Seismic Retrofitting of Deficient Canadian Buildings

By

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B.S. Civil Engineering, McGill University, 2007
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ABSTRACT

Many developed countries such as Canada and the United States are facing a significant infrastructure crisis. Most of their facilities have been built with little consideration of seismic design and durability issues. As a result, these structures are vulnerable to earthquake loadings and are in urgent need of repair and retrofit.

This thesis provides a literature review of the vulnerability of Canadian infrastructures built prior to the development of seismic design provisions in actual codes of practice and standards. It describes the performance of typical structures under earthquake loading, such as unreinforced masonry buildings, flat slab concrete buildings and steel frame buildings. It then presents the most common retrofitting strategies applicable to low-rise buildings commonly found in major Canadian cities. A case study assessing the performance of hybrid base isolation systems is then presented.

The performance of passive and semi-active hybrid base isolation system is evaluated through the use of a SIMULINK computer model of a typical two-story concrete frame building. A significant reduction in interstory displacement is achieved using the passive system and further reduction in base displacement and base shear is accomplished using the semi-active system.

Thesis Supervisor: Jerome Joseph Connor
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1.0 INTRODUCTION

Earthquakes are natural events that often lead to catastrophic fatalities and economic losses. In Canada, several studies concluded that seismic events represent the most important threat to public security and safety. Over the past 20 years, a better understanding of the behavior and performance of existing structures under seismic excitations has resulted in major advancements in earthquake design. Major developments in international and national building codes and standards have evolved to account for these severe loading conditions. During this process, a performance-based design approach was used, the desired performance level of structures dictating the structural design of the building (Mitchell, 2007).

However, most of the aging infrastructures in Canada were built prior to the development of these design codes and are, therefore, in urgent need of repair and retrofit. In fact, an investment of $49 billion would be needed for the rehabilitation of all Canadian infrastructures (Mufti, 2003). This amount may seem enormous, but a rapid investment in existing infrastructures will be rewarding in the long term. According to an estimate by the NIBS in 2005, one dollar wisely spent on hazard mitigation will result in a four dollar saving for the society (Mitchell, 2007). The choice of the most suitable and cost-effective mitigation strategy for each retrofit project is a major challenge for structural engineers.

The main objective of this thesis is to provide the reader with a general understanding of the seismic retrofit of deficient structures, with a focus on Canadian buildings. After a brief overview of the current state of Canadian infrastructures and the seismic risk in the country, Section 2 assesses the vulnerability of unreinforced masonry buildings, concrete frame buildings, and steel frame buildings built prior to the development of seismic design provisions. Section 3 provides a general description of seismic mitigation strategies used for the retrofit of existing structures and Section 4 evaluates their application to each building type mentioned above. Hybrid base isolation systems, which represent one of the most promising alternatives for the retrofit of existing buildings, are presented in Section 5. Their performance is evaluated based on the results of a SIMULINK computer model of a typical three-story concrete frame building.

2.0 EARTHQUAKES AND CANADIAN INFRASTRUCTURES

2.1 CURRENT STATE OF INFRASTRUCTURES

The wide majority of Canadian infrastructures have been built after the Second World War in response to the increasing Canadian population (Mirza and Haiter, 2003). Up to the 1969's, the 'baby boom', combined with the rise in immigration levels, has led to a growing need for new infrastructures all around the country (Mitchell, 2007). However, most of the facilities built prior to the 1970's have been designed with inadequate knowledge of seismic design and material durability, and are in desperate need of repair, rehabilitation or renewal (Hollaway, 2003).

As a result, Canada, like many other developed countries, is now facing a significant infrastructure crisis. Over the last decades, all levels of government have continued to invest in new infrastructures, neglected existing ones. The number of deficient structures is now increasing at a faster rate than our ability to repair them (Günes, 2004). In Canada, only 20% of the $11 billion dedicated to infrastructures in the 1980s was spent on the rehabilitation and maintenance of existing facilities, the rest being spent on the seemingly more urgent need for new infrastructures (Mirza and Haiter, 2003).

Canadian aging infrastructures consist of a wide variety of facilities such as roads, bridges, buildings, sewers, and water systems, and are financed by the federal, provincial and municipal governments. The lack of national infrastructure policies regulating their repair, maintenance, and rehabilitation is leading to major safety concerns. Many structures are highly vulnerable to natural hazards, such as earthquakes, floods, landslides, and ice and snow storms; or man-made hazards, such as wars, terrorist attacks, and other damaging accidents (Mirza and Haiter, 2003).

2.2 EARTHQUAKE HAZARD

2.2.1 Seismic Risk

Unlike other natural hazards, such as floods and hurricanes, earthquake events are hardly predictable. Their probability of occurrence at a specific time and location is even harder to determine due to the long return intervals between the few events that occurred in the past (Allen, 2007). The main goal of the seismological community is to collect data from past earthquakes that can be used to accurately predict the magnitude, location and date of occurrence of future events. Researchers consider past earthquake disasters, along with information available on tectonic and geological characteristics of the region, to predict maximum anticipated seismic levels (NRCAN, 2009). Structural engineers then use this information to design earthquake resistant buildings.

Past Earthquake Disasters

Natural disasters, such as hurricane Katrina in New Orleans (2005) or the Samutra earthquake and tsunami of the Indian Ocean (2004), have destroyed several cities around the world. Among all types of natural disasters, earthquakes are the most destructive and account for 60% of all natural hazard fatalities (Allen, 2007). Over the past century, earthquake events have been responsible for more than 2 million deaths (USGS, 2009).

Even if the seismic risk in Canada is moderate compared to that in countries such as Japan, Pakistan or Italy, its infrastructures are highly vulnerable to earthquake hazards. It has been shown that a major earthquake is Canada's largest potential natural disaster. In fact, Canada's largest cities -- Montreal, Ottawa, Quebec, Toronto, Vancouver, and Victoria -- are located in regions of moderate to high seismic risk (see Figure 1). Accordingly, approximately 65% of the total country's population is grouped in regions with more than 75% of Canada's seismic risk. In a more localized perspective, a seismic event is the most important threat to public safety and security for many of the largest Canadian cities, including Montreal and Quebec City (Mitchell, 2007).

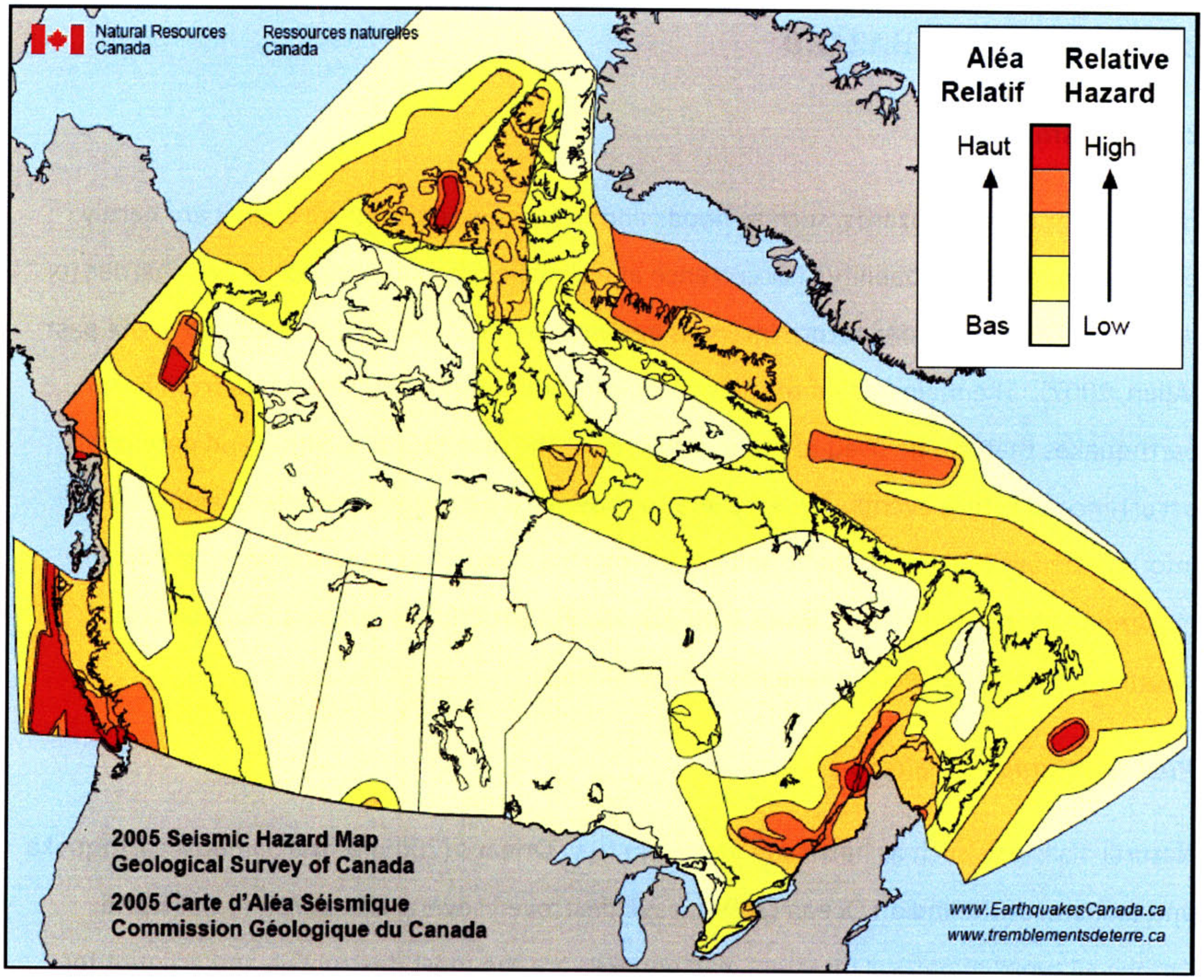


Figure 1: Seismic Hazard Map of Canada (NRCAN, 2005)

Soil Conditions

As shown in Figure 2, most of the zones of high seismic risk are located in the provinces of British Columbia, Ontario, and Quebec. The seismic zones in western Canada are subjected to long-duration ground motions typical of the Cascadia subduction zones. The soil in this part of the country is critical because it consists if thick, unconsolidated glacial and fluvial deposits (Mitchell, 2007).

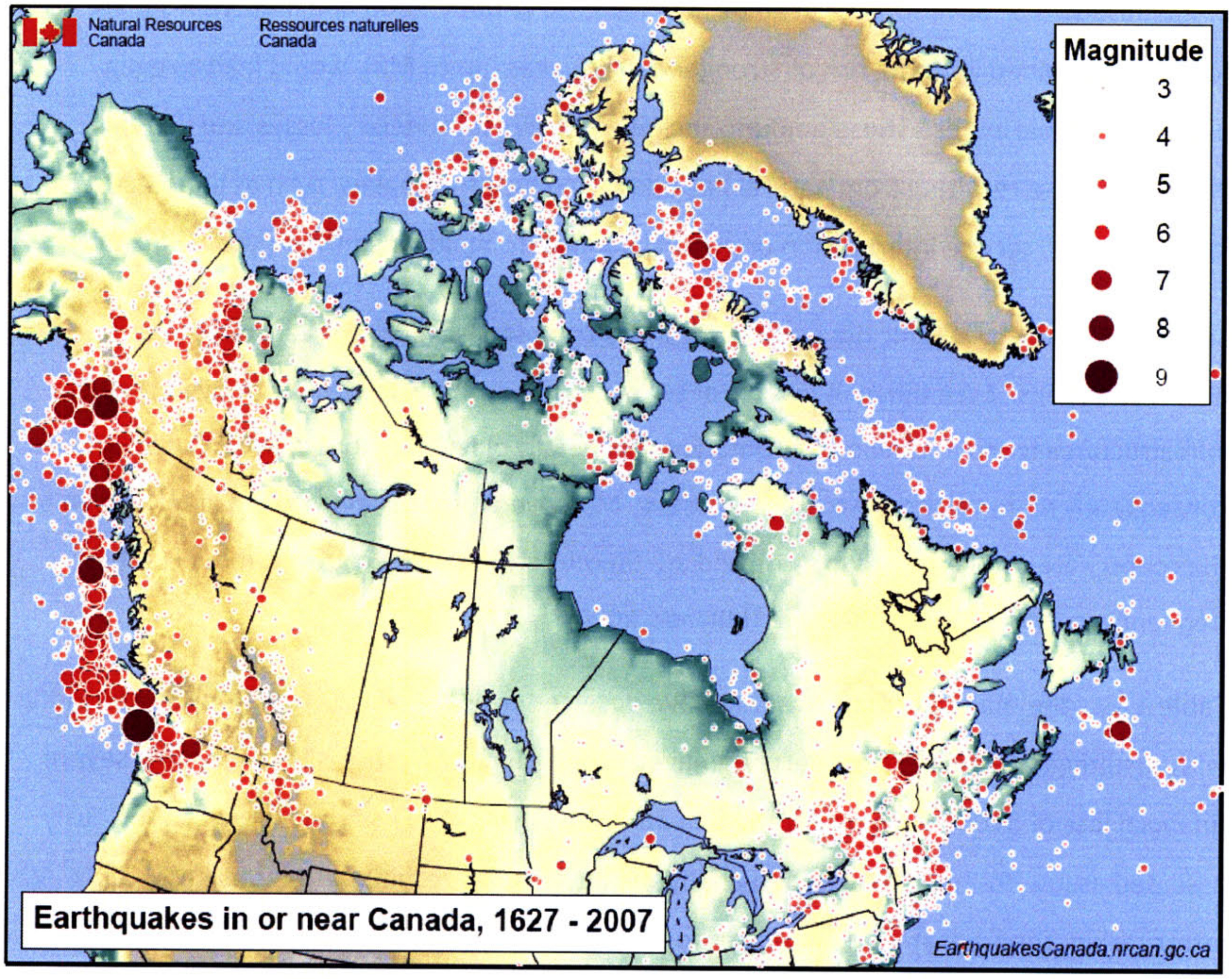


Figure 2: Earthquakes in or near Canada, 1627-2007 (NRCAN, 2005)

In eastern Canada, ground motions of high frequency content are the most common. The risk of motion amplification is increased due to the presence of pockets of soft marine sediments left from the ancient Champlain Sea. Hence, extended damage in regions far away from the earthquake epicenter can be expected. For example, the earthquake that occurred in Saguenay in 1988 caused damage in Montreal, more that 350 km away. In Ontario, soil amplification has been found to highly exceed typical amplification factors (Mitchell, 2007).

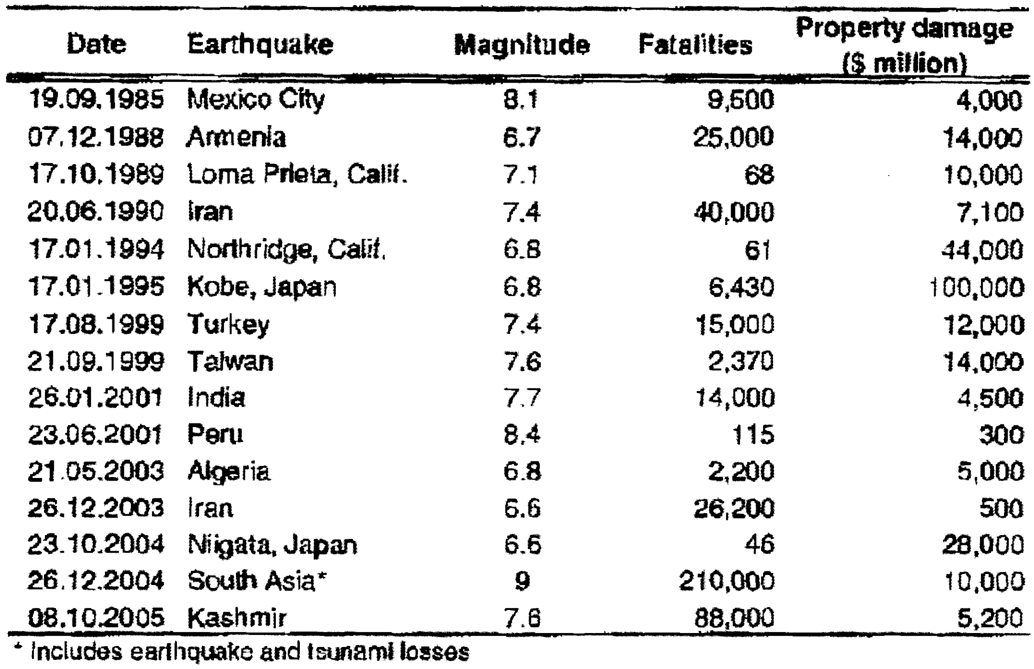
Financial Losses

The cost of repair and retrofit of deficient Canadian infrastructures is high, but the potential financial losses resulting from an earthquake are even higher. According to a study commissioned by the Munich Reinsurance Company of Canada in 1992, national financial losses in the order of $15 to $30 billion dollars could result from a relatively small earthquake of magnitude 6.5. Seventeen years later, the increase in population and industrial square footage, combined with the risk of stronger earthquakes (up to M8), would lead to even higher economic losses. These amounts include only direct monetary losses and do not include damage resulting from natural hazards following earthquakes, such as liquefaction, land slide, fire, service interruption or general damage to building content (Mitchell, 2007).

Scientists expect the seismic risk in eastern Canada to be similar to that of Vancouver, but for different reasons. Even though the rates of seismicity in British Columbia are relatively greater, infrastructures found in Quebec and Ontario are generally older and more deficient, and consequently more vulnerable to seismic losses. Moreover, most buildings found in this region of the country are categorized as low-rise and are therefore vulnerable to ground motion at high frequencies associated with this zone (Mitchell, 2007).

Table 1 lists the major earthquakes that have occurred over the last 20 years worldwide, along with resulting fatalities and property damage. Especially noteworthy is that the most severe financial loss of $100 billion occurred in Japan in 1995, where the seismic risk was previously assessed as low-to-moderate. When adding financial losses resulting from business interruption, the total amount increases to $150 billion (Mitchell, 2007).

Table 1: Fatalities and property damage resulting from major earthquakes over the last 20 years (Mitchell, 2007)



Date	Earthquake	Magnitude	Fatalities	Property damage ($ million)
19.09.1985	Mexico City	8.1	9,500	4,000
07.12.1988	Armenia	6.7	25,000	14,000
17.10.1989	Loma Prieta, Calif.	7.1	68	10,000
20.06.1990	Iran	7.4	40,000	7,100
17.01.1994	Northridge, Calif.	6.8	61	44,000
17.01.1995	Kobe, Japan	6.8	6,430	100,000
17.08.1999	Turkey	7.4	15,000	12,000
21.09.1999	Taiwan	7.6	2,370	14,000
26.01.2001	India	7.7	14,000	4,500
23.06.2001	Peru	8.4	115	300
21.05.2003	Algeria	6.8	2,200	5,000
26.12.2003	Iran	6.6	26,200	500
23.10.2004	Niigata, Japan	6.6	46	28,000
26.12.2004	South Asia*	9	210,000	10,000
08.10.2005	Kashmir	7.6	88,000	5,200

* Includes earthquake and tsunami losses

2.2.2 Vulnerability of Infrastructures

As mentioned above, most Canadian facilities were built prior to the 1970's, when there were no seismic provisions in design codes. Of greater concern is the safety of critical infrastructures, such as hospitals, schools and bridges. In the case of an earthquake or any other natural or man-made hazard, these structures must remain fully operational and functional. However, most of these infrastructures are likely to experience severe damage or even collapse at the occurrence of a major earthquake (Mitchell, 2007).

Generally, structures are most likely to be damaged when their natural frequency is close to the frequency content of the earthquake (NRCAN, 2009). As a result, high-rise structures would be damaged by large and distant earthquake events with low frequency content, while low-rise buildings would suffer from high frequency excitations of moderate magnitude happening locally (see Figure 3). Spectral and peak ground accelerations as provided by the National Building Code of Canada are presented in Figure 4.

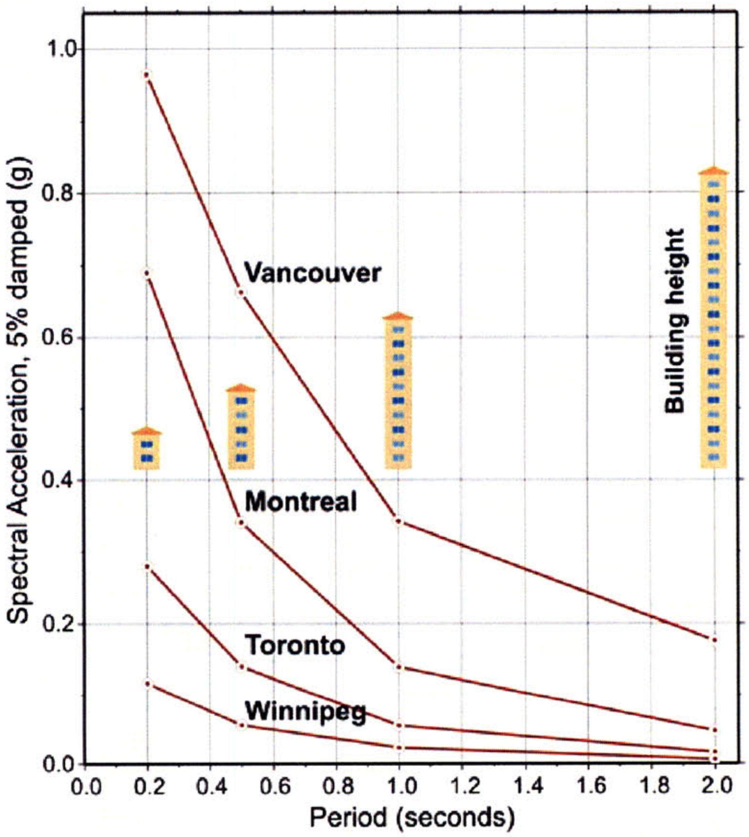


Figure 3: Hazard spectra for four Canadian cities (NRCAN, 2005)

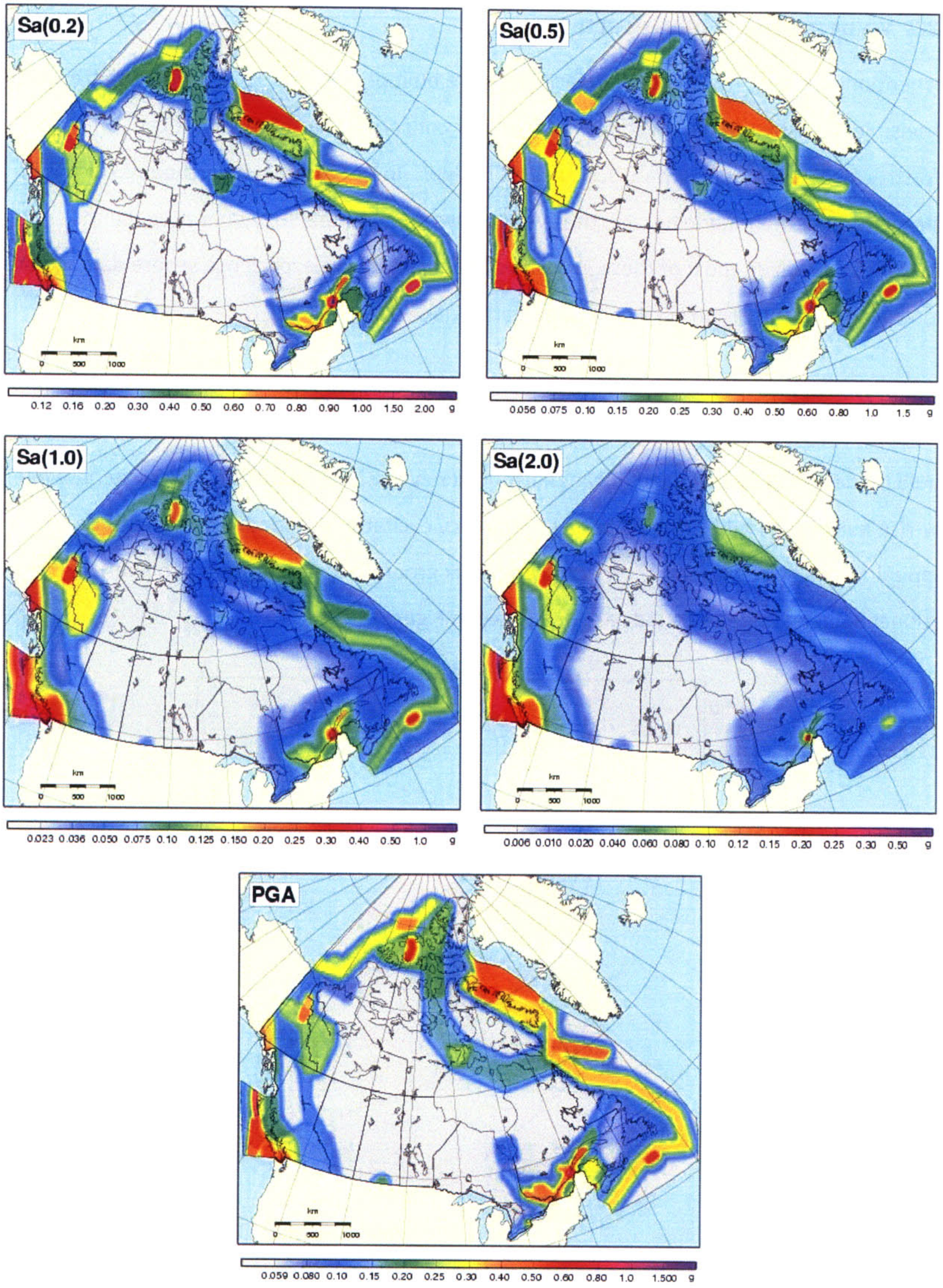


Figure 4: Spectral acceleration for various periods for a 2500 years return period for firm ground condition (NRCAN, 2005)

To assess the vulnerability of Canadian structures under these loads, a performance-based assessment method is used. Mitchell (2007) describes the vulnerability assessment as:

> the evaluation of the actual state of a structure, with consideration of uncertainties of structural performance, and the need for seismic retrofit to conform to a required performance level.

This method is extremely useful for the evaluation of existing buildings, especially for critical facilities that must remain safe and operational after an earthquake (Mitchell, 2007).

2.2.3 Codes and Guidelines

Over the past 20 years, the seismic design of new structures in Canada has evolved dramatically through several additions to the National building Code of Canada and CSA Design Standards (Mitchell, 2007). Conforming to these codes, engineers can now design safer structures that would resist moderate to strong earthquakes, while respecting the acceptable level of structural damage (Cheung *et al.*, no date). However, most buildings built earlier do not meet these current standards, and their performance during an earthquake is hardly predictable. Recently, past earthquake events such as Northridge (USA 1994), Kobe (Japan 1995) or Chi-Chi (Taiwan 1999) demonstrated that moderate earthquakes have a disastrous impact on older buildings (Cheung *et al.*, no date). These buildings would have probably survived if they had been properly retrofitted to accomplish a desired performance level.

Recent provisions in design codes have been developed using the performance-based design approach. Mitchell (2007) defines the performance-based seismic design as:

> a process that explicitly evaluates how a building is likely to perform, given the potential seismic hazard it is likely to experience, considering uncertainties inherent in quantification of potential hazard and uncertainties in assessment of the actual building response.

Several international networks and organizations are currently working on reducing seismic risks. The Applied Technology Council (ATC) and the Network for Earthquake Engineering Simulation (NEES) in the United States, and the Seismic Retrofit Solutions Program in New

Zealand are some examples (Mitchell, 2007). In Canada, The Public Works and Government Services Canada (PWGSC) are investigating the use of emerging technologies in several retrofit projects (Cheung *et al.*, no date).

2.3 TYPE OF STUCTURES

In Canada, the most common types of deficient and aging structures are unreinforced masonry buildings, concrete buildings and steel structures. A proper understanding of the properties and behavior of each building type under seismic load is the first step to seismic retrofit. Common structural deficiencies found in Canadian buildings are poor concrete confinement, inadequate shear resistance, poor lap splices, inadequate connections in precast concrete buildings, poor details in steel frame and unreinforced masonry constructions, 'soft stories' in all building types, and generally poor design and detailing of aging buildings and bridges . An overview of each of the three building types and their structural deficiencies is presented below (Mitchell, 2007).

2.3.1 Masonry Buildings

Masonry buildings consist of more that 70% of the total inventory worldwide and are widely found across Canada (ElGawady *et al.*, 2004). This structural system has been used for a wide variety of buildings and critical facilities, including schools, hospitals, government buildings and port facilities. These buildings are highly vulnerable to earthquake events, and a large number will require seismic retrofit over the next ten years (see Figure 5) (Mitchell, 2007).

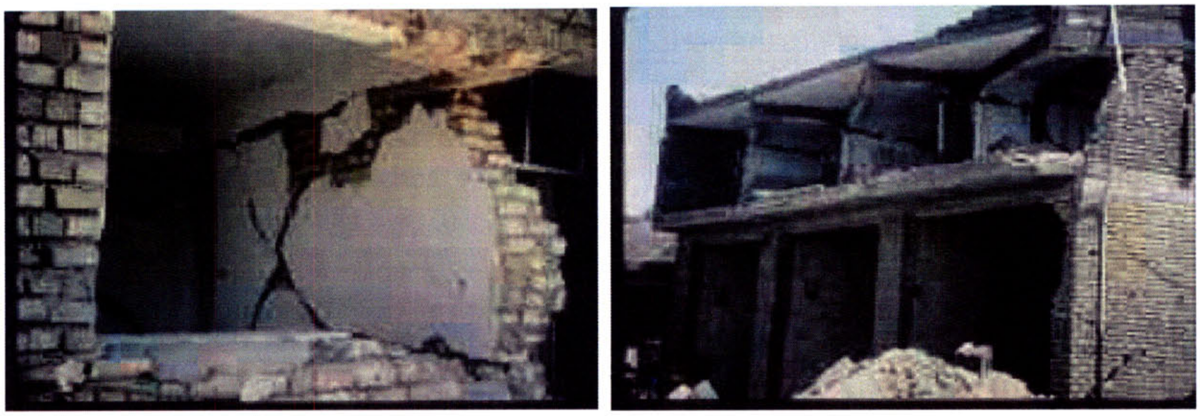


Figure 5: Typical damage to unreinforced masonry walls (NGDC, 2009)

Masonry structures, often called load bearing structures, are effective in transferring vertical loads to the ground through exterior and interior bearing walls. However, they are not designed to carry large lateral loads. Under dynamic loads, masonry walls behave in a brittle manner and experience severe cracking, up to the point where they do not contribute to the lateral resistance of the building anymore (Matsagar and Jangid, 2008).

Among the most popular types of masonry structures found across Canada, the infilled masonry frame is one of the most common. These structures consist of a typical concrete frame with interior unreinforced masonry block partitions. The major problem of these structures is their lack of ductility and energy absorption. They are vulnerable to fast and high energy loadings typical of earthquake events due to the brittleness of masonry units and the low ductility of concrete frames (Mitchell, 2007).

2.3.2 Concrete Frame Buildings

Several concrete buildings found in major Canadian cities are structurally deficient. Most of them have been designed to carry gravity loads only, without consideration of lateral loads, resulting in severe damage (see Figure 6 and 7). The weakest elements of such structures are usually the columns, which have poor lateral confinement and shear resistance. Moreover, the location of lap splices and the number of reinforcing bars are often inadequate. Other deficiencies include the lack of confinement and shear reinforcement in joints, poor anchorage of longitudinal reinforcement of beams into joints, and shear resistance of beams (Mitchell, 2007).

For building without beams, flat concrete slabs experience additional problems such as punching shear failures. For buildings with concrete shear walls, insufficient and inadequate reinforcement is common. Regions of plastic hinging experience inadequate lap splice, while the ends of the walls lack concentrated vertical bars and confining ties (Mitchell, 2007).

Figure 6: beam-column joint failure in typical reinforced concrete frames (Saaticioglu *et al.,* 2001)

Figure 7: Shear failure of RC walls (Ireland *et al.*, 2006)

2.3.3 Steel Frame Buildings

Low-rise steel frame buildings are another type of structure found in almost every Canadian city. Their structural advantages include their low mass-to-stiffness ratio, their high ductility and their high energy absorption capacity. However, the Northridge earthquake demonstrated the vulnerability of such structures, especially for low-rise buildings. Common deficiencies include inadequate diaphragm strength and rigidity, incomplete and insufficient lateral force resisting system, excessive lateral flexibility, inadequate global strength and ductility, and inadequate foundation systems (see Figure 8). The three common types of steel structures are concentrically braced frame (CBF), eccentrically braced frame (EBF), and moment resisting frame (MRF). An overview of their advantages and disadvantages is presented below (Sarno and Elnashai, 2002).

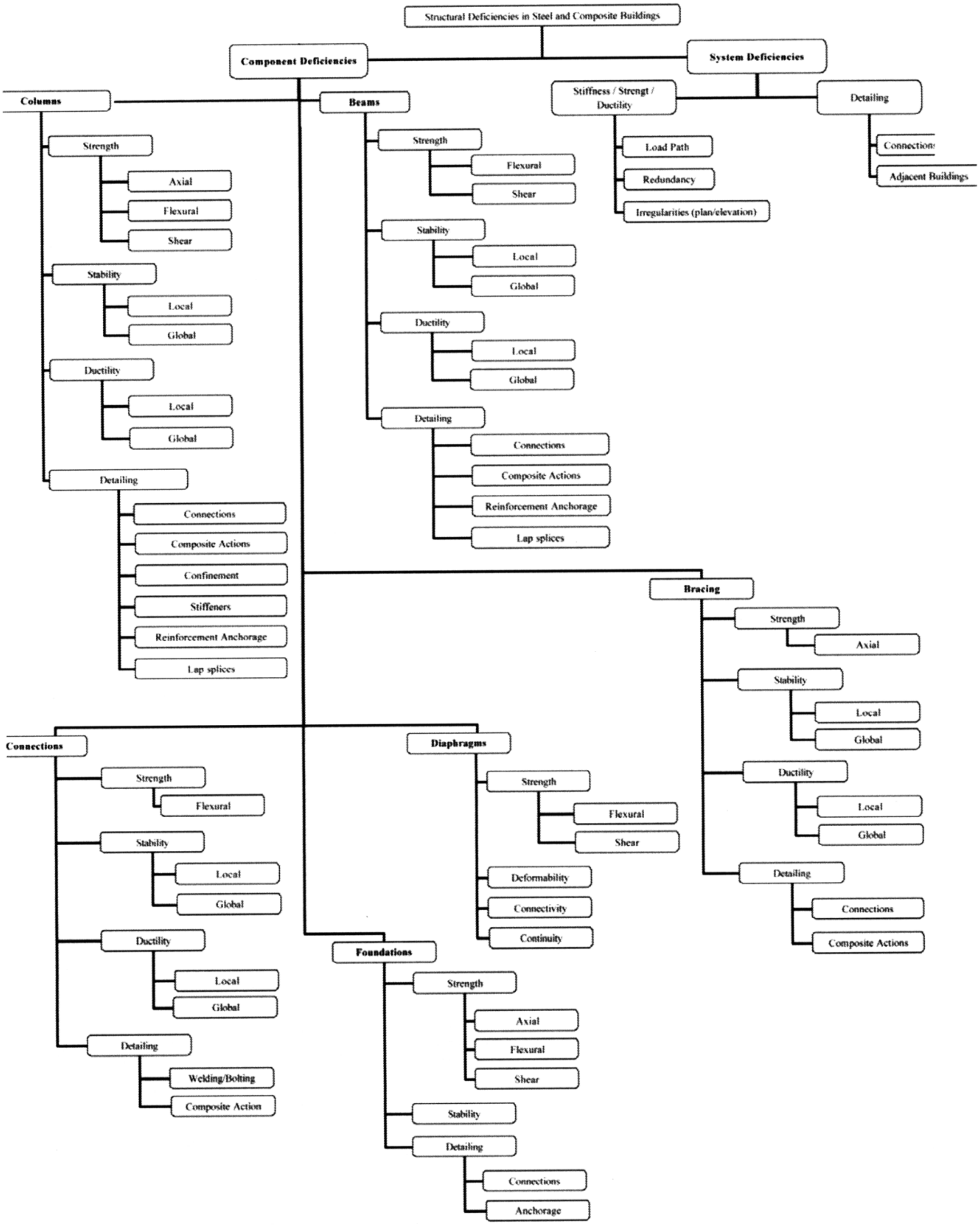


Figure 8: Structural deficiencies for steel and composite buildings (Adapted from Sarno and Elnashai, 2002)

Moment Resisting Frame

Moment resisting frames (MRF) are generally ductile and have sufficient strength. However, they often have poor lateral stiffness, leading to large interstory drifts (Sarno and Elnashai, 2002). The damage experienced by MRF is usually limited to nonstructural components of the building, such as claddings or mechanical and electrical utilities. However, the 1994 Northridge earthquake revealed that welded beam-to-column connections in MRF buildings can behave in a brittle manner and result in catastrophic failures (Bruneau 2005; Sarno and Elnashai, 2002). Other problems include excessive story drifts and shears, column yielding and plastification, instability, and failure of diaphragm shear connections (Sarno and Elnashai, 2002).

Concentrically Braced Frame

Unlike MRF, concentrically braced frames (CBF) usually have both adequate stiffness and strength. However, they are not as ductile as MRF, their deformation/energy absorption capacity being limited by the buckling failures of the braces (Sarno and Elnashai, 2002). In this case, both structural and non structural elements suffer damage from a significantly large earthquake event. Common structural types of damage include fracture of braces and connection elements, poor detailing of connections, buckling and yielding of braces, and failure of diaphragm shear connections (see Figure 9).

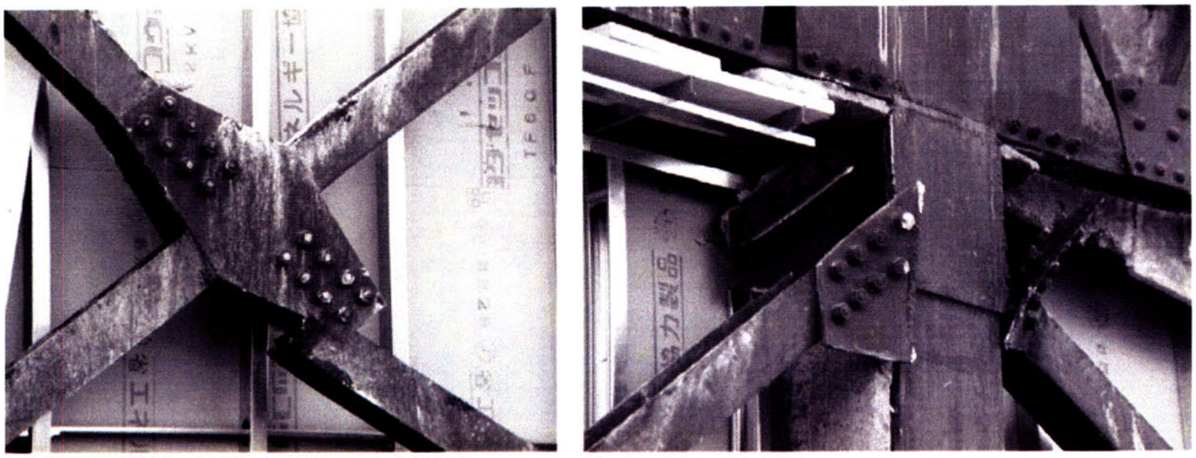


Figure 9: Damage to CBF in Kobe earthquake (Sarno and Elnashai, 2002)

Eccentrically Braced Frame

Eccentrically braced frames behave like CBF with added energy dissipation capability and, consequently, increase in ductility. They are the most effective type of steel structures used to reduce the seismic response and structural damage to buildings. This bracing configuration allows for the formation of a plastic hinge in the link beam between the braces. This permanent bending or shear yielding deformation prevents the buckling of the braces and results in additional energy absorption. These structures experience permanent damaged in the link beams after an earthquake (Sarno and Elnashai, 2002).

3.0 RETROFITTING STRATEGIES

Most developed countries are facing the new challenge of rehabilitating and retrofitting their infrastructures. Indeed, the number of infrastructures worldwide continues to increase, while existing facilities are aging (Darby, 1999). The factors contributing to the need for seismic retrofit are listed below(Basler *et al.* 2001; Gunes 2004).

- revisions in seismic design codes,
- inadequate structural design,
- general aging,
- change in structural systems,
- increase in applied loads,
- structural damage,
- fire damage,
- excessive deflection,
- environmental deterioration, and
- inadequate maintenance.

This thesis focuses on the retrofit of buildings designed with inadequate knowledge of seismic design, a major problem in the majority of Canadian cities.

The seismic retrofit of existing structures is a challenging problem due to the high level of uncertainties encountered during the design process, including uncertainties due to the condition of the soil, the performance of the deficient structure, and the performance of the retrofitting system. Major design considerations include (Allen 2002; Mitchell 2007):

- level of seismic hazard,
- vulnerability of the structure,
- desired performance level,
- use of the building and disruption of occupants,
- importance and historical value of the building,
- design codes and methods used during the initial construction,
- current codes requirements,
- cost and duration of construction,
- structural compatibility to existing building,
- reversibility of the intervention,
- capacity of the foundation system, and
- availability of materials and technologies.

The major steps to the implementation of a retrofitting strategy are shown in Figure 10. The first task consists of evaluating the seismic risk, the vulnerability of the structure, and its behavior under lateral loads. The trade-off between the performance level and the cost of the system is then chosen and the most suitable strategy is selected. Finally, the proposed scheme is evaluated through a time history analysis and, if needed, the design is modified to achieve the desired performance level (Sarno and Elnashai, 2002).

The response of the structure is dependant on the type and the number of devices used and their overall distribution throughout the building. In many cases, their cost is the main factor dictating their choice. There is, therefore, an urgent need for the development of cost-efficient retrofit strategies and innovative materials that are effective, reliable and durable (Gunes, 2004).

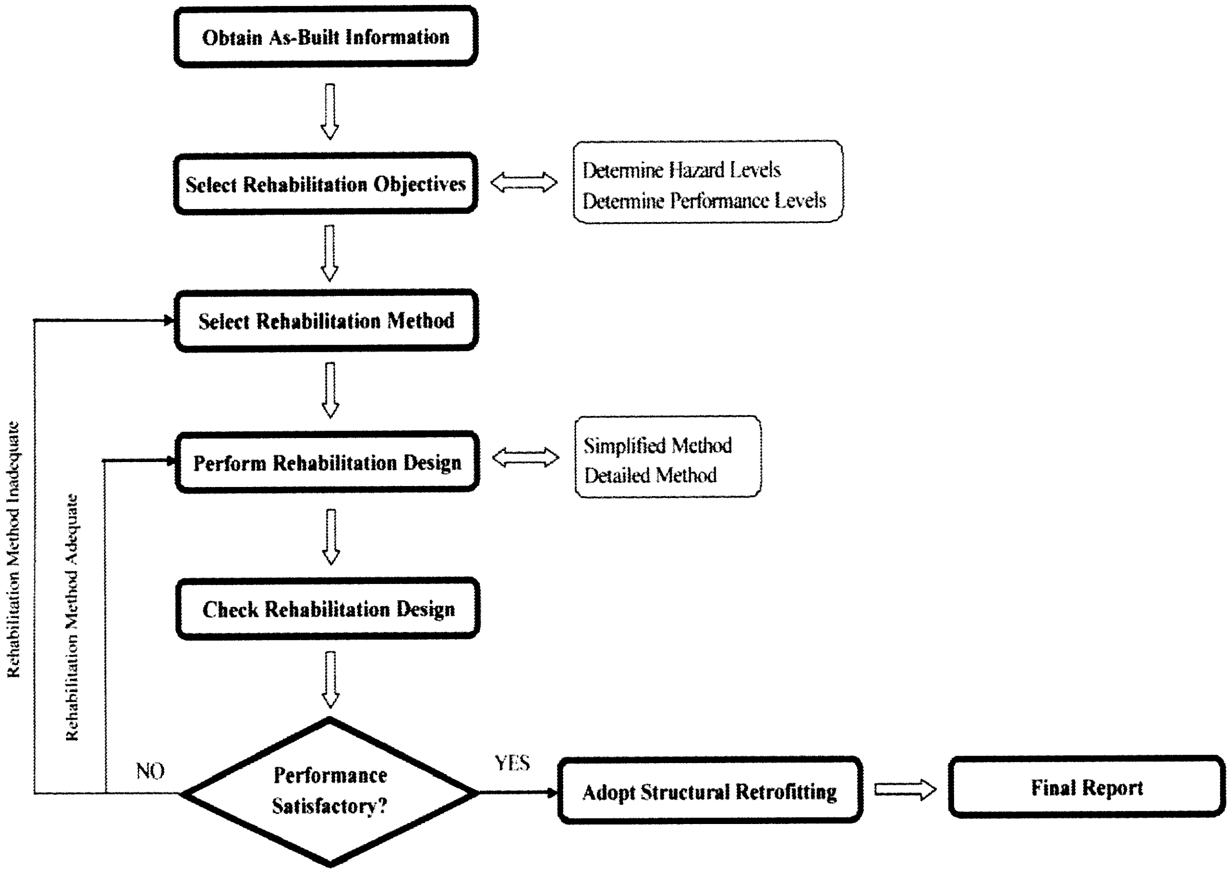


Figure 10: Standard process for seismic rehabilitation (Sarno et Elnashai, 2002)

Many innovative strategies for the retrofit of existing buildings are now available. They can be divided into two categories: passive control and active control. Passive control strategies are currently the most common and are presented in the following section. Active control strategies will be discussed briefly in Section 3.2.

3.1 PASSIVE STRATEGIES

Passive strategies are commonly used to reduce the motion of new and existing structures. Recently, the structural technology trend has switched from the design of conventional earthquake resistant structures to structurally controlled structures. Instead of designing a strong and often overdesigned building capable of sustaining extreme loads, the new objective is to reduce the vibration load applied to the building (Forrai *et al.*,2001).

Several types of passive strategies are now available, ranging from conventional methods, such as the addition of stiffness to the building, or non-conventional methods, such as the addition of passive dampers or base isolators. They either increase the lateral capacity of the structure or decrease the load demand on the structural elements of the building. None of these methods requires an external source of energy to work properly. (Sarno and Elnashai, 2002).

3.1.1 Conventional strategies

Conventional strategies have been widely used for the retrofit of steel and concrete buildings over the past decades, and are still used today despite the fact that non-conventional strategies would be more effective in many cases. Modifications can either be made to the critical structural elements of the building, such as the addition of stiffness, strength or ductility, or a completely new lateral load resisting system can be provided (Allen, 2007). The owner may also simply decide to reduce the load on the building by imposing a change in use or applying occupancy restrictions on the users (Allen, 2007).

The most common strategy consists of adding stiffness to the building. However, by increasing the stiffness, the seismic force demand on the structure is also increased. The walls or bracing elements have to be larger and stiffer, which again increases the demand. Non-structural elements can be transformed into structural ones, existing structural elements can be modified, and, finally, new components can be added. In most cases, the structural load path is modified, meaning that structural elements such as beam-column joints and building foundations have to be strengthened, at relatively high costs (Islam and Gupta, 2004).

Reducing the mass of the system is another interesting strategy to reduce the load demand on the building, but it also has some limitations. Several options are available to reduce the mass of a building, including the replacement of heavy cladding with lighter cladding, the replacement of masonry partition walls with lighter partitions or the removal of top floors and partial demolition of the building. However, a significant mass reduction is needed to improve the performance of structures, which is often not achievable (Islam and Gupta, 2004).

All these methods are often disruptive to the building and difficult to achieve. They also involve costly operations, such as heavy demolition, lengthy construction time and occupant relocation (Cheung *et al.*, no date). For all the aforementioned limitations, this thesis focuses on the use of non-conventional strategies for the retrofit of existing structures.

3.1.2 Damping Strategies

Damping mitigation strategies are increasing in popularity and are widely used for the seismic retrofit of existing buildings. They increase the energy absorption/dissipation capacity of structures by absorbing a large portion of the seismic energy that would otherwise be applied to the structural elements of the building. They reduce the vibration response by using the motion of the structure to develop control forces. Dampers usually use the relative motion of two structural elements to which they are linked to dissipate energy. Basic dissipation mechanisms include yielding of steel, viscoelastic action in rubber-like materials, shearing of viscous fluids, and sliding friction (Symans and Constantinou 1999, Connor, 2002).

Damping strategies offer many advantages over conventional strategies for the retrofit of existing buildings. First, they are less intrusive and can be installed relatively silently and quickly, without causing significant disruption to the building (Occhiuzzi 2009; Islam *et al.* 2004). Their application also has a reduced impact on building foundations due to the general reduction in seismic demand on the structural elements of the superstructure (Islam and Gupta, 2004). Consequently, this strategy is effective in reducing the overall construction cost and duration. Finally, their high performance level, combined with the possible achievement of immediate occupancy after an earthquake event, often results in the most suitable and economical retrofitting strategy (Mitchell, 2007).

Viscous Damping

Viscous dampers are used to reduce the earthquake induced motion of new and existing buildings. First used in the military industry in the late 1890s, they have been successively developed for structural applications and successfully applied to a wide variety of structures around the world (see Figure 11). Their success is due to their high effectiveness. Indeed, an optimized design can result in a 50% reduction in interstory drift and a 30% reduction in base and interstory shear (Lee and Taylor, 2001). They reduce both stress and deflection, a significant advantage over other mitigation devices. They also reduce the seismic force applied on buildings, thereby reducing potential injuries caused to people during an earthquake event. Other advantages include their high level of predictability at any temperature, their relatively small size, their ease of installation and maintenance, and their long service life (Taylor devices, 2009).

Figure 11:Viscous damper applications (Taylor, 2009)

Viscous dampers are velocity-dependent, meaning that they generate a damping force proportional to the velocity they experience. The magnitude of the force is given by:

$$F = CV^n, \tag{1}$$

where *C* is the damping coefficient of the device, *V* is the velocity experienced by the damper and n is a constant varying from 0.3 to 1.95. For each retrofit project, the optimal values of C and n must be found, and the optimal number and location of dampers must be determined (Lee and Taylor, 2001).

A typical viscous damper is shown in Figure 12. The mechanism by which energy is dissipated is relatively simple. As the damper experiences the relative velocity of the two stories to which it is attached, the piston rod pushes on the fluid. The latter then moves from one chamber to the other through a controlling valve at a rate proportional to the velocity of the rod. The amount of energy dissipated can be controlled by varying the opening of the valve. Viscous dampers act at 90 degrees out of phase with structural stiffness forces, which means that the damping force is a maximum when the building is at its initial position and when the velocity is maximum (Cheung *et al.,* no date). The dampers can be installed in parallel with base isolators; on a diagonal or as part of a chevron brace in a structural bay; horizontally at the top of a chevron brace; or even horizontally between two adjacent buildings (Lee and Taylor, 2001).

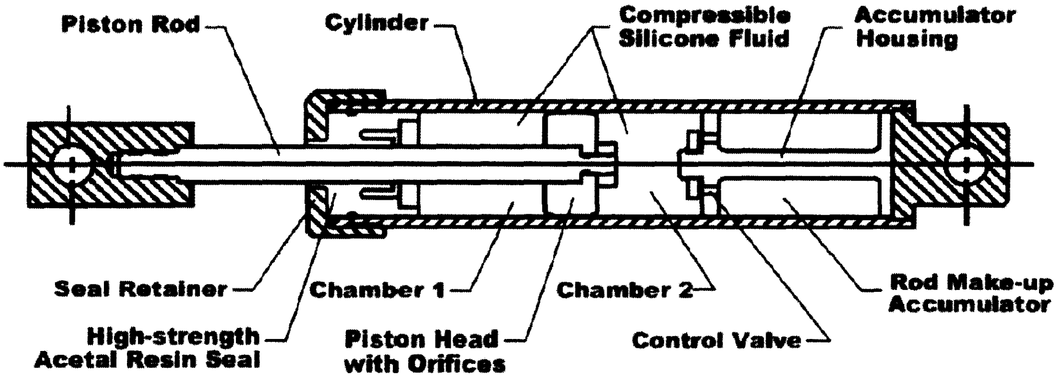


Figure 12: Viscous damper (Taylor, 2009)

Viscoelastic Dampers

Viscoelastic dampers result from a combination of elastic (displacement dependant) and viscous (velocity dependant) materials. They were used for the first time in 1969 to reduce wind vibrations on the Twin World Trade Center Towers in New York. Experimental testing has shown that they can reduce interstory drifts and structural accelerations by 50%. They are temperature dependant and, most importantly, they add stiffness to the system. Hence, they are not recommended for soft structures for which an increase in stiffness increases the response of the structure (Nielsen *et al.*, 1996).

Viscoelastic dampers consist of several layers of a viscoelastic material bonded to steel plates (Connor, 2002). Earthquake vibrations are reduced by the shearing action of the viscoelastic layers, and transformed into heat absorbed by the steel plates (Nielsen *et al.*, 1996).

Friction Dampers

Friction dampers behave similarly to the shock-absorbing system of a car: they both dissipate energy through friction and transform it into heat (Cheung *et al.*, no date). They are effective in reducing the resonance risk of buildings, since their natural period varies with the amplitude of vibration. They offer many advantages, such as independence to velocity and temperature, simple dissipation mechanism, low cost, ease of installation, and durability. Finally, for a given force and displacement, they dissipate more energy than other damping devices, meaning that fewer dampers can be used to achieve a desired performance level (Pasquin *et al.*, 2004).

Friction dampers developed for structural application can be found in the form of tension cross bracing, single diagonal bracing and chevron bracing (Pasquin *et al.*, 2004). They consist of a series of coated steel plates bolted together (see Figure 13) (Cheung *et al.,* no date). Under normal loading conditions (including wind loads), the plates do not slip. However, at a given earthquake load, they start to slip and dissipate a large amount of energy that would otherwise be released by the yielding of other structural elements (Pasquin *et al.*, 2004). The maximum force exerted by the damper is constant and is chosen at the design stage.

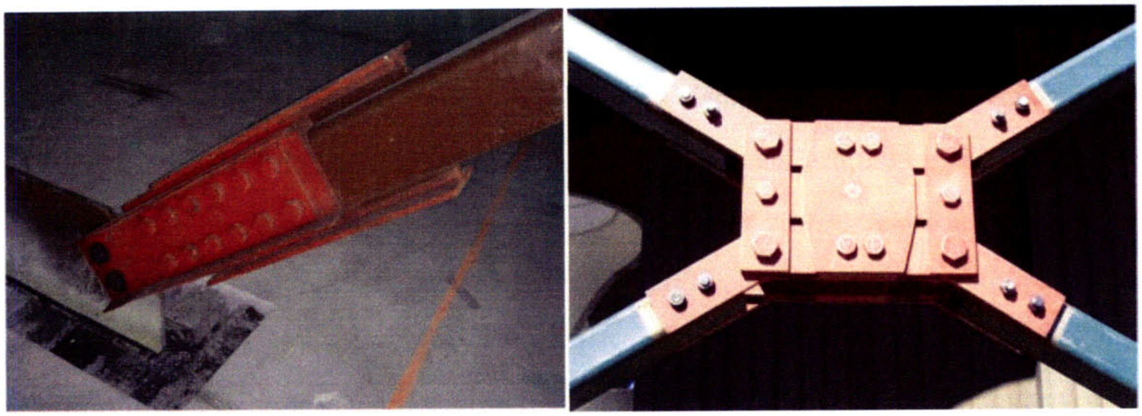


Figure 13: Friction dampers (Symans and Constantinou, 1999)

Hysteretic Dampers

Hysteretic dampers dissipate energy from inelastic deformations. The maximum force exerted by the damper is limited by the yield strength of the material used. These dampers usually consist of a low-strength steel core member of high ductility that is surrounded by a jacket in order to increase the buckling load of the core member (see Figure 14). The moment of inertia of the jacket is chosen such that buckling if prevented until the point where the core yields. Hence, these dampers are effective in both tension and compression (Connor, 2002).

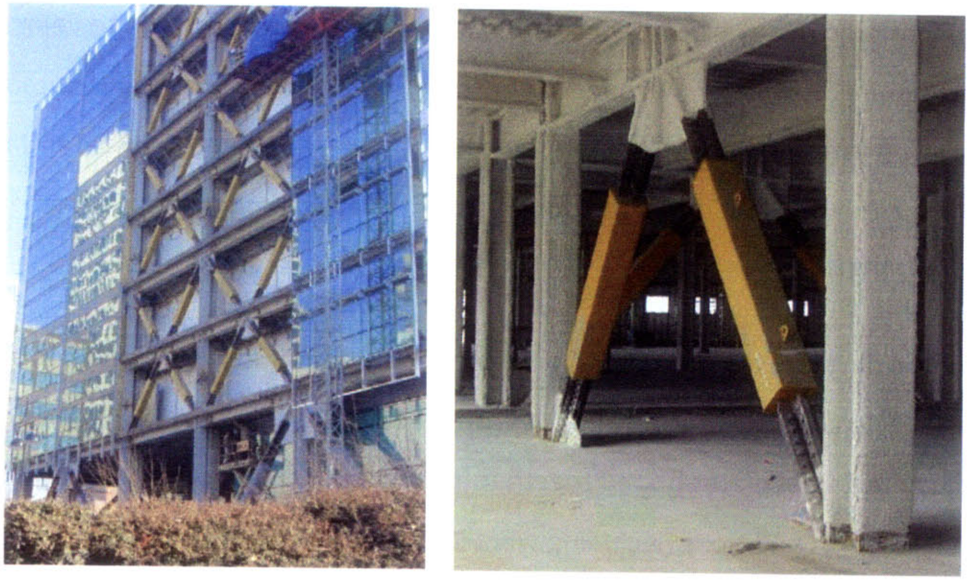


Figure 14: Hysteretic dampers (Symans and Constantinou, 1999)

Tuned mass Dampers

Tuned mass dampers (TMD) provide an effective and clever way to reduce the dynamic response of structures. First introduced by Fraham in 1909 for the ship industry, many other applications, including structural ones, have been developed during the last century (Connor, 2002). TMD consist of a mass, a damper, and a spring attached to a primary structure (see Figure 15). The mass is relatively small compared to the total mass of the building, in the order of a few percent (Hoang *et al.*, 2008). The damper is tuned to a frequency close to the fundamental frequency of the structure and oscillates out of phase with the primary structure, reducing its dynamic response. TMD can be used to reduce the vibration of a critical floor section locally or to reduce the total vibration of the building.

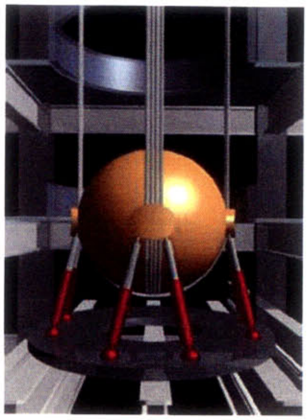


Figure 15: TMD in Taipei 101 (Taipei 101, 2009)

3.1.3 Base Isolators

The concept of seismic base isolation emerged in the late nineteenth century, but engineers had to wait until the mid-1980s for their commercialization (Connor, 2002). In recent years, base isolation has been found to be one of the most promising alternatives for the reduction of the dynamic response of new and existing structures. Following earthquake disasters such as the one experienced by the city of Kobe in 1995, base isolation is becoming a standard practice in countries of high seismic risk, such as Japan (Connor, 2002).

During an earthquake, underreinforced structures experience large interstory drifts that affect the integrity of the structure, as well as large lateral accelerations that may cause harm to the occupants or damage to the building content (Matsagar and Jangid, 2008). The base isolation strategy mitigates such effects by isolating the building or part of it from ground excitations. The acceleration of the superstructure is reduced while the lateral displacement across the isolation layer is increased (Symans and Constantinou, 1999). It has been shown that the acceleration of an isolated structure can be reduced by up to 80% compared to a non-isolated structure (LA DPW Engineering, 2005).

A base isolated structure experiences small deformations above the ground level, but significantly large deformations at the isolation level, which may affect other components of the building. In this case, the superstructure acts mostly as a rigid body, all floors moving together (see Figure 16). Hence, engineers must consider the configuration of the entrances of the building and the location of utility services, such as gas lines or sewers, to prevent any disruption that may result from excessive deformations across the isolation layer. They must also respect the minimal gap distance between the retrofitted structure and adjacent buildings, to avoid possible collisions (Matsagar and Jangid, 2008).

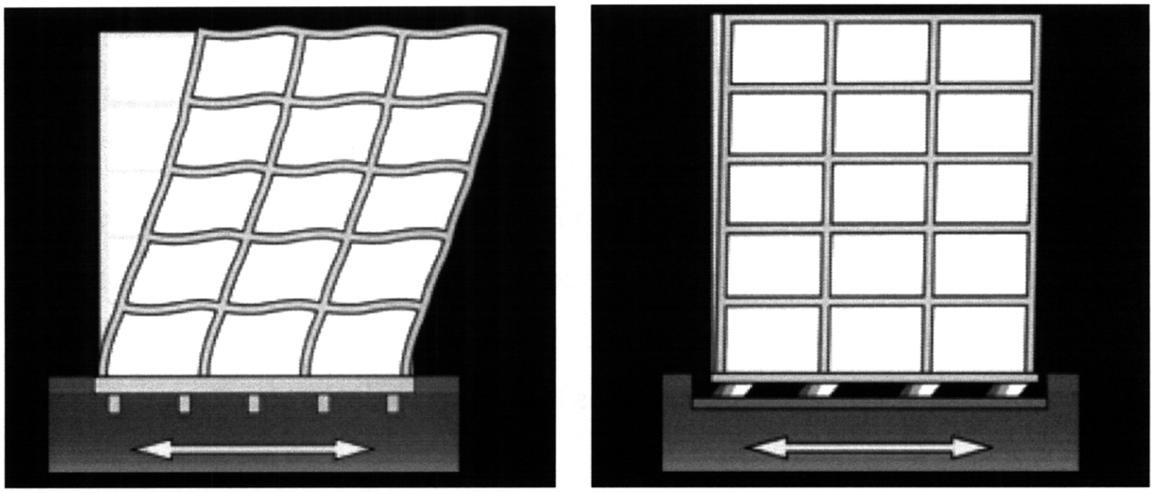


Figure 16: Effects of base isolation (a) Fixed base building, (b) base isolated building (Wang, 2002)

Base isolation offers many advantages over other mitigation strategies. Unlike conventional strategies that result in significant damage and potential down time after a strong earthquake, base isolation systems minimizes the damage caused to the superstructure and allows for immediate occupancy after an earthquake. A rather more significant advantage is their application to historic buildings. The installation of the isolation system does not affect their appearance and preserves their architectural qualities. Furthermore, it has a minimal impact on the building use since the work is being performed at the base of the structure only (Matsagar and Jangid, 2008).

Two distinct isolation systems are available: sliding systems and elastomeric bearing systems. Both systems are stiff in the vertical direction, but flexible in the horizontal direction.

Sliding System

The principle behind the sliding system consists of cutting off the load transmission path of the structure. The sliding interfaces provide a discontinuity between the ground motion and the superstructure and limit the transfer of shear forces to the building. Two types of sliding systems are available: the sliding isolation bearing system and the friction pendulum system (FPS) (see Figure 17). They both have been used for several projects around the world for new and existing constructions, but remain less popular than elastomeric bearing systems (Wang 2002; Kelly 1998).

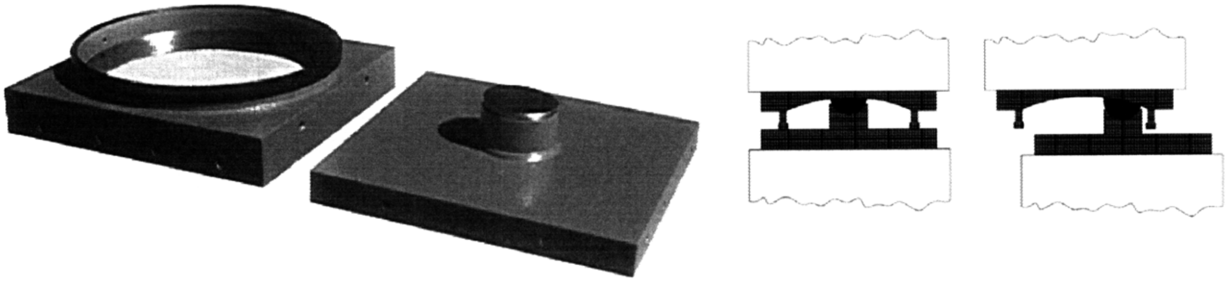


Figure 17: Friction pendulum systems (Wang, 2002)

Elastomeric Bearings System

The Elastomeric system decouples the building from the seismic ground motion by providing a soft layer of elastomeric bearings between the building structure and its foundations (Kelly, 1998). It is laterally flexible, resulting in a layer of low stiffness that reduces the fundamental period of the superstructure and moves it far from the predominant frequency content of the earthquake. Resonance problems are thereby avoided (Wang, 2002).

The first mode of vibration of isolated structures consists of the deformation of the elastomeric system only, the superstructure acting as one rigid element (Kelly, 1998). Higher modes of vibration are orthogonal to the first mode and to the ground motion, and do not participate in the deformation of the building.

Several types of bearings can be used, such as high-damping rubber bearing (HDRB) or lead-rubber bearing (LRB) (see Figure 18). HDRB are the most common (Matsagar and Jangid, 2008). They consist of several layers of circular steel and rubber plates squeezed between two larger and thicker steel plates. The bearings are flexible horizontally, but very stiff vertically. Their fabrication is relatively simple and they have a large resistance to environmental effects. LRB consists of HDRB with an added lead core that dissipates additional energy. Damping is provided through the yielding of the lead core, thereby absorbing part of the ground motion energy (Matsagar and Jangid, 2008).

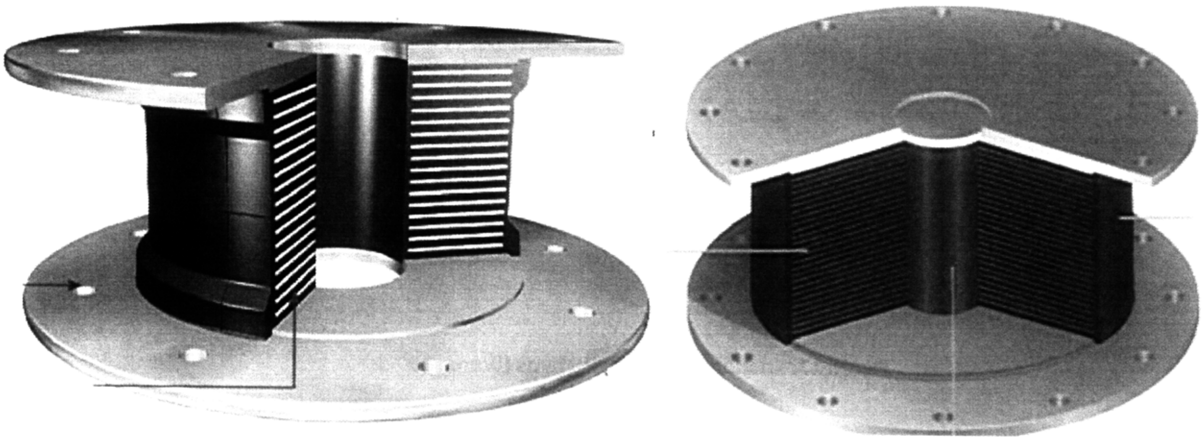


Figure 18: HDRB and LRB (Wang, 2002)

Some conditions must be fulfilled for the implementation of base isolation systems: a squat geometry with significantly high column loads, a site that allows for horizontal displacement of more than 200mm at the ground level, and wind loads in the order of less than 10% of the weight of the structure (Deb, 2004).

3.1.4 Fiber Reinforced Polymers

Fiber Reinforced Polymers (FRPs) were first introduced in the market for their military and aerospace applications. In the early 1940, they found their way to civil engineering in response to the increasing need for stronger and more durable materials. Recently, major advancements made during the 1990's led to an increasing use of this innovative material for both new and existing infrastructures, at a time where the use of traditional materials often resulted in higher costs and intrusiveness (Sieble, no date).

Some of the most important advantages of FRPs include their high strength to weight ratio, high flexibility and versatility, low maintenance, ease of installation, low installation time, corrosion resistance and low life-cycle costs. They are considered to be a one of the most cost-effective and efficient seismic mitigation strategies for a wide variety of applications (Cheung *et al.,* no date). Some of these methods are presented below. Especially noteworthy is that all of them are used without adding mass to the system, a significant advantage over conventional strengthening strategies.

CFRP Sheets and Strips

FRPs are often used to increase the strength and ductility of structural members such as columns, beams and walls. FRP sheets can be wrapped around columns and beams, while FRP strips are glued onto beams and walls (see Figure 19). Past experiments have demonstrated that these interventions do not add stiffness to the structural elements, a serious advantage when an increase in seismic force demand is undesirable (Cheung *et al.*, no date).

Reinforced concrete columns in older concrete buildings often have problems of inadequate confinement, inadequate amount of horizontal reinforcement, or inadequate lap spice of vertical reinforcement, which can all be solved using a simple CFRP jacket (see Figure 20) (Cheung *et al.*, no date). This method can also be used masonry brick columns, which represents a fast and effective way to mitigate potential damage caused by future earthquakes. Even thought CFRPs are most commonly used for concrete or masonry structures, they can also be used for steel frame buildings as an alternative method to the addition of structural steel plates (Hollaway, 2003).

Figure 19: Retrofit of beam-column joint, slab and wall in RC buildings using CFRP (Sika, 2009)

Figure 20: Retrofit of a RC column using a FRP jacket (Sps Repair, 2009)

FRPs can also be used in confined areas, where it may be difficult to bring on site a long and heavy piece of rigid metal frame or where space for adequate bolted or welded connections is not available. With their high flexibility and their ease of transportation, handling, and installation, FRP may be the most suitable solution for a wide variety of applications.

Fiber Reinforced Cement

Fiber Reinforced Cement (FRC) is an attractive solution for the strengthening of existing structures. This is particularly true for unreinforced masonry structures, which often suffer serviceability problems. Conventional retrofitting strategies for such structures consist of either replacing the interior load bearing walls with a lighter frame, or to apply a structural overlay, both leading to an increase in stiffness (Cheung *et al.*, no date). When such effect is undesirable, an effective solution is to add a FRP overlay onto the masonry wall. This system consists of a thin layer of FRP reinforced by a high strength fiberglass mesh. Such operation increases the strength and ductility of the masonry wall without adding stiffness to the building(Cheung *et al.*, no date).

CFRP External Prestressing

CFRP External Prestressing is another efficient way to improve the confinement of deficient concrete columns. This method consists of wrapping the existing column with prestressed cable strands. These cables exert an active pressure onto the column face and increase its lateral confinement, and, consequently, their performance under earthquake loadings (Cheung *et al.*, no date).

3.2 ACTIVE AND SEMI-ACTIVE STRATEGIES

Passive control strategies have shown their great advantages in a wide variety of applications. However, their incapacity to adapt to future dynamic loads and the difficulty to modify them once they are designed and installed leave space for potential improvements. Indeed, passive systems are often overdesigned to account for the unpredictability of dynamic loadings (Connor, 2002).

Recently, active and semi-active control systems have been developed to account for these problems and are used to intelligently control the response of structures. Connor (2002) describes the active structural control system as:

> 'one that has the ability to determine the present state of the structure, decide on a set of actions that will change this state to a more desirable one, and carry out these actions in a controlled manner and in a short period of time'.

These systems can adapt to unpredictable earthquakes while maintaining desired performance levels and limiting the number of failures of critical structural components of a building (Connor, 2002).

3.2.1 Active Control

The first attempt to the application of active control in the structural field has been made in the 1960s by Eugene Freyssinet, who developed a principle of structural stabilization of tall buildings by the use of prestressing tendons as control devices (Soong, 1987). The principle behind active control systems consists of the supply of an external power source to reduce the seismic response of buildings. They control the power input force on the structure and adjust it in order to bring the response closer to the desired response (Connor 2002; Sarno and Elnashai 2002). To achieve this, a monitor first measures the external loading excitation by the use of sensors. The data collected are then directed to a cognitive controller system that identifies the state of the system, decides on the course of action and develops the action plan to be transferred to an actuator system using a pre-determined control algorithm. The actuators then carry out the instructions from the controller, and apply a time-varying force on the structure (Connor, 2002).

The control force can either be determined based on the excitation monitoring feedback only, or can be adjusted based on the actual response feedback of the structure. Those two control systems are called open-loop control and feedback control, respectively. Not only the feedback control system monitors the input excitation, but it also monitors the response of the building. It uses this information to make continual corrections to the applied control force and adjust it to minimize the response of the structure (Connor, 2002). A schematic representation of an active control feedback system is presented in Figure 21.

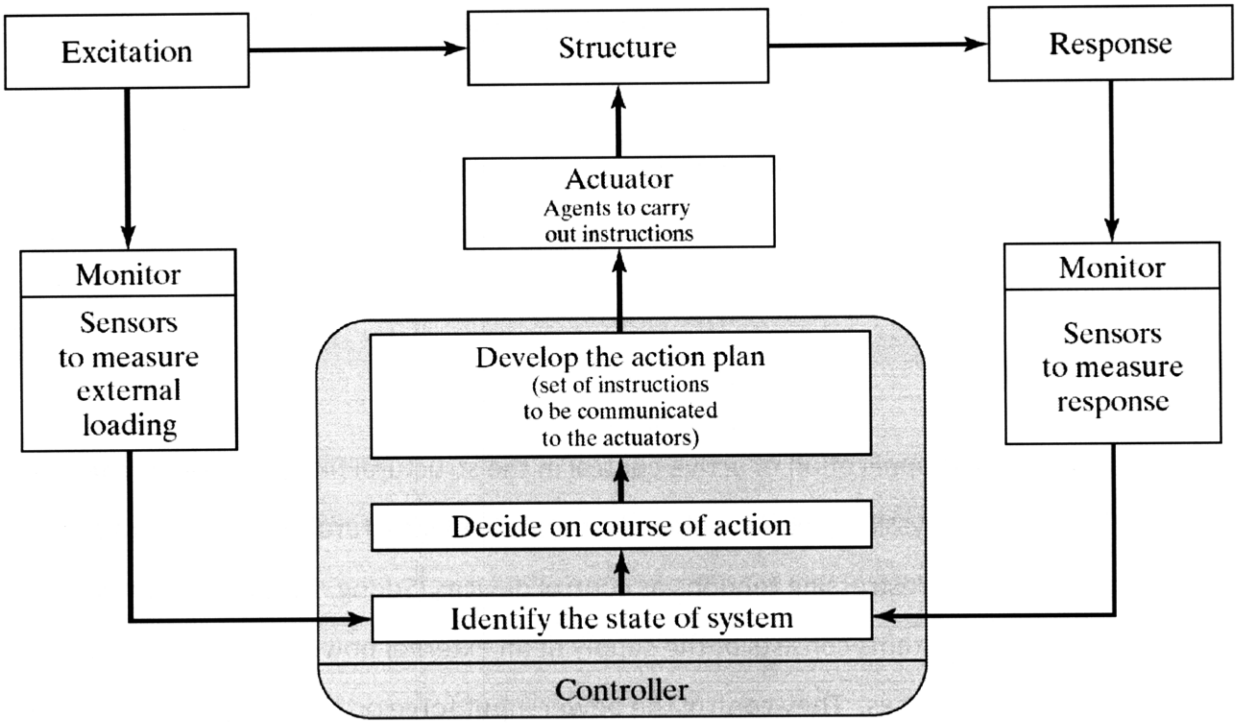
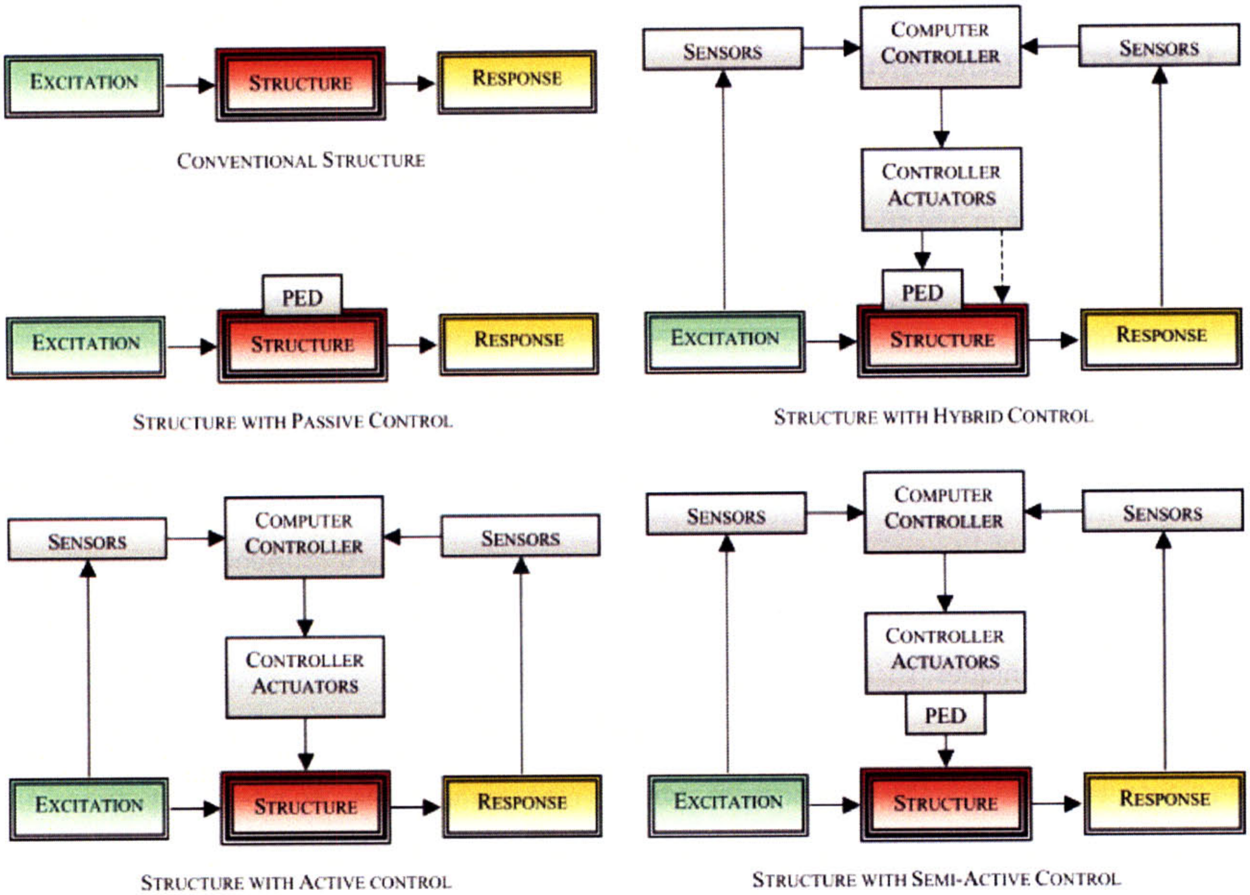


Figure 21: Components of an active control system (Connor, 2002)

The actuators can be either electrohydraulic or electromechanical, and supply the control forces to the structure. In the case of small structures, a power input in the order of tens of kilowatts is generally required. For large structures, it may reach many megawatts. One major disadvantage of active control systems is their high demand on power sources for the operation of the actuators. Of greater concern is the fact that the source of power may not always be available during the earthquake. Moreover, by adding energy to the system, they can also cause instability problems (Symans *et al.*, 1999).

3.2.2 Semi-Active Control

Semi-active control systems for a structural application have been developed by Hrovat *et al.* (1983). They are a compromise between passive and active control systems, combining the reliability of passive control with the adaptability of active control. Similarly to active control systems, they require an external energy source, but this external power is much smaller than the amount required for active control, in the order of tens of watts (Symans and Constantinou. 1999, Connor, 2002). In fact, semi-active devices do not add energy to the system and can be seen as controllable passive devices (Sarno and Elnashai, 2002). They are used when the power demand for the active control system is too large and cannot be afforded. Figure 22 presents the possible control schemes for the control of structures.



Conventional Structure

Structure with Passive Control

Structure with Hybrid Control

Structure with Active control

Structure with Semi-Active Control

Figure 22: Application of control schemes to mitigate structural vibrations. PDE = passive energy dissipation (Sarno and Elnashai, 2005)

4.0 RETROFITTING STRATEGIES: APPLICATIONS AND BENEFITS

Two distinct approaches can be used for the seismic retrofit of existing buildings: the local modification of individual structural members and the global retrofit of the structural system. Local modifications are preferably used for the repair of structural members damaged by an earthquake. They usually consist of adding strength, stiffness or ductility to the members. Global modifications are generally used to either increase the overall capacity of the structure or to reduce the force demand on the building during an earthquake (Allen, 2007).

Factors to consider for the selection of the most suitable mitigation strategy are the type of the building, the desired performance level, the site conditions, and construction and installation costs. The next section provides a general overview of the applicability of passive devices to masonry, concrete and steel buildings.

4.1 MASONRY BUILDINGS

Masonry buildings are highly vulnerable to earthquake loads. Under fast and strong dynamic loads, they behave in a brittle manner, resulting in catastrophic failures. Common strategies used for the seismic retrofit of such structures are the addition of a new lateral resisting system, the coating of existing walls or the isolation of the building from ground motion.

The construction of a new lateral load resisting system is often exhaustive and intrusive. Indeed, the addition of concrete shear walls or steel bracings is even impossible in many cases (Taghdi *et al.*, 1998). A better strategy consists of applying a surface treatment on the existing walls to increase their ductility and their lateral resistance. This can be achieve by the use of conventional techniques, such as ferrocement, post-tensioning, shotcrete, or added steel plates (ElGawady *et al.*, 2004). However, these techniques are also impractical and costly. Moreover, they add mass to the structure, which is often undesirable. An interesting alternative that is increasing in popularity is the use of fiber reinforced polymers. The application of FRP strips to existing walls can increase their ductility and flexural capacity under both in plane and out-of-plane loadings, without significantly increasing the mass of the building (Willis *et al.*, 2009).

Figure 23: Damage to URM walls retrofit with vertical FRP strips (Willis *etal.*, 2009)

Another strategy consists of isolating the building from ground excitation using base isolators. This method is advantageous when it is impossible to modify the superstructure, either because building occupants can not be relocated, because overall conditions do not allow for such modifications, or because the historic value of the building has to be preserved. However, the installation of the base isolators can also be quite exhaustive. The building must first be underpinned to support the superstructure while needle beams are cast directly under the masonry walls in which holes have been created. The beams are supported by the base isolators installed at specified spacing. Temporary support can then be removed as the loads are gradually transferred from the walls to the needle beams to the base isolators. Figure 24 shows a typical arrangement of such system (Kelly 1998; Matsagar and Jangid 2008).

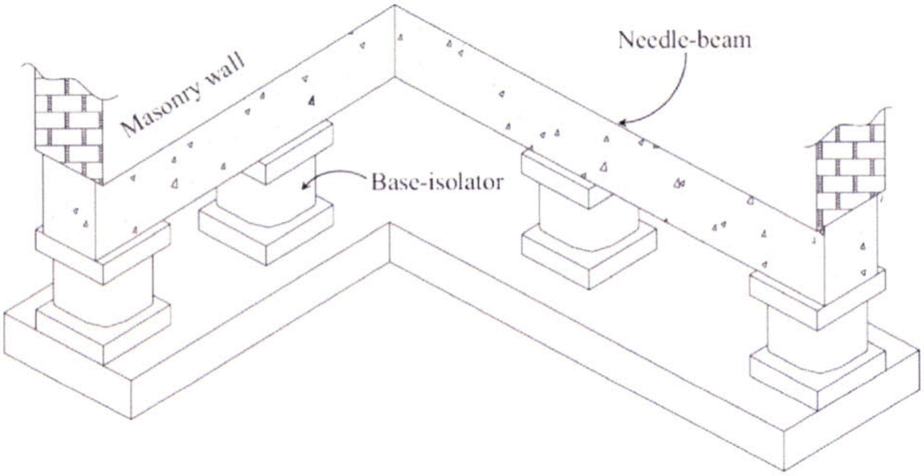


Figure 24: Details of base isolated masonry structure (Matsagar and Jangid, 2008)

4.2 CONCRETE FRAME BUILDINGS

To prevent damage in reinforced concrete buildings, all seismic strategies described in Section 3 can be used. First, concrete shear walls can be strengthened or steel bracings can be added to increase the stiffness of the building. However, as mentioned previously, increasing the stiffness increases the force demand on the building and consequently the force on some structural elements, such as columns and foundations. In many cases, strengthening these elements would be too costly and time consuming (Bai and Hueste, 2007).

To overcome these problems, a selective weakening strategy has been proposed (Ireland *et al.*, 2006). It consists of reducing the load demand on the building by modifying the inelastic deformation mechanism of the structural elements to achieve a higher performance level. To be effective, this method must be used in combination with dissipation devices.

Increasing the dissipation capacity of the building is the next alternative. The choice between passive or active strategies and between all types of structural dampers depends on the desired performance level and the cost of the devices. However, in some cases, the addition of damping may increase the force applied to the building, in which case this strategy would be undesirable (Bai and Hueste, 2007).

It may happen that only few structural elements need to be strengthened and that the overall retrofit of the structure would be unnecessary and costly. In such cases, fiber reinforced polymers offer many advantages. Concrete columns can be strengthened by the use of FRP jackets; punching shear failure in flat concrete slab can be prevented by applying FRP strips on the slab around the column; and finally the performance of beam-column joints can be increased by applying FRP sheets around the joints (Bai and Huste, 2007).

Finally, base isolation can also be used, in which case a base-raft and several base isolators are installed as shown in Figure 25. Here, the columns are cut at their base and sit on the base raft on top of base isolators.

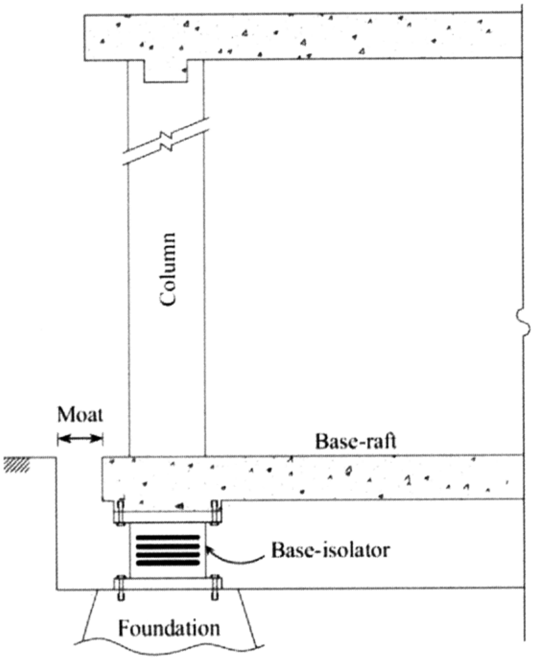


Figure 25: Details of base isolated concrete frame structure (Matsagar and Jangid, 2008)

4.3 STEEL FRAME BUILDINGS

Steel frame buildings can be retrofitted using several approaches that can be divided into two main categories: local modifications of structural components or global modifications of the structural system. The main concept is to achieve sufficient ductility by providing enough inelastic dissipative zones and ensuring the linear behavior of other members and connections. Local modifications alone usually do not allow for an adequate retrofit level. However, their retrofit is essential to avoid brittle failures and reach a minimum ductility level. In the case of columns and beams, the member should be locally retrofitted to avoid premature failures and achieve a minimum stability level Unfortunately, due to the large number of members and connections in a typical frame structure, it is not cost-effective to repair them all. Hence, it is often suitable to select a global strategy that does not require too many local interventions (Sarno and Elnashai, 2002).

Figure 26 presents the possible retrofit strategies for steel and composite buildings. One of the most common strategies consists of limiting structural deformations by increasing the stiffness of the structure through the addition of steel braces or shear walls However, increasing the stiffness increases the force demand on the structure, which is highly undesirable. Reducing the mass of the building is also an interesting option, but is usually harder to achieve (Sarno and Elnashai, 2002).

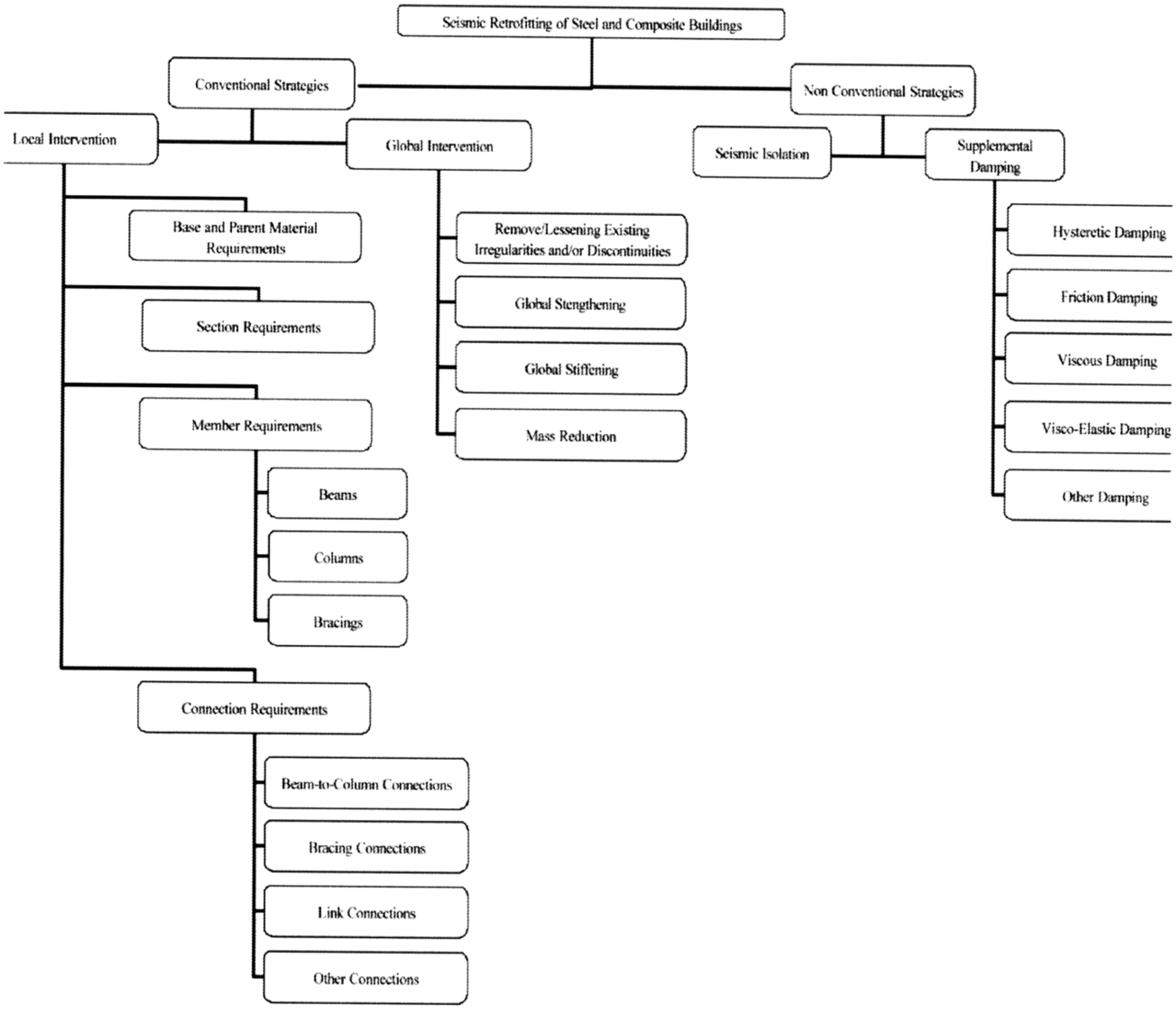


Figure 26: Retrofitting strategies for steel and composite buildings (Sarno and Elnashai, 2002)

One of the most effective strategies consists of increasing the dissipation capacity of the structure by the addition of damping devices. This is particularly true for steel structures. Their lateral flexibility and, consequently, their relatively large deformation can be used to dissipate energy at low-level loadings. According to Sarno and Elnashai (2002), structural damping of steel buildings can be increased up to 50.0% for viscous and viscoelastic dampers and up to 20% for hysteretic and friction dampers. Force concentrations on the structural elements of the building are also reduced. However, achieving such a high damping level would be expensive (Connor, 2002).

Table 2 presents a summary of the various retrofitting strategies for moment resisting frame, concentrically braced frame and eccentrically braced frame buildings. In addition to the strategies presented above, base isolation or innovative materials can also be used but are less popular for steel buildings.

Table 2: Retrofitting strategies for steel frame structures (Sarno and Elnashai, 2002)

MRFs	CBFs	EBFs
• Encase columns and/or beams in RC. • Strengthen/weaken beam-to-column and column-base connections. • Augment composite slab participation. • Add braces to one or more bays at each story. • Add concrete or masonry infills to one or more bays at each story. • Add concrete shear wall. • Add new MRFs to the exterior of the building. • Use base isolation systems. • Add supplemental damping devices.	• Replace or modify braces with insufficient strength/ductility. • Replace tension only systems with tension/compression braces. • Encase columns and/or beams in RC. • Change the brace configuration. • Add concrete or masonry infills to one or more bays at each story. • Add concrete shear wall. • Reinforce inadequate connections. • Use base isolation systems. • Add supplemental damping devices.	• Add cover plates to beam flanges at link location. • Add doubler plates and/or web stiffeners to the link web. • Change the brace configuration.

5.0 CASE STUDY: HYBRID BASE ISOLATION SYSTEMS

Base isolation is one of the most promising alternatives for the seismic retrofit of existing structures. Many experiments and field applications have shown their high effectiveness under severe earthquake loadings. In addition to significantly reduce the acceleration and displacement experienced by the superstructure, it is the only technique that is not intrusive to the superstructure and does not affect its appearance, a serious advantage in the case of historical buildings with patrimonial value. Moreover, the work can be carried out while the building is occupied. Most importantly, it is the easiest way to shift the fundamental frequency of the building far from the frequency content of the earthquake, an important advantage for low-rise stiff concrete structures subject to high frequency earthquakes typically found in Canada. For all the aforementioned reasons, the base isolation strategy is used for the case study presented below. The elastomeric bearing system has been chosen because it is the most commonly used (Matsagar and Jangid, 2008).

5.1 PASSIVE SEISMIC ISOLATION SYSTEMS

Passive applications of base isolation have been previously reviewed in Section 3.1.3. Their reliability under large earthquake loadings has been proven throughout many experiments and field applications. However, the performance of such systems under other service loads is sometimes critical. For example, wind loads applied to the superstructure may result in excessively large displacements of the entire structure due to the presence of a low stiffness layer at the base of the building (Connor, 2002).

Hybrid Isolation systems have been designed to overcome this problem. A hybrid system generally consists of a combination of a base isolation system with a passive damping system (viscous, hysteretic or friction dampers) (Symans *et al.* 1999; Connor 2002). The isolation system reduces the fundamental frequency and spectral acceleration of the structure, while the damping system limits its displacement response. The dampers and the base isolators act in parallel to increase the overall energy dissipation capacity of the building (see Figures 27 and 28). This type of mitigation strategy has been accepted by the engineering community and is now being used in a growing number of applications due to its high effectiveness in reducing the motion of buildings and its intrinsic reliability (Symans *et al.*, 1999).

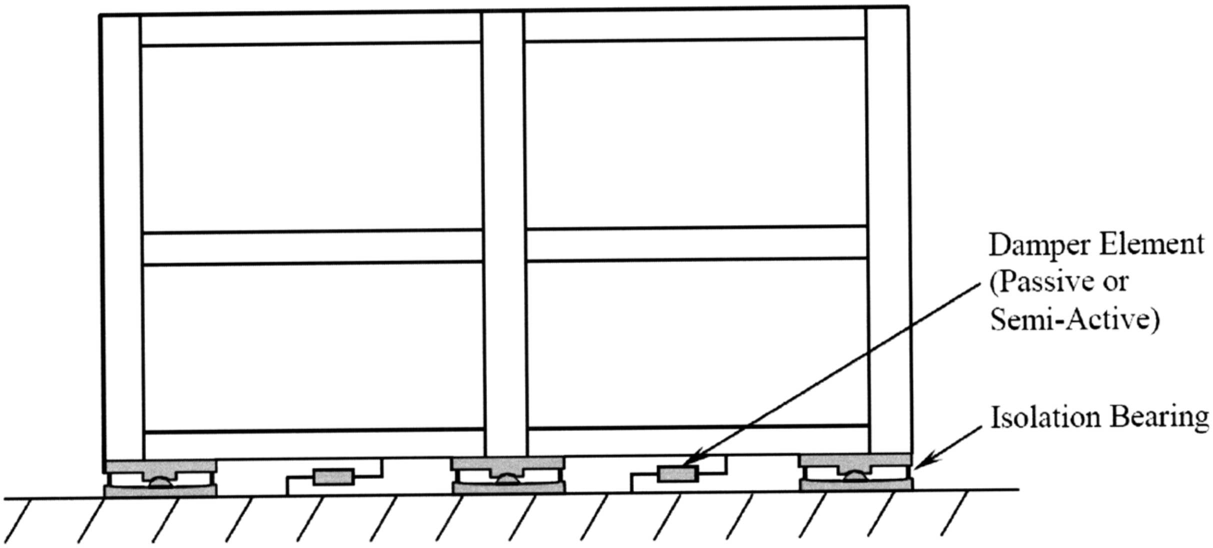


Figure 27: Building Structure Containing Hybrid Seismic Isolation System (Symans *et al.*, 1999)

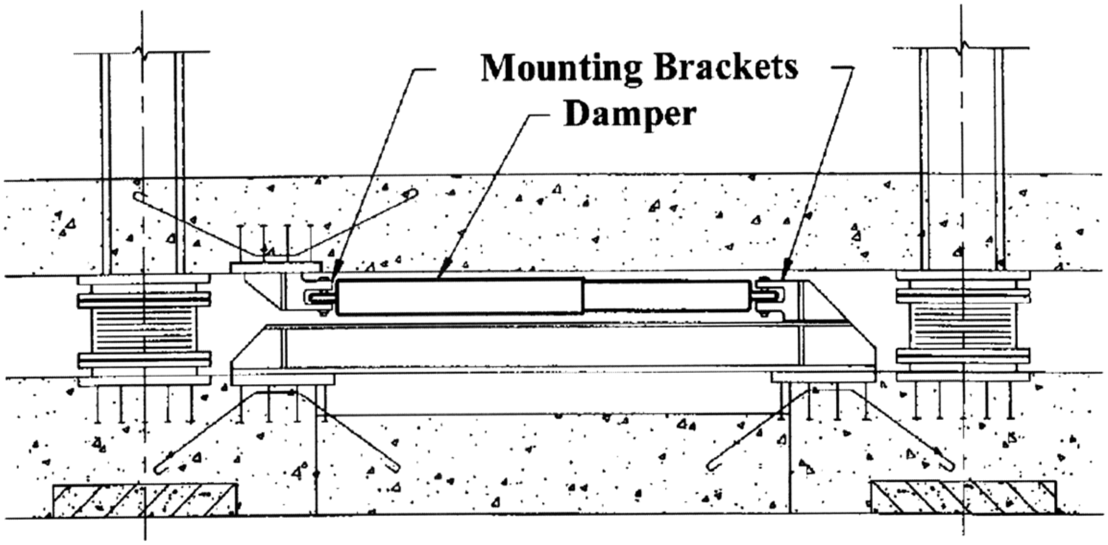


Figure 28: Dampers in parallel with base isolators (Lee and Taylor, 2001)

5.2 SEMI-ACTIVE HYBRID SEISMIC ISOLATION SYSTEMS

The performance of passive hybrid isolation systems can be improved by replacing passive dampers with semi-active dampers to control the energy dissipation of the building. They overcome the limitation of passive systems designed for extreme seismic events. Semi-active systems can adapt to a wide variety of earthquakes of small or large scale. Such systems consist of a combination of a passive base isolation system with a semi-active damping system (see Figure 27) (Symans *et al.*, 1999).

Common types of semi-active dampers used in base isolation applications include variable-orifice fluid dampers, magneto-rheological dampers and electromagnetic friction dampers. Other control devices can be used, such as variable stiffness devices or controllable friction devices. Many experiments have shown their high performance under disparate ground motions of various intensities and characteristics, which cannot be achieved using passive damping. It has been shown that the base displacement and base shear of isolated buildings can be reduced by half of the original values (Wilson, 2005).

The concept of semi-active control has been defined previously in section 3.2. The mechanical properties of the dampers are adjusted based on feedback of sensors measuring the ground excitation and/or the instantaneous response of the structure. The damper used for this case study consists of a magneteorological (MR) damper (see Figure 29).

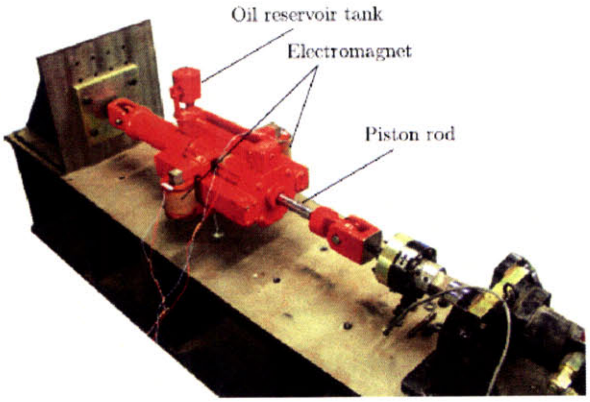


Figure 29: 20 kN MR Fluid Damper (Rodriguez *et al.*, 2009)

MR dampers are similar to linear viscous dampers, except that the fluid inside the piston is filled with MR fluid containing small polarizable particles. When a magnetic field is applied, the viscosity of the fluid and the properties of the dampers are modified. Coil is wrapped around the piston head to adjust the magnitude of the magnetic field. When the magnetic field is applied, the fluid becomes semi-solid; when no current is supplied to the coil, the MR damper acts as a simple viscous damper(Lin et al, no date). They are divided into an upper chamber and a lower chamber by a piston. The MR fluid flows from one chamber to the other through an orifice at both ends (see Figure 30).

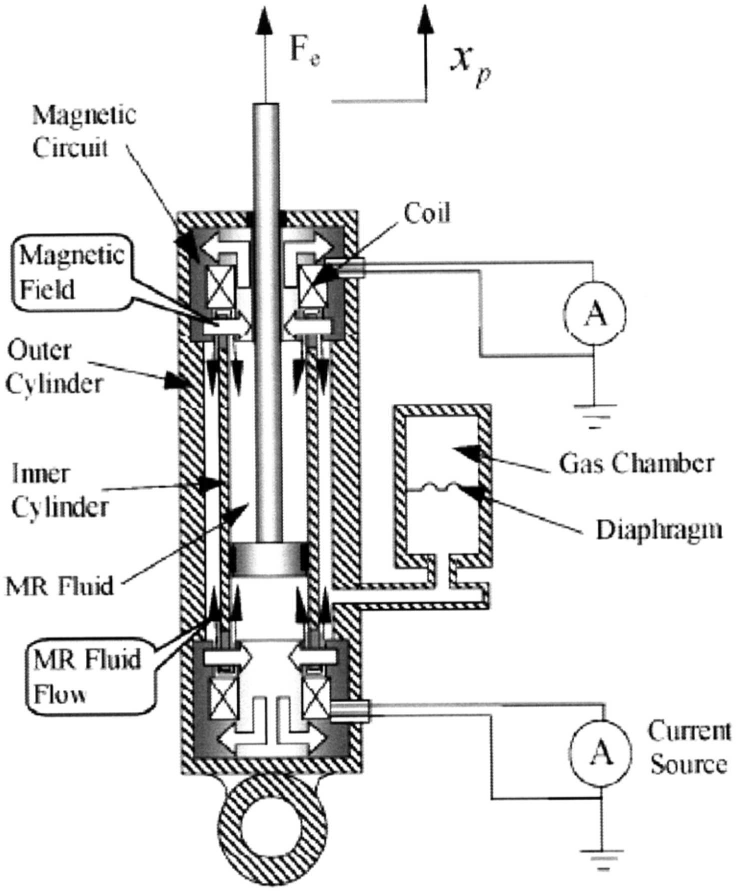


Figure 30: Configuration of the MR Damper (Choi and Sung, 2008)

5.3 EXPERIMENTAL MODEL

The building analyzed here consists of a two-story, concrete frame building modeled as a three-degree-of-freedom system. For the purpose of this analysis, it is assumed that the superstructure would remain elastic during the earthquake excitation while the isolation level would behave in an inelastic manner. The structure consists a square building, four bays wide, with columns spaced at every 6 meters. The estimated properties of the buildings are presented below.

Dimensions: 24m x 24m
Height: 12m
Mass/Floor: 300 000 Kg
Fundamental period 0.3 sec.
Initial damping: 2% of critical damping

The performance of four different models is evaluated (see Figure 31). The first model consists of the primary, fixed-base structure, with no additional damping. In the second model, a base isolation layer is added. The third model is a passive hybrid system that consists of a MR damper without energy supply acting in parallel with the base isolators. In this case, the MR damper acts like a simple viscous damper. Finally, the same system is evaluated with a semi-active control of the MR damper. A power supply is provided to modify the properties of the damper, which properties are listed in Table 3.

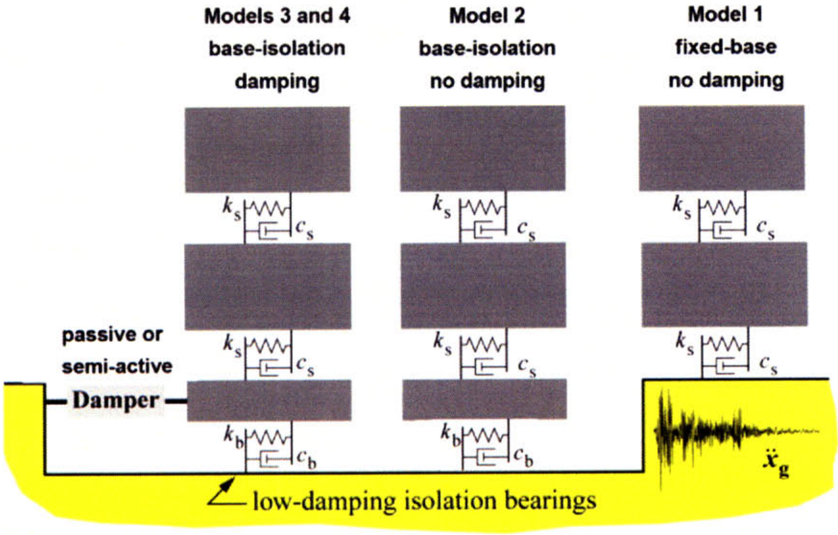
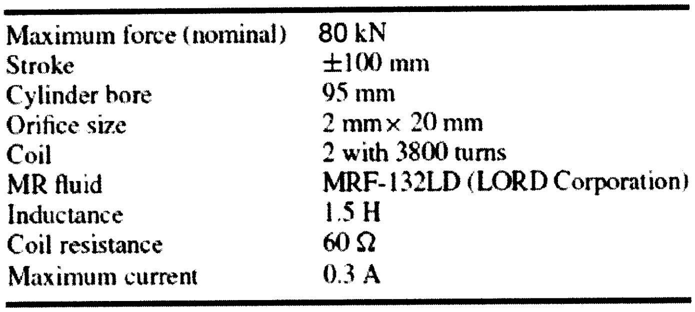


Figure 31: For models analyzed (Adapted from Ramello *et al.*, no date)

Table3: 80kN MR Damper Properties (Adapted from Rodriguez et al, 2009)



Maximum force (nominal)	80 kN
Stroke	±100 mm
Cylinder bore	95 mm
Orifice size	2 mm × 20 mm
Coil	2 with 3800 turns
MR fluid	MRF-132LD (LORD Corporation)
Inductance	1.5 H
Coil resistance	60 Ω
Maximum current	0.3 A

The building is modeled in the SIMULINK program from Matlab. The response has been evaluated for a variety of earthquakes. The result of the analysis of the building excited by the Northridge earthquake is presented in Section 6. This earthquake is chosen because of its high frequency content typical of Canadian earthquakes. This type of earthquake is critical for stiff low-rise buildings commonly found across Canada.

6.0 RESULTS

6.1 PRIMARY STRUCTURE

The displacements of the three levels of the primary, fixed-base structure are plotted in Figure 32. The period of the primary structure is 0.22 sec. As shown, the displacement of the roof far exceeds the limit of H/200 (0.04m) for a building of this type. Notice that the interstory displacements are also significantly large. The acceleration of the three levels far exceeds the acceptable limit. Indeed, an acceleration of almost 5g is experienced by the roof (see Figure 33).

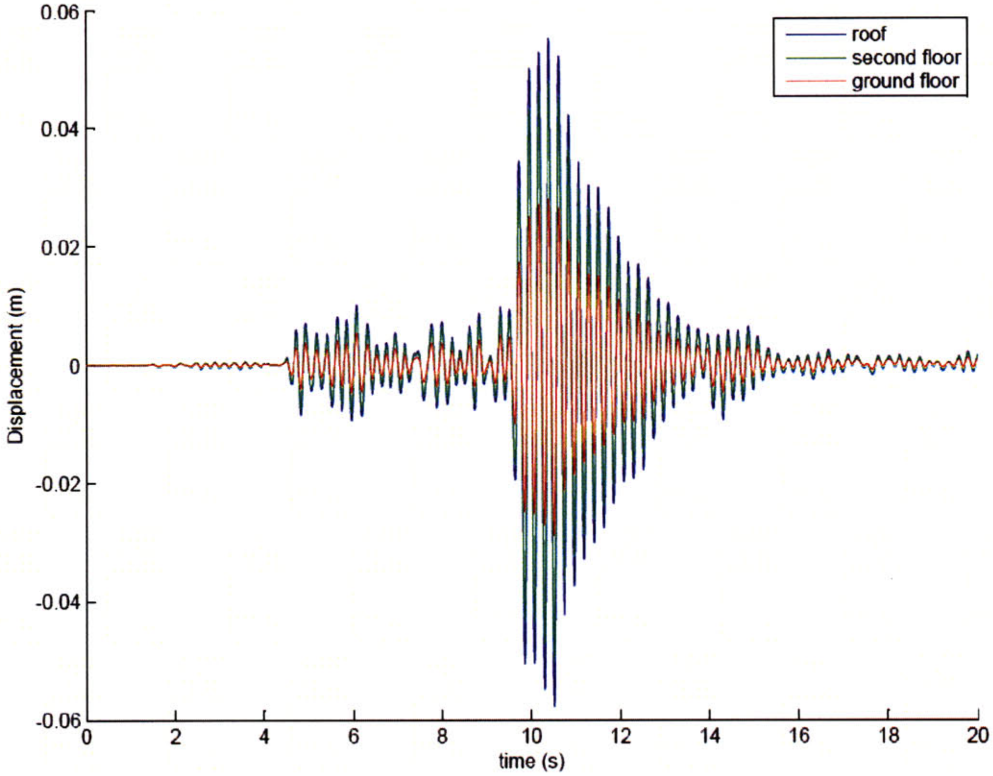


Figure 32: Displacement response of the primary structure, no isolation, no damping (Northridge EQ)

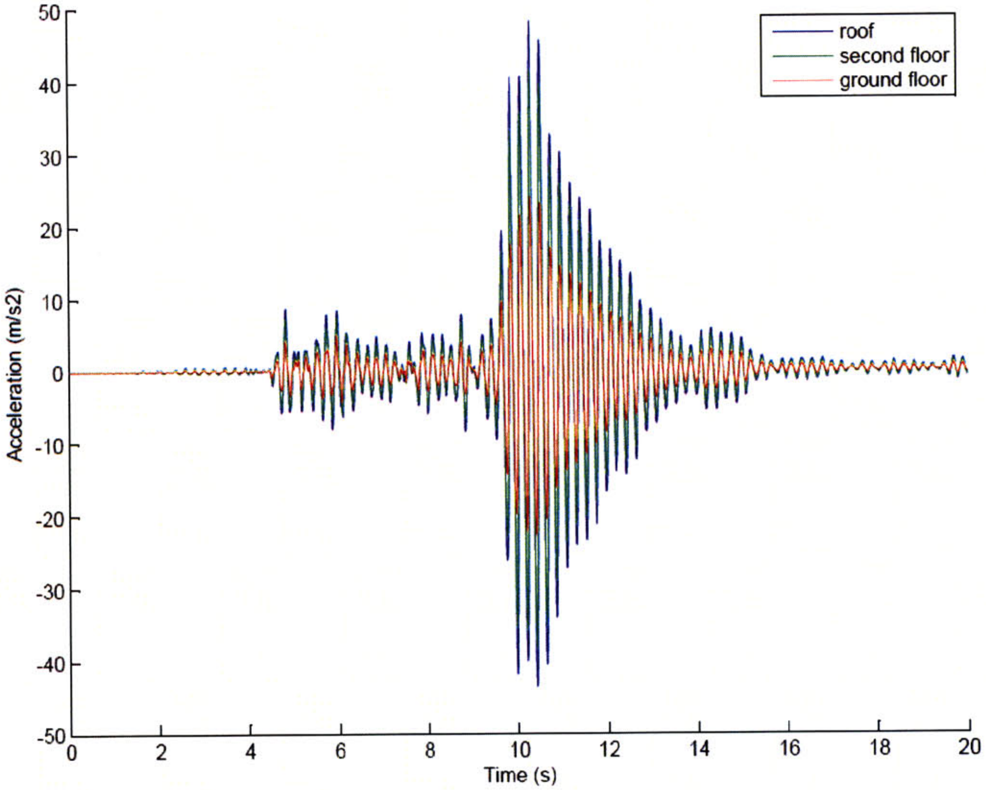


Figure 33: Acceleration response of the primary structure, no isolation, no damping (Northridge EQ)

6.2 BASE ISOLATED STRUCTURE

The same building with an additional base isolation layer is analyzed. In this case, the period is increased from 0.22 sec to 0.59 sec. The interstory displacement in this case is highly reduced, the superstructure acting as one rigid element (see Figure 34). The total displacement of the second floor and the roof are also reduced. However, as mentioned previously, the performance of base isolated structures under lower frequency earthquakes can be critical. Therefore, the same structure is analyzed under the ElCentro Earthquake. As shown in Figure 35, the total displacement increases from 0.035m to 0.07m. Therefore, the implementation of a hybrid base isolation system that will dissipate additional energy is highly suitable and is presented in the next section.

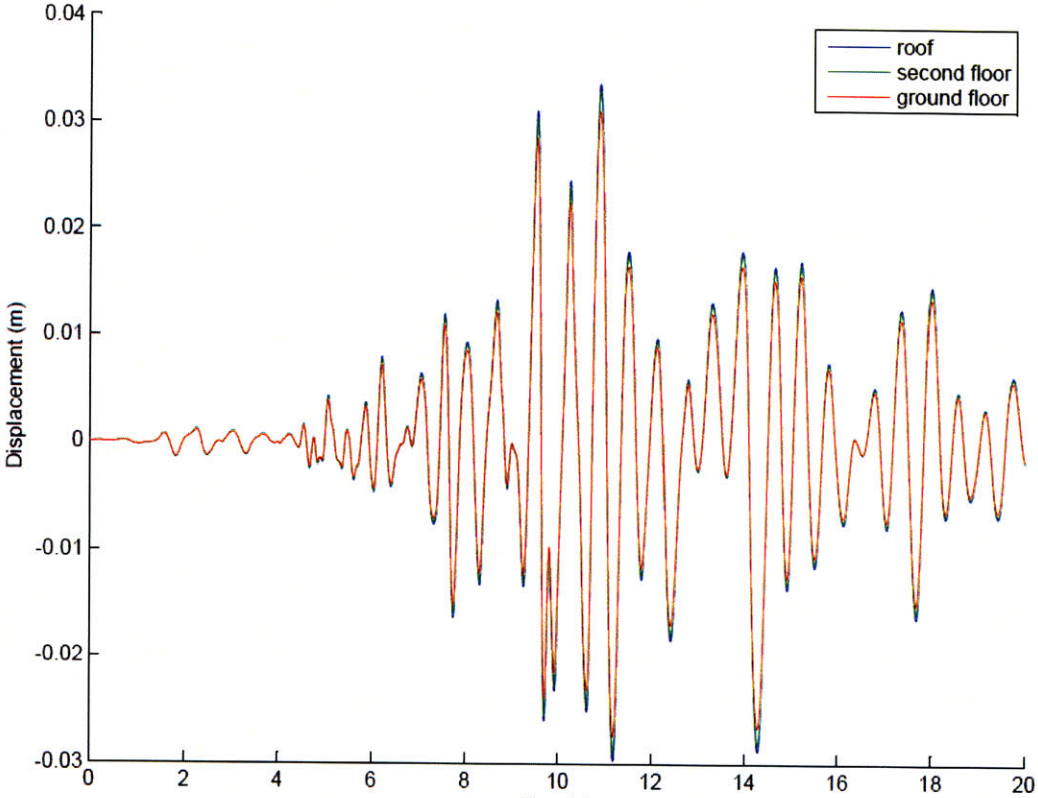


Figure 34: Displacement of the isolated structure, no damping (Northridge EQ)

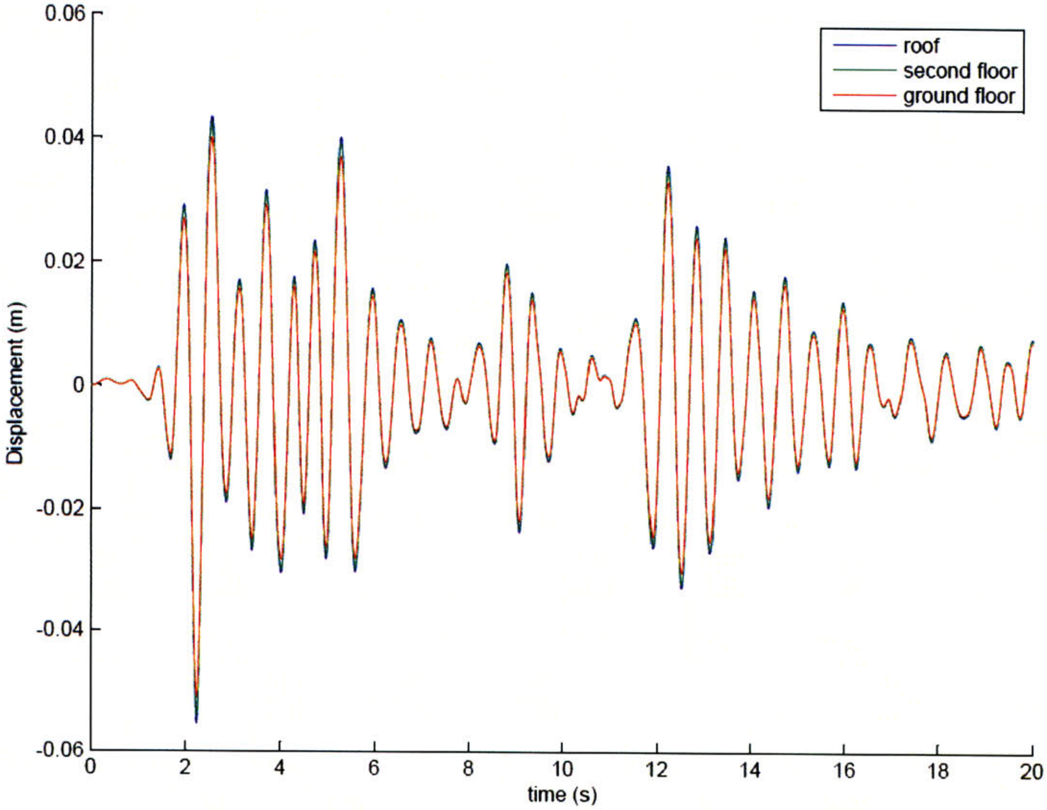


Figure 35: Displacement of the isolated structure, no damping (ElCentro EQ)

The roof acceleration of the primary structure and the base isolation structure is provided in Figure 36. As seen, the acceleration is significantly reduced, going from almost 5g to 1.1g.

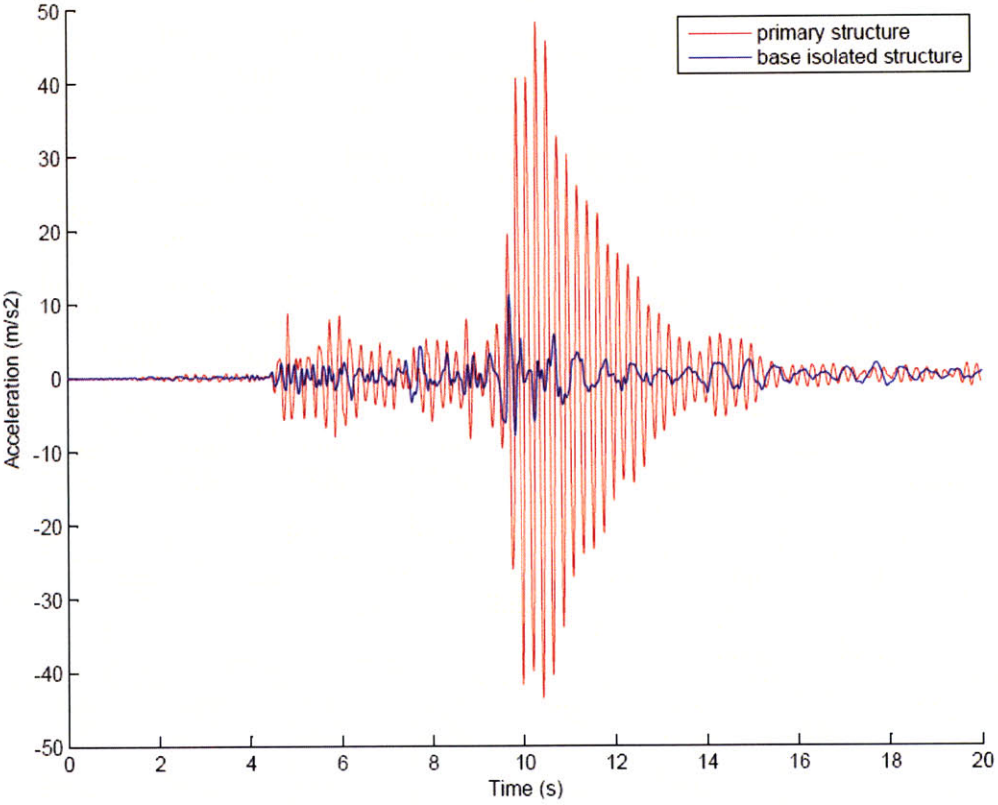


Figure 36: Acceleration of the structure with and without base isolation (Northridge EQ)

6.3 PASSIVE HYBRID ISOLATED STRUCTURE

The performance of the structure presented above can be increased by the addition dampers. For the purpose of this analysis, MR dampers without control are used to provide the additional passive damping. As shown in Figures 37, the maximum displacement response of the damped structure under the Northridge earthquake is reduced from 0.034m to 0.025, a reduction of 26%. For the ElCentro earthquake, the maximum displacement is reduced from 0.055m to 0.038m, a reduction of 31% (see Figure 38). In this case, 10 dampers are used, with a damping coefficient of 180 kNs/m and a stiffness coefficient of 5.5 kN/m, providing a maximum force of around 80 kN.

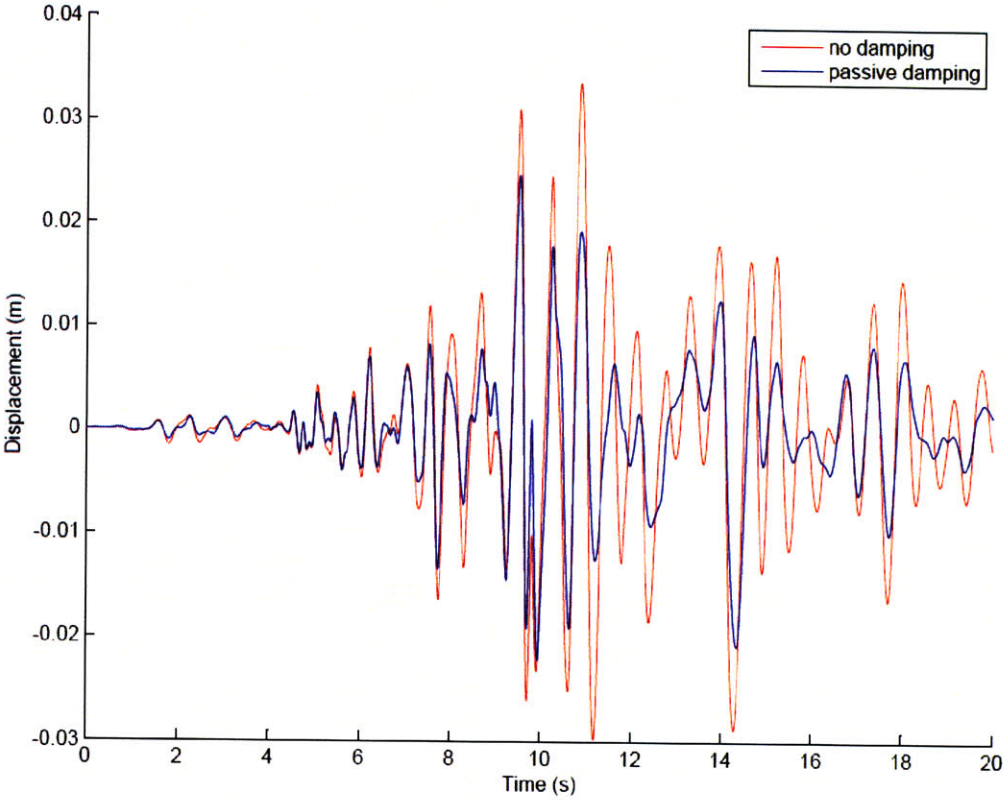


Figure 37: Displacement of the isolated structure with and without damping (Northridge EQ)

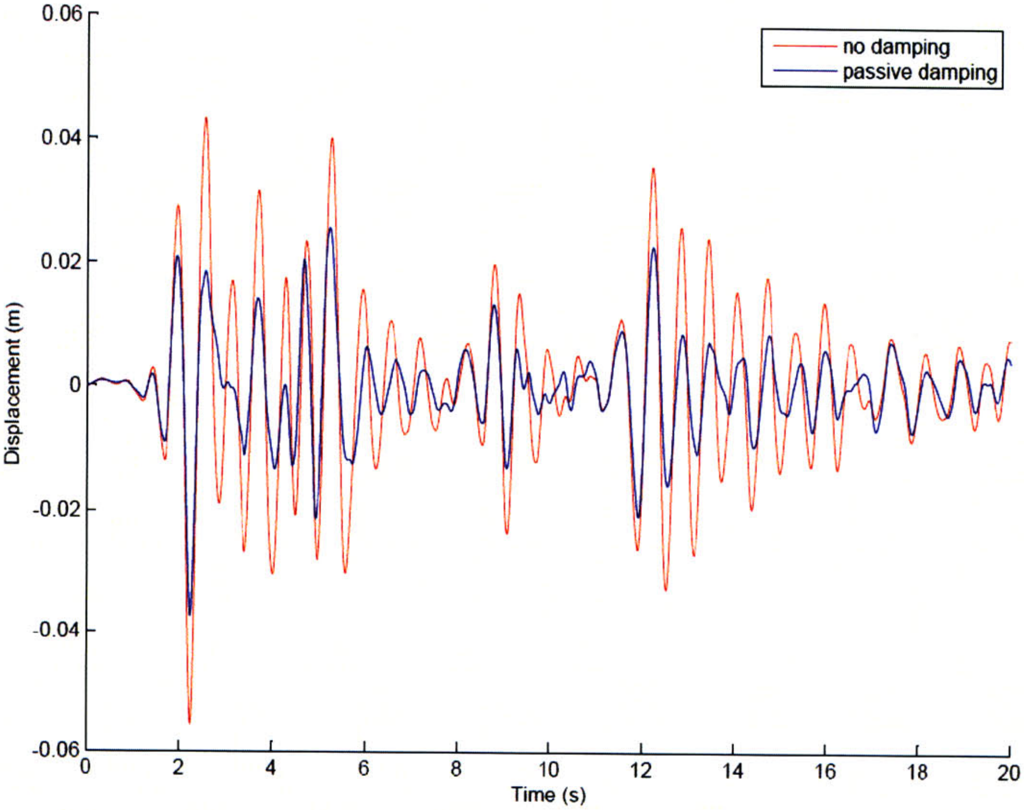


Figure 38: Diaplacement of the isolated structure with and without damping (ElCentro EQ)

Following is a comparison of the roof acceleration for the base isolated structure with and without damping. Under the Northridge earthquake, the acceleration is barely reduced (see Figure 39). However, as shown in Figure 40, the acceleration in the case of the ElCentro earthquake is reduced from 0.95g to 0.77g.

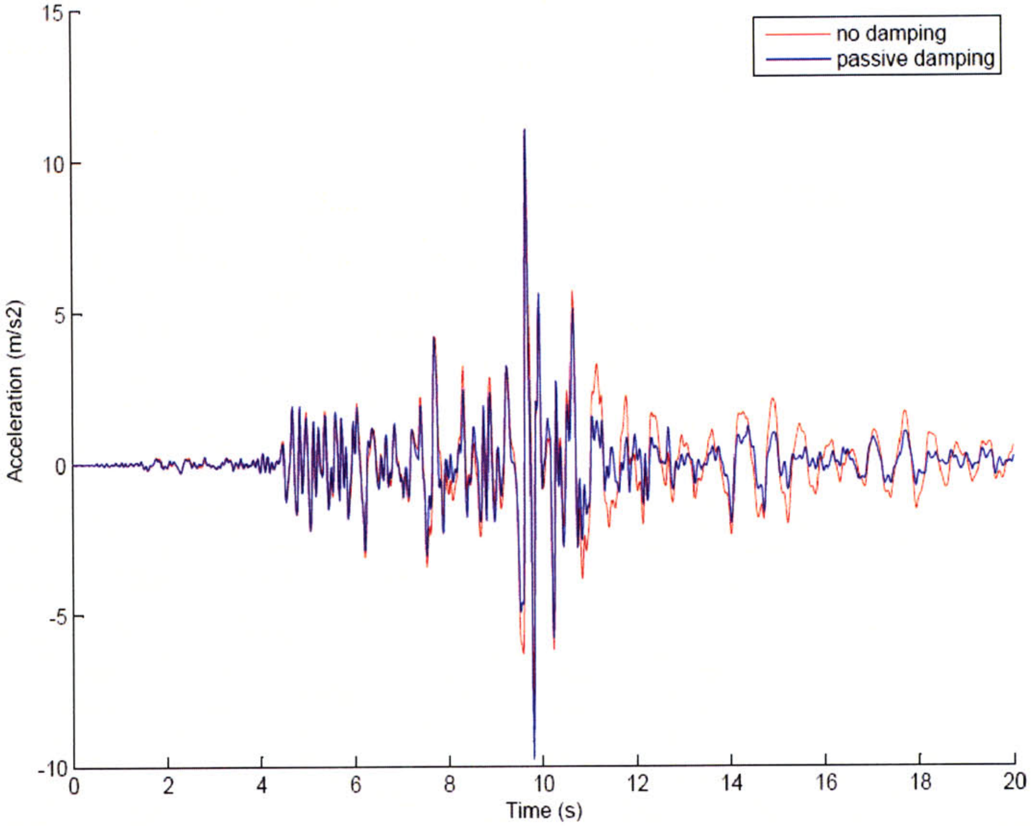


Figure 39: Acceleration of the base isolated structure with and without damping (Northridge EQ)

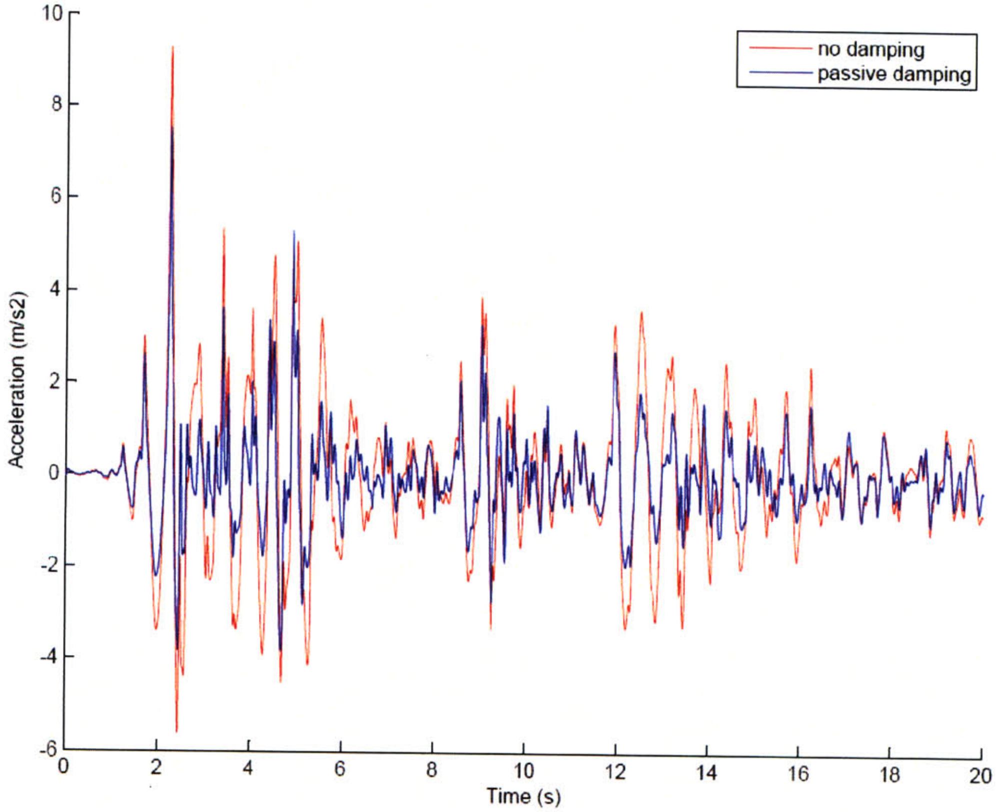


Figure 40: Acceleration of the base isolated structure with and without damping (ElCentro EQ)

6.4 SEMI-ACTIVE HYBRID ISOLATED STRUCTURE

The same system is now analyzed with a semi-active control. The damping forces for the various currents provided are listed below. Again, ten MR dampers are used to reduce the dynamic response of the structure.

0.5A – 80 kN
0.4A – 60 kN
0.3A – 20 kN
0.0A – 10kN

As shown in Figure 41, the maximum displacement under the Northridge earthquake in now reduced from 0.034m to 0.025m to 0.019m, which represents a reduction of 44%. In the case of the ElCentro earthquake, the maximum displacement is reduced from 0.055m to 0.038m to 0.022m, a reduction of 60% (see Figure 42).

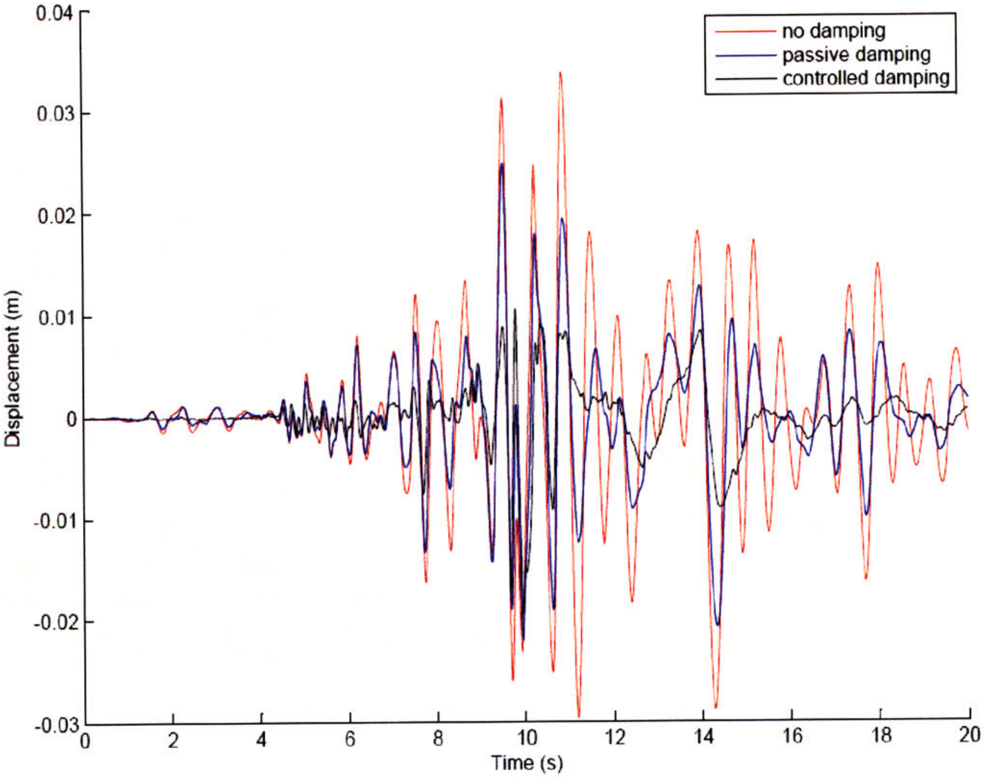


Figure 41: Displacement comparison for no damping, passive damping and semi-active damping (Northridge EQ)

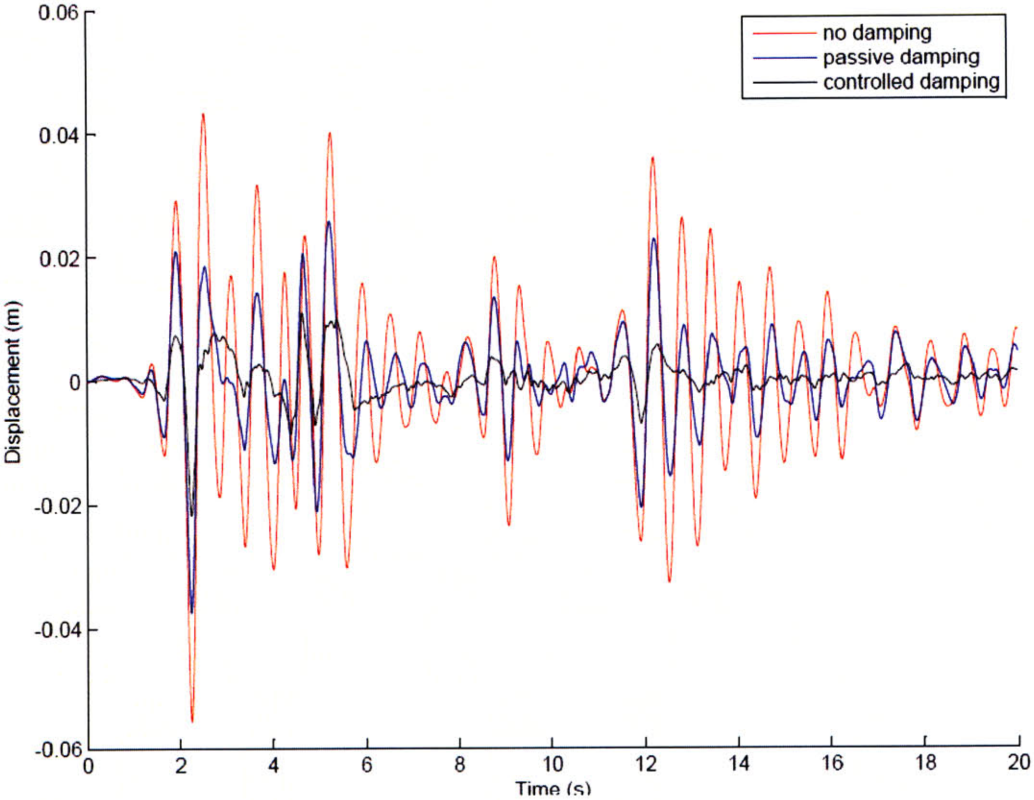


Figure 42: Displacement comparison for no damping, passive damping and semi-active damping (ElCentro EQ)

Unfortunately, providing semi-active control is often done at the expense of interstory displacement. Even if the overall displacement of the superstructure is reduced (see Figure 43), the interstory displacement is increased from 6.8 mm to 9.9 mm (see Figure 44). However, this displacement is still lower than the allowable limit of h/200.

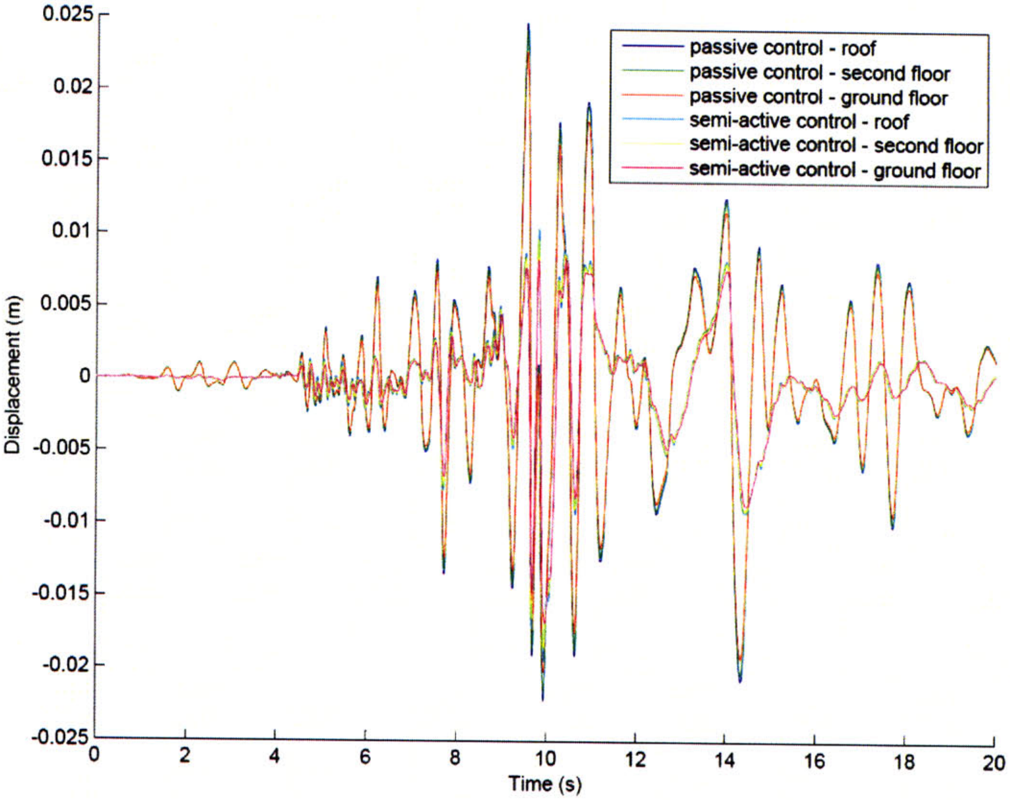


Figure 43: Displacement of each of the three stories with passive and semi-active damping

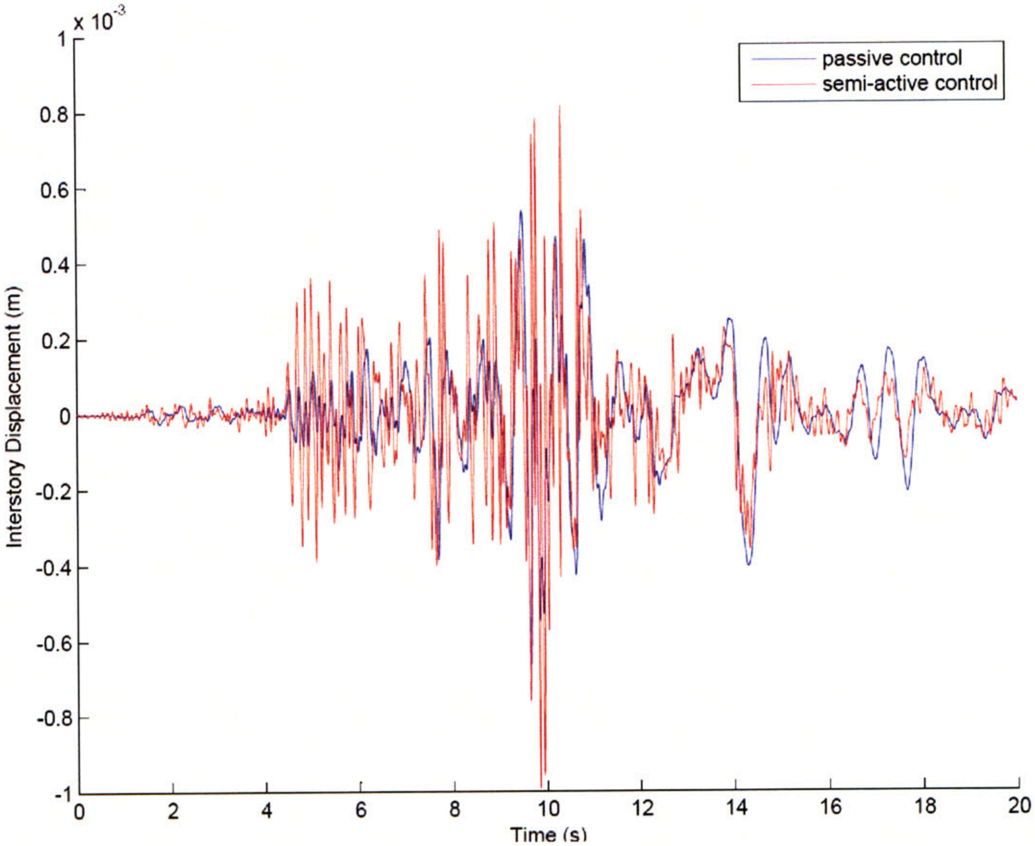


Figure 44: Interstory displacement between the second floor and the roof (Northridge EQ)

Again, roof accelerations are compared. For the Northridge earthquake, the acceleration the structure controlled by the semi-active damping system is increased form 1.2g to 1.33g (see Figure 45). We can see that the damping system is not effective in reducing the acceleration of the superstructure. On the opposite, the damping system effectively reduce the acceleration of the structure under the ElCentro earthquake. The acceleration is reduced from 0.95g to 0.77g to 0.65g (see Figure 46).

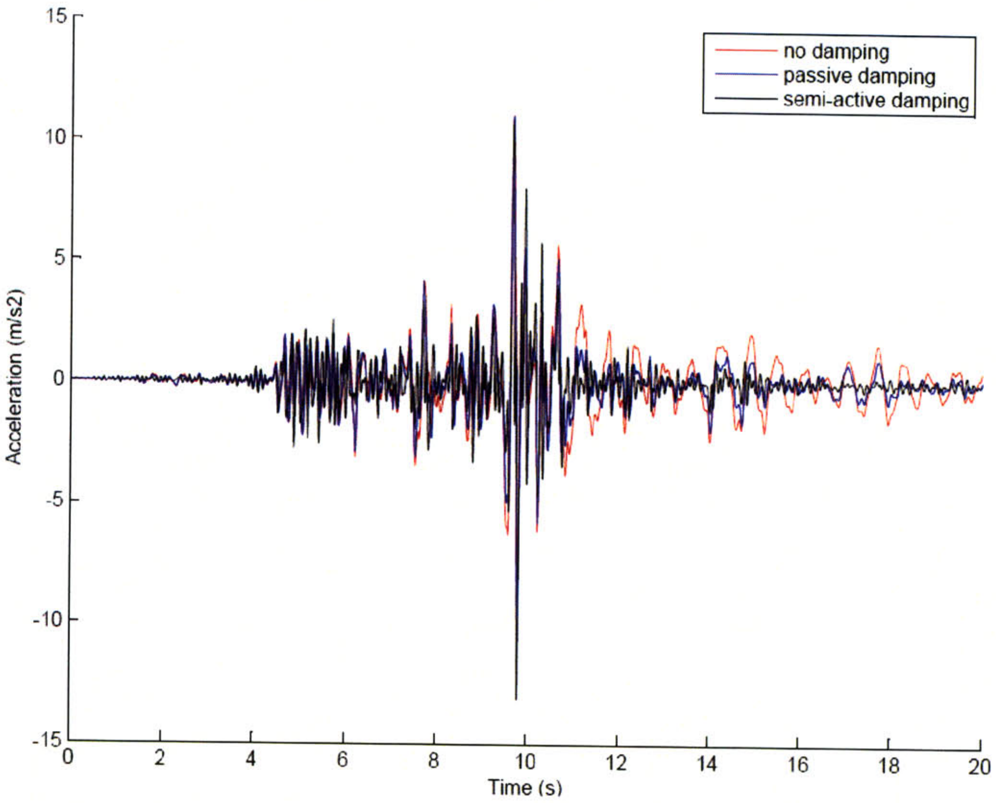


Figure 45: Acceleration of the structure without damping, with passive damping and with semi-active damping (Northridge EQ)

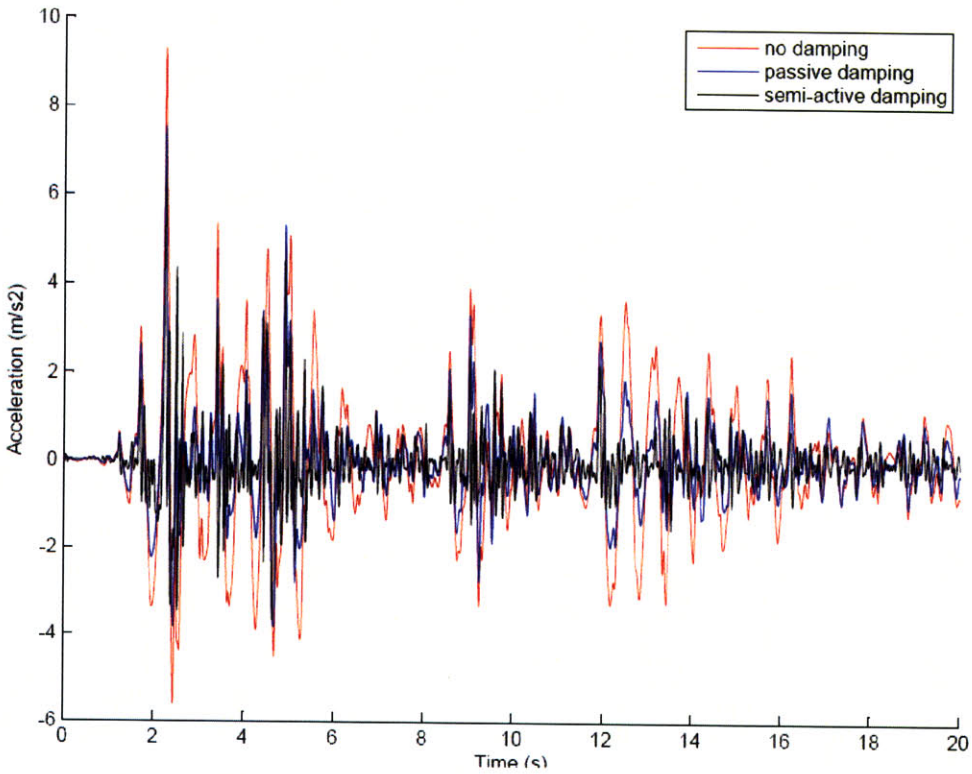


Figure 46: Acceleration of the structure without damping, with passive damping and with semi-active damping (ElCentro EQ)

Finally, the force provided by the passive MR damper and the semi-active MR damper as a function of displacement and velocity is provided in Figures 47, 48, 49 and 50.

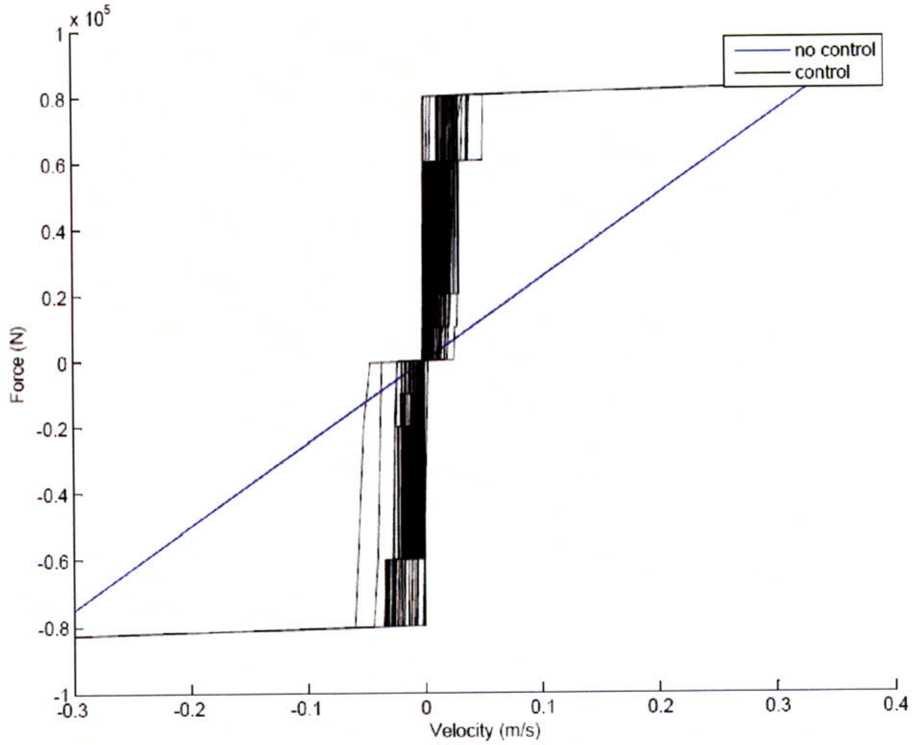


Figure 47: Force vs velocity (Northridge EQ)

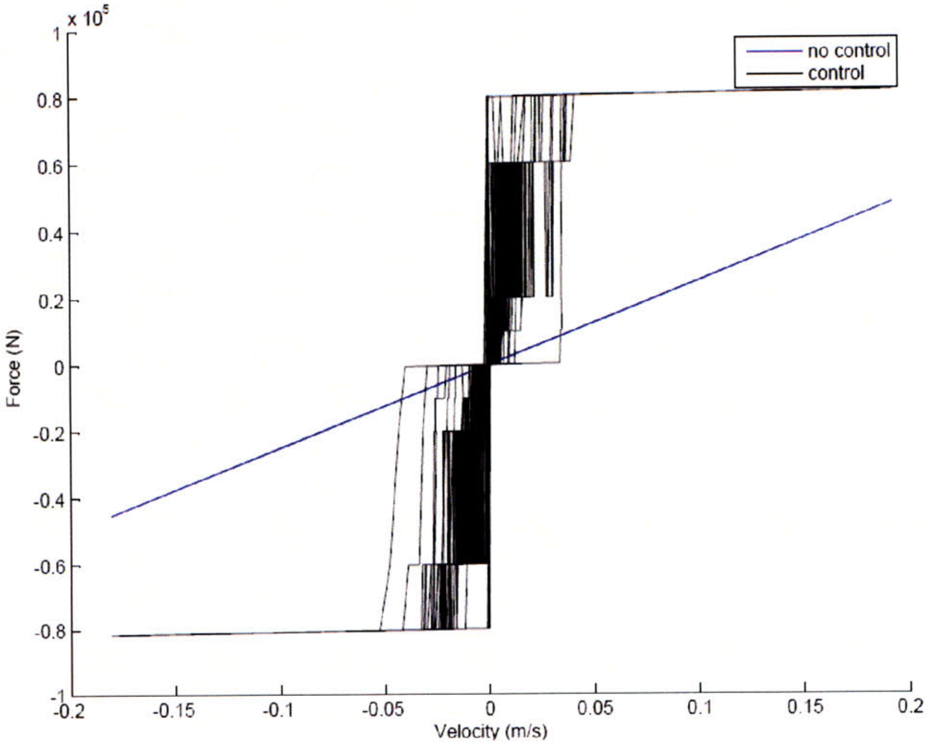


Figure 48: Force vs velocity (ElCentro EQ)

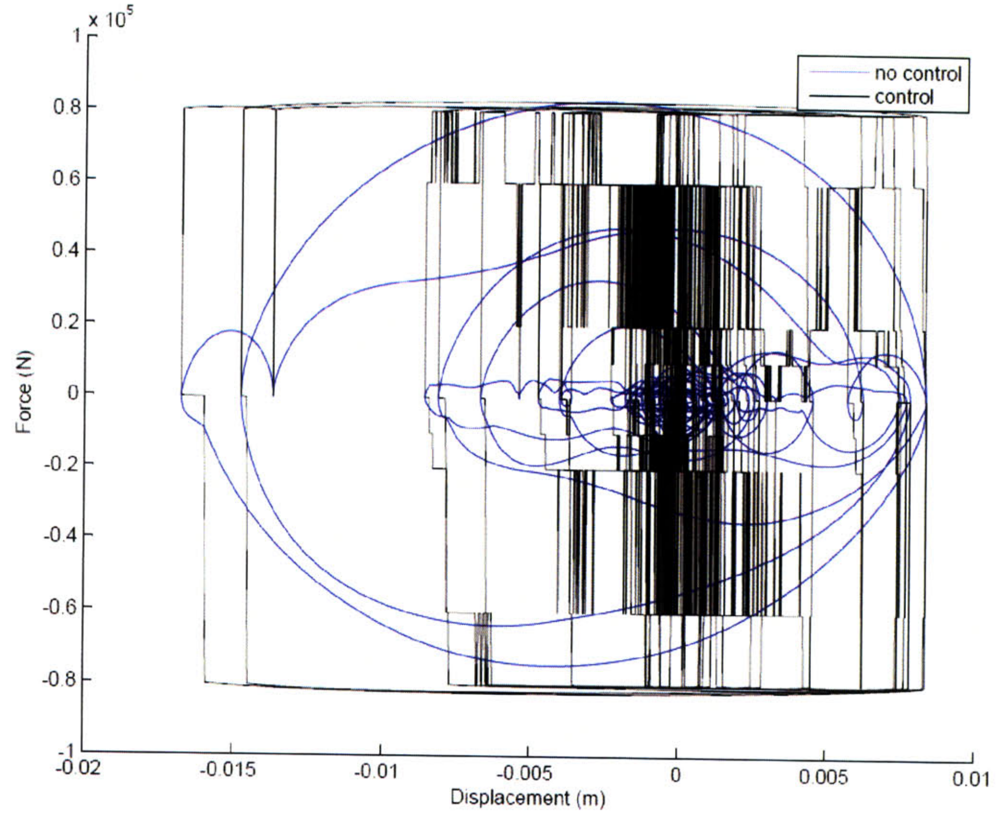


Figure 49: Force vs displacement (Northridge EQ)

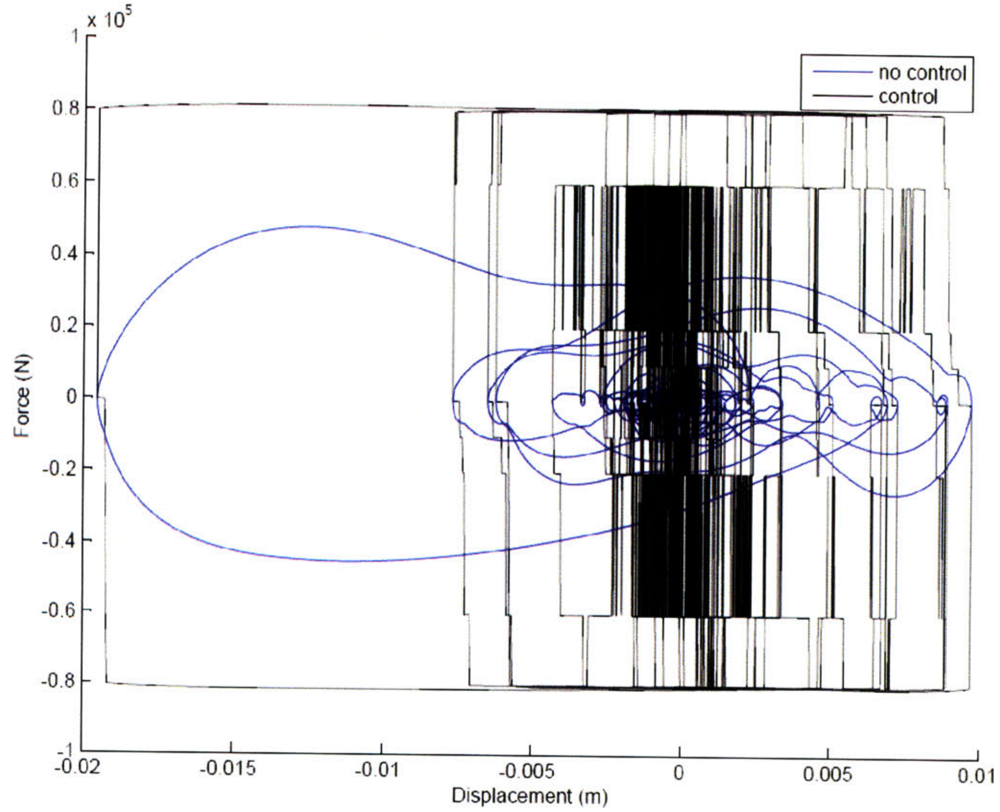


Figure 50: Force vs dis placement (ElCentro EQ)

7.0 CONCLUSION

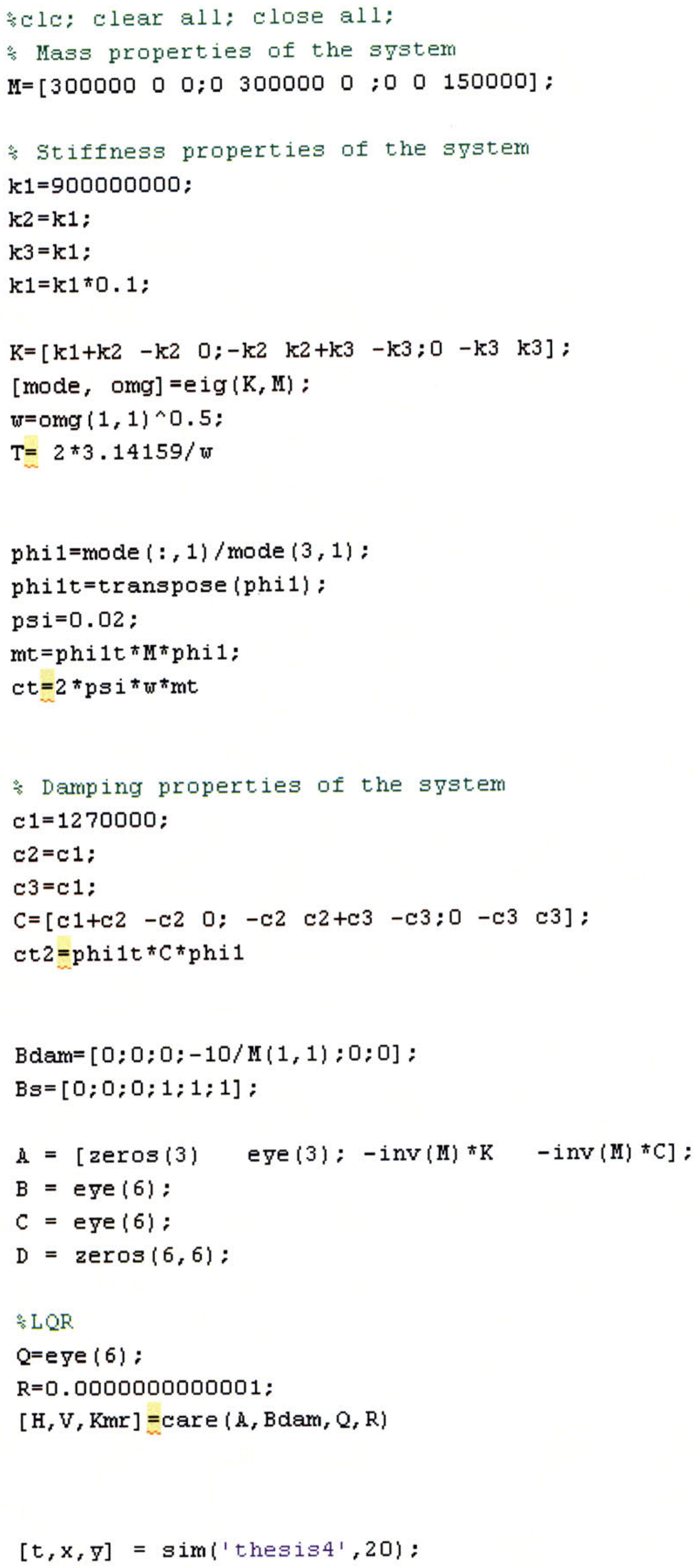
The vulnerability of unreinforced masonry buildings, concrete frame buildings and steel frame buildings commonly found in Canada that are susceptible to earthquake damage has been assessed. The application of conventional and innovative mitigation strategies for their seismic retrofit has been presented and the advantages and disadvantages of passive, active and semi-active control strategies have been discussed. Finally, the performance of hybrid base isolation system has been evaluated through the modelisation of a typical two-story, base isolated concrete frame building.

Over the last decades, considerable progress has been made in seismic design, evolving from no provisions in design codes to stringent structural performance criteria. Resulting from these advancements, the trend in structural engineering has switched from stiffness-increase strategies to force-demand reduction strategies. Following the success of recent experiments and field applications, non-conventional strategies such as base isolation or adding damping are gaining in popularity. Indeed, these technologies are cost-effective and far less intrusive to the building occupants.

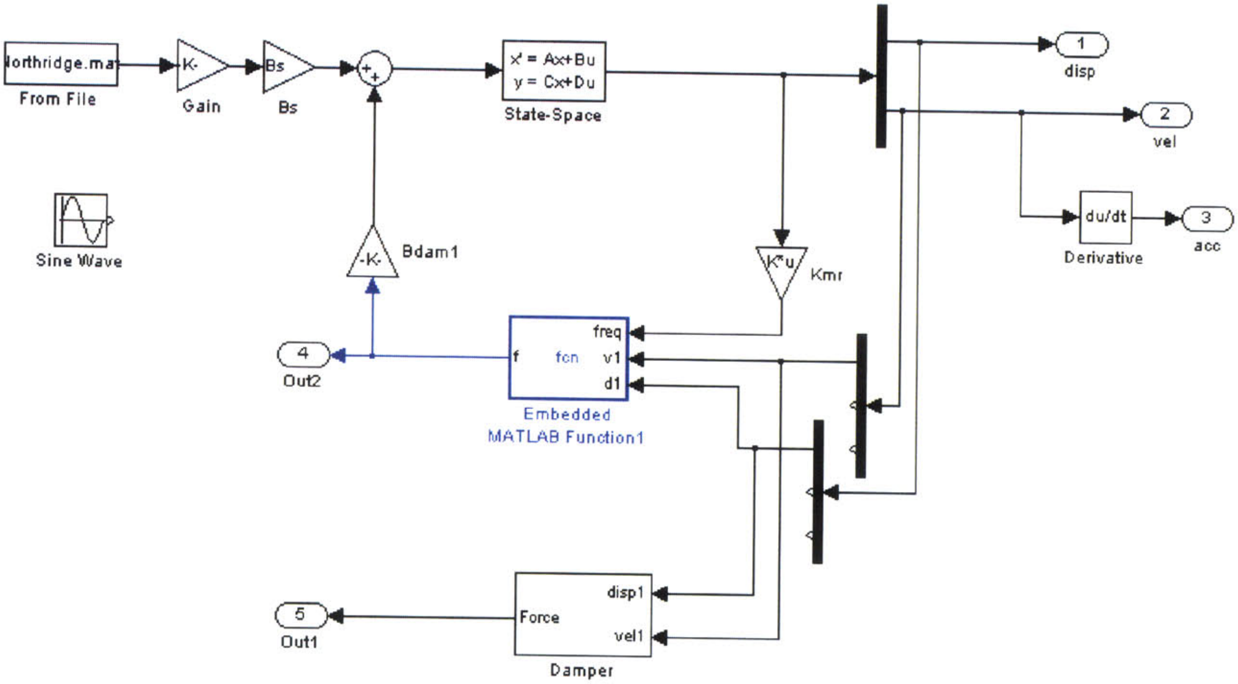
Recently, 'smart' structures have made their apparition in the civil engineering field. The future of structural engineering is leading toward active and semi-active structural control. The objectives are to reduce the cost and maintenance of these strategies, to eliminate the reliance on energy supply sources, to further increase their reliability and robustness, and to gain general acceptance within the engineering community (Forrai *et al.,* 2001). As the number of building in need for seismic retrofit is increasing around the world, it appears evident that non-conventional strategies will keep on moving forward toward future implementations and diverse applications.

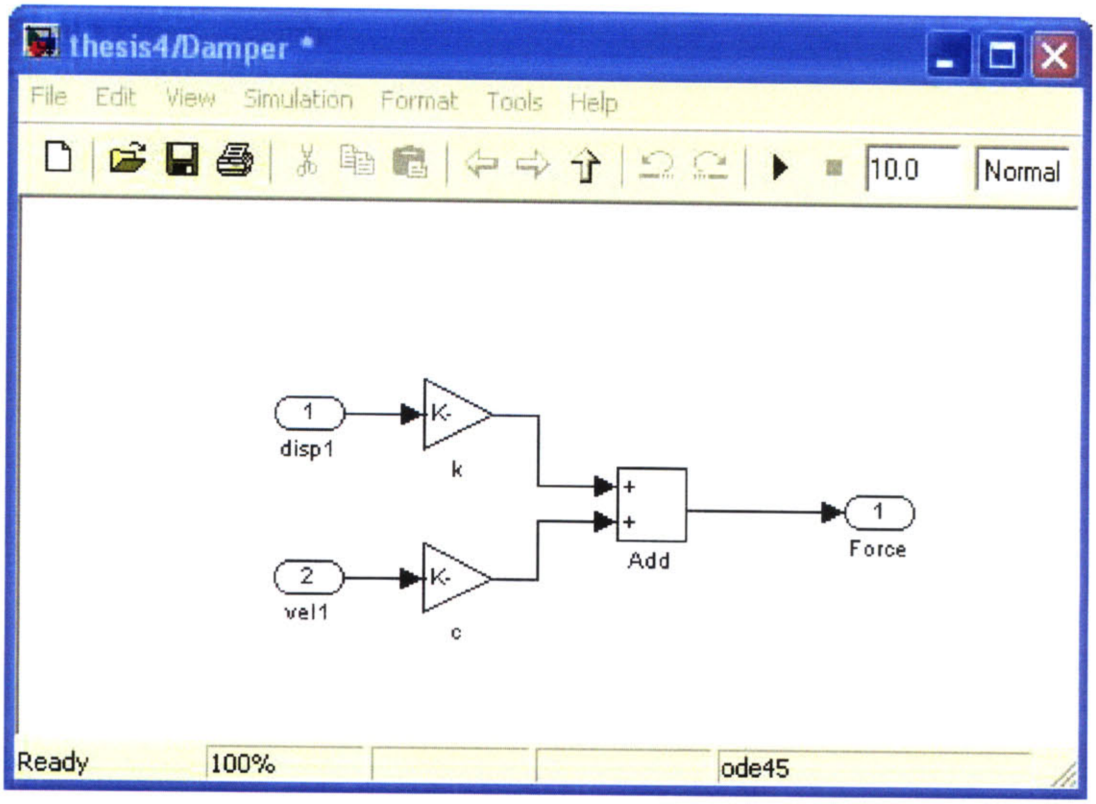
APPENDIX A

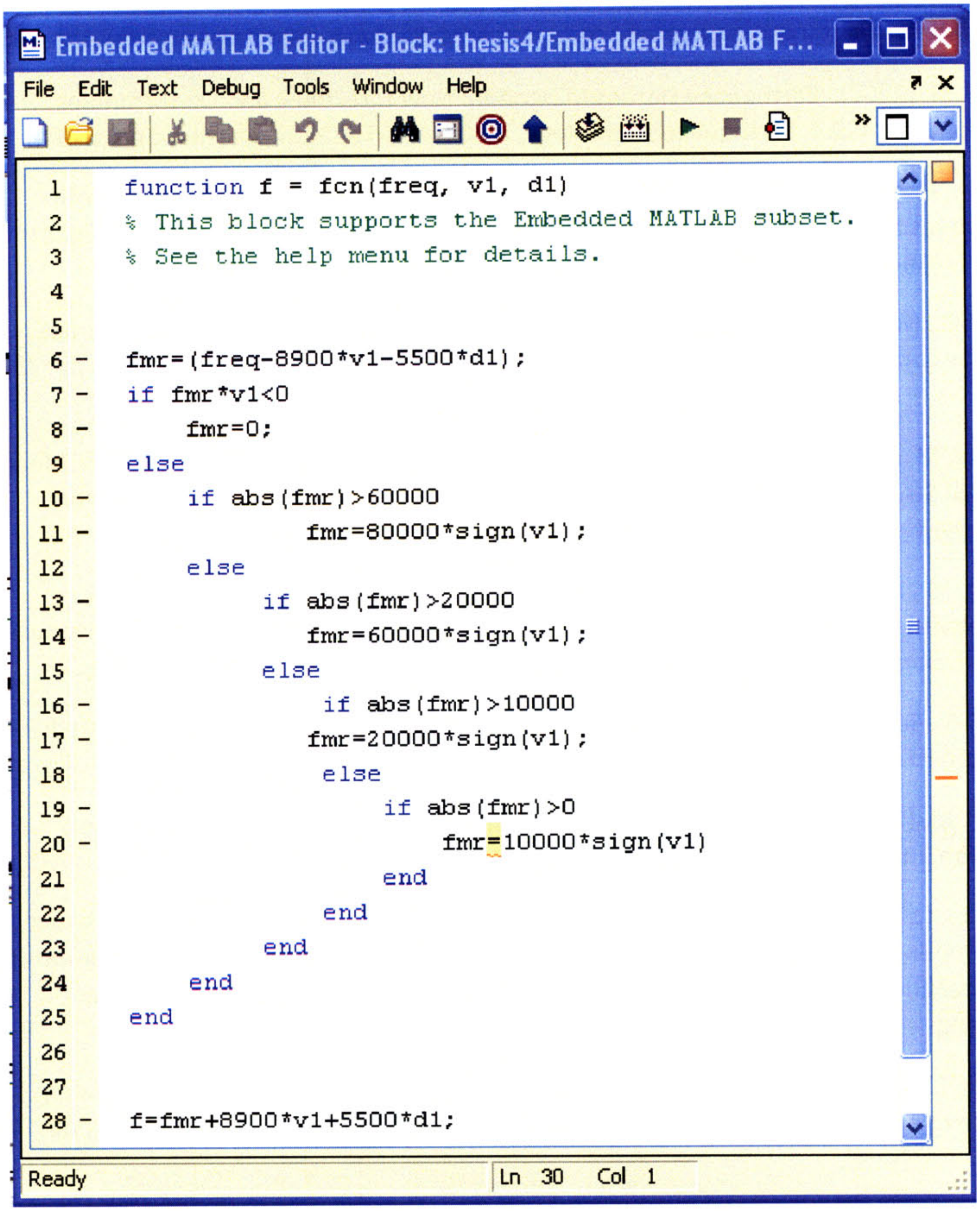
SIMULINK MODEL



```
%clc; clear all; close all;
% Mass properties of the system
M=[300000 0 0;0 300000 0 ;0 0 150000];

% Stiffness properties of the system
k1=900000000;
k2=k1;
k3=k1;
k1=k1*0.1;

K=[k1+k2 -k2 0;-k2 k2+k3 -k3;0 -k3 k3];
[mode, omg]=eig(K,M);
w=omg(1,1)^0.5;
T= 2*3.14159/w


phi1=mode(:,1)/mode(3,1);
phi1t=transpose(phi1);
psi=0.02;
mt=phi1t*M*phi1;
ct=2*psi*w*mt


% Damping properties of the system
c1=1270000;
c2=c1;
c3=c1;
C=[c1+c2 -c2 0; -c2 c2+c3 -c3;0 -c3 c3];
ct2=phi1t*C*phi1


Bdam=[0;0;0;-10/M(1,1);0;0];
Bs=[0;0;0;1;1;1];

A = [zeros(3)    eye(3); -inv(M)*K    -inv(M)*C];
B = eye(6);
C = eye(6);
D = zeros(6,6);

%LQR
Q=eye(6);
R=0.000000000001;
[H,V,Kmr]=care(A,Bdam,Q,R)



[t,x,y] = sim('thesis4',20);
```


Northridge.mat
From File
K-
Gain
Bs
Bs
+
+
x' = Ax+Bu
y = Cx+Du
State-Space
1
disp
2
vel
du/dt
Derivative
3
acc
Sine Wave
-K-
Bdam1
K*u
Kmr
4
Out2
f
fcn
freq
v1
d1
Embedded
MATLAB Function1
5
Out1
Force
disp1
vel1
Damper


thesis4/Damper *
File
Edit
View
Simulation
Format
Tools
Help
10.0
Normal
1
disp1
K-
k
2
vel1
K-
c
+
+
Add
1
Force
Ready
100%
ode45



```
function f = fcn(freq, v1, d1)
% This block supports the Embedded MATLAB subset.
% See the help menu for details.


fmr=(freq-8900*v1-5500*d1);
if fmr*v1<0
    fmr=0;
else
    if abs(fmr)>60000
            fmr=80000*sign(v1);
    else
         if abs(fmr)>20000
            fmr=60000*sign(v1);
         else
             if abs(fmr)>10000
            fmr=20000*sign(v1);
             else
                 if abs(fmr)>0
                     fmr=10000*sign(v1)
                 end
             end
         end
    end
end


f=fmr+8900*v1+5500*d1;
```

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