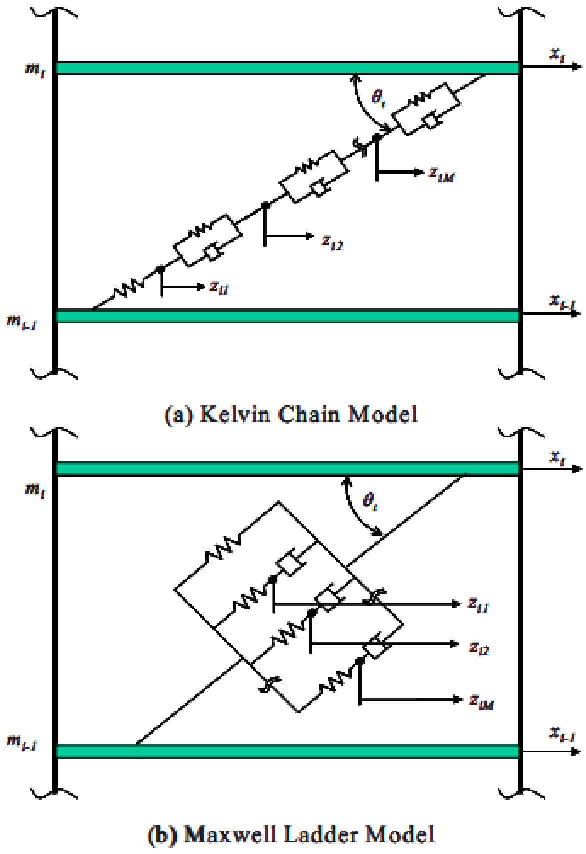

Figure 2.12 Auxiliary coordinates of the Kelvin chain and Maxwell ladder models (Singh and Chang, 2009)

Ragni et al. (2011) investigated the displacement based design method for the seismic design of steel frames equipped with dissipative braces. Attention was focused on concentric braced steel frames with pinned beam-to-column joints in which the bracing system (with viscoelastic or elastoplastic dissipative devices) was the main seismic resistant component. The proposed design method utilized an equivalent continuous model where flexural deformability and shear deformability were related respectively to columns and diagonals of the bracing system. In this way, the analytical expressions of the required flexural and shear stiffness distributions were obtained. It was reported that these expressions were quite simple and could be conveniently used in preliminary design of dissipative diagonal braces and columns. Examples were shown for steel frames with dissipative braces based on elastomeric dampers (viscoelastic devices) and steel frames with buckling-restrained braces (elastoplastic devices). Two design arrangements were considered for the frame: (case a) the different designed profiles were adopted for each floor and (case b) the same designed profile was assumed for two consecutive floors as common in many realistic situations. For the viscoelastic braces, the stiffness equal to the design values was used in the finite element model. Figure 2.13 illustrates the maximum inter-storey drifts and maximum brace elongations at each level obtained from the analysis. Results of time history analyses were illustrated and discussed in order to evaluate the effectiveness of the proposed displacement based design procedure. It was found that in the case of braces with dampers, satisfactory results were achieved. In the cases of buckling restrained bracing systems, a lower (but still satisfactory) agreement between design and response results was observed in consequence of the localization of inter-storey drifts typical of elastoplastic bracing systems.

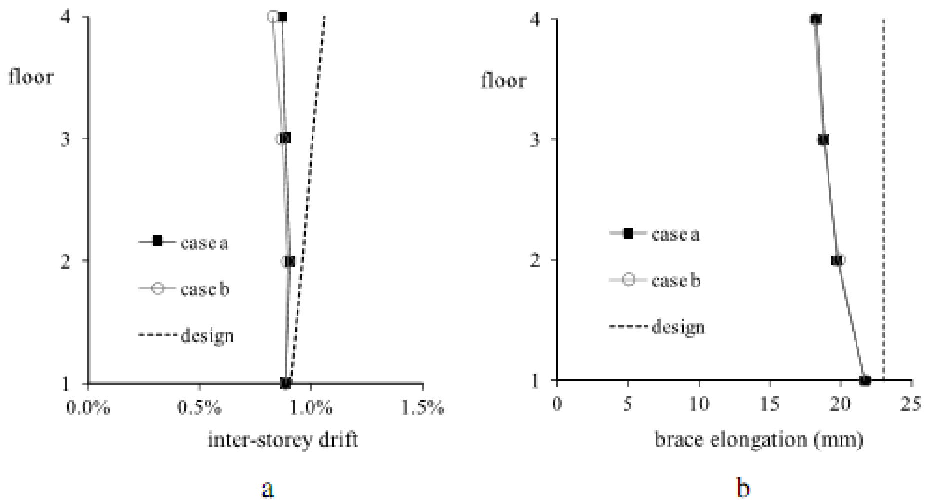

Figure 2.13 Results of viscoelastic braces: a) inter-storey drifts and b) brace elongations for the 4-storey frames (Ragni et al., 2011)

CHAPTER 3

3. CASE STUDY

3.1. Description of Building

In this study, the effectiveness of viscoelastic dampers (VED) in seismic protection of the structures was investigated. For this purpose, the 5 and 12 storey steel frame systems having the same storey height of 3.8 m with 8 m bay spacing were designed as moment resisting frame with and without viscoelastic damper systems. Conventional frame structure is denoted as reference frame, RF while the frame designed with viscoelastic dampers is called as viscoelastically damped frame, VEDF. Figure 3.1 shows the plan views of the model structures while Figures 3.2 and 3.3 demonstrate the elevation views of the 5 and 12 storey model structures of RF and VEDF, respectively.

In all frame systems, the elastic modulus and yield stress of the steel were used as 210 GPa and 240 MPa, respectively. The section of the frame's elements was taken as a box section and the details of the cross-sectional area are given in Table 3.1. As seen in Figures 3.2 and 3.3, two viscoelastic dampers at each floor level were inserted on outer span of the frames. Moreover, the viscoelastic dampers having the same stiffness were utilized at each floor level, and the required stiffness and damping coefficient were evaluated considering that the modules of storage and loss were the same for viscoelastic dampers.

During modelling and analyzing of the frame systems, the finite element software DRAIN-2DX (Prakash et al., 1993) has been used. Dynamic analysis of the model structures indicated that the first natural periods of 5 and 12 storey reference frames were found as 0.67 and 1.25 s, respectively. Those for the frames with viscoelastic damper were noted as 0.83 and 1.76 s.

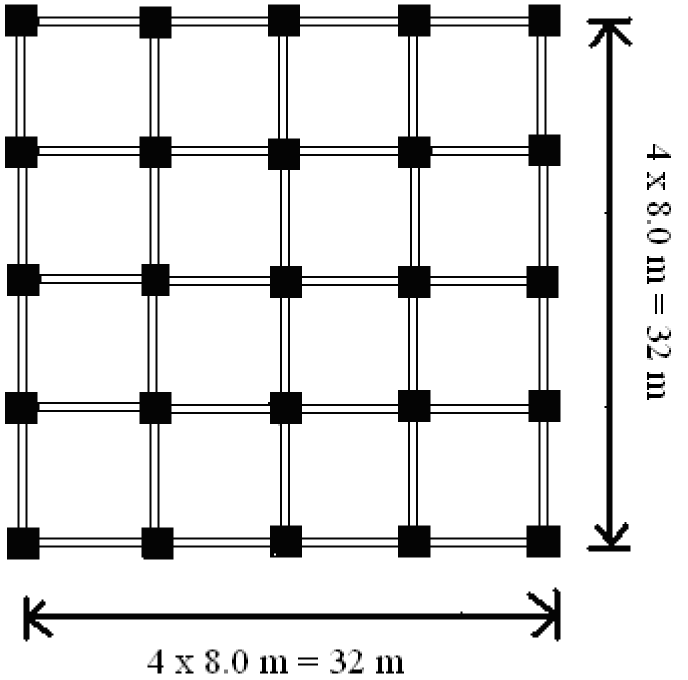

Figure 3.1 Plan views of five and twelve storey steel building frames

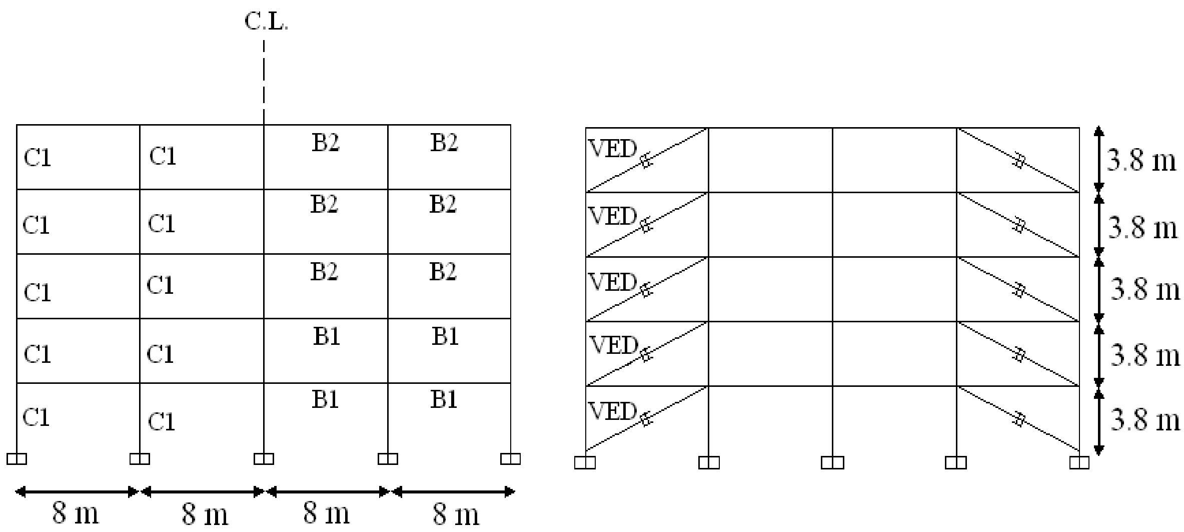

Figure 3.2 Five-storey steel building frames equipped with and without viscoelastic dampers

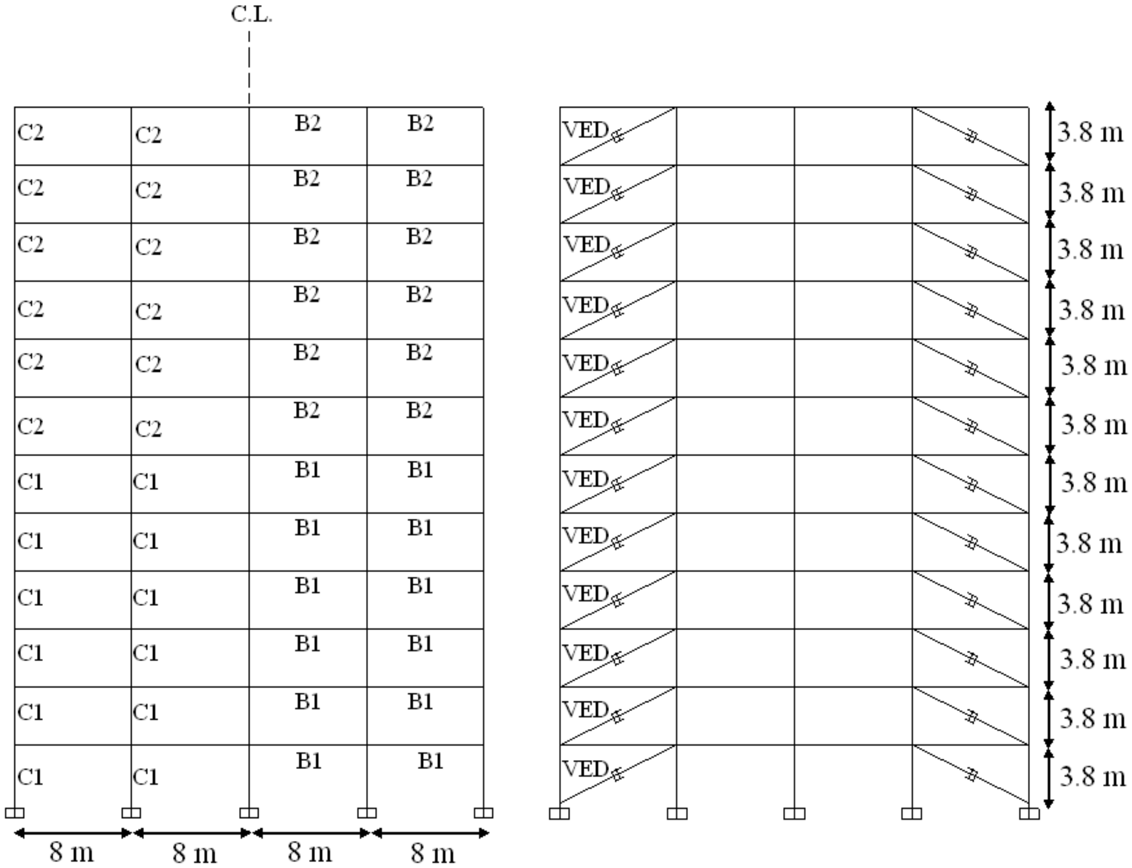

Figure 3.3 Twelve-storey steel building frames equipped with and without viscoelastic dampers

Table 3.1 Section properties of the frame systems

Frame systems	C1 Box section (mm)	C2 Box section (mm)	B1 Box section (mm)	B2 Box section (mm)	VED Stiffness (N/mm)
5 storey-RF	590x590x25	-	530x265x25	440x220x25	-
5 storey-VEDF	510x510x20	-	460x230x20	390x185x20	7610
12 storey-RF	670x670x25	590x590x25	640x320x25	570x285x25	-
12 storey-VEDF	510x510x25	440x440x25	480x240x25	480x240x25	9214

3.2. Design of Steel Frames with and without Viscoelastic Dampers

In the design of the 5 and 12 storey steel frames with and without viscoelastic dampers, direct displacement-based design procedure proposed by Lin et al. (2003) was utilized. According to this displacement-based seismic design procedure, an inelastic system is modeled as an equivalent linear system which is composed of an equivalent stiffness and an equivalent hysteretic damping ratio. Thus, a substitute structure which has the same ultimate force and displacement characteristics with the inelastic structure can be formed. Since the equivalent properties of the substitute structure are elastic, a set of elastic displacement response spectrum can be utilized for the design. Therefore, the substitute structure approach allows the inelastic system to be designed by an elastic displacement response spectrum (Lin et al., 2003).

The details of the procedure of the displacement-based design method used in this study is given hereafter and in order to further clarify the iteration procedure of the method, the flowchart is given in Figure 3.4.

In the first step of this direct displacement-based design method, the target roof displacement (Δ_u, ultimate displacement) was determined. The moment resisting frame without viscoelastic damper system (Reference frame denoted as RF) and with viscoelastic damper system (Viscoelastic damped frame designated as VEDF) were designed for the same target roof displacement which was taken as 1% of the building height. Then, a yield roof displacement (Δ_y) for the designed buildings was assumed. The initial ductility (μ) was calculated from the ratio of the target roof displacement to yield displacement. At the first iteration cycle, the yield displacement was assumed arbitrarily. At the end of the iterations, it would converge to a fixed value, no matter what value of the yield displacement was used at the first iteration cycle.

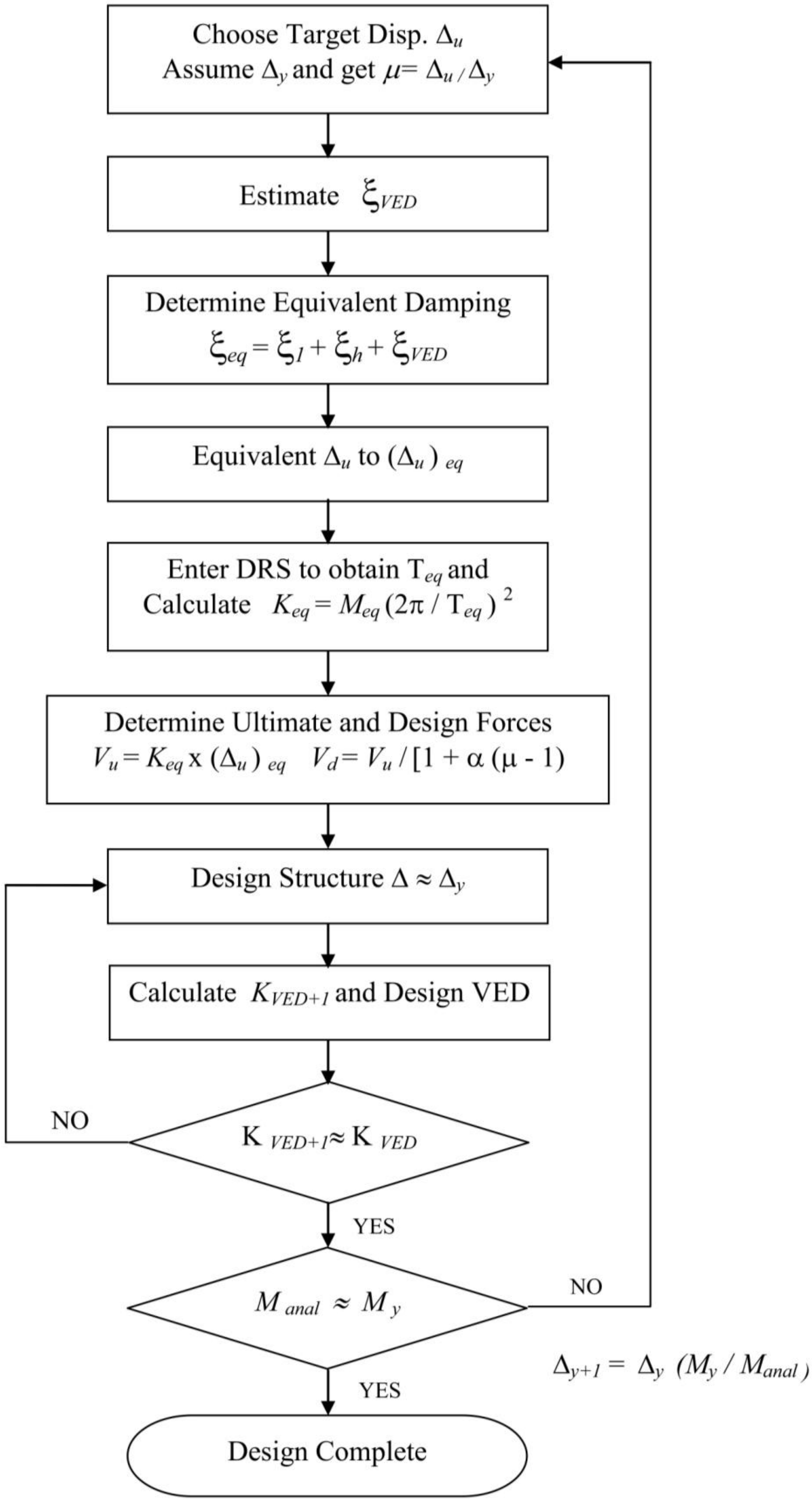

Figure 3.4 Displacement-based design flowchart for building with VED (Lin et al., 2003)

In the next step, the additional effective viscous damping ratio coming from viscoelastic damper systems must be assumed. For all frames with viscoelastic dampers, the effective damping ratio provided by viscoelastic dampers was assumed to be 10%. Then, total equivalent damping ratio (ξ_{eq}) was calculated as sum of the inherent damping ratio (ξ_i), hysteretic damping ratio (ξ_h), and effective viscous damping ratio provided by viscoelastic damper systems (ξ_{VED}). Thus, the total equivalent damping ratio is expressed as:

$$\xi_{eq} = \xi_i + \xi_h + \xi_{VED} \tag{3.1}$$

The inherent damping ratio (ξ_i) was assumed to be 2% for the steel frames designed (Chopra, 1995). According to the studies of Jennings (1968), Iwan and Gates (1979), Lin et al. (2003), the equivalent hysteretic damping ratio may be derived based on the dissipated energy by the inelastic hysteretic deformations. Thus, the hysteretic damping (ξ_h) in the structures was determined according to the equation given below which is based on the Takeda hysteretic model with a bilinear stiffness ratio (α) and a ductility ratio (μ) (Kowalsky et al., 1994; Lin et al., 2003). Hence, the hysteretic damping ratio is stated as:

$$\xi_h = \frac{1}{\pi}\left[1 - \left(\frac{1-\alpha}{\mu}\right) + \alpha\right] \tag{3.2}$$

Following this step, the target roof displacement and the mass of the multi degree of freedom (MDOF) frames were converted to the equivalent target displacement and the equivalent mass of a single degree of freedom (SDOF) substitute structure. In order to use the elastic displacement response spectrum, the target roof displacement and the mass of the MDOF structure with dampers must be equivalent to a SDOF system (Lin et al., 2003). Considering only the fundamental mode and assuming a uniform storey height with a uniform mass distribution and a triangular displacement shape, the equivalent target displacement for the equivalent SDOF system was calculated as given in Eqn. (3.3) (Miranda, 1999; Lin et al., 2003).

$$\left(\Delta_u\right)_{eq} = \Delta_u * \frac{2N+1}{3N} \tag{3.3}$$

Similarly, by considering the mass participation in the fundamental mode, the equivalent mass for the equivalent SDOF system was calculated in accordance with the Eqn. (3.4) (Tsai and Chang, 1999; Lin et al., 2003).

$$M_{eq} = \left(\sum_{i=1}^{N} m_i h_i\right) / h_N \tag{3.4}$$

Here, N is the number of storey in the building, m_i is the mass of i^{th} storey and h_i is the height from the base to the i^{th} storey.

The next step of the procedure involves the determination of the equivalent period (T_{eq}) and equivalent stiffness (K_{eq}) of the SDOF substitute structure. The equivalent period of the SDOF system were found out from the elastic displacement spectrum by using the values of equivalent target displacement $\left(\Delta_u\right)_{eq}$ and total equivalent damping ratio ε_{eq}. The total equivalent stiffness at maximum response displacement was obtained by the equation given below (Lin et al., 2003).

$$K_{eq} = \left(\frac{2\pi}{T_{eq}}\right)^2 \tag{3.5}$$

Since the SDOF substitute structure is elastic, based on a bilinear force-displacement model design force (i.e., yield force, V_y) and the ultimate force capacity (V_u) were evaluated in accordance with the following equations of (3.6) and (3.7), respectively (Lin et al., 2003).

$$V_u = K_{eq} * \left(\Delta_u\right)_{eq} \tag{3.6}$$

$$V_d = V_y = \frac{V_u}{1+\alpha\left(\mu-1\right)} \tag{3.7}$$

According to this design approach, since the design force (V_d or V_y) still falls in the elastic range, the structure can be designed and analyzed in an identical way to the conventional approach (Lin et al., 2003). Therefore, the frames were designed based on V_d and Δ_y. The design forces were distributed over the height of the building according to Eqn. (3.8), in which w_x is the mass of x^{th} storey. The structural members were proportioned, such that the building produces a roof displacement of Δ_y when subjected to the laterally distributed force found by using Eqn. (3.8).

$$F_x = V_d \frac{w_x h_x}{\sum_{i=1}^{N} w_i h_i} \tag{3.8}$$

For energy dissipation devices, it is convenient to obtain the effective viscous damping ratio (ξ_{VED}) provided by energy dissipation device by equating the energy dissipated per cycle of a periodic excitation to the corresponding value of linear viscous damping (Chopra, 1995) as given in equation below:

$$\xi_{VED} = \frac{c}{c_{cr}} = \frac{c}{2m\omega} = \frac{W}{2\pi m\omega\varpi u_o^2} = \frac{1}{4\pi\varpi/\omega}\frac{W}{W_s} \tag{3.9}$$

In the Eqn. (3.9), W equals to the energy dissipated by the added linear damping device ($W = \pi c\varpi u_o^2$) that undergoes a cycle of periodic motion ($u = u_o \sin\varpi t$) of amplitude u_o and frequency ϖ. The parameters c_{cr}, m, ω, and W_s show the critical damping coefficient, mass, natural circular frequency, and the maximum strain energy of the system, respectively. For structures subjected to earthquake loading, $\varpi/\omega = 1$, then the equation becomes as given below:

$$\xi_{VED} = \frac{W}{4\pi W_s} \tag{3.10}$$

The effective viscous damping ratio (ξ_{VED}) provided by viscoelastic dampers and the effective stiffness of the viscoelastic energy dissipation devices (K_{VED}) can be derived as (Lin, 2000):

$$\xi_{VED} = \frac{1}{4\pi}\frac{W_{viscoelastic}}{W_s} = \frac{1}{4\pi}\frac{\sum_j \pi\eta k'_j u_{o,j}^2}{\sum_i \frac{1}{2}F_i u_i} = \frac{1}{2}\frac{\sum_j \eta k'_j u_{o,j}^2}{\sum_i F_i u_i} \tag{3.11}$$

$$K_{VED,j} = k'_j = \frac{nG'_j A_j}{t_j} \tag{3.12}$$

where η , k'_j , n, G'_j , A_j and t_j are correspondingly the loss factor, storage stiffness, number of viscoelastic slab, storage modulus, cross-section area and thickness of the viscoelastic devices j.

In practice, the dynamic behavior of viscoelastic dampers is generally represented by a spring and a dashpot connected in parallel (Valles et al. 1996; Soong and Dargush, 1997; Kim and Choi, 2006). For the linear spring-dashpot representation of the viscoelastic damper, the stiffness K_d and the damping coefficient C_d are obtained as follows:

$$K_d = \frac{G'(w)A}{t} \tag{3.13}$$

$$C_d = \frac{G''\left(w\right)A}{wt} \tag{3.14}$$

where $G'\left(w\right)$ and $G''(w)$ are shear storage and shear loss modulus, respectively; A and t are total shear area and the thickness of the material, respectively; and w forcing frequency, for which the fundamental natural frequency of the structure is generally used in time domain analysis. The ratio of the shear storage modulus to shear loss modulus is called as loss factor.

By using the Eqn. (3.11) and Eqn. (3.12), the effective stiffness of viscoelastic damper systems were determined. The loss factor was taken as 1.0. Then, viscoelastic dampers in the structures were modeled analytically, and from the step of design based on V_d and Δ_y, all procedures were repeated until finding convergence in the value of stiffness of viscoelastic dampers.

End moment against the yield moment of each member was checked. Although the damped structure will deflect Δ_y at the roof as assumed under the laterally distributed force, it may not be a real yield point of the damped structures (Lin et al., 2003). Thus, in order to make sure that the roof displacement obtained was really a yield point, the end moment of each member must be checked against its yield moment capacity. If it was not the case, an iteration procedure according to Eqn. (3.15) was imposed. Starting from the assumption of the yield displacement the all the steps of the procedure were repeated.

$$\Delta_{y+1} = \Delta_y \frac{M_y}{M} \tag{3.15}$$

3.3. Natural Earthquake Records

To investigate the seismic response of the reference frames and the frames with viscoelastic dampers under the earthquake excitations, the frame systems were analyzed through nonlinear time history analysis by using natural ground motions. In the dynamic analyses, 1991 Alkion, 1992 Erzincan, and 1999 İzmit earthquakes were utilized as a ground motion. These natural ground motion records were obtained from European strong-ground motion database (Ambreseys et al. 2004a; Ambreseys et al. 2004b). From the database, earthquake ground motions recorded at a significant distance from the fault (D > 10 km) and recorded on firm soils (shear wave velocities > 180 m/s) were selected. The characteristics of the natural earthquake records used are summarized in Table 3.2. The earthquakes were listed by their location, magnitude (M_w), peak ground acceleration (PGA), and peak ground velocity (PGV), etc. Additionally in the selection of the earthquake ground motions, minimum limits were imposed on the earthquake magnitude (M_w > 6.5) and on the peak ground velocity (PGV > 15 cm/s) and acceleration (PGA > 2 m/s^2). In the nonlinear dynamic analysis of each frame, the records of natural ground motions were scaled considering first mode elastic spectral acceleration, $S_a(T_1)$ as being compatible with the elastic design acceleration spectrum of 10% probability of exceedence in a 50-year period.

Table 3.2 Properties of selected natural earthquake records

Eartquake location	Record station	Soil type	M_w	Date	D (km)	PGA (m/s^2)	PGV (cm/s)
İzmit/Turkey	Yarımca-Petkim	C	7.6	17/08/1999	20	3.05	58.50
Erzincan /Turkey	Erzincan-Headquarters of Meteorology	B	6.7	13/03/1992	13	5.31	84.57
Alkion/ Greece	Korinthos-OTE Building	C	6.6	24/02/1981	20	3.07	22.61

CHAPTER 4

4. DISCUSSION OF THE RESULTS

In this study, the nonlinear time history analysis was carried out using the finite element software DRAIN-2DX for both conventional and viscoelastically damped structures. As a case study, 5 and 12 storey steel moment resisting frame structures were selected and analyzed by using natural ground motion records. In the dynamic analyses, three different natural ground motion records, namely 1991 Alkion, 1992 Erzincan, and 1999 İzmit earthquakes were utilized. As a result of these analyses, inter-storey drift ratios, maximum plastic rotation of beams and columns, and energy time history plots obtained for each moment resisting frame without viscoelastic damper system (Reference frame as RF) and with viscoelastic damper system (viscoelastic damped frame as VEDF) were evaluated and discussed below.

Figures 4.1, 4.2, and 4.3 show the results of the roof displacement time history of the 5 storey frames equipped with and without viscoelastic dampers under three different earthquake ground motions of İzmit, Erzincan, and Alkion, respectively. In the same way, the outcomes of the roof displacement time history of the 12 storey frames are presented in Figures 4.4, 4.5, and 4.6.

The analysis of the results indicated that the frames with viscoelastic dampers gave lower displacement values than the reference ones for all earthquake motions. As observed in Figures 4.1 to 4.3, Erzincan earthquake acceleration induced a severe deformation than the other earthquake accelerations for both the 5 storey RF and VEDF. However, the magnitude of the displacement for the viscoelastically damped frames within the Erzincan earthquake duration was much smaller than that for the reference frames. The similar trend was also observed for the İzmit and Alkion earthquakes.

Moreover, for the RF, the maximum roof displacements were 0.22, 0.15, and 0.08 m under Erzincan, İzmit, and Alkion earthquakes, respectively while those were 0.11, 0.08, and 0.06 m for the VEDF.

As seen from Figures 4.4 to 4.6, generally, the seismic response of the 12 storey frames had similar trends in comparison to that of the 5 storey frames. The frames with added viscoelastic dampers had considerably lower roof displacement that the reference frames for all earthquakes. İzmit earthquake acceleration caused a greater roof displacement than the other earthquake accelerations (followed Erzincan and then Alkion earthquakes) for the 12 storey VEDF. However, for the 12 storey reference frame, the effects of the earthquake ground motions on displacement time history were altered. Results indicated that for the VEDF, the maximum roof displacements were 0.22, 0.19, and 0.13 m under Erzincan, İzmit, and Alkion earthquakes, respectively while those were 0.22, 0.26, and 0.24 m for the RF.

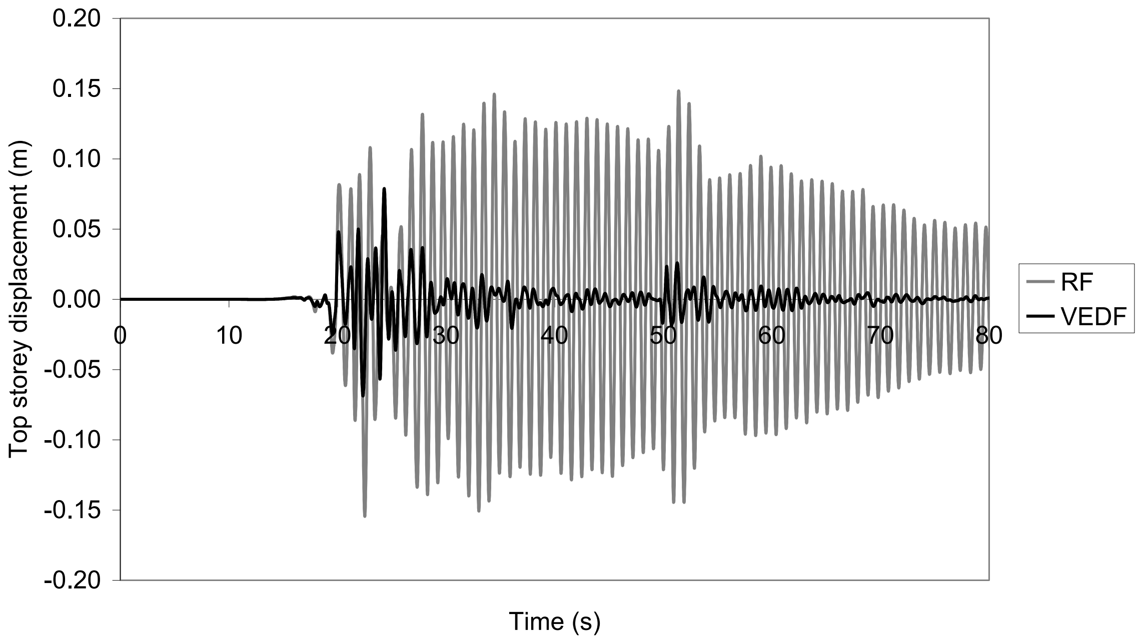

Figure 4.1 Displacement time history of the roof of the 5 storey frames under İzmit earthquake

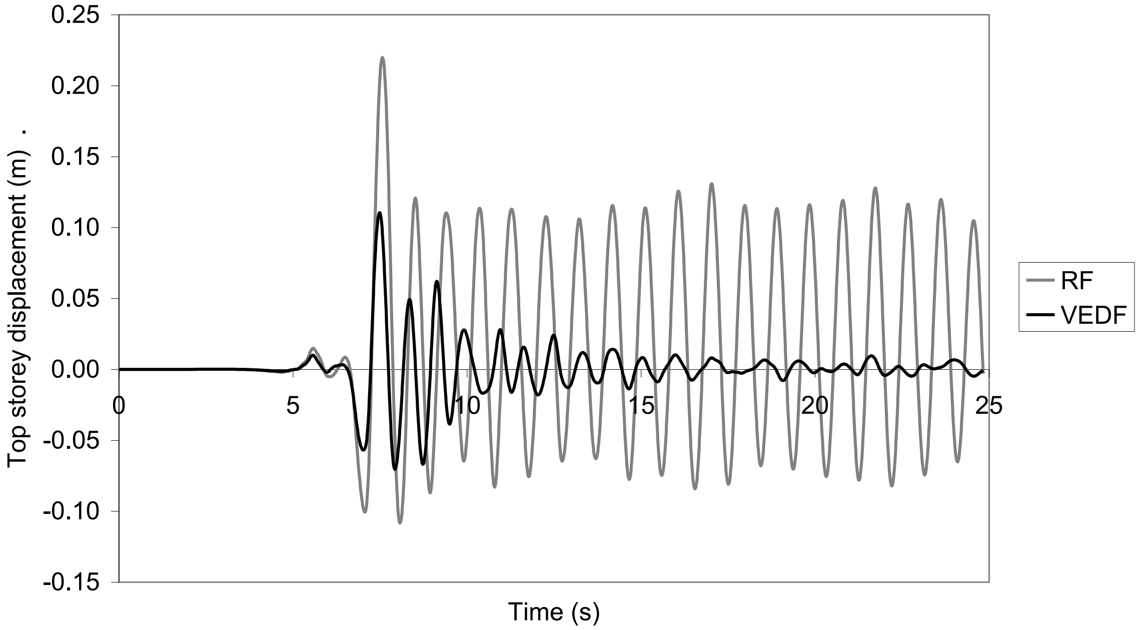

Figure 4.2. Displacement time history of the roof of the 5 storey frames under Erzincan earthquake

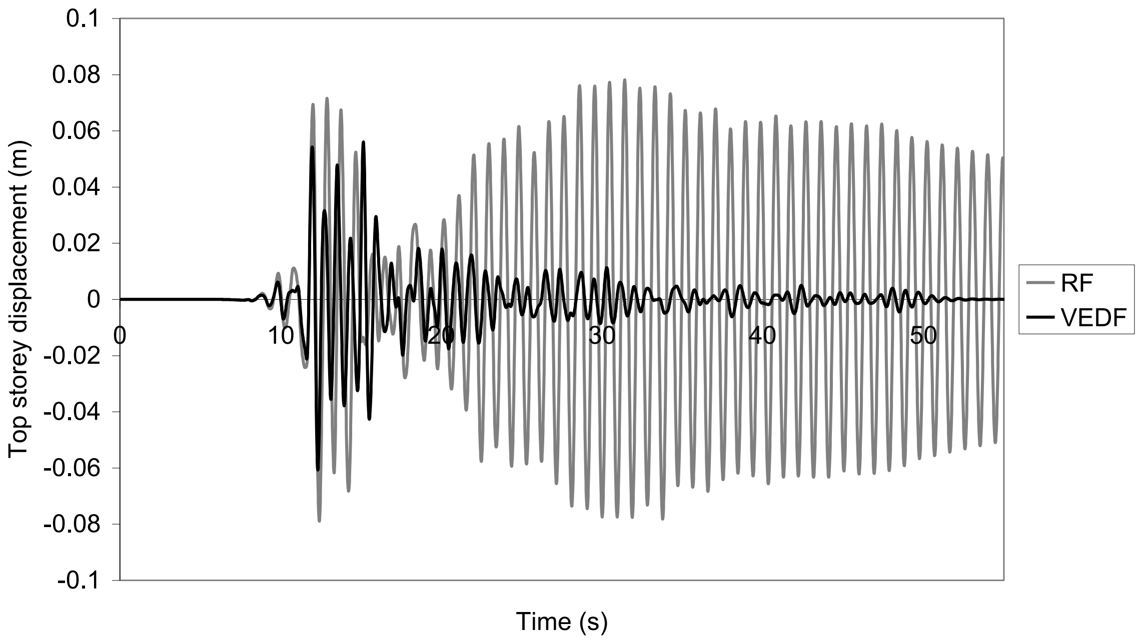

Figure 4.3 Displacement time history of the roof of the 5 storey frames under Alkion earthquake

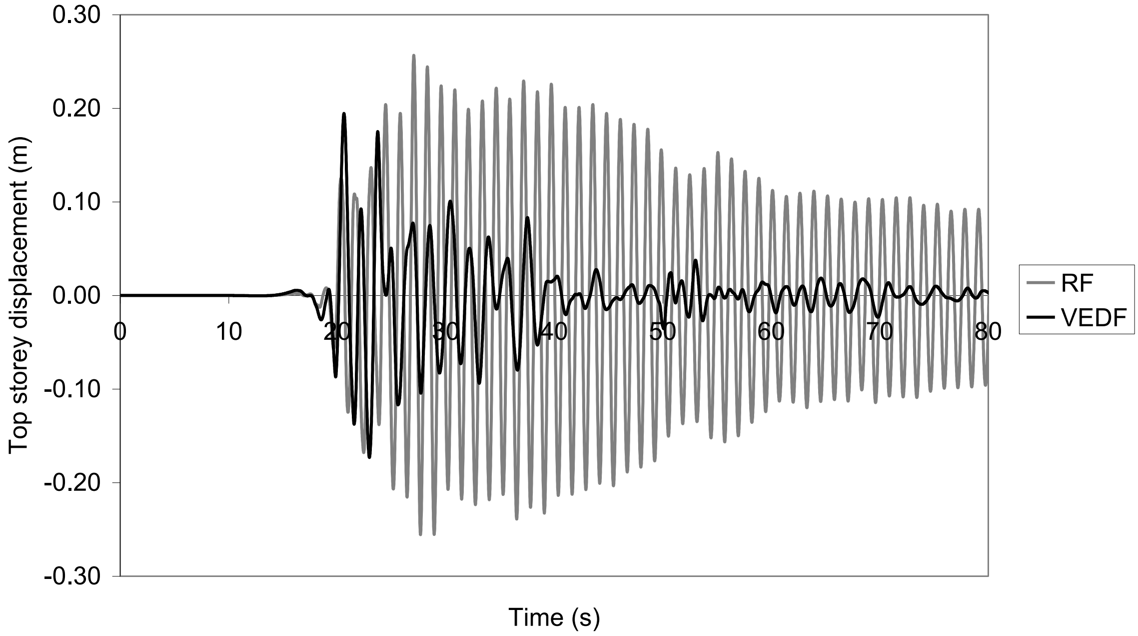

Figure 4.4 Displacement time history of the roof of the 12 storey frames under İzmit earthquake

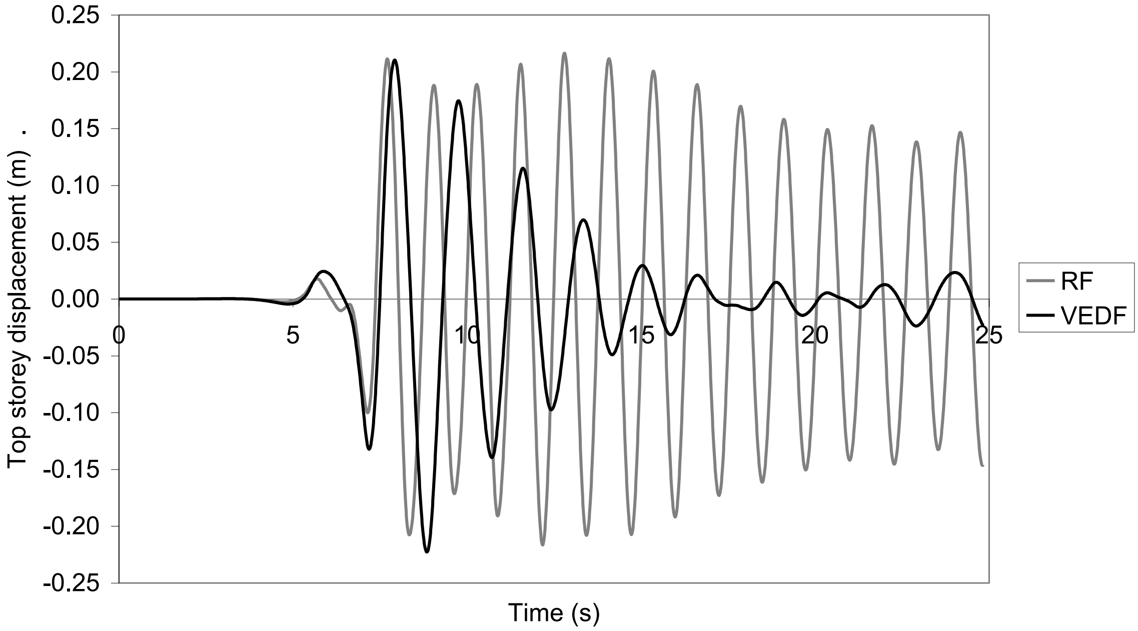

Figure 4.5 Displacement time history of the roof of the 12 storey frames under Erzincan earthquake

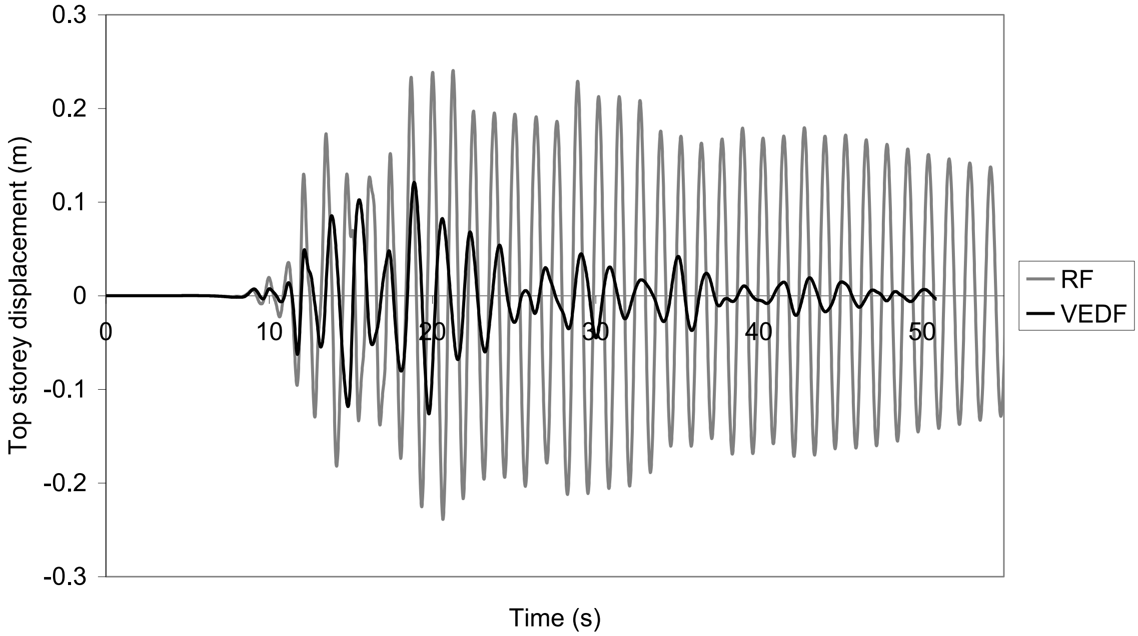

Figure 4.6 Displacement time history of the roof of the 12 storey frames under Alkion earthquake

Variations in the maximum interstorey drift ratios for 5 and 12 storey model structures with and without viscoelastic dampers under earthquake excitation are presented in Figures 4.7 and 4.8, respectively. It was observed from these plots that one of the main goals of using damper in steel building was reducing interstory drift ratio to improve integrity and serviceability of structure during vibration excitation. This aspect strongly affects the design of the structure. Moreover, the use of viscoelastic dampers had a tendency to distribute the drifts more uniformly along the height of the frames, especially for 5 storey model structures. It was found that the ratio of the maximum interstorey drifts in 5 storey frames equipped with viscoelastic dampers to those in the 5 storey reference frames ranged from 0.42 to 0.60, depending mainly upon ground motion records to be applied. Similarly, this ratio varied between 0.49 and 0.94 for the 12 storey model structures. As a result, viscoelastically damped model structures exhibited significantly lower displacements.

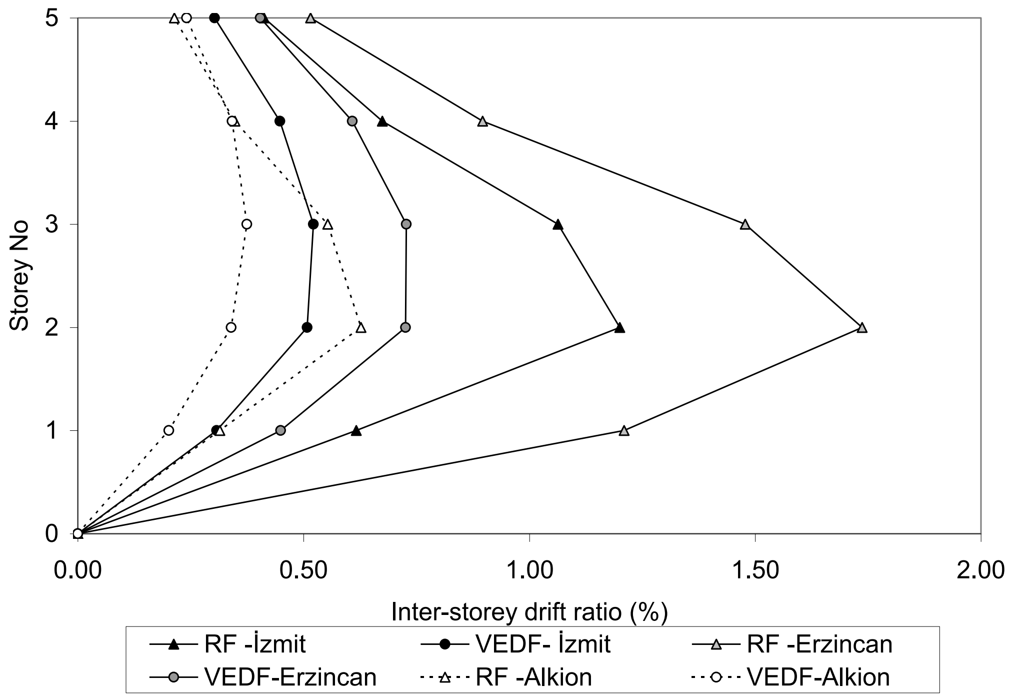

Figure 4.7 Maximum interstorey drift ratios of 5 storey frames with and without viscoelastic dampers

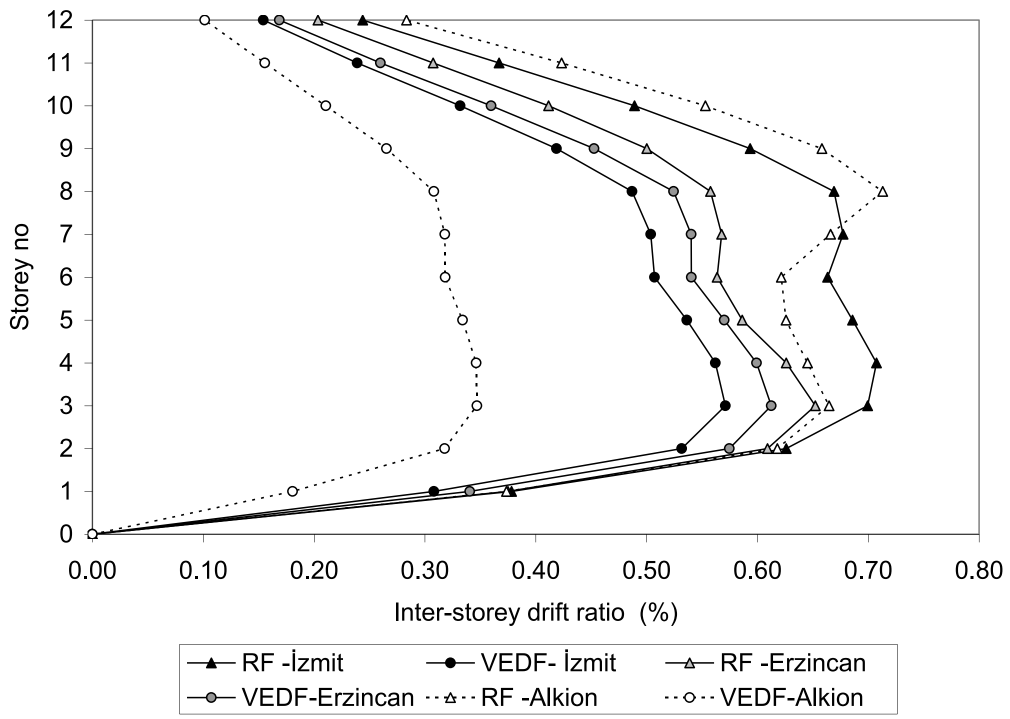

Figure 4.8 Maximum interstorey drift ratios of 12 storey frames with and without viscoelastic dampers

The variations of the maximum plastic rotations and number of the plastic hinges computed for beams and columns of the 5 and 12 storey RF and VEDF under the earthquake excitation are given in Table 4.1. As seen in Table 4.1, generally, for both 5 and 12 storey RF, the values of maximum plastic rotations were found in column and beam elements when the frame was hit by Erzincan earthquake. Adding viscoelastic dampers into the moment resisting frames was found generally effective in reducing the maximum plastic rotations in the column and beam elements. Similarly, it was observed that the number of the plastic hinges was significantly decreased for the viscoelastically damped frames under all ground motion records. FEMA 355E (2000) also recommends that the maximum plastic rotations experienced by the moment resisting frame should not be exceed the target plastic rotation of 3.0%. As seen in Table 4.1, all values were less than the limit of 3.0%. This implied that both the column and beam elements of RF and VEDF had satisfactory resistance to the seismic forces.

Tablo 4.1 Maximum plastic rotations and number of plastic hinges occurred in the columns and beams of the frame structures

Types of frame structure		İzmit earthquake		Erzincan earthquake		Alkion earthquake	
		Maximum plastic rotations (%)	Number of plastic hinges	Maximum plastic rotations (%)	Number of plastic hinges	Maximum plastic rotations (%)	Number of plastic hinges
5 storey frames	RF-column	0.051	5	0.656	5	0.000	-
	VEDF-column	0.000	-	0.019	1	0.000	-
	RF-beam	0.403	8	0.712	9	0.000	-
	VEDF-beam	0.000	-	0.089	5	0.000	-
12 storey frames	RF-column	0.077	5	0.075	5	0.081	5
	VEDF-column	0.029	1	0.062	4	0.000	-
	RF-beam	0.193	30	0.107	12	0.153	27
	VEDF-beam	0.000	-	0.000	-	0.000	-

The input energy imparted to a structure by an earthquake is mainly dissipated by yielding of the structural members when inelastic behavior develops and by damping energy associated with the conventional members as well as any passive energy dissipation systems if available. The energy terms can be defined by integrating the equation of motion of a structure as given below (Chopra, 1995).

$$\int_0^x m\ddot{x}(t)\, dx + \int_0^x c^* \dot{x}\, dx + \int_0^x f_s^*(x,\dot{x})\, dx = -\int_0^x m\ddot{x}_g(t)\, dx \tag{4.1}$$

Here, $f_s^*(x,\dot{x})$ are the resisting forces in the yielding members of the system, m =mass matrix, c*= damping matrix for the structure-control system which includes the effect of inherent structural damping and that of the passive energy dissipation systems; $\ddot{x}_g(t)$ is the horizontal ground acceleration, and x, $\dot{x}$ and $\ddot{x}$ are the relative displacement, velocity and acceleration response of the dynamic degrees of freedom. Eqn (4.1) can be described in terms of the energy of the system as:

$$E_K(t) + E_C(t) + E_Y(t) + E_S(t) = E_I(t) \tag{4.2}$$

where $E_K(t)$, $E_C(t)$, $E_Y(t)$, $E_S(t)$ and $E_I(t)$ are the kinetic energy associated with the motion of the structure relative to the ground, the damping energy, the yield strain energy, the elastic strain (recoverable strain) energy and the input energy (Tzan and Pantelides, 1997).

The energy time-history plots showing the variation of the kinetic energy, strain energy (sum of the elastic strain energy and yield strain energy) and damping energy, for the 5 storey model structures of RF and VEDF subjected to the İzmit, Erzincan, and Alkion earthquakes are presented in Figures 4.9 to 4.14 while those for the 12 storey model structures of RF and VEDF are shown in Figures 4.15 to 4.20. For both 5 and 12 storey VEDFs, it was observed that the energy significantly reduced due to the installation of viscoelastic dampers into the frame structures.

As seen on the figures, the input energy is always positive, but it does not always increase with time since the change in the ground displacement could be on the opposite direction to the absolute acceleration. And when this occurs, earthquake behaves as a reactive force to aid the structure balancing itself.

When the model structures of RF and VEDF were compared, as expected, it was observed that the high amount of the energy on building designed with viscoelastic dampers was compensated by the damping energy while that for the conventional frame was achieved by the strain energy. As a result, the structures designed with viscoelastic dampers had higher damping energy, and this significantly affected the elastic and inelastic behavior of the model structures under earthquake excitations. Thus, the damage occurred on structural elements would be decreased.

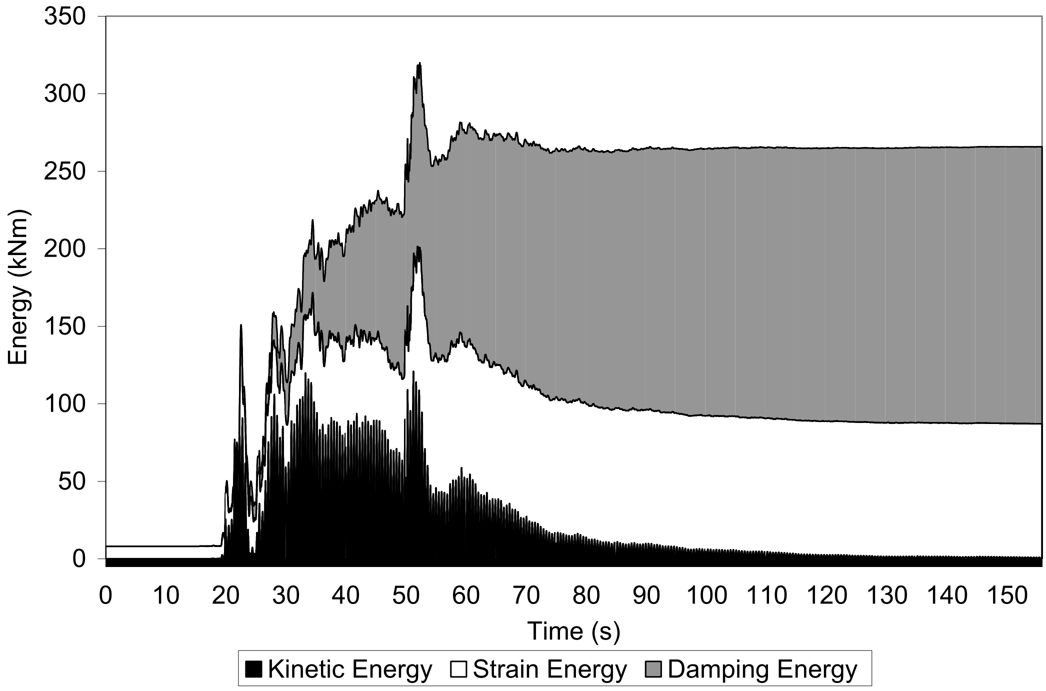

Figure 4.9 Energy time history of 5 storey RF under İzmit earthquake

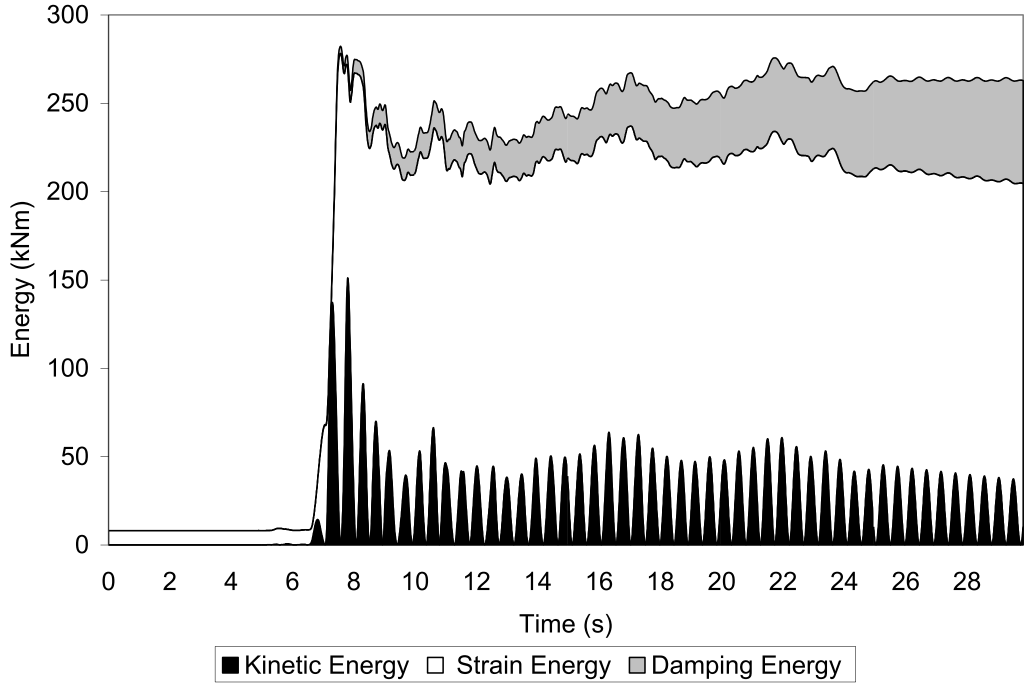

Figure 4.10 Energy time history of 5 storey RF under Erzincan earthquake

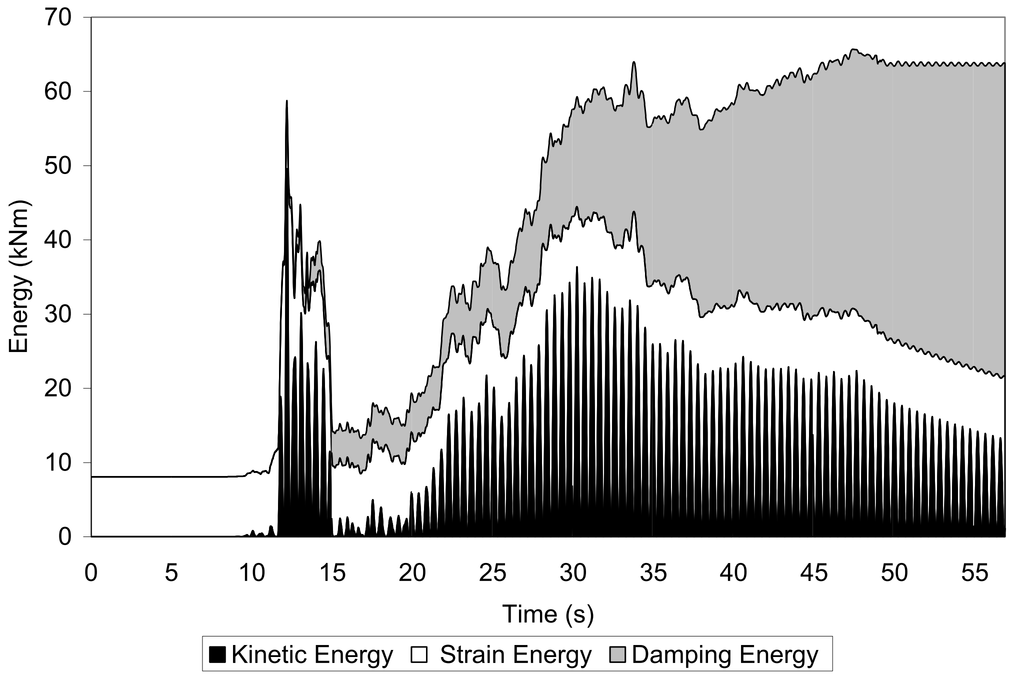

Figure 4.11 Energy time history of 5 storey RF under Alkion earthquake

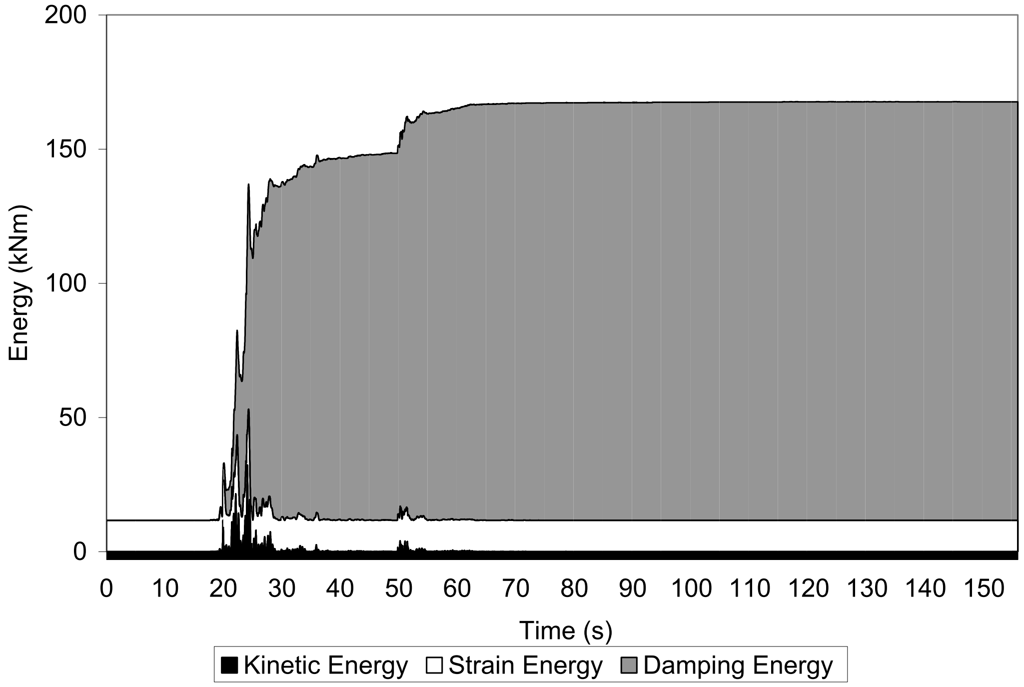

Figure 4.12 Energy time history of 5 storey VEDF under İzmit earthquake

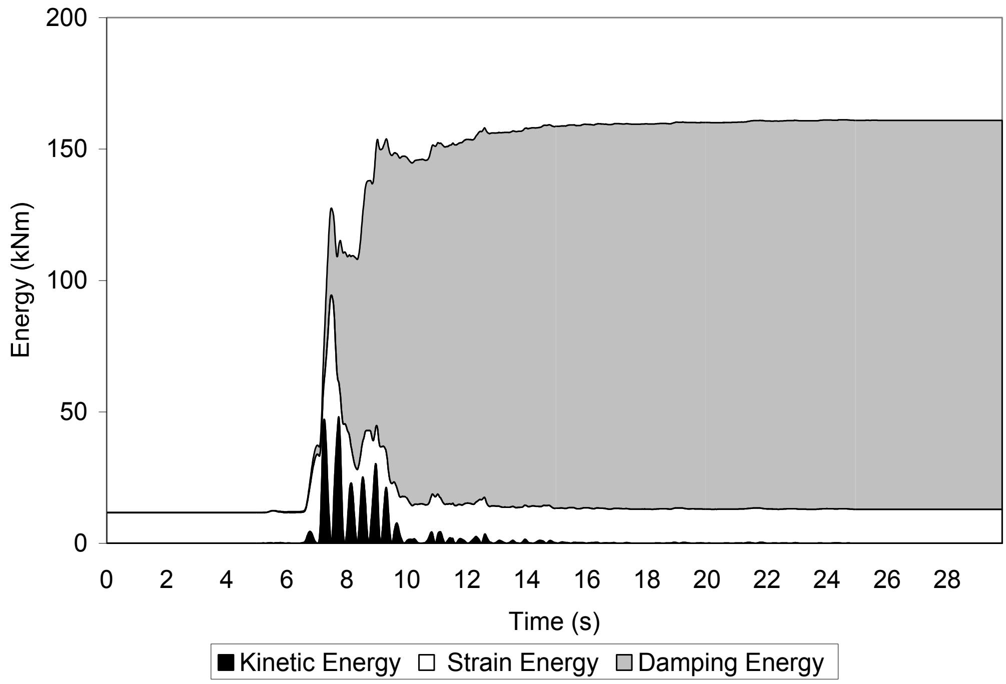

Figure 4.13 Energy time history of 5 storey VEDF under Erzincan earthquake

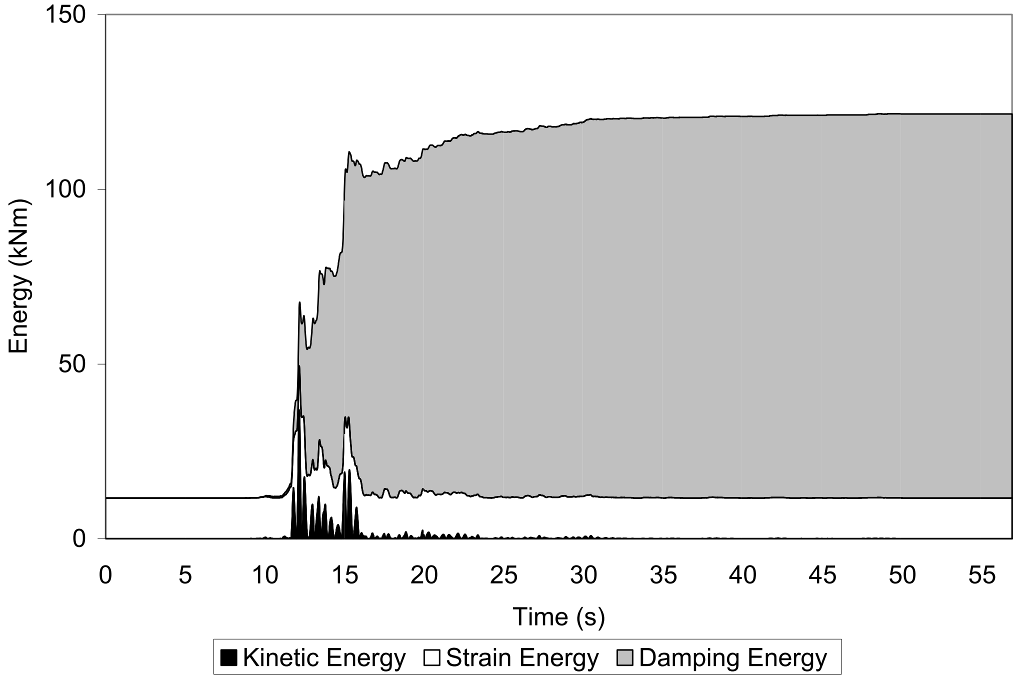

Figure 4.14 Energy time history of 5 storey VEDF under Alkion earthquake

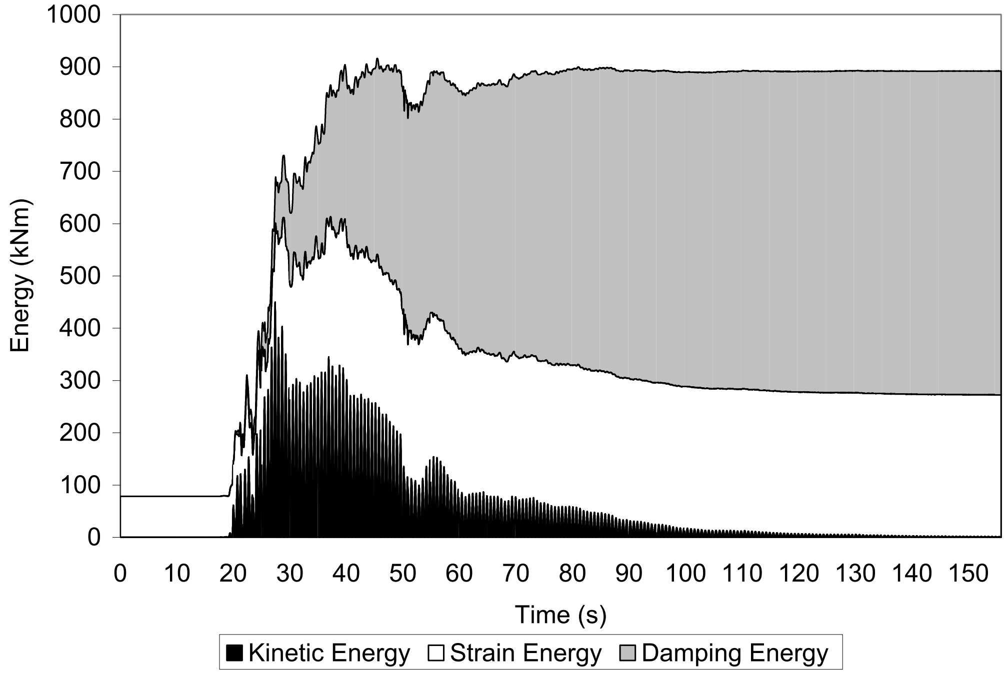

Figure 4.15 Energy time history of 12 storey RF under İzmit earthquake

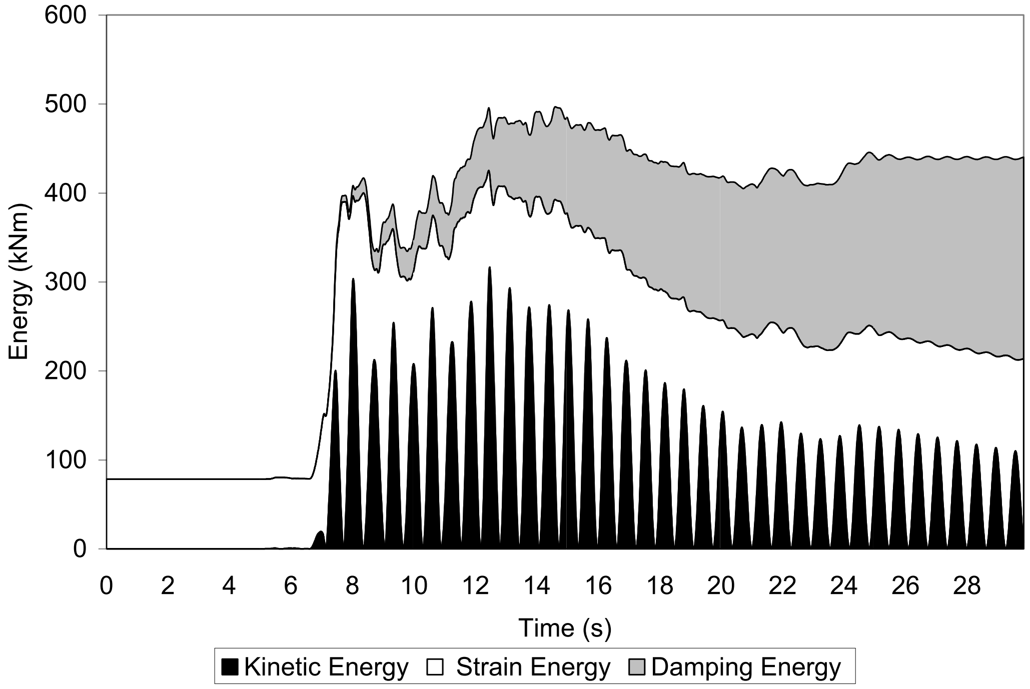

Figure 4.16 Energy time history of 12 storey RF under Erzincan earthquake

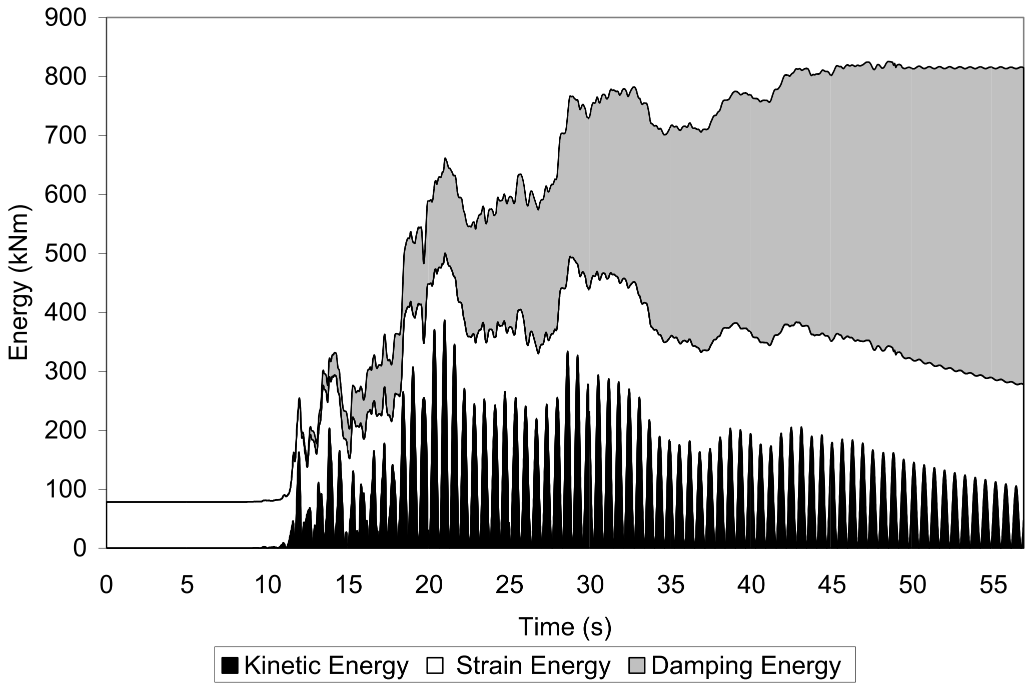

Figure 4.17 Energy time history of 12 storey RF under Alkion earthquake

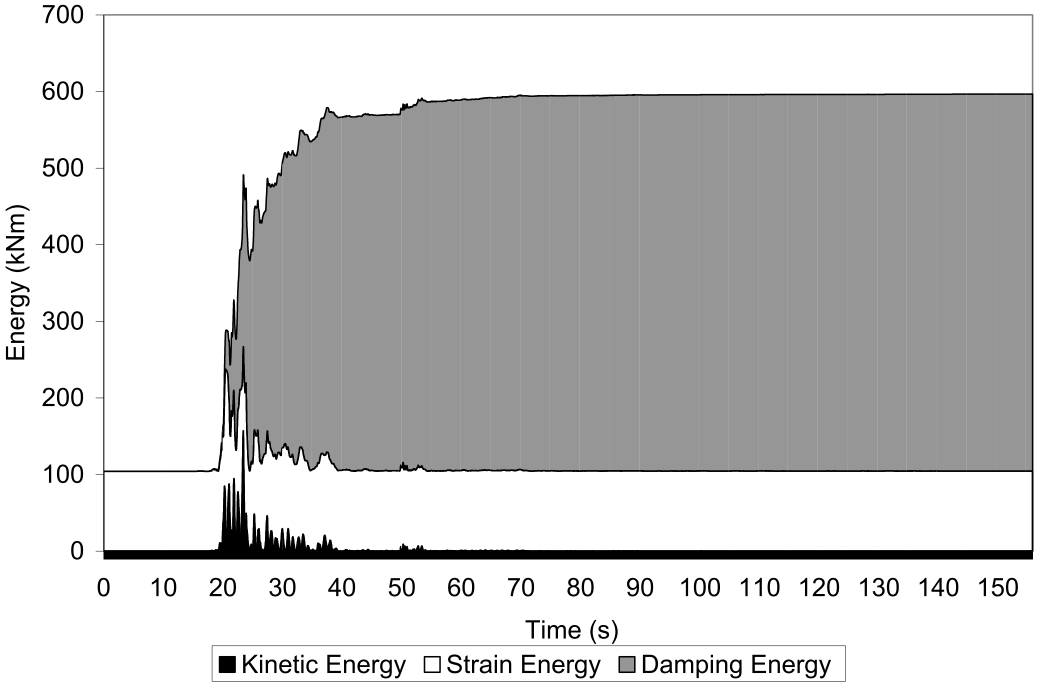

Figure 4.18 Energy time history of 12 storey VEDF under İzmit earthquake

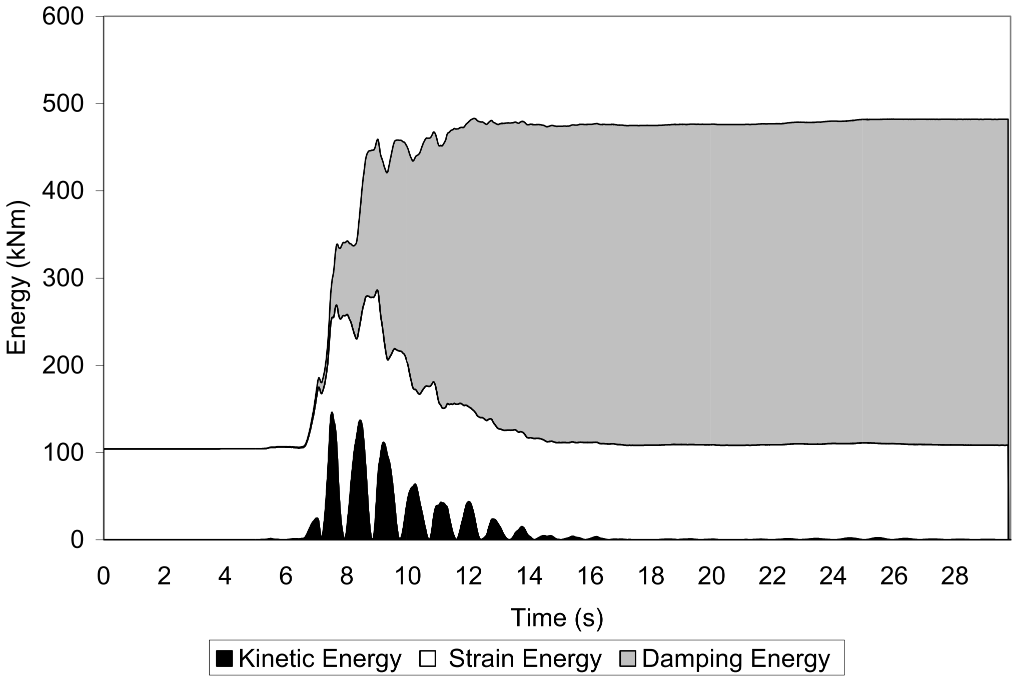

Figure 4.19 Energy time history of 12 storey VEDF under Erzincan earthquake

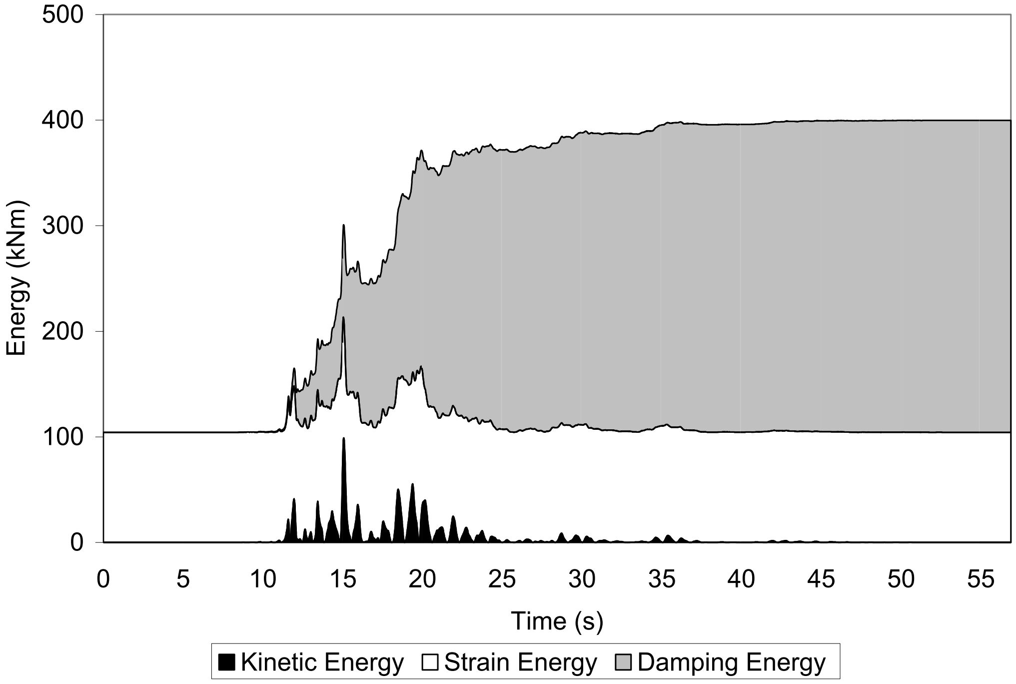

Figure 4.20 Energy time history of 12 storey VEDF under Alkion earthquake

CHAPTER 5

5. CONCLUSIONS

In this thesis, the modelling and structural response of the 5 and 12 storey conventional and viscoelastically damped frames under earthquake excitations were studied. All computer simulation and nonlinear time history analysis were performed by means of a finite element software of DRAIN-2DX. In the dynamic analyses, 1991 Alkion, 1992 Erzincan, and 1999 İzmit earthquakes were utilized as a ground motion record. As a result of these analyses, roof displacement time history, inter-storey drift ratios, maximum plastic rotation of beams and columns, and energy time history plots were examined under different earthquakes for all frame systems. Based on the results obtained in this study and subsequent analysis, the following conclusions could be drawn:

1. Analysis of the results indicated the cross-sectional area of the members used in modelling the structures was reduced when the frame structures were designed with viscoelastic dampers for both 5 and 12 storey buildings. This caused that the viscoelastically damped frames had slightly higher first modes of vibration in comparison to the conventional frames.

2. The nonlinear time history analysis performed on the model structures indicated that each earthquake record exhibited its own peculiarities, dictated by frequency content, duration, sequence of peaks and their amplitude. The dispersion in the results of different ground motions depended on the characteristics of both the structure and the record.

3. Results of the nonlinear time history analysis showed that Erzincan earthquake acceleration induced a severe deformation than the other earthquake accelerations for both the 5 storey frame systems with and without

viscoelastic dampers. However, the viscoelastically damped model structures exhibited significantly lower displacements. For the 12 storey frame systems with visoelastic dampers, the effects of the earthquake ground motions on displacement time-history had similar tendency but that was altered for the 12 storey reference frame.

4. From the results of this study, it was found that the ratio of the maximum interstorey drifts in the 5 storey frames equipped with viscoelastic dampers to those in the 5 storey reference frames were in the range of 0.42 to 0.60, depending mainly upon ground motion records to be applied. Similarly, this ratio varied between 0.49 and 0.94 for the 12 storey model structures. Thus, it was concluded that viscoelastically damped frame has sinificantly lower drift values in comparison to that of the reference frame. This indicates the efficiency of using passive energy dissipation systems for seismic protection of structures.

5. According to the energy time-history results, for both the 5 and 12 storey model structures, it was pointed out that the input energy significantly diminished due to the installation of viscoelastic dampers into the frame systems. On the other hands, the added dampers had the effect of increasing the amount of dissipated seismic energy.

6. This study revealed that viscoelastic dampers increased the stiffness and the damping of structures, and decreased the seismic responses of structures effectively. Moreover, the passive energy dissipation system was found to assist in reducing the peak structural response, plastic rotations and the number of plastic hinges. However, the seismic responses of the structure with and without viscoelastic dampers depended mainly on the design parameters used in the model as well as the characteristics of the earthquakes.

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