Influence of Damping Systems
On Building Structures
Subject to Seismic Effects

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Abstract

In order to control the vibration response of high rise buildings during seismic events, energy absorbing passive damping devices are most commonly used for energy absorption. Today there are a number of types of manufactured dampers available in the market, which use a variety of materials and designs to obtain various levels of stiffness and damping. Some of these include friction, yielding, viscoelastic and viscous dampers. These dampers are usually installed between two load bearing elements (walls or columns) in new buildings. In existing buildings, which require retrofitting, they could be installed in cut-outs of shear walls, as evidenced from recent investigations. An effective damping system can result in higher levels of safety and comfort, and can also lead to considerable savings in the total cost of a building.

This thesis treats seismic mitigation of multistorey buildings using embedded dampers. Three types of damping mechanisms, viz, friction, viscoelastic, and combined friction-viscoelastic were investigated. Finite element methods were employed in the analysis using the program ABAQUS version 6.3. A direct integration dynamic analysis was carried out to obtain the damped and undamped responses of the structure in terms of deflections and accelerations at all storeys in order to evaluate the effectiveness of the damping system in mitigating the seismic response. The damping mechanisms have been modelled as (i) a linear spring and dash-pot in parallel for the viscoelastic damper, (ii) a contact pair with friction parameter for a friction damper and (iii) a hybrid damper consisting of both a viscoelastic and a friction damper. The earthquake events used in this study have been applied as acceleration time-histories at the base of the structure in the horizontal plane. Concrete material properties were chosen to represent the model as many high-rise buildings are constructed by using reinforced concrete.

Several medium and high-rise building structures with embedded dampers in different configurations and placed in various locations throughout the structure were subjected to different earthquake loadings. Influence of damper type and properties, configuration and location were investigated. Results for the reduction in tip deflection and acceleration for a number of cases demonstrate the feasibility of the technique for seismic mitigation of these structures for a range of excitations, even when the dominant seismic frequencies match the natural frequency of the structure. Results also provide information which can be used for optimal damper placement for seismic mitigation.

Keywords

Seismic response; Friction damper; Viscoelastic damper; Damping; Configuration
Publications

**International Refereed Journal Papers:**


**International Refereed Conference Papers:**


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Symbols

\begin{align*}
A & \quad \text{Shear Area} \\
A_g & \quad \text{Tributary Area of the Slab} \\
C & \quad \text{Structural Damping Matrix} \\
C_d & \quad \text{Damping Coefficient of VE Damping Device} \\
C_c & \quad \text{Damping Coefficient of the Corrector} \\
C_M & \quad \text{Damping Coefficient of the Main Spring} \\
C_0 & \quad \text{Zero Frequency Damping Coefficients} \\
D & \quad \text{Control Force-Location Matrix} \\
f & \quad \text{Magnification Factor} \\
f_e & \quad \text{Vector of Forces in the Supplemental Device} \\
f_c & \quad \text{External Excitation Vector} \\
f_e' & \quad \text{Compressive Strength} \\
E_c & \quad \text{Young’s Modulus} \\
F & \quad \text{Overall Force} \\
F_D & \quad \text{Force along the Axis of the Damper} \\
F_M & \quad \text{Force in the Main Spring} \\
F_{vd} & \quad \text{Force in the Device} \\
G & \quad \text{Gravity Load} \\
G' & \quad \text{Viscoelastic Damper Shear Storage Modulus} \\
G'' & \quad \text{Viscoelastic Damper Shear Loss Modulus} \\
h & \quad \text{Participation Matrix which Consists of Vector of Floor Masses} \\
h & \quad \text{Height above the Structural Base of the Structure to Level x} \\
I & \quad \text{Importance Factor} \\
J & \quad \text{Optimal Placement and Design of the Control System} \\
K & \quad \text{Structural Stiffness Matrix} \\
k_d & \quad \text{Axial Stiffness of Damping Device} \\
m & \quad \text{Lumped Mass} \\
M & \quad \text{Total Mass Matrix of the Structure} \\
p & \quad \text{Contact Pressure} \\
Q & \quad \text{Live Load on the Structure} \\
t & \quad \text{Temperature} \\
t & \quad \text{Thickness of Viscoelastic Material} \\
T & \quad \text{Natural Period} \\
u & \quad \text{Interstorey Drift Vector} \\
u_d & \quad \text{Damper Relative Displacement} \\
u_g & \quad \text{Earthquake Ground Displacement} \\
U & \quad \text{Vector of Displacement of Structure} \\
\dot{U} & \quad \text{Vector of Velocity} \\
\ddot{U} & \quad \text{Vector of Acceleration} \\
U_i^d & \quad \text{Real Value of Displacement Response Associate with the Modal Shape} \\
U_i^v & \quad \text{Real Value of Velocity Response Associate with the Modal Shape}
\end{align*}
$V$ Velocity
$W$ Weight
$x$ Displacement of Damper
$X$ Modal Response
$\Delta c_i$ Damping Coefficient of the Damper Installed at the $i$ Floor Deflection
$\gamma$ Shear
$\mu$ Coefficient of Friction
$\rho$ Damper Angle
$\rho$ Density (kg/m$^3$)
$\tau$ Shear Stress
$\tau_{\text{max}}$ Maximum Shear Stress
$\nu$ Poisson’s Ratio
$\omega$ Circular Frequency
$\psi_i$ Displacement Mode Shape
$\psi_i$ Velocity Mode Shape
$\zeta$ Modal Damping Ratio

---

**Abbreviations**

ADAS Added Damping and Stiffness
ASCE American Society of Civil Engineers
EDD Energy Dissipation Device
EDR Energy Dissipating Restraint
EDS Energy Dissipation System
FEA Finite Element Analysis
FEM Finite Element Method
FEMA Federal Emergency Management Administration
FVED Friction-Viscoelastic Damper
LCVA Liquid Column Vibration Absorber
LED Lead Extrusion Damper
MCEER Multidisciplinary Center for Earthquake Engineering Research
MDOF Multi Degree of Freedom
MTMD Multi Tuned Mass Damper
PGA Peak Ground Acceleration
OCT Optimal Control Theory
SDOF Single Degree of Freedom
TDA Transient Dynamic Analysis
TLD Tuned Liquid Damper
TLCD Tuned Liquid Column Damper
TMD Tuned Mass Damper
SMA Shape Memory Alloy
VDS Viscous Damping System
VE Viscoelastic
Influence of Damping Systems on Building Structures Subject to Seismic Effects

Statement of Original Authorship

The work contained in this thesis has not been previously submitted for a degree or diploma at any higher education institution. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made.

Signature:

____________________________________

Date:

____________________________________
I am extremely grateful and deeply indebted to my supervisor Prof. David Thambiratnam for his enthusiastic and expertise guidance, constructive suggestions, encouragements throughout the course of this study and the valuable assistance in many ways. Without such assistance this study would not have been what it is. His immense patience and availability for comments whenever approached, even amidst his heavy pressure of work throughout the entire period of study, deserves grateful appreciation. Adjunct Professor, Nimal Perera, is to be mentioned with thanks for kindly agreeing to serve as associate supervisor.

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Chapter 1

Introduction
Chapter 1: Introduction

1.1 Background to the study

Earthquakes are one of nature’s greatest hazards to life on this planet and have destroyed countless cities and villages on virtually every continent. They are one of man’s most feared natural phenomena due to major earthquakes producing almost instantaneous destruction of buildings and other structures. Additionally, the damage caused by earthquakes is almost entirely associated with man made structures. As in the cases of landslides, earthquakes also cause death by the damage they induce in structures such as buildings, dams, bridges and other works of man. Unfortunately many of earthquakes give very little or no warning before occurring and this is one of the reasons why earthquake engineering is complex.

On average about 200 large magnitude earthquakes occur in each decade (www.iris.edu). About 10-20% of these earthquakes occur mid ocean, and hence cause no problems for human settlements. Others occur in the areas away from towns and cities and so similarly cause few problems. The problem occurs when an earthquake hits highly populated areas. Unfortunately as the population of the earth increases the chances of this happening also increases. At the start of the century, less than one in three large earthquakes killed someone, it has now risen to two in three and this upward tend shows no signs of abating.

Some of the major problems relating to earthquake design are created by the original design concept chosen by the architect. No engineer can truly transform a badly conceived building into an earthquake resistant building. The damages which have occurred during earthquake events clearly demonstrate that the shape of a building is crucial to how they respond. The ideal aspects of a building form are simplicity, regularity and symmetry in both elevation and plan. These properties all contribute to a more predictable and even distribution of forces in a structure while any irregularities are likely to lead to an increased dynamic response, at least in certain locations of the structure. Also buildings, which are tall in comparison to their plan area, will generate high overturning moments while buildings with large plan areas may not act as expected due to differences in
ground behaviour, which are not always predictable. This causes different parts of the building to be shaken differently creating obvious problems. Torsion from ground motion could be of great concern due to eccentricity in the building layout. For instance if the centre of mass (gravity) is not in the same position as the centre of resistance a torsional moment about a vertical axis will be created which will have to be designed for. In order to achieve satisfactory earthquake response of a structure, three methods can be identified as being practical and efficient. These are; isolation, energy absorption at plastic hinges and use of mechanical devices to provide structural control.

The first type, the method of structural isolation is very efficient, but expensive and difficult to carry out (Di Sarno et al., 2005). The principle behind isolation is to change the natural period of the structure, substantially decouple a structure from the ground motion input and therefore reduce the resulting inertia force the structure must resist. This is done by the insertion of energy absorbing material between the substructures and superstructures, which will reduce the amount of seismic forces transmitted. In traditional structures, subjected to random and/or unpredictable loads, plastic hinges are provided. These plastic hinges, which suffer inelastic deformation are generally concentrated at the beam-column joints and are thus associated with damage to the primary structural elements.

On the other hand, installations of mechanical energy absorbers, which are most promising and on which this study concentrated, do as their name suggests. They absorb the energy from the earthquake reducing the effects on the critical components of the structure (FEMA 274). After the earthquake these absorbers, which do not themselves support the structure, are replaced leaving the building undamaged. Once again cost is a factor as neither of the concepts can be justified by cost alone.

There are two types of structural control provided by the addition of mechanical devices, active and passive control. Active control requires a power supply to activate the dampers and hence may be undependable during seismic events where the power supply could be disrupted. For this reason, dampers with active control
have been tested on tall buildings subjected to wind induced loading rather the more unpredictable cyclic loading caused by earthquakes. On the other hand, *passive energy dissipation systems* have emerged as special devices that are incorporated within the structure to absorb a portion of the input seismic energy. As a result, the energy dissipation demand on primary structural members is often considerably reduced, along with the potential for structural damage (FEMA 274).

![Fig. 1.1 Example of typical passive energy dissipating devices](Based on Soong T. T. et al., 1997)

The idea of utilizing separate *passive energy dissipating devices* within a structure to absorb a large portion of the seismic energy began with the conceptual and experimental work of Kelly et al. (1972). Today there are various types of manufactured passive dampers available “off the shelf”. These use a variety of materials to obtain various levels of stiffness and damping. These dampers have been reviewed in Soong and Dargush (1997), Constantinou et al. (1998), Symans and Constantinou (1999), Sadek (1999) and Soong and Spencer (2002). Some of these include viscoelastic (VE), viscous fluid, friction and metallic yield dampers. These dampers have different dynamic characteristics and so will affect the seismic response of structures differently.

The characteristic of VE and *viscous dampers* are that, they dissipate energy at all levels of deformation and over a broad range of excitation frequencies. *Friction*
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dampers, on the other hand, dissipate energy only when the slip force is reached and exceeded. Metallic yield dampers dissipate energy through the inelastic deformation of the material. A combination of these dampers can be used within the structural system to effectively damp out the high and low frequency content of earthquakes. This is commonly referred to as a hybrid system.

1.2 Research Problem

As mentioned in the previous section, the use of passive energy dissipation devices has become very popular in the recent years. However, the vast majority of applications was realised within frame structures, while investigations on use of damping devices within cut outs of shear wall is still very limited. For this reason the aim of this research is to investigate the behaviour of multi-storey frame-shear wall building structures under earthquake loads with damping devices strategically located within the cut outs of the shear wall. The research will evaluate the influence of different damping systems on the overall seismic response of the structure.

Friction, VE, and hybrid friction-VE damping mechanisms of different size, configuration and placed at various locations were treated. The damping mechanisms were modelled as a contact pair with friction parameter for a friction damper, a linear spring and dash-pot in parallel for the VE damper, and a hybrid damper consisting of both a friction and a VE damping mechanism.

Finite element techniques with time history analysis were employed to investigate the effect of these damping systems under seismic loading of five different earthquake excitations. For the purposes of this study, the program selected for the numerical analysis was ABAQUS / Standard version 6.3. This is a general purpose finite element program that has been designed to solve a wide range of linear and non-linear problems involving static, dynamic, thermal and electrical response of systems. In conjunction with this program MSC / PATRAN Version 2004 was used as the pre-processor for generating the geometry, element mesh, boundary conditions and loading conditions, and as the post-processor for viewing.
the results of the analysis, and generating graphs of the response of buildings. A direct integration dynamic analysis was selected to obtain the response of the structures under earthquake loading. This analysis assembles the mass, damping and stiffness matrices and solves the equation of dynamic equilibrium at each point in time. The response of the structure was obtained for selected time steps of the input earthquake accelerogram.

To study the effectiveness of the damping systems in mitigating the seismic response, the tip displacements and accelerations of each structure were acquired from the results of the analysis and compared with those of undamped structure.

1.3 Aims and objectives

The main aim of this project is to generate fundamental research information on the seismic performance of building structural systems having passive damping devices installed within shear walls.

Additional objectives are:

- Create computer models of building structures – damper systems for investigation
- Study the effects of important parameters such as damper properties, locations and configuration of the dampers and earthquake types
- Use the research findings to propose more effective damping system for seismic mitigation.

1.4 Method of investigation

- This research was carried out using computer simulations. Finite element models of the shear wall of buildings was set up and analysed under five earthquake excitations
- The damping mechanisms were modelled as a linear spring and dash-pot in parallel for the VE damper, a contact pair with friction parameter for a friction damper and a hybrid damper consisting of both a VE and a friction damping mechanism
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- Size and material properties of the structure, damping properties, configuration and location of dampers, and earthquake types, were parameters in this investigation and the influence of these was studied.
- Evaluation of results and reporting major findings.

1.5 Scope of research

This research investigates seismic response of building structures with embedded dampers. The main response parameters are tip deflection and tip acceleration of the structure. The scope of this investigation is as follow:

- Building structures
  - Frame shear wall structure is the range of height 48 m to 96 m
  - The structures have natural frequencies within the range of dominant frequencies of the earthquakes treated here

- Damping mechanisms
  - Friction, viscoelastic and combined friction-viscoelastic dampers are considered

- Damper configurations
  - Diagonal, chevron braced, combined diagonal-chevron braced and lower toggle configurations are used

- Damper locations
  - Across the height of the structures with the dampers installed in one, two or three at a time

- Seismic records
  - Five different earthquakes, each with different duration of strong motion and range of dominant frequencies were used. All the seismic records were scaled to have the same peak ground accelerations to facilitate comparison and to suite Australian (low) seismic conditions.

The comprehensive investigation treating all the above parameters will provide results, which can be used to establish:
- The feasibility of using embedded dampers in seismic mitigation and
- Placement of dampers for the best results for different structures.
1.6 Layout of thesis

The material contained in this thesis is presented as seven chapters. They are the following:

**Chapter 1  Introduction**  
Presents the background and introduction to the topic, defines research problem, states the aims and objectives and outlines the method of investigation used in this research project.

**Chapter 2  Literature Review**  
Highlights a review of previous literature published on the behaviour of a passive energy dissipation devices used in building structures under seismic loading. It then identifies the need and scope of the present research.

**Chapter 3  Model Development and Verification of Results**  
This chapter describes the method of investigation and the building structures and damping systems used in this research project. The development of finite element models and some model calibration are also presented.

**Chapter 4  Results – High-Rise Structures**  
The results of finite element analyses of high-rise frame-shear wall structures, 24 storey high, embedded with five types of damping systems obtained under variety of earthquake excitations are presented.

**Chapter 5  Results – 18-Storey Structures**  
The results of finite element analyses of 18-storey frame-shear wall structures embedded with six types of damping systems obtained under variety of earthquake excitations. Evaluation of results and application of findings to establish damping systems for best results.

**Chapter 6  Results – 12-Storey Structures**  
The results of finite element analyses of 12-storey frame-shear wall structures embedded with six types of damping systems obtained under different earthquake excitations. Evaluation of results and application of findings to establish best damping model.

**Chapter 7  Conclusions and Recommendations**  
Highlights the major results and the main contributions of this research and makes some recommendations for further research.
Chapter 2

Literature Review
Chapter 2: Literature Review

2.1 Introduction

The surface of the Earth consists of 12 solid and rigid plates 60-200 km thick. These plates are floating on top of a more fluid zone, constantly moving, sinking at the boundaries, and being regenerated. This movement build up large tectonic stresses, resulting in many tiny shocks and a few moderate earth tremors. In some parts, strain can build up for hundreds of years, producing great earthquakes when it finally releases. Tectonic plates are somewhat flexible. The motion between them is not confined entirely to their own boundaries. Earthquake motion also occurs away from plate boundaries. These earthquakes are mostly caused by localised stresses, variations in temperature and strength within plates. Large and small earthquakes can also occur on faults not previously recognized.

An earthquake is in technical terms, the vibration, generated by a sudden dislocation of segment of the crust. The crust may first bend and then, when the stress exceeds the strength of the rocks, breaks and moves to a new position. In the process of breaking, vibrations called seismic waves are generated. These seismic waves travel outward from the source of the earthquake along the surface and through the Earth at varying speeds and frequency depending on the material.
through, which they move. The two general types of vibrations produced by earthquakes are surface waves, which travel along the Earth's surface, and body waves, which travel through the Earth. Surface waves usually have the strongest vibrations and probably cause most of the damage done by earthquakes.

Body waves are of two types, compressional and shear. Both types pass from the focus of an earthquake to distant points on the surface, but only compressional waves travel through the Earth's molten core. Compressional waves, called “P wave” travel at great speeds and push tiny particles of Earth material directly ahead of them or displace the particles directly behind their line of travel. Shear waves, called “S waves” do not travel as rapidly through the Earth's crust and mantle as do compressional waves, and they ordinarily reach the surface later. Instead of affecting material directly behind or ahead of their line of travel, shear waves displace material at right angles to their path and therefore sometimes called "transverse" waves.

If the earthquake occurs in a populated area, it may cause many deaths, injuries and extensive property damage. However, the earthquakes of large magnitude do not necessarily cause the most intense surface effects. The earthquake's destructiveness in a given region depends on local surface and subsurface geologic conditions. An area underlain by unstable ground is likely to experience much more noticeable effects than an area equally distant from an earthquake's epicentre but underlain by firm ground such as granite. In addition to magnitude and the local geologic conditions level of destruction depends on other factors. These factors include the focal depth, the distance from the epicentre, the density of population, constructions in the area shaken by the quake and the design of building structures.

2.2 Seismic design concept

In conventional seismic design, acceptable performance of a structure during earthquake excitation is based on the lateral force resisting system being able to absorb and dissipate energy in a stable manner for a large number of cycles.
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(Constantinou et al., 1998). Conventional design approach is not applicable in situations when a structure must remain functional after earthquake. In such cases, the structure must be designed with sufficient strength to minimize the inelastic deformations, however, this approach is very expensive. Furthermore, in such structures, special precautions need to be taken in safeguarding against damage of important secondary system, which are needed for continuing serviceability. Over the past couple of decades the astounding developments in alternate design strategies have been made, which incorporate, earthquake protective systems in the structure. These protective systems are in the form of:

1) Seismic isolation systems:
   - Elastomeric bearings
   - Lead rubber bearings
   - Combined elastomeric and sliding bearings
   - Sliding friction pendulum system
   - Sliding bearings with restoring force

2) Supplemental energy dissipation devices:
   - Passive energy dissipation
     - Metallic dampers
     - Friction dampers
     - VE solid dampers
     - VE or viscous fluid dampers
     - Tuned mass dampers
     - Tuned liquid dampers
   - Semi-active and Active systems
     - Active bracing systems
     - Active mass dampers
     - Variable stiffness and damping systems
     - Smart materials

Conventional design approach, seeks to prevent occurrence of inelastic deformations by allowing structural members to absorb and dissipate the transmitted earthquake energy by inelastic cyclic deformations in specially created regions. This strategy implies that some damage may occur, possibly to the extent that the structure is no longer repairable.

The method of structural isolation deflects or filters out the earthquake energy by interposing a layer with low horizontal stiffness between the structure and the
foundation. The structural isolation system includes wind resistant and tie down system and also includes supplemental energy dissipation devices to transmit force between the structure above the isolation system and the structure below the isolation system. The structural isolations are suitable for a large class of structures that are short to medium height, and whose dominant modes are within a certain frequency range. However, in an earthquake rich in long period components, it is not possible to provide sufficient flexibility for the reflection of the earthquake energy. Several building and bridges have now been installed with base isolation systems (for example Tokyo Port Terminal, Tohoku Electric Power Company, University of Southern California Teaching Hospital, Los Angeles, Marina Apartments San Francisco).

Another approach to improving earthquake response performance is that of supplemental energy dissipation systems. The primary reason for introducing energy dissipation systems into building frames is to reduce the displacement and damage in the structure. Displacement reduction is achieved by adding either stiffness and/or energy dissipation to the building structure. In these systems, mechanical devices are incorporated into the frame of the structure and absorb the energy from the earthquake reducing the drift as well effects on the critical components of the structure. These mechanical energy dissipating devices have been found to be quite promising and their applications form the focus of this study.
2.3 Analysis procedures

Today, there are a number of methods available for building structures subjected to seismic loading (Newmark and Hall, 1987, Krinitzsky et al., 1993, Fagan, 1992). Four of them, namely linear static, linear dynamic, non-linear static, non-linear dynamic, are described in great details in FEMA 273/274.

2.3.1 Static analysis procedures

A static analysis is a quick and easy way to obtain an approximate response of a structure. Generally, this method determines the distribution of the earthquake base shear force, in a given direction, throughout the height of a structure. According to linear static procedures, static lateral forces are applied to the structure to obtain design displacements and forces (FEMA 273/274). The method is based on two important assumptions. First, it is implied that an adequate measure of design actions can be obtained using a static analysis, even though seismic response is dynamic. Second, it is implied that an adequate measure of design actions can be obtained using a linearly-elastic model, even though nonlinear response to strong ground shaking may be anticipated. The guidelines provide also criteria to determine when nonlinear procedures are required as an alternative. Preferably, the evaluation of a nonlinear deformations should be carried out using nonlinear procedures that explicitly account for nonlinear deformations in deformed components. As an option, the guidelines permit evaluation to be carried out using linear procedures. In a linear procedure, there is a direct relation between internal forces and internal deformation for all types of loading. Hence, when using linear procedures, it is simpler to express acceptability in terms of internal forces rather than internal deformations (FEMA 273/274).

2.3.2 Dynamic analysis procedures

Similarly as the linear static, also the linear dynamic procedure uses the same linearly-elastic structural model. The linear dynamic procedure provide greater insight into the structural response, however, similarly to linear static procedure, it does not explicitly account for effect of nonlinear response. On the other hand, in
the case of nonlinear dynamic procedure, the nonlinear load-deformation behaviour of individual components and elements is modeled directly in the mathematical model. There are two general types of dynamic analysis to choose from for structures subjected to seismic ground motions, which are the response spectrum and type history analysis.

The response spectrum is defined as a graphical relationship of the maximum acceleration response of a SDOF elastic system for various values of natural period, $T$, obtained by changing the structural properties, with damping to the dynamic motion of force (Newmark and Hall, 1987). The response spectrum requires dynamic analysis of a structure to establish modal frequencies and mode shapes. Using standard mathematical procedures (Clough and Penzien, 1993) and a response spectrum corresponding to the damping in the structure, the modal frequencies and shapes are used to establish spectral demands. The spectral demands are then used to calculate displacements, forces, storey shears and base reactions for each mode of response considered. Consequently, all these results are combined by using established rule to calculate total response quantities (FEMA 273/274).

The time history analysis determines the response of a structure due to forces, displacements, velocities or accelerations that vary with time. There are two types of this method, first is direct integration and the second, modal superposition (Clough and Penzien, 1975). Modal superposition is only suitable for linear analysis, whereas direct integration can be used also for nonlinear analysis. The direct integration utilize a step-by-step solution of equation of motion, which is generally described as:

$$ M\ddot{U} + C\dot{U} + KU = F(t) $$

(2.1)

where, $M, C, K$ are the mass, the damping, and the stiffness matrices, respectively, $U, \dot{U},$ and $\ddot{U}$ are the displacement, velocity and acceleration vectors, respectively, $F(t)$ is the vector of applied forces, which may varied with time. The most popular integration scheme is the Newmark-$\beta$ method, which is implicit and unconditionally stable. The following approximations are made in this method:
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\[
\{U\}_{k+dt} = \{U\}_k + \left(\{U\}_k + \{\dot{U}\}_{k+dt}\right) \frac{dt}{2}
\]  \hspace{1cm} (2.2)

\[
\{U\}_{t+dt} = \{U\}_t + \{\dot{U}\}dt + \left(\frac{1}{2} - \beta\right)\{\ddot{U}\}_t + \beta\{\ddot{U}\}_{t+dt}\left(dt\right)^2
\]  \hspace{1cm} (2.3)

where, \(dt\) is the time step of the analysis, and \(\beta\) is the structural damping depend on an amplitude decay factor, but usually a value of 0.25 is used.

2.4 Passive energy dissipation devices

The function of Seismic Isolation and Energy Dissipation System is to reduce structural response due to earthquake, wind and other dynamic loads. These devices, which are also known as motion control system, can absorb part of the energy induced in the structure, reducing energy dissipation demand on the primary structural members, and thus reducing the structural deflection and non-structural deformations. The term control systems denotes what was previously termed energy dissipation system, whereas the terms passive and semiactive denote, respectively, systems that require no externally supplied power and systems that require minimal externally supplied power to operate.

Passive control system develops control forces at the points of attachment of the system. The power needed to generate these forces is provided by the motion of the points of attachment during dynamic excitation. The relative motions of these points of attachment determine the amplitude and direction of the control forces. On the other hand, an active control system develops motion control forces. However, the magnitude and direction of these forces are determined by a controller based on information from sensors and control strategy and supplied by the active control system. An active control system should, in principle provide for more versatile response control, however, they require a power supply to be activated and hence may be undependable during seismic events where the power supply could be disrupted. For this reason, dampers with active control have been tested on tall buildings subjected to wind induced loading rather than the more unpredictable cyclic loading caused by earthquakes.
A semi-active Control System generally originated from passive control systems, which was modified to allow for adjustment of their mechanical properties (shearing of viscous fluid, orificing of fluid or sliding friction). The mechanical properties of semi-active control systems may be adjusted by a controller. Power source required for the semi-active control system is typically very small and remotely related to the power output of the system.

In the last two decades, variety of energy dissipation devices (EDD) has been developed to enhance safety and reduce damage during earthquake excitations. These devices utilize a wide range of material and technologies as a means to increase the damping, stiffness and strength characteristics of structures. Energy dissipation may be achieved either by the transformation of kinetic energy to heat or by the transferring of energy among vibrating modes.

Fig. 2.3 Example of semi-active control system
(Based on Kobori et al., 1995)
The first mechanism includes either hysteretic devices, which dissipate energy without considerable rate dependence, and VE or viscous devices, which, in contrast, exhibit substantial rate dependence. Hysteretic devices operate on principles such as yielding of metals and frictional sliding, while VE or viscous devices involving deformation of VE solids or viscous fluids and those employing fluid orificing. The second mechanism consists of re-centering devices that utilize either a preload generated by fluid pressurization or internal springs, or a phase transformation to produce modified force-displacement response that includes a natural re-centering component.

### 2.4.1 Hysteretic energy dissipation devices

A variety of hysteretic energy dissipation devices has been proposed and developed to enhance structural protection. Most of these devices generate rectangular hysteresis loop (Fig. 2.4), which indicates that behaviour of friction dampers is close to that of Coulomb friction. The simplest models of hysteretic behaviour involve algebraic relation between force and displacement. Hence, hysteretic systems are often called displacement dependant.

![Idealized force-displacement loops of hysteretic devices](Based on Soong et al., 1997)

In general, hysteresis devices dissipate energy through a mechanism that is independent of the rate of load frequency, number of load cycles, or variation in temperature. In addition, hysteresis devices have high resistance to fatigue. Hysteresis devices include metallic dampers that utilize the yielding of metals as the dissipative mechanism, and friction dampers that generate heat through dry
sliding friction. The both types of devices are inherently non-linear, that means, that the force output does not scale with the displacement, and significant path dependence is apparent. The non-linearity of hysteretic devices must be considered in structural analysis as well as in design. It should also be noted that energy dissipation occurs only after a certain threshold force is exceeded.

2.4.1.1 Metallic dampers

Metallic dampers utilize the hysteretic behaviour of metals in the inelastic range. The resisting forces of the dampers, consequently, depend on the nonlinear stress-strain characteristics of the metallic material. A wide variety of damping devices that utilize shear, flexure and material deformation in the plastic range have been developed and tested. The most desirable characteristic of these devices are their stable hysteretic behaviour, low-cycle fatigue property, long term reliability, and relative insensitivity to change in temperature. In addition, these devices are relatively inexpensive and their properties will remain stable over the long lives of structures. Disadvantages of these devices are their limited number of working cycles and their non-linear response.

Fig. 2.5 Two examples of X-shaped plate dampers
(Based on Whittaker et al., 1991)

X-shaped plate dampers are probably the most popular among the metallic dampers. The parallel metallic plate used to be installed within a frame bay between a chevron brace and the overlying beam. Consequently, the dampers
primarily resist the horizontal forces related to inter-storey drift via flexural deformation of the individual plates. Beyond a certain level of force, the plates yield and hence provide substantial energy dissipation. The tapered shape of the plates generates nearly uniform yielding over the entire plate surface.

X-shaped plate dampers consisting of multiple X-shaped steel plates were introduced by Bechtel Power Corporation. Comprehensive experimental studies of these dampers have been carried out by Bergman et al., (1987) and Whittaker et al., (1991). Some applications of non-linear structural analysis to building frames incorporating metallic dampers can also be found in Xia and Hanson (1992), Jara et al., (1993). There are also several interesting alternatives to the modified Newton-Raphson algorithm. Variations of the steel cross-bracing dissipaters have been developed in New Zealand, Italy and USA. The results of these studies showed that the X-shaped plate dampers exhibited good performance and proved to be stable under large axial loads in the device.

![Triangular shaped damper and its hysteresis loops](Based on Tsai et al., 1997)

**Triangular plate dampers** were originally developed in New Zealand and used in several base isolation applications. Later, after experimental work carried out by Tsai and Hong (1992), they were also used in buildings. Triangular steel yielding devices have also been used in Japan. Device similar to the triangular plate...
dampers was developed by Obayashi Corporation (Soong and Constantinou, 1994). Another device, a yielding damper consisting of a short lead tubes loaded to deform in shear was developed for developed by Sakurai et al., 1992).

*Yielding steel bracing systems* fabricated from round steel bars for cross-braced structure have been developed in New Zealand (Skinner et al., 1980; Tyler, 1985). Energy is dissipated by inelastic deformation of the rectangular steel frame in the diagonal direction of the tension brace, as shown in Fig. 2.7. Several modifications of the steel cross–bracing dissipater have been developed and installed in Italy (Ciampi, 1991).

![Fig. 2.7 Yielding steel bracing system](Based on Tyler et al., 1985)

All these metallic yielding dampers may be effective in reducing the response of structures to earthquake loading. The post-yielding deformation range of these dampers is a major concern, which should be addressed to insure that the dampers can sustain a sufficient number of cycles of deformation without premature fatigue. Another problem, which should be worked out carefully, is the stable hysteretic behaviour of the dampers under repeated inelastic deformation.

*Lead extrusion damper* (LED) represents another class of dampers, which utilized the hysteretic energy dissipation properties of metal. The process of extrusion consists of forcing a lead piston through a hole or an orifice, thereby changing its shape. LEDs were first suggested by Robinson et al., (1987), as a passive energy
dissipation device for base isolated structures in New Zealand. As it can be seen from Fig. 2.8, there are two types of LEDs introduced by Robinson. The first device consists of a thick-walled tube and a co-axial shaft with a piston. There is a constriction on the tube between the piston heads and the space between the piston heads is filled with lead. The central shaft extends beyond one end of the tube. When external excitation occurred, the piston moves along the tube and the lead is forced to extrude back and forth through the orifice formed by the constriction of the tube.

![Fig. 2.8 Two examples of lead extrusion devices](Based on Robinson et al., 1987)

The second type of LED is similar to the first one, except that the extrusion orifice is formed by a bulge on the central shaft rather than by a constriction in the tube. The shaft is supported by bearings which also serve to hold the lead in place. As the shaft moves, the lead must extrude through the orifice formed by the bulge and tube. Similar to most friction devices, the hysteretic behaviour of LED is essentially rectangular.

The load deformation relationship of LED is stable and unaffected by the number of loading cycles. They have a long life and do not have to be replaced or repaired after an earthquake excitation since the lead in the damper returns to its
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undeformed state after excitation. The lead extrusion devices are insensitive to environmental conditions and aging effects.

2.4.1.2 Friction dampers

A wide variety of friction dampers has been developed and installed in building structures. Friction dampers can provide mechanism for dissipation of large amount of energy and they have good performance characteristics and their behaviour is less affected by the load frequency, number of load cycles, or changes in temperature. Friction damper exhibit rigid-plastic behaviour and force response is modelled by Coulomb friction.

X-braced damper was proposed by Pall et al., (1982). In this type of damper, the braces in a moment resisting frame incorporated frictional devices. When load is applied to this damper, the tension brace induces slippage at the friction joint. Consequently, the four links force compression brace to slip. Energy is dissipated in both braces even though they were designed to be effective in tension only. However, this is only valid if the slippage of the device is sufficient to completely straighten any buckled braces.

![Bracing-damper system](image)

*Fig.2.9 Pall friction damper*  
*(Based on Pall and Marsh, 1982)*

*Bracing-damper system*, which is a more detailed model for the device in which each member of the bracing-damper system is represented by element reflecting
its individual axial and bending characteristics. The structural braces are assumed to yield in tension, but buckle elastically in compression was proposed by Filiatrault and Cherry (1987). Bending stiffness is included to maintain stability of the damper mechanism. The authors conducted some physical experiments from which, made obvious, that seemingly minor fabrication details can significantly affect the overall performance at the friction damper.

*Improved Pall friction damper* was developed by Wu et al., (2005). The construction details, operation and damper force of the improved damper were compared with those of the original Pall friction damper. The analyses and validation experiment carried out by authors showed that the resisting forces generated by these two dampers were identical with identical frictional forces. Hence the improved damper replicated the mechanical properties of the original damper, but offers some advantages in terms of ease of manufacture and assembly.

*Uniaxial friction damper* (Fig. 2.10) manufactured by Sumitomo Metal Industries Ltd., utilizes a slightly more sophisticated design. The pre-compressed internal spring exerts a force that is converted through the action of inner and outer wedges into a normal force on the friction pads. These copper alloy friction pads contain graphite plug inserts, which provides dry lubrication. This helps to maintain a consistent coefficient of friction between the pads and the inner surface of the stainless steel casing.

![Uniaxial friction damper](Based on Sumitomo Metal Industries Ltd 1992)
Aiken and Kelly (1990) indicated that the response of these dampers is extremely regular and repeatable with rectangular hysteresis loops. Furthermore, the effect of loading frequency and amplitude, number of cycles, or ambient temperature on damper response was reported as insignificant. The reduction in displacements, however, depend on the input ground motion because friction dampers are not activated and do not dissipate energy for forces smaller than threshold. The placement of the dampers did not influence base shears significantly.

*Sumitomo friction damper* was investigated by Aiken and Kelly (1990). The authors performed experimental and numerical examinations of this damper installed on 1/4-scale 9 storey steel frame in conjunction with chevron brace assembly. The performance of the friction dampers was outstanding. The hysteresis loops showed very consistent, nearly ideal Coulomb behaviour throughout the duration of the test and approximately 60% of the input energy was dissipated in the dampers.

![Fig. 2.11 Installation of uniaxial friction damper in steel frame](Based on Sumitomo Metal Industries Ltd., 1992)

*Energy Dissipating Restraint* (EDR) shown in Fig. 2.12 was manufactured by Fluor Daniel, Inc. The design of this friction damper is superficially similar to the Sumitomo damper, since this device also includes an internal spring and wedges encased in a steel cylinder. However, there are some novel features of the EDR that produce very different response characteristics. The EDR utilizes steel and bronze friction wedges to convert the axial spring force into normal pressure
acting outward on the cylinder wall. Therefore, the frictional surface is created by the interface between the bronze wedges and the steel cylinder. Internal stops are installed within the cylinder in order to create the tension and compression gaps. The length of the internal spring can be changed during operation to provide a variable friction slip force.

*Fig. 2.12 Energy dissipating restraint (Based on Fluor Daniel, Inc., 1993)*

*Slotted bolted damper*, proposed by Fitzgerald et al., (1989) can be seen in Fig. 2.13. This damper allows slip to occur in slotted bolted connections. The connection consists of gusset plate, two back to back channels, cover plates, and bolts with washers. The sliding interface consists of steel.

*Fig. 2.13 Three examples of slotted bolted connections (Based on Fitzgerald et al., 1989, Grigorian et al., 1993, and Constantinou 1991)*
Grigorian and Popov (1993), introduced and tested a slotted bolted damper with the sliding interface consisted of brass and steel. A frictional characteristic of modified interface were a significantly more stable than those of steel interface. Earthquake simulator tests of a three-storey steel building model with the slotted connection have been carried out by Grigorian and Popov who showed the effectiveness of the device in reducing the seismic response. Utilizing of graphite impregnated bronze plates proposed by Constantinou et al., (1991), is another alternative to improve the frictional characteristic of the slotted bolted damper. *Rotating slotted bolted friction damper* with inclined slotted holes has been successfully proposed and tested by Butterworth (1999).

*Concentrically braced frame* proposed by Yang and Popov (1995) is representative of another type of friction dampers. Concentrically braced frames offer one of the most efficient lateral loads resisting system available, which combine strength, stiffness, low weight and simple construction. Unfortunately, under seismic loading the applying of light tension-only tends to leads to ‘soft storey’ failure due to irrecoverable tensile yielding. To remedy this disadvantage, the authors suggested the used of specially detailed sliding plates moving in the vertical plane.

### 2.4.2 Viscoelastic dampers

*VE dampers* have force displacement characteristics that are function of either the relative velocity between the ends of the damper or the frequency of the motion. However, the response of these devices may also be a function of relative displacement. VE devices exhibit stiffness and damping coefficients, which are frequency dependant. Moreover, the damping force in these devices is proportional to velocity, that the behaviour is viscous. Research and development of the VE dampers for seismic applications began in the early 1990s. Over the last few years extensive experimental program has been designed and carried out for steel frames and lightly reinforced concrete frames. VE dampers are mostly used in structures where the damper undergoes shear deformations.
Solid VE dampers are constructed from constrained layers of acrylic polymers or copolymers and designed to produce damping forces through shear deformations in the VE material. When deformed, the VE materials exhibit the combined features of an elastic solid and viscous liquid i.e. they return to their original shape after each cycle of deformation and dissipate a certain amount of energy as heat.
were found to significantly improve the response of the frame and reduce inter-storey drifts and storey shears. These results have led to the development of design procedures for structures fitted with supplemental VE dampers.

*Bitumen rubber compound VE damper* developed by Showa and Shimizu corporations induces large damping forces to shear deformation and can sustain shear strains of about 300%. The test results indicated a 50% reduction in the seismic response of the frame. *Super-plastic silicone rubber VE shear damper* developed and tested by Kumagai-Gumi Corporation at the top connection of a wall panel to the surrounding frame. Tests of ½ scale 3-storey steel frame show response reductions of up to 60%.

*Fluid VE devices*, such as viscous shear walls, operate by shearing VE fluids. They possess the response characteristics similar to solid VE dampers, except that the fluid VE dampers do not exhibit stiffness when loads with low frequencies are applied (Fu and Kasai, 1999). The dampers, which utilise the viscous properties of fluids have been developed and used in several structural applications. Force-displacement loop of viscous damping devices is shown in Fig.2.16.

![Fig. 2.16 Idealized force-displacement loop of viscous devices](Based on Soong et al., 1997)

*Viscous-damping wall system* was developed by Sumitomo Construction Company in Japan. The device consists of an outer steel casing attached to the lower floor, filled with a highly viscous fluid. An inner moving steel plate hanging from the upper floor is contained within the steel casing. The viscous damping force is induced by the relative velocity between the two floors. Arima et
al., (1988) conducted experimental test on a full scale 4-storey steel frame with and without viscous damping walls. The results of frame fitted with viscous walls revealed response reductions of 66 to 80%. A 4-storey reinforced concrete building with viscous damping walls was constructed in Tsukuba, Japan and has since been monitored for earthquake response. The viscous damping walls installed in the 78 m high steel frame building in Shizuoka City, Japan, provided 20 to 35% damping and reduced the building response up to 70 to 80% (Miyazaki and Mitsusaka, 1992).

Fluid viscous dampers operate on the principle of fluid flow through orifices. Viscous dampers can dissipate large amount of energy over a wide range of load frequencies. The main advantage of viscous dampers is that, they can reduce building deflection and stress at the same time. This is because the force from the dampers is completely out of phase with stresses due to flexing of the structure. Viscous dampers are relatively insensitive to temperature changes. However, these dampers are not suitable for stiff structures due to high damper force requirement.

Fluid viscous dampers, which operate on the principle of fluid flow through orifices, are installed in a number of structural applications (some of them can be seen from Figs. 2.18-2.20).
Viscous dampers of varieties of materials and damping parameters were proposed and developed for seismic protection. These dampers possess linear viscous behaviour when force response of viscous dampers is proportional to velocity, so structural design is generally straightforward. Comprehensive experimental and analytical studies of buildings fitted with viscous dampers manufactured by Taylor Devices, Inc. have been carried out by Lee and Taylor (2002). They affirmed that the addition of currently available viscous dampers into a structure could provide damping as high as 35% critical.

In a study conducted by Pong, (1994) indicated that the major disadvantage of using fluid dampers as energy absorbing devices is that the peak structural response cannot be reduced significantly if it occurs in the early stages of excitation because of the dependence of the damper’s resisting force on the velocity. To overcome this shortcoming, Pong suggested using a combination of tapered-plate energy absorber and fluid dampers.
2.4.3 Dynamic vibration absorbers

*Dynamic vibration absorbers* have been examined in several numerical and experimental studies. In order to reduce input of seismic energy, the dynamic vibration absorbers involve mass, stiffness and damping. Their dynamic characteristics must be tuned to those of the primary structure. The most popular representatives of this category of devices are *Tuned Mass Damper (TMD)* and *Tuned Liquid Damper (TLD)*.

*TMD* consists of a mass, which moves relative to the structure and is attached to it by a spring and a viscous damper in parallel, as shown in Fig. 2.21. The structural vibration generates the excitation of the TMD. As a result, the kinetic energy is transferred from the structure to the TMD and is absorbed by the damping component of the device. The TMD usually experience large displacements.
TMD incorporated into a structure where the first mode of the structural response dominates, is expected to be very effective. The optimum tuning and damping ratios that result in the maximum absorbed energy have been studied by several investigators. TMDs have been found effective in reducing the response of structures to wind and harmonic loads and have been installed in a number of buildings. Numerical and experimental results reveal that the effectiveness of TMDs on reducing the response of the same structure under different earthquake excitations or of different structures under the same earthquake is considerably different. As a result, there is not a general agreement about the effectiveness of TMDs for seismic applications.

The *tuned liquid damper* (TLD) and *tuned liquid column damper* (TLCD), similarly to a TMD impact indirect damping to the system and thus improve structural response. A TLD dissipate energy by means of viscous actions of the fluid and wave breaking. In the case of TLCD, energy is dissipated by the passage of liquid through an orifice with inherent head loss characteristics.

TLDs (Fig. 2.22) consist of rigid tanks filled with liquid, where the energy is absorbed by the sloshing motion and dissipates it through viscous action of the liquid. TLDs have several advantages over the previously described TMDs, which are:

- Reducing the motion in two directions simultaneously.
- Do not require large stroke lengths.
- No activation mechanism is required.
- Minimum maintenance cost.

TLD systems are not very sensitive to the actual frequency ratio between primary and secondary system. However, the relatively small mass of fluids compared to the large mass of TMDs requires larger spaces to achieve the same damping effect. TLCD contain liquid-filled tube-like containers, which are rigidly attached to the structure. Energy is dissipated by the movement of the liquid in the tube through an orifice. The vibration frequency of the device can be adjusted by changing the liquid column length (Sakai, 1989). TLCDs are relatively simple to
implement in existing buildings since they do not interfere with vertical and horizontal load paths. TLCDs do not require the space for large stroke lengths. Damping is increased by adjusting the orifice opening.

In a study conducted by Sadek (1999), the optimum parameters of TLCDs for seismic applications depend on the results of a deterministic response analysis of a series of SDOF structures to earthquake ground accelerations. The results of this study indicate that the use of the optimum parameters reduces the displacement and acceleration responses up to 47% (mass ratio of 0.04).
2.4.4 Phase transformation dampers

Phase transformation dampers are energy dissipating devices, which include a new class of materials referred to as shape memory alloy (SMA). Some of the most promising characteristics of the martensitic and superelastic modes of SMA behaviour are:

- High stiffness for small strain (elastic loading)
- Reduced stiffness for intermediate strain (due to formation of martensite)
- High stiffness at large strain (elastic loading of martensite)

Since the superelastic state ideally displays a hysteretic effect with zero residual strain, an energy absorbing device made of this material would theoretically provide a self-centering mechanism. Other attractive properties associated with SMA include their insensitivity to environmental temperature changes when properly heat treated, and their excellent fatigue and corrosion resistance properties. These metals are capable to produce large control forces despite its slow response time. Since most structures deal with low frequency content, it is possible to take advantage of these metals without much compromise.

![Stress-strain response of shape memory alloys](Based on Li, J. and Samali, B., 2000)

Aiken (1992) incorporated small loops of Nitinol to study possibility of using this material as a passive energy dissipation device for structures. The results of the testing included two types of behaviour of special interest: large strain behaviour and cyclic superelastic behaviour. The advantage of this behaviour is that, for low seismic excitation, the structure behaves elastically, for moderate earthquakes, the
Nitinol dissipate large amount of energy, while remaining elastic. Finally, in the case of large earthquakes, the structure stiffens and again dissipates large amounts of energy. Behaviour of Nitinol during a moderate earthquake can be seen in Fig. 2.24.

While no actual structural applications have taken place, intensive research and experimental work were undertaken by Samali and Wu (2000). The results of these investigations are more than encouraging, however, these results also revealed that SMA is very sensitive to earthquake excitation type and can produce change in stiffness and consequently change the first natural frequency of the structure toward the earthquake dominant frequency. For these reasons, a careful design is necessary for all implementations of SMA in structural control.

2.4.5 Hybrid dampers

The characteristics of VE and viscous dampers are that they dissipate energy at all levels of deformation and over a broad range of excitation frequencies (Chang et al., 1993). On the other hand, friction dampers dissipate energy only when the threshold force is reached and exceeded (Aiken et al., 1991). Yielding dampers dissipate energy through the uniform deformation of the steel (Tsai et al., 1992). A combination of these damping mechanisms can be apply within the structure to efficiently dampen out broad range of frequency content of seismic excitations (Hisano et al., 2000, Ribakov 2004, Shao et al., 1999). Such device is commonly referred to as a hybrid damping system. However, due to a variety of practical problems, development and utilizing of hybrid damping systems is still rather limited. As example of such problems can be used hybrid friction-VE damper. When the VE material dissipates energy, it heats and softens. The frictional element does not, so at a certain point the frictional element is not pushed hard enough to slip. Since these difficulties, only a limited number of hybrid damping systems have found practical application.

_Hybrid friction-VE damper_ developed by company Damptech (2000) is probably, the most popular representative of this category. This damper combines the
Influence of Damping Systems on Building Structures Subject to Seismic Effects

advantages of pure frictional and pure VE energy dissipation mechanisms. The damper consists of friction pads and VE polymer pads separated by steel plates as shown in the Fig. 2.25. A prestressed bolt in combination with disc springs and hardened washers is used for maintaining the required clamping force on the interfaces as in the original FD concept. Intensive testing is still underway with different VE materials. It has been experimentally confirmed that well designed friction-VE damper develop less than the half of peak response displacement of its non-damped counterpart and is also capable of dissipating 75%-90% of seismic input energy.

![Fig. 2.25 Hybrid friction-VE damper](Based on Damptech, 2000)

2.5 Implementation of supplemental devices to building structures

Since the *Energy dissipation devices (EDDs)* are relatively new in the structural design and were introduced to the market by the individual producers, there are no clear design procedures for their implementation to building structures. As it was described in Section 2.3, the guidelines for building structures subjected seismic loading identify four types of analysis. The most common method is linear procedures, which require modifying the seismic excitation on the structure with the respect of the added damping, in the form of the estimate effective damping. The effective damping is usually formulated by applying device on a single degree of freedom (SDOF) system, and taken as the damping ratio of the first mode (FEMA 273/274). This oversimplified procedure, however, is not taking in account the complex structure-device interaction.
In order to investigate the effect of implementation of EDDs on buildings response, both theoretical formulation and numerical simulation can be used. The overall effect of additional damping from the supplemental devices is generally investigated by applying the damping matrix into the original equation of motion. As a result of implementation of damping devices into the building structure, the dynamic properties of the original structural system are changed, including the natural frequencies, mode shapes and modal damping ratios. Since the added damping device often bring non-proportional damping into the system, the dynamic characteristics can not be solved by using orthogonal property of the mode shapes so that the multi degree of freedom (MDOF) system is transformed into a set of SDOF modal systems. A mathematically equivalent state space method can be used to obtain the structural characteristics and seismic response.

The seismic response reductions depend on the structure and its performance requirement, which are displacement, acceleration or element force. For the device configuration with optimal seismic response reduction, the structure has to be analysed carefully first. Subsequently, based on its characteristics and the defined performance index, an optimal device distribution is generated. The optimisation algorithm search the best device configuration from all the possible device location and combinations based criteria, such as minimum response under seismic excitation, under design spectra compatible earthquakes, or under a varied range of earthquakes (FEMA 273/274).

To better understand the effect of added non-proportional damping to the system, several buildings have been studied. These analyses show that the most effective device distributions very much depend on the configuration of the structure and the building performance indices. Currently, the most common retrofit design procedure using EDD is to specify a damping ratio first before the design. After that, the devices are added to the structural system to increase the damping ratio to the specified value. The structure response is then checked for reduction. By this procedure, it is possible to design device configurations with acceptable building seismic response. However, sometimes, due to the oversimplification of the
original system, the neglected higher mode might play an important role in the structural response (Lee et al., 2001).

Hence, the retrofitting design of using additional EDD is not a straightforward matter and there is no general rule, which can be applied to all type of structure, the dynamic characteristics of structure need to be analysed carefully to fulfil the performance objective of the design. The higher modes contributing to this response also need to be analysed, so that the most effective device location is found and the devices are placed at those positions. The largest added damping should not necessarily guarantee the best response reduction. The common practice of EDD design procedure based on maximum effective damping, not always lead to most effective design. Constantinou et al., (2001) defined a design procedure as follows:

1. Structural analysis and simplification
2. Performance index (displacement, velocity, acceleration, combined response)
3. Definition of the constraints of added EDDs
4. Search for optimal configuration of the EDD. This search is based on:
   - Increase model damping for a particular mode
   - Modify a particular mode shape
   - Modify the total modal contribution to a particular mass point
5. Validation

While following this procedure, the designer should be aware that optimal distributions of damping devices throughout the building cause the structure to be non-proportionally damped. To find the effective configuration, it is necessary to calculate characteristics of each potential configuration, compare these with the original structural characteristics, and identify for each configuration. For mass point of interest, the designers should check, which natural frequencies contribute most in the response and examine each corresponding steady state response vector to identify the largest contribution vector. The comparisons should be made among modal shapes, loading factors, natural frequencies and damping ratios. These comparisons reveal the most important modes in the response of the structure.
In addition, the damping distribution is very different if the optimisation target is chosen as the displacement or acceleration. The displacement response is dominated by main modes, therefore, with a higher damping ratio and higher modal participation factor, the displacement response is further reduced (Uriz and Whittaker, 2002). On the other hand, in the case of acceleration response, which includes even higher mode contribution, the response is increasing.

As mentioned previously, displacements are reduced as the effective damping ratio is increased. Many designers believe that fitting more dampers at the level of maximum inter-storey drift will achieve optimised structural response. However, such approach can only be accurate for proportionally damped or SDOF structures. It has been shown by Uriz and Whittaker (2002) that for MDOF systems, higher damping ratios could, in several cases, increase the seismic response. For structures with added EDDs, the damping is no longer proportional or negligibly small. To study the damping effects on MDOF building structures, the analysis method must consider the effects of non-proportional damping.

For non-proportionally damped MDOF systems, the damped mode shape is not orthogonal to the damping matrix in the n-dimensional physical domain. The complex mode shapes are orthogonal in the state-space domain, where real value cannot be directly applied” (Constantinou et al., 2001). The double modal superposition approach developed by Gupta and Law (1986) is used in some analysis, where the response is the superposition of modal displacement and modal velocity. These modes are not the undamped system modes, nor the complex system modes. The conventional modal analysis routine of fast calculation is employed in this approach and the damping effects can be clearly explained by using structural dynamic parameters.

The response of an MDOF non-proportionally damped system damping can be affected by many factors, which include modal shape, modal response, natural frequencies and damping ratio. The damping ratio and natural frequency in these equations are determined by the state vector eigen-value solution. The state vectors for different supplemental damping devices are different, because the
natural frequency, damping ratio and modal shape, are not constant. It can be concluded, that the system response is not always reduced when the damping ratio is increased.

To demonstrate how system characteristic change due to added damping, the research team at Buffalo University (2001) analyzed a four DOF frame structure embedded with linear viscous dampers. The dampers were arranged in 13 different damper configurations (Table 2.1), with four damping ratios (5%, 10%, 20% and 30%).

Table 2.1 Damper location (After Lee et al., 2001)

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<th>Placement/Case</th>
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<td>1st storey</td>
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The results of first complex modal damping ratio for small damping ratios (5%) showed, the complex damping ratio remained nearly the same, however, when the damping ratio was increased to 10%, a dramatic change were observed in case 4, while the other cases stayed unchanged. Case 3, 4, 7, 10, and 12 have very different values when the controlling ratio increased to 20%. Lastly, the first complex damping ratio for cases 1, 3, 4, 7 and 12 became 99% when the ratio was increased to 30%. The results revealed that, as the damping ratio became higher, dramatic changes resulted (in cases 1, 3, 4, 7 and 12) in the fundamental mode collapse. Consequently, the second mode shape turned into dominant in the system response.

As regards to the relationship between the maximum displacement response and the controlling effective damping, except for the case 4, the response decreased as the damping ratio increased. However, as the controlling damping ratio was higher than 15%, different damper configurations resulted in different response reductions even though they may achieve the same effective damping ratio value.
The authors emphasize the limitations of using the damping ratio as the key seismic response reduction measurement and they recommended evaluation of performance, based on response history analysis for all non-proportionally damped structures.

Optimal control theory (OCT) using a linear quadratic regulator is adapted in study conducted by Ribakov et al., (2001) to design the linear passive viscous dampers in accordance to the deformation and velocity. The main difficulty in optimal design of passive viscous damped structures is that the optimal solution requires different levels of damping at each storey, which can be inconvenient and expansive. The aim of this research, was to obtain the required properties of the optimal viscous devices by using the OCT. Detail of the viscous damping system can be seen from Fig. 2.26, where the angle between the damping correctors and the vertical plane was chosen so that the viscous damping system (VDS) consisting of standard viscous dampers produced optimal control forces.

![Viscous damping system - construction scheme](Based on Ribakov et al., 2001)

To investigate the effectiveness of the proposed design technique, simulations of 7-storey shear framed structure with stiff beams were carried out. The results of numerical simulation showed that its performance with VDS including standard dampers was close to the optimal design. Reductions up to 50% in the peak displacements and 60% in the peak accelerations were obtained. The results show a promising method of using off-the-shelf viscous dampers in VDS. Thus, the
damping at each floor is controlled and provides an improved behaviour of the structure during an earthquake.

2.6 Research on seismic mitigation with passive dampers

Though there is a lack of comprehensive guidelines implementing effects of energy dissipation devices on the structural dynamic behaviour, it is still possible to find some partial studies. Chang et al., (1993) investigated the seismic behaviour of steel frame structure with added VE dampers and developed design guidelines based on obtained results. Hahn et al., (1992) developed parametric optimization analyses to determine the optimum damping coefficient for the supplemental dampers to establish the effect of different damper distributions throughout the building height. The results from these analyses showed that dampers should be placed in the lower storeys of the building if they are of uniform storey stiffness.

Several studies introduced complex mathematical formulations to optimise the performance index based on ‘optimal control theory’. Gluck et al., (1996), and De Silva (1981) proposed optimal control theory to establish the optimum damping coefficient and most effective damping coefficients distribution for viscous dampers under random earthquake ground motion. These studies, which utilize a fully populated optimal gain matrix, were created by minimising a quadratic performance index and the parameters related to the supplemental damper coefficients were derived by approximations.

There are also sequential approaches to obtain the optimal damping coefficients distribution and capacity of the devices. Zhang et al., (1992), and Wu et al., (1997) proposed method of the optimal placement of viscous dampers for a building with specified storey stiffness. This method is based on the intuitive criterion that additional damper is placed sequentially on the storey with the maximum inter-storey drift and as a result the damper is optimally installed if placed at a position of largest relative displacement across the damper. This
approach was further developed by Shukla et al., (1999), whose study included investigation of the effect of earthquake excitation frequencies content.

Tsuji and Nakamura (1996) developed algorithms to reduce objective functions step by step. According Gurgoz et al., (1992) optimal placement of viscous dampers should be based on an energy criterion. Ashour et al., (1987) suggested that the optimal distribution of the supplemental devices should be at locations, which maximise the damping ratio of the fundamental mode, because this mode contribution to the structure’s response is the most significant. Natke (1993) carried out topological optimization to obtain optimal layout of the structure equipped with VE dampers. In a study conducted by Pong, (1994) a finite element model for fluid dampers was introduced and used in the analysis of a ten-storey frame. The results showed that the addition of fluid dampers to the first floor of a building absorb more energy than those at the upper floors.

Kasai and Watanabe (1998) proposed a simplified theory to predict and compare the seismic performance of VE and elastoplastic passive dampers. Closed-form expressions for an equivalent period and an equivalent damping constant for these systems were suggested by idealizing them as linear SDOF systems. Consequently, the authors extended this theory to design of MDOF VE and elastoplastic systems. Singh and Moreshi (2002) used a generic algorithm to determine the optimal size and location of viscous and VE dampers, defining different forms of performance functions. Yang et al., (2002) proposed two optimal design methodologies for passive energy dissipation devices based on active control theories leading to the determination of the optimal damper placements and their capacities.

Hanson et al., (1993) studied influence of placement of supplemental friction dampers on seismic performance of structures. They suggested that the distribution of the friction load should be similar to the storey stiffness. As a result, such distribution should significantly reduce the structural displacement. However, such distribution can be less efficient in reducing the acceleration near the top of the structure. The optimal distribution of friction dampers was obtained
from a nonlinear dynamic analysis (Filiatrault et al., 1990), while a performance index to calculate response reduction was defined as a function of the strain energy of the undamped and damped structural systems.

Henry et al., (2001) introduced a new method to simultaneously determine the number of necessary dampers and the optimal placements and design of these dampers needed to retrofit existing structures. He suggested starting with assumption that dampers are placed between every storey of the structure. Dampers are then eliminated in parts on their gains and their effect on the building’s response. Dampers with smaller gains are eliminated first, as their effect on the building’s response is insignificant. Dampers with large gains are eliminated according to the building’s response with and without those dampers.

A design method for metallic yielding dampers ADAS devices by utilizing optimal control theory was developed by Ribakov et al., (1999). In this work the optimal properties of ADAS devices were proposed to be obtained by assuming equal dissipation in viscous and elasto-plastic actions of the linear viscous and nonlinear ADAS devices, respectively. It was shown that this approach yields similar behaviour of a structure with optimal viscous dampers and ADAS devices.

The design of friction damped frames was studied by group of researchers at University of California at Berkeley (Bhatti et al., 1981, Austin et al., 1985). The analysing was formulated as a constrained optimization problem and nonlinear technique was applied to define the best results for various objective functions. An 11-storey steel frame building structure was used as an optimal design example. The building had been designed with 120 supplemental dampers. 24 dampers were placed on the first storey, and number of damper was gradually decreasing as the storey height increased. If the performance index was selected as optimal storey drift, the first storey mass contributed less to the dominant mode of the system response. Better results were obtained when most of the dampers from the first storey were removed, and added to the 6th to 11th storeys. With this optimised distribution, the total system damping was increased by 4%.
Furthermore, the damping ratio of the dominant mode was increased by 19%, and the largest storey drift, which occurred in the 6th floor, was reduced by an average of 11% for a series of spectra-compatible earthquake. However, if the performance index was selected as acceleration, the optimal distribution would be different.

The research undertaken by Madsen et al., (2001) was concentrated on using dampers within tall buildings that contain shear walls to enhance their seismic response. This new method of retrofitting buildings involves the use of VE dampers located within the shear wall of the building structure. According to results of this project, it was shown that it is more effective to place VE dampers in the lowest storeys. The theory behind this being the rigid and highly damped lower part of a multi-storey building modulates the dynamic excitation resulting from strong ground motions more effectively. This has also the effect of reducing the stiffness at the base of the structure, increasing the natural period and so reducing the amount of seismic energy that is attracted to it. Therefore, the most effective position for the dampers is in the lower storeys.

Due to some practical problems, there is only very limited number of publications concentrated of effective utilizing of combined energy dissipation systems. Nevertheless, some studies are very promising. Hisano et al., (2002) studied and analysed the effects of vibration reduction control by combined hysteretic-viscous damping system on high-rise building. The effects of vibration reduction were analysed in two different cases. In one case, the two different types of dampers (displacement dependant hysteretic lead damper and velocity dependant viscous damper) were used independently. In the other case, the combined damping system, which involves both types of the dampers, was applied to high-rise building.

In the planning of the structure, firstly, the number of dampers, which could absorb a design energy earthquake vibration in the viscous damper, was obtained. Subsequently, some of the viscous dampers were replaced with the hysteretic dampers, while the total number of the dampers remained unchanged. The number
of the combined dampers was determined by considering the effects of lead dampers on the periodic characteristics of the building. The internal damping of the building was assumed to be proportional to the instantaneous stiffness of $h = 2\%$. The building equipped with viscous dampers, lead dampers and the hybrid damping system respectively, was examined under the Hachinohe and El Centro earthquake excitations. The results from the analysis showed that the deflection and acceleration responses obtained from the combined damping system were somewhere between those obtained from the case where only viscous damper was used independently and those obtained from the case where only lead dampers were used independently.

A 12-storey concrete building was considered for retrofit by a number of engineers. Shao and Miyamoto (Shao et al., 1999), who were also involved in the study, suggested that passive dampers could be the most cost-effective solution. During preliminary study several damping systems were selected and studied for seismic retrofit. Linear and non-linear time history analyses were performed. Performance comparisons of earthquake response parameters were analysed. The results of this study revealed that the best performance was achieved by the combination of nonlinear viscous dampers with supplemental friction devices. These systems met the performance target with great saving over the previously proposed retrofit schemes. Friction damping system had significant saving over the viscous damping system due to the damper unit price difference. In contrast, viscous dampers with supplemental friction dampers would have 25% lower floor acceleration responses over friction damping system. These higher floor acceleration responses could increase the cost of the tile wall strengthening. Based on these results the authors suggested that combination of a nonlinear viscous damper system with friction damper revealed great potential for the further seismic retrofit.

All these studies have made some contributions to optimal use of energy dissipation devices. However, none of them address the performance requirements sufficiently so that engineers can design an optimal damper placement based on particular performance index of a particular building.
2.7 Configurations of passive energy dissipation devices

As it can be seen from previous sections, a number of energy dissipating devices have been employed to provide enhanced protection for new and retrofit buildings. These devices, which are made of varieties of materials and utilize different damping mechanisms, are generally installed in diagonal or chevron brace configurations. In recent years, several new configurations, which amplify damper displacements or velocities was developed. These new configurations offer certain advantages, either in terms of cost of the energy dissipation devices, or in terms of architectural considerations such as open space requirements. In the case of stiff structures treated under seismic or wind loads, structure experience only small displacement, while the required damping forces are large. To address this larger damping devices were installed, which resulted in increased cost. In addition, energy dissipation devices of diagonal or chevron brace configuration, in many cases, cannot be installed in certain areas due to open space requirements.

To amplify displacement and hence lower input force demand in energy dissipation devices the team of researchers at Buffalo University developed several new configurations, which utilize innovative mechanisms. These new configurations include the toggle-brace and the scissor-jack energy dissipation system. These configurations are more complex in their application since they require more care in their analysis and detailing.

The theory and development of these systems has been described in Constantinou et al. (2001) and Constantinou and Sigaher (2000). The following is a brief description of these new configurations and comparison with the typical chevron brace and diagonal configurations. The toggle-brace and scissor-jack are systems, which magnify the damper force through shallow truss configurations and consequently release the magnified force to the structural frame. Fig. 2.27 illustrates various damper configurations in a framing system and expressions for the magnification factors $f$. 
It can be seen that:

\[ u_D = f \cdot u \quad (2.4) \]

\[ F = f \cdot F_D \quad (2.5) \]

where, \( u_D \) is damper relative displacement, \( u \) is interstorey drift, \( F \) is force exerted on the frame, \( F_D \) is force along the axis of the damper and \( f \) is magnification factor. The significance of the magnification factor can be clearly demonstrated in the case of linear viscous dampers, for which:

\[ F_D = C_0 \dot{u}_D \quad (2.6) \]

where, \( C_0 \) is damping of the viscous damper, \( \dot{u}_D \) is relative velocity between the ends of the damper. The damping ratio under elastic conditions for a single-storey frame is:

\[ \beta = \frac{C_0 f^2 g T}{4 \pi W} \quad (2.7) \]

where, \( W \) is weight of the frame and \( T \) is fundamental period. This means, the damping ratio is proportional to the square of the magnification factor. As results, the viscous damper in \textit{toggle-brace} and \textit{scissor-jack} configuration can achieve magnification factors \( f = 2 \) to 3 without any significant sensitivity to changes in the geometry of the system. On the other hand, the conventional chevron-brace and diagonal configurations have magnification factors less than or equal to unity.

The formulas of the magnification factors introduced in Fig. 2.27 were confirmed by physical testing in laboratories at the Buffalo University. Example of
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...instalment of energy dissipating device with lower toggle brace assembly in structural frame is shown in Fig. 2.28.

Fig. 2.28 Application of the damper with lower toggle braced assembly
(Based on Taylor Devices Inc., 2001)

The numerical examples of the upper and lower toggle configurations were conducted by Marcelin (2003) on a 1-storey structure equipped with an upper and lower toggle viscous dampers. The results from this parametric study showed that the toggle brace configurations are capable of reducing both the maximum and root mean squared displacement of the buildings response. The reduction in root mean squared displacement for the toggle brace system was 34 %, while only 12% reduction for the chevron system. Similarly for the maximum displacement, when the chevron and diagonal system had a reduction of 20% and 23% respectively, as compared to 42 % and 47% for the lower and upper toggle brace system.

In addition, similar trends were noted for the velocities, when yet again the toggle systems were about twice as effective as the conventional configurations. The results from this study revealed that the conventional diagonal and chevron configuration are capable of reducing the building response; nevertheless, the toggle brace damper configurations are evidently more efficient energy dissipation systems.
Hata et al., (2001) proposed and developed another type of the toggle damping mechanisms. The main object of his experimental work was to develop toggle damping mechanism, which is a kind of amplifier device with lever mechanisms, damper and auxiliary masses. The proposed model (Fig.2.29) was a portal frame mounted on a unidirectional shaking table incorporating an obtuse-type toggle mechanism having viscous dampers. The model was composed of a frame whose columns were made of flat springs to confine the deformation in the in-plane directions. The lower arms of the toggle mechanism were connected to the frame with uni-axial pins so that the auxiliary mass could not vibrate in the out-of-plane directions. The pins of the upper arms were composed of universal joints to absorb the out-of-plane vibration and torsional vibration of the main frame mass and installation accuracy errors.

The model was subjected to loading with sinusoidal waves and input earthquake waves. A comparison between the acceleration response ratios of this model and a normal single-degree-of-freedom viscous damping system showed that the resonance magnification of the model by shown experiment was around 1.5 times, which corresponds to a damping of approximately 40% when evaluated as a normal single-degree-of-freedom system. The experiment results agreed well with the theoretical magnification and input-reducing effects. As a result, these small
dampers were considered to have a significant potential of providing large damping for buildings.

Gluck (1996) proposed the utilizing of lever mechanisms to improve the damping characteristics of a structure by magnifying the drifts and drifts velocities, which are transferred to the dampers. Ribakov et al., (2000, 2003) further developed a method for design of passively controlled structures using optimal control and viscous dampers with the dampers connected to the structure through lever arms. These lever arm system was then used to magnify efficiency of damping devices. The system consisted of a chevron brace, a lever arm connected by means of hinges and to the storey diaphragm and the chevron brace, respectively. A connection of chevron brace and lever arm was realized through steel plates. At the time, when the structure experienced a horizontal drift at the top storey, the lever arm rotated around hinge, and as a result a magnified displacement and drift velocity were transferred to the damper placed at the bottom of the lever.

Ribakov et al., (2003) conducted a numerical simulation of 7-storey steel structure equipped with this damping system under varieties of seismic excitations. Reduction in peak displacement in the range from 40 to 75% compared to the uncontrolled structure was achieved with this damping system. A proposed damping system yields significantly higher energy dissipation in the structural system with smaller devices. Even more, the large damping without changes in structural stiffness can be very valuable for retrofit of existing structures (Ribakov et al., 2003). In order to reduce the control forces and energy required for control, they proposed also magnifying configurations for connecting semiactive and active dampers (Gluck and Ribakov 2000, Ribakov 2000).
2.8 Conclusions to the literature review

2.8.1 Summary of the literature review

This literature review looked at the background of seismic activity and seismic effect on building structures. It briefly described the analysis methods currently in use and assumes their suitability. Seismic mitigation principles are discussed next, followed by a description of a number of available passive energy dissipation devices using a range of material and damping mechanisms that work on principles such as frictional sliding, yielding of metals, shear deformation of VE materials, shearing VE fluid or flow of viscous liquid through orifices. The reviewing was followed by recent research on seismic mitigation using passive dampers installed mostly in lower–rise structures. A number of strategies and recommendations of effective use of passive dampers, as well as numerical and experimental results is described. Finally, some of the possible configurations of dampers are presented. The vast majority of applications of damping devices presented in this literature review was realised within frame structures, whereas the research of damping devices placed within cut outs of shear walls is still very limited.

2.8.2 Proposed research

The main aim of this research project is to bridge existing knowledge gaps by undertaking comprehensive investigation of several high and medium-rise structures with different damping devices embedded within cut outs of shear walls. To further extend understanding of damping devices embedded within cut outs of shear walls, these structures are treated under a variety of different earthquake excitations and the results are compared in order to capture their advantages in creating efficient damping systems. The research will address the needs of local industries. It aims to carry out a comprehensive investigation on seismic mitigation of high and medium-rise structures with different damping devices embedded within cut outs of shear walls at different locations across the height of each structure. The results from this research are presented in the next chapters.
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Chapter 3

Model Development
Chapter 3: Model Development

3.1 Introduction

This study comprehensively investigates the seismic response of 18 and 12 storey frame-shear wall structures with dampers located within cut-outs. Three types of damping mechanisms were investigated. The first damping mechanism involves the use of friction damper, the second damping mechanism involves the use of VE damper and the third damping mechanism involves the use of a hybrid system consisting of friction and VE damper. The use of both components of the hybrid damper, namely the displacement dependant friction part, which has the capacity to manage large deformation, and the velocity dependant VE part, which has the capacity to manage even small deformation can allow effective control of the building’s vibration response. In order to further validate the feasibility of the procedure used in this study, three additional structural models, namely 24-storey frame-shear wall structural models, 96m high shear wall models and 18-storey frame models with embedded damping systems have also been treated.

3.2 Finite element analysis

Finite Element (FE) methods have been employed in this research to model, analyse and investigate the effects of the six types of damping devices on the seismic response of structures. For the purpose of this study, the program selected for the numerical analysis has been ABAQUS Standard Version 6.3. In conjunction with this computer program, MSC/PATRAN 2003 has been used as the pre-processor for generating the geometry, element mesh, boundary conditions and loading conditions of the model, and as the post-processor for viewing the results of the analysis.

A direct integration dynamic analysis was selected to obtain the response of the structure under seismic loading. This analysis assembles the mass, stiffness and damping matrices and solves the equations of dynamic equilibrium at each point in time. The response of the structure is obtained for selected time steps of the input earthquake accelerogram. The dynamic procedure in ABAQUS/Standard
uses implicit time integration. To study the effectiveness of the damping system in mitigating the seismic response of the buildings in this study, the maximum displacements and accelerations at the top of the structures are obtained from the results of the analysis and compared with those of the undamped building structure.

3.3 Loading and boundary conditions.

3.3.1 Boundary conditions

The earthquake events used in this study were recorded as time-history accelerations in the horizontal plane. The acceleration was applied in the x-direction at the base of the structure, as shown in Figure 3.1. The support at the base of the structure was restrained against translation in the y-direction, and rotation about the z-axis, thereby allowing only the x-direction translation.

![Fig. 3.1 - Model location of applied acceleration](image)

3.3.2 Material properties

Concrete material properties were chosen for the models since many multi-storey buildings in Australia are constructed by using reinforced concrete. The concrete had a compressive strength, $f'_c$, of 32 MPa, Young’s modulus, $E_c$ of 30,000 MPa, which reflects an assessment assuming predominantly elastic response with little cracking, Poisson’s ratio, $\nu$ of 0.2, and density, $\rho$ of 2500 kg/m$^3$. No internal
damping was considered for the concrete since it was assumed small in relation to the damping provided by the damping devices. Structural steel was used to model friction dampers and hybrid dampers with yield strength, \( f'\) of 350 MPa, and Young’s modulus, \( E_c \) of 207,000 MPa, Poisson’s ratio \( \nu \) of 0.3 and density, \( \rho \) of 7700 kg/m³.

3.4 Damper models

3.4.1 Friction damper model

The first damping mechanisms employed in this study have been represented by friction dampers. The initial focus of this research was on the development of a model, which represents the real behaviour of friction dampers. This task was achieved by modelling the frictional contact between two tubes, which slide one inside the other. Some structural details are explained later in this chapter.

The extended version of the classical isotropic Coulomb friction model is provided in the computer program ABAQUS, (the program available to the author) for use with all contact analyses. In the basic form of the Coulomb friction model, two contacting surfaces can carry shear stresses up to certain magnitude across their interface before they start sliding relative to one another. In two-dimensional contact problems, the direction of frictional slip must lie in the plane, and hence, there are only two options: slip to the right or left. The contact problem is therefore in linear range, since all the states are governed by linear equations and nonlinearity is introduced only through the inequalities that trigger changes of state.

The Coulomb friction model assumes that no relative motion occurs if the equivalent frictional stress is less than the critical stress, which is proportional to the contact pressure in the form:

\[
\tau_{crit} = \mu \cdot p
\]

where, \( \mu \), is the coefficient of friction, and \( p \), is the contact pressure.
3.4.2 Viscoelastic damper model

The second damping mechanisms employed in this study are represented by *VE dampers*. Dampers are modelled as a linear spring and dash-pot in parallel (known as the Kelvin model) where the spring represents stiffness and the dashpot represents damping. Abbas & Kelly (1993) define the stiffness and damping coefficients as follows:

\[
k_d = \frac{G' A}{t} \quad (3.2)
\]

\[
C_d = \frac{G^* A}{\omega t} \quad (3.3)
\]

where, \( A \) is the shear area of the VE material, \( t \) is the thickness of the VE material, \( \omega \) is the loading frequency of the VE damper, \( G' \) is the shear storage modulus, and \( G^* \) is the shear loss modulus. The following equations were used to obtain the moduli of the VE material as defined by Abbas and Kelly (1993):

\[
G' = 16.0 \omega^{0.51} \gamma^{-0.23} e^{(72.46/Temp)} \quad (3.4)
\]

\[
G^* = 18.5 \omega^{0.51} \gamma^{-0.20} e^{(73.89/Temp)} \quad (3.5)
\]

where, \( \gamma \) is the shear strain.

This model approximates the behaviour of a VE damper under vibratory loading to within 10%, which was considered sufficiently accurate for the purposes of this study. As it can be seen from equations 3.5 and 3.6, the temperature of VE material significantly influences its mechanical characteristics. However, experimental results conducted by Chang et al., (1992) have shown that variation in the damper temperature due to dynamic excitation become negligible after several loading cycles as an equilibrium temperature is reached between the surroundings and the damper. In the present study the temperature was kept constant at 21°C during the entire investigation.

3.4.3 Hybrid friction-viscoelastic damper model

The third damping mechanisms employed in this study is represented by *hybrid dampers* consisting of both a VE and a friction damper model in series.
3.5 18-storey frame-shear wall structure.

3.5.1 Description of 18-storey frame-shear wall structure

The structural models, treated in this thesis have been predominantly represented by two types of frame-shear wall structures. The first set of models designated by H0 represents two-dimensional 18-storey frame-shear wall structures. After the preliminary convergence study, the concrete shear walls were constructed from 2016 S4R5 shell elements using shell elements of designation S4R5, having 4 nodes per element and 5 degrees of freedom at each node. The dimensions of the shear walls were 6m wide and 0.4 m thick. The columns and beams were located on either side of the wall, as seen in Fig. 3.2 and had cross-sectional dimensions of 0.75 x 0.75 m and 0.75 x 0.45 m respectively, and the beam spans were 6.0 m. The height between storeys was set at 4.0 m, which made the overall height of the structures to be 72.0 m. A lumped mass of 20,000 kg at each beam-column and beam-shear wall junction was used to account for mass transferred from slabs and beams.

![Fig. 3.2 - 18-storey frame-shear wall structure](image)

3.5.2 Damper placement in 18-storey frame-shear wall structure

One of the main aims of this study was to investigate the efficiency of energy dissipating dampers in vibration control for variety of placements under different earthquake loads. For this purpose twelve different damper placements were used to study the influence of location on the seismic response of these models. These
models were designated by H1, H3, H5, H7, H9 and H11 for single damper placements and by H1-3, H4-6, H7-9, H10-12, H13-15 and H16-18 for three dampers placement within the models. As can be seen in Figs. 3.3 and 3.4, the designating numbers correspond to location of the storey at which dampers were placed. The undamped structure (Fig. 3.2) was also analysed in order to compare results.

3.6 12-storey frame-shear wall structure

3.6.1 Description of 12-storey frame-shear wall structure

The second set of models was represented by 12-storey frame-shear wall structure designated by M0 (Fig. 3.5). The shear walls were constructed from 1344 S4R5 shell elements of the same parameters as in the previous models, and columns and beams with cross-sectional dimensions reduced to 0.6x0.6 m and 0.6x0.45 m respectively. The spans of the beams were 6.0 m and the height between storeys
was 4.0 m gave overall height 48.0 m. Lumped masses of 20,000 kg at each beam-column junction of the frames were used as before.

![Fig.3.5 - 12-storey frame-shear wall structure](image)

### 3.6.2 Damper placement in 12-storey frame-shear wall structure

In the 12-storey structural models twelve different damper placements were used to study the influence of location on the seismic response. These placements are designated by M1, M3, M5, M7, M9 and M11 for single damper placement (Fig.3.6) and by M1-2, M3-4, M5-6, M7-8, M9-10 and M11-12 for placement of two dampers within these models (Fig. 3.7).

![Fig. 3.6 - Placements of single damper within 12-storey frame-shear wall structures](image)

![Fig. 3.7 - Placements of two dampers within 12-storey frame-shear wall structures](image)
3.7 Shear wall structure

In order to further demonstrate the feasibility of this study and if necessary extend the technique to taller structures three additional types of structural models with embedded dampers were also investigated. The first of these structures chosen to test the feasibility of this technique was the shear wall designated W0, modelled using 2560 two-dimensional shell elements. The dimensions of the shear wall were 96 m high, 15 m wide and 0.5 m thick. A total of four different damping placements were used to study the influence of location on the seismic response of these models. These were designated by W1, W2, W3 and W1-3 in which the damper was placed in the lower, middle, upper and all three parts of the structure, respectively, as shown in Fig. 3.8.

![Fig. 3.8 - Placement of dampers within shear wall structures](image)

3.8 24-storey frame-shear wall structures.

The second structural types were represented by 24-storey frame-shear wall structures of designation T0. The shear walls were modelled as before, while the columns and beams of the frame had cross-sectional dimensions of 0.9 x 0.9 m and 0.9 x 0.45 m respectively, and the spans were 8 m, as shown in Fig. 3.9. Lumped masses at each beam-column junction of the frames to account for mass transferred from slabs were 25,000 kg. Placement of dampers and cut-out details were as in the study with shear wall models.
3.9 18-storey frame structures

The third structural types investigated in this study were represented by 18-storey frame structures designated by F (Figs. 3.10, 3.11), where the columns and beams of the frame had cross-sectional dimensions of 0.75 x 0.75 m and 0.75 x 0.45 m respectively, and the spans were 6 m. The lumped masses at each beam-column junction were 20,000 kg. Twelve different damper placements were used. These placements are designated by F1, F4, F7, F10, F13 and F16 for single damper placement (Fig. 3.10) and by F1-3, F4-6, F7-9, F10-12, F13-15 and F16-18 for placement of three dampers within these models (Fig. 3.11).
3.10 Dampers within 18 and 12-storey frame-shear wall structures

Fig. 3.12 shows a six different damping systems, which were primarily proposed and studied for medium-rise structures designated by H (18-storey) and M (12-storey). Seismic analyses of these frame-shear wall structures were carried out with one type of damping system at a time. Four different configurations of the dampers namely, diagonal, chevron brace, lower toggle and a hybrid configuration consisting of the friction damper oriented horizontally and the VE damper mounted diagonally were considered. Furthermore, twelve different damper placements were used in each set of models to study the influence of location on the seismic response of these models. The undamped structure was also analysed in order to compare results.
3.10.1 Structural model with friction damper – diagonal configuration

Details of the diagonal friction damper located within shear wall of the frame-shear wall model can be seen in Fig. 3.13 where a 3.5 m wide by 3.5 m high wall section was cut out and replaced by the damper. This damper was modelled as a pair of diagonal tubes each with a thickness of 50 mm, and with one tube placed within the other.

- The outer tube having an inner diameter of 200 mm and length 3.75 m was modelled using 264 S4R5 shell elements.
- The inner tube having an outer diameter of 198 mm and length 3.75 m was modelled using 252 S4R5 shell elements.

The radial clearance between the tubes was 1mm, the contact area in the unloaded state was 3.71 m² and the coefficient of friction between the tubes was assumed to be 0.25. The connection between each tube and the shear wall was modelled using a MPC (Multi-Point Constraint) Pin type connecting element, which provides a pinned joint between two nodes. This MPC makes the displacement of the two nodes equal but allows the differential rotations, if these exist, independent of each other. A MPC Slider type connecting element was chosen to ensure frictional sliding between the tubes in a determined direction. This MPC keeps a node on a straight line defined by two other nodes such that the node can move along the line, and the line can also change length.

![Fig. 3.13 - Structural details of diagonal friction damper](image-url)
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Fig. 3.14 shows the details of the MPC connection between the damper and shear wall in the computer model.

The response of this model as well as all others were investigated under the El Centro, Hachinohe, Kobe, Northridge and San Fernando earthquake excitations described later in this chapter.

3.10.2 Structural models with VE damper – diagonal configuration

The concrete frame-shear wall was modelled using the same FE mesh, material properties and dimensions as in the previous models. Detail of the diagonal VE damper located within the cut out of the shear wall can be seen in Fig. 3.15. The properties of the damper for 18-storeys models were at first calculated as \( k_d = 10 \times 10^6 \text{ N/m} \) and \( c_d = 63 \times 10^6 \text{ Ns/m} \) based on double layer damper in parallel with dimensions of 1,850 mm by 300 mm by 10 mm and the values \( G' = 900,000 \text{ Pa} \) and \( G'' = 300,000 \text{ Pa} \). These moduli were calculated using the loading frequency \( f = 0.614 \text{ Hz} \), which corresponded to the fundamental frequency of this structure model. In a similar manner, damping properties of VE dampers located in the 12-storeys models (with \( f = 1.05 \text{ Hz} \)), were calculated. The values for this structure had \( k_d = 10 \times 10^6 \text{ N/m} \) and \( c_d = 38 \times 10^6 \text{ Ns/m} \) with dimensions of 1,670 mm by 300 mm by 10 mm and the values \( G' = 950,000 \text{ Pa} \) and \( G'' = 450,000 \text{ Pa} \). The results for both structure were evaluated and in order to facilitate comparisons,
The approximate average values of $k_d = 10 \times 10^6$ N/m and $c_d = 50 \times 10^6$ Ns/m, were used in all the subsequent analyses.

**Fig. 3.15 - Structural details of diagonal VE damper**

### 3.10.3 Structural model with friction damper – chevron brace configuration

The concrete frame-shear wall model was created using the same FE mesh, material properties and dimensions as before. The only difference was in the size of the cut out, which was reduced to 3.5 m wide by 2.5 m high. Fig. 3.16 shows the details of the frame-shear wall model with a friction damper of chevron brace configuration.

The friction damper was modelled as a pair of horizontal tubes, where one tube was placed within the other. Based on the geometric orientation of the chevron bracing system, the relative displacement along the direction of the damper was equal to the inter-storey drift of the floors between which it was connected. The ratio between inter-storey drift and relative damper displacement was related through the magnification factors. In the case of the chevron brace, the value of the magnification factor $f = 1$, whereas in the case of the diagonal brace the magnification factor is $f = \cos \theta$, where $\theta$ is the damper angle to the horizontal.
In order to create the chevron brace damper with approximately similar damping force to that of the diagonal dampers, the chevron brace friction dampers was constructed as described below.

- The outer tube was constructed from 264 S4R5 shell elements, the inner diameter of this tube was 200 mm and its length was 2.8 m.
- The inner tube was constructed from 276 S4R5 shell elements, the outer diameter of this tube was 198 mm and its length was 2.85 m.

The thickness of both tubes was 50 mm, the radial clearance between the tubes was 1 mm and the contact area in the unloaded state was 2.49 m², which represents 67% of the contact area for diagonal friction damper. The coefficient of friction between tubes was assumed to be 0.25. The connection between each tube and the shear wall was modelled using a MPC Pin type connecting element, and a MPC Slider type connecting element was chosen to ensure frictional sliding between the tubes in a determined direction.

### 3.10.4 Structural model with VE damper – chevron brace configuration

The concrete frame-shear wall model was created as in the previous case. The details of the damper placed within the shear wall are shown in Fig. 3.17. The damper was oriented horizontally in the upper part of the cut out, attached at one end directly to the left side of the shear wall and attached at the other end to the upper edge of the shear wall via an MPC Rigid connection. While comparing to
diagonal VE dampers, the values of spring and dashpot remained the same, whereas the overall length was reduced to 67% of length for diagonal VE damper.

3.10.5 Structural model with hybrid friction-VE damper

The concrete frame-shear wall models were modelled using the FE mesh, material properties and dimensions as before. The friction component of the hybrid friction-VE damper was modelled as a pair of horizontal tubes, with one tube placed within the other.

- The outer tube was constructed from 384 S4R5 shell elements, the inner diameter of this tube was 200 mm and its length was 1.500 m.
- The inner tube was constructed from 155 S4R5 shell elements, the outer diameter of this tube was 198 mm and its length was 1.485 m.

The thickness of both tubes was 50 mm, the radial clearance between the tubes was 1 mm and the contact area in the unloaded state was 1.67 m², which represents 67% of the contact area for chevron brace friction dampers. The coefficient of friction between the tubes was 0.25. The direction of frictional sliding was determined by Slider type MPCs, as shown in Fig. 3.18.

The VE part of the hybrid damper, which represented both spring and dashpot elements was oriented with one end attached to a brace assembly placed in the
middle of the upper edge of the cut out, and the other end attached to the lower left-hand corner of the cut out. This oriented the damper at 45° to the horizontal, while its length was 2.475 m, which represents 50% of length for the diagonal VE damper. The values of damping and stiffness were kept the same as in the previous models.

3.10.6 Structural model with VE damper – lower toggle configuration

The concrete frame-shear wall models were created using the FE mesh, material properties and dimensions as before. The only difference was in the size of the cut out, which was enlarged to 3.5 m wide by 3.0 m high. Detail of the lower toggle VE damper located within the cut out of the frame-shear wall model can be seen in Fig. 3.19.

As it was mentioned previously the magnification factor for chevron brace represents $f = 1$ and for the diagonal bracing $f = \cos \theta$, where $\theta$ is the angle of damper to the horizontal. In the case of the lower toggle configuration, the effect of the magnification factor on the damping force is notably higher. The reason is because in the toggle configuration the relative displacement of the dampers is 2-3 times the interstorey drift, which basically means that a magnification factor is more than two times greater than unity.
Based on this, the lower toggle VE damper was created with its length of 2.250 m, which represent 45% of the length of diagonal VE damper and the parameters of spring and dashpot were kept the same as previously. The damper was oriented at $45^\circ$ to the horizontal and had one end attached to the lower arm of the steel holder and the other end attached to the lower right-hand corner of the cut out. The arms of the brace assembly are created from 100 x 5 SHS, which are connected to each other by MPC Pin and the connection to the shear wall was realised by 6 mm pre-bent plate.

All these structural models have natural frequencies, which match those of typical high rise buildings and hence enable a simple conceptual study for evaluating seismic mitigation.

### 3.11 Dampers installed within 96 m high shear wall structures

Five different damping systems shown in Fig. 3.20 were investigated for shear wall structure designated by W. Seismic analyses of the shear walls were once again carried out with one type of damping system at a time. Three different configurations of the friction and VE dampers namely, diagonal, chevron brace and a hybrid configuration consisting of the friction damper oriented horizontally and the VE damper mounted diagonally were considered. Furthermore, four different damper placements were used to study the influence of location on the
seismic response of these models. The undamped structure was also analysed in order to compare results.

3.11.1 Structural models with friction damper – diagonal configuration

Details of the damper located within the shear wall can be seen in Fig. 3.21, where a 12.0 m wide by 12.46 m high wall section was cut out and replaced by a diagonal friction damper.

Fig. 3.20 - Damping systems installed in shear wall structure

Fig. 3.21 - Structural details of diagonal friction damper within structure type W
The damper was modelled as a pair of diagonal tubes each with a thickness of 50 mm, and with one tube placed within the other.

- The outer tube having an inner diameter of 200 mm and length 14.5 m was modelled using 239 S4R5 shell elements.
- The inner tube having an outer diameter of 198 mm and length 15 m was modelled using 263 S4R5 shell elements.

The radial clearance between the tubes was 1 mm, the contact area in the unloaded state was 16.4 m² and the coefficient of friction between the tubes was 0.25. The connection between each tube and the shear wall as well as the direction of frictional sliding was determined by Slider and Pin type MPCs.

### 3.11.2 Structural model with VE damper – diagonal configuration

Detail of the diagonal VE damper located within the cut out of the shear wall can be seen in Fig. 3.22.

![Fig. 3.22 - Structural details of diagonal VE damper within structure type W](image)

The properties of the damper for model type W1 were at first calculated as $k_d = 80 \times 10^6$ N/m and $c_d = 107 \times 10^6$ Ns/m based on double layer damper in parallel with dimensions of 1,530 mm by 300 mm by 10 mm and the values $G' = 872,647$ Pa and $G'' = 1,240,081$ Pa. These moduli were calculated using the loading frequency.
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\[ f = 0.531 \text{ Hz}, \] which corresponded to the fundamental frequency of this structure. In a similar manner, damping properties of VE dampers located in the structure type W2 (with \( f = 0.598 \text{ Hz} \)), W3 (with \( f = 0.735 \text{ Hz} \)), and W1-3 (with \( f = 0.536 \text{ Hz} \)) were calculated. The seismic responses of the structures were noticeably close when \( c_d \) was within the range \( 40 \times 10^6 \) to \( 140 \times 10^6 \text{ Ns/m} \) and \( k_d \) within the range \( 30 \times 10^6 \) to \( 120 \times 10^6 \text{ N/m} \) hence in order to facilitate comparisons, approximate average values of \( k_d = 100 \times 10^6 \text{ N/m} \) and \( c_d = 100 \times 10^6 \text{ Ns/m} \), respectively, were determined and used in all the subsequent analyses for the structures type W.

### 3.11.3 Structural models with friction damper – chevron brace configuration

The concrete shear wall was modelled using the same FE mesh, material properties and dimensions as before. The only difference was in the size of the cut out, which was reduced to 12.0 m wide by 8.0 m high. The friction damper showed in Fig. 3.23 was modelled as a pair of horizontal tubes, where one tube is placed within the other.

![Structural details of chevron brace friction damper within structure type W](image)

- The outer tube was constructed from 311 S4R5 shell elements, the inner diameter of this tube was 200 mm and its length was 11.125 m.
- The inner tube was constructed from 287 S4R5 shell elements, the outer diameter of this tube was 198 mm and its length was 11.000 m.
The thickness of both tubes was 50 mm, the radial clearance between the tubes was 1 mm, the contact area in the unloaded state was 13.2 m² and the coefficient of friction between them was determined to be 0.25. The connection between each tube and the shear wall was modelled using a MPC Pin type connecting element, and a MPC Slider type connecting element was chosen to ensure frictional sliding between the tubes in a determined direction. The details of the MPC connection between the damper and shear wall in the computer model are also shown in Fig. 3.23.

### 3.11.4 Structural model with VE damper – chevron brace configuration

The concrete shear wall was modelled as in the previous case. The parameters of the damper are presented in Fig. 3.24. The damper placed within the shear wall was oriented horizontally in the upper part of the cut out, attached at one end directly to the left side of the shear wall and attached at the other end to the upper edge of the shear wall via an MPC Rigid connection.

![Fig. 3.24 - Structural details of chevron brace VE damper within structure type W](image)

### 3.11.5 Structural model with hybrid friction-VE damper

The concrete shear wall was modelled using the same FE mesh, material properties and dimensions as before. The friction component of the hybrid damper shown in Fig. 3.25 was modelled as a pair of horizontal tubes, with one tube
placed within the other. The material properties and dimensions, except for length, were the same as in the chevron brace friction damper. The contact area in the unloaded state was 5.4 m². The direction of frictional sliding was determined by Slider and Pin type MPCs.

The VE part of the hybrid damper, which represented both spring and dashpot elements was oriented with one end attached to a brace assembly placed in the middle of the upper edge of the cut out, and the other end attached to the lower left-hand corner of the cut out. This oriented the damper at 40° to the vertical, while its length was 9.0 m. Damping and stiffness were kept the same as in the diagonal VE dampers.

![Fig. 3.25 - Structural details of hybrid friction-VE damper within structure type W](image)

### 3.12 Dampers within 24-storey frame-shear wall structures

Details of the diagonal VE damper located within the cut out of the shear wall of 24-storey frame-shear wall structure designated T are shown in Fig. 3.26. The properties of the damper for model type T1 were at first calculated as $k_d = 50 \times 10^6$ N/m and $c_d = 37 \times 10^6$ Ns/m based on double layer damper in parallel with dimensions of 980 mm by 300 mm by 10 mm and the values $G' = 850,000$ Pa and $G'' = 250,000$ Pa. These moduli were calculated using the loading frequency $f = 0.401$ Hz, which corresponded to the fundamental frequency of the structure model type T1. In a similar manner, damping properties of VE dampers located in
the model type T1-3 (with $f = 0.370 \, \text{Hz}$), and T3 (with $f = 0.586 \, \text{Hz}$) were calculated. These frequencies would give varying values for $k_d$ and $C_d$ but in order to facilitate comparisons, approximate average values of $k_d = 50 \times 10^6 \, \text{N/m}$ and $c_d = 40 \times 10^6 \, \text{Ns/m}$ respectively, were determined and used in all the subsequent analyses for the structures type T.

3.13 Dampers within 18-storey frame structures

The concrete 18-storey frame structure designated by F0 was created using the same FE mesh, material properties as in the previous structures. The dimensions of this structure are described in chapter 3.9.1. Detail of the diagonal VE damper located within the frame can be seen in Fig. 3.27. The properties of this damper embedded within the 18-storey frame structure were at first calculated as $k_d = 20 \times 10^6 \, \text{N/m}$ and $c_d = 14 \times 10^6 \, \text{Ns/m}$ based on double layer damper in parallel with dimensions of 706 mm by 300 mm by 10 mm and the values $G' = 850,000 \, \text{Pa}$ and $G'' = 250,000 \, \text{Pa}$. These moduli were calculated using the loading frequency $f = 0.440 \, \text{Hz}$. The seismic responses of the structures were noticeably close when $c_d$ was within the range $10 \times 10^6$ to $60 \times 10^6 \, \text{Ns/m}$ and $k_d$ within the range $1 \times 10^6$ to $30 \times 10^6 \, \text{N/m}$ hence in order to facilitate comparisons, approximate average
Influence of Damping Systems on Building Structures Subject to Seismic Effects

values of \( c_d = 50 \times 10^6 \) Ns/m and \( k_d = 10 \times 10^6 \) N/m respectively, were determined and used in all the subsequent analyses for the structures type F.

![Fig. 3.27 - Structural details of diagonal VE damper within structure type F](image)

3.14 Input earthquake records

In general, earthquakes have different properties such as peak acceleration, duration of strong motion and ranges of dominant frequencies and therefore have different influences on the structure. In order to ensure that the chosen mitigation procedure is effective under different types of excitations, five, well-known earthquakes records were used in this study. These were all applied for the first 20 s of their duration, during which the strong motion took place.

For more consistent comparison, all earthquake records were scaled to peak ground acceleration (PGA) of 0.1 g for the taller structures of designation W and T and 0.15 g for the medium-rise structures of designation H and M, respectively. Duration of the strong motion and range of dominant frequencies were kept unchanged and were evaluated by Welch’s method (1967) based on Fast Fourier Transform Techniques, using the computer program MATLAB Version 6.3.

The earthquake records, which were selected to investigate the dynamic response of the models, are:
El Centro (1940) with duration of strong motion in the range of 1.5-5.5 secs and dominant frequencies in the range 0.39-6.39 Hz,

Hachinohe (1994) with duration of strong motion in the range of 3.5-7.5 secs and dominant frequencies in the range 0.19-2.19 Hz,

Kobe (1995) with duration of strong motion in the range of 7.5-12.5 secs and dominant frequencies in the range 0.29-1.12 Hz,
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- Northridge (1994) with duration of strong motion in the range of 3.5-8.0 secs and dominant frequencies in the range 0.14-1.07 Hz and

![Northridge earthquake record and its dominant frequencies](image)

- San Fernando (1971) with duration of strong motion in the range of 4.5-9.5 secs and dominant frequencies in the range 0.58-4.39 Hz.

![San Fernando earthquake record and its dominant frequencies](image)

3.15 Verification of results

At the present time, structural control research is generally greatly diversified in terms of specific applications and required objectives. A common guideline for the comparison of results from various algorithms and devices still does not exist. Preferably, each proposed control strategy should be verified by experimental testing under conditions that are close to the realistic physical structure. However, in the case of medium or high-rise structure it is unrealistic, both logistically and economically, to conduct appropriate experimental tests.
One of the available alternatives to examine the validity of research results is to use the analytical benchmark models proposed by the American Society of Civil Engineers (ASCE). A committee of ASCE on Structural Control developed a benchmark study, focusing on the comparisons of structural control algorithms for the benchmark structural control problems (Balas 1997, Lu and Skelton 1997, Smith, et al. 1997, Spencer, et al. 1997 Spencer, et al. 1998 and Wu, et al. 1997). Some of these algorithms have been experimentally confirmed at the Laboratory of University of Notre Dame's (Baker, et al. 1999). The primary objective of this project was to develop benchmark models to provide systematic and standardized means by which a variety of control methods can be examined. Realising of these objectives allow implementation of innovative control approaches for dynamic hazard mitigation.

The researchers from Faculty of Engineering, University of Technology, Sydney participated in this benchmark project and have published several experimental works, which were conducted in their University’s laboratories. Some of these testings were conducted on the five-storey benchmark model subjected to different earthquake excitations. In order to verify the validity of the present research project, similar model was created and treated under the same earthquake excitations in the computer program ABAQUS. The results are compared and evaluated with the results of experimental testing.

As it can be seen from previous sections of this chapter, six different damping systems were investigated in this study. These damping systems were installed in three different configurations, namely in traditional diagonal and chevron brace and also in a relatively new lower toggle configuration. Marcelin (2001), conducted analytical investigation, confirmed by experimental testing of all three configuration used in present project. Analytical results obtained with the computer program ABAQUS for all three damper configurations are also compared with those of Marcelin (2001) in the following sections.
3.15.1 Benchmark model testing

Shi Lily D., et al., (2000), from University of Technology, Sydney conducted experimental work and numerical analysis of several types of damping systems installed in 5-storey benchmark models. Some of these results are used in present study in order to confirm validity of this research. The building (Fig. 3.33) was modelled as a lumped mass system in which the mass of each storey was concentrated at each floor level and passive viscous dampers were installed in the 1st, 3rd and 5th storey.

The first natural frequencies of the two benchmark models without dampers for different storey masses were 5.5 Hz (BM5.5) and 7.5 Hz (BM5.5). The mass of each storey was \( m_i = 605 \text{ kg and 325 kg} \), respectively, corresponding to frequencies of 5.5 Hz and 7.5 Hz. The frame stiffness was \( K_f = 7.563 \times 10^6 \text{ N/m} \), \( K_2 = K_3 = K_4 = K_5 = 10.066 \times 10^6 \text{ N/m} \) and the stiffness of the damper was \( 1.26 \times 10^6 \text{ N/m} \). Three types of earthquake motions, namely the El Centro (1940) scaled to a PGA of 0.5 g, Kobe (1995) scaled to a PGA of 1.0 g and Northridge (1994) scaled to a PGA of 0.8 g were used as input earthquake excitations.

![Fig. 3.33 Elevation and plan of the five-storey benchmark model](Based on Shi Lily D. et al., 2000)
The damper used in this system consisted of a viscous damper connected to the bracing frame. The damper was expressed by the combination of damper stiffness ($k_d$) and bracing stiffness ($k_b$) joined in series. The equivalent stiffness of the entire device ($k_{hi}$) was defined as:

$$k_{hi} = \frac{k_d k_b}{k_d + k_b}$$  \hspace{1cm} (3.6)

The peak values of tip deflection and tip acceleration of experimental testing conducted on the BM7.5 and BM5.5 subjected to three earthquake excitations are presented in Table 3.1

<table>
<thead>
<tr>
<th>Control</th>
<th>Earthquake Type</th>
<th>BM7.5 Tip Deflection (mm)</th>
<th>BM5.5 Tip Deflection (mm)</th>
<th>BM7.5 Tip Accelerations (g)</th>
<th>BM5.5 Tip Accelerations (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Control</td>
<td>Kobe</td>
<td>7.70</td>
<td>17.90</td>
<td>1.22</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td>El Centro</td>
<td>9.30</td>
<td>13.60</td>
<td>1.88</td>
<td>1.38</td>
</tr>
<tr>
<td></td>
<td>Northridge</td>
<td>7.50</td>
<td>14.40</td>
<td>1.32</td>
<td>1.62</td>
</tr>
<tr>
<td>Passive Damping</td>
<td>Kobe</td>
<td>7.60</td>
<td>15.70</td>
<td>1.09</td>
<td>1.18</td>
</tr>
<tr>
<td>System</td>
<td>El Centro</td>
<td>9.60</td>
<td>12.90</td>
<td>2.10</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>Northridge</td>
<td>7.60</td>
<td>13.00</td>
<td>1.09</td>
<td>1.18</td>
</tr>
</tbody>
</table>

The comparison of responses for undamped benchmark model and benchmark model fitted with viscous dampers showed that dampers were able to reduce peak values of tip deflection on the top of the structure without increasing floor accelerations by much. However, deflection and acceleration reductions depend on the type of earthquake and structure. The dampers performed significantly better for building model BM5.5 than for BM7.5. Nevertheless, it has to be noted that tip acceleration and tip deflection of the damped models were in several cases higher then those of the undamped models. This trend was evident in some of the models treated in this thesis.

3.15.2 Results of analytical investigation of benchmark models

The analytical model created in computer program ABAQUS was constructed of steel. The material properties were compressive strength, $f_c'$ of 350 MPa, and
Young’s modulus, $E_c$ of 207,000 MPa, Poisson’s ratio $\nu$ of 0.3 and density, $\rho$ of 7700 kg/m³. The columns were modelled from solid steel with cross-sectional dimensions of 30 x 30 mm, the beams were made of steel profiles SHS 150 x 150 x 4 mm and floors were modelled of steel plate thickness of 12 mm. The stiffness of the dampers installed in the 1st, 3rd and 5th storeys was 1.26 x $10^6$ N/m. To adjust natural frequencies to 5.5 and 7.5 Hz, additional lumped mass was applied at each beam-column joint. The results of tip deflection and acceleration obtained by analytical models in computer program ABAQUS are presented below.

Time history responses of tip deflection and tip acceleration for the undamped BM7.5 and BM5.5 fitted with three stiffness dampers under the Northridge earthquake excitations (scaled to 0.8 g) are illustrated in Fig. 3.34. From this figure it can be seen that in terms of peak values of tip deflection and term root mean squared deflection only minor reductions were obtained. On the other hand, reductions in tip acceleration and root mean squared acceleration were more substantial.

![Fig. 3.34 – Tip deflection and acceleration response of the undamped BM5.5 and BM5.5 fitted with three viscous dampers](image)

The results of peak values of tip deflection and tip acceleration for the analytical models of the BM7.5 and BM5.5 subjected to the same three earthquake excitations as it was for experimental testing are presented in Table 3.2. As it can be seen from these results, in terms of tip deflection for BM7.5, the placement of the dampers in the 1st, 3rd and 5th storeys resulted in decrease in tip deflection under the Kobe earthquake. On the other hand, in the case of the El Centro and
Northridge earthquake excitations tip deflections were increased. The dampers performed significantly better when installed in BM5.5, where the reductions in tip deflection were experienced under all three earthquake excitations.

Table 3.2 – Seismic response of analytical models

<table>
<thead>
<tr>
<th>Control</th>
<th>Earthquake Type</th>
<th>BM7.5</th>
<th>BM5.5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tip Deflection</td>
<td>Tip Accel.</td>
<td>Tip Deflection</td>
</tr>
<tr>
<td></td>
<td>(mm)</td>
<td>(g)</td>
<td>(mm)</td>
</tr>
<tr>
<td>No Control</td>
<td>Kobe</td>
<td>7.90</td>
<td>1.51</td>
</tr>
<tr>
<td></td>
<td>El Centro</td>
<td>8.70</td>
<td>1.63</td>
</tr>
<tr>
<td></td>
<td>Northridge</td>
<td>7.30</td>
<td>1.42</td>
</tr>
<tr>
<td>Passive Damping System</td>
<td>Kobe</td>
<td>7.70</td>
<td>1.47</td>
</tr>
<tr>
<td></td>
<td>El Centro</td>
<td>8.80</td>
<td>1.72</td>
</tr>
<tr>
<td></td>
<td>Northridge</td>
<td>7.40</td>
<td>1.26</td>
</tr>
</tbody>
</table>

As regards to tip acceleration, in the cases of BM7.5 the dampers produced significant reductions under the Northridge earthquake and small reduction was experienced also under the Kobe earthquake, whereas increase in acceleration was experienced under the El Centro earthquake. In the cases of the BM5.5 the dampers produced significant acceleration reductions under the Kobe and Northridge earthquake excitations, whereas under the El Centro earthquake, slight acceleration increase was recorded.

While comparing tip deflection and tip acceleration obtained from computer program ABAQUS with results from experimental testing (Tables 3.1 and 3.2), in terms of tip deflection the results were in reasonably good agreement and in all the cases followed similar trends. As regards to tip acceleration reduction, the range of the results obtained from physical testing and from analytical computer model was slightly more open, nevertheless, all the results were, once again, in reasonably good agreement.

3.15.3 Effectiveness of damper configurations

The damper fitted in three configurations, namely diagonal, chevron brace and lover toggle are investigated in present study. Traditional diagonal and chevron brace configurations restrict the amount of damping being applied to a structure. On the other hand, the toggle brace configuration (Fig. 3.35) is capable of
magnifying the damping effect of the damper on the structure response and allows the damping device to be used more efficiently. Numerical examples of the efficiency of varieties of configuration were highlighted in Section 2.7.

![Fig.3.35 – Undamped frame and frame fitted with damper of chevron brace, diagonal and lower toggle brace configuration](Based on Marcelin S., 2001)

As referred in Section 2.7, the force generated by a viscous damper is directly proportional to the relative velocity between each end of the damper. Therefore, the larger the relative displacement, the larger the damping force produced. Because of the geometric orientation of the chevron and the diagonal bracing systems, the relative displacement along the direction of the viscous damper is either equal to (case of the chevron brace) or less than (case of the diagonal brace) the inter-storey drift of the floors between which they are connected. On the other hand, in the case of the toggle brace configuration, the relative displacement along the direction of damper is 2.5-3 times the interstorey drift.

The numerical examples of the diagonal, chevron brace and lower toggle configurations were examined by Marcelin (2001) on a 1-storey frame structure equipped with viscous damper fitted in three different configurations (one at time). The floor mass was $m = 1.458 \times 10^4$ kg, the elastic stiffness was $k = 1.28 \times 10^6$ kNs/m and the external damping was $c = 2.27 \times 10^4$ kN/m.

The results of peak values of tip deflection for the analytical models of the 1-storey frame subjected to the El Centro earthquake excitations are presented in Tables 3.3. As shown in Table 3.3, in terms of the tip deflection reduction, the frame fitted with the toggle brace system was twice as effective as the traditional configurations. The chevron and diagonal system had reductions of 15.4% and
12.7 % respectively as compared to 34 % for the lower toggle brace system. The results of this parametric study showed that the traditional diagonal and chevron configuration were capable of reducing the building response. However, the toggle brace damper configuration was significantly more effective.

Table 3.3 – Results of frame deflection (after Marcelin)

<table>
<thead>
<tr>
<th>Brace Configuration</th>
<th>Uncontrolled</th>
<th>Diagonal</th>
<th>Chevron</th>
<th>Lower Toggle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Deflection</td>
<td>0.059</td>
<td>0.052</td>
<td>0.050</td>
<td>0.039</td>
</tr>
<tr>
<td>% Deflection Red.</td>
<td>12.71</td>
<td>15.42</td>
<td>34.75</td>
<td></td>
</tr>
</tbody>
</table>

### 3.15.4 Results of analytical investigation of damper configurations

The analytical model created in computer program ABAQUS was constructed of steel. The material properties were compressive strength, $f'_c$ of 350 MPa, Young’s modulus, $E_c$ of 207,000 MPa, Poisson’s ratio $\nu$ of 0.3 and density, $\rho$ of 7700 kg/m$^3$. The columns and beam were made of steel profiles SHS 750 x 500 x 5 mm and floors were modelled of steel plate thickness of 15 mm. The floor mass was $m = 1.458 \times 10^4$ kg and the elastic stiffness was $k = 1.28 \times 10^6$ kNs/m.

![Image of graph showing deflection over time](image)

**Fig. 3.36 – Tip deflection of the undamped frame and frame fitted with viscous damper of lower toggle configuration.**

Fig. 3.36 illustrates the seismic responses of the undamped frame and the frame with viscous damper of the lower toggle configuration in terms of the peak values of tip deflection. The figure shows high performance of the lower toggle damper.
with tip deflection reduction of 27.8%. Furthermore, in terms of root mean squared displacement the performance of the damper was even higher.

The results for the frames fitted with the damper in all three configurations (one at time) compared with the undamped frame are presented in Table 3.4. The results revealed high tip deflection reduction experienced by all three dampers. However, the performance of the damper of lower toggle configuration was significantly higher. These results are in good agreement with the results obtained from the parametric study described in previous section.

Table 3.4 – Frame deflection (computer program ABAQUS)

<table>
<thead>
<tr>
<th>Brace Configuration</th>
<th>Uncontrolled</th>
<th>Diagonal</th>
<th>Chevron</th>
<th>Lower Toggle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Deflection</td>
<td>0.072</td>
<td>0.061</td>
<td>0.062</td>
<td>0.052</td>
</tr>
<tr>
<td>% Deflection Red.</td>
<td>15.27</td>
<td>13.89</td>
<td>27.78</td>
<td></td>
</tr>
</tbody>
</table>

As mentioned previously, it is unrealistic, both logistically and economically, to conduct appropriate experimental tests of medium or high-rise structures under seismic conditions. For these reasons, the validity of the method used in the present study was verified on benchmark models. It was not possible to exactly model the benchmark structures as some of the structural details were not available. Despite this, the present computer results with viscous dampers agreed reasonably well with those from the literature study and confirmed the validity of the investigated method.
Influence of Damping Systems on Building Structures Subject to Seismic Effects
Chapter 4

Results – High Rise Structures
Chapter 4: Results - High-rise structures

4.1 Introduction

The results from the finite element analyses of two types of high-rise structure are presented in this chapter. First type is represented by a 96 m high shear wall structure (described in the Section 3.7) embedded with five different damping systems (Section 3.11), namely friction and VE diagonal dampers, friction and VE chevron brace dampers and hybrid friction-VE dampers. These damping systems were installed within cut outs of shear wall at four different damper placements (Section 3.7.1) Seismic analyses were carried out with one type of damper at one placement at a time. Efficiency of these damping systems was investigated under five different earthquake excitations. This was the first structure treated in this research to determine feasibility of the procedure. The second type is represented by a structure, which includes the same shear wall with frames on both sides. This structure was fitted with diagonal VE dampers embedded within the cut outs of the shear wall at four different damper placements. Influence of the properties of the VE dampers on seismic mitigation with this structure was investigated under the five earthquake excitations. This analysis was used to establish optimum damper properties.

4.2 Seismic responses of 96 m high shear wall structure

There are various ways of assessing seismic response but computation of tip deflection is a reasonable measure of the overall effect of seismic response. Working back from tip deflection to equivalent base shear and moment is one way of ‘averaging out’ the seismic effects of varying accelerations up the wall. Hence any reduction is worthwhile in overall seismic design force. The results show that the value of reduction of tip deflection is dependent on the complex characteristics of the time histories used for assessment. Hence the benefits can only be legitimately assessed if the analysis is carried out for suite of time histories.

The undamped shear wall structural model was in this study created in order to compare its results with those of the shear wall structure embedded with five
damping systems. The results of the tip deflection and tip acceleration of this structure obtained under five earthquake excitations are presented in Table 4.1.

**Table 4.1 - Tip deflection and tip acceleration of the undamped structure.**

<table>
<thead>
<tr>
<th></th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S.Fernando</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection(m)</td>
<td>0.161</td>
<td>0.356</td>
<td>0.168</td>
<td>0.143</td>
<td>0.141</td>
</tr>
<tr>
<td>Acceleration(m/s²)</td>
<td>4.87</td>
<td>5.33</td>
<td>5.96</td>
<td>5.57</td>
<td>4.42</td>
</tr>
</tbody>
</table>

The results for all the shear wall structural models obtained under each of the five earthquakes records are presented below.

Figs. 4.1 to 4.5 illustrate tip deflection and tip acceleration under the El Centro earthquake excitations in this structural model of designation W1, which has the damper placed in the lower part of the structure, compared with the undamped structure. These figures clearly demonstrated that incorporation of the dampers to the structure reduced the peak values of tip deflections and tip accelerations under all five seismic loads. On the other hand, the performance of the dampers obtained under different excitations varied significantly. The highest tip deflection reduction was achieved for the diagonal VE damper, whereas the poorest results were obtained by the chevron brace friction damper. In terms of tip acceleration reduction the best results, once again, occurred for the diagonal VE damper, whereas the smallest reduction was recorded for hybrid friction-VE damper.

*Fig. 4.1 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal friction damper under the El Centro earthquake*
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Fig. 4.2 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE damper under the El Centro earthquake

Fig. 4.3 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with chevron brace friction damper under the El Centro earthquake

Fig. 4.4 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with chevron brace VE damper under the El Centro earthquake
The reductions in tip deflection and tip acceleration obtained with all damper types, configurations and at all locations for each of the five earthquake records compared with that of the undamped structure are presented below.

Fig. 4.6 illustrates the average percentage reduction in the peak values of tip deflections obtained by all the structural models compared with that of the undamped structure. Physical parameters of the diagonal VE dampers (spring stiffness, $k_d = 100 \times 10^6$ N/m, dashpot damping, $c_d = 100 \times 10^6$ Ns/m) and the diagonal friction dampers (friction coefficient, $\mu = 0.25$ and the geometry of the sliding tubes) were determined after a preliminary analysis where each was acting with approximately the same damping force. The hybrid friction-VE damper was created to represent 52% of the damping force of the diagonal VE damper, and 33% of the damping force of the chevron brace friction dampers. Finally, the parameters of the chevron brace friction dampers and chevron brace VE dampers represent 67% of the damping force of the diagonal dampers, based on geometry.

From this point of view, the results displayed good performance for all five types of damping systems. Overall, the highest reduction was achieved by models with hybrid friction-VE dampers with an average reduction of 22.2%. The second highest reduction was experienced by shear wall structure embedded with chevron brace friction dampers. It was followed by structures fitted with diagonal friction and diagonal VE dampers, which each showed relatively similar performance.
Finally, the lowest performance was recorded for structures fitted with chevron brace VE dampers, with an average tip deflection reduction of 12.3%.

While comparing shear wall structures fitted with VE dampers and structure fitted with friction dampers, VE dampers performed better in nearly all cases in the lower and middle parts of the structure, while friction dampers performed obviously better in the upper part and throughout the structure. The hybrid damping system displayed the highest performance and the most reliable behaviour.

Fig. 4.7 illustrates the average percentage tip deflection reduction of the models with respect to the damper locations.
The best performance was experienced in the shear wall structure of designation W1-3 with dampers placed in all three parts of the structure, with an overall reduction of 24.7%. Unexpectedly high performance with an overall reduction of 19.9% occurred for shear wall type W3 with dampers placed in the upper part of the structure. This was followed by an overall reduction of 19.1% for shear wall type W2 with dampers placed in the middle part of the structure. Considerably the lowest overall tip deflection reduction of 8.5% was obtained for shear wall type W1 representing dampers placed in the lower part of the structure.

The efficiency of the damping systems under a variety of earthquake loadings was also evaluated and results can be observed from Fig. 4.8.

It can be seen that the greatest average reduction in tip deflection of 26.3 %, was experienced under the Kobe earthquake excitation, which was characterised by a strongly narrow dominant frequency range (0.29-1.12 Hz). The second highest average deflection reduction of 19.8% occurred under the San Fernando earthquake, which, in contrast to the Kobe earthquake, had a wide band of dominant frequencies (0.58-4.39 Hz). The results of the shear wall structure obtained under the Northridge earthquake, which had a strongly dominant narrow frequency range (0.14-1.07 Hz), were slightly lower, with an average reduction of 19.2%. In the case of the El Centro earthquake excitation, which displays a wide band of dominant frequencies (0.39-6.39Hz), the efficiency of the models was slightly lower with an average reduction of 15.9%. The lowest performance of
11.2% occurred under the Hachinohe earthquake excitation, which had a moderate dominant frequency range (0.19-2.19Hz).

The primary objective of this study was to investigate the tip deflection of the structure. However, the percentage reductions in the peak values of the tip accelerations were also studied and the results are presented in Figs. 4.9-4.11.

Fig. 4.9 shows that all types of damping systems produced good results. Overall, the best result was achieved by the structure embedded with diagonal friction dampers, with an average tip acceleration reduction of 42.8%. The second highest reductions occurred for the shear wall structures embedded with diagonal VE dampers, this was followed by shear wall structures embedded with hybrid friction-VE dampers and structures embedded with the chevron brace VE damper. Finally, the poorest overall result was experienced by shear wall structures embedded with the chevron brace friction dampers, with an overall tip acceleration reduction of 18.1%. While comparing shear wall structures embedded with VE and friction dampers, it can be concluded that the greater tip acceleration reduction was achieved with VE dampers in the lower and middle parts of the structure, or with friction dampers in the upper part and throughout, though not as significantly as in the case of reduction in the tip deflection.

Fig. 4.9 - Average tip acceleration reductions for five types of damping systems.
In terms of damper placement (Fig. 4.10), the best results, with an overall tip acceleration reduction of 42.5% was achieved by structure type W1 representing the model with dampers placed in the lower part of the structure (Fig. 3.8), while type W1-3, with dampers placed in the all three parts of the structure, displayed a reduction of 36.1%. A noticeably lower overall reduction of 27.1% occurred for shear wall structure type W2 with the dampers placed in the middle part of the structure, and lastly the poorest result was experienced by structure type W3 with the dampers placed in the upper part of the structure, with an overall tip acceleration reduction of 16.4%.

The efficiency of the damping systems in reducing tip acceleration under a variety of earthquake loading is illustrated in Fig. 4.11.

![Fig. 4.10 - Average tip acceleration reductions (under five earthquakes) for different damper locations.](image)

![Fig. 4.11 - Average tip acceleration reductions (for five types of damping system) under different earthquake excitations.](image)
The highest overall tip acceleration reduction of 37.7% was achieved under the Hachinohe earthquake excitation and only slightly lower reduction of 35.1% was experienced under the San Fernando earthquake excitation. The overall reduction occurred for the Northridge earthquake was lower with an average of 33.8%, this was followed by 26.8% overall tip acceleration reduction for the Kobe earthquake and finally the lowest reduction with average value of 22.8% was experienced under the El Centro earthquake excitation.

Three types of damping mechanisms, viz, VE, friction, and hybrid friction-VE dampers, located within cut outs of shear walls, to mitigate the seismic response of shear wall structures were investigated in this section. These structural systems was modelled and analysed under five different earthquakes, using finite element techniques. The effects of damper type, configuration and location on the seismic response were studied. The results of this investigation confirmed that substantial reductions in deflection and acceleration of the structure could be achieved by all three types of dampers in all configurations and at all locations. However, responses under earthquakes with varying frequency content and strong motion duration have yielded a wide range of results and some interesting features.

In terms of reduction in the tip deflection, the best performance was experienced when dampers were placed in the upper level, while greatest reduction in the peak values of tip acceleration was achieved when dampers were placed in the lower level. VE dampers performed better than friction dampers in the lower and middle parts of the structures, while friction dampers performed better in the upper parts of the structure and throughout. Hybrid dampers were overall the most efficient and also had the most stable performance.

The natural frequencies of most of the shear wall structural models (with cut outs at different location) were within the frequency range of the dominant modes of the earthquakes. Hence, this study encountered resonant structural vibration, in most cases during the strong motion, and has demonstrated the possibility of mitigating seismic response of structures by an appropriately embedded damping systems.
The complete sets of results in terms of percentage reductions in tip deflections and tip accelerations for the shear wall structures are presented in Tables 4.2 and 4.3 and in Figs. 4.12 and 4.13 respectively.

**Fig. 4.12 – Percentage reductions in tip deflection of all shear wall structures**

**Table 4.2 - Percentage reductions in tip deflection of the all models**

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Influence of Damping Systems on Building Structures Subject to Seismic Effects

Fig. 4.13 – Percentage reductions in tip acceleration of all shear wall structures

Table 4.3 - Percentage reductions in maximum tip acceleration of the all models

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4.3 Seismic responses of 24-storey frame-shear wall structure

In order to further demonstrate the feasibility of the procedure used in this study, the second type of high-rise structure represented by 24-storey frame-shear wall structure (discussed in Section 3.8.1) embedded with diagonal VE dampers was also investigated. The dimensions and parameters of the shear walls were chosen as before and also diagonal VE dampers were embedded at locations similar to that in the study with shear wall structure. The seismic responses of the frame-shear wall structures were analysed under the same five earthquake excitations. The natural frequencies of these models (with different cut-outs) ranged from 0.434–0.727 Hz and were within the frequency range of dominant modes of the earthquakes considered in this study.

4.3.1 Damping properties of VE dampers

The main aim of the following sections is to illustrate the influence of the damping properties of the dampers on structural performance under earthquake load. The results from the finite element analysis for the 24-storey frame-shear wall structures embedded with diagonal VE dampers are presented below. These structures included four different damper placements, as shown in Fig. 3.9. The peak values of tip deflection and tip acceleration for three of them, namely T1 with the damper placed in the lowest part of the structure, T3 with the damper placed in upper part of the structure and T1-3 with the dampers placed in the all three parts of the structure were studied under the El Centro earthquake excitation.

4.3.2 Frame-shear wall structure type T1

The design of structure designated by T1 was discussed in details in sections 3.8.1 and 3.12. This structure, which has damper positioned over the storeys 4 to 6, had the large change in the natural frequency compared to undamped structure, when the natural frequency was reduced from 0.602 Hz to frequencies varying in range from 0.401 Hz to 0.472 Hz, depending on the stiffness of the damper. This change in the natural frequency was anticipated, since reducing the stiffness at the
lowest part of the structure increases the ductility, and thus considerably reduces the natural frequency of the structure. The range of the most dominant frequencies for the El Centro earthquake, as it can be seen from Fig. 3.28, were in the range 0.5-0.7 Hz what means that instalment of the damper partially detune the structure out of this range.

A dynamic analysis of this structure with damper placed over the storeys 4 to 6 were conducted under the El Centro earthquake excitation. The damper was created as elastic spring and dashpot in parallel, as described in Section 3.12. The values of spring were after preliminary study determined to be in the range from 10,000 to 200 x 10^6 N/m and values of dashpot to be in the range from 10,000 to 200 x 10^6 Ns/m. A summary of the results of the seismic response of the structure embedded with these dampers is provided in Tables 4.4 and 4.5. Table 4.4 shows the percentage reductions in the peak values of tip deflections experienced by this structure compared with results of the undamped structure, while Table 4.5 shows the percentage reductions in the peak values of tip acceleration experienced by the same structure.

The results of percentage reduction in tip deflection of the structure embedded with damper of varying properties display overall very high performance. The results reveal the high level of sensitivity of the structure to varying damping properties of dashpot. The best performance with the highest reduction of 38.9% was recorded for dashpot with damping parameter of $c_d = 70 \times 10^6$ Ns/m. The second highest reductions were recorded for dashpot with damping parameter of $c_d = 60 \times 10^6$ Ns/m, it was followed by $c_d = 50 \times 10^6$ Ns/m and $c_d = 80 \times 10^6$ Ns/m with reductions only slightly lower. In general, it can be stated that dashpot with the values of damping in the range from $c_d = 20 \times 10^6$ Ns/m to $c_d = 140 \times 10^6$ Ns/m experienced very high and stable performance, while decrease in the performance was significant when value of the damper was moved out of this range.
Table 4.4 - Percentage reductions in tip deflection of structure type T1

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*Tip deflection of the undamped structure type T0 represents 0.188 m
With regards to the spring parameters, the results make obvious that the tip deflection of the structure noticeably depend less on the change in the stiffness of the spring. Influence of the damper stiffness was insignificant in the range of values in which the dashpot displayed the highest performances. Outside this range, the best results were recorded for the spring with stiffness of $k_d = 80 \times 10^6$ N/m. It was followed by $k_d = 100 \times 10^6$ N/m, $k_d = 120 \times 10^6$ N/m and $k_d = 60 \times 10^6$ N/m. For convenience see also Fig. 4.14.

A summary of the results of the percentage reductions in tip acceleration for the same structure is shown in Table 4.5. In general, the results revealed the significant tip acceleration reductions. However, the level of sensitivity of the structure to varying damping properties of dashpot was noticeably high. The best performance, with the highest reduction of 64.4% was achieved by dashpot with damping parameter of $c_d = 20 \times 10^6$ Ns/m. The tip acceleration reductions for the dashpots with values from $c_d = 10 \times 10^6$ Ns/m to $c_d = 50 \times 10^6$ Ns/m were only slightly lower. However, as it can be seen from Fig. 4.15, the decrease in tip acceleration reductions beyond this range was substantial.
Table 4.4 - Percentage reductions in tip acceleration of structure type T1

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*Tip acceleration of the undamped structure type T0 represents 2.97 m/s²*
With respect to spring parameter, the result illustrate low sensitivity to change of the spring stiffness in the cases when values of dashpot were equal or higher than $c_d = 10 \times 10^6$ Ns/m. On the other hand, when damping of dashpot was below these values, the changes in spring parameters caused noticeably varied results. The highest tip acceleration reductions were recorded for spring of $k_d = 10 \times 10^6$ N/m this was followed by $k_d = 1 \times 10^6$ N/m and $k_d = 1 \times 10^5$ N/m.

### 4.3.3 Frame-shear wall structure type T3

The frame-shear wall structure designated by T3 (discused in Section 3.8), which has damper positioned over the storeys 19 to 21, experienced the smaller change in the natural frequency compared to undamped structure, when the natural frequency was altered from 0.602 Hz to frequencies varying in range from 0.586 Hz to 0.612 Hz, depend on stiffness of the damper. These frequencies were within the range of the most dominant frequencies of the El Centro earthquake (0.5-0.7 Hz), hence this structure was treated at least partially in resonant conditions.

The damper was created as elastic spring and dashpot in parallel, as described in Section 3.12. The parameters of spring and dashpot were determined to be the same as in the previous structure. A summary of the results of reductions in the
peak values of tip deflections obtained by this structure compared with results of the undamped structure under the El Centro (1940) earthquake excitation is provided in Tables 4.6.

The results illustrate high performance of the dampers. However, this performance was more than 10% lower than in the case of previously studied structure T1. From the presented results it can be seen, that the best performance with the highest tip deflection reduction of 28.1% was recorded for dashpot with damping parameter $c_d = 30 \times 10^6$ Ns/m. The subsequent increase or decrease in damping of dashpot caused relatively regular decrease in tip deflection reductions. The performance of the dampers was distinctly lower, when the damping value of dashpot was less than $c_d = 10 \times 10^6$ Ns/m.

In terms of spring parameter (Table 4.6), the highest deflection reductions were recorded for spring with values of $k_d = 10 \times 10^6$ N/m and $k_d = 1 \times 10^6$ N/m. Consequent increase in stiffness of spring resulted in a gradual decrease in tip deflection reductions. In addition, in the case when the value of dashpot was lower than $c_d = 10 \times 10^6$ Ns/m the performance of the damper decreased considerably for the all spring parameters.

Fig. 4.16 - Percentage reductions in tip deflection of the structure type T3
Table 4.6 - Percentage reductions in tip deflection of the structure type T3

| $C_d \times 10^6$ Ns/m | 0.05 | 0.1 | 1.0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 100 | 120 | 140 | 160 | 180 | 200 |
|-------------------------|-----|-----|-----|----|----|----|----|----|----|----|----|-----|-----|-----|-----|-----|-----|-----|
| Cd [10^6 Ns/m]          |     |     |     |    |    |    |    |    |    |    |    |     |     |     |     |     |     |     |
| 0.05                    | 5.4 | 5.4 | 4.8 | 20.4 | 26.3 | 27.8 | 27.2 | 25.7 | 25.7 | 24.0 | 22.8 | 21.0 | 19.5 | 18.6 | 16.8 |     |     |
| 0.1                     | 5.4 | 5.4 | 4.8 | 20.4 | 26.3 | 27.8 | 27.5 | 26.3 | 25.7 | 24.6 | 23.4 | 21.0 | 19.5 | 18.6 | 16.8 |     |     |
| 1.0                     | 4.2 | 4.8 | 4.8 | 20.4 | 26.3 | 28.1 | 27.5 | 26.3 | 25.7 | 24.6 | 23.4 | 21.6 | 19.8 | 18.6 | 16.8 |     |     |
| 10                      | -4.2 | -2.4 | 2.4 | 18.0 | 24.6 | 26.9 | 26.3 | 25.7 | 25.1 | 24.0 | 23.4 | 21.6 | 19.8 | 18.6 | 16.2 |     |     |
| 20                      | -1.8 | -0.6 | 1.8 | 16.2 | 22.8 | 25.7 | 25.1 | 24.6 | 24.0 | 23.4 | 22.8 | 21.0 | 19.8 | 18.6 | 16.8 |     |     |
| 30                      | 0.0 | 6.6 | 9.0 | 14.4 | 21.0 | 24.0 | 24.0 | 24.0 | 23.4 | 22.8 | 22.2 | 21.0 | 19.2 | 16.8 | 16.2 |     |     |
| 40                      | 2.4 | 3.6 | 5.4 | 13.8 | 19.8 | 22.8 | 23.4 | 23.4 | 23.4 | 22.8 | 22.2 | 21.0 | 19.2 | 16.8 | 16.2 |     |     |
| 50                      | 1.2 | 0.0 | 5.4 | 12.6 | 19.2 | 22.2 | 22.8 | 22.8 | 22.2 | 22.2 | 21.6 | 20.4 | 19.2 | 17.4 | 16.2 |     |     |
| 60                      | 0.0 | 0.0 | 6.6 | 12.0 | 18.0 | 21.0 | 21.6 | 21.6 | 21.6 | 21.6 | 20.4 | 19.2 | 18.0 | 16.2 |     |     |     |
| 80                      | 3.6 | 3.6 | 6.6 | 12.0 | 16.2 | 19.2 | 20.4 | 20.4 | 21.0 | 21.0 | 21.0 | 19.8 | 19.2 | 18.0 | 16.2 |     |     |
| 100                     | 2.4 | 3.0 | 6.0 | 10.8 | 15.0 | 17.4 | 19.2 | 19.2 | 20.4 | 20.4 | 20.4 | 19.2 | 17.4 | 17.4 | 16.2 |     |     |
| 120                     | 3.0 | 4.2 | 6.6 | 10.2 | 13.8 | 16.2 | 18.6 | 18.6 | 19.2 | 19.8 | 19.8 | 19.2 | 18.6 | 17.4 | 16.2 |     |     |
| 200                     | 4.2 | 4.2 | 6.0 | 9.6 | 11.4 | 13.2 | 15.6 | 15.6 | 16.8 | 17.4 | 17.4 | 17.4 | 17.4 | 18.8 | 15.6 |     |     |

*Tip deflection of the undamped structure type T0 represents 0.188 m
A summary of reductions in the peak values of tip acceleration obtained by the structure with dampers embedded over the storeys 19 to 21 compared with results of the undamped structure obtained under the El Centro earthquake excitation is illustrated in Tables 4.7

The tip acceleration reductions achieved by this structure were very high, nevertheless, values of the reductions were in average by 20% lower than it was for the structure with damper embedded in the lower storeys. From these results it can be seen, that the highest tip acceleration reduction of 44.4% occurred for dashpot with damping parameter $c_d = 10 \times 10^6$ Ns/m. The subsequent increase or decrease in the damping of dashpot caused relatively regular decrease in reductions. Moreover the tip acceleration reductions of the structure became even insignificant when the value of the dashpot represented $c_d = 1 \times 10^5$ Ns/m on one side or $c_d = 200 \times 10^6$ Ns/m on the other side of the range, respectively.

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**Fig. 4.17 - Percentage reductions in tip acceleration of the structure type T3**
Table 4.7 - Percentage reductions in tip acceleration of the structure type T3

| \( C_d [10^6 \text{ Ns/m}] \) | 0.05 | 0.1 | 1.0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 100 | 120 | 140 | 200 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.05 | 0.3 | 11.4 | 24.6 | 42.3 | 34.3 | 24.2 | 19.9 | 16.8 | 13.8 | 11.1 | 9.1 | 6.4 | 5.1 | 4.0 | 1.3 |
| 0.1 | 0.3 | 11.4 | 25.9 | 43.1 | 34.3 | 24.2 | 19.9 | 16.8 | 13.8 | 11.1 | 9.1 | 6.4 | 5.1 | 4.0 | 1.3 |
| 1.0 | 0.3 | 7.1 | 24.6 | 44.4 | 35.4 | 25.6 | 19.2 | 15.8 | 12.8 | 10.8 | 8.4 | 6.7 | 5.4 | 3.7 | 0.7 |
| 10 | 21.9 | -2.0 | 22.2 | 43.4 | 37.0 | 24.9 | 19.2 | 14.1 | 13.5 | 10.4 | 8.1 | 7.4 | 6.4 | 5.7 | 0.7 |
| 20 | -22.2 | -19.9 | 19.2 | 42.1 | 36.7 | 24.2 | 18.5 | 15.2 | 13.1 | 10.4 | 8.4 | 7.1 | 5.7 | 4.0 | 1.3 |
| 30 | -10.1 | -9.1 | 22.9 | 41.1 | 35.4 | 24.2 | 18.5 | 15.2 | 12.5 | 10.8 | 8.4 | 6.7 | 5.7 | 4.0 | 1.7 |
| 40 | -3.7 | -6.1 | 23.6 | 40.1 | 34.3 | 25.3 | 17.8 | 15.2 | 12.5 | 10.1 | 8.4 | 6.7 | 6.1 | 4.7 | 2.0 |
| 50 | -7.4 | -8.4 | 23.2 | 39.1 | 34.0 | 24.9 | 17.5 | 14.8 | 12.1 | 10.1 | 8.8 | 7.1 | 6.1 | 5.1 | 2.0 |
| 60 | -12.8 | -11.4 | 20.9 | 38.4 | 33.7 | 23.9 | 17.2 | 14.8 | 11.8 | 9.8 | 8.8 | 7.4 | 6.7 | 5.7 | 2.4 |
| 80 | 13.5 | 14.1 | 27.9 | 38.4 | 34.3 | 23.6 | 17.5 | 14.5 | 11.8 | 9.8 | 9.1 | 7.4 | 6.7 | 5.4 | 2.0 |
| 100 | 6.7 | 5.7 | 25.9 | 37.7 | 33.3 | 23.2 | 17.5 | 14.5 | 11.4 | 9.8 | 9.1 | 7.1 | 6.1 | 4.7 | 1.7 |
| 120 | 2.4 | -3.0 | 20.2 | 36.0 | 32.7 | 22.6 | 17.5 | 13.8 | 16.5 | 10.1 | 8.4 | 6.7 | 5.1 | 4.4 | 1.0 |
| 200 | -15.8 | -13.1 | 16.5 | 28.3 | 26.6 | 20.9 | 17.2 | 13.8 | 11.4 | 9.8 | 8.1 | 6.4 | 6.1 | 4.4 | 2.7 |

*Tip acceleration of the undamped structure type T0 represents 2.97 m/s²*
As regards to the effect of the different spring parameters on tip acceleration reduction of the structure, the best results occurred for springs with stiffness of $k_d = 1 \times 10^5$ N/m and $k_d = 1 \times 10^6$ N/m. On the other hand, the subsequent increase in stiffness of the spring caused a gradual decrease in tip acceleration reductions. Nevertheless this statement is fully accurate only when the values of dashpots were from $10 \times 10^6$ to $80 \times 10^6$ Ns/m, while the results experienced out of this dashpot’s range were obviously irregular without any recognizable trend.

4.3.4 Frame-shear wall structure type T1-3

The design of structure designated by T1-3 was discussed in details in sections 3.8. This structure, which has damper positioned over the storeys 4 to 6, 10 to 12 and 19 to 21, experienced the large change in the natural frequency compared to undamped structure, when the natural frequency was reduced from 0.602 Hz to frequencies in range from 0.370 Hz to 0.458 Hz, depend on stiffness of the damper. The installation of dampers within the cut outs of shear wall detune structure from the range of the most dominant frequencies, which for the El Centro earthquake, as it can be seen in Fig. 3.28, were in the range 0.5-0.7 Hz.

The diagonal VE dampers were created as elastic spring and dashpot in parallel, as described in Section 3.12. The values of spring and dashpot were determined to be in the same range as it was in the previous case. A summary of the results of the percentage reductions in the peak values of tip deflections obtained by this structure compared with results of the undamped structure are presented in Table 4.5 and in Fig. 4.18.

The results reveal noticeably stable performance of dampers within the broad range of damping properties of dashpot. The best performance with the highest reduction of 41.3% was achieved for dashpot with damping parameter of $c_d = 80 \times 10^6$ Ns/m. The performance of the dampers with dashpots within the range from $c_d = 20 \times 10^6$ Ns/m to $c_d = 200 \times 10^6$ Ns/m were only slightly lower. The performance of dampers with dashpot $c_d = 10 \times 10^6$ Ns/m were considerable lower, however,
still adequately high. On the other hand, the performances of the dampers with the dashpots under this value were unfavourable with high increase in tip deflections.

With respect to spring parameter, Table 4.8 illustrates, that when values of dashpots were in the range from $c_d = 20 \times 10^6$ Ns/m to $c_d = 200 \times 10^6$ Ns/m, the highest reductions in tip deflection were recorded for the dampers with spring of stiffness $k_d = 1 \times 10^6$ N/m. The similar tip deflections were also experienced when stiffness of spring was $k_d = 1 \times 10^5$ N/m and $k_d = 5 \times 10^4$ N/m. On the other hand, a subsequent increase in stiffness of the spring over the value $k_d = 1 \times 10^6$ N/m caused a gradual decrease in tip deflection reductions. For the damping of dashpot smaller than $c_d = 20 \times 10^6$ Ns/m the unfavourable increase in deflections were eliminated only in the case when the value of the stiffness of spring was equal or higher than $c_d = 80 \times 10^6$ Ns/m.

![Deflection reductions of the structure type T1-3](image)

*Fig. 4.18 - Percentage reductions in deflection of the structure type T1-3*
Table 4.8 - Percentage reductions in tip deflection of the structure type T1-3

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*Tip deflection of the undamped structure type T0 represents 0.188m.
The results for percentage reduction in tip acceleration of the structure embedded with damper of varying properties under the El Centro earthquake excitation is provided in Tables 4.9. The results revealed the significantly high tip acceleration reductions experienced for broad range values of spring and dashpot with the highest reduction of 69.4%. As regards to influence of dashpot, the highest tip acceleration reductions were recorded for the value of $k_d = 20 \times 10^6$ N/m. The subsequent increase or decrease of the damping in dashpot resulted in relatively regular decrease in the tip acceleration reduction.

In terms of spring parameter it can be seen, that while the damping of dashpot was within the range $c_d = 10 \times 10^6 - 200 \times 10^6$ Ns/m the highest acceleration reductions were recorded for the spring $k_d = 10 \times 10^6$ N/m and $k_d = 1 \times 10^6$ N/m. Further increase in stiffness of spring reasoned in slight decrease in the tip acceleration reductions. In the case when the value of the dashpot was less than $c_d = 10 \times 10^6$ Ns/m the highest tip acceleration reduction was recorded for the springs $k_d = 50 \times 10^6$ N/m and $k_d = 60 \times 10^6$ N/m.

![Acceleration reductions of the structure type T1-3](image)

*Fig. 4.19 - Percentage reductions in tip acceleration for the structure type T1-3*
Table 4.9 - Percentage reductions in tip acceleration for the structure type T1-3

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*Tip acceleration of the undamped structure type T0 represents 2.97 m/s².*
4.3.5 Parameters of VE dampers - Summary of finding

The influence of the damping properties of the diagonal VE dampers on performance of the 24-storey frame-shear wall structure under earthquake load were investigated. The dampers experienced a high performance over the wide range of damping parameters. However, from the results it is evident, that the damper stiffness, $k_d$, has clearly less impact on the response of the structure than the damping coefficient, $c_d$.

In the case of the structure with the damper embedded in the upper storeys the maximum tip deflection reduction of 28.1% was achieved for dashpot with damping parameter $c_d = 30 \times 10^6$ Ns/m and spring with stiffness of $k_d = 10 \times 10^6$ N/m. The subsequent increase or decrease in these parameters resulted in relatively regular decrease in tip deflection reduction. The performance of the damper embedded in the lower storeys was even higher with maximum tip deflection reduction of 38.9%. This reduction was obtained for dashpot with damping parameter $c_d = 70 \times 10^6$ Ns/m and spring with stiffness of $k_d = 1 \times 10^6$ N/m. In addition, to higher maximum tip deflection reduction, the range of the parameters at which the damper achieved the performance close to the maximum values was noticeably wider.

The highest performance, as it was expected, occurred for the structure with the dampers placed in the all three parts of the structure. The maximum tip deflection reduction of 41.3% occurred for the dashpot with damping parameter $c_d = 80 \times 10^6$ Ns/m and spring with stiffness of $k_d = 10 \times 10^6$ N/m. While comparing these results with results recorded for the damper placed in the lower storeys, it is obvious, that fitting of additional two dampers in the middle and upper parts of the structure produced only slightly increased in maximum deflection reduction. On the other hand, as it can be seen from Figs. 4.14 and 4.16, the fitting of these two additional dampers extended the range of the parameters at which the damper experienced the maximum performance.
In terms of tip acceleration, the significantly high reductions were experienced in all three damper placements. In the case of the structure with the damper embedded in the lower storeys the highest tip acceleration reduction of 64.4% occurred for the dashpot $c_d = 20 \times 10^6 \text{ Ns/m}$ and spring $k_d = 10 \times 10^6 \text{ N/m}$, while additional increase or decrease in damping resulted in noticeably decrease in tip acceleration reductions. An influence of change in the damper stiffness was less significant.

Tip acceleration reductions experienced by the structure with the damper embedded in the upper storeys was noticeably lower. The maximum reduction of 44.4% occurred for the damper with the dashpot $c_d = 10 \times 10^6 \text{ Ns/m}$ and spring $k_d = 1 \times 10^5 \text{ N/m}$. The tip acceleration reductions experienced for other values of dashpot were significantly lower. The change in the damper stiffness did not considerably affect the results.

The highest tip acceleration reduction was experienced by the structure with the dampers embedded in all three parts of the structure. The maximum reduction of 69.4% was experienced for damper with dashpot $c_d = 20 \times 10^6 \text{ Ns/m}$ and spring $k_d = 10 \times 10^6 \text{ N/m}$. The results for other values of damping revealed noticeably lower tip acceleration reductions. An influence of the change in the damper stiffness was, yet again, relatively insignificant.

The results for tip acceleration reduction, similarly to results for tip deflection reduction, reveal significantly higher efficiency of the damper positioned in the lower storeys. While comparing the tip acceleration reductions experienced by all three structures, the results imply that instalment of the damper in the middle and upper storeys had only small influence on the structural performance over the whole range of investigated dampers parameters.

Influence of the damper parameters on seismic response of these structures was, in a smaller scale conducted under the Hachinohe and Kobe earthquake
excitations. The tip deflection and tip acceleration reduction occurred under these earthquake excitations were significantly different. However, the ranges of the most effective damping parameters were relatively close to those experienced under the El Centro earthquake.

### 4.3.6 Response of the structure under five earthquake excitations

The 24-storey frame-shear wall structural model was further investigated under five earthquake excitations. Based on results reported in the previous sections it can be seen that the structures experienced the highest performance when $C_d$ was within the range $10 \times 10^6$ to $100 \times 10^6$ Ns/m and $k_d$ within the range $1 \times 10^6$ to $60 \times 10^6$ N/m. Hence in order to facilitate comparisons, approximate average values of $k_d = 40 \times 10^6$ N/m and $c_d = 50 \times 10^6$ Ns/m, respectively were determined and used in all subsequent cases.

Fig. 4.20 illustrates the percentage reductions in the peak values of tip deflections experienced by the structures embedded with the dampers at four different placements.

![Tip Deflection Reduction](image)

*Fig. 4.20 – Percentage reduction in tip deflection of 24-storey frame-shear wall structure in terms of damper placements*

The results showed significant performance of the structures for all damper placements. The high average tip deflection reduction of 20.0% was achieved by the structure with the damper placed in the lower storeys. A slightly lower average
tip deflection reduction of 17.8% occurred for the structure with the damper placed in the middle storeys, while the lowest efficiency with still relatively high an average reduction of 14.1% was experienced by the structure with the upper storeys damper placement. Clearly the highest average tip deflection reduction, as it was expected, was obtained by the structure with the three dampers placement.

As it can be seen in Fig. 4.21, the dampers achieved the highest tip deflection reductions under the El Centro earthquake excitation however the reduction experienced under the Kobe and San Fernando earthquakes were only slightly lower. The reductions occurred under the Northridge earthquake excitation were also relatively high, while clearly the lowest reductions were experienced under the Hachinohe earthquake.

![Fig. 4.21 – Percentage reduction in tip deflection of 24-storey frame-shear wall structure under five earthquake excitations](image)

Fig. 4.22 shows the results of peak values in tip acceleration reduction experienced by the same structures. The structures achieved significant overall tip deflection reduction; however the range of the particular results was noticeably wide. The average tip deflection reduction of 18.1% occurred for the structure T3, which had the damper placed in the upper storeys. Noticeably higher reduction of 26.4% was experienced by the structure T2 with the damper placed in the middle storeys. Clearly the highest tip deflection reductions were achieved by the structures T1, which had the damper placed in the lower storeys and structure T1-3, with the dampers placed in the lower, middle and upper storeys, which both
obtained relatively similar results under all earthquake excitations. Based on this, it can be suggest that in the case of the structure T1-3 the dampers placed in the middle and upper storeys were rather ineffective in reducing tip acceleration.

**Fig. 4.22** – Percentage reduction in tip acceleration of 24-storey frame-shear wall structure in terms of damper placements

Fig.4.23 presents the same results in terms of tip acceleration reduction experienced under different earthquake excitations. Clearly the highest reductions were achieved under the San Fernando earthquake. It was followed by the reductions occurred under the Kobe and El Centro earthquakes. The reductions recorded under the Northridge earthquake were slightly lower and the lowest reductions were, yet again, recorded under the Hachinohe earthquake.

**Fig. 4.23** – Percentage reduction in tip acceleration of 24-storey frame-shear wall structure under five earthquake excitations
The dampers demonstrate a high performance in the vast majority of cases. In general, the dampers achieved clearly better results in both investigated parameters when placed in the lower storeys, however, in the case of the Hachinohe earthquake, where the dampers experienced the lowest reductions in both investigated parameters reverse trend was observed.

While comparing the results from 24-storey frame-shear wall structure embedded with diagonal VE dampers with results obtained from 96m tall shear wall structure embedded with the same dampers, surprisingly higher tip deflection and tip acceleration reductions were obtained by 24-storey frame-shear wall structure. Nevertheless, both types of structure revealed similar trends in both investigated parameters.
Chapter 5

Results - 18-Storey Structures
Chapter 5: Results - 18-storey structure

5.1 Introduction

The results from the finite element analysis of two types of medium-rise structure are presented in this chapter. First type is represented by an 18-storey frame-shear wall structure (described in Section 3.5.1) embedded with six different damping systems (Section 3.10), namely friction and VE diagonal dampers, friction and VE chevron brace dampers, hybrid friction-VE dampers and VE lower toggle dampers. These damping systems were embedded in cut outs of shear walls at twelve different damper placements (Section 3.5.2). Seismic analyses were carried out with one type of damping system at one placement a time. The efficiency of these damping systems were investigated under the five different earthquake excitations. The second type is represented by an 18-storey frame structure of the same parameters as it was for an 18-storey frame-shear wall, except the shear wall is replaced by one bay of frame. This structure was fitted with diagonal VE dampers installed at 12 different placements. The efficiency of the dampers fitted in this structure ws investigated under the same five earthquake excitations.

5.2 Seismic response of 18-storey frame-shear wall structure

As it was mentioned previously, there are various ways of assessing seismic response. However, computation of tip deflection is a reasonable measure of the overall effect of the earthquake. For this reason any reduction in tip deflection represents a worthwhile reduction in overall seismic design force. Though tip deflection is more important in assessing overall seismic response, the reductions in the peak values of the tip accelerations at the top of the structure are also investigated.

Figs. 5.1-5.5 illustrate the tip deflection and tip acceleration of the structure designated by H1-3 with the diagonal VE dampers fitted in the lowest three storeys compared with tip deflection and tip acceleration of the undamped structure obtained under five earthquake excitations. From these graphs it is evident that the dampers embedded into the cut-outs of shear walls, in most cases,
Influence of Damping Systems on Building Structures Subject to Seismic Effects

significantly reduced the tip deflection and tip acceleration throughout the duration of the earthquakes.

![Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the El Centro earthquake](image1)

**Fig. 5.1** - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the El Centro earthquake

![Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the Hachinohe earthquake](image2)

**Fig. 5.2** - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the Hachinohe earthquake

![Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the Kobe earthquake](image3)

**Fig. 5.3** - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the Kobe earthquake
Fig. 5.4 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the Northridge earthquake

Fig. 5.5 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal VE dampers under the San Fernando earthquake

Figs. 5.6-5.8 illustrate tip deflection and tip acceleration responses of six damping systems embedded in the same structure under the El Centro earthquake excitation. From these graphs, as well as from numerous other results obtained with dampers at different placements, it was evident that the different damping properties of the friction and VE dampers resulted in significantly different seismic responses.

Fig. 5.6 shows the typical tip deflection and tip acceleration responses of structure embedded with diagonal friction and diagonal VE dampers. It can be seen, that the diagonal friction dampers surpassed the diagonal VE dampers in their ability to reduce the intensity of the initial strong motion. In contrast, advantage of the
diagonal VE dampers was in gradually decreasing the tip deflection and tip acceleration of the structure.

*Fig. 5.6 - Tip deflection and tip acceleration responses of the structures embedded with diagonal friction and VE dampers under the El Centro earthquake*

Fig. 5.7 compares the seismic responses of the same structure embedded with chevron brace friction and chevron brace VE dampers, respectively. Both structures displayed relatively close tip deflection responses. On the other hand, their tip acceleration responses were more diverse. The structure embedded with the chevron brace friction dampers revealed tip acceleration response more typical for the structures fitted with friction dampers, whereas, tip acceleration response of the structure fitted with the chevron brace VE dampers was more typical for the structure fitted with VE dampers.

*Fig. 5.8 - Tip deflection and tip acceleration responses of the structures embedded with chevron brace friction and VE dampers under the El Centro earthquake*
Influence of Damping Systems on Building Structures Subject to Seismic Effects

Tip deflection and tip acceleration responses of the structures embedded with hybrid friction-VE dampers and lower toggle VE dampers, respectively, are illustrated in Fig. 5.8. The seismic response of the structure fitted with hybrid friction-VE dampers was mostly, as expected, somewhere, between the response of the chevron brace friction dampers and the diagonal VE dampers. On the other hand, the seismic responses of the lower toggle VE dampers were, in the vast majority of cases, noticeably close to the seismic responses of the structure fitted with friction dampers.

![Graph showing comparison between Hybrid dampers vs. Lower toggle VE dampers](image)

5.2.1 Undamped structure

The undamped structural model was in this study created in order to compare its results with the results of the structures fitted with six damping systems. The tip deflection and tip acceleration of this structure experienced under five earthquake excitations are presented in Table 5.1

Table 5.1 - Tip deflection and tip acceleration of the undamped 18-storey structure.

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Fig. 5.9 shows the interstorey drifts of the undamped 18-storey structure under five earthquake excitations. The highest interstorey drifts, with the maximum
interstorey deflection of 0.0326 m occurred under the Hachinohe earthquake. The second highest interstorey drifts of 0.0221 m was recorded under the El Centro earthquake. This followed by the Northridge earthquake with the maximum interstorey deflection of 0.0204 m and the Kobe earthquake with the maximum interstorey deflection of 0.0128 m. The lowest interstorey drifts were recorded under the San Fernando earthquake with the maximum interstorey deflection of 0.0100 m. In Fig. 5.9 it can also be seen that the maximum interstorey drifts occurred between storeys 14 and 15. This was followed by drifts between storeys 13 and 14, then 15 and 16, 16 and 17, 12 and 13, 11 and 12, etc.

![Fig. 5.9 - Interstorey drifts of undamped structure under the five earthquake excitations](image)

5.2.2 Diagonal friction dampers

The results of the tip deflection and tip acceleration experienced by the structures embedded with diagonal friction damper are presented below.

Fig. 5.10 illustrates time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures embedded with diagonal friction dampers, which were placed in storeys 1 to 3 and 13 to 15 respectively. In these graphs, it can be seen that the dampers embedded
in the upper storeys (with higher interstorey drifts) produced clearly better results in both investigated parameters than the dampers fitted in the lowest storeys.

Fig. 5.10 Tip deflection and acceleration responses of the structures with different placement of the diagonal friction dampers under the El Centro earthquake

The percentage reductions in the peak values of the tip deflection experienced by the structures fitted with the diagonal friction damper embedded at six different locations under five different earthquakes are presented in Fig. 5.11.

Fig. 5.11 – 18 storey structure embedded with 1 diagonal friction damper -percentage reductions in tip deflection under different earthquake records

As it can be seen from this figure, the damper displayed extraordinary performance under the El Centro, Hachinohe and Northridge earthquake excitations. While working back to inter-storey drifts obtained by the undamped structure (Fig. 5.9), it is evident, that these three earthquake excitations caused
clearly the higher interstorey drifts. In contrast, the efficiency of the dampers occurred under the Kobe and San Fernando earthquakes, which caused noticeably lower interstorey drifts, was obviously poor. The best performance occurred under the Hachinohe earthquake with average tip deflection reduction of 15.26%. The reductions experienced under the El Centro and Northridge earthquakes were slightly lower, while in contrast, unfavourable increase in average tip acceleration was recorded under the Kobe and San Fernando earthquake excitations.

The results of the same structure in terms of tip deflection reduction at different damper placements are illustrated in Fig. 5.12. From these results it is evident, that efficiency of the damper under the El Centro, Hachinohe and Northridge earthquakes was directly related to an increase in inter-storey drift. On the other hand, in the cases of the Kobe and San Fernando earthquakes, in which the structure experienced noticeably lower inter-storey drifts, the dampers were effective when embedded in the 1st or 4th storey, whereas, in the upper placements adverse increase in tip deflection was recorded. This may be attributed to inadequate compensation for removed stiffness at cut out and/or partial resonance of the damped structure and insufficient push on the friction damper to make it fully operational.

The tip deflection reductions for the structure embedded with three diagonal friction dampers fitted at six different placements are shown in Fig. 5.13. The
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overall results were, as it was expected, noticeably higher than for the structure embedded with one damper, nevertheless, the similar trends were experienced. The highest tip deflection reductions were obtained under the Hachinohe earthquake excitations and significantly high reductions also occurred under the El Centro and Northridge earthquakes. On the other hand, performance of the dampers under the Kobe and San Fernando earthquakes were, once again, unfavourable with noticeably increase in deflection. The reason may be, once again, attributed to inadequate compensation for removed stiffness at cut out and/or change in natural frequency of the structure.

Fig. 5.13 – 18 storey structure embedded with 3 diagonal friction dampers -percentage reductions in tip deflection under different earthquake records

Fig. 5.14 presents the results of tip deflection reduction in terms of different dampers placements. The results displayed similar trends as it was for the structure embedded with one damper. The highest average reduction was achieved by the structure with the dampers fitted in the top storeys, while the lowest average reduction was recorded for the structures with dampers placed in storeys 10 to 12. The results achieved under the El Centro, Hachinohe and Northridge earthquakes fully support Hanson’s theory (1993), which recommends placement of friction dampers at levels of maximum interstorey drift. On the other hand, in case of the Kobe and San Fernando earthquakes, an adequate efficiency was displayed only when the dampers were fitted in the lowest storeys.
The percentage reductions in the peak values of the tip acceleration for the structures fitted with the diagonal friction damper are shown in Fig. 5.15. The structure experienced a wide range of results. The highest average tip acceleration reduction of 11.95% was achieved under the Hachinohe earthquake. The reductions recorded under the El Centro and Northridge earthquakes were clearly lower and rather inconsistent, while the reductions occurred under the Kobe and especially under the San Fernando earthquakes were obviously unfavourable. Nevertheless, the increase in tip acceleration to a level lower than 10 m/s² (in this case, 5 m/s²) did not represent any significant decrease in structural safety.
In terms of damper placement (Fig. 5.16), tip acceleration reductions occurred under the El Centro earthquake were increasing when the damper was moved towards the top of the structure. The highest reductions for the Northridge earthquake was obtained when the damper was placed in the upper storeys, whereas, in the case of the Hachinohe earthquake, the highest reductions occurred when the damper was placed in the 1st or 10th storeys. In the cases of the Kobe and San Fernando earthquakes, clearly the poorest results, with strong increase in tip acceleration, was experienced for the upper storeys placements.

Fig. 5.16 – 18 storey structure embedded with 1 diagonal friction damper
-percentage reductions in tip acceleration for different damper placements

The tip acceleration reduction for structure embedded with three diagonal friction dampers are shown in Fig. 5.17.

Fig. 5.17 – 18 storey structure embedded with 3 diagonal friction dampers
-percentage reductions in tip acceleration under different earthquake records
The highest acceleration reduction of 18.22% was achieved under the Hachinohe earthquake. The reductions under the El Centro and Northridge earthquakes were only slightly lower. The reductions experienced under the Kobe earthquake were rather inconsistent and the poorest result with tip acceleration increase by 16.73% was recorded, yet again, under the San Fernando earthquake excitation.

In terms of efficiency of different damper placement (Fig. 5.18) the greatest tip acceleration reductions occurred when the dampers were placed in storeys 1 to 6 and 13 to 15, respectively. The tip acceleration reductions for the dampers placed in storeys 7 to 12 were rather inconsistent with some significant results. Clearly the worst results, with massive tip acceleration increases were, once again, experienced by the structure with the dampers placed in the top storeys. This tip acceleration increase was mainly due to the operating principle of the friction dampers, which caused transfer of acceleration to the ambient structural elements as well as decrease in stiffness of the top storeys due to the cut out in the shear wall.

Diagonal friction dampers embedded in the 18-storey structure were investigated in this section. The overall performance of the dampers was adequately high; nevertheless, wide range of results was experienced. The dampers achieved a high performance in both investigated parameters under earthquake excitations, which caused high deflections. In these excitations, tip deflection reductions were
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gradually increasing when the dampers were moved towards the storeys with the highest interstorey drifts. In contrast, the diagonal friction dampers were noticeably less effective under the earthquake excitations, which cause only small deflections. This may be attributed to inadequate compensation for removed stiffness at cut out and/or partial resonance of the damped structure and insufficient push on the friction damper to make it fully operational.

In terms of tip acceleration reduction, the efficiency of the dampers was rather inconsistent without any obvious trends. Significant tip acceleration reductions were experienced when the dampers were placed in storeys 1 to 6 and 13 to 15, respectively. In contrast, the results for other placements were relatively poor. The worst results with regard to increase in tip acceleration occurred for the uppermost storey placements. This was mainly due to the operating principle of the friction dampers, which caused transfer of acceleration to the ambient structural elements, as well as decrease in stiffness of the top storeys due to the cut out in the shear wall.

5.2.3 Diagonal VE dampers

The results of the tip deflection and tip acceleration experienced by the structures embedded with diagonal VE dampers are presented below.

Time history responses of tip deflection and tip acceleration under the El Centro earthquake excitation in the undamped structure and in the structures with the diagonal VE dampers placed in storeys 1 to 3 and 13 to 15, respectively, are illustrates in Fig. 5.19. The graphs show the high performance of the dampers embedded in the lowest storey, while their performance was clearly less significant when placed in the upper storeys. In addition to high tip deflection and tip acceleration reductions obtained by placing the dampers in the lowest storey, their even higher performance in terms of root mean square deflection and root mean square acceleration can be observed.
Fig. 5.19 – Tip deflection and acceleration responses of the structures with different placement of the diagonal VE dampers under the El Centro earthquake

Fig. 5.20 shows the tip deflection reduction for the structure embedded with the diagonal VE damper. The structures experienced reasonable overall performance with a few significant results. The highest average reduction of 4.91% occurred under the Kobe earthquake excitation. The average reduction for the El Centro earthquake was only slightly lower, whereas, the reductions obtained under the Hachinohe, Northridge and San Fernando earthquakes were rather insignificant.

Fig. 5.20 – 18 storey structure embedded with 1 VE diagonal damper -percentage reductions in tip deflection under different earthquake records

The results of the same structure with respect to damper placement are presented in Fig. 5.21. The damper obtained the highest performance when was placed in the lowest storey, while it performance gradually decreased when was moved towards the top of the structure. This trend was experienced under all earthquake...
excitations except for the San Fernando earthquake, where the highest tip
deflection reduction occurred when the damper was placed in the 7th storey.

The tip deflection reduction for the structure embedded with three diagonal VE
dampers are illustrated in Fig. 5.22. The highest average reduction of 19.27% was
obtained under the El Centro earthquake. The tip deflection reductions
experienced under the Kobe earthquake were, except for top placements, also
significantly high. On the other hand, the tip deflection reductions recorded for the
Northridge and Hachinohe earthquakes were a noticeably poorer. The lowest
average tip deflection reduction of 4.07% occurred, similarly as it was in the case
of one damper placements, under the San Fernando earthquake.
The results of tip deflection reduction in terms of damper placement are presented in Fig. 5.23. The best performance occurred when the dampers were placed in the lowest three storeys, while their subsequent repositioning towards the top storeys responded in a gradually decrease in tip deflection reduction under all earthquake excitations. According to a study conducted by Ashour (1987), the optimal placement of dampers should be the one that maximizes the damping ratio of the fundamental mode, as this mode’s contribution to the structure’s overall response is always significant. The results experienced under the all earthquake excitations confirmed the best performance experienced by the dampers placed in the lowest storeys, whereas, when they were moved towards the top of the structure a gradual decreased in their efficiency was experienced. The results for the diagonal VE dampers were in accordance with Ashour’s study.

![Fig. 5.23 – 18 storey structure embedded with 3 diagonal VE dampers -percentage reductions in tip deflection for different damper placements](image)

Fig. 5.24 shows the percentage reduction in tip acceleration for the structure embedded with the diagonal VE damper placed at 6 different locations. The highest average reduction of 11.21% was recorded under the Hachinohe earthquake. The reductions under the Northridge earthquake were also adequately high, while the reductions obtained under the Kobe, San Fernando and El Centro earthquakes were rather insignificant.
The results for percentage reductions in tip acceleration in terms of damper placement are shown in Fig. 5.25. The highest acceleration reductions occurred when damper was placed in the lowest storey, whereas, decrease in tip acceleration reductions was experienced as the damper was moved towards the top of the structure.

The tip acceleration reductions experienced by the structure embedded with three diagonal VE dampers are presented in Fig. 5.26. In overall, the dampers obtained a very high tip acceleration reduction. The highest average reduction of 22.37% was obtained under the Hachinohe earthquake. However, average reductions
occurred under the Northridge, Kobe and El Centro earthquakes were also a comparably high. Clearly the lowest tip acceleration reductions were, yet again, experienced under the San Fernando earthquake.

Fig. 5.26 – 18 storey structure embedded with 3 VE diagonal dampers
-percentage reductions in tip acceleration under different earthquake records

In terms of tip acceleration reduction at different damper placements (Fig. 5.27), the diagonal VE dampers achieved the highest reduction when placed in the lowest three storeys, while their moving towards the top of the structure caused decreases in tip acceleration reductions. This trend was in agreement with previously investigated structure fitted with one diagonal VE damper.

Fig. 5.27 – 18 storey structure embedded with 3 diagonal VE dampers
-percentage reductions in tip acceleration for different placements
The seismic responses of the 18-storey frame-shear wall structure embedded with one and three diagonal VE dampers, respectively, were investigated in this section. In general, the diagonal VE dampers revealed a high and consistent performance. However, the range of particular results was relatively wide. In terms of tip deflection reductions, the highest performance was obtained under the El Centro and Kobe earthquakes, while the reductions occurred under the Northridge and Hachinohe earthquake excitations were clearly lower. As regards to tip acceleration reduction, a reverse trend was experienced, when the best results occurred under the Hachinohe and Northridge earthquakes, whereas, the reductions under the Kobe and El Centro earthquakes were noticeably lower. The poorest results in both investigated parameters were recorded under the San Fernando earthquake. Based on the results presented in this section, it can be stated that the most characteristic feature experienced by diagonal VE dampers was their clearly highest performance when placed in the lowest storeys and a consequent decrease in efficiency, as they were moved towards the top of the structure.

While comparing the results of tip deflection reductions for diagonal VE dampers with those of the diagonal friction dampers, the diagonal VE dampers were consistent under all earthquake excitations, whereas, the diagonal friction dampers were rather ineffective under the Kobe and San Fernando earthquakes excitations. The best performance of the diagonal VE dampers was obtained when the dampers were fitted in the lowest storeys and gradually decreased as the dampers were moved towards the top of the structure. In contrast, the performance of the diagonal friction dampers increased when the dampers were moved towards the storeys with higher interstorey drifts.

In terms of tip acceleration reductions, diagonal friction dampers achieved better results under the El Centro earthquake, while diagonal VE dampers reached better results under other earthquakes. The both types of dampers produced the highest reductions when dampers were placed in the lowest storey. In the case of diagonal VE dampers, the acceleration reductions were regularly decreased as the dampers
were moved towards the uppermost storeys, while in the case of the diagonal friction dampers open range of the results was recorded.

5.2.4 Chevron brace friction dampers

The results of tip deflection and tip acceleration experienced by the structures fitted with chevron brace friction damper are presented below.

The time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with chevron brace friction dampers placed in storeys 1 to 3 and 13 to 15, respectively, are illustrate in Fig. 5.28. These graphs show moderate performance of the dampers. The seismic responses of the dampers in both placements was relatively close, however, the dampers fitted in the upper storeys revealed slightly better results in both investigated parameters.

![Fig. 5.28 – Tip deflection and acceleration responses of the structures with different placement of the chevron brace friction dampers under the El Centro earthquake](image)

The results of the percentage reductions in the peak values of the tip deflection experienced by the structures fitted with the chevron brace friction damper are illustrated in Fig. 5.29. The damper performed well, showing a high level of reliability and a slightly higher overall performance than it was for the structures fitted with the diagonal friction damper or the diagonal VE damper. The highest average tip deflection reduction of 9.79% was obtained under the Kobe earthquake. The average reductions experienced under the other earthquakes
oscillated in range from 3.62% under the Hachinohe earthquake to 5.70% under the San Fernando earthquake.

![Graph showing deflection reduction under different earthquakes](image)

**Fig. 5.29 – 18 storey structure embedded with 1 chevron brace friction damper -percentage reductions in tip deflection under different earthquake records**

The best performance with an average tip deflection reduction of 5.95% occurred when the damper was embedded in the 13th storey. However, as it can be seen in Fig. 5.30, the average tip deflection reductions obtained by the structure with the damper fitted in the other placements were within 1%.

![Graph showing deflection reduction for different placements](image)

**Fig. 5.30 – 18 storey structure embedded with 1 chevron brace friction damper -percentage reductions in tip deflection for different placements**

Tip deflection reductions occurred under the El Centro, Hachinohe and Northridge earthquakes increased as the damper was moved towards the
uppermost storey, whereas under the Northridge earthquake reverse trend can be observed. In the case of the Kobe earthquake, the damper was the most efficient in the lowest placements and the deflection reductions occurred for the middle and upper storey placements were constant.

Fig. 5.31 illustrates the tip deflection reduction for the structure fitted with three chevron brace friction dampers. The dampers obtained a high performance in the vast majority of cases. The average tip deflection reductions were confined in very narrow range from 6.96% occurred under the Hachinohe earthquake to 9.73% under the Northridge earthquake.

With respect to damper efficiency at different placements, two major trends can be observed. The first trend, which was observed under the Kobe and San Fernando earthquakes imply the highest damper efficiency occurred when the dampers were placed in the lowest storeys and their efficiency rapidly decreased as they were moved toward the top of the structures. The second trend, observed under the El Centro, Hachinohe and Northridge earthquake excitations, support the theory that the damper efficiency is increased when they are located in regions of large inter-storey drift. As it can be seen in Fig. 5.32, these two reverse trends resulted in extraordinary close average tip deflection reduction for all damper placements.
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Fig. 5.32 – 18 storey structure embedded with 3 diagonal VE dampers - percentage reductions in tip deflection for different placements

Fig. 5.33 shows the tip acceleration reduction for the structure embedded with the chevron brace friction damper. The highest average reduction of 10.09% was achieved under the El Centro earthquake. The reductions under the Northridge and San Fernando earthquake were inconsistent and less significant. The poorest results with increase in tip acceleration were recorded under the Hachinohe and Kobe earthquakes.

Fig. 5.34 shows the same results in terms of different damper placements. The highest average tip acceleration reductions occurred when the damper was placed
in the 4th storey, while, its repositioning towards the top or bottom of the structure caused increase in tip acceleration. The poorest result, with small increase in average tip acceleration occurred when the damper was placed in the 1st storey.

![Graph showing percentage reductions in tip acceleration for different placements](image1)

**Fig. 5.34 – 18 storey structure embedded with 1 chevron brace friction damper -percentage reductions in tip acceleration for different placements**

The tip acceleration reduction for the structure embedded with three chevron brace friction dampers are illustrated in Fig. 5.35. The highest average reduction of 10.35% was experienced under the Kobe earthquake. The average reductions recorded under the El Centro and Hachinohe earthquakes were slightly lower, whereas the reductions under the San Fernando and Northridge earthquakes were in the vast majority of cases relatively insignificant.

![Graph showing percentage reductions in tip acceleration under different earthquake records](image2)

**Fig. 5.35 – 18 storey structure embedded with 3 friction chevron brace dampers -percentage reductions in tip acceleration under different earthquake records**
As it can be seen from Fig. 5.36, the highest average tip acceleration reduction occurred when the dampers were placed in storeys 4 to 6. The tip acceleration reductions for the other damper placements were considerably lower and rather inconsistent. The poorest result, with the increase in tip acceleration was experienced when the dampers were fitted in the highest storeys. This increase in tip acceleration was mainly due to the operating principle of the friction dampers and additionally, by decrease of stiffness of the top storeys due to the cut-out in the shear wall.

An 18-storey frame-shear wall structure embedded with one or three chevron brace friction dampers were investigated. The results for tip deflection reduction, which was the main investigated parameters, revealed consistent and adequately high efficiency of the dampers under all earthquake excitations. In terms of damper placements, it can be seen, that performance of the dampers under the El Centro, Hachinohe and Northridge earthquakes i.e. excitations causing highest interstorey drifts (Fig. 5.9), was directly related to increase in interstorey drift. On the other hand, in the case of the Kobe and San Fernando earthquakes, reverse trend can be observed.

The results for tip acceleration reductions were comparative to those of tip deflection reductions only under the El Centro and Hachinohe earthquake. In the
case of the Kobe earthquake, significant tip acceleration reductions were obtained when the dampers were placed in the lower storey, while the reductions for other damper placements were rather insignificant. The tip acceleration reductions occurred under the Northridge and San Fernando earthquakes were inconsistent and in average insignificant.

While comparing the results of tip deflection reductions experienced by structure fitted with chevron brace friction dampers with those of the structure fitted with the diagonal friction dampers, the chevron brace friction dampers performed significantly better under the Kobe and San Fernando earthquakes, whereas the diagonal friction dampers performed clearly better under the El Centro. The highest difference in favour of the diagonal friction damper occurred under the Hachinohe earthquake excitation.

With regards to damper placements, both types of dampers revealed similar trends under the El Centro, Hachinohe and Northridge earthquakes, in which increase in efficiency occurred when the dampers were moved towards the storeys with a higher interstorey drifts. In the case of the Kobe and San Fernando earthquakes reverse trend occurred for the structures fitted with chevron brace friction dampers, while the results for the structure fitted with the diagonal friction dampers were rather inconsistent.

In comparison of the tip deflection reduction experienced by the structure fitted with the chevron brace friction dampers and the structure fitted with the diagonal VE dampers, the chevron brace friction dampers performed better under the Northridge and San Fernando earthquakes, whereas the diagonal VE dampers performed better under the Hachinohe, Kobe and particularly under the El Centro earthquakes. On the other hand, the structure fitted with diagonal VE dampers experienced a gradual decrease in tip deflection reduction when the dampers were moved from the lowest to the uppermost storeys under all earthquake excitations, the structure fitted with the chevron brace friction dampers experienced the
similar trend only under the Kobe and San Fernando earthquakes, while under other earthquake excitations reverse trend occurred.

In terms of tip acceleration reductions, the results of the structure fitted with the chevron brace friction dampers revealed noticeably lower reductions than the structure fitted with the diagonal friction dampers and diagonal VE dampers. The highest tip acceleration reductions occurred when the dampers were placed in the lower storeys, while the lowest reductions were recorded when the dampers were placed in the uppermost storeys. This was in agreement with the results of the diagonal friction and VE dampers.

5.2.5 Chevron brace VE dampers

The results of the tip deflection and tip acceleration experienced by the structures fitted with chevron brace VE damper are presented below.

Time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with the chevron brace VE dampers placed in storeys 1 to 3 and 13 to 15 respectively, are illustrated in Fig. 5.37. The graphs revealed relatively close and rather insignificant performance of the dampers fitted in both placements.

Fig. 5.37 Tip deflection and acceleration responses of the structures with different placement of the chevron brace VE dampers under the El Centro earthquake

Fig. 5.38 shows the tip deflection reduction for the structure embedded with the chevron brace VE damper placed at 6 different locations. The damper revealed a
good performance and high reliability. The highest average reduction of 8.12% occurred under the Kobe earthquake, while the deflection reductions for the other earthquakes were confined in narrow range from 4.07% under the El Centro earthquake to 4.70% under the Hachinohe earthquake.

![Graph showing percentage reductions in tip deflection under different earthquake records](image1)

**Fig. 5.38 – 18 storey structure embedded with 1 VE chevron brace damper - percentage reductions in tip deflection under different earthquake records**

Fig. 5.39 shows the results of the same structure in terms of efficiency of the damper at different placements.

![Graph showing percentage reductions in tip acceleration for different placements](image2)

**Fig. 5.39 – 18 storey structure embedded with 3 chevron brace friction damper - percentage reductions in tip acceleration for different placements**

Interestingly, the overall result followed the trend more typical for friction dampers i.e. the highest tip deflection reduction occurred in storeys with highest interstorey drift and was decreased when the dampers were moved towards the
bottom or the top of the structure. The results obtained under the El Centro, Hachinohe and Northridge earthquakes were in agreement with the overall trend, whereas in the case of the San Fernando earthquake, reverse trend was experienced. The tip deflection reductions occurred under the Kobe earthquake were consistent throughout all the placements.

Fig. 5.40 shows the tip deflection reductions for the structure embedded with three chevron brace VE dampers installed at 6 different placements. The highest average tip deflection reduction of 10.37% occurred under the Kobe earthquake. The average reduction obtained under the El Centro, Hachinohe and Northridge earthquakes were also adequately high. Clearly the lowest performance, with average reduction of 4.93%, occurred under the San Fernando earthquake.

As it can be seen from Fig. 5.41, in the cases of the Kobe and San Fernando earthquakes, the highest tip deflection reductions occurred when the dampers were placed in the lowest storeys, while strong decrease in tip deflection was experienced when the dampers were moved toward the top of the structures. This trend was in agreement with results of the structure embedded with the diagonal VE dampers. On the other hand, the reverse trend was experienced under the El Centro, Hachinohe and Northridge earthquake excitations. These two reverse trends resulted in remarkably narrow range of average tip deflection reductions at all placements.
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Fig. 5.41 – 18 storey structure embedded with 3 chevron brace VE dampers
-percentage reductions in tip deflection for different placements

Fig. 5.42 – 18 storey structure embedded with 1 VE chevron brace damper-
-percentage reductions in tip acceleration under different earthquake records

As can be seen from Fig. 5.43, the highest tip acceleration reductions were obtained when the damper was placed in the 4th storey. In contrast, the poorest result occurred when the damper was placed in the 1st storey. In the case of the El...
Centro, Northridge and San Fernando earthquakes the results did not followed any apparent trends. The worst results, with regular increase in tip accelerations from the uppermost to the lowest placement were obtained under the Hachinohe and Kobe earthquakes. Nevertheless, this trend was mostly caused by changes of structural stiffness due to cut outs, while the damper was essentially ineffective throughout the all placements.

![Graph showing acceleration reductions for different placements](image1.png)

**Fig. 5.43 – 18 storey structure embedded with 1 chevron brace VE damper -percentage reductions in tip acceleration for different placements**

Fig. 5.44 shows the results of tip acceleration reductions for the structure fitted with three chevron brace VE dampers.

![Graph showing acceleration reductions for different earthquake records](image2.png)

**Fig. 5.44 – 18 storey structure embedded with 3 chevron brace VE dampers -percentage reductions in tip acceleration under different earthquake records**
A highest average reduction of 10.94% was achieved under the El Centro earthquake. The tip acceleration reductions experienced under the other earthquake excitations were mostly insignificant.

As it can be seen from Fig. 5.45, the highest tip acceleration reduction occurred when dampers were placed in the lowest storeys, while a decrease in deflection reduction was experienced as the dampers were moved towards the top of the structures. Nevertheless, no particular trend was apparent under any earthquake excitations.

![Fig. 5.45 – 18 storey structure embedded with 3 chevron brace VE dampers -percentage reductions in tip acceleration for different placements](image)

Chevron brace VE dampers embedded in 18-storey frame-shear wall structure were investigated in this section. The results of tip deflection reduction revealed consistent and adequately high efficiency of the dampers. In the cases of the El Centro, Hachinohe and Northridge earthquakes, the highest tip deflection reductions were experienced when the dampers were placed in the uppermost storeys, whereas the reductions were gradually decreased when the dampers were moved towards the bottom of the structure. In the cases of the Kobe and San Fernando earthquake excitations, rather reverse trend can be observed.

Chevron brace VE dampers were created to represent 66.7% of the damping force of the diagonal VE dampers; however, their overall tip deflection reductions were
comparatively high. In addition, chevron brace VE dampers provided the higher reliability and readability than the diagonal dampers. Nevertheless, outstanding performance of the diagonal dampers occurred under the El Centro and Hachinohe earthquakes was not achieved.

With regards to tip acceleration reductions, high and consistent results occurred under the El Centro earthquake, while the results obtained under the other earthquake excitations were mostly insignificant. In terms of damper placements, the structure achieved the highest tip acceleration reductions when the dampers were placed in the lowest storeys and gradually decreased when the dampers were moved towards the top of the structure.

While comparing with other previously discussed damping systems, tip acceleration reductions obtained by the chevron brace VE dampers were noticeably lower. The highest tip acceleration reductions occurred when the dampers were placed in the lower storeys, while the lowest reductions were recorded when the dampers were placed in the uppermost storeys. This was in agreement with the results of diagonal friction and VE dampers.

5.2.6 Hybrid friction-VE dampers

The results of the tip deflection and tip acceleration experienced by the structures fitted with hybrid friction-VE dampers are presented below.

Fig. 5.46 shows time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with the hybrid friction-VE dampers placed in storeys 1 to 3 and 13 to 15, respectively. As it can be seen from these graphs, the dampers embedded in both placements achieved only moderate reductions of the seismic responses. The dampers placed in the upper storeys obtained slightly higher tip deflection reduction, while the dampers placed in the lowest storeys obtained clearly better results in terms of tip acceleration reduction.
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Fig. 5.46 Tip deflection and acceleration responses of the structures with different placement of the hybrid friction-VE dampers under the El Centro earthquake

The results of the tip deflection reduction for the structure embedded with the hybrid friction-VE damper are presented in Fig. 5.47. The damper revealed adequately high and consistent performance. The highest average tip deflection reduction of 8.47% occurred under the Kobe earthquake. The reductions obtained under the Northridge and San Fernando earthquakes were also adequately high. On the other hand, the tip deflection reductions occurred under the El Centro and Hachinohe earthquakes were mostly insignificant.

As can be seen from Fig. 5.48, the damper achieved clearly the best performance when placed in the 7th storey. Tip deflection reductions for the other placements were clearly less significant. In the case of the Kobe earthquake, consistent performance occurred at all placements, whereas the tip deflection reductions
recorded under the other earthquake excitations were rather complex without any obvious trends.

![Graph showing placement of dampers and percentage reduction in tip deflection](image)

**Fig. 5.48** – 18 storey structure embedded with 1 hybrid friction-VE damper - percentage reductions in tip deflection for different placements

Fig. 5.49 shows tip deflection reductions for the structure embedded with three hybrid dampers. The dampers obtained adequately high and stable overall performance. The highest average reduction of 9.92% was achieved under the Kobe earthquake. The deflection reductions occurred under the Northridge and San Fernando earthquakes were slightly lower, whereas the reductions experienced under the El Centro and Hachinohe earthquakes were only moderate.

![Graph showing earthquake records and percentage reduction in tip deflection](image)

**Fig. 5.49** – 18 storey structure embedded with 3 hybrid dampers - percentage reductions in tip deflection under different earthquake records

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As regards of efficiency of the dampers at different placements, from Fig. 5.50 it can be observed that the average tip deflection reductions were confined within exceptionally narrow range, and no preferable location was recognised. In the case of the El Centro, Hachinohe and Northridge earthquakes the highest average reductions occurred for the dampers placed in storeys 13 to 15, while in the cases of the Kobe and San Fernando earthquakes, the highest average reduction occurred when the dampers were placed in the lowest storeys.

![Graph](image)

**Fig. 5.50 – 18 storey structure embedded with 3 hybrid friction-VE dampers -percentage reductions in tip deflection for different placements**

The results of tip acceleration reduction for structure embedded with one hybrid friction-VE damper are presented in Fig. 5.51.

![Graph](image)

**Fig. 5.51 – 18 storey structure embedded with 1 hybrid damper -percentage reductions in tip acceleration under different earthquake records**
The best performance with the greatest average acceleration reduction of 26.83% was obtained under the El Centro earthquake, in contrast the reductions under the other earthquakes were in the vast majority of cases only miniscule.

Clearly the highest average tip acceleration reductions occurred when the damper was placed in the 7th storey (Fig. 5.52) and in contrast, the lowest reduction was recorded for the damper placed in the 1st storey. The range of the particular results obtained from all damper placements was widely open and no particular trends were recognised.

![Fig. 5.52 – 18 storey structure embedded with 1 hybrid friction-VE damper -percentage reductions in tip acceleration for different placements](image)

Fig. 5.53 shows the tip acceleration reduction of the structure fitted with three hybrid friction-VE dampers.

![Fig. 5.53 – 18 storey structure embedded with 3 hybrid dampers -percentage reductions in tip acceleration under different earthquake records](image)
The highest reduction of 15.30% was once again displayed for the El Centro earthquake excitation. Relatively high average reductions were also experienced under the Hachinohe and Kobe earthquakes, whereas the reductions occurred under the Northridge and San Fernando earthquakes were mostly inadequate.

As can clearly be seen in Fig. 5.54, the highest tip acceleration reduction was obtained by the structure with the dampers placed in storeys 4 to 6, while a gradual decrease in reductions was experienced when the dampers were moved towards the top or bottom of the structure. These results were in some cases rather reverse to results of tip acceleration reduction experienced by the structure fitted with one hybrid friction-VE damper.

![Fig. 5.54 – 18 storey structure embedded with 3 hybrid friction-VE dampers -percentage reductions in tip acceleration for different placements](image-url)

18-storey frame-shear wall structure embedded with the hybrid friction-VE dampers was investigated in this section. Based on the results, it can be seen that the expected high and reliable performance of the hybrid friction-VE dampers was achieved only partially. In terms of the tip deflection reduction, the structure fitted with three hybrid dampers experienced a consistent performance under all earthquake excitations. In the cases of the El Centro, Hachinohe and Northridge earthquakes the damper efficiency increased when the dampers were moved towards the storeys with higher interstorey drifts (only dampers placed in storeys 4 to 6 were slightly out of this trend). In the cases of the Kobe and San Fernando earthquake excitations rather reverse trends can be observed. The results for the
structure fitted with one damper were in majority of cases inconsistent and significant reductions were recorded only for the damper fitted in the 7th storey.

While comparing the hybrid friction-VE dampers with the other previously discussed damping systems, the hybrid dampers surpass the others under the San Fernando earthquake excitations. Their tip deflection reductions obtained under the Kobe and Northridge earthquakes were somewhere in the middle, whereas in the cases of the El Centro and Hachinohe earthquakes the hybrid dampers were the least effective among all investigated damping systems.

As regards to tip acceleration reductions for the hybrid dampers, the best overall results and also the most consistent acceleration reductions occurred under the El Centro earthquake excitations. In the case of the Hachinohe and Kobe earthquakes the high acceleration reductions were experienced for lower storey placements, while the results for the others placements were only moderate. The results occurred under the Northridge and San Fernando excitations were inconsistent and mostly insignificant. While comparing the tip acceleration reductions of the structures fitted with the hybrid dampers with reductions experienced by the structures fitted with other damping systems, it can be seen, that average tip acceleration reductions of hybrid dampers were somewhere in the middle under all earthquake excitations.

It is evident, that the structures embedded with a hybrid friction-VE dampers follow a trends close to these of the structures embedded with the chevron brace friction dampers. This means that whereas the friction component of the hybrid dampers operated appropriately, the VE component of the hybrid dampers remained essentially ineffective.
5.2.7 Lower toggle VE dampers

The results of the tip deflection and tip acceleration experienced by the structures fitted with lower toggle VE dampers are presented in this section.

Time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with the lower toggle VE dampers placed in storeys 1 to 3 and 13 to 15 respectively, are illustrates in Fig. 5.55. Higher performance in both studied parameters was obtained by the dampers embedded in storeys 13 to 15. It is worth noticing that while the seismic responses of the structure with the dampers fitted in storeys 13 to 15 displayed features more typical for VE dampers, the seismic responses of the structure with dampers fitted in the lowest storeys were closer to typical response of the structure fitted with friction dampers.

![Graphs showing tip deflection and acceleration responses](image)

Fig. 5.55 – Tip deflection and acceleration responses of the structures with different placement of the lower toggle VE dampers under the El Centro earthquake

Fig. 5.56 illustrates the tip deflection reductions for the structure embedded with the lower toggle VE damper placed at 6 different locations. The highest average reduction of 12.93% was obtained under the Kobe earthquake. The reductions occurred under the Northridge and San Fernando earthquakes were also an adequately high and relatively stable. On the other hand, the deflection reductions experienced under the El Centro and Hachinohe earthquakes were noticeably lower.
As can be seen in Fig. 5.57, the highest average tip deflection reduction and the most consistent performance was obtained by the damper placed in the 13th storey. The performance of the damper in the other placements was only slightly lower. The tip deflection reductions obtained under the Hachinohe earthquake gradually increased when the damper was moved from the bottom to the top of the structure, while, in the case of the San Fernando earthquake, reverse trend was experienced. The tip deflection reductions occurred under the other earthquakes were rather stable, without any apparent trends.

Fig. 5.57 – 18 storey structure embedded with 1 lower toggle VE damper -percentage reductions in tip deflection for different placements
The results for tip deflection reduction of the structure fitted with three lower toggle VE dampers are presented in Fig. 5.58. The dampers displayed very high performance with the features similar to those of the structure fitted with one lower toggle VE damper. The highest average reduction of 18.75% occurred under the Kobe earthquake. The reductions obtained under the El Centro, Northridge and San Fernando earthquakes were also significantly high and even the lowest average tip deflection reduction, recorded under the Hachinohe earthquake, was adequately high.

As it can be seen from Fig. 5.59 the highest average tip deflection reductions of 14.54% occurred when the dampers were placed in the lowest three storeys. The tip deflection reductions for other placements were only slightly lower. In the cases of the El Centro and Hachinohe earthquakes the highest average tip deflection reductions occurred when the dampers were placed in the uppermost storeys, while a gradual decrease in deflection reduction was experienced when the dampers were moved towards the bottom of the structure. A reverse trend occurred under the Kobe and San Fernando earthquakes. In the case of the Northridge earthquake, the performance remained relatively consistent at all damper placements.
The results of tip acceleration reduction for the structure embedded with the lower toggle VE damper (Fig. 5.60) were in contrast to results of tip deflection reductions. The greatest average acceleration reductions of 24.25% occurred under the El Centro earthquake. The tip acceleration reductions obtained under the Northridge and San Fernando earthquakes were also adequately high, while the acceleration reductions experienced under the Hachinohe and Kobe earthquake excitations were rather insignificant.

As can be seen from Fig. 5.61 the highest tip acceleration reductions occurred when the damper was fitted in the 4th storey. Consecutively, a slight decrease in acceleration
reduction was experienced, when the damper was moved towards the top or the bottom of the structure. The tip acceleration reductions occurred under the El Centro earthquake were significantly high in all placements, while the reductions experienced under the other earthquake excitations were rather inconsistent.

![Fig. 5.61 – 18 storey structure embedded with 1 lower toggle VE damper -percentage reductions in tip acceleration for different placements](image1)

The results of the tip acceleration reduction for the structure embedded with three lower toggle VE dampers are presented in Fig. 5.62. The highest average tip acceleration reduction of 21.18% occurred under the El Centro earthquake. The reductions obtained under the Kobe earthquake were also comparatively high. The tip acceleration reductions experienced under the other earthquake excitations were noticeably lower; nevertheless, still adequately high.

![Fig. 5.62 – 18 storey structure embedded with 3 lower toggle dampers- -percentage reductions in tip acceleration under different earthquake records](image2)
The greatest tip acceleration reductions were achieved when the dampers were placed in storeys 13 to 15 and 1 to 6, respectively. In contrast, an increase in average tip acceleration by 6.5% was experienced when the dampers were repositioned towards the uppermost storeys.

![Graph showing percentage reductions in tip acceleration for different placements.](image)

**Fig. 5.63 – 18 storey structure embedded with 3 lower toggle VE dampers**

- percentage reductions in tip acceleration for different placements

The results of the lower toggle VE dampers embedded in the 18-storey structure were presented in this section. The dampers revealed a very high performance in both investigated parameters and under all earthquake excitations. The highest tip deflection reductions of the structure embedded with three lower toggle VE dampers occurred when the dampers were placed in the lowest three storeys. The reductions for the other placements were only slightly lower. In the cases of the El Centro and Hachinohe earthquakes the highest average tip deflection reductions occurred when the dampers were placed in the uppermost storeys and were gradually decreased when the dampers were moved towards the bottom of the structure. A reverse trend was experienced under the Kobe and San Fernando earthquakes. In the case of the Northridge earthquake the performance remained rather consistent for all placements. Similar trends were also experienced for the structure fitted with one lower toggle VE damper.

In terms of tip acceleration reduction, the best results were experienced under the El Centro earthquake. The reductions under the Kobe earthquake were comparatively high, while the reductions occurred under the other earthquake
excitations were noticeably lower. Regards to placements, the greatest tip acceleration reductions occurred when the dampers were placed in storeys 13 to 15 and great results were also experienced for the structure with dampers placed in storeys 1 to 6. In contrast, an increase in average tip acceleration was experienced when the dampers were embedded in the uppermost storeys. This may be attributed to inadequate compensation for removed stiffness at cut out in location close to investigated point.

The VE damper was created to represent only 42% of the damping force of the diagonal VE damper. However, its overall performance was noticeably higher and also more reliable. The results for the reduction in tip deflection followed similar trends to those of the structures fitted with chevron brace friction dampers. With regard to reduction in tip acceleration the results comply with a trend, which is closest to that of the diagonal VE dampers.

To provide extra comparisons, structures embedded with the lower toggle friction dampers were also analysed. The results revealed a noticeable level of similarity to those of the lower toggle VE dampers and for that reason are not presented in this study; nevertheless, the time history graphs of both types of dampers make more obvious that amplifying force of toggle brace assembly altered damping response of friction (or VE) dampers to operate in a relatively similar way.

5.2.8 Summary of findings in the 18-storey structure

The results from the 18-storey frame-shear wall structure embedded with the six different damping systems installed at twelve different placements were investigated in this chapter. In general, the dampers obtained substantial reductions in the peak values of the tip deflection and tip acceleration, however, the range of the results was considerably wide.

The highest tip deflection reductions were recorded, as it was expected, for the lower toggle VE dampers. The results of these dampers reveal high levels
reliability under all excitations and when placed in all locations. The reductions for the diagonal VE damping system were even higher for the lower storey placements, however, their efficiency considerably decreased when placed towards the top of the structure. The most consistent performances in all placements and under all seismic excitations were displayed for both types of the chevron brace dampers.

The hybrid damping system acted in a similar way to that of the friction chevron brace damper, which indicates that only the friction part of this damping system was working properly, while the VE part remained rather ineffective. Lastly, the results of diagonal friction dampers reveal the highest sensitivity to placement and also to variations in seismic excitations. These dampers achieved the highest reductions under the Hachinohe earthquake, which caused the highest structural deflections from all excitations. On the other hand, in the case of the San Fernando earthquake, which caused the lowest structural deflection, involvement of the diagonal friction dampers was rather unfavourable. This may be attributed to inadequate compensation for removed stiffness at cut out and/or partial resonance of the damped structure and insufficient push on the friction damper to make it fully operational.

The overall results for the all damping system embedded in the 18-storey frame-shear wall structure in terms of reduction in the peak values of the tip deflection are illustrated in Figs. 5.64 -5.71.

Fig. 5.64 - Average percentage tip deflection reductions for all damping systems in all (1 damper) placements
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Fig. 5.65 - Average percentage tip deflection reductions for all damping systems (1 damper placement) under different earthquake excitations

Fig. 5.66 - Average percentage tip deflection reductions for all damping systems in term of damper placements

Fig. 5.67 - Average percentage tip deflection reductions of all damping systems under different earthquake excitations
Fig. 5.68 - Average percentage tip deflection reductions for all damping systems
in all (3 damper) placements

Fig. 5.69 - Average percentage tip deflection reductions for all damping systems
(3 dampers placement) under different earthquake excitations

Fig. 5.70 - Average percentage tip deflection reductions for all damping systems
in terms of damper placements
As regards of the peak values of the tip acceleration for the same structures, the highest reductions were recorded for the structures fitted with the diagonal VE dampers. The tip acceleration reductions of these dampers were greatest when placed in the lower parts of the structure. The average reductions for the lower toggle VE dampers were close to those of diagonal VE dampers; however, their reductions in the lower storeys were noticeably lower. The diagonal friction dampers displayed, once again, the highest sensitivity to variation of the dampers placement and seismic excitations. The slightly lower overall tip acceleration reduction was attributed to their ineffectiveness in the uppermost storeys and poor results experienced under the San Fernando earthquake.

The hybrid friction-VE dampers and the chevron brace friction damper followed similar trends with rather inconsistent acceleration reductions under the El Centro, Hachinohe and Kobe earthquakes, while the reductions experienced under the Northridge and San Fernando earthquakes were quite small. The lowest tip acceleration reductions were obtained by the chevron braced VE dampers where adequate reductions were recorded only under the El Centro earthquake excitation.

The overall results for all damping systems embedded in the 18-storey frame-shear wall structure in terms of reduction in the peak values of the tip acceleration are illustrated in Figs. 5.72-5.79.
Fig. 5.72 - Average percentage tip acceleration reductions for all damping systems in all (1 damper) placements

Fig. 5.73 - Average percentage tip acceleration reductions for all damping systems (1 damper placement) under different earthquake excitations

Fig. 5.74 - Average percentage tip acceleration reductions for all damping systems in terms of damper placement
Fig. 5.75 - Average percentage tip acceleration reductions for all damping systems under different earthquake excitations

Fig. 5.76 - Average percentage tip acceleration reductions for all damping systems in all (3 dampers) placements

Fig. 5.77 - Average percentage tip acceleration reductions for all damping systems (3 dampers placement) under different earthquake excitations
Fig. 5.78 - Average percentage tip acceleration reductions for all damping systems in term of damper placements

Fig. 5.79 - Average percentage tip acceleration reductions for all damping systems under different earthquake excitations

The results presented in this chapter imply the diagonal friction dampers experienced the best performance when placed in the storeys with the highest interstorey drift. On the other hand, the diagonal VE dampers obtained clearly the best results when placed in the lowest storeys. In the case of the hybrid friction-VE dampers anticipated high and reliable performance was accomplished only partially, when the higher consistency was achieved at the expense of lower efficiency. Based on this finding combined damping system consisting of the diagonal friction damper placed in the 16th storey and the diagonal VE damper placed in the 1st storey was also investigated. Figs. 5.80-5.84 show the time history response of the 18-storey frame-shear wall structure fitted with this
combined damping system compared with responses of the undamped structure and structure with the diagonal VE dampers embedded in the lowest three storeys.

**Fig. 5.80 - Tip deflection and acceleration responses of the undamped structure and the structures fitted with three diagonal VE dampers and the structure fitted with one friction damper and one VE damper respectively under the El Centro earthquake**

**Fig. 5.81 - Tip deflection and acceleration responses of the undamped structure and the structures fitted with three diagonal VE dampers and the structure fitted with one friction and one VE damper respectively under the Hachinohe earthquake**

**Fig. 5.82 - Tip deflection and acceleration responses of the undamped structure and the structures fitted with three diagonal VE dampers and the structure fitted with one friction and one VE damper respectively under the Kobe earthquake**
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For convenience the results of the combined damping system are also presented in Table 5.2.

**Table 5.2 - Percentage tip deflection and tip acceleration reduction of the structure fitted with one diagonal friction damper and one diagonal VE damper**

<table>
<thead>
<tr>
<th></th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S.Fernando</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Accel. Red.</td>
<td>15.55</td>
<td>25.85</td>
<td>14.80</td>
<td>13.28</td>
<td>8.82</td>
<td>15.66</td>
</tr>
</tbody>
</table>

As can be seen from the graphs, the structure fitted with the combined damping system obtained adequately high and consistent tip deflection and tip acceleration reductions under all earthquake excitations. In comparison with the responses of the structure fitted with the diagonal VE dampers, the combined damping system...
obtained the higher tip deflection reduction (which is more important parameter) under the Hachinohe, Northridge and San Fernando earthquakes, at the same time as in terms of tip acceleration reduction it was under the San Fernando earthquake. To emphasize these results, it should be point out that the combined damping system consist of only two dampers.

From the results presented in this chapter it can be seen that the friction dampers in the vast majority of cases suppressed the VE dampers in their ability to reduce the intensity of the initial strikes, while the advantage of the VE damper was in more gradual decreasing seismic response of the structure. While closely studying the time history responses of all damping systems it can be seen the combined damping system effectively involved the advantages of both damping mechanisms to obtain significant and consistent performance. In the light of these, it can be suggested, that the combined damping system, which include the diagonal friction damper fitted in the storey with maximum interstorey drift and the diagonal VE damper fitted in the storey the closest to the source of excitation can produce better and more reliable result than individual friction or VE dampers.

5.3 Seismic response of 18-storey frame structure

As it can be seen in the literature review (Chapter 2), a number of studies have investigated the use of damping devices within frame structure. On the other hand, instalment of the dampers in the cut outs of the shear wall is a relatively new method with only limited investigation. In order to better comprehend the findings from this, study an additional structural model represented by the 18-storey frame structure (designated by F) embedded with diagonal VE dampers (described in the Section 3.9) was also investigated. The dimensions of the 18-storey frame structure were kept the same as for previously discussed 18-storey frame-shear wall structure, except for substitution of the shear wall with an interior bay, which had the same dimensions as the exterior bays. Its results were compare with results of the 18-storey frame-shear wall.

Figs. 5.85-5.89 show tip deflection and tip acceleration under all five earthquake excitations in the structure of designation F1-3 with the diagonal VE dampers.
fitted in the lowest three storeys compared with the seismic responses in the undamped structure.

Fig. 5.85 – Tip deflection and acceleration responses of the undamped structure and the structures fitted with diagonal VE dampers under the El Centro earthquake

Fig. 5.86 – Tip deflection and acceleration responses of the undamped structure and the structures fitted with diagonal VE dampers under the Hachinohe earthquake

Fig. 5.87 – Tip deflection and acceleration responses of the undamped structure and the structures fitted with diagonal VE dampers under the Kobe earthquake
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Fig. 5.88 – Tip deflection and acceleration responses of the undamped structure and the structures fitted with diagonal VE dampers under the Northridge earthquake

Fig. 5.89 – Tip deflection and acceleration responses of the undamped structure and the structures fitted with diagonal VE dampers under the San Fernando earthquake

From these graphs it is evident that the dampers embedded in the lowest storeys of the frame structure in most cases significantly reduced the tip deflection and acceleration of the structure throughout the duration of the earthquakes. The best performance can be observed under the Northridge earthquake, at the same time as the poorest performance was displayed under the Hachinohe earthquake.

The results of peak values of tip deflection reduction for the 18-storey frame structure embedded with the diagonal VE damper are show in Fig. 5.90. The greatest performance with an outstanding average reduction of 30.5% was achieved under the Northridge earthquake. The high performance was also experienced under the El Centro earthquake. On the other hand, deflection reductions, which occurred under the other earthquake excitations, were insignificant.
Fig. 5.90 – 18 storey frame structure embedded with diagonal VE damper -percentage reductions in tip deflection under different earthquake records

Fig. 5.91 illustrates the tip deflection reduction for the same structures with respect to damper placement. The highest average tip deflection reduction of 13.2% occurred when the damper was fitted in the lowest storey. The reductions for the 4th storey, 7th storey and 10th storey placement were comparatively high, whereas the reductions for the upper storey placements were clearly lower.

The tip deflection reduction for the structure embedded with three diagonal VE dampers are presented in Fig. 5.92. The dampers revealed similar trends as in the case of one damper placement. Clearly the higher average tip deflection reductions of 34.4% occurred under the Northridge earthquake. The high tip
deflection reductions were also recorded under the El Centro earthquake. In contrast, the reductions obtained under the San Fernando earthquake were only moderate and the poorest performances occurred under the Kobe and Hachinohe earthquakes.

The average tip deflection reductions for different placements were by some means varied by significant results obtained under the Northridge and El Centro earthquakes; nevertheless, the highest tip deflection reduction occurred in the majority of the cases when the dampers were placed in storeys 4 to 6 and 7 to 9, respectively. The deflection reductions for the other placements were decreased as the dampers were moved towards the top or bottom of the structure.
The results of peak values of tip acceleration reduction for the same structure embedded with the diagonal VE dampers are shown in Fig. 5.94. Greatest average reduction of 31.0% was achieved under the San Fernando earthquake. High average reduction was also obtained under the Hachinohe earthquake. On the other hand, acceleration reductions recorded under the other earthquakes were obviously less significant.

**Fig. 5.94 – 18 storey frame structure embedded with diagonal VE damper -percentage reductions in tip acceleration under different earthquake records**

The highest average tip acceleration reduction of 18.5% was obtained when the damper was placed in the lowest storey (Fig. 5.95). The average reduction for the uppermost placement was only slightly lower. The tip acceleration reductions for the other placements were noticeably lower; nevertheless, still adequately high.

**Fig. 5.95 – 18 storey frame structure embedded with diagonal VE damper -percentage reductions in tip acceleration for different placements**
Tip acceleration reductions for the structure embedded with three dampers are presented in Fig. 5.96. The highest average tip acceleration reduction of 27.9% was, yet again, achieved under the San Fernando earthquake. The average reduction occurred under the Hachinohe earthquake was only slightly lower. The tip acceleration reductions recorded under the Northridge and El Centro earthquakes were also relatively high. On the other hand, reductions that occurred under the Kobe earthquake were, except for uppermost placement, relatively poor.

![Fig. 5.96 – 18 storey frame structure embedded with 3 diagonal VE dampers -percentage reductions in tip acceleration under different earthquake records](image)

Fig. 5.97 shows the tip acceleration reductions for the frame structure embedded with three dampers at 6 different placements.

![Fig. 5.97 – 18 storey frame structure embedded with 3 diagonal VE dampers -percentage reductions in tip acceleration for different placements](image)
Interestingly, the results revealed some trends, which are in disagreement with those for the structures with one damper placement. The lowest tip acceleration reductions occurred when the dampers were placed in the lowest storeys. The tip acceleration reductions for structures with damper placed in storeys 13 to 15 and 10 to 12, respectively, were slightly higher. The tip acceleration reductions for the structures with the dampers placed in storeys 4 to 6 and 7 to 9, respectively, were noticeably higher. Clearly the highest tip acceleration reductions were obtained when the dampers were placed in the uppermost storeys.

Diagonal VE dampers embedded in the 18-storey frame structure were investigated in this section. Overall, the dampers experienced a high performance in both investigated parameters. However, the range of the results was relatively wide. In terms of tip deflection reductions both types of the structure i.e. the structure fitted with one damper and the structure fitted with three dampers experienced similar trends. Clearly the highest deflection reductions were experienced under the Northridge earthquake. The tip deflection reductions recorded under the El Centro earthquake were also relatively high, whereas, the reductions occurred under the other earthquakes were only moderate. In both structures the dampers performed noticeably better when placed in the lower storeys, however, there was no particular placement with significantly the best results under all excitations.

With regards to tip acceleration reduction, both structures (i.e. structure fitted with one and three dampers) experienced similar trends. The best results were recorded under the San Fernando and Hachinohe earthquakes. The reductions occurred under the Northridge and El Centro earthquakes were noticeably lower. The poorest results occurred under Kobe earthquake. The highest tip acceleration reductions were experienced when the dampers were placed in the uppermost storey. This was anticipated, since the placement of the dampers in the top storey increased the stiffness in the area of measurement. In the case of the lowest storey placements the lowest tip acceleration reductions were obtained. This was also anticipated, since the dampers fitted in the lowest storeys increased the stiffness at the base, and thus, decreased the ductility of the structure, which caused detuning.
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of the structural natural frequency to the range of dominant frequencies of the earthquakes.

While comparing the results of the 18-storey frame structure embedded with the diagonal VE dampers with the results of the 18-storey frame-shear wall structure embedded with the same dampers, a number of different trends can be observed. In general, the dampers placed in the frame structure produced slightly better results in both investigated parameters. This was expected, because the structure did not experience a decrease of stiffness due to cut out of shear wall. For convenience the results of tip deflection and tip acceleration of both structures embedded with three diagonal VE dampers are, once again, shown in Figs. 5.98 and 5.99.

![Chart showing Tip Deflection Reductions in “H” and “F” Structures](image)

Fig. 5.98 – 18-storey frame-shear wall structure vs. 18-storey frame structure - percentage reductions in tip deflection under different earthquake records

In Fig. 5.98 it can clearly be seen that the frame structure obtained the higher tip deflection reductions under the Northridge earthquake excitation and that slightly better results were also experienced under the San Fernando earthquake. On the other hand, the frame-shear wall structure performed better under the Hachinohe, Northridge and, the most noticeably, under the Kobe earthquakes. Generally, it can be stated that in terms of tip deflection reduction under different earthquake excitations, the better overall performance was achieved by the dampers placed in the frame structure. On the other hand, the performance of the dampers placed in the frame-shear wall structure was slightly more consistent. As regards to
efficiency of the dampers in different placements, the dampers placed in frame–shear wall structure were highly effective in the lowest storeys and their efficiency was decreased when they were moved towards the top of the structure. In the case of frame structure, this trend was less obvious and the dampers revealed more stable performance throughout the all placements.

Fig. 5.99 shows the results of tip acceleration reduction obtained by the same structures. The dampers fitted in the frame structure achieved clearly higher tip acceleration reductions under the El Centro, Hachinohe and San Fernando earthquakes, whereas, in the cases of the Kobe and Northridge earthquakes, clearly higher tip acceleration reductions were achieved by the dampers fitted in the frame-shear wall structure. On the whole, tip acceleration reductions obtained by the dampers fitted in the frame-shear wall structures were slightly lower, but, once again, more consistent.

The frame structure embedded with three diagonal VE dampers experienced the highest tip acceleration reductions when the dampers were placed in the uppermost storey and the similar result was experienced for the structure embedded with one damper. This trend was in contrast with the results obtained from all previously examined structures for which the diagonal VE dampers obtained the highest acceleration reductions when placed in the lowest storeys,
whereas uppermost storeys placement caused an increase in tip acceleration. The main reason for this tip acceleration increase, experienced by previously investigated structures, can be attributed to decrease in stiffness of the top storeys due to the cut-out in the shear wall. Nevertheless, diversities in tip acceleration reductions obtained by the dampers in other placements were also obvious.
Chapter 6

Results – 12-Storey Structures
Chapter 6: Results - 12-storey structures

6.1 Introduction

The results from the finite element analysis of the 12 storey structures embedded with the different damping systems installed at different placements are presented in the following sections.

A 12-storey frame-shear wall structure described in Section 3.6.1 was modelled and analysed. A total of six different damping systems (Section 3.10), namely friction and VE diagonal dampers, friction and VE chevron brace dampers, hybrid friction-VE dampers and VE lower toggle dampers embedded within cut outs of shear walls of the structure, as shown in Fig. 3.12, were studied. These damping systems were placed within cut outs of shear wall at twelve different damper placements (Section 3.5.2) Seismic analyses were carried out with one type of damping system at one placement a time. The efficiency of these damping systems were investigated under the five different earthquake excitations.

6.2 Seismic responses of 12-storey frame-shear wall structure

The results presented in this section show the typical time history responses of the 12-storey structure embedded with different damping systems.

Figs. 6.1-6.5 illustrate the tip deflection and tip acceleration under five earthquake excitations in the structure of designation M 9-10 with the diagonal friction dampers fitted in storeys 9 and 10, compared with tip deflection and tip acceleration in the undamped structure. From these graphs, it is evident that the dampers embedded into the cut-outs of shear walls can significantly reduce the tip deflection and tip acceleration throughout the duration of the earthquakes. On the other hand, as it can be seen from the structural response recorded under the San Fernando earthquake excitation, instalment of the dampers can in same cases increase the seismic response of the structure.
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Fig. 6.1 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal friction dampers under the El Centro earthquake

Fig. 6.2 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal friction dampers under the Hachinohe earthquake

Fig. 6.3 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal friction dampers under the Kobe earthquake
Influence of Damping Systems on Building Structures Subject to Seismic Effects

Fig. 6.4 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal friction dampers under the Northridge earthquake

Fig. 6.5 - Tip deflection and tip acceleration response of the undamped structure and the structure embedded with diagonal friction dampers under the S. Fernando earthquake

Figs. 6.6-6.8 show tip deflection and tip acceleration under the El Centro earthquake excitation for the same 12-storey frame-shear wall structure embedded six different damping systems.

Fig. 6.6 shows the tip deflection and tip acceleration responses of the 12-storey structure with the diagonal friction and diagonal VE dampers placed in storeys 9 and 10. The results for the 18-storey structure, discussed in previous chapter, demonstrated superiority of the diagonal friction dampers over the diagonal VE dampers when placed in the upper storeys. As it can be seen in this figure, the 12-storey structure embedded with the same dampers treated under the El Centro excitations showed the results, which were in agreement with the results for 18-storey frame-shear wall structure.
Fig. 6.6 - Tip deflection and tip acceleration responses of the structures embedded with diagonal friction and VE dampers under the El Centro earthquake

Fig. 6.7 shows tip deflection and tip acceleration responses of the structures embedded with the chevron brace friction and chevron brace VE dampers under the El Centro earthquake excitation. The seismic responses of the dampers in both investigated parameters were relatively close. The friction dampers performed slightly better in the first half of the excitation, i.e. during the strong seismic excitation, while the VE dampers were better performing during smaller excitation.

The results discussed in the previous chapter, demonstrated the better ability of friction dampers to reduce the intensity of the initial strong motion, while advantage of the VE dampers was in gradually decreasing the tip deflection and tip acceleration of the structure. However, as it can be observed from Fig. 6.7 and from many other results presented latter in this chapter, these trends were in the case of the 12-storey structure, noticeably less obvious.

Fig. 6.7 - Tip deflection and tip acceleration responses of the structures embedded with Friction and VE chevron brace dampers under the El Centro earthquake
Tip deflection and tip acceleration responses of the structures embedded with the hybrid friction-VE dampers and the structure embedded with the lower toggle VE dampers respectively, are illustrated in Fig. 6.8. Both types of dampers achieved good overall performance. The structure fitted with the hybrid dampers obtained slightly higher tip deflection reduction; whereas the structure fitted with the lower toggle VE dampers achieved better results in terms of tip acceleration reduction.

![Graph showing tip deflection and acceleration responses of the structures embedded with Hybrid damping systems and VE lower toggle dampers under the El Centro earthquake](image)

**Fig. 6.8 - Tip deflection and acceleration responses of the structures embedded with Hybrid damping systems and VE lower toggle dampers under the El Centro earthquake**

### 6.2.1 Undamped structure

The undamped structural model was created in order to compare its results with the results of the structures fitted with the damping systems. The results of the tip deflection and tip acceleration of this structure obtained under five earthquake excitations are presented in Table 6.1

<table>
<thead>
<tr>
<th></th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S.Fernando</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection (m)</td>
<td>0.206</td>
<td>0.374</td>
<td>0.154</td>
<td>0.145</td>
<td>0.141</td>
</tr>
<tr>
<td>Acceleration (m/s²)</td>
<td>5.69</td>
<td>6.61</td>
<td>6.93</td>
<td>5.76</td>
<td>3.51</td>
</tr>
</tbody>
</table>

Fig. 6.9 shows the interstorey drifts of the undamped 12-storey structure under five earthquake excitations. The highest interstorey drifts were displayed under the Hachinohe earthquake with the maximum interstorey deflection of 0.0396 m. The second highest interstorey drifts were experienced under the El Centro
earthquake with the maximum interstorey deflection of 0.0226 m. This was followed by the Kobe earthquake with the maximum interstorey deflection of 0.0173 m and the San Fernando earthquake with the maximum interstorey deflection of 0.0170 m. The lowest interstorey drifts occurred under the Northridge earthquake with the maximum interstorey deflection of 0.0149 m. From the Fig. 6.9 it can also be seen that the maximum interstorey drifts were displayed between storeys 10 and 11. This was followed by drifts between the storeys 9 and 10, then 11 and 12, 8 and 9, 7 and 8, etc.

![Interstorey Drifts of Undamped Structure](image)

*Fig. 6.9 - Interstorey drifts of undamped structure under the five earthquake excitations*

### 6.2.2 Diagonal friction dampers

The results of the tip deflection and tip acceleration experienced by the structures embedded with the diagonal friction damper are presented below.

Fig. 6.10 illustrates time history responses of tip deflection and tip acceleration under the El Centro earthquake excitation in the undamped structure and in the structures with the diagonal friction dampers placed in storeys 1 and 2, and 9 and 10, respectively. In these graphs it can be seen that the dampers in both placement produced clearly better results than undamped structure. While comparing seismic responses for both damper placements, the dampers embedded in the storeys 9 to 10 with higher interstorey drifts produced slightly better results in tip deflection,
which is the primary investigated parameter, while in terms of tip acceleration, clearly the better result occurred for the dampers fitted in the lowest storeys. In addition, it is worth noticing, that both responses remained generally in phase with responses of undamped structure.

Fig. 6.10 Tip deflection and acceleration responses of the structures with different placement of the diagonal friction dampers under the El Centro earthquake

Fig. 6.11 illustrates the results of the percentage reductions in the peak values of the tip deflection obtained by the structures fitted with the diagonal friction damper at six different locations under five earthquake excitations. The damper obtained the greatest performance under the El Centro earthquake. The performance of the damper achieved under the Kobe, Northridge and San Fernando earthquakes were, except for the lowest placement, also very high. On the other hand, the performance under the Hachinohe earthquake was unexpectedly poor.

Fig. 6.11 – 12 storey structure embedded with 1 diagonal friction damper -percentage reductions in tip deflection under different earthquake records
In terms of damper placements (Fig. 6.12) the highest tip deflection reductions occurred when the damper was placed in the 11th storey, and a regular decrease in deflection reduction was experienced as the damper was moved towards the bottom of the structure. This trend was relatively regular under the Northridge and San Fernando earthquakes, whereas the trends occurred under the El Centro and Kobe earthquakes, these trend were more complex. In the case of the Hachinohe earthquake adequate tip deflection reductions occurred only for the lower storey placements.

The percentage reductions in the peak values of the tip deflection obtained by the structures fitted with the three diagonal friction dampers are presented in Fig. 6.13.

![Diagram showing percentage reductions in tip deflection for different damper placements](image1)

**Fig. 6.12 – 12 storey structure embedded with 1 diagonal friction damper -percentage reductions in tip deflection for different damper placements**

![Diagram showing percentage reductions in tip deflection under different earthquake records](image2)

**Fig. 6.13 – 12 storey structure embedded with 2 diagonal friction dampers -percentage reductions in tip deflection under different earthquake records**
The dampers display a very high efficiency in the vast majority of the cases. The highest average tip deflection reduction of 27.86% was achieved under the El Centro earthquake. The reductions occurred under the Northridge and San Fernando earthquakes were also adequately high, while tip deflection reductions experienced under the Hachinohe and Kobe earthquakes were clearly less significant.

The results of tip deflection reduction in terms of efficiency of the dampers in different placements are illustrated in Fig. 6.14. The dampers placed in the lowest storeys produced in the majority of cases, only minor tip deflection reductions. On the other hand, tip deflection reductions significantly increased as the dampers were moved towards the storeys with the higher interstorey drifts. This trend was, yet again, relatively regular under the Northridge and San Fernando earthquakes, while, in the cases of the El Centro and Kobe earthquakes this trend was less obvious.

Fig. 6.14 – 12 storey structure embedded with 2 diagonal friction dampers -percentage reductions in tip deflection for different damper placements

Fig. 6.15 shows the tip acceleration reduction for 12-storey structure embedded with the diagonal friction damper. While this structure revealed very good overall performance in terms of tip deflection reduction, the results of the same dampers in terms of tip acceleration reductions were rather unfavourable. High reductions were experienced under the El Centro and Kobe earthquakes, while tip acceleration reductions occurred under the other earthquake excitations were
mostly unfavourable. The poorest result with strong increase in average tip acceleration by 16.12% was recorded under the San Fernando earthquake.

As can be seen from Fig. 6.16, the poorest result with increase in average tip acceleration occurred for the lowest damper placement, while its repositioning towards the storeys with higher interstorey drifts resulted in increase in tip acceleration reduction. On the other hand, in the case of the El Centro and San Fernando earthquake excitations rather reverse trend can be observed.

The results of tip acceleration reduction for structures fitted with two diagonal friction dampers are illustrated in Fig. 6.17. The highest tip acceleration reductions occurred under the El Centro earthquake. This was followed by, still
adequately high, acceleration reductions occurred under the Northridge and Kobe earthquakes. On the other hand, the results recorded under the Hachinohe and San Fernando earthquake excitations were unfavourable.

The highest tip acceleration reductions can be observed when the dampers were placed in the lowest storeys (Fig. 6.18), while a gradual increase in tip acceleration occurred as the dampers were moved towards the top of the structure.
structures (with one damper placement and with two damper placement) experienced significantly high tip deflection reduction under the El Centro, Northridge and San Fernando earthquake excitations, while the reductions occurred under the Hachinohe and Kobe earthquakes were less significant. In terms of efficiency of the dampers in different placements, the results, once again, confirmed previous finding, which imply that the diagonal friction dampers placed in the lowest storeys produced, in the majority of cases, only minor tip deflection reductions. On the other hand, the reductions significantly increased, when, the dampers were moved towards the storeys with the higher interstorey drifts.

The results for tip acceleration reductions were rather inconsistent. The highest tip acceleration reductions were obtained under the El Centro earthquake excitations. The acceleration reductions experienced under the Northridge and Kobe earthquakes were also adequately high, on the other hand, the reductions occurred under the Hachinohe and San Fernando earthquakes were unfavourable. The highest tip acceleration reductions were achieved when the dampers were placed in the lowest storeys and were gradually decreased as the dampers were moved towards the top of the structure.

While comparing these results with the results of the diagonal friction dampers fitted in the 18-storey frame-shear wall structure, several different trends can be observed. In terms of tip deflection reduction, the biggest difference occurred under the Hachinohe earthquake. Based on the results obtained from the 18-storey structure, it was anticipated that the 12-storey structure fitted with the diagonal friction dampers should achieve a significant performance primarily under earthquake excitations with higher interstorey drifts. However, as it can be seen from Figs. 6.11- 6.14 the performance of dampers fitted in the 12-storey structure under the Hachinohe excitations was rather insignificant.

The diverse damper performance was also experienced under the San Fernando earthquake, where high deflection reductions achieved by the dampers embedded
in the 12-storey structure were in contrast with massive increase in tip deflection experienced by the same dampers fitted in the 18-storey structure.

In terms of tip acceleration reductions the highest difference in damper efficiency was, yet again, experienced under the Hachinohe earthquake, when, the dampers placed in 18-storey structure experienced significant tip deflection reductions, while the dampers fitted in the 12-storey structure were rather ineffective. As regards to efficiency of the dampers in different placements, the trends experienced in the 12-storey and 18-storey structures were rather reverse under all earthquake excitations.

6.2.3 Diagonal VE dampers

The results of the tip deflection and tip acceleration reduction experienced by the 12-storey structures embedded with the diagonal VE damper are presented below.

Time history responses of tip deflection and tip acceleration under the El Centro earthquake excitation in the undamped structure and in the structures with the diagonal VE dampers placed in storeys 1 and 2 and 9 and 10, respectively, are illustrated in Fig. 6.19.

From these graphs it can be seen that the structures embedded with the diagonal VE dampers produced clearly better results than undamped structure. While
comparing an efficiency of the dampers embedded in the two different placements, than clearly a higher efficiency, in both investigated parameters, was experienced by the dampers fitted in the lowest storeys. These results were in agreement with previous findings.

Fig. 6.20 shows the tip deflection reductions for the structure embedded with the diagonal VE damper. The greatest average tip deflection reduction of 17.47% was achieved under the El Centro earthquake excitation. The tip deflection reductions occurred under the other earthquakes were in the vast majority of cases only moderate.

Fig. 6.21 shows the damper efficiency in different placements. In general, the highest deflection reduction occurred when the damper was positioned in the middle storeys, while its repositioning towards the top or bottom of the structure resulted in decrease in tip deflection reduction. This trend can be observed under the Hachinohe, Northridge and San Fernando earthquakes. In the case of the Kobe earthquake an efficiency of the damper was regularly increasing when the damper was moved from the lowest storey towards the top of the structure, while under the El Centro earthquake excitation rather reverse trend can be observed.
Percentage reductions in tip deflection for the same structures fitted with two diagonal VE dampers are illustrated in Fig. 6.22. The greatest average reduction of 29.6% occurred under the El Centro earthquake. The average tip deflection reduction obtained under the Kobe earthquake was also reasonably high. On the other hand, the reductions recorded under the San Fernando and Hachinohe earthquakes were only moderate. The poorest results with only small tip deflection reductions occurred under the Northridge earthquake.
Fig. 6.23 shows high average tip deflection reductions for the structure with two diagonal VE dampers located in the lower and middle storeys, whereas placing the dampers in the uppermost storeys resulted in noticeably lower performance. In general, the structure displayed only a small sensitivity to damper placements and tip deflection reductions for the lower and middle storeys placement were within an exceptionally narrow range.

![Graph showing % Deflection Reduction for different damper placements](image)

**Fig. 6.23 – 12 storey structure embedded with 2 diagonal VE dampers -percentage reductions in tip deflection for different damper placements**

Fig. 6.24 shows the tip acceleration reduction for the structure embedded with the diagonal VE damper.

![Graph showing % Acceleration Reduction under different earthquake records](image)

**Fig. 6.24 – 12 storey structure embedded with 1 diagonal VE damper -percentage reductions in tip acceleration under different earthquake records**
The highest average reduction of 7.38% occurred under the El Centro earthquake. The reductions obtained under the Kobe, Hachinohe and San Fernando earthquakes were noticeably lower. The poorest result and unfavourable increase in average tip acceleration was experienced under the Northridge earthquake.

As can be seen in Fig. 6.25, the highest acceleration reduction occurred for the damper placed in the lowest storeys and a decrease in acceleration reduction occurred when the damper was moved towards the top of the structure. This trend can be observed under the El Centro, Hachinohe and San Fernando earthquakes, while, the results obtained under the Kobe and Northridge earthquakes were rather complex and did not reveal any obvious trend.

![Graph showing acceleration reductions for different damper placements](image)

**Fig. 6.25 – 12 storey structure embedded with 1 diagonal VE damper -percentage reductions in tip acceleration for different damper placements**

The tip acceleration reductions obtained by the structure embedded with two diagonal VE dampers are presented in Fig. 6.26. The highest average reduction of 18.10% was, yet again, achieved under the El Centro earthquake. The reductions occurred under the Hachinohe and Kobe earthquakes were also adequately high, whereas, the reductions obtained under the San Fernando earthquake were clearly lower. The poorest result with increase in tip acceleration was recorded under the Northridge earthquake excitation.
In terms of damper placements (Fig. 6.27) the highest tip acceleration reductions occurred for the dampers placed in the lowest storeys, while, when the dampers were moved towards the top of the structure a gradual increase in tip acceleration was recorded. This trend can be observed under the El Centro and Hachinohe earthquakes. In the case of the San Fernando earthquake, the highest reductions occurred for the middle storey placements, while under the Kobe earthquake the reductions for the middle storey placements were the lowest. The poorest reductions occurred under the Northridge earthquake excitations.

The diagonal VE dampers embedded in the 12-storey frame-shear wall structure were investigated in this section. In terms of tip deflection reductions the extraordinary performance was experienced under the El Centro earthquake. High
tip deflection reductions occurred also under the Kobe earthquake, while the reductions recorded under the other earthquakes were obviously less significant. In terms of tip acceleration reductions, the best results were, yet again, achieved under the El Centro earthquake excitation, while in contrast, the poorest result with an increase in tip acceleration was experienced under the Northridge earthquake.

The results for 12-storey structure embedded with the diagonal VE dampers, once again, confirmed trends, which indicated that the highest tip deflection and tip acceleration reductions occurred when the dampers were placed in the lowest storeys and gradually decreased as the dampers were moved towards the top of the structure.

While comparing these results with the results of the diagonal VE dampers embedded in the 18-storey structure, in both types of structure the dampers clearly gave the highest tip deflection reductions under the El Centro and Kobe earthquakes. The dampers embedded in the 18-storey structure obtained the highest tip deflection reductions when placed in the lowest storey and a regular decrease in deflection reductions occurred as they were moved towards the top of the structure. In the case of the 12-storey structure consistency and regularity of this feature was less obvious. Furthermore, in the cases of the Kobe and Northridge earthquakes, rather reverse trends were experienced. In terms of tip acceleration reduction, the diagonal VE dampers embedded in the 12-storey structure achieved higher reductions under the El Centro earthquake, while the diagonal VE dampers embedded in the 18-storey structure achieved the higher tip acceleration reduction under the other earthquakes.
6.2.4 Chevron brace friction dampers

The results of tip deflection and tip acceleration experienced by the structures fitted with the chevron brace friction damper are presented below.

Fig. 6.28 display the time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with the chevron brace friction dampers placed in storeys 1 and 2, and 9 and 10, respectively. The graph of tip deflection illustrates a high performance of the dampers placed in the upper storeys, while the dampers placed in the lowest storeys were rather ineffective. In terms of tip acceleration the better results were, yet again, displayed for the dampers placed in the upper storeys. Nevertheless, the dampers fitted in both placements were not able to effectively reduced intensity of tip acceleration related to initial strong motions.

The results of the percentage reductions in the peak values of the tip deflection experienced by the structures fitted with the chevron brace friction damper are illustrated in Fig. 6.29. The performance of the damper under the Hachinohe earthquake was unfavourable. On the other hand, the damper performed effectively and reliably under the other earthquake excitations. The highest average tip deflection reduction of 13.83% occurred under the Kobe earthquake and the average tip deflection reductions under the El Centro, Northridge and San Fernando earthquakes were only slightly lower.
Fig. 6.29 – 12 storey structure embedded with 1 chevron brace friction damper -percentage reductions in tip deflection under different earthquake records

Fig. 6.30 shows high performance of the damper in all placements. The lowest tip deflection reductions occurred when the damper was fitted in the lowest storey, while a increase in reductions was experienced when the damper was moved towards the storeys with higher interstorey drifts. This trend was displayed under the El Centro, Northridge and San Fernando earthquakes, while the deflection reductions experienced under the Kobe earthquake were very consistent.

Fig. 6.30 – 12 storey structure embedded with 1 chevron brace friction damper -percentage reductions in tip deflection for different placements

The percentage reductions in tip deflection for the structures fitted with two chevron brace friction dampers are illustrated in Fig. 6.31. The greatest performance, with an average deflection reduction of 21.5%, was obtained under the El Centro earthquake. This was followed by still adequately high reductions.
recorded under the Kobe, Northridge and San Fernando earthquakes. Conversely, the reductions occurred under the Hachinohe earthquake were rather insignificant.

From Fig. 6.32, it can be seen that the highest average tip deflection reduction occurred for the structures with dampers placed in storeys 7 and 8. As the dampers were moved towards the top or bottom of the structure, slight decreases in efficiency were experienced. This trend was, yet again, displayed under the El Centro, Northridge and San Fernando earthquakes, while the deflection reductions occurred under the Kobe and Hachinohe earthquakes were rather constant.
The results of tip acceleration reduction for the 12-storey structure embedded with the chevron brace friction damper are illustrated in Fig. 6.33. A significant average tip acceleration reductions of 16.71% occurred under the Kobe earthquake. The results obtained under the other earthquake excitations were unfavourable with an increase in tip acceleration. The massive increase in average tip acceleration by 50.45% was recorded under the San Fernando earthquake. As mentioned previously, the reduction in structural tip deflection is primary purpose for incorporation of the dampers to the structure, while the increase in tip acceleration to a level lower than 10 m/s² (in this case 5 m/s²) did not represent any decrease in structural safety.

Fig. 6.33 – 12 storey structure embedded with 1 chevron brace friction damper - percentage reductions in tip acceleration under different earthquake records

Fig. 6.34 shows the tip acceleration reductions experienced by the same structure in terms of efficiency of the dampers in different placements. Overall results were strongly influenced by the results occurred under the El Centro and San Fernando earthquakes, in which strong increase in tip acceleration was recorded for all the placements. In the case of the Kobe earthquake tip acceleration reductions were gradually increased when the damper was moved towards the storeys with higher interstorey drifts. On the other hand, the results displayed under the other earthquake excitations seem to be more influenced by decrease in stiffness due to cut-outs in shear wall than by an ability of the damper to decrease tip acceleration.
Percentage reductions in tip acceleration for the structure embedded with two chevron brace friction dampers are illustrated in Fig. 6.35. The results display relatively high acceleration reductions under the Kobe earthquake. However, the reductions occurred under the other earthquakes were considerably poor. Clearly the worst results occurred under the San Fernando earthquake, where increase in average tip acceleration by 54.6% was experienced. However, even this increase (to 5.44 m/s² for uppermost placement) did not represent any decrease in structural safety.

As can be seen from Fig. 6.36 an increase in tip acceleration was recorded for all damper placements. The lowest tip acceleration increase was recorded when
dampers were placed in storeys 5 and 6, while, the highest increase occurred when the dampers were placed in the uppermost storeys. Nevertheless, the tip acceleration reductions of chevron brace friction damper seem to be, yet again, more influenced by decrease in the stiffness due to cut-outs in shear wall than by ability of the dampers to reduce tip acceleration.

\[
\begin{array}{cccccc}
\text{EC} & \text{HA} & \text{KO} & \text{NO} & \text{SF} & \text{Avg} \\
\text{Avg} & -16.68 & -11.59 & -6.22 & -17.25 & -16.31 \\
\text{Avg} & -32.3 & & & & \\
\end{array}
\]

\(\text{Fig. 6.36} \quad \text{12 storey structure embedded with 2 chevron brace friction dampers} \)

The chevron brace friction dampers embedded in the 12-storey frame-shear wall structures were investigated in this section. The results for tip deflection reduction, revealed adequately high efficiency of the dampers. The highest and the most consistent performance in terms of tip deflection reductions were achieved under the Kobe earthquake excitations. The reductions obtained under the El Centro, Northridge and San Fernando earthquakes were also adequately high. On the other hand, the efficiency of the dampers under the Hachinohe earthquake was rather poor. In terms of damper placements, the performance of the dampers under the El Centro, Northridge and San Fernando earthquakes regularly increased when the dampers were moved from bottom to the top of the structure. In the case of the Kobe earthquake the performance was relatively consistent.

In terms of tip acceleration reduction, the chevron brace friction dampers obtained very poor results. The tip acceleration reductions, which occurred under the Kobe earthquake excitations, were adequately high and relatively consistent. However,
the results obtained under the other earthquake excitations were considerably poorer. The worst results with strong increase in tip accelerations were recorded under the San Fernando earthquake. Nevertheless, even this increase in tip acceleration to the value of 5 m/s², did not represent any significant decrease in structural safety/performance and these dampers were efficient in reducing tip deflection and hence improve overall structural performance.

The overall reductions in tip deflection for chevron brace friction dampers were comparatively high to those of the diagonal friction dampers and diagonal VE dampers embedded in the 12-storey structure. The dampers displayed a slightly lower level of sensitivity to the placement and a relatively higher consistency than the diagonal dampers. On the other hand, in terms of tip acceleration reduction, the efficiency of the chevron brace friction dampers was considerably poorer with a significant increase in tip acceleration in a majority of the cases.

While comparing the results of tip deflection reduction for chevron brace friction dampers embedded in the 12-storey frame-shear wall structure, with the reductions experienced by the same dampers embedded in the 18-storey frame-shear wall structure, the dampers fitted in the 18-storey structure were obviously more effective under the Hachinohe earthquake. In contrast, the dampers fitted in the 12-storey structure were more effective under the other earthquakes. In the case of the El Centro and Northridge earthquakes, both structures revealed a regular increase in tip deflection reductions when the dampers were moved from the bottom towards the top of the structure, whereas no common trends were observed under the other earthquake excitations.

As mentioned previously tip acceleration reduction for the 12-storey structures embedded with the chevron brace friction dampers were considerably poor with adequate results achieved only under the Kobe earthquake. In contrast, the same dampers embedded in the 18-storey structure experienced relatively high tip acceleration reductions in the majority of the cases.
6.2.5 Chevron brace VE dampers

The results of the tip deflection and tip acceleration reductions experienced by the structures fitted with the chevron brace VE damper are presented below.

Time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with the chevron brace VE dampers placed in storeys 1 and 2, and 9 and 10, respectively, are illustrated in Fig. 6.37. This figure illustrate significant reduction in tip deflection experienced by the dampers fitted in the upper storeys, whereas, the dampers remained rather ineffective when fitted in the lowest storeys. In terms of tip acceleration response the better results were, once again, obtained by the dampers placed in the upper storeys, however, as it can be observed from this graphs, the dampers, in both placements, were not able to reduced intensity of tip acceleration related to initial strong motions.

Fig. 6.37- Tip deflection and acceleration responses of the structures with different placement of the chevron brace VE dampers under the El Centro earthquake

The results of the percentage reductions in the peak values of the tip deflection experienced by the structures fitted with the chevron brace VE damper are illustrated in Fig. 6.38. The damper revealed an adequately high overall performance. The highest average tip deflection reduction of 14.95% was obtained under the Kobe earthquake. The average tip deflection reductions occurred under the El Centro, Northridge and San Fernando earthquakes were also adequately high, whereas the performance of the damper occurred under the
Hachinohe earthquake was obviously poor with insignificant results in all the placements.

**Fig. 6.38** – 12 storey structure embedded with 1 chevron brace VE damper -percentage reductions in tip deflection under different earthquake records

Fig. 6.39 shows relatively regular increase in damper efficiency when the damper was moved from bottom to the top of the structure. This trend occurred under the El Centro, Northridge and San Fernando earthquakes, whereas in the case of the Kobe earthquake reverse trend can be observed.

**Fig. 6.39** – 12 storey structure embedded with 1 chevron brace VE damper -percentage reductions in tip deflection for different placements

Tip deflection reductions experienced by the same structures fitted with two chevron brace VE dampers are displayed in Fig. 6.40. The best performance with
an average reduction of 18.18% was achieved under the Kobe earthquake. The reductions occurred under the El Centro, Northridge and San Fernando earthquakes were also reasonably high, whereas the reductions recorded under the Hachinohe earthquake were, once again, only minuscule.

![Graph showing percentage reductions in tip deflection under different earthquake records](image)

**Fig. 6.40** – 12 storey structure embedded with 2 chevron brace VE dampers - percentage reductions in tip deflection under different earthquake records

Fig. 6.41 shows the results of tip deflection reduction in the different damper placements. The dampers experienced a gradual increase in efficiency when were moved from the bottom to the top of the structures. This overall trend was clearly demonstrated under the El Centro, Northridge and San Fernando earthquakes, whereas the tip deflection reductions obtained under the Hachinohe and Kobe earthquakes were rather constant throughout all the placements.

![Graph showing percentage reductions in tip deflection for different placements](image)

**Fig. 6.41** – 12 storey structure embedded with 2 chevron brace VE dampers - percentage reductions in tip deflection for different placements
The results of tip acceleration reduction for the 12-storey structure embedded with the chevron brace VE damper are illustrated in Fig. 6.42. The reductions occurred under the Kobe earthquake were very consistent and adequately high. On the other hand, unfavourable results with increase in tip acceleration were recorded under the other earthquake excitations. The poorest results with a massive increase in average tip acceleration by 36.02% was experienced under the San Fernando earthquake. However, the increase in tip acceleration to a level, in this case, lower than 5 m/s² did not represent any decrease in structural safety/performace.

Fig. 6.42 – 12 storey structure embedded with 1 chevron brace VE damper -percentage reductions in tip acceleration under different earthquake records

Fig. 6.43 shows the same results in terms of different damper placements.

Fig. 6.43 – 12 storey structure embedded with 1 chevron brace VE damper -percentage reductions in tip acceleration for different placements
High increase in tip acceleration was recorded in the vast majority of placements. Nevertheless, this increase was the lowest when the damper was placed in the 3rd storey and a gradual increase was experienced, when the damper was moved towards the top or bottom of the structure. This trend can be observed under the El Centro and San Fernando earthquakes, whereas, the trends experienced under the other earthquake excitations were more complex.

Fig. 6.44 illustrates tip acceleration reduction for the 12-storey structure embedded with two chevron brace VE dampers. The acceleration reductions under the Kobe earthquake were, yet again, adequately high; nevertheless, the results, which occurred under the other earthquake excitations, were evidently poor. Clearly the poorest result, with an increase in the average tip acceleration by 36.6%, was experienced under the San Fernando earthquake. However, this increase did not influence the level of the structural safety.

In terms of damper placement (Fig. 6.45), the lowest acceleration increase was recorded when the dampers were placed in storeys 5 and 6, while their repositioning towards the top or bottom of the structure caused gradual increase in tip acceleration. This pattern can be, yet again, observed under the El Centro and San Fernando earthquakes, whereas the reverse trend occurred under the Kobe earthquake. In the cases of the Hachinohe and Northridge earthquakes no particular trend were recognised.
Chevron brace VE dampers embedded in 12-storey frame-shear wall structure were investigated in this section. The results of both investigated parameters were very close to those of chevron brace friction dampers. In terms of tip deflection reduction the dampers experienced a high overall performance. The dampers similarly to chevron brace friction dampers remained rather ineffective under the Hachinohe earthquake. In contrast, the highest and most consistent tip deflection reductions were, yet again, achieved under the Kobe earthquake excitations. In terms of damper placement some features more common for friction dampers were experienced. Prime such feature was a regular increase in performance when the dampers were moved from the bottom towards the top of the structure. This can be observed under the El Centro, Northridge and San Fernando earthquakes.

The results of tip acceleration reduction experienced by the 12-storey structure fitted with the chevron brace VE dampers were noticeably poorer. The tip acceleration reductions occurred under the Kobe earthquake excitation were adequately high and consistent. In contrast, the results obtained under the other earthquake excitations were an obviously poor. The worst result with a strong increase in tip acceleration was recorded for the San Fernando earthquake. However, even this poor result did not influence the level of the structural safety and the dampers, efficient in reducing tip deflection, were improving overall structural performance.
Tip deflection reductions experienced by the chevron brace VE dampers embedded in the 12-storey structure were adequately high compared to those of the diagonal friction or diagonal VE dampers, moreover, chevron brace VE dampers were more consistent and less sensitive to damper placements. In terms of tip acceleration reduction, the chevron brace VE dampers experienced unfavourable results, what was in contrast with a high acceleration reductions obtained by the diagonal dampers.

While comparing tip deflection reductions experienced by the chevron brace VE dampers fitted in the 12-storey frame-shear wall structures, with the reductions recorded for the chevron brace VE dampers embedded in the 18-storey frame-shear wall structures, the dampers embedded in the 18-storey structure were obviously more effective under the Hachinohe earthquake. In contrast, the dampers embedded in the 12-storey structure were more effective under the other earthquakes. In the cases of the El Centro and Northridge earthquakes, both structures revealed a regular increase in tip deflection reductions when the dampers were moved from the bottom towards the top of the structure. On the other hand, under the other earthquake excitations no common trends were experienced.

Tip acceleration reduction obtained by the 12-storey structure embedded with chevron brace VE dampers were unfavourable, when adequate results occurred only under the Kobe earthquake. The tip acceleration reduction obtained by the 18-storey structure embeded with the same dampers were, except for the El Centro earthquake, inconsistent and mostly insignificant; however, evidently better than those of the 12-storey structure.

6.2.6 Hybrid friction-VE dampers

The results of the tip deflection and tip acceleration reductions experienced by the structures fitted with the hybrid friction-VE dampers are presented below.
Fig. 6.46 shows time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with hybrid friction-VE dampers placed in storeys 1 and 2, and 9 and 10, respectively. The graph illustrates substantial reduction in tip deflection obtained by the dampers in both placements; nevertheless, tip deflection reduction obtained by the dampers placed in the lower storeys was slightly higher. In terms of tip acceleration, the better results was, also, displayed for the dampers placed in the lowest storeys, however, the dampers fitted in both placements only slightly reduced intensity of acceleration related to initial strong motions.

The results of the tip deflection reduction for the structure embedded with the hybrid friction-VE damper are presented in Fig. 6.47. The best performance with average reduction of 11.69% was achieved under the El Centro earthquake. The reductions occurred under the other earthquakes were noticeably lower and without any significant results.

Fig. 6.47 – 12 storey structure embedded with 1 hybrid friction-VE damper -percentage reductions in tip deflection under different earthquake records
The same results in terms of efficiency of the damper in different placements are presented in Fig. 6.48. The highest overall tip deflection reduction occurred when the damper was embedded in the 3\textsuperscript{rd} storey and the reductions regularly decreased as the damper was moved towards the top or bottom of the structure. This trend can be observed under the Hachinohe and El Centro earthquakes, while under the Northridge and San Fernando earthquake excitations, reverse trend was experienced. In the case of the Kobe earthquake, the poorest result occurred when the damper was placed in the 5\textsuperscript{th} storey and a regular increase in tip deflection reduction occurred as the damper was moved towards the top or bottom of the structure.

![Graph showing percentage reductions in tip deflection for different placements](image)

\textit{Fig. 6.48 – 12 storey structure embedded with 1 hybrid friction-VE damper -percentage reductions in tip deflection for different placements}

The percentage reductions in the peak values of the tip deflection experienced by the structures fitted with two hybrid friction-VE dampers are illustrated in Fig. 6.49. In general, the dampers experienced relatively high and very consistent performance. The best results were, once again, recorded under the El Centro earthquake with an average tip deflection reduction of 18.35\%. Adequately high reductions were also experienced under the Kobe earthquake. The performance of the dampers under the other earthquakes was noticeably poorer and the lowest average reduction of 6.5\% was recorded under the Northridge earthquake excitation.
In terms of efficiency of the dampers in different placements, the results presented in Fig. 6.50 show a consistent performance for the dampers fitted in the all placements. Nevertheless, the dampers fitted in the lower storeys were slightly more efficient. The tip deflection reductions obtained under the El Centro and Kobe earthquakes increased progressively as the dampers were moved from the top to the bottom of the structures, while, in the case of the Northridge and San Fernando earthquakes reverse trend was experienced. The tip deflection reductions occurred under the Hachinohe earthquake were relatively constant.

Fig. 6.49 – 12 storey structure embedded with 2 hybrid friction-VE dampers -percentage reductions in tip deflection under different earthquake records

Fig. 6.50 – 12 storey structure embedded with 2 hybrid friction-VE dampers -percentage reductions in tip deflection for different placements
The results of tip acceleration reduction for the structure embedded with the hybrid friction-VE damper are presented in Fig. 6.51. The best result with the greatest average tip acceleration reduction of 17.16% was obtained under the Hachinohe earthquake. In contrast, the tip acceleration reductions occurred under the Kobe, Northridge and San Fernando earthquakes were in the vast majority of the cases only miniscule. The poorest results with increase in tip acceleration, was experienced under the El Centro earthquake excitation.

Fig. 6.51 – 12 storey structure embedded with 1 hybrid friction-VE damper -percentage reductions in tip acceleration under different earthquake records

Fig. 6.52 presents the results of the same structure in terms of tip acceleration reduction in different damper placements.

Fig. 6.52 – 12 storey structure embedded with 1 hybrid friction-VE damper -percentage reductions in tip acceleration for different placements
The highest average reductions occurred when the damper was placed in the 9th storey, while, the lowest reduction was recorded when the damper was placed in the 1st storey. Nevertheless, the range of the particular results for different placements was inconsistent and no obvious trends were recognised.

Fig. 6.53 presents the percentage reduction in tip acceleration for the same structure fitted with two hybrid dampers. The highest average reduction of 19.62% occurred, once again, under the Hachinohe earthquake. The reductions under the Northridge and Kobe earthquakes were noticeably lower, although still adequately high. The lowest tip acceleration reductions occurred under the El Centro and San Fernando earthquakes.

The highest average tip acceleration reductions occurred when the dampers were installed in the storeys 3 and 4. In contrast, the least effective placement was in the top storeys. The same trend was experienced under the Hachinohe earthquake and partly under Kobe earthquake. In the case of the Northridge earthquake the lowest reduction occurred, yet again, for the highest storey placement, while the highest reduction was obtained when the dampers were placed in storeys 9 and 10. In the case of the El Centro earthquake, the dampers were effective only in the lowest storeys. Finally, the tip acceleration reductions obtained under the San Fernando earthquake were inconsistent and without any obvious trends.
Hybrid friction-VE dampers embedded in the 12-storey frame-shear wall structure were investigated in this section. The dampers experienced consistent and reliable performance under all excitations. The highest tip deflection reductions occurred under the El Centro earthquake, while the poorest reductions were experienced under the Northridge earthquake. The dampers revealed a consistent performance in all placements; nevertheless, their efficiency in the lower storeys was slightly higher.

In terms of tip acceleration reductions, the greatest reductions were achieved under the Hachinohe earthquake, while the reductions experienced under the El Centro and San Fernando earthquakes were relatively poor. In terms of tip acceleration reduction in different placements, the structure embedded with one damper experienced the best results when the damper was placed in the 9th or 11th storey, while the structure with two dampers placement obtained the highest tip acceleration reductions when the dampers were placed in storey 3 and 4, and 9 and 10, respectively.

While comparing the performance of the hybrid friction-VE dampers embedded in the 12-storey frame-shear wall structures with the performance of the same dampers embedded in 18-storey frame-shear wall structures, a number of different features experienced under all earthquake excitations can be observed. In terms of
tip deflection reductions the dampers embedded in the 12-storey structures performed better under the El Centro, Hachinohe and Kobe earthquake excitations. In contrast, the dampers embedded in the 18-storey structures performed slightly better under the Northridge and San Fernando earthquakes. With regards to damper placements both types of the structures experienced similar trends under the Kobe earthquake and partially under the Northridge earthquake. In contrast, rather reverse trends were experienced under the El Centro and San Fernando earthquakes, while the trends occurred under the Hachinohe earthquake were without any common features.

In terms of tip acceleration reductions, diversity in results experienced by both types of structures was even higher and no common features were observed under any earthquake excitations. The dampers embedded in the 12-storey structure achieved higher tip acceleration reductions under the Hachinohe and Kobe earthquake excitations. On the other hand, the dampers embedded in the 18-storey structure obtained higher tip acceleration reductions under the San Fernando and El Centro earthquake excitations. The dampers embedded in the 18-storey structure obtained clearly the highest tip acceleration reductions when placed in storeys 4 to 6, while the dampers embedded in the 12-storey structure experienced relatively constant reductions in most of the placements.

While comparing the hybrid friction-VE dampers with the other damping systems embedded in the 12-storey structure, the hybrid dampers experienced tip deflection and tip acceleration reductions, which were close to average under the all earthquake excitations.

The results for hybrid friction-VE dampers embedded in the 12-storey structure were relatively close to those of the 12-storey structure fitted with the VE diagonal dampers. Based on the presented results, it can be suggested, that whereas, the VE part of the hybrid damper operated appropriately, the friction part remained less effective. The high efficiency of the diagonal VE part and the low efficiency of the chevron brace friction part of the hybrid damper in the 12-storey structure were in contrast to the results of the hybrid friction-VE damper fitted in
the 18-storey frame-shear wall structures (where only the chevron brace friction dampers operated effectively). A contrast in performances makes it obvious that creating a highly effective hybrid friction-VE damper is rather complex and so requires a more comprehensive study.

6.2.7 Lower toggle VE dampers

The results of the tip deflection and tip acceleration experienced by the structures fitted with lower toggle VE dampers are presented in this section.

Time history responses of tip deflection and tip acceleration under the El Centro earthquake in the undamped structure and in the structures with the lower toggle VE dampers placed in storeys 1 and 2, and 9 and 10, respectively, are illustrated in Fig. 6.55.

The graphs illustrate good performance of the dampers in both placements and in both investigated parameters. In terms of peak value of tip deflection, clearly better result occurred for the structure with the dampers placed in the lowest storeys, while in terms of peak value of tip acceleration slightly better results were experienced by the structure with the dampers placed in the upper storeys.

The tip deflection reductions for the structure embedded with the lower toggle VE damper placed at 6 different locations are presented in Fig. 6.56. As can be seen in this figure, the damper revealed very high and consistent performance. The
highest average tip deflection reduction of 15.29% occurred under the El Centro earthquake. The reductions obtained under the other earthquakes were also adequately high and consistent.

Fig. 6.56 – 12 storey structure embedded with 1 lower toggle damper -percentage reductions in tip deflection under different earthquake records

Fig. 6.57 shows the result for tip deflection reduction in terms of different placements.

The average tip deflection reduction for the lower three damper placements were exceptionally close. In addition, similarly narrow range of average tip deflection reductions can be observed for the three upper placements. These close ranges were based on two reverse trends. In the case of the El Centro, Hachinohe and
partially Kobe earthquakes the highest tip deflection reductions occurred for the lowest storey placement and the reductions were decreased as the damper was moved towards the top of the structure. On the other hand, in the cases of the Northridge and San Fernando earthquake excitations rather reverse trend was experienced.

The percentage reduction in tip deflection for structures fitted with two lower toggle VE dampers are presented in Fig. 6.58. The dampers exhibit a high and consistent performance. The highest average reduction of 21.83% was achieved under the El Centro earthquake. The high average deflection reductions were also obtained under the other earthquakes, when even the lowest average reduction, occurred under the Northridge earthquake, represented value of 11.86%.

![Graph showing percentage reductions in tip deflection under different earthquake records](image)

**Fig. 6.58 – 12 storey structure embedded with 2 lower toggle dampers -percentage reductions in tip deflection under different earthquake records**

In terms of efficiency of the dampers in different placements (Fig. 6.59), the highest average tip deflection reduction occurred for the structure with dampers fitted in storeys 3 and 4; nevertheless, the reductions obtained by the structure with the dampers fitted in the other placements were also very high. The tip deflection reductions recorded under the Northridge and San Fernando earthquakes gradually increased when the dampers were moved from the bottom to the top of the structures, whereas under the El Centro earthquake the reverse trend occurred. In the case of the Kobe earthquake, the dampers performed
significantly better in the lower storeys, while under the Hachinohe earthquake, the performance was rather consistent for all the placements.

Fig. 6.59 – 12 storey structure embedded with 2 lower toggle VE dampers -percentage reductions in tip deflection for different placements

Fig. 6.60 shows the results of tip acceleration reduction for structure embedded with the lower toggle VE damper.

Fig. 6.60 – 12 storey structure embedded with 1 lower toggle damper -percentage reductions in tip acceleration under different earthquake records

The greatest average tip acceleration reductions of 21.74% was achieved under the Hachinohe earthquake. The tip acceleration reductions obtained under the Kobe and Northridge earthquakes were still relatively high. On the other hand, the reductions occurred under the San Fernando earthquake were rather unfavourable.
Surprisingly, the poorest results with slight increase in tip acceleration was experienced under the El Centro earthquake excitation.

As can be seen from Fig. 6.61 the highest tip acceleration reductions occurred when the damper was fitted in the 9th storey. Adequately high reductions were also achieved when the damper was fitted in storey 3 or 5, whereas the tip acceleration reductions obtained for the other damper placements were clearly less significant.

The percentage reductions in tip acceleration for the structures embedded with two lower toggle VE dampers are presented in Fig. 6.62.

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**Fig. 6.61 – 12 storey structure embedded with 1 lower toggle VE damper**

- percentage reductions in tip acceleration for different placements

**Fig. 6.62 – 12 storey structure embedded with 2 lower toggle dampers**

- percentage reductions in tip acceleration under different earthquake records
The dampers produced significant overall tip acceleration reduction. However, a wide range of results was experienced. The average tip acceleration varied from a 3.4% increase occurred under the San Fernando earthquake to significant 21.8% tip acceleration reduction obtained under the Hachinohe earthquake.

Fig. 6.63 shows the results of structural tip acceleration reduction in terms of different damper placements. The highest average reduction occurred when the dampers were fitted in storeys 9 and 10. The tip acceleration reductions for the structure with the dampers fitted in the lower storeys were also very high, while, in contrast, a significant increase in average tip acceleration was experienced, when the dampers were placed in the uppermost storeys. Based on previous results, it is obvious that this acceleration increase was caused by decrease in top storey stiffness due to cut out.

The lower toggle VE dampers embedded in the 12-storey structure were investigated in this section. The dampers revealed very high and consistent performance. In terms of tip deflection reduction, the highest damper efficiency was achieved under the El Centro earthquake. However, the reductions obtained under the other earthquake excitations were also significantly high. With regards to tip acceleration reductions, the greatest reductions were achieved under the Hachinohe earthquake. The reductions occurred under the Kobe and Northridge earthquakes were also very high. On the other hand, in the cases of the El Centro
and San Fernando earthquakes, the results were inconsistent with a strong increase in tip acceleration for the uppermost storeys placement.

The result presented in this section confirmed superiority of the lower toggle VE dampers among all investigated damping systems embedded in the 12-storey structure. The lower toggle VE dampers experienced the highest performance in both investigated parameters under the Hachinohe, Kobe and Northridge earthquake excitations and their performance obtained under the El Centro earthquake was only slightly lower than it was for the diagonal friction and diagonal VE dampers. In the case of the San Fernando earthquake, high tip deflection reduction was, yet again, recorded, while in terms of tip acceleration reduction poor results were experienced.

While comparing these results with the results obtained by the lower toggle VE dampers fitted in the 18-storey frame-shear wall structures some different features can be observed. The 12-storey structure experienced the highest tip deflection reduction under the El Centro earthquake, whereas its tip acceleration reductions under this earthquake excitation were rather insignificant. In the case of the 18-storey structures reverse results were experienced. The 12-storey structures treated under the Hachinohe earthquake revealed a high and consistent performance in both investigated parameters, whereas the performances of the 18-storey structures were rather unreliable. In the 18-storey structures tip deflection reductions obtained under the San Fernando earthquake were gradually decreasing when the dampers were moved towards the top of the structure, while under the El Centro earthquake reverse trend was recorded. In the case of the 12-storey structures reverse trends can be observed.

6.2.8 Summary of findings in the 12-storey structure

The results from the 12-storey frame-shear wall structure embedded with the six different damping systems installed in twelve different placements were investigated in this chapter. In terms of reduction in the peak values of tip deflection, which is primary investigated parameter, substantial reductions were
In terms of tip acceleration reduction the range of the results was noticeably wide and some unfavourable results were experienced; nevertheless, even this increase in tip acceleration did not represent any decrease in structural safety/performance and these dampers were efficient in reducing tip deflection, and in improving overall structural performance.

The highest tip deflection reductions and the most consistent performances were recorded for the lower toggle VE dampers. The tip deflection reductions for the diagonal friction dampers were generally comparable to those of the lower toggle VE dampers; nevertheless, in few cases this remained rather ineffective. Slightly lower overall deflection reductions for the chevron brace friction dampers and chevron braced VE dampers than those of the diagonal friction dampers were due to their poor performances under the Hachinohe earthquake. The widest range of results was displayed for the diagonal VE dampers. However, their overall reduction also remained at a satisfactory high level. The reductions for the hybrid damping system were comparable to those of the other damping systems; however, its results suggested that, whereas the VE part of hybrid damping system operated effectively, the friction part of this damping system was less effective.

The overall results for the all damping systems embedded in the 12-storey frame-shear wall structure in terms of reduction in the peak values of the tip deflection are shown in Figs. 6.64-6.71.

Fig. 6.64 - Average percentage tip deflection reductions for all damping systems in all (1 damper) placements
Influence of Damping Systems on Building Structures Subject to Seismic Effects

Fig. 6.65 - Average percentage tip deflection reductions for all damping systems (1 damper placement) under different earthquake excitations

Fig. 6.66 - Average percentage tip deflection reductions for all damping systems in term of damper placements

Fig. 6.67 - Average percentage tip deflection reductions of all damping systems under variety of earthquake excitations
Influence of Damping Systems on Building Structures Subject to Seismic Effects

Fig. 6.68 - Average percentage tip deflection reductions for all damping systems in all (2 dampers) placements

Fig. 6.69 - Average percentage tip deflection reductions for all damping systems (2 dampers placement) under different earthquake excitations

Fig. 6.70 - Average percentage tip deflection reductions for all damping systems in terms of damper placements
While in terms of tip deflection reduction for the 12-storey structures, all damping systems performed exceptionally well, the results in terms of the peak values of tip acceleration reduction were considerably poorer. The highest reductions were recorded for the lower toggle VE dampers. However, even these dampers remained unreliable under the San Fernando earthquake. The reductions for the diagonal VE dampers were rather uneven with an increase of tip accelerations under the Northridge earthquake. The reductions for the hybrid friction-VE damper were generally high, except for the El Centro and San Fernando earthquakes, where slight increases in acceleration were recorded. The tip acceleration reductions of the diagonal friction dampers were uneven and in many cases rather insufficient. The poorest results with high increase in tip acceleration were recorded for the chevron brace VE dampers and chevron brace friction dampers, which performed effectively only under the Kobe earthquake.

The overall results of all damping systems for the 12-storey frame-shear wall structure in terms of reduction in the peak values of the tip acceleration are illustrated in Figs. 6.72-6.79.
Fig. 6.72 - Average percentage tip acceleration reductions for all damping systems in all (1 damper) placements

Fig. 6.73 - Average percentage tip acceleration reductions for all damping systems (1 damper placement) under different earthquake excitations

Fig. 6.74 - Average percentage tip acceleration reductions for all damping systems in terms of damper placement
Influence of Damping Systems on Building Structures Subject to Seismic Effects

Fig. 6.75 - Average percentage tip acceleration reductions for all damping systems under variety of earthquake excitations

Fig. 6.76 - Average percentage tip acceleration reductions for all damping systems in all (2 dampers) placements

Fig. 6.77 - Average percentage tip acceleration reductions for all damping systems (2 dampers placement) under different earthquake excitations
The results presented in this chapter, yet again, confirmed that in terms of tip deflection reduction, which was the more important investigated parameter, the friction dampers performed more efficiently when placed in the storeys with higher interstorey drift, while the VE dampers were more efficient when placed close to source of excitations. The hybrid dampers experienced consistent performance in the all placements. With respect to these findings, similarly as in the case of the 18-storey frame-shear wall structure, the combined damping system, which consisted of the diagonal friction damper placed in the 11th storey and the diagonal VE damper placed in the 1st storey was also investigated.
The time history responses of the 12-storey structure fitted with the combined damping system compared with the responses of the undamped structure and also with the responses of the structure with the diagonal VE dampers fitted in the lowest two storeys are illustrated in Figs. 6.80-6.84. As can be seen from the graphs, the structure embedded with combined damping system obtained high and consistent performance in both investigated parameters and under all earthquake excitations. While comparing with the responses for the structure fitted with the diagonal VE dampers, in terms of tip deflection, the combined damping system performed stronger under the Hachinohe, Kobe, Northridge and San Fernando earthquakes, at the same time as in terms of tip acceleration reduction under the Kobe, Northridge and San Fernando earthquakes.

**Fig. 6.80 - Tip deflection and acceleration responses of the undamped structure and the structures fitted with two diagonal VE dampers and the structure fitted with one friction damper and one VE damper respectively under the El Centro earthquake**

**Fig. 6.81 - Tip deflection and acceleration responses of the undamped structure and the structures fitted with two diagonal VE dampers and the structure fitted with one friction damper and one VE damper respectively under the Hachinohe earthquake**
Influence of Damping Systems on Building Structures Subject to Seismic Effects

**Fig. 6.82** - Tip deflection and acceleration responses of the undamped structure and the structures fitted with two diagonal VE dampers and the structure fitted with one friction damper and one VE damper respectively under the Kobe earthquake.

**Fig. 6.83** - Tip deflection and acceleration responses of the undamped structure and the structures fitted with two diagonal VE dampers and the structure fitted with one friction damper and one VE damper respectively under the Northridge earthquake.

**Fig. 6.84** - Tip deflection and acceleration responses of the undamped structure and the structures fitted with two diagonal VE dampers and the structure fitted with one friction damper and one VE dampers respectively under the San Fernando earthquake.
Table 7.2 - Percentage tip deflection and tip acceleration reduction of the structure fitted with one diagonal friction damper and one diagonal VE damper

<table>
<thead>
<tr>
<th></th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S.Fernando</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Defl. Red.</td>
<td>26.70</td>
<td>12.30</td>
<td>27.26</td>
<td>20.69</td>
<td>17.73</td>
<td>20.93</td>
</tr>
<tr>
<td>% Accel. Red.</td>
<td>26.53</td>
<td>15.13</td>
<td>33.47</td>
<td>9.02</td>
<td>11.36</td>
<td>19.10</td>
</tr>
</tbody>
</table>

As stated previously the friction dampers in the vast majority of cases suppressed the VE dampers in their ability to reduce the intensity of the initial strong strikes, while the advantage of the VE damper was in more gradual decreasing seismic response of the structure. The hybrid friction-VE dampers were the most reliable; nevertheless, their overall results were slightly lower than those of the other damping systems. While comparing the time history responses for all the damping systems, it seems more obvious that in the cases of the hybrid friction-VE dampers, in the vast majority of the cases only one part of the damper was operating efficiently. On the other hand, the time history responses of the combined damping system demonstrated a successful involvement of advantages of both damping mechanisms to obtain significant and consistent performance.

Based on this as well as on the results of the 18-storey frame-shear wall structure discussed in the previous chapter, it can be suggested, that the combined damping system, which include the diagonal friction damper fitted in the storey with maximum interstorey drift and the diagonal VE damper fitted in the storey the closest to the source of excitation can produce better and more reliable result than individual friction or VE dampers.
Conclusions and Recommendations
Chapter 7: Conclusions and Recommendations

7.1 Scientific Contribution from this Research

The main contribution of this research was to establish that seismic mitigation of building structures can be achieved by using dampers embedded within cut outs of the shear wall. This is a novel method of seismic control and the feasibility of this approach has been considered and amply demonstrated for several cases.

A strategy for protecting buildings from earthquakes is to limit the tip deflection, which provides an overall assessment of the seismic response of the structure. Different building structures require different damping systems for the best results. However, the present study demonstrated that some trends common for all investigated structures can be observed. To this end, findings of the present study revealed that:

- VE dampers are most effective when placed in the lowest storeys.
- Friction dampers are most effective when placed close to regions of maximum interstorey drift.
- The combined damping system, which consists of the diagonal friction damper being placed in the storey with the highest interstorey drift and the diagonal VE damper placed in the lowest storey, is clearly more effective than the hybrid friction-VE dampers placed in the same cut outs.

With regards to efficiency of damper configurations:

- Diagonal dampers experienced highest sensitivity to placement and variations in seismic excitations. These dampers achieved the most significant performances under earthquakes, which caused high deflection.
- Chevron brace dampers were by 30% more effective in terms of tip deflection reduction.
- Lower toggle dampers obtained significantly highest seismic mitigation.

This study has shown that it is possible achieve seismic mitigation, under all earthquake excitations, for all the structures considered in this thesis, by using appropriate damper types suitably located within the structure. The large amount and variety of information generated in present study can enable effective use of dampers to mitigate seismic response in high and medium-rise building structures,
including those whose natural frequencies are within the range of the strong motion of the seismic excitation.

7.2 Discussion
In order to control the vibration response of the medium and high rise structures during earthquake events, passive dampers as energy absorption devices are mostly used. There have been several studies undertaken to develop a method, which optimises the use of energy dissipating dampers in vibration control of buildings under earthquake loads. However, the basic theories behind these methods are mostly not supporting each other and in many ways are rather contradictory. Even more, there are numerous types of dampers available commercially as well as numerous types of high-rise buildings with varied properties, which could be treated under seismic loads. In the light of this, there was a great necessity for further development of methods to determine the effective use of dampers in medium and high rise structures.

Despite the availability of sophisticated computer facilities, determining the type of damping devices and their optimal placement and size still remains highly an iterative trial and error process. What makes the problem even more difficult is the uncertainty of seismic inputs as the forces of nature can vary tremendously. The range of the results presented in this study illustrates the complexity of the problem of optimization in the use of damping devices.

This study has investigated the use of friction, VE and hybrid friction-VE damping devices to mitigate the seismic input energy within medium and high-rise structures. These damping devices were embedded in a variety of different placements (one at a time). Damper properties such as stiffness, damping coefficient, location, configuration and size were varied and results for tip deflections and accelerations were obtained. Six different damping systems, namely friction and VE diagonal dampers, friction and VE chevron brace dampers, hybrid friction-VE dampers and lower toggle VE dampers were placed within cut outs to treat seismic mitigation. Finite element techniques were used to
model the dampers and the structures to obtain the dynamic responses under five different earthquake excitations, using time history analyses.

7.2.1 96 m high shear wall structure

Five different types of structure were investigated in this study. At the initial stage 96 m high, 15 m wide and 0.5 m thick shear wall structure was considered. Five types of damping systems, namely diagonal friction and VE dampers, chevron brace friction and VE damper and hybrid friction-VE dampers, located within cut outs of shear walls, to mitigate the seismic response of shear wall structures were investigated. The results of this investigation confirmed that substantial reductions in tip deflection and acceleration of the structure could be achieved by all five types of dampers in all locations. However, responses under earthquakes with varying frequency content and strong motion duration yielded a wide range of results.

Reductions of up to 45.4% in the peak values of the tip deflections and up to 71.7% in the peak values of the tip accelerations were obtained for the shear wall structures. In terms of reduction in the tip deflection, the best performance was observed when the dampers were placed in the upper level, while greatest reductions in the peak values of tip acceleration were achieved when dampers were placed in the lower level. VE dampers performed better than friction dampers in the lower and middle parts of the structures, while friction dampers performed better in the upper parts of the structure and when placed throughout the structure. The dampers of chevron brace configuration were created to represent only 67% of damping force of diagonal dampers, however, their results in terms of tip deflection reduction were comparatively high. Hybrid dampers were overall the most efficient and also had the most consistent performance.

7.2.2 24-storey frame-shear wall structure

The second structural model investigated in this study was represented by the 24-storey frame-shear wall structure. In this model, the shear wall was modelled as before and was attached on both sides by frame. The structure was fitted with
diagonal VE dampers placed in the cut outs of the shear wall and treated under five earthquake excitations. The reductions of up to 45.7% and 76.4% in tip deflections and tip accelerations respectively, were obtained. Clearly the better results, in both investigated parameters, occurred when the damper was placed in the lower storeys.

The influence of the properties of the dampers on performance of the 24-storey frame-shear wall structures embedded with diagonal VE dampers under the El Centro earthquake excitation was also investigated. The dampers obtained a high performance over the wide range of damping parameters. From the results it is evident, that the damper stiffness, \( k_d \), has clearly less impact on the response of the structure than the damping coefficient, \( C_d \). When the damper was placed in the upper storeys, the increase in damping coefficient \( C_d \) produced strong increase in tip deflection reduction for the lower values (of \( C_d \)) to the peak at \( 30 \times 10^6 \text{ Ns/m} \) and than flattened out. Similar results occurred when the damper was placed in the lower storeys. However, the range of increasing tip deflection reduction with increase in the damping coefficient was extended to the higher values of \( 70 \times 10^6 \text{ Ns/m} \). In the case of the structure with the dampers fitted in the all three parts of the structure, impact of the damping coefficient was similar as in previous cases, however, the range of increasing efficiency was extended up to value of \( 80 \times 10^6 \text{ Ns/m} \).

Influence of the damper parameters on seismic response of these 24-storey frame-shear wall structures was also calculated in a smaller scale under the Hachinohe and Kobe earthquake excitations. The tip deflection and tip acceleration reductions under these earthquake excitations were significantly different. However, the ranges of the most effective damping parameters were relatively close to those experienced under the El Centro earthquake.

While comparing the results of 96 m high shear wall structure and 24-storey frame-shear wall structures, the results obtained from 24-storey frame-shear wall structure were slightly better than those of the shear wall structure fitted with
diagonal VE dampers. In the case of the frame-shear wall structure higher tip deflection reduction occurred for the lower storey placement, while in the shear wall structure reverse trend was observed. In terms of tip acceleration reductions the dampers were more efficient in the lower storeys of each structure.

### 7.2.3 18-storey frame-shear wall structure

Primary objective of this investigation was concentrated on the efficiency of the dampers embedded within cut outs of the medium rise structures, which are more common for Australian conditions. The 18-storey and 12-storey frame-shear wall structures embedded with six types of damping devices, namely diagonal friction and VE dampers, chevron brace friction and VE dampers, hybrid friction-VE dampers and lower toggle VE dampers were predominantly investigated. The dampers were embeded into twelve different placements (one at a time) within cut outs of shear walls to mitigate the seismic response of these medium-rise building structures.

In the 18-storey frame-shear wall structure, reductions of up to 36% in the peak values of tip deflections and 47% in the peak values of the tip accelerations were obtained. The highest tip deflection reductions were obtained, as it was expected, for the structure fitted with the lower toggle VE dampers. The dampers reveal a high level of reliability under all excitations and when fitted in all the placements. The performance of the diagonal VE dampers was even higher for the lower storey placements, however, their performance considerably decreased when moved towards the uppermost storeys. The most consistent performances in all placements and under all seismic excitations were revealed for both types of chevron brace dampers. The results of the hybrid friction-VE dampers were close to those of the friction chevron brace dampers, which indicate that only the friction part of this damping system was working efficiently, while the efficiency of the VE part was, in most cases, lower.

The results of the diagonal friction dampers reveal the highest sensitivity to placement and also to variations in seismic excitations. These dampers achieved the highest performance from all damping systems under the Hachinohe
earthquake, which caused the highest structural deflections from all excitations. On the other hand, involvement of these dampers under the San Fernando earthquake excitation, which causes the lowest structural deflection, was rather unfavourable. These poor results may be attributed to inadequate compensation for removed stiffness and/or partial resonance of the damped structure and insufficient push on the friction damper to make it fully operational.

The highest reductions in peak values of the tip acceleration for the same structure were recorded when it was fitted with the diagonal VE dampers. The tip acceleration reductions of these dampers were greatest when placed in the lower storeys. The average reductions for lower toggle dampers were close to those of the diagonal VE dampers; however, their reductions for the lowest storeys placements were noticeably lower. The diagonal friction dampers displayed once again the highest sensitivity to variation of the damper placements and seismic excitations. Their overall slightly lower tip acceleration reduction was attributed to their poor effectiveness under the San Fernando earthquake excitation as well as their ineffectiveness in the uppermost storey placement, where increase in tip acceleration occurred under each of earthquake excitations. This tip acceleration increase was mainly due to the operating principle of the friction dampers, which caused transfer of acceleration to the ambient structural elements as well as decrease in stiffness of the top storeys due to the cut-out in the shear wall.

The hybrid friction-VE damper and the friction chevron brace dampers followed similar trends with rather inconsistent acceleration reductions under the El Centro, Hachinohe and Kobe earthquakes, while the reductions under the Northridge and San Fernando earthquakes were quite small. The lowest tip acceleration reduction was displayed for the chevron brace VE dampers where satisfactory reductions were recorded only under the El Centro earthquake excitation.

As the results for the diagonal friction damper were inconsistent, a combined damping system consisting of the diagonal friction damper placed in the 16th storey and the diagonal VE damper placed in the 1st storey was also analysed under the same five earthquake excitations. The combined damping system
achieved significant reductions in both investigated parameters under all five earthquake excitations. Furthermore, to emphasize the significance of these results, it should be pointed out that this combined damping system consisted of only two dampers.

### 7.2.4 18-stoey frame structure

In order to further demonstrate feasibility of the procedure used in this study, additional structural models represented by 18-storey frame structures were also treated with embedded diagonal VE dampers. The dimensions of the model were kept to be the same as for previously discussed 18-storey frame-shear wall structure except for substitution of the shear wall with an interior bay. The dampers revealed good overall performance, however, the range of the results obtained under different earthquake excitations varied significantly. On the other hand, the dampers produced relatively consistent results at all placements.

The results for the 18-storeys frame-shear wall structure embedded with the diagonal VE dampers and for the 18-storey frame (only) structure embedded with the same dampers were compared. The frame structure produced slightly better results in both investigated parameters. This was expected, because the structure did not experience decrease of stiffness due to cut out of shear wall. In general, the dampers placed in the frame-shear wall structure were slightly more consistent in terms of efficiency under different earthquake excitations, whereas the dampers placed in the frame structure produced more consistent results through all the placements.

### 7.2.5 12-storey frame-shear wall structure

In the 12-storey frame-shear wall structure reductions of up to 43% in the peak values of tip deflections and 50% in the peak values of the tip accelerations were obtained. In terms of reduction in tip deflection, which is primary investigated parameter, substantial reductions were obtained. On the other hand, in terms of tip acceleration reduction some unfavourable results were experienced. The highest tip deflection reductions and also the most consistent performances were recorded.
for the lower toggle VE dampers. The tip deflection reductions for the diagonal friction dampers were generally comparable to those of the lower toggle VE dampers; however, in few cases this remained rather ineffective. Slightly lower overall deflection reductions for the chevron brace friction dampers and chevron braced VE dampers than those of the diagonal friction dampers were due to their poor performances under the Hachinohe earthquake. The widest range of results was displayed for the diagonal VE dampers, however, their overall reduction also remained at a satisfactory high level. The reductions for the hybrid damping system were comparable to those of the other damping systems; nevertheless, its results suggested that whereas the VE part of damping system operated effectively, the friction part of this damping system was clearly less ineffective.

In terms of tip acceleration reduction the highest reductions were recorded for the lower toggle VE dampers, however, even these dampers remained unreliable under the San Fernando earthquake. The reductions for the diagonal VE dampers were rather inconsistent with same acceleration increases under the Northridge earthquake. The reductions for the hybrid friction-VE damper were generally high except for the El Centro and San Fernando earthquakes where slight increases in acceleration were recorded. The tip acceleration reductions of the diagonal friction dampers were uneven and in many cases an increase in tip acceleration was experienced. The poorest results were recorded for the chevron brace VE dampers and chevron brace friction dampers, which obtained acceleration reductions under the Kobe earthquake only.

The results of the 12-storey frame-shear wall structure confirmed that in terms of tip deflection reduction, the friction dampers performed more efficiently when placed in the storeys with higher interstorey drift, while the VE dampers were more efficient when placed close to source of excitations. The hybrid dampers experienced consistent performance in the all placements. With respect to these findings, the combined damping system, which consisted of the diagonal friction damper placed in the 11th storey and the diagonal VE damper placed in the 1st storey was also investigated. The significant tip deflection and acceleration reduction of this combined damping system demonstrated successful involvement
of advantages of both damping mechanisms to obtain significant and consistent performance under all earthquake excitations.

7.2.6 Conclusion

Five high to medium-rise structures embedded with six damping systems were investigated under five different earthquake records. Each of these damping systems performed in a different manner and also their performance varied considerably when treated in different structures. However, some specific features can be observed. The friction dampers in the huge majority of cases surpassed the VE dampers in their ability to reduce the intensity of the initial strong strikes. In contrast, the VE dampers gradually decreased the deflection and acceleration of the structure. The performance of the friction dampers increased with higher interstorey deflection, while the best performance of VE dampers was achieved when placed in the lowest storeys. In addition, the diagonal friction dampers performed better under the earthquakes, which caused higher deflections of the structure. In contrast, the performance of these dampers under earthquakes that caused a lower structural deflection was less favourable. The performance of the diagonal VE dampers was noticeably less sensitive to this aspect. With regard to the reductions of the tip acceleration, both damping systems experienced the best performance in the lowest storeys, while their performance gradually decreased as the dampers were moved towards the uppermost storeys.

Despite the fact, that both types of the chevron brace dampers were created to represent only 66.6% of the damping force of the diagonal dampers, their overall tip deflection reduction was comparatively high and even more reliable than those of the diagonal dampers. On the other hand, both types of chevron brace dampers were clearly the least effective in terms of tip acceleration reduction. The hybrid friction-VE dampers performed in a more stable and reliable manner than the diagonal and chevron brace dampers, nevertheless their overall reductions were in the majority of cases, slightly lower. The results of these dampers in an 18-storey structure indicated that only the friction part of the hybrid damper was operating properly. On the other hand, in the 12-storey structure it was only the VE parts.
The lower toggle VE damper displayed the highest performance and reliability from all damping systems.

A number of analyses of the several different structure types fitted with different damping systems and treated under different earthquake excitations were carried out to gain a comprehensive understanding of the effectiveness of the dampers and their placement. This study treated the structural response under a range of seismic excitations even when the dominant seismic frequencies matched the natural frequency of the structure. It has been shown that it is possible to have seismic mitigation, under all earthquake excitations, by using certain damper types appropriately located within the structure.

7.3 Recommendations for Further Research

The following are suggestions for further research in this area:

- The method of optimising the location of the dampers within the structure be further investigated.
- The study be extended by involving the other toggle configurations proposed by the other authors (Ribakov et al., 2003, Kubota et al., 2001, etc.) study.
- Investigation of performances of the damping systems under synthesized excitations with a wide range of frequencies and peak ground accelerations.
- Extend the study on use of semi-active damping systems. These are designed to alter the properties to suit the intensity and frequency content of the earthquake, in order to obtained more efficient performance.
List of References


Shao, D., M.S., S.E., H. Kit Miyamoto, M.S., S.E. “Viscous Damper Versus Friction Damper for Retrofit of a Non-ductile Reinforced Concrete Building with Unreinforced Masonry Infill.” *SEAOC 1999 Convention*


Appendix

Results of 12-Storey and 18-Storey Structures

Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with three chevron brace VE dampers

<table>
<thead>
<tr>
<th>Model</th>
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Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with three chevron brace friction dampers

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Percentage tip acceleration reduction of the 18-storey frame-shear wall structure fitted with three chevron brace VE dampers

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Percentage tip acceleration reduction of the 18-storey frame-shear wall structure fitted with three chevron brace friction dampers

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Percentage tip acceleration reduction of the 18-storey frame-shear wall structure fitted with three chevron brace VE dampers

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Influence of Damping Systems on Building Structures Subject to Seismic Effects

### Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with one chevron brace VE damper

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Influence of Damping Systems on Building Structures Subject to Seismic Effects

Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with three diagonal VE dampers

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Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with three diagonal friction dampers

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Percentage tip acceleration reduction of the 18-storey frame-shear wall structure fitted with three diagonal VE dampers

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Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with three diagonal friction dampers

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### Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with one diagonal friction damper

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### Percentage tip acceleration reduction of the 18-storey frame-shear wall structure fitted with one diagonal friction damper

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### Percentage tip acceleration reduction of the 18-storey frame-shear wall structure fitted with three hybrid friction-VE dampers

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### Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with one hybrid friction-VE damper

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### Percentage tip acceleration reduction of the 18-storey frame-shear wall structure fitted with one hybrid friction-VE damper

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### Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with three lower toggle VE dampers

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### Percentage tip deflection reduction of the 18-storey frame-shear wall structure fitted with one lower toggle VE damper

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### Percentage tip acceleration reduction of the 18-storey frame structure fitted with three diagonal VE dampers

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### Percentage tip deflection reduction of the 18-storey frame structure fitted with one diagonal VE damper

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### Percentage tip acceleration reduction of the 18-storey frame structure fitted with one diagonal VE damper

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Influence of Damping Systems on Building Structures Subject to Seismic Effects

### Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with two chevron brace VE dampers

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Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with one chevron brace VE damper

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Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with one chevron brace friction damper

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Percentage tip acceleration reduction of the 12-storey frame-shear wall structure fitted with one chevron brace VE damper

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Percentage tip acceleration reduction of the 12-storey frame-shear wall structure fitted with one chevron brace friction damper

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Influence of Damping Systems on Building Structures Subject to Seismic Effects

### Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with two diagonal VE dampers

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### Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with two diagonal friction dampers

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### Percentage tip acceleration reduction of the 12-storey frame-shear wall structure fitted with two diagonal friction dampers

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Percentage tip acceleration reduction of the 12-storey frame-shear wall structure fitted with two diagonal friction dampers
### Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with one diagonal VE damper

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### Influence of Damping Systems on Building Structures Subject to Seismic Effects

#### Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with two hybrid friction-VE dampers

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<td>3.81</td>
</tr>
<tr>
<td>H 9</td>
<td>3.90</td>
<td>4.17</td>
<td>3.02</td>
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<td>4.66</td>
</tr>
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<td>3.27</td>
<td>5.43</td>
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<td>5.81</td>
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</table>

#### Percentage tip acceleration reduction of the 12-storey frame-shear wall structure fitted with one hybrid friction-VE damper

<table>
<thead>
<tr>
<th>Model</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S. Fernando</th>
</tr>
</thead>
<tbody>
<tr>
<td>H 1</td>
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<td>0.03</td>
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<td>-4.79</td>
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<td>-1.21</td>
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### Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with two lower toggle VE dampers

<table>
<thead>
<tr>
<th>Model</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S. Fernando</th>
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<tbody>
<tr>
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<td>H 7-8</td>
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<td>16.63</td>
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<td>13.82</td>
<td>18.69</td>
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### Percentage tip acceleration reduction of the 12-storey frame-shear wall structure fitted with two lower toggle VE dampers

<table>
<thead>
<tr>
<th>Model</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S. Fernando</th>
</tr>
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### Percentage tip deflection reduction of the 12-storey frame-shear wall structure fitted with one lower toggle VE damper

<table>
<thead>
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<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S. Fernando</th>
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</tr>
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### Percentage tip acceleration reduction of the 12-storey frame-shear wall structure fitted with one lower toggle friction damper

<table>
<thead>
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<th>Model</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S. Fernando</th>
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</thead>
<tbody>
<tr>
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### Percentage tip deflection and tip acceleration reduction of the 18-storey structure fitted with one diagonal friction damper and one diagonal VE damper

<table>
<thead>
<tr>
<th>Model</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S. Fernando</th>
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</thead>
<tbody>
<tr>
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<td>19.18</td>
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### Percentage tip deflection and tip acceleration reduction of the 12-storey structure fitted with one diagonal friction damper and one diagonal VE damper

<table>
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<tr>
<th>Model</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Kobe</th>
<th>Northridge</th>
<th>S. Fernando</th>
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<tbody>
<tr>
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<td>27.26</td>
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<td>15.13</td>
<td>33.47</td>
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<td>11.36</td>
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