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POSTDOCTORAL STUDIES**

**Jean-Michel Carrière**

-----  
AUTEUR DE LA THÈSE / AUTHOR OF THESIS

**M.A.Sc. (Civil Engineering)**

-----  
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**Seismic Retrofit of Existing Reinforced Concrete Moment Resisting Frame Structures Using Diagonal  
Prestressing**

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TITRE DE LA THÈSE / TITLE OF THESIS

**Dr. Murat Saatcioglu**

-----  
DIRECTEUR (DIRECTRICE) DE LA THÈSE / THESIS SUPERVISOR

-----  
CO-DIRECTEUR (CO-DIRECTRICE) DE LA THÈSE / THESIS CO-SUPERVISOR

**EXAMINATEURS (EXAMINATRICES) DE LA THÈSE / THESIS EXAMINERS**

**Dr. D. Lau**

-----  
**Dr. N. Naumoski**

-----  
**Dr. D. Palermo**

-----  
**Gary W. Slater**

-----  
Le Doyen de la Faculté des études supérieures et postdoctorales / Dean of the Faculty of Graduate and Postdoctoral Studies

**Seismic Retrofit of Existing Reinforced Concrete Moment  
Resisting Frame Structures using Diagonal  
Prestressing**

By

Jean-Michel Carrière, P.Eng.

A thesis presented to the  
School of Graduate Studies and Research of the University of Ottawa in  
Partial fulfillment of the requirements for the degree of  
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## Abstract

The national building code of Canada (NBCC) has been under constant evolution since its first issue in 1941. Over the years, knowledge related to structural engineering and more specifically seismic response has continued to evolve through research and field investigations. Recently, the 2005 NBCC has been made available with a new perspective on seismic design, including uniform hazard spectra provided for each city for static and dynamic analyses.

Due to a greater understanding of seismic response of structures and the Canadian seismicity, the NBCC has increased the values of elastic base shears over the years. Between the 1970 and 2005 editions of NBCC, the elastic design base shear has increased by a factor as high as 2.6. A structure designed in 1970, if subjected to 2005 NBCC compatible design earthquake record would potentially be labeled as seismically deficient and fail under code specified loads. Multiple technologies currently exist to retrofit seismically deficient structures. The purpose of this research is to explore the possibilities of using diagonal prestressing, in various configurations, to increase structural stiffness and decrease lateral drift demands under seismic loading.

Shallouf (2005) successfully demonstrated experimentally that a 1 bay, 1 storey R/C frame with masonry infill panels, designed based on the ACI 318-1963 building code, hence seismically deficient, could be retrofitted with prestressed cables to reduced lateral drift. In addition, analytical modeling of the 1 storey frame and analysis of a 5 storey structure in a high seismic region (i.e. Vancouver) were used to illustrate the effectiveness of the approach. This analytical model was successful in reducing lateral drift under various prestressing patterns.

The objective of this research is to continue investigating the effectiveness of the retrofit methodology for structures in two different seismic zones, such as Vancouver and Ottawa. Also, it is examined for buildings with different heights. Structures varying between five and fifteen storeys are analyzed. The results indicate the effectiveness of high-strength prestressing cables as lateral bracing elements, while the effect of prestressing the cables changing with the dynamic characteristics of the building and their interaction with the frequency of the exciting force, i.e., earthquake record.

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# Chapter 1

## Introduction

### 1.1 General

High rise concrete buildings have been designed and built for a little over 100 years. Since the first high rise structure was constructed in Cincinnati in 1903, both the quality of materials as well as the design methodology have been significantly improved to the point where we can now realistically model, in 3 dimensions, an entire structure with a computer model to ascertain different types of stress throughout the entire structure. Hand in hand with the improvements in structural mechanics and design, the understanding of earthquakes has also increased during the last three decades, and the knowledge in the area continues to expand.

For as long as buildings have been built, they have been subjected to seismic loads throughout the ages. The understanding of earthquakes and their corresponding effects on building structures has become a large component of modern building design where structures continue to grow in size and height. Where seismic events are more likely to occur, structural design is often governed by the earthquake loads rather than typical gravity loads. Over the years, the building codes and standards have been modified to reflect the most up to date research in the field of seismic engineering. As earthquake loads have become more and more important in structural

design, engineers have looked back on previously built structures to determine if they are safe to sustain a design earthquake that may occur in the future.

As infrastructures continue aging, engineers are consistently reminded that a large inventory of existing buildings were designed and built under older building codes and practices rather than contemporary codes and standards. Advances in construction techniques such as improved shear, confinement and reinforcement splice designs in potential plastic hinge regions lead one to believe that reinforced concrete structures lacking these new criteria would therefore be seismically deficient and would not successfully survive a serious seismic event. This perception has been confirmed by observations of structural response during previous earthquakes. Therefore, seismic retrofit techniques are being developed to effectively upgrade the capacity of existing structures to design earthquake loads and deformations.

Most current retrofit strategies offer effective solutions to safeguard seismically deficient structures against significant structural damage. However, these techniques may result in architecturally unattractive solutions, sometimes causing a significant reduction in usable space with negative economic implications. In addition, the majority of current retrofit techniques may easily disrupt day to day operations, inducing additional economic losses to building owners. The techniques for non-ductile reinforced concrete frame buildings include, but not limited to, the addition of reinforced concrete infill walls, the addition of structural steel braces and the application of fibre-reinforced polymer (FRP) sheets over existing masonry infill walls, all intended to improve the lateral resistance of structures while controlling inelastic deformations. The “Literature Review” section of this Chapter explores a variety of different retrofitting techniques that have been successfully implemented in non-ductile reinforced concrete buildings.

Cheung and Foo (2001) published a three-step approach to the seismic risk mitigation strategy of the Public Works and Government Services Canada (PWGSC), currently being implemented for PWGSC buildings in Canada. They state that the average age of PWGSC buildings is 40 years, indicating that a large number of buildings were built based on pre-1970s building codes. The three-step process consists primarily of screening, evaluation and retrofitting. The screening step determines if a building is seismically deficient based on an assigned seismic priority index

(SPI). Following the screening of a medium or high risk structure, an in depth evaluation may have to be undertaken to determine the level of seismic deficiency. Once the evaluation is complete, a proper retrofit technique may be selected to increase the seismic resistance of elements, while also reducing lateral drift. As the aging infrastructure continues to grow, as well as our understanding into seismic events and their effects on building structures increase, the evaluation and retrofitting of older reinforced concrete structures will be a continuously improving area of study.

As previously noted, the National Building Code of Canada (NBCC) has undergone a constant evolution since its first edition in 1941. Prior to 1941, building regulations were governed by provinces. However, no set standards were truly in place and therefore the building owners remained reluctant to encourage the government to develop a standard building code while they were able to construct whatever they basically wanted. The first publication of the NBCC did contain seismic provisions, which were contained in appendices. These provisions were based on the 1937 United States Uniform Building Code (UBC). The first true inclusion of seismic provisions was in 1953 where the base shear coefficients were based on zones. These coefficients were simply double for Zone 2 and multiplied by 4 for Zone 3 and were also included in 1960 and 1965 editions. This remained as the approach until 1970, at which time a non-dimensional multiplier was introduced. The 1970 NBCC seismic provisions were utilized in the current investigation, as the buildings considered in the current investigation were assumed to be designed in that era. This edition of NBCC was the last edition prior to the introduction of zonal peak accelerations in 1975 and zonal peak velocities in 1985. These resulted from significant advances in the understanding of Canadian seismicity.

Further modifications were introduced to the NBCC in 1990 and 1995, including force reduction factors and calibration factors. The most recent edition of the NBCC was released in 2005 and is regarded as a great overhaul of seismic provisions, where soil conditions have become increasingly more important and dynamic analysis has become the preferred option.

## **1.2 Objective and Scope**

The objective of the current research project is to investigate the effectiveness of diagonal pressing non-ductile reinforced concrete frame buildings as a seismic retrofit strategy. The scope includes;

- Selection of three non-ductile reinforced-concrete frame buildings, with 5, 10 and 15-storey heights.
- Design of the structures using the 1970 NBCC seismic provisions.
- Selection of suitable computer software to conduct dynamic inelastic response history analysis.
- Modeling the buildings for computer analyses.
- Selection of earthquake ground motion records compatible with current Canadian seismicity as reflected by the Uniform Hazard Response Spectra given in NBCC-2005.
- Dynamic inelastic response history analyses of buildings under selected earthquake records for various retrofitting schemes.
- Evaluation and assessment of results.
- Presentation of results.

## **1.3 Literature Review on Seismic Retrofitting**

There are a number of retrofitting techniques that are currently proposed and being used to protect seismically deficient buildings against earthquakes. The following provides an overview of these retrofitting methods and the principles behind them.

Cheung, Foo and Granadino (2000) wrote a paper introducing seismic retrofit strategies currently used by Public Works and Government Services Canada (PWGSC). Accordingly, the current retrofit technologies being used in practice consist of friction dampers, fluid viscous dampers, Carbon Fibre Reinforced Polymer (CFRP) technology and steel jacketing. Friction dampers may be seen as steel X bracing in beam-column R/C frames. Fluid viscous dampers are a newer technology, similar to that of friction dampers. CFRP is a relatively new retrofit method, which

now has its own CSA standard CAN/CSA-S806-02 identifying it as a workable solution to structural retrofit designs. Finally, the column jacketing consists of retrofitting an existing column with a steel jacket to increase the confining capacity of concrete. The paper discusses 5 case studies where the above technologies were used to produce a cost effective solution to retrofit older structures build under less stringent building codes. The authors also discuss the classic retrofit solutions, consisting of adding reinforced concrete shear walls, masonry infill panels or other bracing elements which are intrusive and time consuming, increasing overall costs. It is shown that in each case, the overall cost of the retrofit project is drastically reduced to the new retrofit techniques.

Daudey and Filiatrault (2000) conducted an evaluation of reinforced concrete bridge piers under seismic loading and investigated the effects of retrofitting through steel jackets. This retrofitting technique focuses on the column element and its ductility rather than the frame action studied in the current project. However, the ductility capacity of individual elements is just as important in ensuring a ductile frame action. Five rectangular column specimens were tested. The first column represented as-built conditions without retrofitting. The other four columns were retrofitted with steel jackets of elliptical shape. The main contribution of the steel jackets was the improved ductility capacity in plastic hinge regions at the ends of columns. Steel jacket retrofitting allowed for confinement of the concrete, thus maintaining the capacity to form plastic hinges without drastically increasing strength and stiffness. The ductility ratio increased from 2 for the unretrofitted column to 6 for the steel jacket retrofitted columns. The four retrofitted specimens all indicated equally good results. Failure typically occurred through the slip of longitudinal bars at the base of column between the column and the footing dowels. Therefore, it is clear that this retrofit technique significantly increases the ductility of the column, thus increasing its energy dissipation capacity. This method is dependant on the expected failure mechanism of columns. The intent is to prevent premature shear failure while the formation of flexural plastic hinge is ensured as the weakest link. It may be important to note that, while the steel jacketing aims for increasing deformability, the research conducted in this thesis aims for increasing strength and reducing lateral drift demands. Although the design concepts are different, both constitute valid retrofit strategies.

Chang and Chang (2004) explored the effectiveness of retrofitting reinforced concrete columns using FRP jackets modified with steel plates. Providing steel plates at column plastic hinge zones prior to the application of CFRP jackets increased stiffness dramatically. The steel plates were adhered to the column through epoxy. In conjunction with the CFRP wrap, the confinement of concrete column was significantly enhanced. Multiple columns were tested by the researchers under a lateral displacement control history. Failure of columns, both with and without the retrofit, generally consisted of splice failures at the base of column longitudinal reinforcement. The experimental results showed improvements in strength, ductility and energy dissipation.

Bonelli (2004) reported on research that involved the use of reinforced concrete structural walls as a solution for retrofitting reinforced concrete frames. A non-linear analysis was conducted for an existing 6 storey reinforced concrete frame structure with nominal ductility to determine its ductility and shear demands. Further, reinforced concrete infill walls were installed in exterior and interior bays. Through additional non linear analysis, the ductility demand was shown to be greatly reduced. This analysis illustrated the significance of stiffening an existing reinforced concrete frame by adding structural elements. It was reported that, through various research options, multiple retrofitting techniques could be developed with the intention of discovering more efficient and cost saving techniques.

Hiotakis, Lau and Londono (2004) examined the feasibility of retrofitting existing reinforced concrete shear walls with bonded carbon fiber sheets. The experimental development of the project included control specimens, post-failure repaired specimens, as well as pre-failure retrofitted specimens. The repair of 2 failed shear walls allowed for a recovery of the initial strength of shear walls up to 80% to 90%. The overall lateral load carrying capacity was increased as well as the stiffness, thus reducing overall drift under lateral loads. Further, the 5 retrofitted shear walls all exhibited improvements in their lateral capacities as well as their stiffnesses. The flexural capacity of the walls generally improved, leading to greater pre-cracking stiffness and increased cracking load. It can be concluded that retrofitting existing R/C buildings, more specifically shear walls, using CFRP sheets can be achieved with very positive results.

Li (2005), from the University of Michigan, reported on the development of fiber reinforced bendable concrete for retrofitting existing structures. This concrete was said to be 500 times more resistant to cracking and 40 % lighter than conventional concrete. The increased flexibility of the concrete was attributed to the addition of small fibers in the mix, totaling approximately 2% of total volume. Although this concrete mixture was intended mainly for roads and bridge decks, the possibility of increasing flexural capacity of a structural member, using bendable concrete, has attractive characteristics, offering future research potential.

Shalouf (2005) performed experiments on masonry infilled reinforced concrete frames, retrofitted by diagonal prestressing and FRP sheets. Each technique was applied in the lab on a singly-bay, single-storey reinforced concrete frame under different loading conditions. It was shown that the FRP sheets, applied diagonally on the masonry wall, significantly increased the strength of frame with no noticeable increase to ductility. Most of the additional strength gain was lost at the level of drift ratio attained by the companion un-retrofitted frame. As for retrofitting through diagonal prestressing, it was found that the technique greatly reduced lateral drift while significantly increasing lateral strength under statically applied loads. This current project is largely based on the results obtained by Shalouf on diagonally prestressed reinforced concrete frames. The results obtained from three tests were not sufficient to generalize the conclusions; therefore further analytical research has been concluded, as discussed in this thesis. This project does not explore FRP retrofitting but rather focuses mainly on diagonal prestressed cables as a new method of retrofitting.

Yuksel (2005) developed 6 experimental models of a 2 storey, single bay reinforced concrete frame, some in-filled with masonry and retrofitted with diagonal CFRP strips. The frames were loaded under constant axial load and reverse cyclic lateral loading. It is notable to state that masonry infill panels provide significant bracing without any additional retrofitting. However, similarly to Shalouf (2005), the diagonal CFRP strips may have to be placed to increase strength and stiffness to levels that may be needed during strong earthquakes, even if no enhancement could be attained in ductility. This also leads to the conclusion that this type of retrofitting technique should be based on strength rather than ductility.

Pincheira (1992) produced a PhD thesis entitled “Seismic Strengthening of Reinforced Concrete Frames Using Post-Tensioned Bracing Systems”. The noted research is complementary to this report. Pincheira studied the response of 3 non-ductile reinforced concrete frames under inelastic dynamic and static earthquake loading. The study focused on seismic behaviour of buildings in the United States on both soft and firm soils. Further, 5 separate ground motions were considered, which were representative of severe seismic events in the United States. Using the original version of computer software DRAIN 2D, Pincheira undertook both the static and dynamic analysis in an attempt to reduce the drift in the structures. This method has introduced concerns, which are addressed later in this thesis, specifically with respect to the definition of the cable element and the post tensioning loads. Generally, Pincheira concluded that post tensioned bracing can control lateral drifts effectively.

## Chapter 2

# Computer Program DRAIN-R/C

### 2.1 Background

The computer program DRAIN-R/C was used in this investigation to conduct dynamic inelastic response history analysis of selected buildings. The program is a modified version of the general purpose dynamic analysis software DRAIN-2D developed in 1973 at the University of California, Berkeley. The original program was developed to carry out dynamic inelastic response history analysis of structures under seismic loading, with hysteretic models for steel and concrete frame elements, as well as truss members. The program was later modified progressively, to incorporate enhanced hysteretic models for reinforced concrete elements, as well as other functionalities at the University of Ottawa. The resulting software, labeled as DRAIN-R/C (R/C for reinforced concrete) is intended for analysis of reinforced concrete structures. The latest version of the program was converted to visual FORTRAN with user friendly windows-based pre-processor and graphical user interface. The following provides an overview of the chronological development of DRAIN-R/C.

- 1973 - Original development of DRAIN-2D, incorporating the modified version of Takeda's hysteretic model for flexure, by A. E. Kanaan and G. H. Powell at the University of California, Berkeley, California.

- 1993 – Incorporation of additional plastic hinges and hysteretic models for inelastic shear (Ozcebe and Saatcioglu, 1989) and anchorage slip (Saatcioglu et al. 1992) by J. Alsiwat and M. Saatcioglu at the University of Ottawa, Canada.
- 1998 - Incorporation of an infill wall element with hysteretic characteristics (Klingner and Bertero 1978), by A. Shooshtari and M. Saatcioglu at the University of Ottawa.
- 1998 - Incorporation of Axial force-Flexure Interaction Hysteretic Model (Saatcioglu et al. 1983) by A. Shooshtari and M. Saatcioglu at the University of Ottawa, Canada.
- 1998 – Modification of the “P- $\Delta$  effect” option to incorporate secondary moments in the inelastic range of deformations by A. Shooshtari and M. Saatcioglu at the University of Ottawa, Canada.
- 1998 – Introduction of static inelastic (push-over) analysis under incrementally increasing loads, by A. Shooshtari and M. Saatcioglu at the University of Ottawa, Canada.
- 2006 - Introduction of windows-based graphical user interface for easy data entry, by M. Shooshtari and M. Saatcioglu at the University of Ottawa, Canada.
- 2006 - Introduction of an inelastic prestressing cable element with hysteretic capabilities under cyclic loading, by M. Shooshtari and M. Saatcioglu at the University of Ottawa.
- 2006 - Introduction of dynamic impulsive (blast) load option, by M. Shooshtari and M. Saatcioglu.
- 2006 - Introduction of dynamic load option for harmonic forcing functions, by M. Shooshtari and M. Saatcioglu.
- 2006 - Introduction of element loading options for different configurations of gravity loads, by M. Shooshtari and M. Saatcioglu.

For the purposes of this project, non-linear hysteretic behaviour in flexure is considered, whereas inelastic shear behaviour and anchorage slip behaviour are neglected for simplicity.

## 2.2 Background on the Development of DRAIN Programs

The computer software DRAIN-2D (and the modified version DRAIN-R/C) is a general purpose computer program for dynamic analysis of inelastic planar structures. The program consists of a main program and a large number of subroutines. Some of the subroutines were developed independently and added to the base program, bringing functionality and ability to incorporate different types of elements. The program code was designed to allow the incorporation of new subroutines without modifying the main program. The functionalities, such as hinge modeling, can be added as a module and therefore can be easily turned on or off.

The main solution engine of the program is based on the direct stiffness method, where the nodal displacements are unknowns. Each node may have up to 3 degrees of freedom, i.e., translations in X and Y directions and rotation. To simplify the problem, identical degrees of freedom may be applied to multiple nodes. For instance, in the case of a rigid diaphragm, each node on a given floor may be assumed to have equal horizontal displacement. Dynamic analysis is conducted numerically, by utilizing step-by-step linear integration method. Damping is specified as a combination of stiffness and mass dependant damping. Further details on the background of DRAIN-2D and the solution techniques employed can be found elsewhere (Kanaan and Powell 1973).

The following elements are available in the current version of DRAIN-R/C, which is used in this research program:

### ***Truss Element:***

This element yields in tension and may be chosen to either yield or buckle elastically in compression. Originally, the intent was to use the truss element to model a prestressing cable for the purposes of retrofitting. However, it was discovered through analytical studies that the initial prestressing force applied to the truss element was not appropriate for the intended use. The initial forces may be applied to the element prior to the commencement of dynamic analysis. Once the dynamic analysis starts, the effects of element internal loads are released and are no longer affective. This is not representative

of the behaviour of a prestressed cable, which should in reality maintain its internal stress throughout the analysis while also accounting for any variation that may occur in element stresses during response.

***Reinforced Concrete Beam-Column Element:***

This element type is intended for reinforced concrete beams, columns and walls in structures. It consists of elastic flexural beam element and three inelastic springs at each end with hysteretic models for flexure, shear and anchorage slip assigned to each, respectively. The springs are initially assigned very high stiffness values and they do not rotate as the member experiences elastic deformations. Upon yielding, however the appropriate springs start rotating following inelastic stiffnesses as prescribed by the hysteretic models. Each hysteretic model requires specific set of input that that defines the onset of yielding and post-yield behaviour. The effects of moment axial force interaction on initial yield level, as well as post-yield stiffness are considered.

***Infill Wall Shear Panel Element:***

This element type is used to introduce the effects of infill panels, typically made of masonry. This type of infill panels are commonly used in buildings and contribute significantly to overall strength and stiffness of the structure. The element provides a diagonal strut, with post-elastic characteristics under reversed cyclic loading. This element was not used in the current investigation.

***Cable Element:***

After the truss element was considered to be inadequate for the simulation of prestressing strands used in the current project, a new cable element was developed and added in 2006 by M. Shooshtari for the purposes of this project. This element type is used to model the cables for purposes of retrofitting seismically deficient non-ductile R/C frame buildings. The equation of motion was modified to properly model the effects of initial prestressing in cables. The original equation of motion is given as:

$$M\ddot{u} + C\dot{u} + Ku = -M \times \ddot{u}_g(t) \quad 2.1$$

Where  $M$ : Mass matrix

- $C$ : Damping matrix
- $K$ : Stiffness matrix
- $\ddot{u}$ : Acceleration
- $\dot{u}$ : Velocity
- $u$ : Displacement

The equation of motion in the program was modified to add an additional force at each time step to take into account the presence of prestressing force. At time step number 1, the equation of motion is given as:

$$M\ddot{u}_1 + C\dot{u}_1 + Ku_1 = -M \times \ddot{u}_g(t_1) + P \quad 2.2$$

Where  $P$ : Initial prestressing force

At the next time step, a change in the displacement of the mass occurs as does a change in the prestressing force. At time step number 2, the equation of motion can be given as:

$$M\ddot{u}_2 + C\dot{u}_2 + Ku_2 = -M \times \ddot{u}_g(t_2) + P' \quad 2.3$$

Where  $P'$ : Prestressing force at time step number 2

To solve the equation of motion for an incremental load, it must be represented as a function of  $\Delta t$ . Therefore, by subtracting equation 2.3 from 2.2, the equation of motion becomes:

$$M\Delta\ddot{u}_2 + C\Delta\dot{u}_2 + K\Delta u_2 = -M \times \Delta\ddot{u}_g(t_2) + \Delta P \quad 2.4$$

Where  $\Delta P$ : Increment of prestressing force

This final equation of motion is used in the new cable element installed as a subroutine in DRAIN-R/C. As can be seen above, the full initial prestressing force must be applied at the initial time step. However, for the concurrent time step, only the increment of prestressing force is required to solve the equation of motion. It is also important to note that steel cables yield beyond their yield strength. Once the element reaches the yielding point, the force increment becomes zero as the cable element is assumed to remain at a constant force beyond yielding. The increment changes once the cable force returns

below the yield point (unloading). Similarly, when the cable tension reaches zero, the cable becomes loose and the increment again becomes zero as there is no compressive capacity in the cable. The above comments do not consider the effects of strain hardening, which was also carried over from the previous truss element model. The strain hardening of the cable beyond the yield point would incur some incremental changes in the prestressing force, which would be dependent of the strain hardening ratio and the hysteretic model of the cable.

### **2.3 Assessment of the Current Version of DRAIN-R/C and Further Development**

As noted in the previous section, a working interactive version of DRAIN-R/C was created at the University of Ottawa. The new version allows for not only graphical interface but also for the creation of input file through a user friendly windows based process. This procedure would allow for the saving and easy modification of input items without having to re-enter the entire data. Also, the time history of seismic event could be input as a separate input file and could easily be substituted for the analysis of the effects of a variety of different earthquake records.

Figure 2.1 shows a typical DRAIN RC interface window with a general 5-storey building and diagonal cable elements. The general view illustrates the node numbers as well as the element numbers. Figure 2.2 shows the graphical user interface with a given earthquake record. As can be seen in the figure, the earthquake record is displayed graphically when input through a separate text file.

One of the concerns with the initial input file was the switches, used to consider various subroutines in the analysis. These switches were occasionally difficult to find, and hard to identify once discovered in the input file amongst the hundreds of numerical values in the file. Even if the user would turn off a certain switch, a poor understanding of the input parameters may invite the user to input a multitude of unnecessary values. In addition, the input of unnecessary values may confuse the program as it may take the incorrect variable in one of its subroutines. The new user interface simply provides options to the user to consider the various

subroutines. If an option is neglected, all input options related to that function are eliminated from the interface to provide a simpler and more concise input process.

An extensive help file is being created to aid the user with the input into the graphical user interface. The only other manual available is the original user's manual, which was written for the original DRAIN-2D program over 30 years ago. This user's manual is very theoretical, does not go into specific input parameters and does not include any pertinent information regarding any additional subroutines related to reinforced concrete structures, as noted in previous sections. Although the understanding of the underlying theory used in a computer program is extremely important, the level of understanding required to use the previous versions of DRAIN-2D and DRAIN-R/C is not practical for everyday use by consulting engineers. In this era when structural analysis is dominated by computers, a practical version of DRAIN-R/C may very well find its place within the domain of structural analysis.

Although the advances in the graphical user interface are extraordinary and very useful from the user's perspective, the program output remains per the original program output. With this said, the program output is all given as text files, sorted with a variety of file extensions for different element properties such as member forces and nodal displacements. Although for most cases, it is relatively simple to find the required output, copy it and paste it into spreadsheet software to produce graphical results this is not an ideal situation. The graphical user interface would greatly benefit from displaying the output for the nodes and elements in the model. If the user wishes to have the internal forces for a cable element, this information should be readily available graphically. In addition, time history analysis is a very powerful tool, and one of the greatest options available within DRAIN RC. Graphical representations of the output, as a function of time, for dynamic analysis would be a strong asset for the practical use of the program and its marketability.

At this point in time, DRAIN-R/C focuses more on academic research rather than everyday engineering design within a consulting environment. However, with the constant increases in computing power and more efficient algorithms, the future versions of DRAIN-R/C, including a

graphical user interface for both input and output, as well as insightful help files may be easily applicable to everyday engineering and structural design.

# Chapter 3

## Selection and Design of Buildings

### 3.1 Introduction

The purpose of this research project is to illustrate the effects of retrofitting seismically deficient buildings with a specific retrofit technique, i.e., diagonal prestressing. Therefore, it is important to select older structures with seismic deficiencies. The buildings selected and analyzed were assumed to be designed and built following the 1970 NBCC requirements. Since the seismic design base shear values have progressively increased over the years, the seismic requirements of the NBCC – 1970 edition would be less stringent than those of the current edition. Therefore, a building designed under the 1970 NBCC would be under-designed based on today's codes, thus requiring retrofitting to upgrade the building to current codes.

### 3.2 Seismic Design Force Levels based on NBCC

Section 4.1.7 of 1970 NBCC requires the following minimum lateral seismic force to act in any direction:

$$V = \frac{1}{4} RKCIFW \quad 3.1$$

Where  $R$  = The value given in the Table of Climatic Data, reflecting seismic region.

$K$  = The value given in Table 4.1.7.A. of NBCC 1970, reflecting ductility in the structure.

$$C = \frac{0.05}{\sqrt[3]{T}} \leq 1.0 \quad 3.2$$

$T = 0.1N$  (Period of structure)

$N$  = number of floors

$I = 1.0$  (Importance factor, 1.0 for ordinary residential, office and industrial buildings)

$F = 1.0$  (Foundation factor, 1.0 for firm soil)

$W$  = Dead load of the structure plus 25% of the roof snow load

The NBCC 2005 gives the static base shear as:

$$V = \frac{S(T_a)M_vIW}{R_dR_o} \quad 3.3$$

Where  $S(T_a)$  is the design spectral response acceleration, expressed as a ratio to gravitational acceleration, for the fundamental lateral period of the building  $T_a$

$T_a$  = Fundamental period

$M_v$  = Higher mode effect factor

$I$  = Importance factor

$W$  = Weight of structure (Dead load plus 25% snow load)

$R_d$  = Ductility related force modification factor

$R_o$  = Overstrength related force modification factor

The numerator in the above expression corresponds to the elastic base shear. The fundamental period for reinforced concrete frames can be computed by the empirical equation given in NBCC 2005, shown below:

$$T = 0.075(h_n)^{\frac{3}{4}} \quad 3.4$$

The next section describes the buildings selected for analysis and discusses the calculated base shear for each structure

### 3.3 Selection of Buildings

The buildings chosen for investigation are all reinforced concrete moment resisting frames. A total of 6 buildings were considered for analysis, divided into two different seismic regions. The city of Ottawa was selected as the building site to represent eastern Canadian seismicity, and the city of Vancouver was selected as the building site that represents western Canadian seismicity. Three building heights were considered in each location, consisting of 5-storey, 10-storey and 15-storey buildings. The buildings have complete symmetry to eliminate possible torsional effects. This also aids in the modeling of structures as series of 2 dimensional plane frames as opposed to 3 dimensional building modeling. All the structures share typical general details such as:

- Each bay has a width of 6.0 m each direction;
- The floor to floor height is 4.0 m throughout;
- Each structure consists of 3 bays in the short direction and 5 bays in the long direction;
- The columns are assumed to be fully fixed at the base.

Based on the information given above, Figure 3.1 shows the typical plan view of all the buildings under consideration, complete with all relevant dimensions. Figure 3.2 shows the typical elevations of the 5, 10 and 15 storey buildings. The elevation views are taken in the short direction as this is the critical direction considered for analysis. The structures will sustain seismic loads in the long direction, and thus will require a lateral load resisting system, sufficiently strong and rigid in the long direction. Further analysis may be considered to compare the adequacy of the structures in their strong axis under earthquake loads.

The buildings were initially designed by Esra Dincer (2003), as part of her M.A.Sc. research at the University of Ottawa for the investigation of lateral drift demands under the 2005 NBCC. All structures composed of reinforced concrete frames (beams and columns). The floor system is assumed to be concrete and is included in design dead loads; however it does not contribute to the lateral resistance of the structure, other than providing overhangs to the beams. Table 3.1 provides additional design data. Table 3.2 through Table 3.7 provide the details of member cross

sections and required reinforcement for all frame elements based on the NBCC-2005. The same buildings were adopted in the current investigation. However, their designs were modified to reflect the differences in designs between the 2005 and 1970 NBCC requirements. Because the governing design load was the earthquake loading, and because the design was based on elastic analysis, it was felt that a linear adjustment of element nominal capacities based on the ratio of design base shears as computed on the basis of NBCC 1970 and NBCC 2005 would provide sufficiently accurate results for the purpose of this investigation in simulating older building performances.

Table 3.8 illustrates the calculated base shears based on NBCC 2005 as well as NBCC 1970. For structures in the west, in particular Vancouver, the ratio of base shears between the 2005 and 1970 values range between 1.6 for the low rise 5-storey building and 1.0 for the high rise 15-storey building. The small difference between the base shear values provided by NBCC 2005 and NBCC 1970 is not surprising, as Vancouver was considered a high seismic zone even in 1970. With regards to the eastern structures, i.e. Ottawa, the base shear coefficients vary between 2.6 for the 5-storey low rise building and 1.3 for the 15-storey high rise building. This is to be expected as the seismic hazard for Ottawa has been assessed to be higher than the earlier design values. However, the base shears for buildings in Ottawa remain to be significantly lower than those for Vancouver.

Since the buildings were initially designed for the 2005 NBCC, and the intent is to undertake analyses of buildings designed based on the 1970 NBCC, the member properties were reduced to achieve the desired strength levels. The flexural capacity of members is assumed to be equal to the required capacity for which the building was designed. This means that the buildings were not over-designed and a reduction in capacity to reflect the older code effect based on applied design loads would be acceptable. The nominal flexural capacities for all frame members were determined using the computer software package RCSection, Version 1.3, developed by Pikaso Software Inc. When reducing the flexural capacities, the cross sectional dimensions were assumed to remain the same with the reductions resulting from reductions in reinforcement. This is consistent with the evolution of reinforced concrete design. In 1970's the design would be based on "Working Stress Method," which would require larger concrete sections. Since most

column axial loads are produced by gravity loads, the axial capacity of the columns is assumed to remain constant. This is important for specifying the axial load – bending moment interaction during dynamic analysis, as required by the hysteretic model. The flexural capacity was calculated using the following assumptions:

- Bi-linear steel stress-strain relationship
- Hognestad's (1951) concrete stress-strain relationship
- Concrete cover, clear to reinforcement is 25mm

It is important to note that the member flexural capacities were reduced in conjunction with seismic loads only, regardless of the required capacity for gravity loads. Therefore, the member capacities may be lower than realistically required for gravity loads, in either era. This assumption was utilized to simplify the building design and create an artificial building with sub standard seismic member capacities.

In addition to the building selection criteria discussed above, each individual building has its own dynamic properties and characteristics. These properties are critical when considering dynamic analysis of structures. To help correlate the results obtained later on in this report, the period of each building is calculated and provided in Table 3.9 and Table 3.10. The period is calculated using a new algorithm included in the most recent version of Drain RC with the graphical user interface.

Table 3.9 illustrates the period of vibration of the 3 separate buildings in Ottawa for the first 3 mode shapes. Only the first 3 mode shapes are considered as they are found to be the most contributing to the overall shape of the structures. As can be seen in the table, as the building height increases, the period also increases.

Table 3.10 illustrates the period of vibration of the 3 separate buildings in Vancouver. Similar to the buildings in Ottawa, the first 3 mode shapes were deemed to be to most critical contributing to the overall shape of the building vibration. It can be seen from the comparison of the periods for Ottawa and Vancouver that the periods are identical between the two cities. The period of the structure is calculated based on the stiffness of the members. Even though the flexural

capacities and reinforcement vary between structures, the cross sectional areas are maintained for practical purposes. Therefore, as stiffness is a function of the cross sectional area, it is clear that the periods will not be changed between the structures in Ottawa and Vancouver. Further, Table 3.10 also includes the periods for the various retrofitted shapes, which are examined and discussed in the following chapter.

# **Chapter 4**

## **Analyses of Buildings and Recommended Retrofit Methodology**

### **4.1 Introduction**

The structures selected and discussed in Chapter 3 were analyzed using computer software DRAIN-R/C presented in Chapter 2, with and without retrofitting. The details of the retrofit methodology employed are presented in this Chapter. The analysis approach and the effects of design parameters considered are also discussed in this Chapter. The analyses were carried out using artificial earthquake records that were generated to reflect Canadian seismicity. These records were selected to be compatible with the Uniform Hazard Spectra specified in the NBCC-2005, and hence reflect the state-of-the art assessment of expected earthquake hazard, with probability of occurrence of 2% in 50 years.

### **4.2 Earthquake Ground Motion Records**

Seismic analysis of structures requires the selection of appropriate ground motion records that are representative of expected seismic activity of the region. Canadian seismicity has recently been re-assessed by the Geological Survey Canada and seismic hazard levels have been updated

and incorporated in the NBCC-2005 in the form of Uniform Hazard Spectra (UHS). Synthetic earthquake records were developed by Atkinson and Beresnev (1998) to be compatible with the UHS. These records can be used as input ground motions for DRAIN-R/C and response time history under these records can be computed.

As discussed later in this chapter, a number of different time histories are used for each geographical area under consideration. The purpose of using multiple earthquake records is to properly represent a variety of different magnitude-distance relationships. It was noted in Atkinson's research that nearby earthquakes did not produce enough long period energy to match the long period band, and distant earthquakes would lose too much high frequency energy to match the short period band. By using time history records that represent both conditions, the worst case scenario for a given structure can be identified and designed satisfactorily based on the new Canadian seismicity.

#### **4.2.1 Artificial Earthquake Records for Ottawa**

For the 3 buildings in Ottawa, a total of 8 earthquake records were selected and used. These records consist of 4 short-duration records, which are under 10 seconds in duration and 4 long duration records, which are between 20 and 25 seconds. Using both short and long duration earthquake records, covering different magnitude and source distance characteristics provides a reasonable distribution of ground motions to induce a multitude of different loading conditions in the structures. As can be seen in the earthquake records, the short duration events have a maximum acceleration of  $410 \text{ cm/s}^2$ . For the long duration earthquakes, the maximum acceleration within the ground motion is  $269 \text{ cm/s}^2$ . As the duration increases, the peak ground acceleration typically decreases. Although the short event may produce a greater inertia force due to the higher acceleration, a longer event may incur a more controlled displacement, with the potential for the building to move in phase with the ground motion. This in turn would also produce high internal forces due to the earthquake motion. Figure 4.1 to Figure 4.8 depict the 8 earthquake records selected.

In order to correlate the ground motions provided above with buildings' dynamic behaviour, the spectral accelerations are critical when studying the effects of seismic events. The natural frequency or period of the given structure is directly related to the amount of force it attracts from the seismic event. Figure 4.9 illustrates the spectral acceleration of the 4 long events for Ottawa. Concurrently, Figure 4.10 shows the spectral acceleration of the 4 short events in Ottawa. Each of the two above figures also shows the average spectral acceleration for all 4 records. Figure 4.11 depicts the average spectral accelerations of the long and short events, as well as the uniform hazard spectra given by the NBCC 2005.

#### **4.2.2 Artificial Earthquake Records for Vancouver**

For the 3 buildings in Vancouver, a total of 12 earthquake records with different magnitude-distance characteristics were selected and used. The 12 earthquake records can be divided into 3 categories, the first being the short duration records, each of them being under 10 seconds. The second category would be the long duration earthquake records, between 15 and 20 seconds. The third series of records are the Cascadia earthquake records, which were created based on the actual earthquake that took place in 1700. The Cascadia earthquake created a 1000 km fault line that spanned from Vancouver Island down to northern California. These earthquake records last between 105 and 145 seconds, considerably longer than the short and long records noted above. The short duration events have a maximum ground acceleration of  $567 \text{ cm/s}^2$  compared to the long duration and Cascadia events, which produce a maximum acceleration of  $254 \text{ cm/s}^2$  and  $117 \text{ cm/s}^2$  respectively. As discussed when reviewing the seismic events for the East, the shorter events tend to have much higher ground accelerations, thus inducing high lateral inertia forces and lateral loads. However, the behaviour is not easily amplified due to the random ground motion. The longer earthquakes, especially those for the Cascadia events are more likely to induce a more controlled motion in the structure, allowing the building to vibrate in each direction with its period and reach higher lateral drifts. Figure 4.12 to Figure 4.23 depict the ground motion records of individual seismic events for Vancouver.

Similar to the spectral accelerations presented for Ottawa, the dynamic behaviour of buildings depends on the dynamic characteristics in Vancouver as well. Figure 4.24 illustrates the spectral

acceleration of the 4 Cascadia events for Vancouver. Concurrently, Figure 4.25 shows the spectral acceleration of the 4 long events in Vancouver. Thirdly, Figure 4.26 consists of the spectral acceleration of the 4 short events in Vancouver. Each of the three above figures also shows the average spectral acceleration for all 4 respective records. Figure 4.27 depicts the average spectral accelerations of the Cascadia, long and short events, as well as the uniform hazard spectra given by the NBCC 2005. The behaviour of the chosen buildings under the seismic events will be discussed later in the project. The effects of retrofitting the structures with diagonal cables, with and without prestress, on the period will be examined as well as its correlation with the spectral accelerations noted above.

### **4.3 Seismic Performance Criterion**

The primary focus of current research is to assess the effectiveness of diagonal prestressing as a seismic retrofit methodology for reducing inter-storey lateral drift. Therefore, the drift ratio is used as a performance criterion. The drift ratio is defined as the maximum lateral displacement divided by height. For inter-storey drift ratio, this definition translates into the ratio of maximum storey displacement to storey height. The NBCC 2005 lateral drift ratio limit is 1.0% for post disaster buildings, 2.0% for schools and 2.5% for all other buildings. Reinforced concrete frame buildings typically yield at approximately 1% lateral drift if they do not suffer from premature shear failures. Building columns in older structures may have insufficient longitudinal reinforcement splice lengths. They often suffer from the slippage of spliced reinforcement starting at about 1% lateral drift as well. However, typical flexure-dominant reinforced concrete frames can experience up to 2% lateral drift without much seismic detailing, before significant strength degradation occurs. Design and detailing requirements specified in modern building codes and standards (ACI318-2005 and CSA A23.3-2004) are intended to result in lateral drift ratios of in excess of 4%. It is important to note that the 1970 NBCC did not specifically limit the lateral drift ratio, but did state that “lateral deflection or drift of a storey relative to its adjacent storeys shall be considered in accordance with accepted practice.” With that said, the buildings would be typically designed to the lateral base shear without a specific drift limit specified.

There were a number of problems with the 1970 NBCC, with respect to reinforced concrete design, which have since been remedied. Important to note would be the advancements with regard to lap splices, specifically at beam – column interfaces, as well as shear and concrete confinement reinforcement requirements. These issues are critical when considering lateral seismic response and resistance of moment resisting frames and are often the primary purpose for retrofitting existing structures.

As indicated earlier, previous research has shown that columns that are flexure dominant, without adequate lap splices i.e. designed prior to early 1970s, would experience reinforcement slip failure within splice regions at approximately 1% lateral drift. This failure mechanism could occur prior to the yielding of flexural reinforcement and would therefore be considered as a premature failure of a column expected to yield and develop its full nominal strength. Similarly, requirements for shear reinforcement in columns have greatly improved over the years, and research has also illustrated that shear failure, in intermediate height - shear dominant columns, would occur at approximately 1% lateral drift. Therefore, buildings designed based on older codes of practice, exhibiting one or more of these deficiencies need to be braced against excessive lateral drift, and preferably to no more than 1%.

A secondary reason for drift control in buildings would be to preserve the architectural finishes or cladding of the building, as well as to protect other brittle non-structural elements against failure. A retrofitted building with drift control (drift ratios of 1% or less) is not likely to experience significant damage to non-structural items. For the above reasons, the goal of current research is to minimize the inter-storey lateral drift of seismically deficient buildings to 1%. This level of lateral drift is considered to lead to acceptable seismic performance.

#### **4.4 Structural Modeling for Dynamic Analysis**

Structures should be modeled properly, prior to analysis, such that the properties of the model would be the same as those of the prototype (actual building). The computer program DRAIN-R/C used for the purpose of dynamic analysis is limited to two dimensional analyses. Therefore, a three dimensional structure must be modeled in two dimensions. This is often done by linking

individual frames in the same plane with rigid links, simulating rigid floors that impose the same lateral displacement in each frame. Furthermore, identical frames can be lumped together to avoid modeling each individual frame. This would be based on the principle that the stiffness of multiple moment resisting frames is additive. For instance, say there are  $n$  number of identical frames included in a 3 dimensional structure, these  $n$  frames may be lumped together for simplicity. To do so, both the axial and flexural stiffness must be increased by a factor of  $n$ . To accomplish this, the values of  $A$  (cross sectional area) and  $I$  (moment of inertia) of each section must be multiplied by  $n$  for analysis in DRAIN-R/C. The cross sectional area is related to the axial stiffness and the moment of inertia is used in the calculation of the flexural stiffness. In the model, the length of each member ( $L$ ) and Young's modulus must remain constant. The key in lumped modeling is to maintain a constant natural frequency. The natural frequency of the structure is given as:

$$\omega = \sqrt{\frac{K(n)}{M(n)}} \quad 4.1$$

Where:  $K$  = stiffness  
 $M$  = mass of structure

Following the above equation, the stiffness must be multiplied by  $n$ , and if we wish to maintain a constant natural frequency, the mass must also be multiplied by  $n$ .

Further, even non identical frames could be modeled together, provided they share similar properties. This would of course introduce some degree of error. This error is avoided due to the symmetry of idealized structures, which are used in this project. For a case where each frame is significantly different in geometry and/or material properties, each frame may be analyzed separately. The number of individual frames required in this project varies due to the structural design. For instance, corner columns will only take 25% of the load of an interior column due to the differences in tributary areas. Similarly, an edge column will only take 50% of the load of an interior column. All interior frames should theoretically be the same for the purposes of design as should the exterior frames. In addition, if shear walls are considered in the design, these will again be applied at the exterior frames and thus be similar for all exterior frames. Therefore, for the purposes of this project, 2 typical frames would be required for each structure. The first

frame would consist of all interior frames lumped together, and the second would consist of the two exterior frames lumped together.

For any number of frames to act concurrently, as they would in a real structure, the lateral forces must be translated within the structure through diaphragm action. Floor systems are the most common application of rigid diaphragm action. In this case, the reinforced concrete floor acts as a rigid axial member thus translating the lateral forces from bay to bay without transferring any moment. Therefore, when using DRAIN RC, the lumped interior frames and the lumped exterior frames will be connected through rigid link members, with a cross sectional area and moment of inertia approaching infinity, a clear span approaching zero. These properties produce an infinitely stiff member, thus accomplishing its goal of a rigid diaphragm. It is important to note that although the diaphragm action is an important part of the 2 dimensional modeling, the nodes at any given floor are not assumed to have identical displacements. This option is given by DRAIN-R/C to simplify the direct stiffness matrices. This option is disregarded due to the addition of cable elements in the exterior bays. Since these cables are obviously allowed to elongate and yield, the reinforced concrete frames must be allowed to sustain axial elongation in relation to its stiffness so that the cable may properly elongate within the frame. Figure 4.28 illustrates an example 10 storey building with the nodes, elements and rigid links shown. The figure clearly differentiates between the interior and exterior frames as they will be different in both sectional properties and retrofitting schemes.

#### **4.5 Analysis of As-Built Structures (Without Retrofitting)**

The six reinforced concrete frame buildings, three in Ottawa and three in Vancouver, with structural properties corresponding to those required by NBCC – 1970, were analyzed using DRAIN-R/C under the selected earthquake records. The inter-storey drift ratios along the height of each structure are shown in Figure 4.29 to Figure 4.31 for each of the 5, 10 and 15 storey buildings in Ottawa. The lateral drift is considered in both the positive and negative directions. These directions correspond to the sign convention given by DRAIN-R/C.

The maximum inter-storey drift for the 5-storey building is 0.48%, as illustrated on Figure 4.29, and is obtained under the Long Earthquake Record #2. This is clearly not a seismically deficient building as the lateral drift is below 1%. The maximum inter-storey drift in the 10-storey building is 0.43%, as seen in Figure 4.30. The critical earthquake record for this building was the Long Earthquake Record #1. The third and final structure under review is the 15 storey building. The maximum inter-storey lateral drift for this building is 0.37%, as shown in Figure 4.31. The results indicate that reinforced concrete buildings, covering the range of structural properties considered in these buildings, with heights ranging between 5 and 15 storeys; do not require seismic retrofitting in Ottawa. These three cases indicate that the applied seismic loads are not critical for buildings in Ottawa, built on the basis of 1970 NBCC, even though the design base shears have been increased by as much as a factor of 2.6 since then.

Figure 4.32 to Figure 4.34 show inter-storey drift results for 5, 10 and 15 storey buildings in Vancouver, under all the artificial earthquake records considered for this location. The first building under review is a typical low rise 5 storey building in Vancouver. The maximum lateral inter-storey drift for the 5 storey building is 2.41%, as shown in Figure 4.32. This drift ratio was produced by the Short Earthquake Record #2 and is considerably deficient if we assume that the columns would have failed beyond a drift of 1% lateral drift, most likely due to the slippage of spliced reinforcement. Therefore, this building is retrofitted by diagonal prestressing as discussed in subsequent section to reduce lateral inter-storey drift.

The second structure considered for Vancouver consists of a medium rise 10-storey building. Through dynamic analysis, the structure is shown to be seismically deficient for a number of different earthquake records as can be seen in Figure 4.33. The maximum drift demand recorded is 7.44% when subjected to the Cascadia #4 Earthquake Record. This is clearly a structure that would have failed regardless of the splice condition, or adequate shear capacity. However, for the analysis of retrofitted 10-storey building, the Short Earthquake Record #3 was used, where the maximum lateral drift was 3.47%. This building in Vancouver forms the second case studied in the next section.

The third and final structure investigated was the 15-storey high rise building in Vancouver. Again, the structure was exposed to all 12 artificial earthquake records and was noted to be seismically deficient for a number of records, as can be viewed in Figure 4.34. The maximum inter-storey drift was 3.38% for the Short Earthquake Record #3. Similar to the 5 and 10-storey buildings, the 15-storey building is retrofitted in the next section to reduce inter-storey drift to 1%.

## **4.6 Analysis of Retrofitted Structures**

### **4.6.1 Retrofit Technique Employed**

As previously indicated, the effectiveness of lateral bracing of seismically deficient non-ductile concrete frame systems, by means of diagonal prestressing of suitable bays, was investigated in this investigation. The retrofit methodology composed of steel cables with various levels of applied prestress. Each cable consists of 7 wire strands with a total cross sectional area of 140 mm<sup>2</sup>/strand. The steel strands have a yield strength of 1600 MPa and ultimate fracture strength of 1860 MPa. However, they were modeled to have bilinear stress-strain relationships, with a zero post-yield slope (no strain-hardening) and a yield strength of 1600 MPa. For the purposes of this research, different levels of initial prestressing were explored for each condition. The variable prestress levels were 0% (0 kN), 4% (10 kN), 22% (50 kN), 45% (100 kN), 67% (150 kN), 89% (200 kN) and 99% (223 kN) of steel yield strength. The values of forces shown are those that were applied to each cable within the middle-bay of each exterior frame, per direction. This range of prestressing represents a broad spectrum of applications of prestressing between 0% and 100%, and is therefore believed to provide insight into the effectiveness of prestressing. The number of strands was also changed as a retrofit design parameter to assess the significance of the level of bracing steel area and associated increases in strength and stiffness. Since the building displaces in two directions, it is possible that the maximum inter-storey drift is higher in one direction for one run and in the other direction for the next. The results plotted in the next section show the absolute maximum inter-storey drifts, which represent worst case outputs of the same analysis.

#### 4.6.2 Analysis of Retrofitted 5-Storey Building in Vancouver

The 5-storey low rise structure in Vancouver showed a lateral inter-storey drift of 2.41% under the Short Earthquake Event #2, when analyzed in as-built conditions, without the retrofit. The same structure was first retrofitted with 4 cables per exterior frame in each (of the diagonal) direction. The inter-storey drift was reduced but remained in excess of 1.36 %, as shown in Figure 4.35. Since the reduction was not sufficient to attain the target drift level of 1%, the number of cables was increased to 6 cables per exterior frame in each direction. In this case the interstorey drift was reduced to approximately 1.0 % as in indicated in Figure 4.36. However, this level of improvement was achieved by excessive prestressing. The structure was next retrofitted with 8 cables per exterior frame in each direction. The interstorey drift in this case was further reduced to 0.9 %, as illustrated in Figure 4.37.

It is worth noting that the relationship between the level of prestressing and the reduction in drift is not linear. In some cases the maximum drift was less without any prestressing than the drift with some prestress in the cables, though increased prestressing almost always gave the highest drift control. Let us examine the case with 8 cables per frame per direction. This arrangement successfully reduced the interstorey drift below 1.0% when 200kN of initial prestressing was applied to each and every cable, at every floor. Without any prestressing produced a maximum lateral drift of 1.16%. Compared to the unretrofitted case, the simple introduction of cables reduced drift from 2.41% to 1.16% and the prestressing resulted in a marginal reduction in drift, bringing overall maximum drift demand to 0.89%. The benefits of adding more cables should be weighed against increasing the level of prestressing to achieve the maximum efficiency in design.

In Chapter 3, Table 3.10 presents the periods of vibration for the unretrofitted structure as well as the periods of the retrofitted buildings. As can be seen in the table, when the number of cables increases, the period decreases, which can be explained through an increase in the structural stiffness. Conceptually, a decrease in the period (or increase in the frequency) would indicate that the structure will attract more lateral forces from the seismic event. Figure 4.26 shows that the spectral accelerations increase as the period varies from the unretrofitted structure up to the

optimal retrofitting scheme. This concept is further explored when discussing the effects of retrofitting on the building base shear.

It is important to note that Table 3.10 does not include any values for the structures with retrofitting under different levels of prestress. The algorithm used in the determination of the building period is based on the building stiffness, and is not affected by the level of prestressing in the cables. At the initial time step, when the period is calculated, the building is at rest, and therefore all the horizontal components of the cables cancel each other, thus having no effect on the calculation of the period. However, it is expected to get differences in structural period during response to earthquake excitation, as the tension force in one diagonal cable increases as the force in opposite diagonal cable decreases, resulting in a net increase in lateral bracing forces.

#### **4.6.3 Analysis of Retrofitted 10-Storey Building in Vancouver**

During the analysis of the 10-storey medium rise building in Vancouver, the lateral inter-storey drift demand of the unretrofitted building under the Short Event Record # 3 was 3.47%. Similar to the 5-storey structure discussed above, the first retrofit case considered was installing 4 cables per exterior frame in each direction with varying levels of prestressing applied to each cable. The results of this first case can be seen in Figure 4.38, which demonstrates that the absolute inter-storey drift is reduced to 1.29% when the initial prestress approached the yield strength. The number of cables was therefore increased in an attempt to further reduce the lateral drift. The second retrofit arrangement was for 6 cables per exterior frame in each direction and the lateral drift was further reduced to 1.09% when the initial level of prestress was about the yield stress. This is shown in Figure 4.39. The number of cables was further increased to 8 cables per exterior frame in each direction which produced a maximum lateral drift of 1.02% when the cables were prestressed to their yield strength, as seen on Figure 4.40.

Examination of drift demands indicates that the presence of cables was sufficient to introduce a significant bracing, even without any prestressing. The use of 8 cables per frame per direction at each floor level reduced the drift demand from 3.47% to 1.23%, a considerable reduction,

approximately one third of the unretrofitted lateral drift demand. The applied prestress does further reduce the level of drift, though a similar drift control may be obtained by increasing the number of cables.

It is interesting to note that the deflected shape has changed significantly between the three retrofit cases. It can be seen that for the first case, the maximum absolute inter-storey drift is sustained in the 4<sup>th</sup> storey, whereas for the second case, the drift is maximum in the 6<sup>th</sup> storey. Further, the third case also shows that the maximum lateral drift corresponds to the 6<sup>th</sup> storey. We can therefore see that the introduction of diagonal cables does have an effect on the deflected shape of building, which implies that it also has an effect on column shear demands. This is further discussed when evaluating the effects of prestressing on seismic base shear demand.

Similar to the 5 storey building, the effects of prestressing and retrofitting are explored for the 10 storey structure under dynamic analysis. As can be seen in Table 3.10, when the number of cables increases, the period decreases, which can be explained through an increase in structural stiffness. Figure 4.26 shows that the spectral accelerations increase as the period varies from the unretrofitted structure up to the optimal retrofitting scheme, thus increasing the quantity of lateral forces attracted under dynamic loading.

#### **4.6.4 Analysis of Retrofitted 15-Storey Building in Vancouver**

The process adopted for the 5 and 10-storey buildings was repeated for the 15-storey building where the intent was again to reduce the maximum lateral inter-storey drift to 1.0%. The earthquake record under consideration for this structure was the Short Event # 3, which produced lateral drift demand of 3.38%. Figure 4.41, Figure 4.42 and Figure 4.43 show the absolute inter-storey drift demands for retrofitted buildings with 4, 6 and 8 cables per exterior frame in each direction, respectively. The results can be summarized as follows:

- 4 Cables: The maximum lateral drift ratio is reduced to 1.88%. This reduced drift value was attained within the third storey level when the cables were not prestressed. The buildings retrofitted with prestressed cables showed slightly higher drift demands within the bottom third of the structure though they showed increased reductions in drift demands in the upper

stories. Clearly, the number of cables was not sufficient to reduce drift demands to or below the target value of 1%.

- 6 Cables: The drift demands were reduced in a similar manner to the 4-cable structure described above, with 1.67% drift ratio. This drift demand was less than the former case because of the increased total area of bracing cables. Though the highest reduction was observed in the structure retrofitted with unstressed cables, structures with prestressed cables showed higher improvements in upper stories.
- 8 Cables: When the number of cables was increased to 8 per exterior frame, per direction, a similar pattern of improvement was observed as the previous two sets of analyses. The maximum lateral drift demand in this case was 1.45% and occurred when the cables were prestressed to about their yield stress.

As can be seen from above, the number of cables that was able to reduce storey drift demand in low rise and medium rise buildings to acceptable levels did not have the desired effect on the 15-storey building. Therefore, a further increase in the number of cables was required. The next retrofitting scheme used contained 12 cables per exterior frame in each direction. The drift demands observed are shown in Figure 4.44. This arrangement allowed further reduction in drift demand to 1.21% when the cables were prestressed to their yield stress. The next step was to increase the number of cables to 24 per exterior frame in each diagonal direction. The results, plotted in Figure 4.45, indicate a maximum drift demand of 1.11% when the maximum prestress was applied. The last trial included 36 cables per exterior frame in each direction. This resulted in a drift demand of 1.11%, as shown in Figure 4.46, which did not change from the 24 cable case. This indicates that beyond a given point, additional cables no longer have a positive effect on inter-storey drift demands. It should be noted that, although the maximum drift remained the same for buildings with 24 and 36 cables, the maximum drift location was different, indicating different deflected shapes in these two sets of buildings.

Figure 4.47 illustrates that drift demands tend to remain the same even for different prestressing levels. The analyses of buildings under different prestressing levels, from 50kN per cable to 223kN per cable all exhibit very similar behaviour within 0.05%, which indicates that the

prestressing no longer has an effect on inter-storey drift and the tendencies are converging onto a given displacement pattern.

The structure with 36 cables per external frame per direction showed a drift demand of 1.47% without any prestressing, which is a considerable reduction from the unretrofitted drift ratio of 3.38%, but slightly greater than the highest prestressing applied, which showed a drift demand of 1.11%. This confirms the earlier observation in 5 and 10 storey buildings, i.e., the installation of cables without prestressing does greatly reduce the maximum inter-storey drift. In some cases, i.e. 4 and 6 cables per exterior frame, the behaviour of the structure is the most efficient when no prestressing is applied to the cables. Therefore, the value of the additional drift reduction due to prestressing is questionable whereas the most efficient design would simply require a slight increase in the number of cables to bypass the entire prestressing process.

Similar to the 10 storey building, the effects on the deflected shape can be seen even better with the 15 storey structure. By examining Figure 4.41 to Figure 4.46, it can be seen that an increase in the number of cables does reduce the maximum inter-storey drift, but it is also evident that the increase is much more evident in the lower levels of buildings. The drift reductions are proportionately much greater for the lower floors compared to the upper floors, which is clearly illustrated when the maximum drift for the 4 cable arrangement takes place between the 3<sup>rd</sup> and 4<sup>th</sup> floors whereas the maximum drift for 36 cables is at the 11th floor. Figure 4.47 and Figure 4.48 illustrate the behaviour of the deflected shape of structure as a function of the number of cables. The figures include cases of no prestressing as well as 223kN of prestressing force per cable. Both figures clearly show that the effects of the cables on the structure are the most influential at the base of the structure, irrespective of prestressing. This may be explained through the effects of column shear. The shear forces along the height of a building are cumulative from the top down, thus providing the greatest amount of column shear at the base of the structure. With the introduction of the cables, the building becomes stiffened, thus reducing lateral deformations and concurrently the columns are exposed to less shear forces since the cables would be assumed to transfer the loads to the working point of the structure, which in this case would be the rigid diaphragm of the reinforced concrete slab. Therefore, as we increase the

number of cables, the area of greatest shear forces would obviously benefit the most from the additional capacity of the cables.

It can be easily seen that although the retrofit of the 15 storey building under the given earthquake load is theoretically possible, it is definitely not practical. The application of 36 cables per exterior frame in each direction is clearly unrealistic unless a smaller number of cables is used in more than two frames.

Same as the 5 and 10 storey buildings examined above, the effects of prestressing and retrofitting are explored for the 15 storey structure under dynamic analysis. As can be seen in Table 3.10, when the number of cables increases, the period decreases, which can be explained through an increase in the structural stiffness. This tendency is again revealed for the 15 storey structure. Figure 4.26 shows that the spectral accelerations increase as the period varies from the unretrofitted structure up to the optimal retrofitting scheme, thus increasing the quantity of lateral forces attracted under dynamic loading.

#### **4.7 Effects of Building Mass on the 5 Storey Structure in Vancouver**

The mass of a structure has a direct link to the behaviour under seismic events. The methods of analysis described in the national building code, to determine the horizontal forces applied to a structure by an earthquake, are all a function of the mass of the structure. Of course there are variations within the distribution of the horizontal loads along the height of a building, but the fact remains that the loads are directly derived from the mass at a given floor, more specifically the inertia force created by the mass of the floor. These equations are discussed in section 4.1 for both the NBCC 1970 as well as the NBCC 2005. As the building mass is increased, the base shear is increased, and therefore the horizontal loads at each floor are also increased. With an increase in the lateral loads, one would assume that the structure would require additional load resisting elements to sustain the addition of lateral forces in the structure. This section will examine the effects of building mass on the requirement of retrofitting through prestressed cables.

The typical low rise 5 storey structure previously discussed was chosen for this analysis for its simplicity and symmetry. The intent was to maintain the two dimensional model as is, and simply increase the number of interior bays within the building to represent an increase in building mass. When conducting the analysis in the short direction of the structure, we can assume that the building period will remain constant as the building is literally being elongated in its long direction and therefore the ratio of the mass relative to the number of moment resistant frames remains constant. If the period remains the same and the mass is increased, this would represent that the stiffness would also be increased proportionally to the mass. As the stiffness is a function of the moment of inertia, this value will be increased proportionally to the mass to achieve the proper modeling. In addition, to achieve the required building model, all material properties for the interior frames was increased by a chosen coefficient, which were 1, 2, 3 and 4. All properties, including the cross sectional area, the moment of inertia, the flexural yield moments, the moment-axial force interaction diagrams as well as the lumped masses and fixed end forces were all increased by the given coefficients. This method is the same used initially when lumping together the interior bays to create the original two dimensional models. The method is simply expanded to create larger structures for the purposes of studying the effects of mass on the building behaviour.

For the typical low rise building, the interstorey lateral drift was successfully reduced below 1.0% with the addition of 8 cables per exterior frame, at each level, in each direction, which were all prestressed to 89% of their yield strength (200kN). This sample case will be the basis for examining the effects of building mass.

The first case under review would be to increase the number of interior moment resisting frames by a factor of 2. The first run of the analysis was conducted using the same number of cables and prestressing as the original structure. The second design run was conducted by doubling the number of cables in the exterior frames to account for the doubling of the mass. This would be an intuitive response to the increase in the mass since the increased mass would attract higher inertia forces, thus creating higher lateral forces that would be required to be resisted by the structure. In this case, the principal lateral resisting elements are the external frames, retrofitted with prestressed diagonal cables. It is important to note that although the majority of the lateral

resistance is undertaken by the external frames, each and every internal reinforced concrete moment resisting frame does contain a limited lateral load resisting capacity. This would lead to a redistribution of the lateral load carrying capacity of the structure and would not necessarily distribute the entirety of the loads to the external frames and thus the intuitive response to simply double the number of cables may not be entirely accurate and would ultimately introduce errors.

The maximum interstorey drift for the original structure was recorded as 0.89%. With the mass amplified by a factor of two, the interstorey drift was increased to 1.35%, while maintaining the same number of cables and prestressing. The number of cables was therefore increased by a factor of two to account for the increased lateral forces and the results were a maximum lateral drift of 0.92%. For this test case, the building drift of the structure with two times the mass was successfully reduced to approximately the same level as the original building by increasing the number of cables linearly with the increase in mass. This may be explained by the fact that the stiffness of the external frames is still relatively close to the total stiffness of the internal frames. The next two cases will further examine the behaviour as the mass is increase by a factor of 3 and 4 respectively.

The next study case was to increase the number of internal frames by a factor of three from the original building. After increasing the mass three fold, and maintaining the number of cables per the original design, the lateral interstorey drift was increased from 0.89% to 1.38%. The number of cables was then increased from 8 to 24 in relation to the 3 times increase in mass. The building drift was reduced down to 1.05%. Although the reduction is very near our intended goal, we would like to replicate the results of the original building. Therefore, the number of cables was further increased to 32, which produced a maximum drift of 1.05%, the same value achieved for 24 cables. To continue with the analysis, the number of cables was then increased to 48, six times the original number of cables for an increase of three times in mass. This resulted in a maximum interstorey drift of 0.92%, which is relatively close to the 0.89% achieved by the original building. These results indicate that the effectiveness of the cables definitely decreases with an increase in building mass and may be explained as described below.

To resist the increased level of shear at each storey, the number of cables is increased to provide more lateral loads resistance. However, these additional cables will further stiffen the external frames, causing them to attract even more horizontal load and thus reducing the effectiveness of the cables, with or without prestressing. This same behaviour was experienced with the 15 storey building in the west, where lateral drift would no longer be reduced beyond a certain point.

The final case study to visualize the effects of building mass was the increase the building mass by a factor of four from the original 5 storey low rise structure. Again, the case building was analyzed with an increased mass while maintaining the original cable configuration. This resulted in an increased interstorey lateral drift of 1.52%, from the original 0.89%. To follow suit with the previous examples, the number of cables was then increased linearly with the mass up to 32 cables per exterior frame, at each level, in each direction, resulting in a lateral drift of 1.16%. As there was still some work to be done by the cables, the number of cables was increased to 48 and 64 cables per exterior frame for maximum lateral drifts of 1.11% and 1.04% respectively. The design analysis now was including eight times the number of cables with an increased mass of only four times the original and the goal drift of 0.89% had yet to be reproduced. The further test the behaviour, the number of cables was increased to 96 per exterior frame. This resulted in an unexpected output of 1.12%, which is an increase in the maximum lateral drift from the previous analysis comprising of 64 cables. This further reinforces the concept that as the building mass increases, the lateral loads proportionally increase as well, requiring a stronger lateral force resisting system, i.e. more diagonal retrofitting cables. However, as stated above, as the number of cables is increased, so is the relative stiffness of the exterior frames, thus attracting a greater seismic force to the frames. This now becomes a vicious cycle that in this case becomes unsolvable using this type of retrofitting system since the cables appear to become ineffective in comparison to the remaining reinforced concrete moment resisting frame.

Figure 4.49 presents a graphical comparison between the increased number of cables relative to the increase in building mass. Using the original 5 storey building used in the analysis of the interstorey drift reduction as a benchmark, ratios are developed for both the number of cables

and the building mass. As the benchmark model achieved the desired interstorey drift using 8 cables per exterior frame, at each floor, and in each direction, the ratio presented in Figure 4.49 is given as a function of 8 diagonal cables. Similarly, the mass is presented as a ratio of the benchmark, for example, 2 would represent a building with twice the building mass as the original benchmark. The tendency of the graph appears to present a significant increase in the required number of cables for a linear increase in the building mass. An estimated relationship between the number of cables and the building mass can be described as a second order polynomial. It can therefore be seen that the effects of diagonal prestressed cables are much more pronounced for building of more conservative size and mass, thus making this type of retrofitting more cost effective for these smaller structures. Further research would be recommended to confirm the above tendencies. If diagonal prestressed cables are not cost effective for buildings or large mass and size, limitations may be provided to aid a potential user or client in the choice of an efficient and cost effective retrofitting method.

#### **4.8 Distribution of Cable Pattern on the 10 Storey Structure in Vancouver**

In the previous sections, it has been shown that the introduction of prestressed cables may indeed act to reduce the lateral seismic displacements caused by artificially created earthquakes. The initial cable arrangement proposed and utilized was to install the cables diagonally in the middle bay of each exterior bay. For each floor, the number of cables remained constant for the ease of analysis as multiple runs were undertaken for each given structure. This cable arrangement is clearly not the most efficient design as can be seen in the Figure 4.35 to Figure 4.46. These figures demonstrate that the effects of the cables are much stronger on some floors than others. This would lead one to believe that if we attain 1.0% maximum lateral interstorey drift between two floors, then any other values of interstorey drift that are below 1.0% are therefore overdesigned and would potentially be structurally workable with a lesser number of cables or reduced levels of prestress. If the 1.0% maximum lateral drift may be obtained by reducing the number of cables or the level of prestress, it would produce a more economical and efficient design.

To further explore this concept, the medium rise, 10 storey building was chosen as the 1.0% maximum lateral interstorey drift was successfully achieved with a reasonable number of cables. Refer to Figure 4.50 for the absolute maximum lateral drift with 8 cables per exterior frames in each direction, at each floor. This is the response achieved in the previous section under the initial cable arrangement. By using an iterative process of reducing the number of cables and the prestress levels, the absolute maximum lateral interstorey drifts were obtained as shown in Figure 4.51. Through the iterative process, it was noted that a reduction in the number of cables had a much greater effect on the lateral drift than reducing the level of prestress. Therefore, once the number of cables was optimally reduced, further tuning was undertaken through the reduction of the prestress levels. This was of course a trial and error process.

Table 4.1 presents the modified cable arrangement producing the graph in Figure 4.51, which is quite close to the ideal situation of a constant lateral interstorey drift of 1.0% at each floor. Ideally, each interstorey drift would be exactly 1.0%, however, this is not the case and will vary for each building and each earthquake record. The concept remains the same as we are trying to find the most efficient design possible. Apart from the roof level, the interstorey drifts are maintained between 0.97% and 1.00%, all within reasonable tolerances.

As previously discussed, the modification to the number of cables has an effect on the deflected shape of the building under analysis. Therefore, for the purposes of achieving a constant drift of 1.0% at each floor, it is not as simple as reducing the number of cables at one floor and expect no response floor the adjacent floors. In some cases, a reduction in the number of cables or the prestress levels would unfortunately increase the lateral drift at the adjacent floors, thus putting them over the 1.0% maximum that we are attempting to maintain. As the iterative process continued, it was necessary to increase the number of cables for a number of floors to allow for a significant reduction in the number of cables at the remaining floors.

To ascertain if the modifications have indeed produced a more efficient design, there are a number of areas that may be quantified and compared. The first would be the total number of cables required in the building structure. For this particular example, some were increased and others were decreased. The final number of cables in the new cable arrangement is 79 cables per

exterior frame in each direction. This is the sum of all the cables at each of the floors. The original cable distribution requires 80 cables per exterior frame. Therefore, a reduction of 1.2% is achieved in the total material costs for this case study.

The second quantifiable item is the average prestress applied to each cable throughout the structure. The original cable arrangement required that all cables be prestressed to 99% of their yield strength. For the new cable arrangement, the prestress levels were reduced for a number of the floor. The average amount of prestress applied to each and every cable in the original cable arrangement was 223kN or 99% of their yield strength. Under the new cable layout the levels of prestress varied between zero and 223kN (99% of yield strength) for an average prestress per cable of 129kN (58% of the cable yield strength) or approximately 58% of the original cable layout. Although the material costs were not greater reduced in this particular case, the labour costs associated with prestressing each individual cable was reduced by almost half the original value.

The trial and error process is difficult and long. A step by step procedure could be developed to facilitate the method to determine the most efficient design.

#### **4.9 Effects on Base Shear for the 5 Storey Structure in Vancouver**

The previous sections of this paper focused mainly on controlling lateral displacements in a structure. However, in addition to lateral displacement, base shear demands are equally important when assessing the effects of seismic events. Through basic engineering principles, forces cannot be induced without displacement. Therefore, the loads that create the lateral displacement must also have an effect on the elements themselves. In this case, we are considering lateral loads due to a seismic event and therefore column shear forces become the dominant load attracted at each floor. Axial load and flexural demands in the beams and columns are treated as secondary to column shear, and will therefore be ignored for the purposes of this thesis.

As previously mentioned, the lateral loads due to an earthquake are cumulative from roof level down to the ground floor. Hence, the greatest and most critical shear forces will be sustained at the base of the building, in this case the ground floor. Although the column cross section and reinforcing would reduce along building height, this would be in consideration of gravity loads only under the 1970 NBCC, and the reduction would not be great enough to induce critical shear failure at an upper level column. Under the most recent 2005 NBCC, column shears may indeed be the governing factor, however, the majority of structures built in present, have drift control through the addition of concrete shear walls, and the columns are not expected sustain high shear forces. This would be applicable to both Eastern (Ottawa) and Western (Vancouver) structures.

The effects of retrofitting a reinforced concrete frame with diagonally prestressed cables is examined and studied for the sample low rise, 5 storey structure in western Canada (Vancouver). This structure was previously labeled as seismically deficient, and under the effects of multiple different earthquakes the maximum lateral interstorey drift was successfully reduced to below 1.0% through the addition of the prestressed cables. The optimum retrofitting for this particular structure was noted to be 8 cables per exterior frame, at each floor, in each direction. The cables were also prestressed to an initial value of 200kN per cable or 89% of their yield strength. Retrofitting the noted structure reduced the lateral drift to the point where the building would most likely behave in an elastic fashion, thus reducing the risk of failure through anchorage slip or shear failure. Both the noted failures modes are common in older reinforced concrete moment resisting frames during significant seismic events. Basic engineering principles and structural frame analysis indicate that the lateral deformations should be proportional to the column moments, which in turn are proportional to the column shear forces. Therefore, as the lateral displacement of the example structure is reduced, there should be a proportional decrease in the column base shear. However, this would only apply as long as the frame in question remains unchanged after retrofitting. This is not the case as diagonal tension members, i.e. cable elements, are introduced on the exterior frames of the structure. These new elements alter the free body diagram of the frame under analysis and create new equilibrium equations at the nodes. Consequently, the principles and equations related to standard frame analysis no longer apply. However, it can be noted that intuitively, as lateral displacements are decreased, the horizontal shears in the reinforced columns would also be decreased since displacements induce forces in

the members. Another item to consider is the structure's overall stiffness with the addition of diagonal tension elements. This topic is further examined later in this chapter.

There were a number of assumptions made when creating the model for the study of base shear. The first and foremost would be that the cables are anchored to the foundations, and are treated independently from the column base. Although the working points of the nodes are concurrent, it is assumed that there is no physical connection between the cable connection and the reinforced concrete column base. This would allow for individual analysis and consideration and therefore, the cables would not impose shear forces on the reinforced concrete columns.

All shear forces are taken from the output files of DRAIN RC. Although the input for DRAIN RC has been updated to a graphical user interface, the output format remains as text files printed to the executable file's folder location. These output files are crude and require some manipulation in order to properly summarize to data. For the case of the column shears, the reinforced concrete elements are relatively easy to extract, however, the cable element forces must be taken as axial tension forces and transmitted to horizontal shear components acting on the working point of the reinforced concrete column.

Figure 2.1 illustrates the node layout as input in DRAIN RC. The node numbers are shown in yellow and start at number 1 for the bottom left node and increase to number 48 at the top right node. To properly analyze and summarize the member forces, a proper understanding of the element force sign convention is required and is illustrated in Figure 4.52. For a right swaying building, at node number 2, the net base shear would be the sum of  $V_{\text{column}}$  plus  $T_{\text{hor cable}}$ . As the cable orientation changes at node number 3, the net base shear at this node would be the sum of  $V_{\text{column}}$  minus  $T_{\text{hor cable}}$ . The next section will examine the base shear in relation to the retrofitting scheme of the structure with regards to both the overall base shear demands as well as the localized shear capacity of the most critical column at the base.

#### 4.9.1 Total Base Shear

The total base shear demand of a building structure, such a reinforced concrete moment resisting frame, with no retrofitting, would be equal to sum of the column shear forces at the base of the building. Each column is assumed to have a rigid (fixed) connection at the foundation level and therefore each column will have three separate reactions, being axial force, bending moment and shear force. With the introduction of the diagonal cables in the exterior frames, there are additional components to the total base shear of the structure as the cables will be anchored into the foundations. The horizontal component of the cable tension must be considered as acting in the same plane as the column shear forces and therefore must be included in the total base shear demand as seen in Figure 4.52.

For the building under consideration, the total base shear demand was calculated for both the building without retrofitting as well as with retrofitting, using the most efficient cable arrangement, reducing lateral interstorey drift ratio to 1.0%. Figure 4.53 illustrates the total base shear time histories for both cases. At a first glance, the total base shear is increased, especially as the earthquake record progresses. Table 4.2 presents maximum base shear demands for the time history shown in Figure 4.53. The results present that the overall base shear demand of the structure is increased by 160.8%. As previously noted, the period of the 5 storey building was noted to decrease as the number of cables increased, therefore, the spectral acceleration of the building, based on the period, was also noted to increase. This would lead to the conclusion that the increase in base shear noted above is directly related to the increased spectral acceleration as the increase in spectral acceleration would case the structure to attract more forces from the seismic event.

To properly compare the effects of the diagonal cables on the base shear, we must first consider the effects of the cables on the overall building stiffness. Figure 4.54 shows the roof displacement of the low rise structure under both retrofitted and unretrofitted events. In addition to clearly seeing a significant reduction in the lateral displacement at the roof level, the period of the retrofitted structure can be estimated and idealized as approximately one third of the period

of the unretrofitted structure. The relationship between the period (or frequency), the mass and the stiffness has been previously discussed and shown as:

$$\omega = \sqrt{\frac{k}{m}} \quad 4.2$$

If the period is assumed to be reduced to one third, the frequency is then tripled. Since the mass remains constant for both models, the stiffness of the structure as a whole would therefore be increased per the relationship shown below:

$$3\omega = \sqrt{\frac{9k}{m}} \quad 4.3$$

To satisfy the equation, the stiffness,  $k$ , must therefore be increased by a factor of 9. Intuitively, by introducing diagonal tension member in the exterior frame, the structure's stiffness would be increased due to its reduced capacity for lateral displacement and is shown as true for the case above. If the stiffness of the structure is increased, it would therefore attract more inertia forces induced by the ground motion, in turn increasing the lateral applied forces. Unfortunately, an increase in the lateral forces required a resisting force, which would consist of the base shear of the building.

Figure 4.55 illustrates the time history of the base shear contribution of the reinforced concrete columns only. This time history is unchanged for the unretrofitted model since the concrete columns are the only elements that contribute to the base shear demand, however, for the retrofitted case, the horizontal component of the cable elements is neglected so to consider the contribution of the reinforced concrete columns only. By comparing only the reinforced concrete columns, the focus can be set on the element, which in the past has proven to be seismically deficient. Even if the lateral drift is controlled to a level where failure through anchorage slip and shear is avoided for flexural dominant columns, the shear forces at the column base must still be resisted. If the column shear are not significantly reduced through the retrofitting technique, it is possible that the columns do not have the shear capacity required and must therefore be retrofitted using other means. The overall base shear demand of the reinforced concrete columns was decreased by 9.3%. The behaviour of each column separately is considered and studied in the next section.

Figure 4.56 illustrates the time history of the base shears of the reinforced concrete columns and the diagonal cables, both for the model with applied retrofitting. This figure demonstrates that the cables are acting in the opposite direction of the column base shears. Both the cable forces and the column shears appear to be in phase, but in different directions. This would indicate that as the ground motion is inducing lateral shear forces in the columns, the diagonal cables are resisting the column shear forces. However, the forces in both elements are not identical nor perfectly symmetrical, and therefore the curves can be superimposed to draw the total base shear demand for the retrofitted structure, as shown in Figure 4.56.

#### **4.9.2 Base Shear of the Critical Columns**

In the previous section, the overall structure was analyzed and the total base shear demands were reviewed and found that retrofitting using prestressed diagonal cables did not reduce, but increase the overall base shear demand. This increase in overall base shear is a combination of all the columns, in addition to the horizontal components of the cables. If the columns were indeed shear dominant columns, the column shear forces would be critical at each and every column.

Table 4.3 tabulates the maximum base shears obtained at the base of each ground floor column. Since the maximums are taken over the entire time history, they do not all correspond to the same point in time. The dynamic analysis indicated that some of the concrete columns are actually sustained increased base shears, potentially contributing to the overall increase in total base shear seen in the previous section. Nodes 1 to 4 consist of the columns lumped at the external frames and nodes 5 to 8 are the columns lumped for the internal frames. The time history of all 8 nodes is illustrated in Figure 4.57 to Figure 4.64. Three out of the four columns that sustain an increase in the maximum applied shear correspond to the exterior frames.

It is important to note that the sum of base shears at each node, as shown in Table 4.3, does not equal the total base shear, as shown in Table 4.2, since each column base shear is a maximum taken over the time history and the maximum values may not correspond to the same time step.

However, the total base shear is calculated for each time step and the values shown are the maximum values found over the entire time history.

As previously noted, the building frequency was increased following the installation of the diagonal prestressed cables. This increase in the frequency can be translated into a linearly proportional increase in the building stiffness. It is known that an increase in the building stiffness will increase the inertia forces at each floor level due to the seismic event. If the horizontal inertia forces are increased at every floor, the cables, in conjunction with the moment resisting frames, must resist these additional loads. At each floor, these additional loads are carried down the building and become cumulative down to the ground floor, or foundation level. Therefore, the advantages of reducing the interstorey drift, which in turn reduced the column bending moment and shear forces, are offset by the increase in stiffness and therefore lateral loads at each floor. It is even noted that in some instances, specifically the exterior frames where the stiffness is the most significantly increased, the maximum column shear at the ground floor actually increases beyond the levels of the unretrofitted structure. Although the base shear is not significantly decreased, retrofitting of the studied low rise structure can be justified by the large and successful decrease in interstorey lateral drift.

In order to justify the increase in the reinforced concrete column base shear, the horizontal component of the cable tension must still be transferred through the node at the beam column interface to satisfy equilibrium. Therefore, this cable force, which may in some cases be quite large, must be partially transferred through the reinforced concrete column and subsequently down to the foundation level where the total base shear demands are quantified and analyzed.

As noted above, the maximum column shear forces were, in some cases, increased beyond the unretrofitted stage due to an increase in the overall building stiffness. It can be concluded that although the base shear demand for each column was not significantly reduced, the reduction in interstorey lateral drift is significant enough to justify even a slight increase in column shear. The overall behaviour of the structure when lateral drift is contained to below 1.0% remains elastic, and anchorage slip and shear failure is typically avoided below 1.0% lateral drift.

# Chapter 5

## Summary and Conclusions

### 5.1 Summary

This research was undertaken in view of a growing number of buildings that were constructed under outdated building codes, some of which have been labeled as seismically deficient. More specifically, these seismically deficient structures were not designed as ductile moment resisting frames. Therefore, the structures exposed to significant seismic events would fail beyond their elastic capacities. This is typical for structures built in the 1970s or earlier. For this reason, a comparison between the most recent 2005 NBCC and the 1970 NBCC is undertaken. For flexure dominant members, experiments have demonstrated that 2.0% lateral drift may be achieved prior to failure. However, for shear and anchorage slip dominant members, the maximum elastic lateral drift is typically constrained to 1.0%. This maximum value of 1.0% was chosen as a benchmark for retrofitting the structures, guaranteeing elastic behaviour, and thus avoiding the concerns of reaching inelastic failure through flexure, shear or anchorage slip.

In an attempt to compare a reasonable number of seismically deficient structures, a total of 3 buildings were chosen in Vancouver. The first structure is a low rise 5 storey building, the second is a medium rise 10 storey building and the third is a high rise 15 storey building. Each of the selected structures is retrofitted using a variety of diagonally prestressed cables in each of

the exterior frames, at all floors. The time history response of the structures is computed using the general program DRAIN RC, which conducts dynamic inelastic analysis. The primary goal of the seismic retrofit technique employed was to reduce the maximum interstorey drift so that the building would remain in the elastic region. A parametric investigation was carried out to examine the effects the parameters outlined below:

- i. The effect of overall building mass on the required number of cables and prestressing required to attain 1.0% interstorey drift for the 5 storey structure.
- ii. The cable arrangement required to achieve the most efficient drift pattern without exceeding 1.0% for the 10 storey structure.
- iii. The effects of diagonal prestressing on base shear. Both the total base shear and the individual column base shears were considered.

Generally, the results indicate that retrofitting structures using diagonal prestressing cables does improve structural response and reduce seismic drift demands.

## **5.2 Conclusions**

### **5.2.1 Lateral Interstorey Drift Demands**

The three buildings in Ottawa did not indicate seismic deficiency. Their behaviour was elastic. Even under increased design base shears to conform 2005 NBCC, the structural response remained within the elastic range, with drift demands well below 1%.

For the structures in western Canada, more specifically Vancouver, all three structures under review were classified as seismically deficient under the 2005 NBCC uniform hazard spectra. The three western structures, displayed reduced interstorey drifts upon retrofitting. The following observations were made based on the analysis results:

- For the 5 storey building, the maximum interstorey drift was reduced from 2.41% to 0.89% for a prestress level of 89% of the yield strength and a total of 8 cables in each bay, in each direction.

- For the 10 storey building, the maximum interstorey drift was reduced from 3.47% to 1.02% for a prestress level of 99% of the yield strength and a total of 8 cables in each bay, in each direction.
- For the 15 storey building, the maximum interstorey drift was reduced from 3.38% to 1.11% for a prestress level of 99% of the yield strength and a total of 24 cables in each bay, in each direction.
- For the 5, 10 and 15 storey structures, the most efficient design are all noted at high prestress levels, ranging from 89% to 99% of the cable yield strength. Although the cables exceed the yield point during the dynamic analysis, thus becoming inelastic, the frame elements in the structure remain in the elastic range.

Therefore, diagonally placed prestressing cables as a method of seismic retrofitting, is an effective technique in reducing interstorey drifts to levels acceptable by the 2005 NBCC. In addition, the interstorey drifts are reduced to a point where the structure behaves elastically, thereby avoiding brittle failure due to inelasticity.

The following conclusions can also be presented with respect to lateral drift demands of retrofitted structures in western Canada:

- The deflected shape and distribution of maximum drift values of depend on the presence and arrangement of prestressing cables. The displacement patterns were noted to vary significantly between unretrofitted and retrofitted structures. This tendency was more evident in taller buildings. When the number of cables was increased at floor levels where maximum interstorey drifts were observed, the maximum drift ratio occurred at other locations.
- As noted above, prestressing the cables do not necessarily reduce interstorey drift. The desired drift control can be achieved with the use of non-prestressed cables. From a cost perspective, it may be more viable to increase the number of cables, rather than to attain drift control with fewer highly prestressed cables. Additional research is needed in this area to explore the feasibility of other cable layouts for increased effectiveness of prestressing.

- With regards to the 15 storey high-rise building in Vancouver, full reduction in drift demands could not be achieved, i.e. the maximum interstorey drift could not be lowered below 1.0%. This could be explained with a sharp increase in force demands with the reduction of retrofitted building period within the high-period range of response spectra, which is the range that governs the 15 storey building response. This could be considered as a vicious circle. For cases such as these, alternative cable arrangements may have to be considered.

### **5.2.2 Building Mass**

The effects of building mass on the effectiveness of the retrofit technique considered were investigated by analyzing the 5 storey low-rise building. The building mass was increased by a factor of 2, 3 and 4 in an attempt to illustrate the relationship between mass, stiffness and the required quantity of diagonally prestressed cables. Using the original retrofitted structure as a benchmark, both the number of required cables and the building mass were varied relative to those of the reference building. The results of increasing the building mass indicated that the relationship between the building mass and the number of cables is not linear. When the mass was increased by a factor of 2, the number of cables had to increase proportionately, thus requiring twice the number of cables to produce similar drift control. As the building mass increased by factors of 3 and 4, the increase in the number of cables was no longer linear and the graph took the shape of a second degree polynomial as seen in Figure 4.49. When the building mass increased, the number of cables required became exponentially larger and would eventually become much too expensive to be cost effective. Therefore, the method of retrofitting consisting of prestressed diagonal cables is most effective for building of lower mass.

### **5.2.3 Cable Arrangement**

The original cable arrangement, which consisted of the same number of cables at each and every floor, was the simplest and most straightforward method of applying the retrofit technique for lateral drift reduction. To further study the effects of varying cable arrangement on the interstorey drift ratios, the 10 storey structure, for which the interstorey drift was successfully

reduced to 1.02%, was chosen as a benchmark model. A number of conclusions were drawn with regards to the cable arrangement, as noted below:

- The initial cable arrangement is not the most efficient as the maximum interstorey drift is not the same at each floor, illustrating that the same number of cables has a different effect on each floor.
- The most efficient design would be one for which each level would have the same interstorey drift, which in the current research project was aimed at 1.0%
- The optimum cable arrangement derived in this thesis was more efficient than the initial cable arrangement in two respects.
  - The number of cables required in each exterior frame, in each direction, was reduced from 80 cables to 79, for a total cable reduction of 4 cables for the entire structure.
  - The average cable prestress, in each cable, was reduced from 223kN to 129kN, for a reduction of approximately 42%, reducing labour costs for installation.

#### **5.2.4 Base Shear**

In order to assess potential changes in total base shear demand due to diagonal prestressing, the 5 storey structure was examined. This structure was successfully retrofitted to reduce the interstorey drift to a maximum ratio of 0.89%. The total base shear demand for the structure was increased by 160.8% due to retrofitting. This value of base shear included the contribution of prestressing cables. This was attributed to the stiffening effect of the cables and resulting shortening of the period of the structure. The base shear demand on columns was reduced by 9.3%. The diagonal cables provided shear resistance in the opposite direction to those sustained by columns. The resulting base shear requirement was computed as the difference between the horizontal component of the cable force and the base shear of concrete columns. It was observed that some of the columns also experienced local increases in shear demands. Half of the columns at the base developed higher shear forces than those for the unretrofitted structure. Therefore, column shear capacities may have to be checked when this retrofit method is employed to ensure that the columns have adequate shear capacity to resist additional shear forces induced by the retrofit technique.

### 5.3 Recommendations for Future Research

The maximum interstorey drift for 3 model structures was successfully reduced to levels that would promote elastic behaviour and eliminate seismic deficiencies. Future research is recommended to expand the inventory of structures analyzed to identify and quantify limitations of the retrofitting technique. Providing additional examples would also reinforce the findings of current research.

To further examine the influence of building mass on the required amount of diagonal cables and level of prestressing, additional analyses are recommended using the 10 and 15 storey buildings considered in the investigation. This future research would further confirm the conclusions and trends observed from the investigation of the 5-storey building.

The modified cable arrangement was identified as more efficient in terms of both the number of cables and the applied level of prestressing. Future research is recommended to confirm the results by expanding the scope to include other building designs, including buildings of varying height, such as the 5 and 15 storey structures presented in this thesis. In addition, the effects of using different earthquake records can also be studied to further assess the interaction of the change in structural period caused by retrofitting and the period of excitation.

The efficient cable arrangement, i.e., one that would provide approximately the same interstorey drift ratio at each floor, was determined using an iterative process. This trial and error procedure is time consuming and repetitive. It is recommended that a parametric study be conducted to establish the relationship between cable arrangement and structural properties. This would help designers in identifying superior cable arrangements without going through trial and error processes.

The structures considered in the current research program are idealized buildings with regular and symmetric floor plans and elevations. Although they were useful models to establish the effects structural and ground motion parameters, there is no substitute for analyzing real buildings which reflects the true practice. It is therefore recommended that the analysis is

expanded to cover a larger numbers of cases and different structures with the challenges of modeling of actual real-life buildings. A more in depth analysis of real life existing structures would be much more representative of true conditions if the modeling was undertaken using a three dimensional software package capable of dynamic non linear analysis. Three dimensional modeling would limit the error by considering torsional effects, geometrical eccentricities and would allow for a more detailed modeling of floor diaphragms.

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## Tables

**Table 3.1: Design Floor Loads**

	Floor	Roof
Dead Load (kN/m <sup>2</sup> )	5.0	3.5
Live Load (kN/m <sup>2</sup> )	2.4	2.2
Concrete Compressive Strength (MPa)	30.0	
Reinforcement Yield Strength (MPa)	400.0	

**Table 3.2: Member Properties for the 5 Storey Frame in Vancouver – NBCC 2005**

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-4	300x450 3-25M top, 2-20M bottom	450x450 8-25M	400x400 8-20M
	5	300x400 3-20M top, 2-15M bottom	450x450 8-25M	400x400 8-20M
Exterior	1-4	300x450 3-25M top, 2-20M bottom	400x400 8-20M	300x300 8-20M
	5	300x400 3-20M top, 2-15M bottom	400x400 8-20M	300x300 8-20M

**Table 3.3: Member Properties for the 10 Storey Frame in Vancouver – NBCC 2005**

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300x450 3-25M top, 2-20M bottom	500x500 12-25M	400x400 8-25M
	6-9	300x450 3-25M top, 2-20M bottom	500x500 12-20M	400x400 8-20M
	10	300x400 3-20M top, 2-15M bottom	500x500 12-20M	400x400 8-20M
Exterior	1-5	300x500 3-25M top, 2-20M bottom	400x400 8-25M	350x350 8-25M
	6-9	300x450 3-25M top, 2-20M bottom	400x400 8-20M	350x350 8-20M
	10	300x400 3-20M top, 2-15M bottom	400x400 8-20M	350x350 8-20M

**Table 3.4: Member Properties for the 15 Storey Frame in Vancouver – NBCC 2005**

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300x450 3-25M top,2-20M bottom	600x600 12-25M	500x500 12-25M
	6-14	300x450 3-25M top,2-20M bottom	600x600 12-20M	500x500 12-20M
	15	300x400 3-20M top, 2-15M bottom	600x600 12-20M	500x500 12-20M
Exterior	1-5	300x500 3-25M top, 2-20M bottom	450x450 8-25M	400x400 8-25M
	6-14	300x450 3-25M top,2-20M bottom	450x450 8-20M	400x400 8-20M
	15	300x400 3-20M top, 2-15M bottom	450x450 8-20M	400x400 8-20M

**Table 3.5: Member Properties for the 5 Storey Frame in Ottawa – NBCC 2005**

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-4	300x450 3-25M top, 2-20M bottom	450x450 8-25M	400x400 8-20M
	5	300x400 3-20M top, 2-15M bottom	450x450 8-25M	400x400 8-20M
Exterior	1-4	300x450 3-25M top, 2-20M bottom	400x400 8-20M	300x300 8-20M
	5	300x400 3-20M top, 2-15M bottom	400x400 8-20M	300x300 8-20M

**Table 3.6: Member Properties for the 10 Storey Frame in Ottawa – NBCC 2005**

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300x450 3-25M top,2-20M bottom	500x500 12-25M	400x400 8-25M
	6-9	300x450 3-25M top,2-20M bottom	500x500 12-20M	400x400 8-20M
	10	300x400 3-20M top, 2-15M bottom	500x500 12-20M	400x400 8-20M
Exterior	1-5	300x500 3-25M top, 2-20M bottom	400x400 8-25M	350x350 8-25M
	6-9	300x450 3-25M top,2-20M bottom	400x400 8-20M	350x350 8-20M
	10	300x400 3-20M top, 2-15M bottom	400x400 8-20M	350x350 8-20M

**Table 3.7: Member Properties for the 15 Storey Frame in Ottawa – NBCC 2005**

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300x450 3-25M top,2-20M bottom	600x600 12-25M	500x500 12-25M
	6-14	300x450 3-25M top,2-20M bottom	600x600 12-20M	500x500 12-20M
	15	300x400 3-20M top, 2-15M bottom	600x600 12-20M	500x500 12-20M
Exterior	1-5	300x500 3-25M top, 2-20M bottom	450x450 8-25M	400x400 8-25M
	6-14	300x450 3-25M top,2-20M bottom	450x450 8-20M	400x400 8-20M
	15	300x400 3-20M top, 2-15M bottom	450x450 8-20M	400x400 8-20M

**Table 3.8: Static Base Shears (kN) Based on the NBCC 2005 and NBCC 1970**

Building type		Static base shear		Reduction factor NBCC 2005 / NBCC 1970
		NBCC 2005	NBCC 1970	
Ottawa	5	1110	432	2.6
	10	1178	756	1.6
	15	1289	966	1.3
Vancouver	5	1353	863	1.6
	10	1633	1513	1.1
	15	1874	1932	1.0

**Table 3.9: Free Vibration Periods for Buildings in Ottawa**

Mode Shape	Period (s)		
	5 storey	10 storey	15 storey
1	1.68	3.37	4.77
2	0.53	1.09	1.54
3	0.29	0.61	0.87

**Table 3.10: Free Vibration Periods for Buildings in Vancouver**

	Mode Shape	Period (s)		
		5 storey	10 storey	15 storey
no retrofit	1	1.676	3.3706	4.7728
	2	0.527	1.0861	1.5395
	3	0.290	0.6138	0.8693
4 cables	1	1.266	2.6139	3.8567
	2	0.410	0.8448	1.2362
	3	0.232	0.4763	0.6884
6 cables	1	1.164	2.4500	3.6677
	2	0.379	0.7866	1.1623
	3	0.217	0.4397	0.6389
8 cables	1	1.090	2.3359	3.5376
	2	0.357	0.7436	1.1070
	3	0.206	0.4124	0.6016
12 cables	1	na	na	3.3690
	2	na	na	1.0277
	3	na	na	0.5483
16 cables	1	na	na	3.2640
	2	na	na	0.9722
	3	na	na	0.5116
24 cables	1	na	na	3.1397
	2	na	na	0.8984
	3	na	na	0.4634
36 cables	1	na	na	3.0430
	2	na	na	0.8327
	3	na	na	0.4209

**Table 4.1: Summary of Efficient Cable Arrangement**

Storey	Number of Cables	Unit Prestress (kN)
10	1	0
9	6	10
8	8	50
7	12	223
6	14	223
5	12	223
4	8	50
3	6	40
2	6	67
1	6	40

**Table 4.2: Summary of Total Base Shear Values**

	Total Base Shear
No Retrofit (kN)	1589.9
With Retrofit (kN)	4145.7
% change (-ve=decrease)	160.8

**Table 4.3: Summary of Base Shear Values at Each First Storey Column**

Max. Base Shear	Node Location							
	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8
No Retrofit (kN)	73.8	133.5	134.8	78.5	201.6	394.4	394.1	232.8
With Retrofit (kN)	76.2	154.8	159.3	74.9	211.2	340.0	360.5	203.6
% change (-ve=decrease)	3.2	15.9	18.2	-4.6	4.7	-13.8	-8.5	-12.5

## Figures

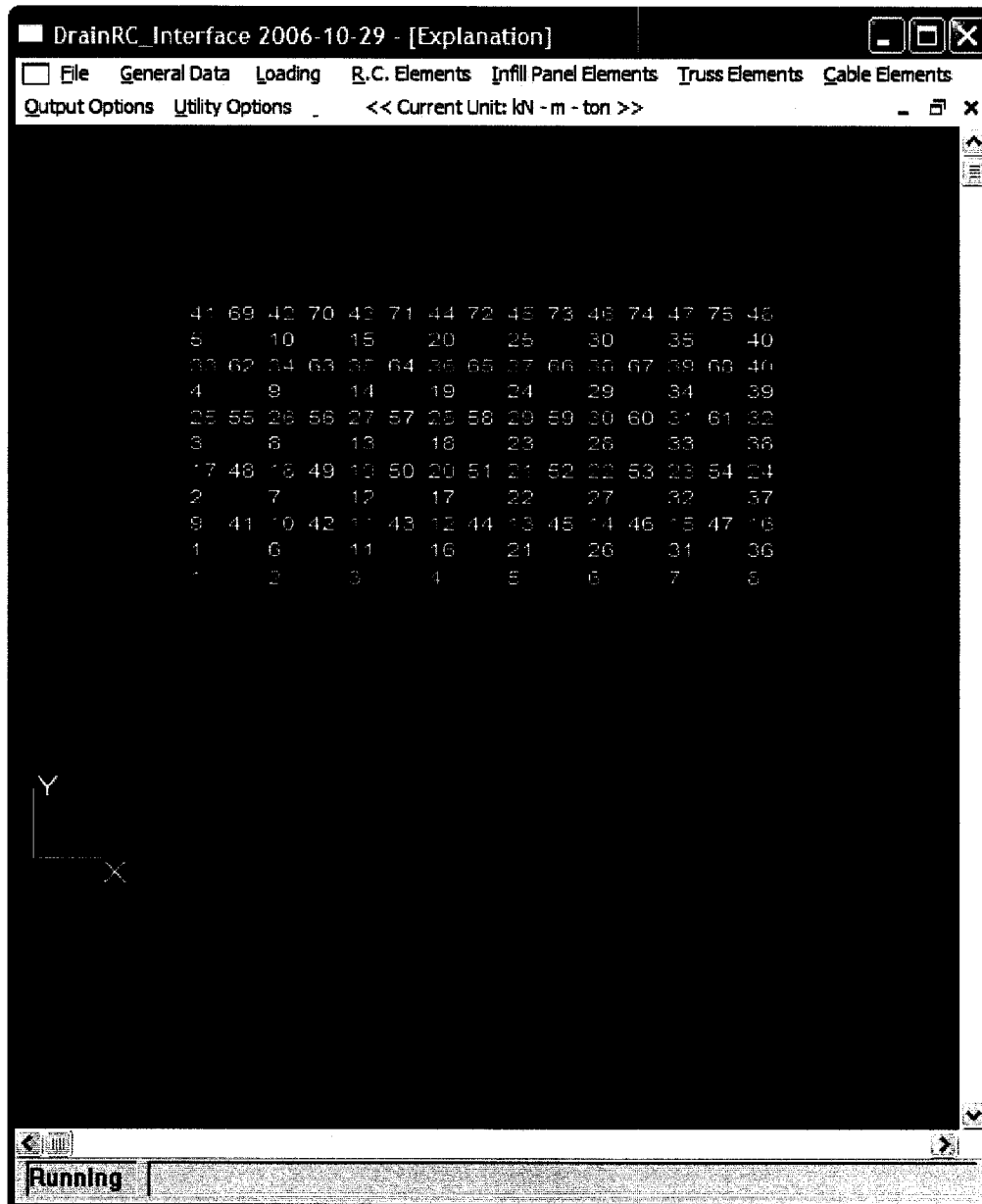
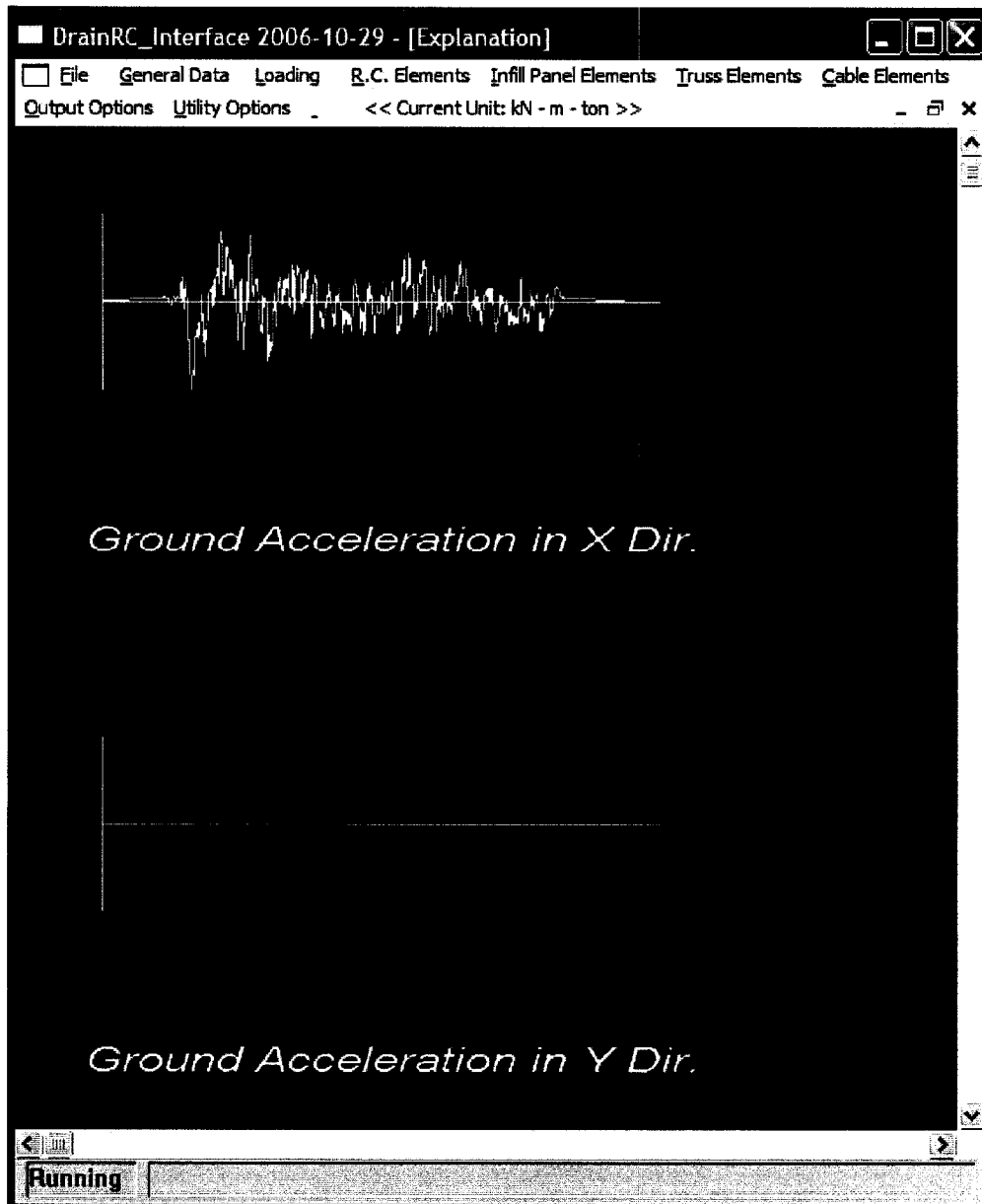
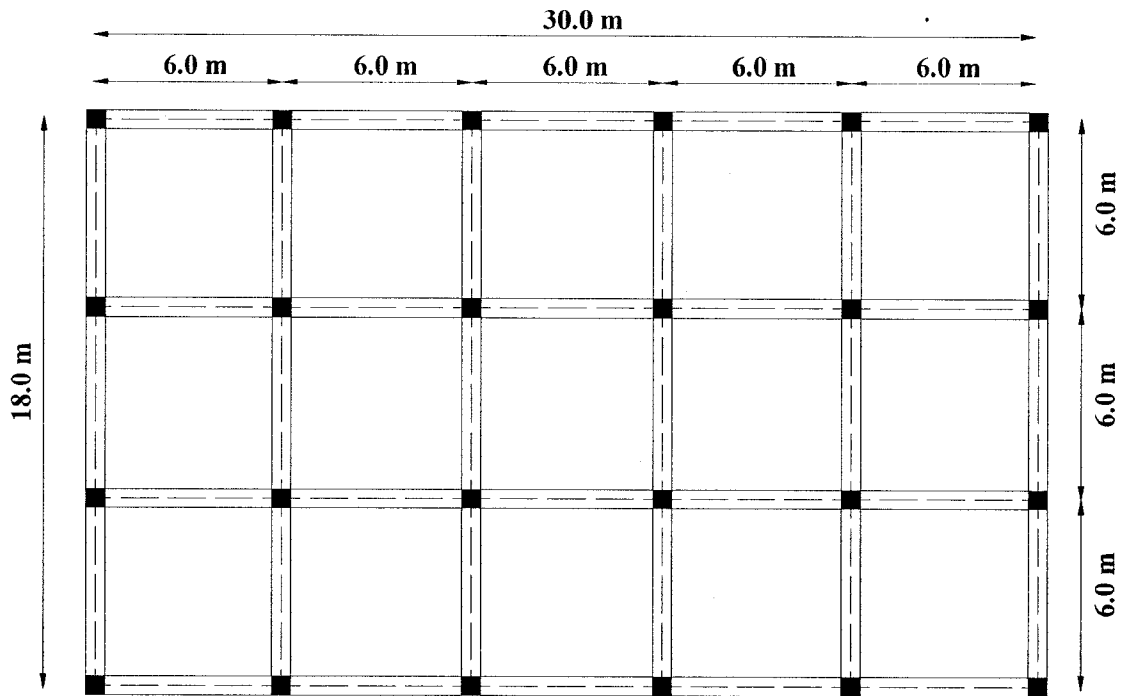


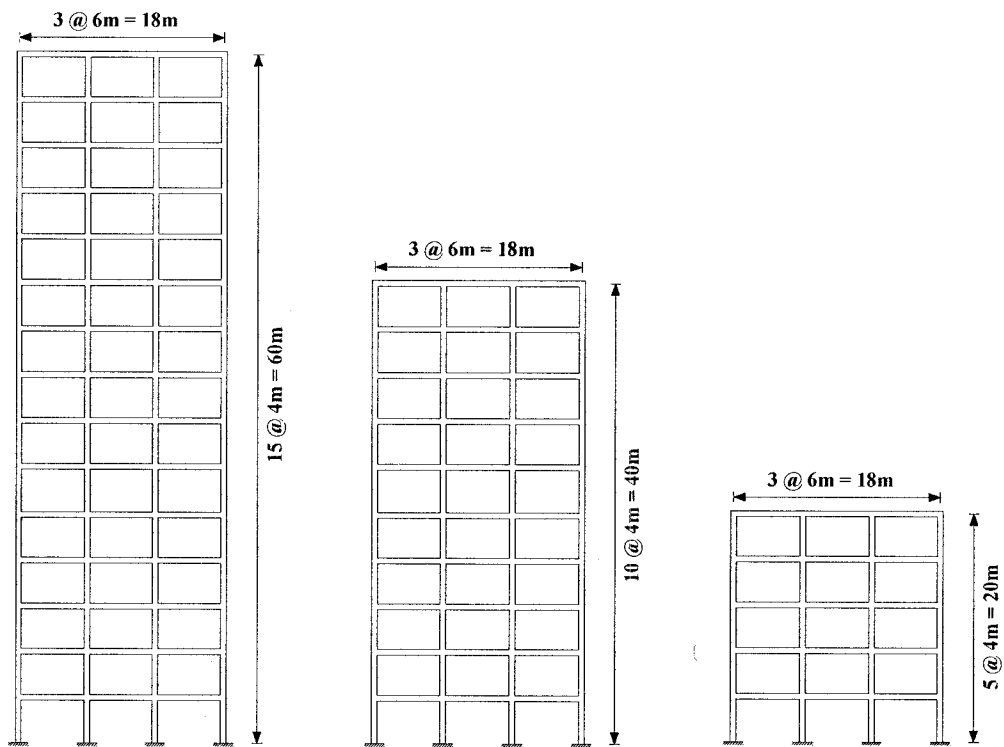
Figure 2.1: DRAIN RC Graphical User Interface - Model



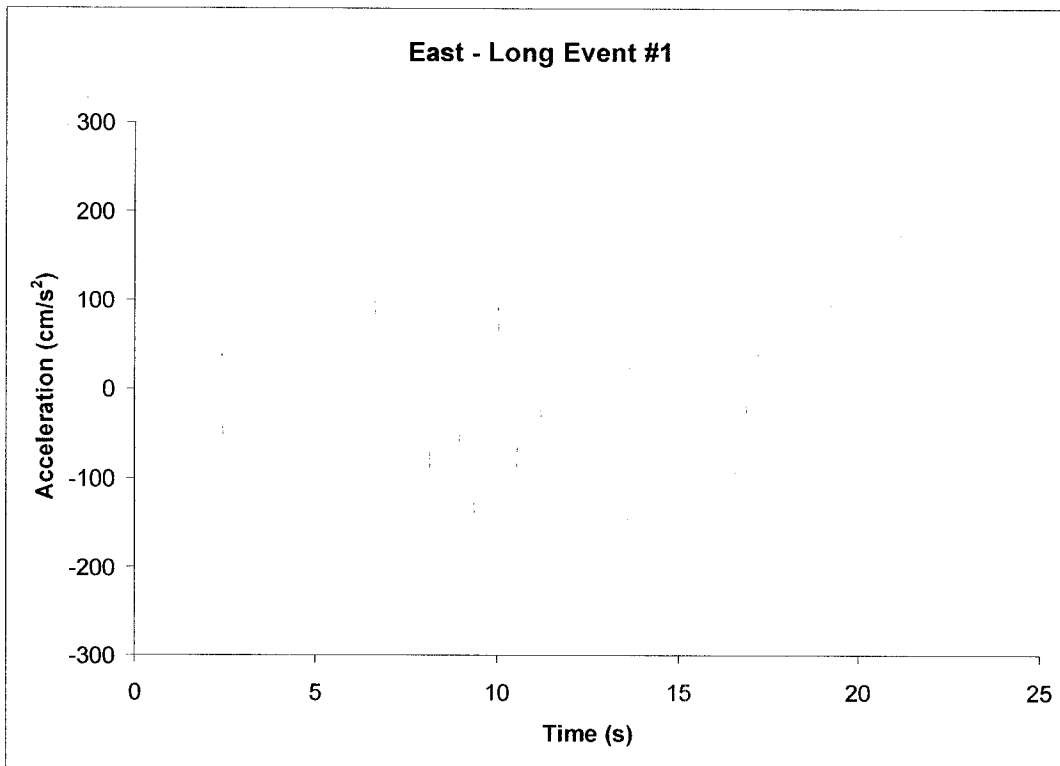
**Figure 2.2: DRAIN RC Graphical User Interface – EQ Record**



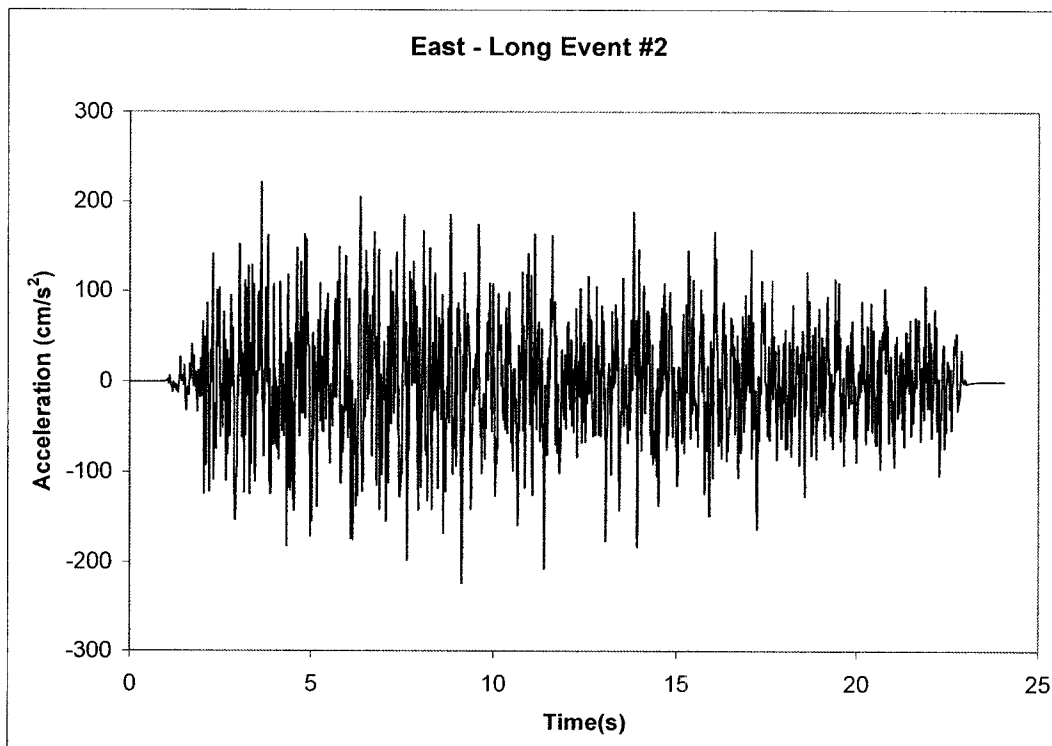
**Figure 3.1: Plan View of the Typical Structure.**



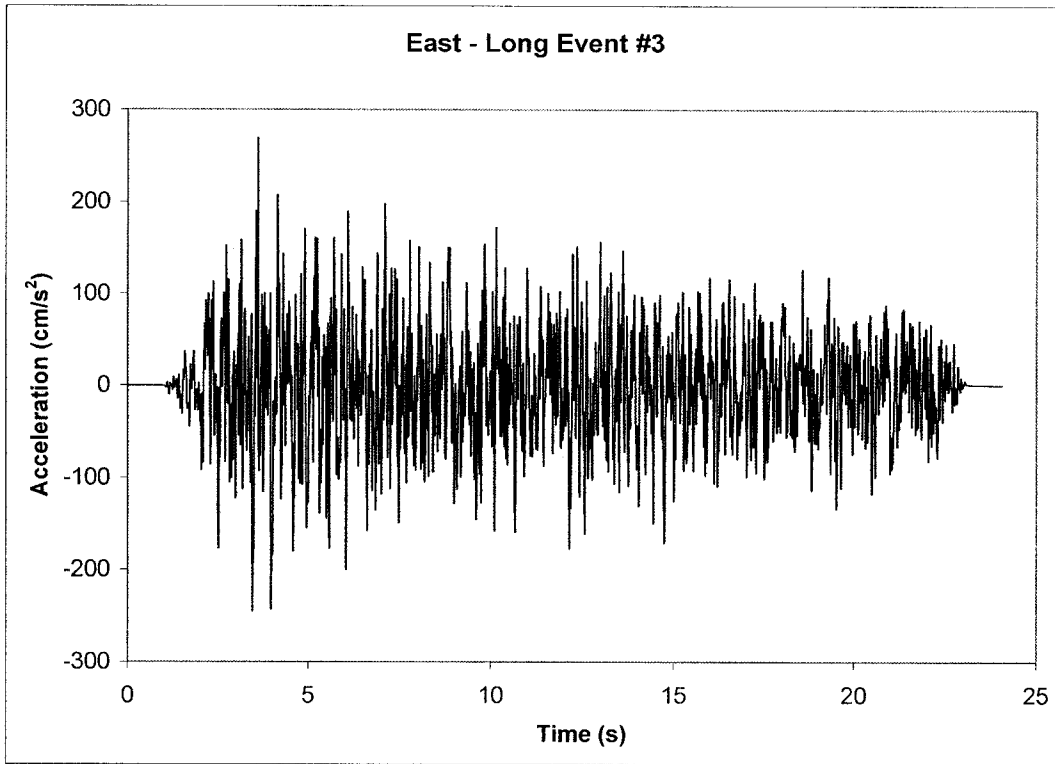
**Figure 3.2: Elevation View of the 5, 10 and 15 Storey Structures**



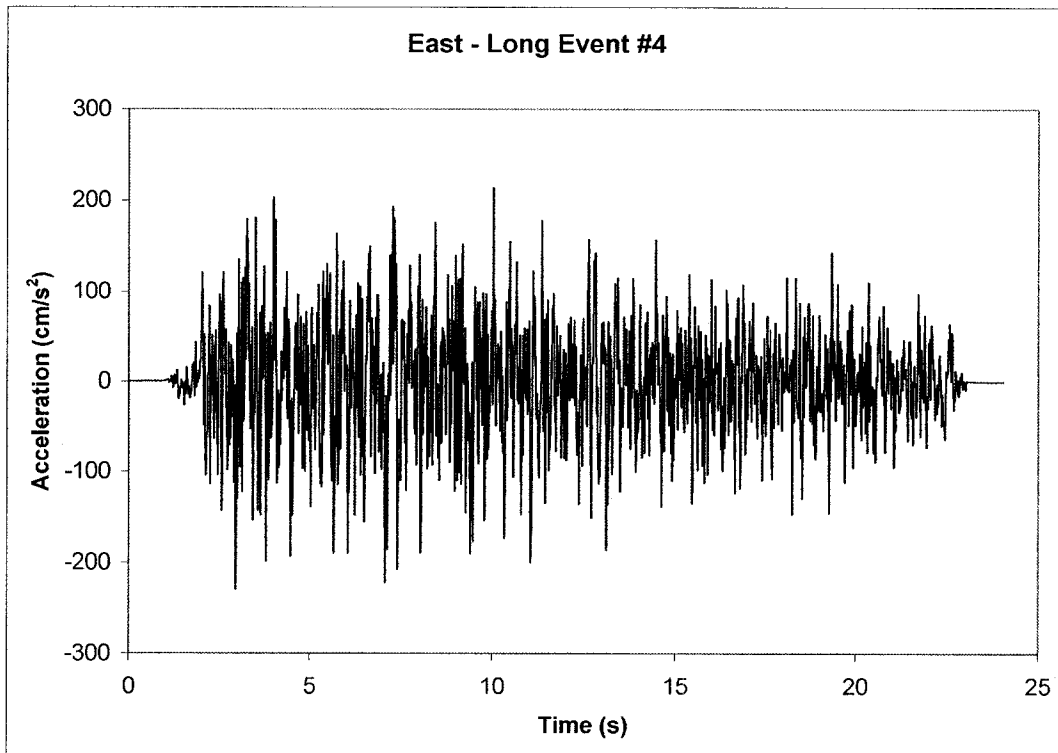
**Figure 4.1: Earthquake Record – Long Event #1 - Ottawa**



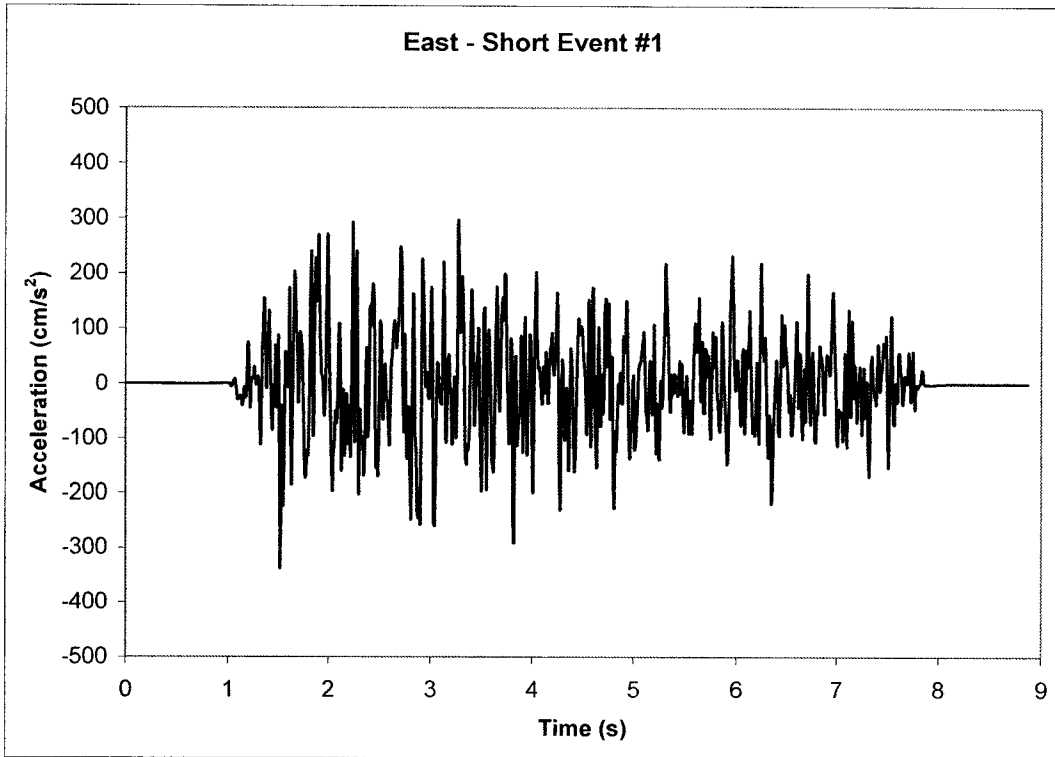
**Figure 4.2: Earthquake Record – Long Event #2 - Ottawa**



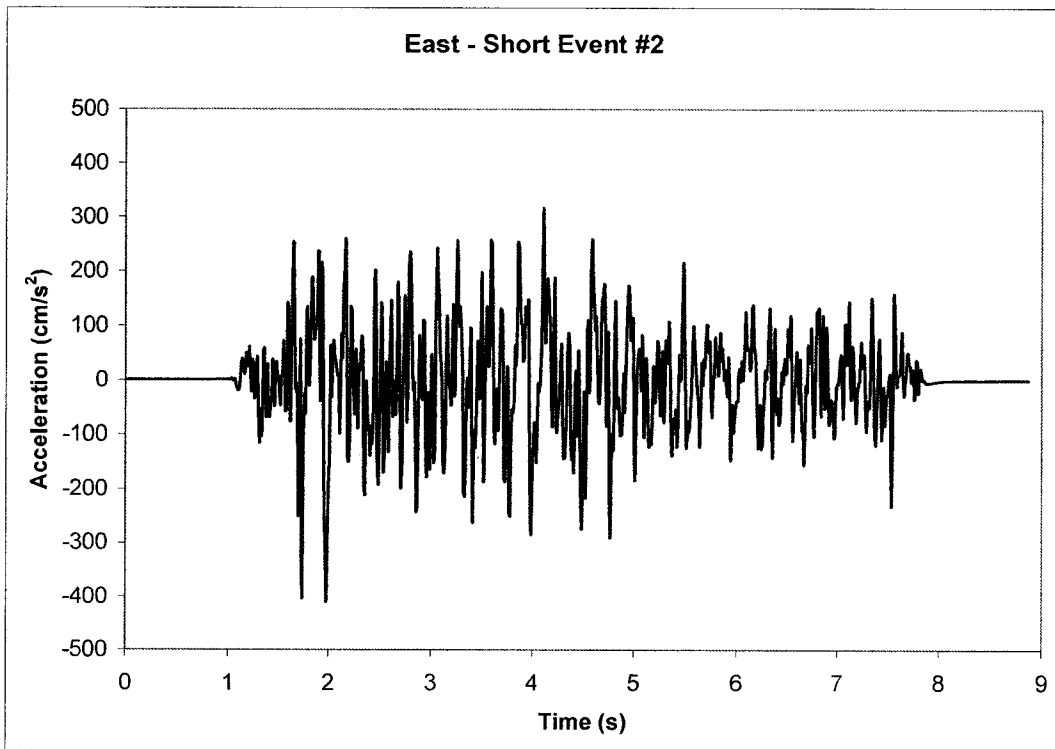
**Figure 4.3: Earthquake Record – Long Event #3 - Ottawa**



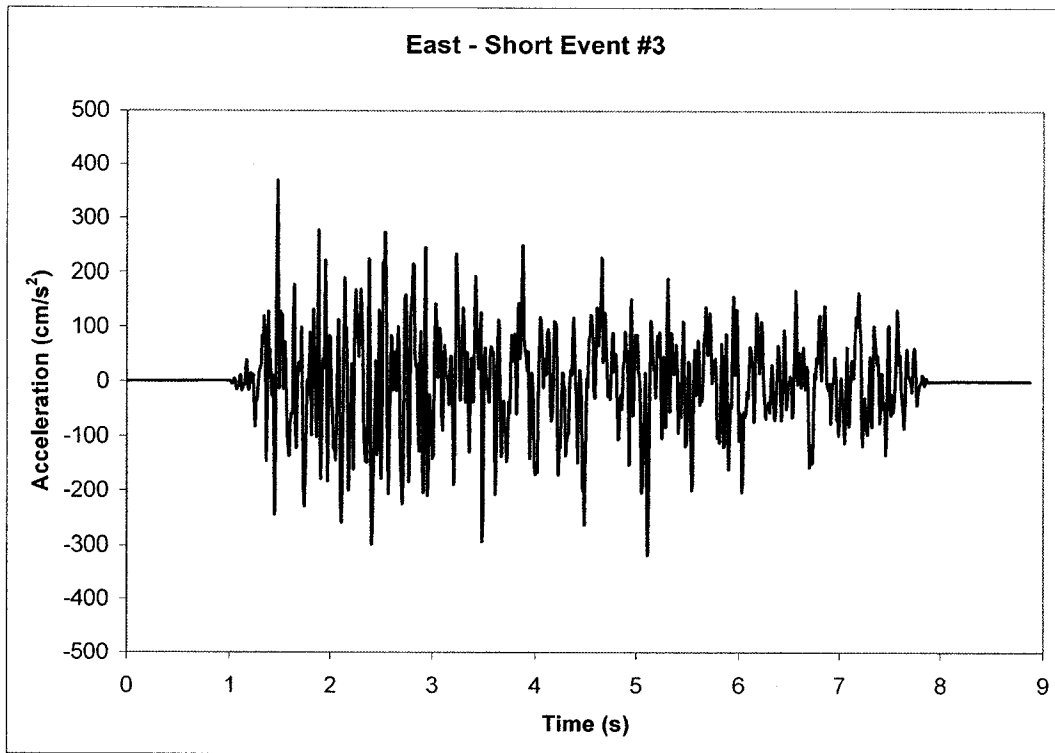
**Figure 4.4: Earthquake Record – Long Event #4 - Ottawa**



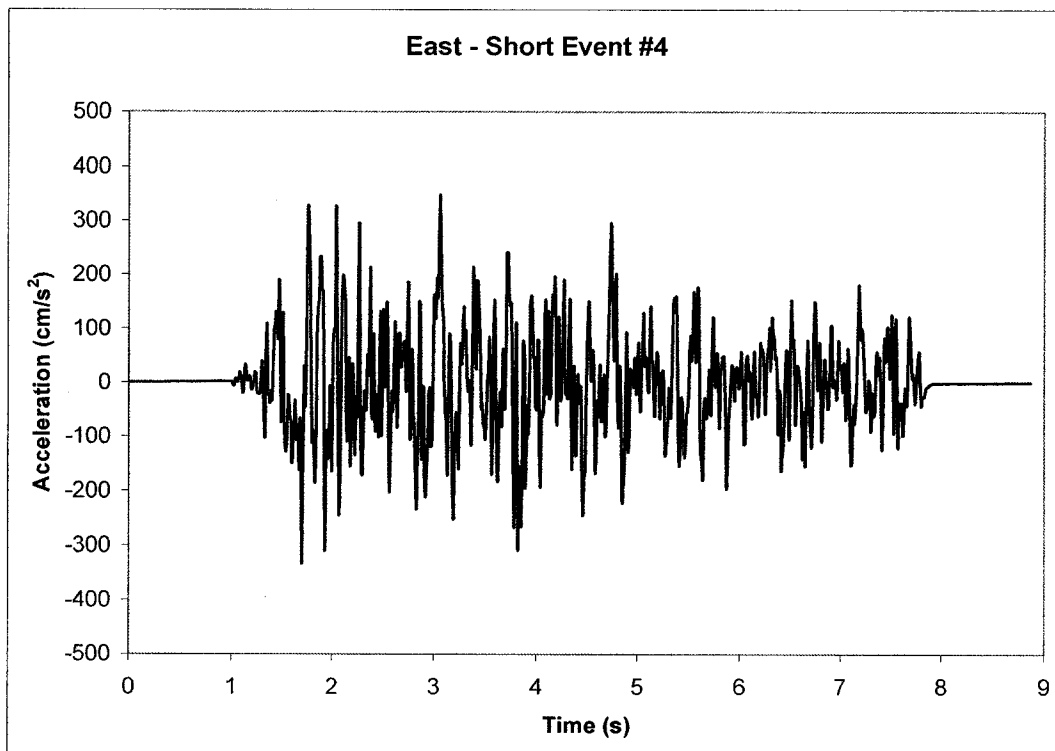
**Figure 4.5: Earthquake Record – Short Event #1 - Ottawa**



**Figure 4.6: Earthquake Record – Short Event #2 - Ottawa**



**Figure 4.7: Earthquake Record – Short Event #3 - Ottawa**



**Figure 4.8: Earthquake Record – Short Event #4 – Ottawa**

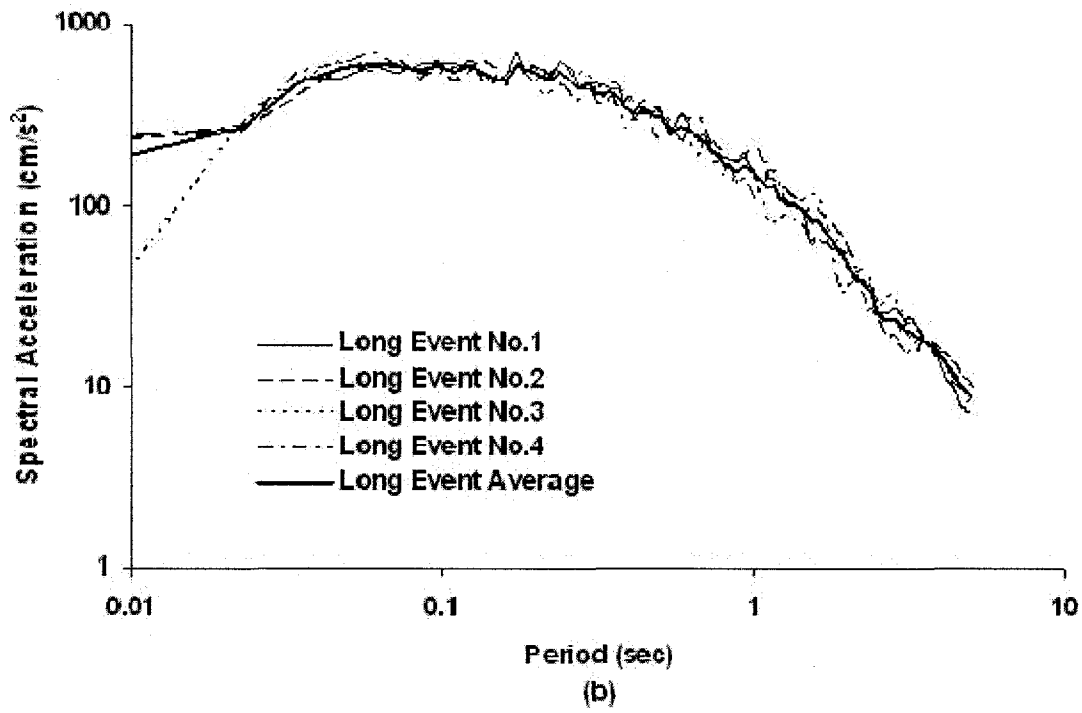


Figure 4.9: Spectral Acceleration – Long Events - Ottawa

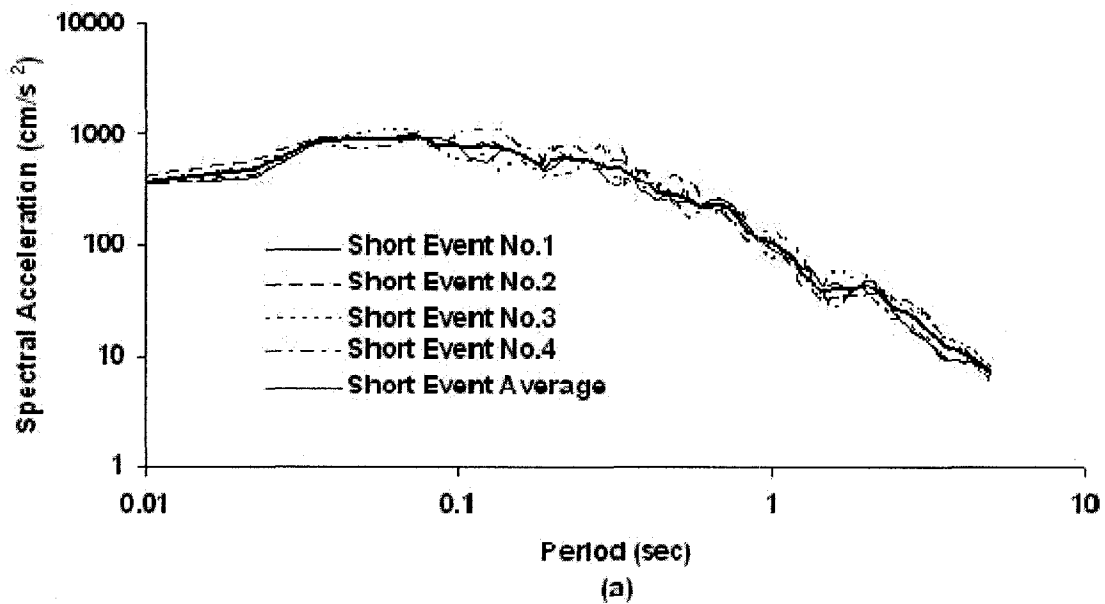
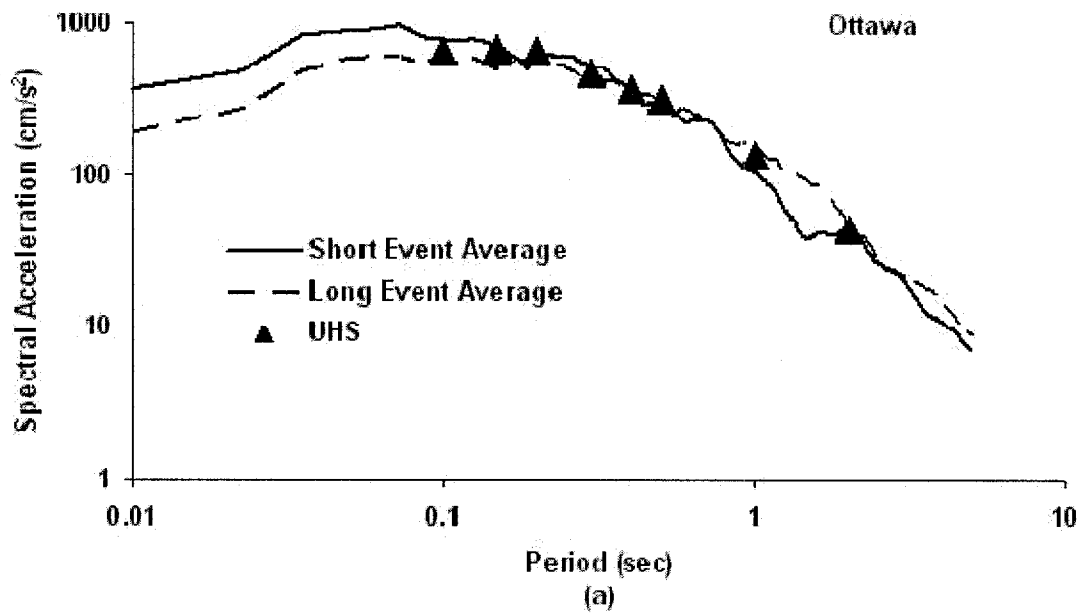
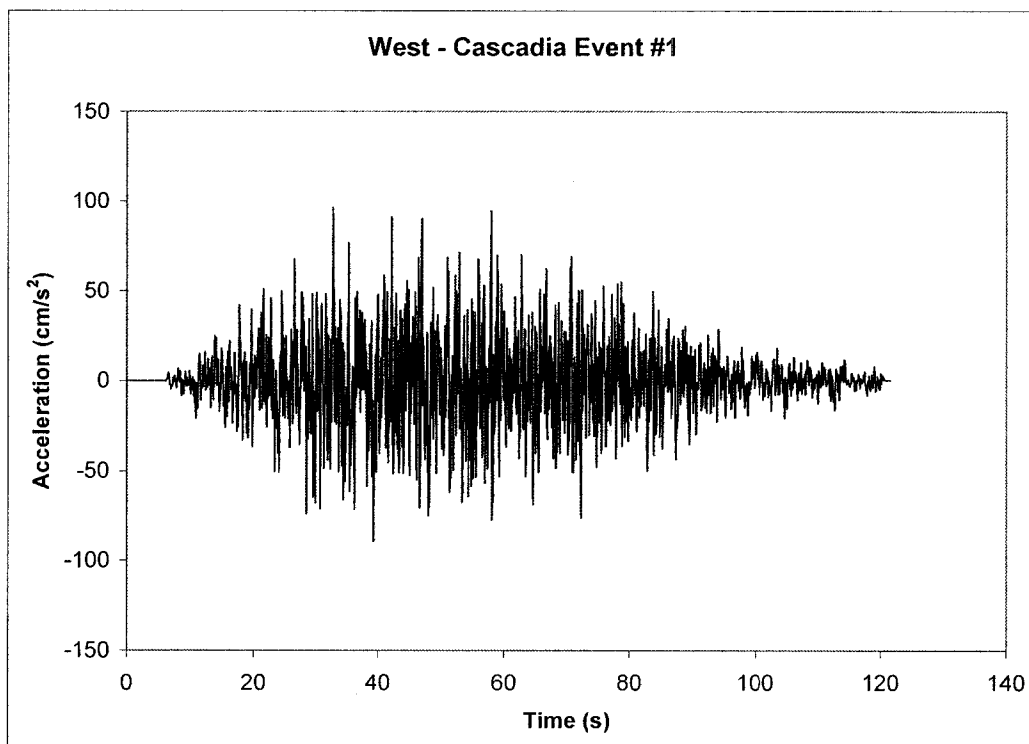


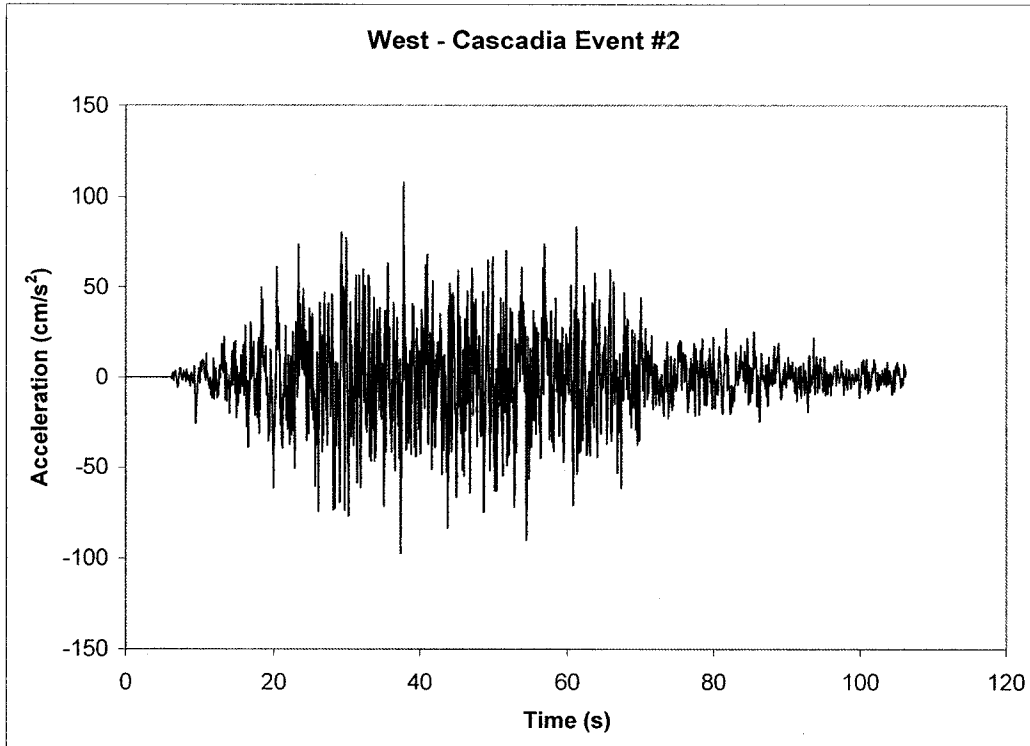
Figure 4.10: Spectral Acceleration – Short Events - Ottawa



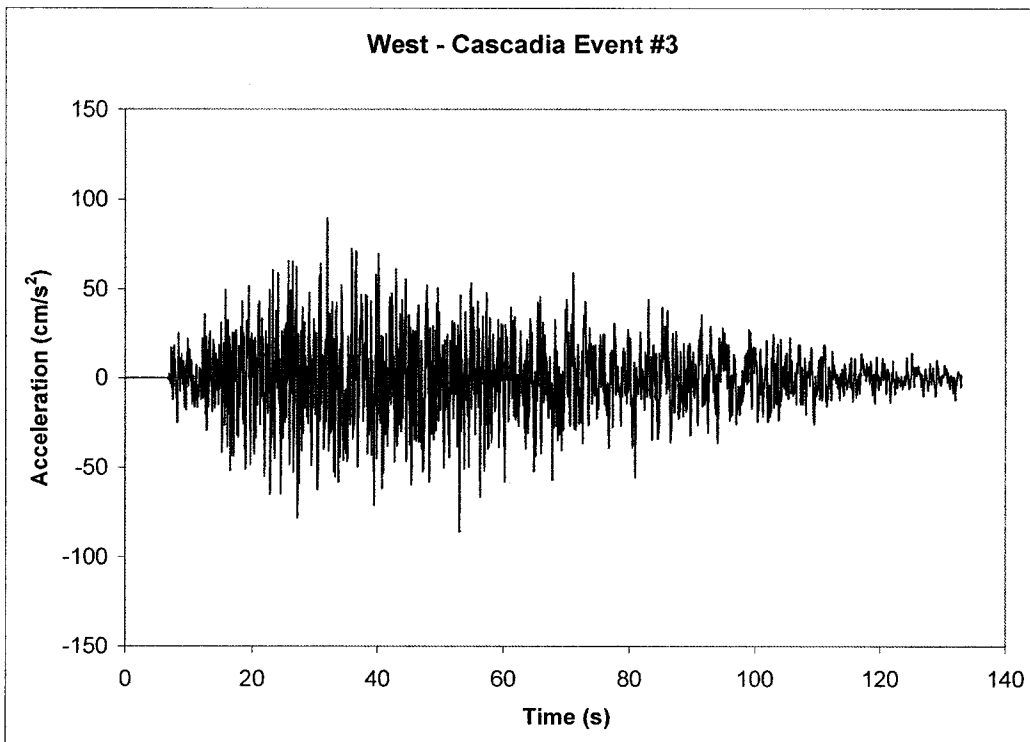
**Figure 4.11: Average Spectral Accelerations - Ottawa**



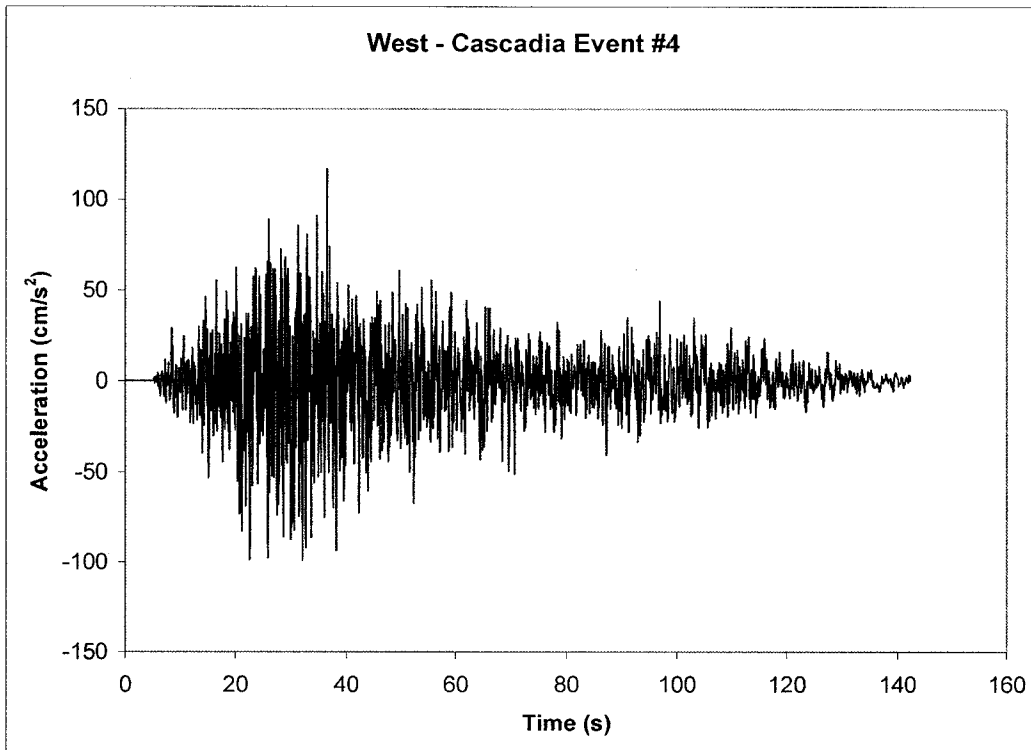
**Figure 4.12: Earthquake Record - Cascadia Event #1 - Vancouver**



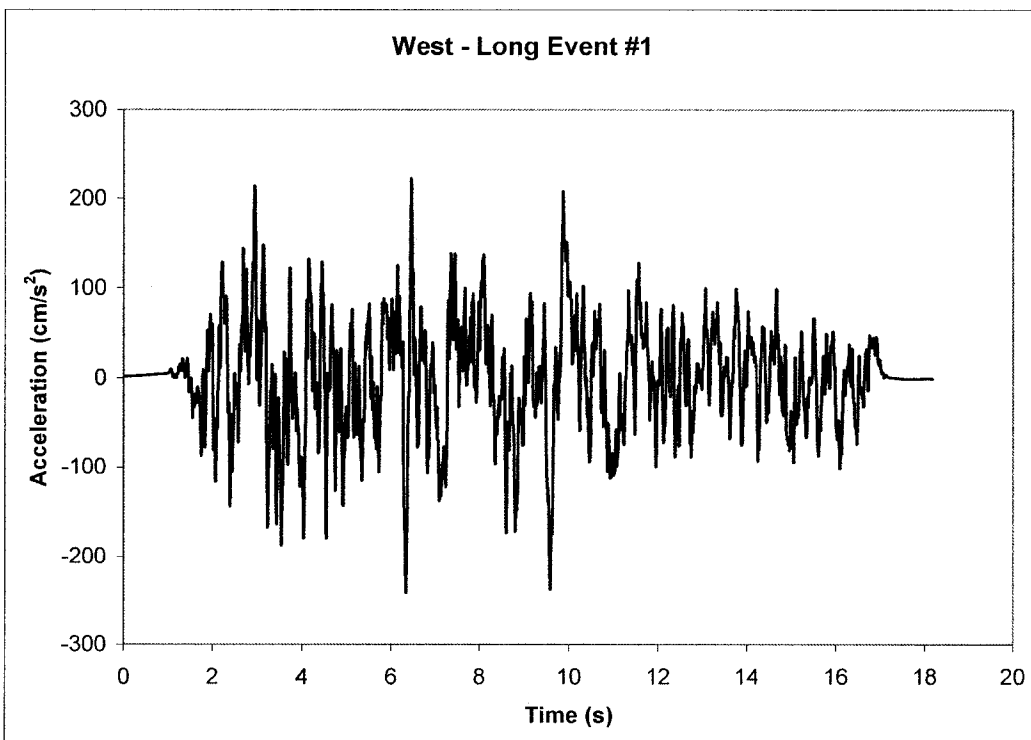
**Figure 4.13: Earthquake Record – Cascadia Event #2 - Vancouver**



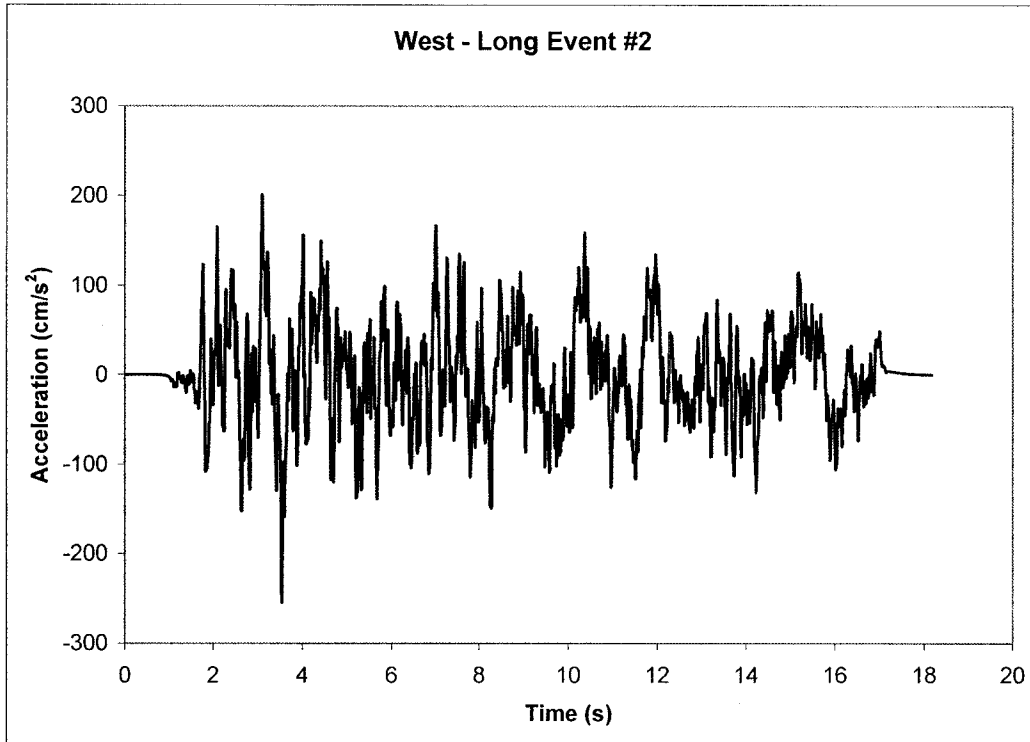
**Figure 4.14: Earthquake Record – Cascadia Event #3 - Vancouver**



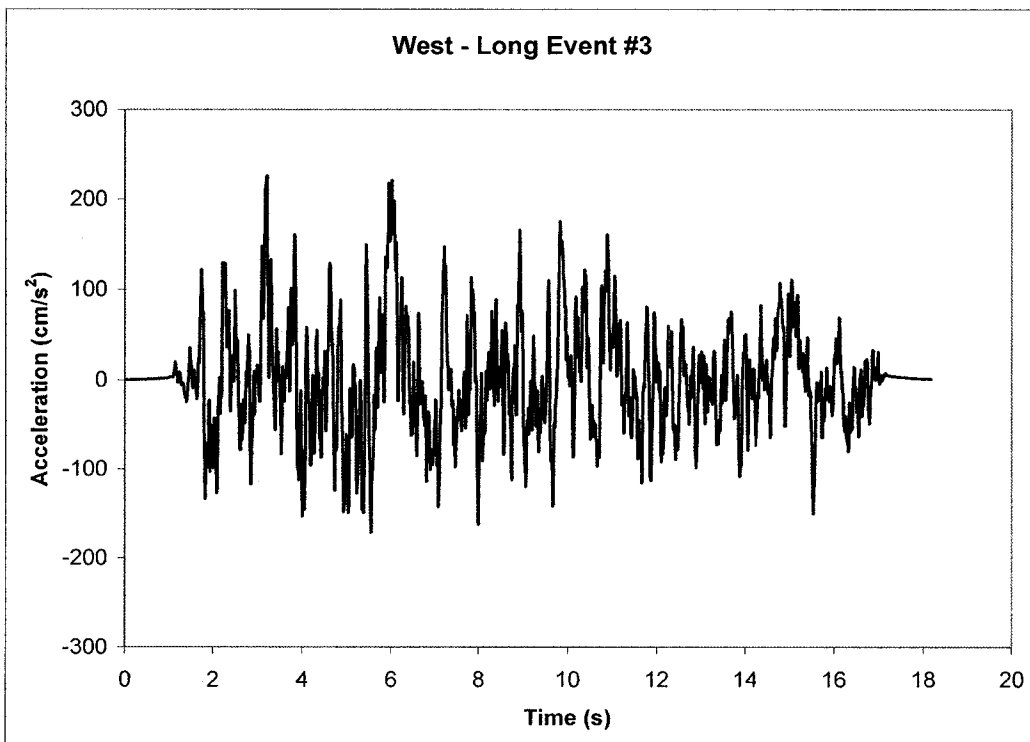
**Figure 4.15: Earthquake Record – Cascadia Event #4 - Vancouver**



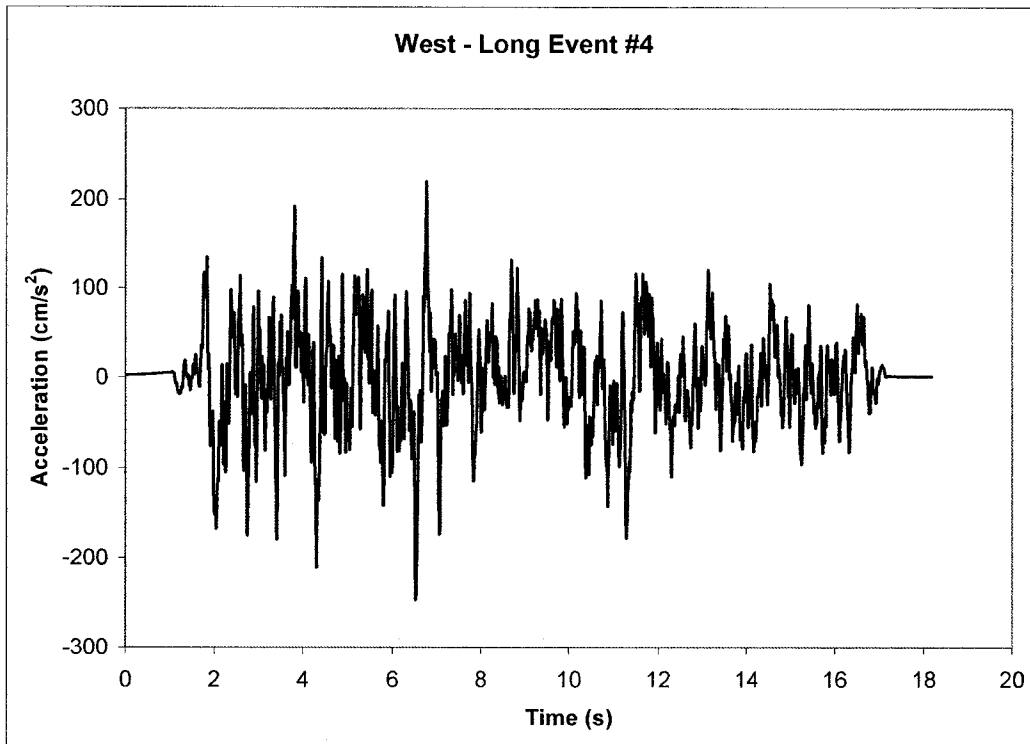
**Figure 4.16: Earthquake Record – Long Event #1 - Vancouver**



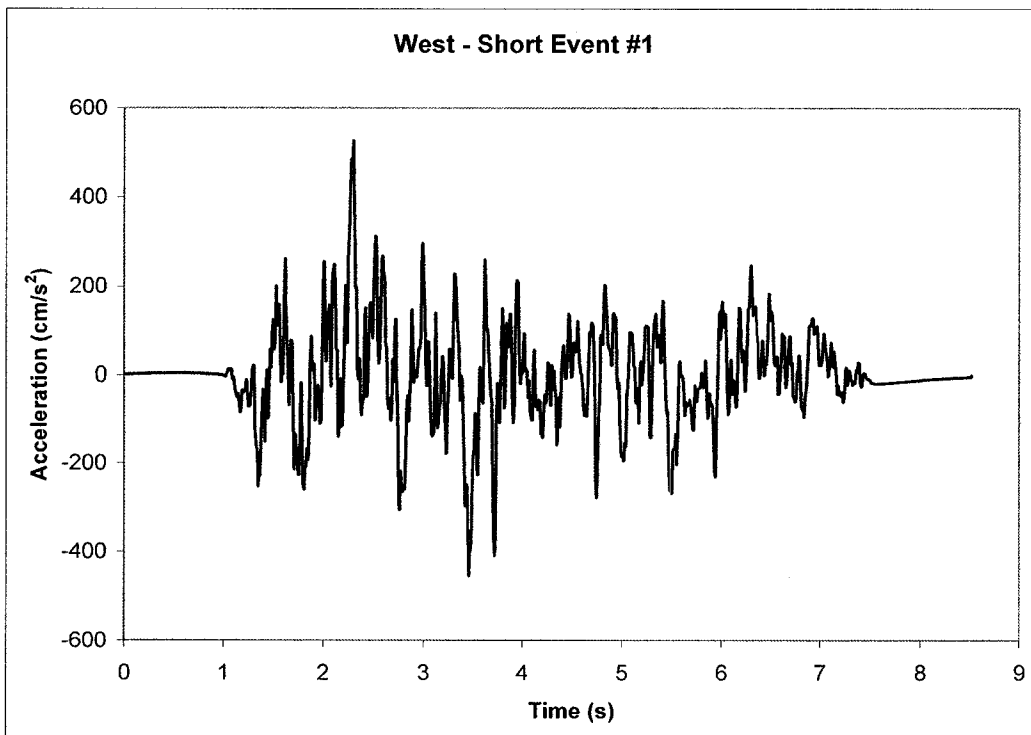
**Figure 4.17: Earthquake Record – Long Event #2 - Vancouver**



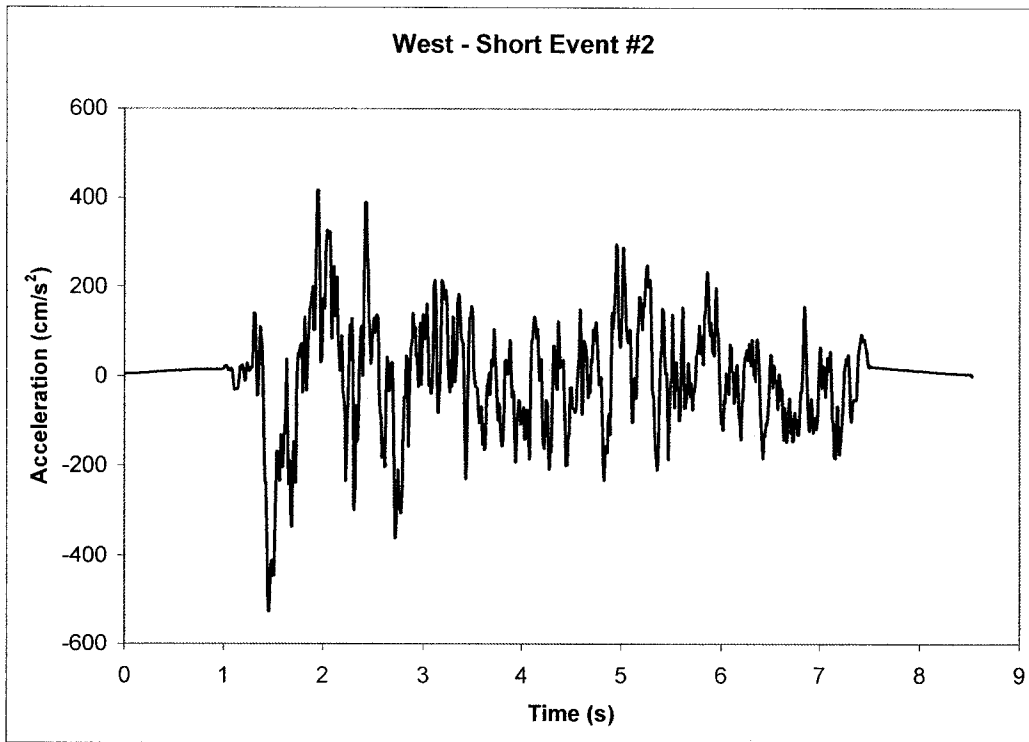
**Figure 4.18: Earthquake Record – Long Event #3 - Vancouver**



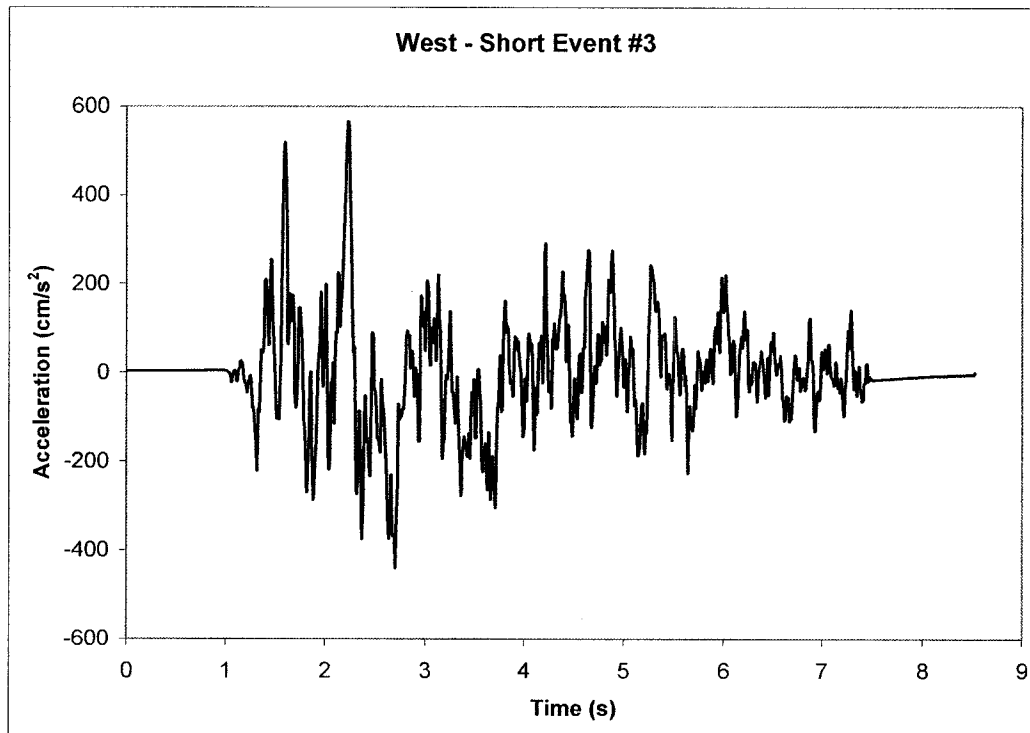
**Figure 4.19: Earthquake Record – Long Event #4 - Vancouver**



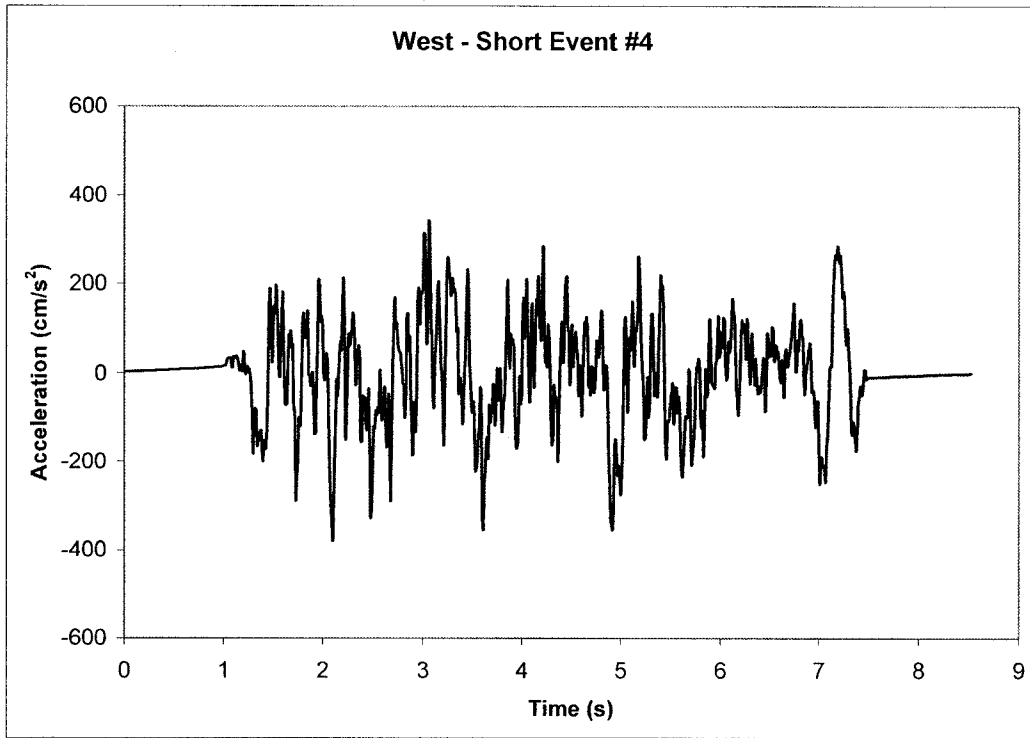
**Figure 4.20: Earthquake Record – Short Event #1 - Vancouver**



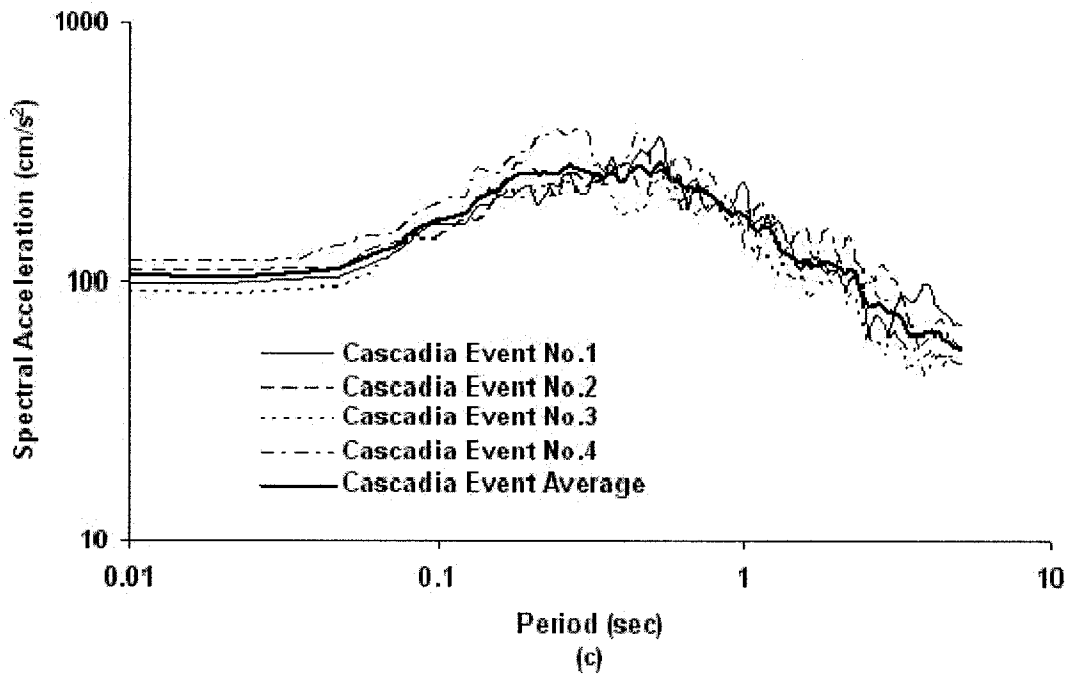
**Figure 4.21: Earthquake Record – Short Event #2 - Vancouver**



**Figure 4.22: Earthquake Record – Short Event #3 - Vancouver**



**Figure 4.23: Earthquake Record – Short Event #4 – Vancouver**



**Figure 4.24: Spectral Acceleration – Cascadia Events - Vancouver**

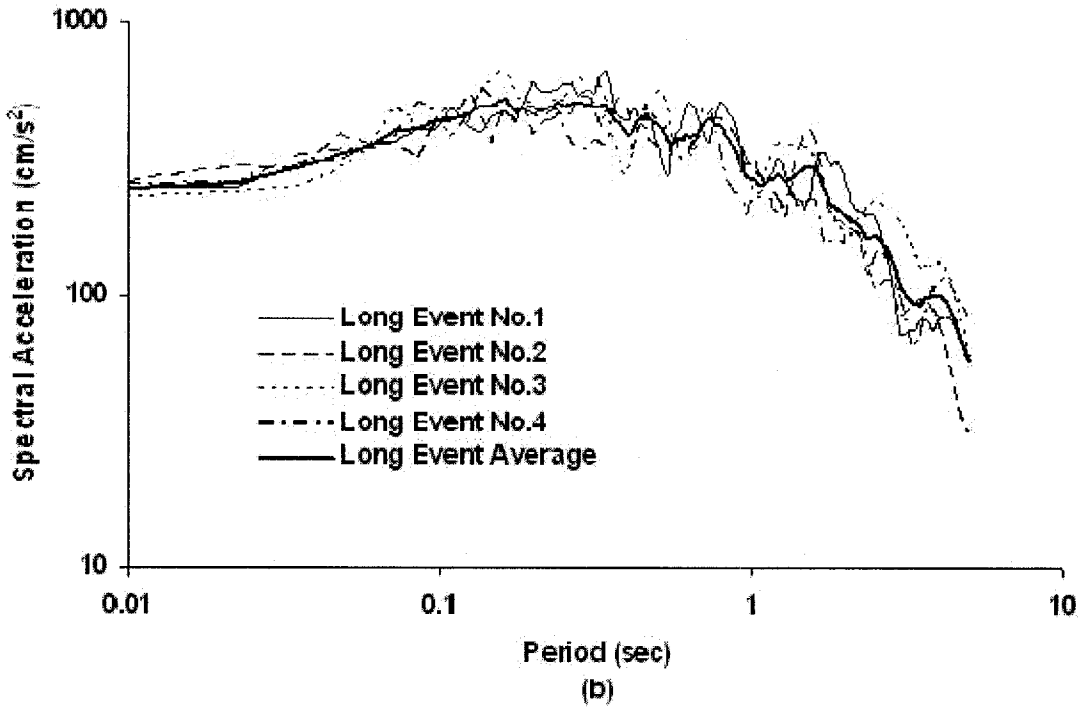


Figure 4.25: Spectral Acceleration – Long Events - Vancouver

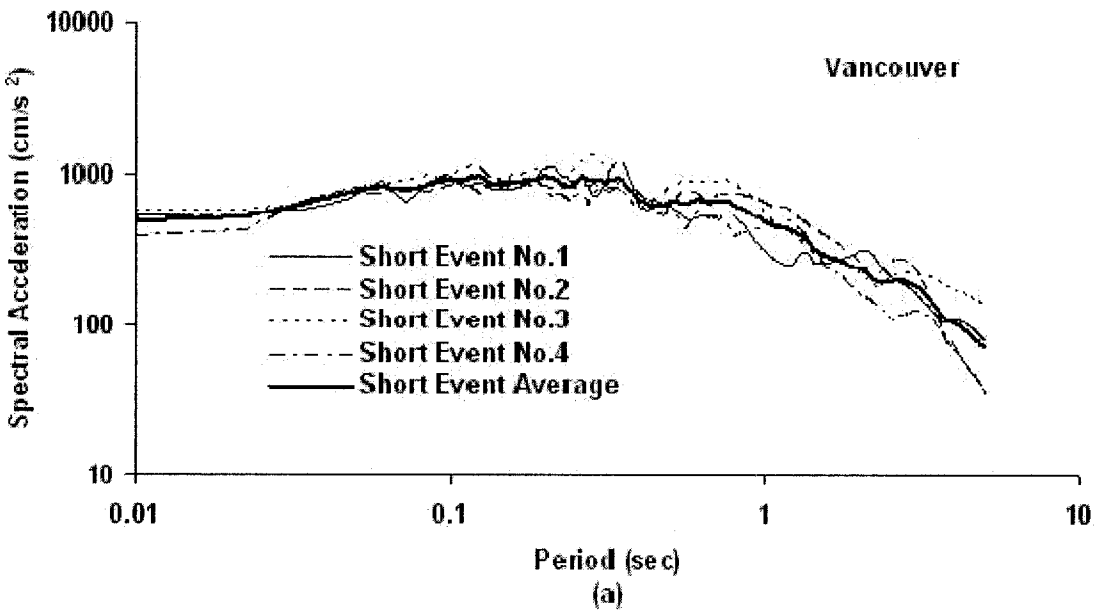


Figure 4.26: Spectral Acceleration – Short Events - Vancouver

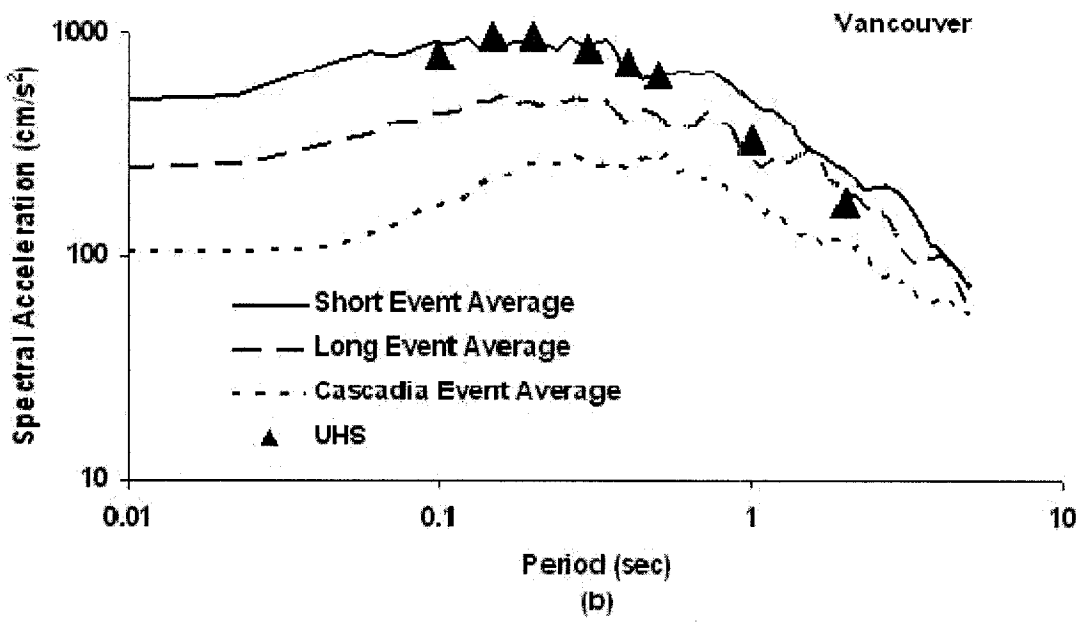
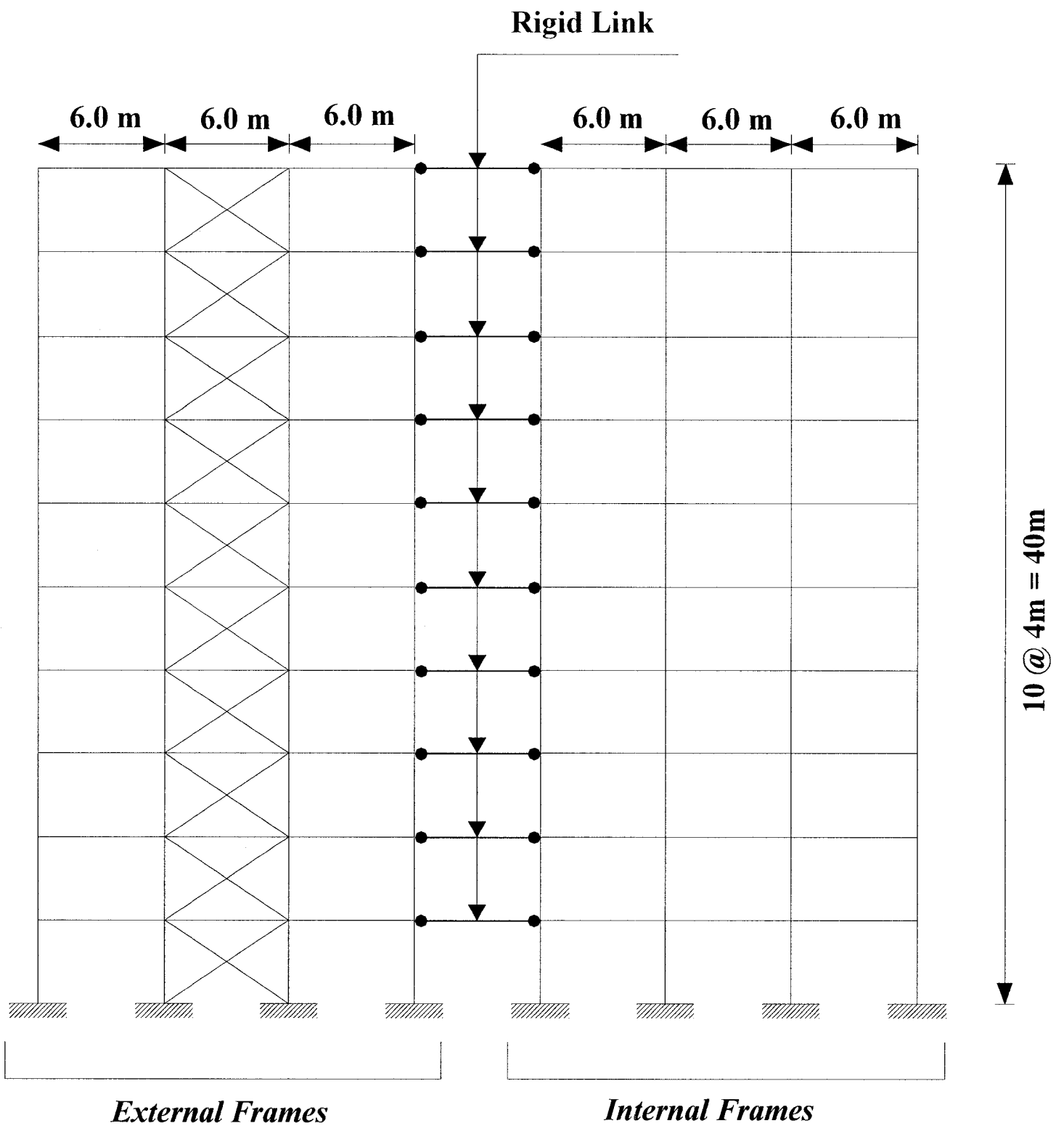
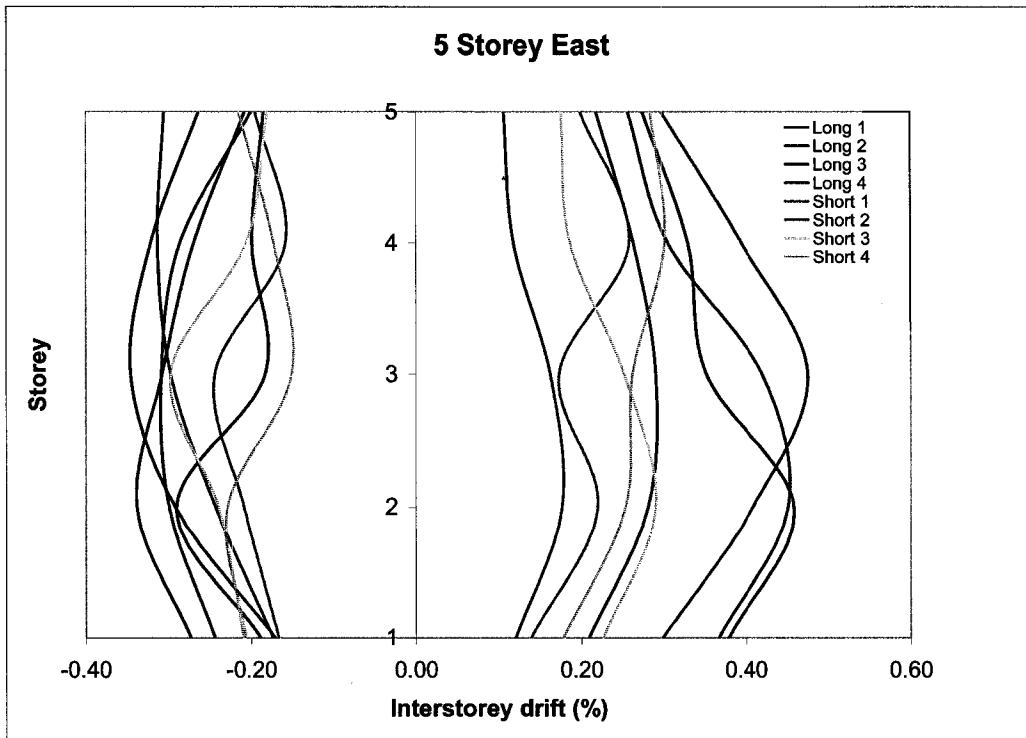


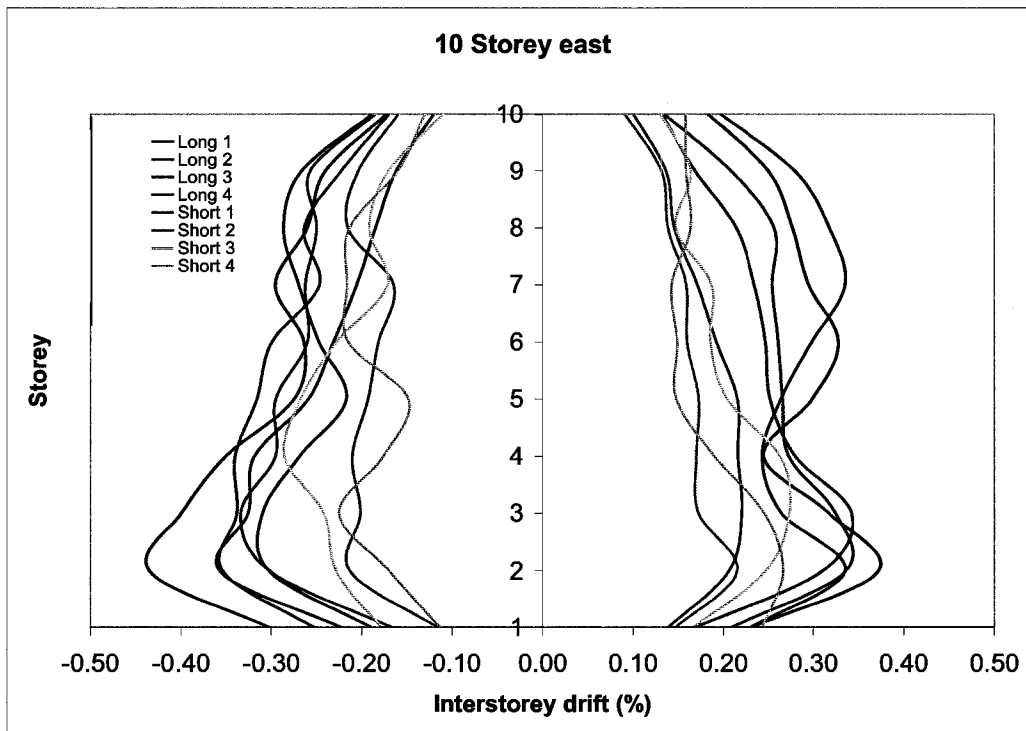
Figure 4.27: Average Spectral Accelerations - Vancouver



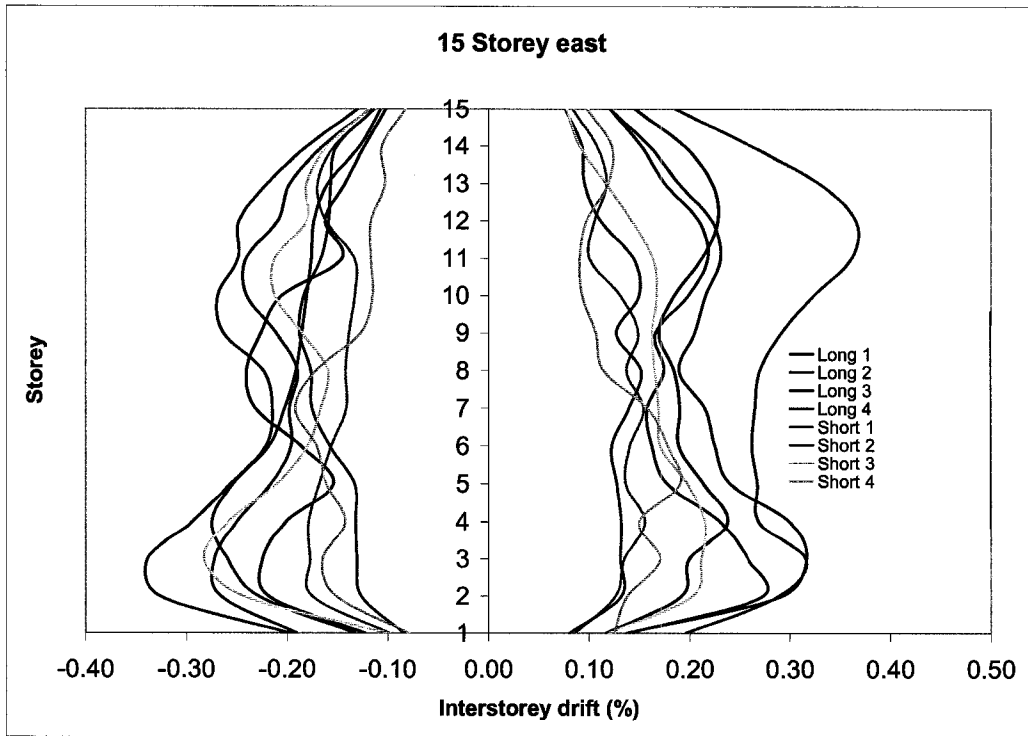
**Figure 4.28: Elevation of the 2 Dimensional Model for a 10 Storey Building**



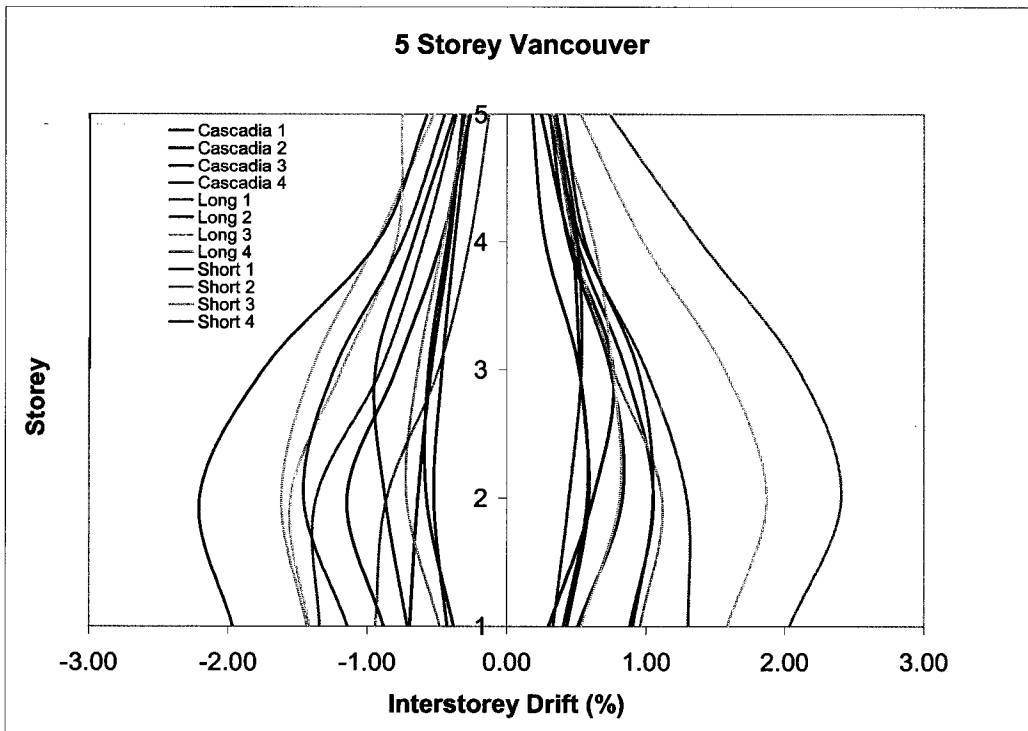
**Figure 4.29: Unretrofitted Interstorey Drift – 5 Storey Ottawa**



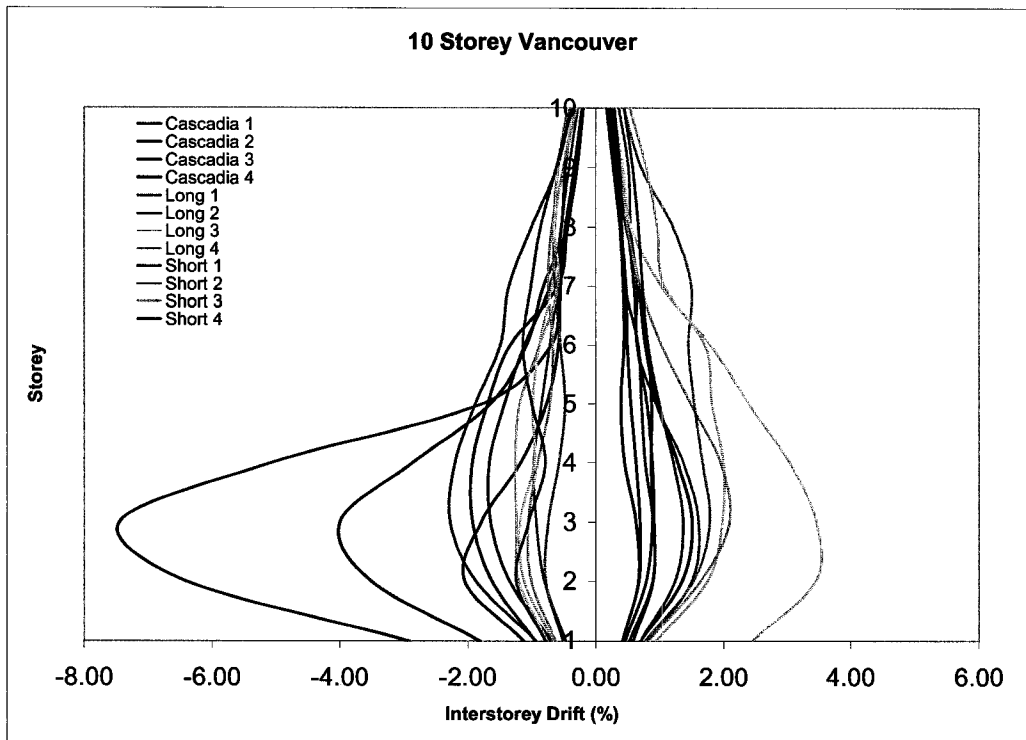
**Figure 4.30: Unretrofitted Interstorey Drift – 10 Storey Ottawa**



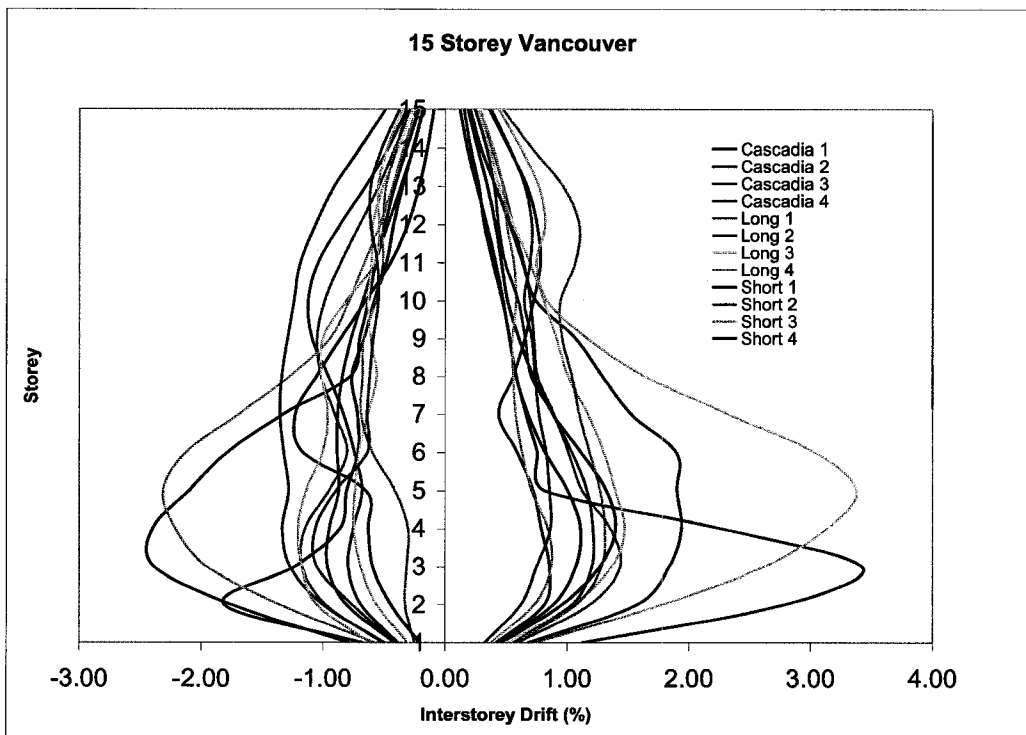
**Figure 4.31: Unretrofitted Interstorey Drift – 15 Storey Ottawa**



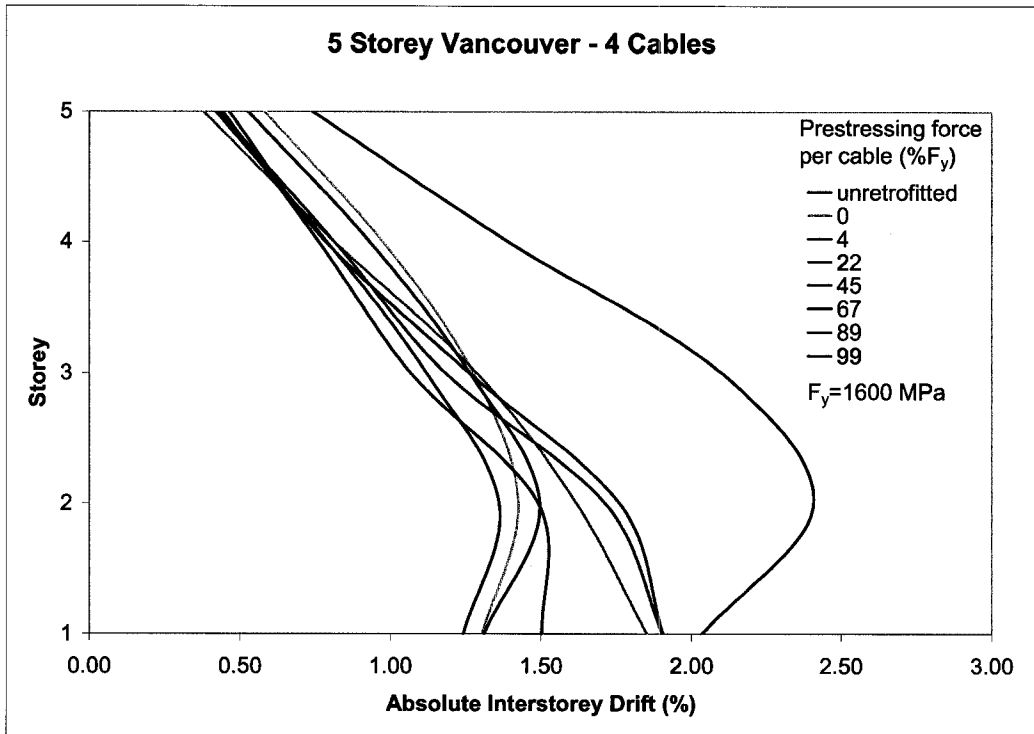
**Figure 4.32: Unretrofitted Interstorey Drift – 5 Storey Vancouver**



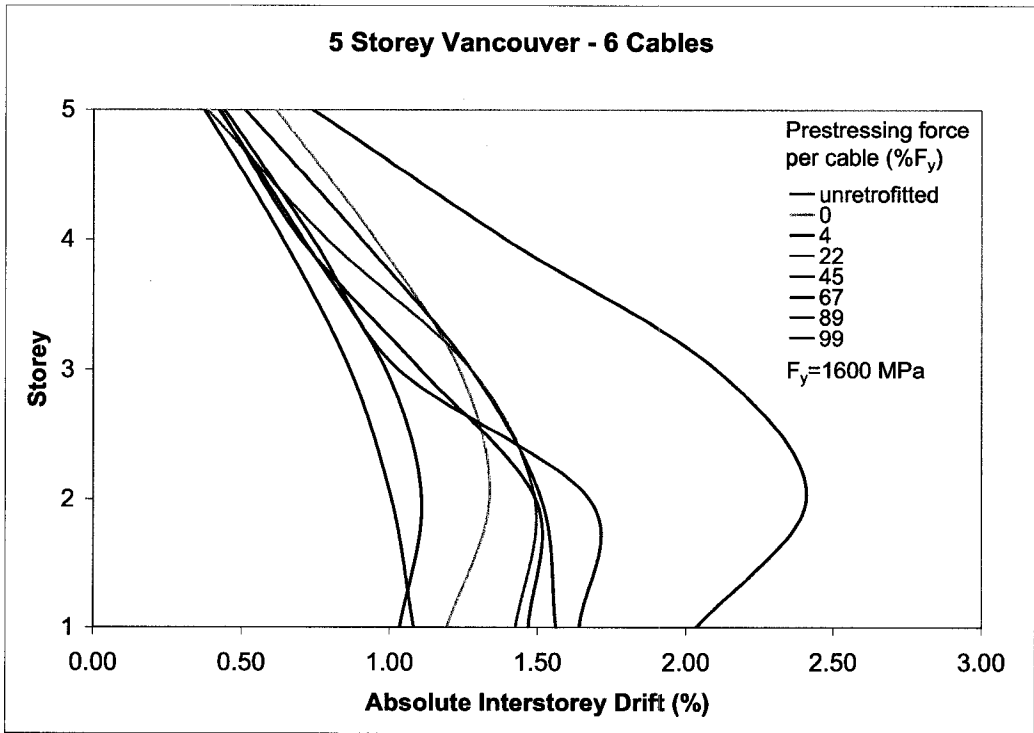
**Figure 4.33: Unretrofitted Interstorey Drift – 10 Storey Vancouver**



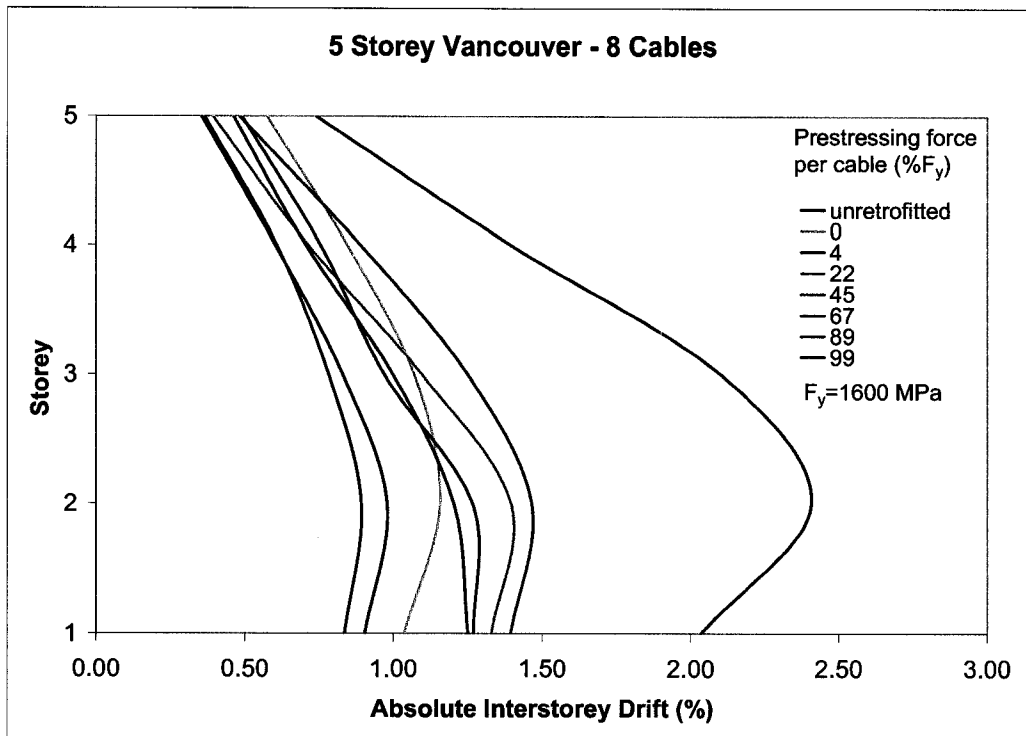
**Figure 4.34: Unretrofitted Interstorey Drift – 15 Storey Vancouver**



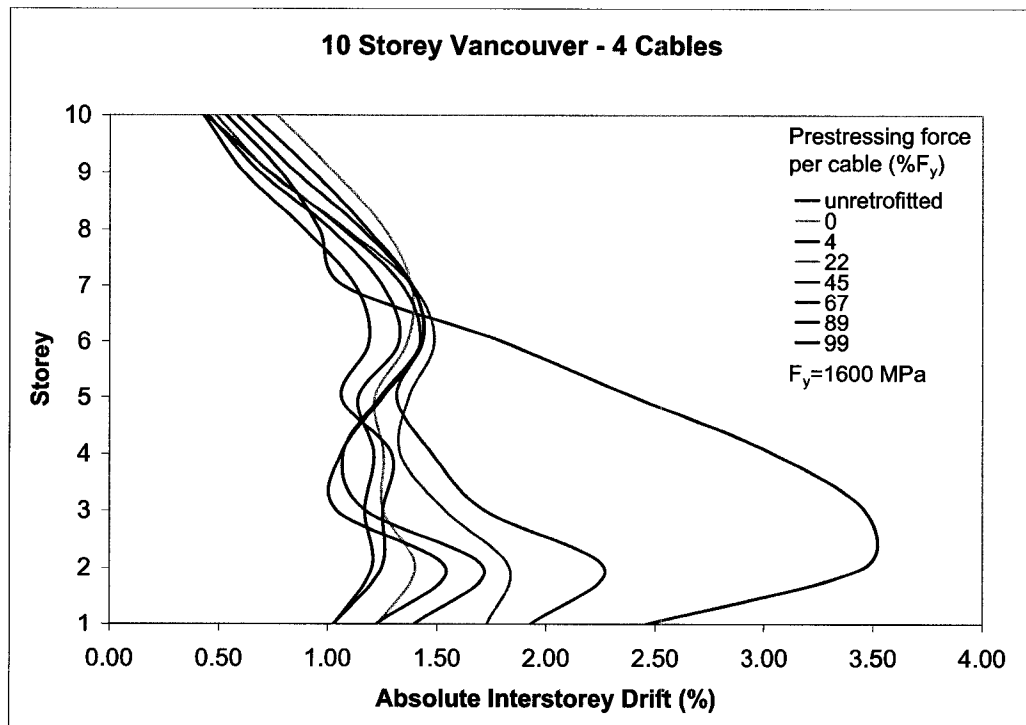
**Figure 4.35: Retrofitted Interstorey Drift – 5 Storey – 4 Cables - Vancouver**



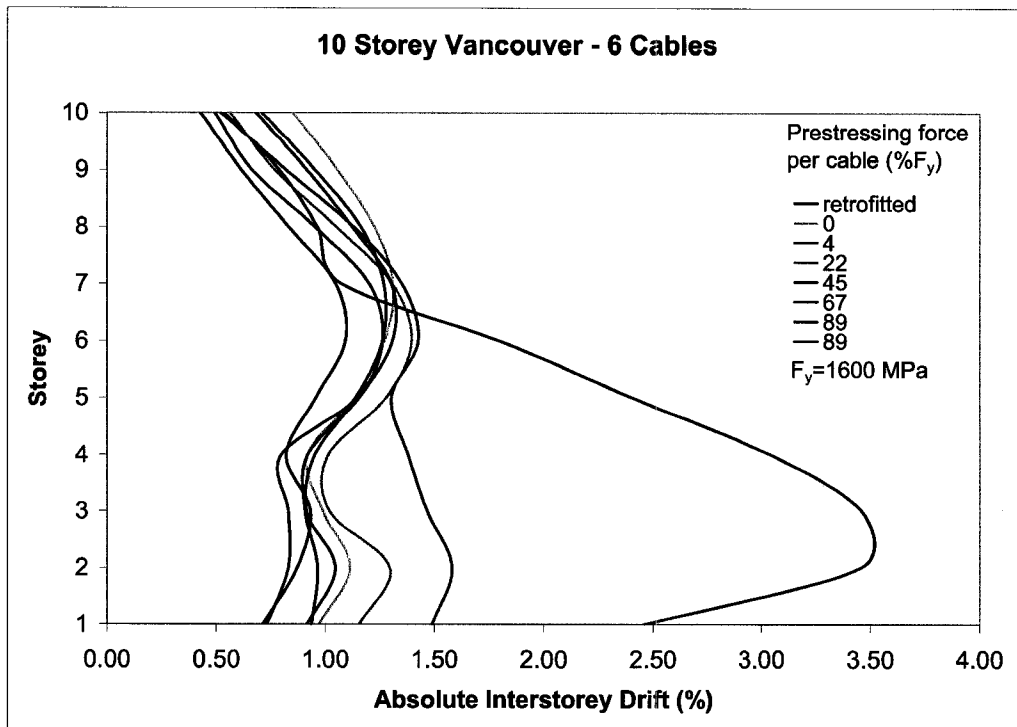
**Figure 4.36: Retrofitted Interstorey Drift – 5 Storey – 6 Cables - Vancouver**



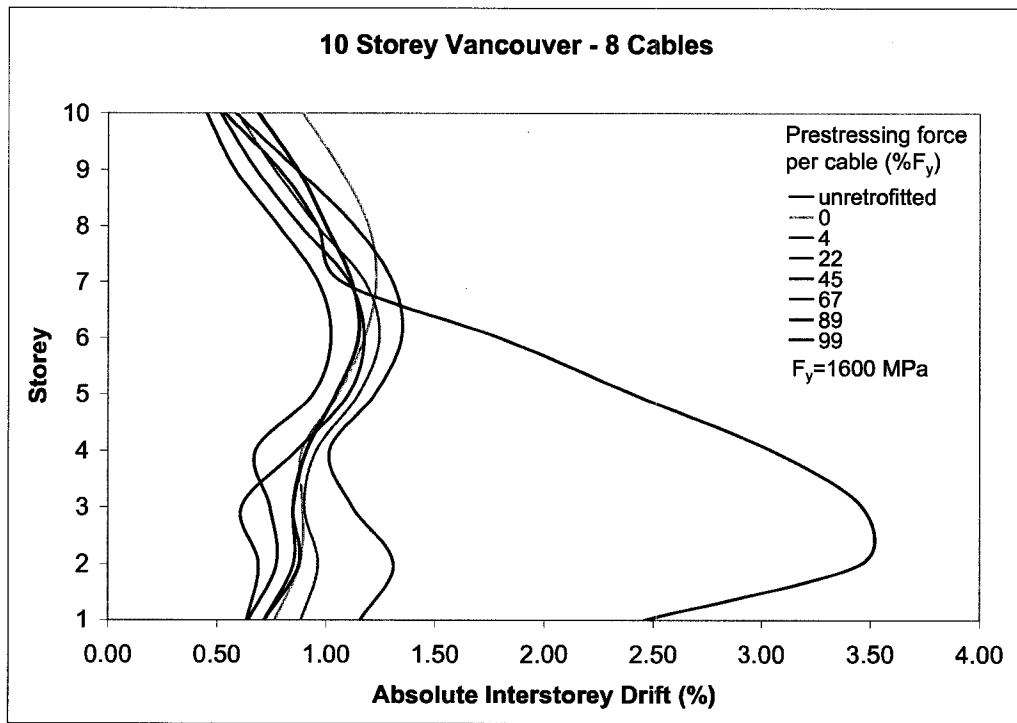
**Figure 4.37: Retrofitted Interstorey Drift – 5 Storey – 8 Cables - Vancouver**



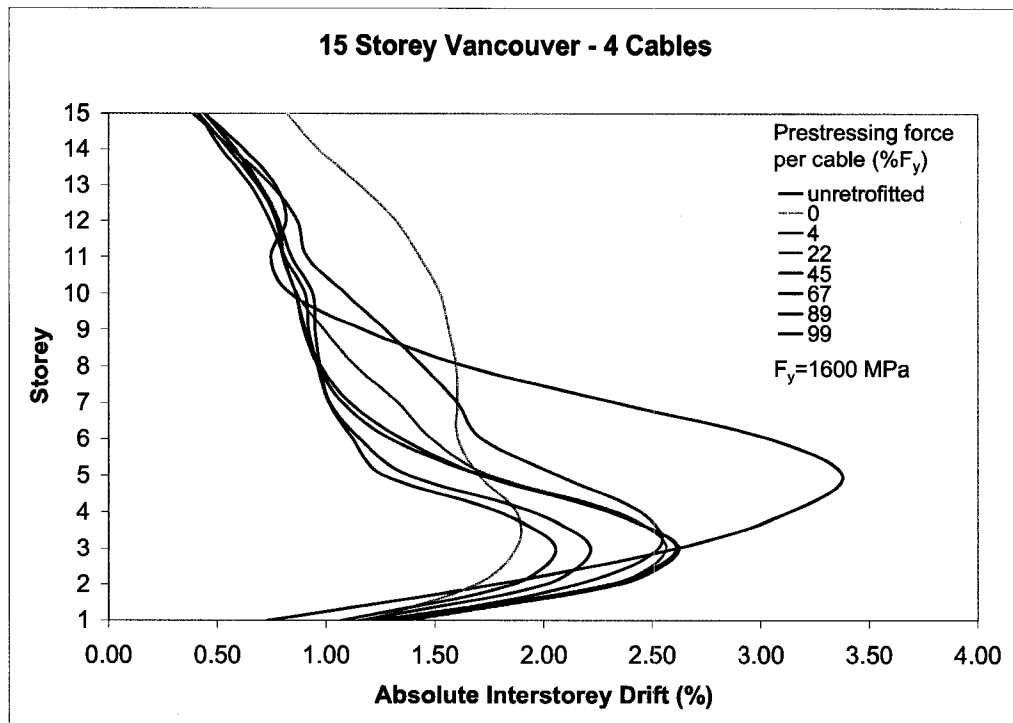
**Figure 4.38: Retrofitted Interstorey Drift – 10 Storey – 4 Cables - Vancouver**



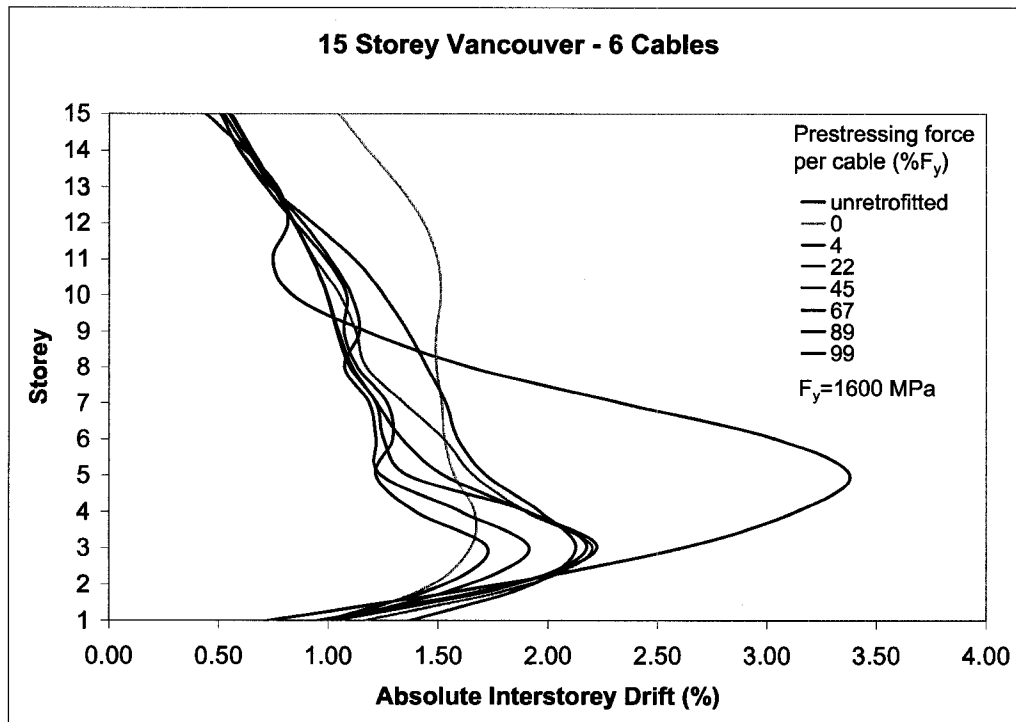
**Figure 4.39: Retrofitted Interstorey Drift – 10 Storey – 6 Cables - Vancouver**



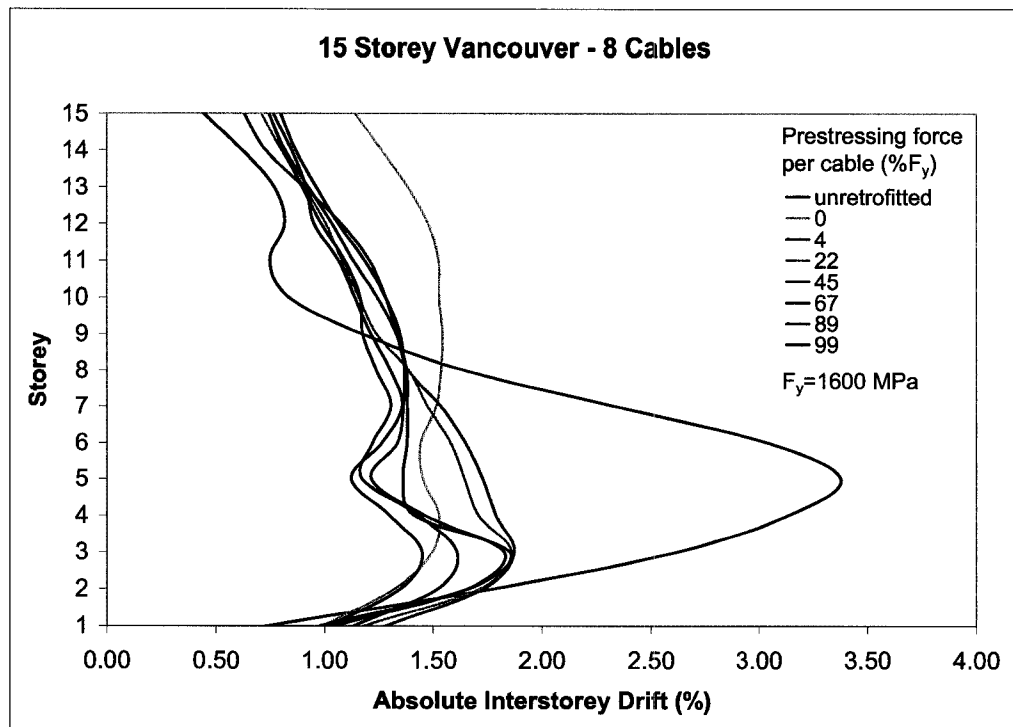
**Figure 4.40: Retrofitted Interstorey Drift – 10 Storey – 8 Cables - Vancouver**



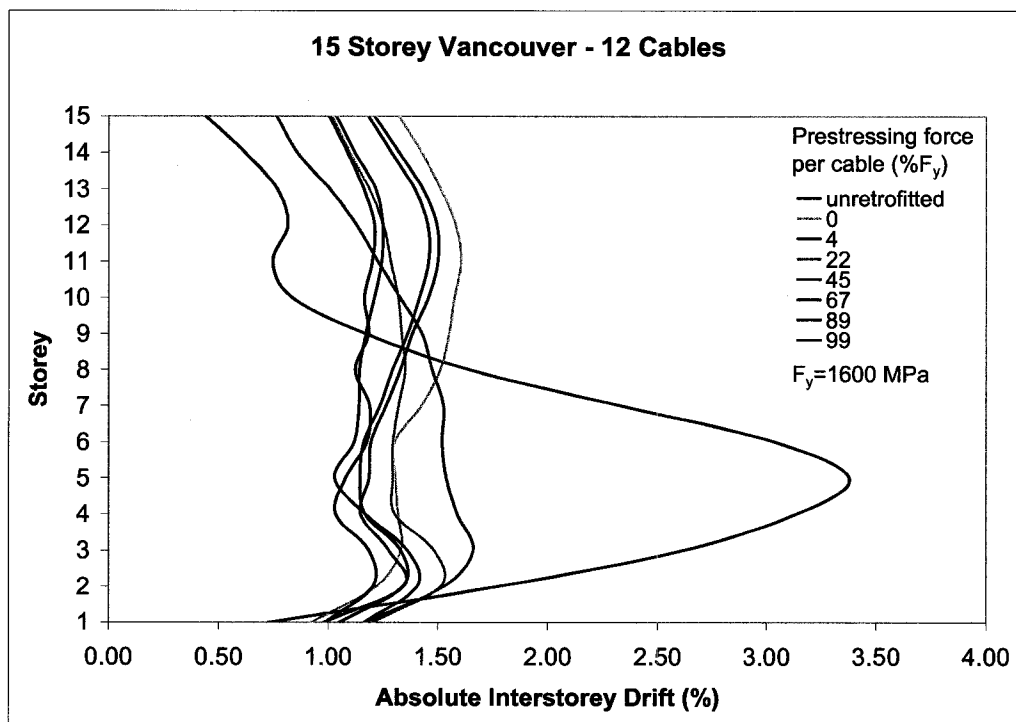
**Figure 4.41: Retrofitted Interstorey Drift – 15 Storey – 4 Cables - Vancouver**



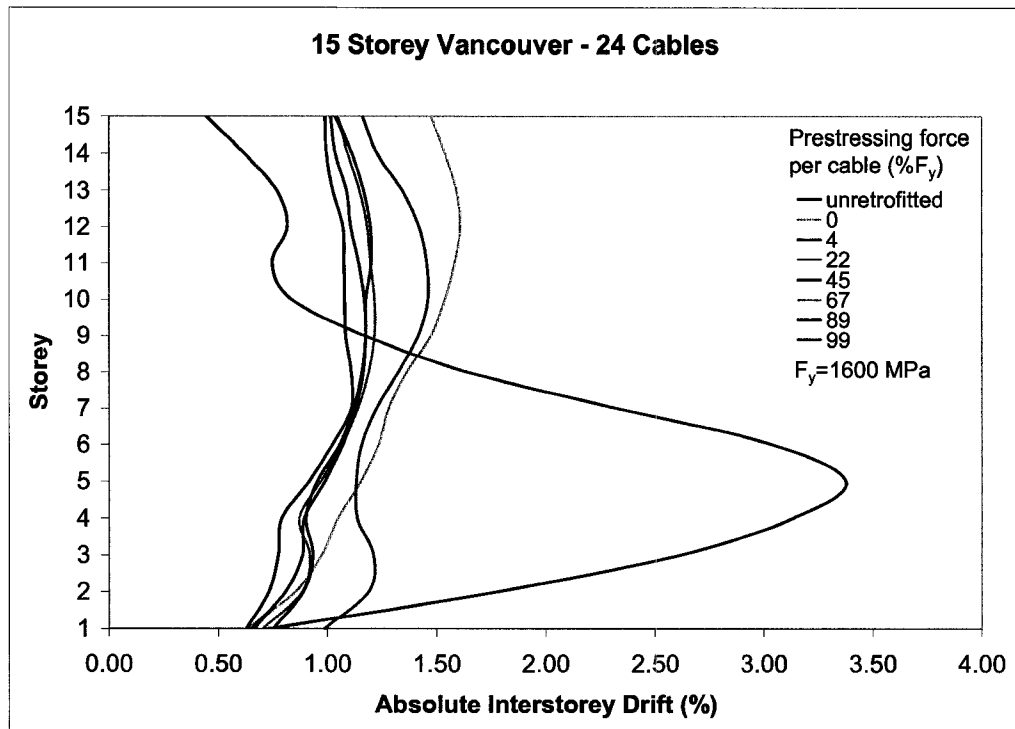
**Figure 4.42: Retrofitted Interstorey Drift – 15 Storey – 6 Cables - Vancouver**



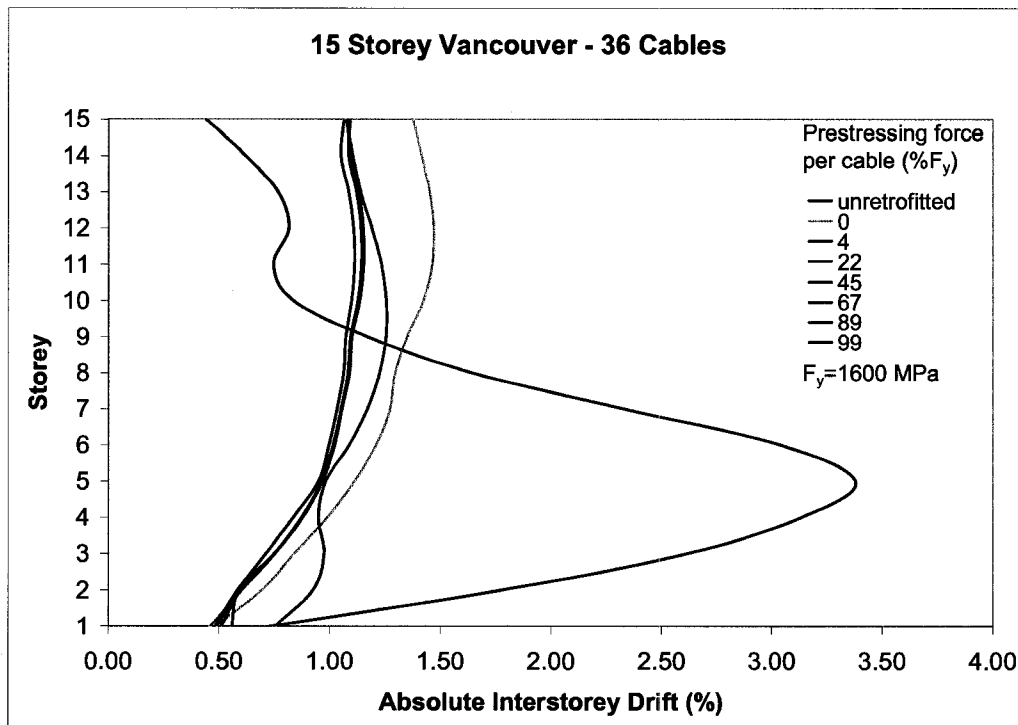
**Figure 4.43: Retrofitted Interstorey Drift – 15 Storey – 8 Cables - Vancouver**



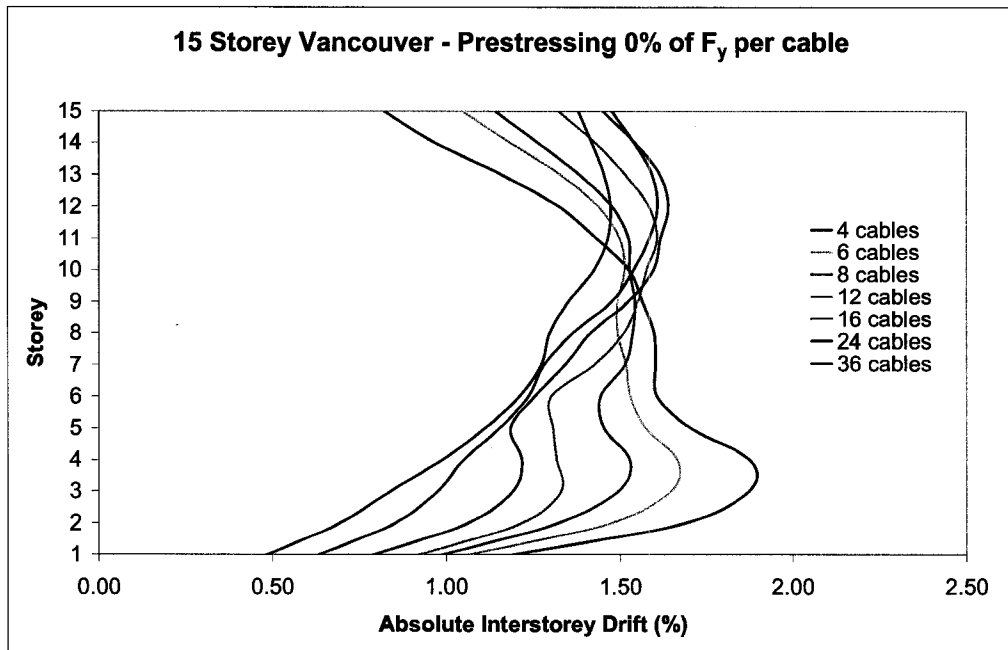
**Figure 4.44: Retrofitted Interstorey Drift – 15 Storey – 12 Cables - Vancouver**



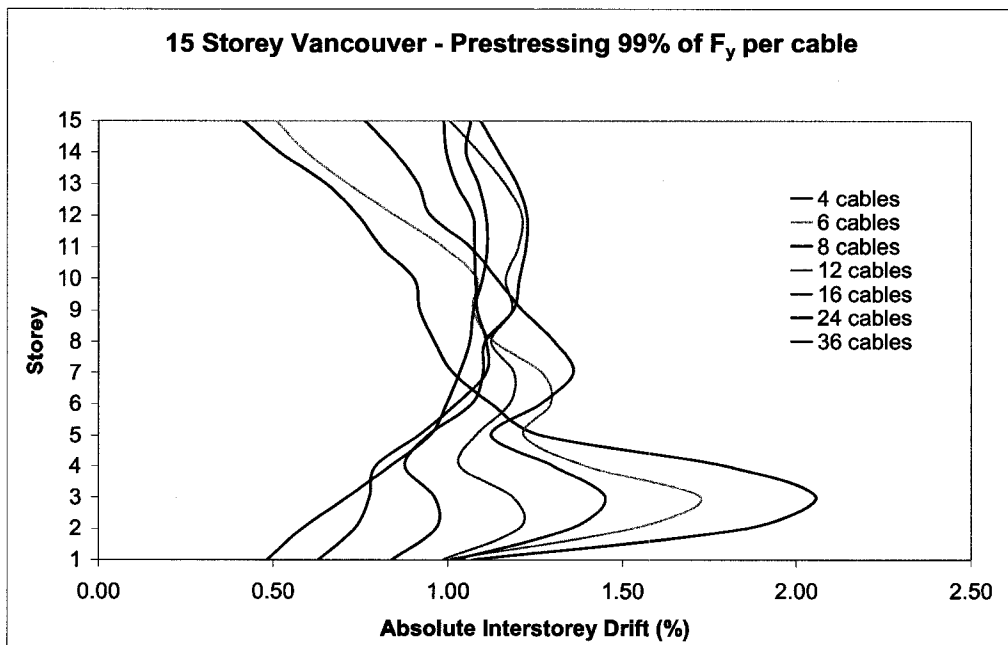
**Figure 4.45: Retrofitted Interstorey Drift – 15 Storey – 24 Cables - Vancouver**



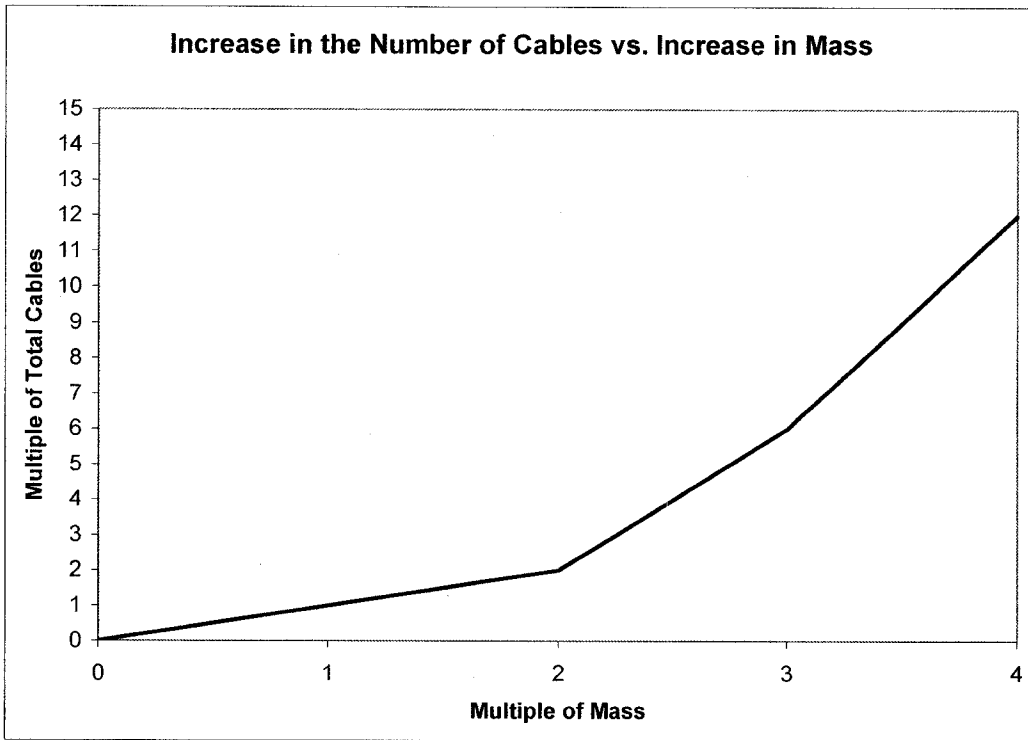
**Figure 4.46: Retrofitted Interstorey Drift – 15 Storey – 36 Cables - Vancouver**



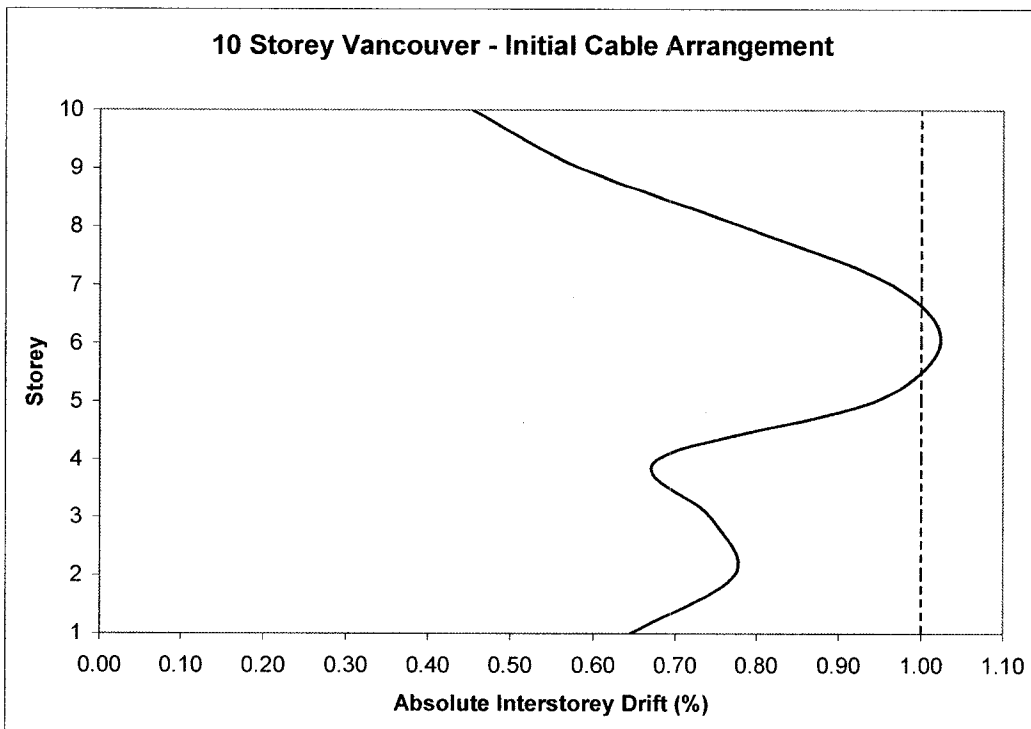
**Figure 4.47: Retrofitted Lateral Drift with Varying Cable Arrangement - Zero Prestress**



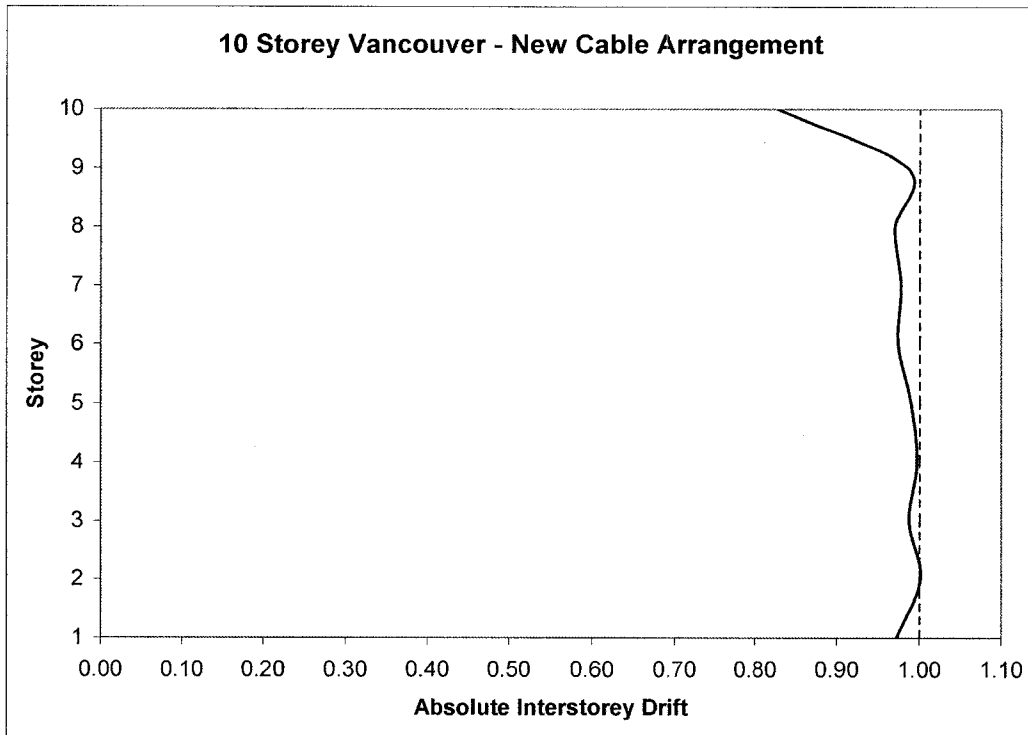
**Figure 4.48: Retrofitted Lateral Drift with Varying Cable Arrangement - 99% Prestress**



**Figure 4.49: Effect of Building Mass on Number of Cables – 5 Storey – Vancouver**



**Figure 4.50: Retrofitted Drift – Initial Cable Arrangement – 10 Storey – Vancouver**



**Figure 4.51: Retrofitted Drift – Efficient Cable Arrangement – 10 Storey – Vancouver**

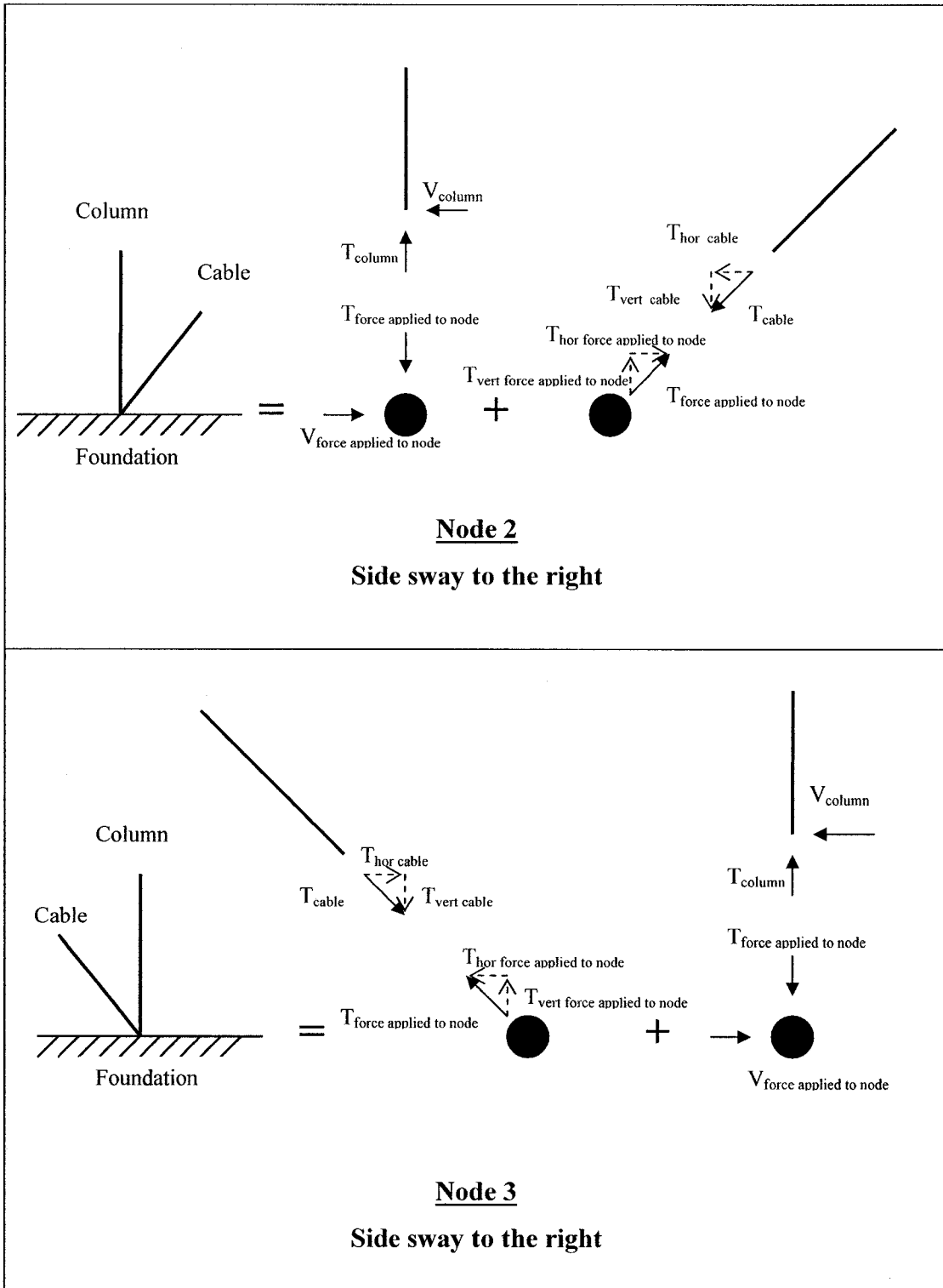
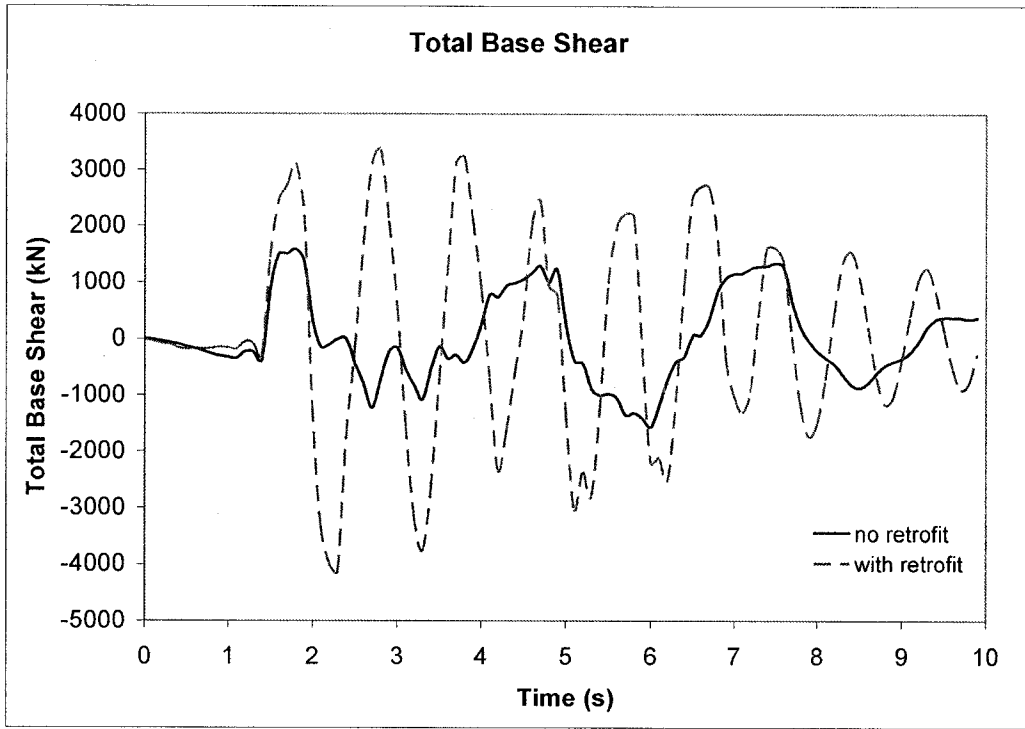
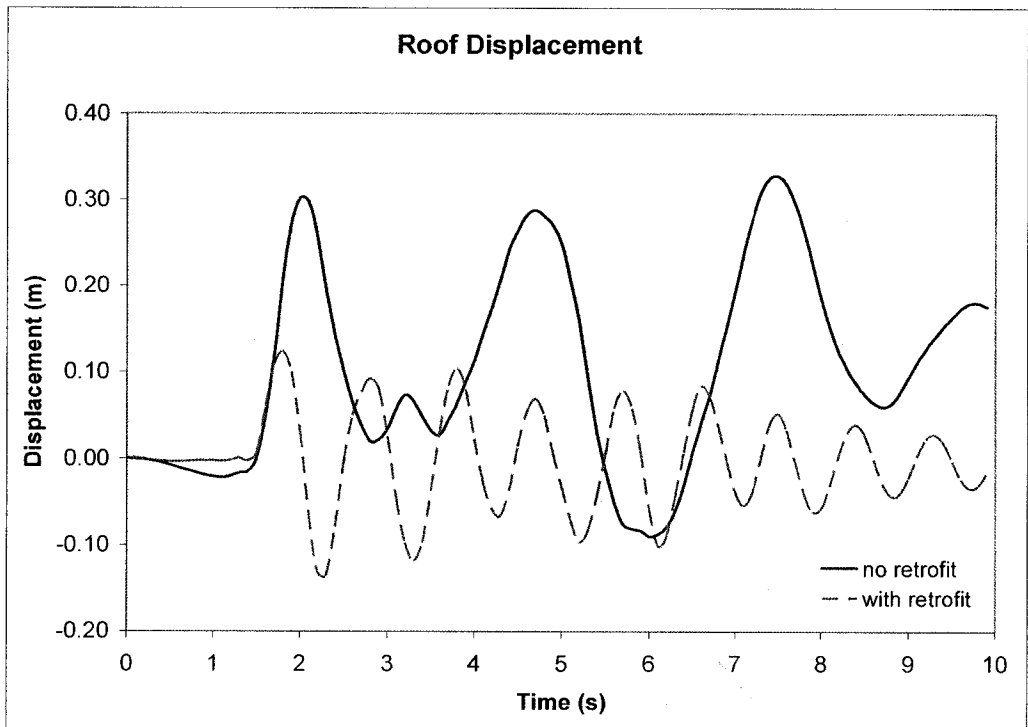


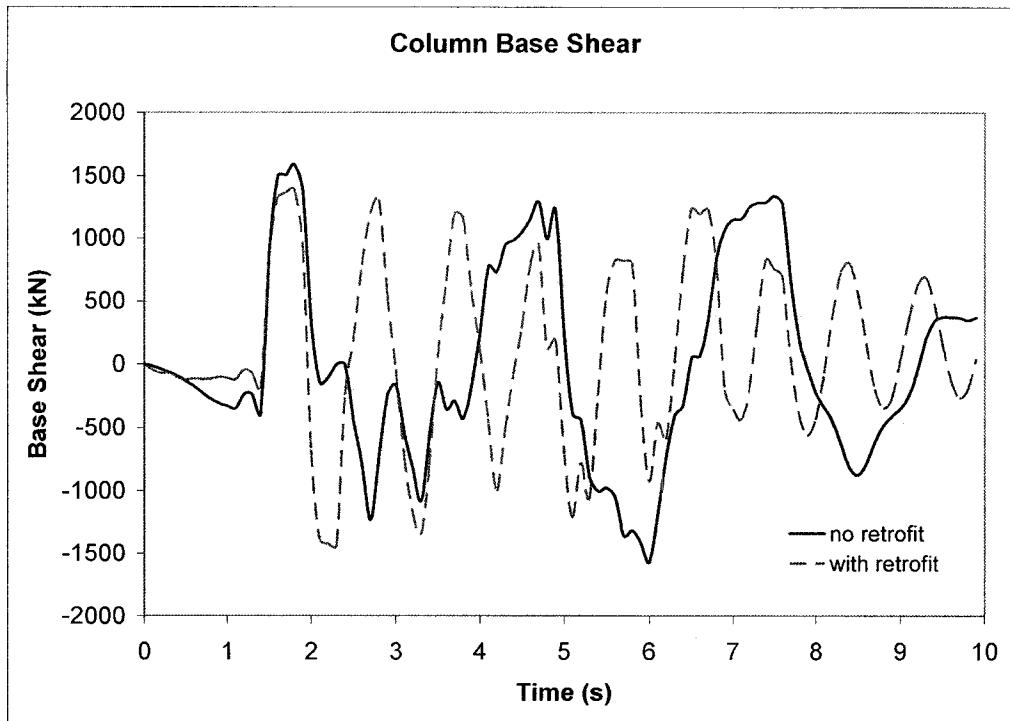
Figure 4.52: Free Body Diagrams of Nodes 2 and 3



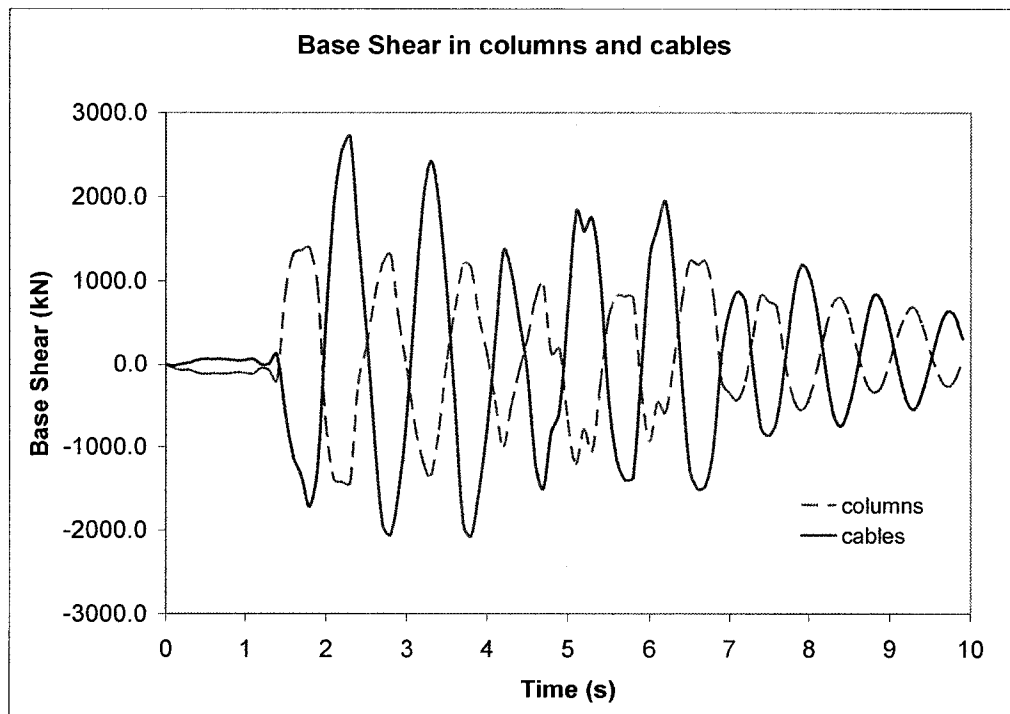
**Figure 4.53: Total Base Shear Demand– 5 Storey – Vancouver**



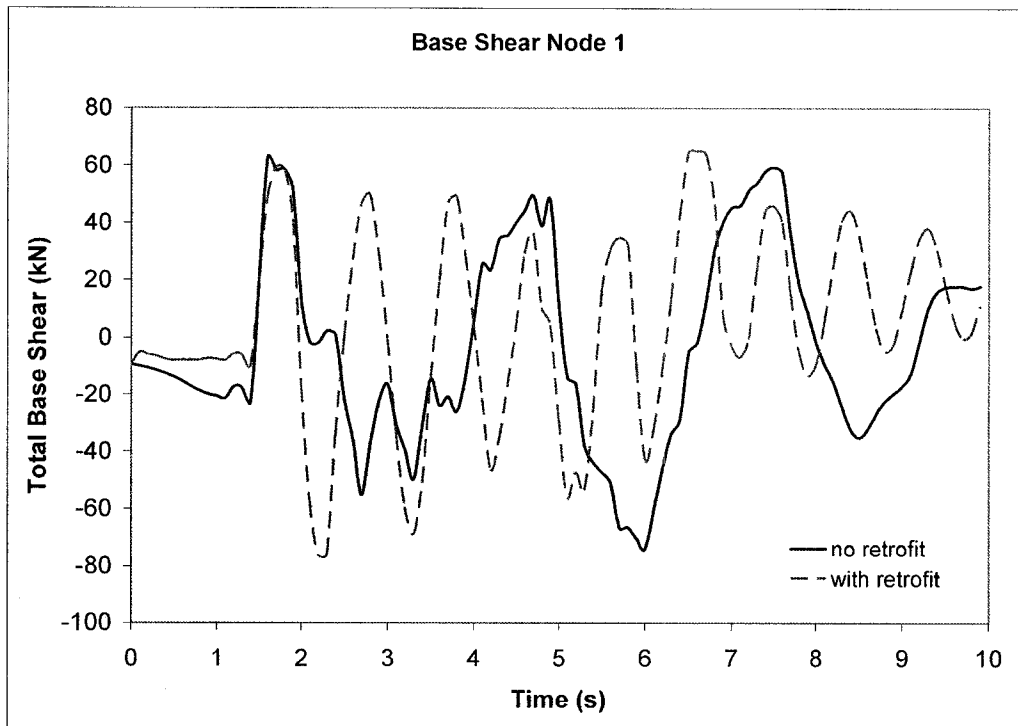
**Figure 4.54: Roof Displacement – 5 Storey - Vancouver**



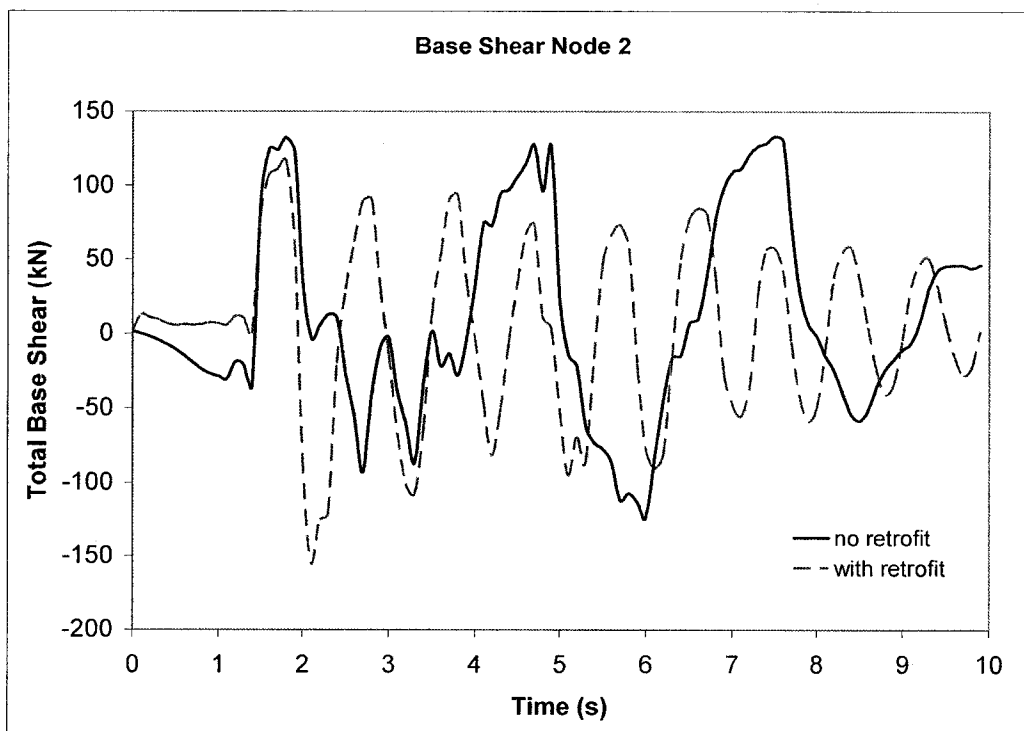
**Figure 4.55: Total Base Shear of Reinforced Concrete Columns – 5 Storey - Vancouver**



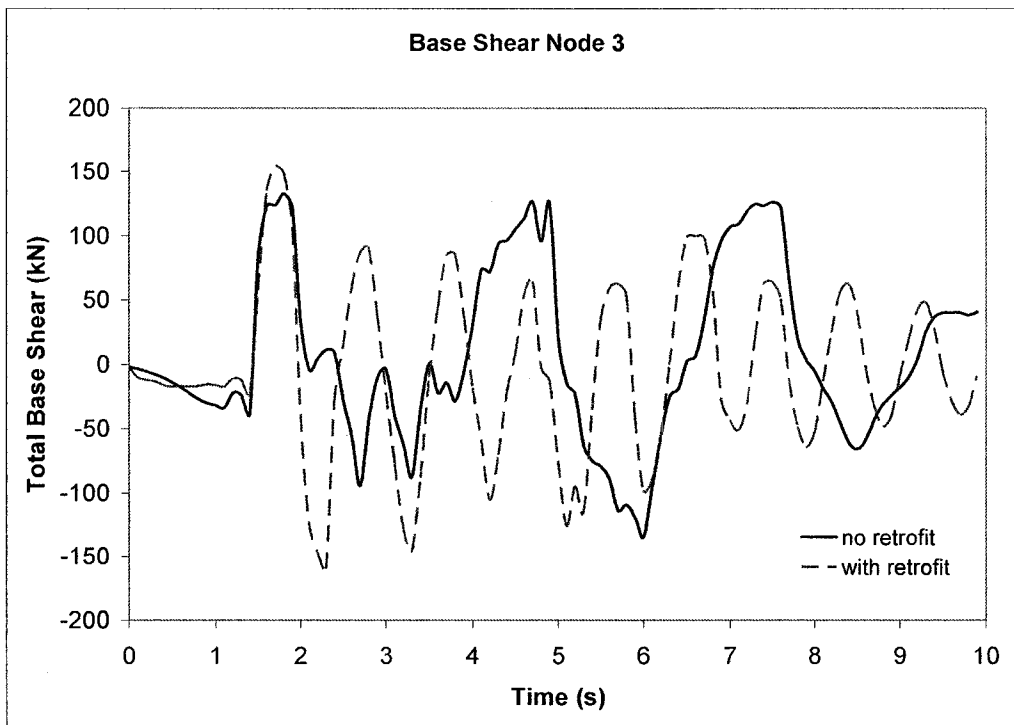
**Figure 4.56: Total Base Shear of Retrofit–Cables vs. Column–5 Storey–Vancouver**



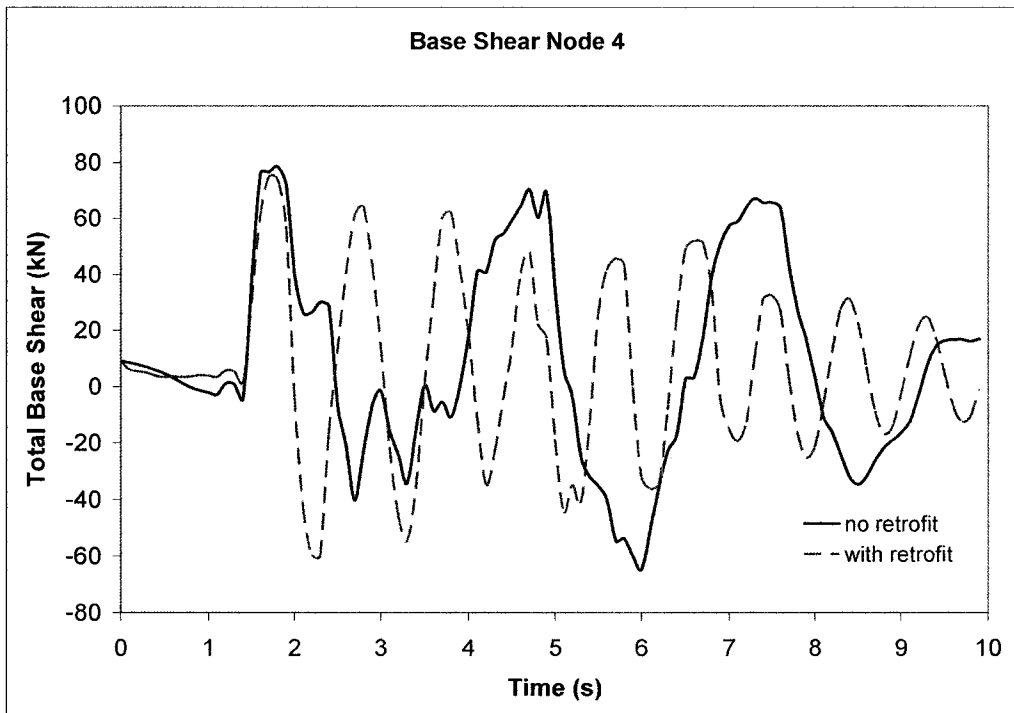
**Figure 4.57: Base Shear Contribution of Reinforced Concrete Column at Node 1**



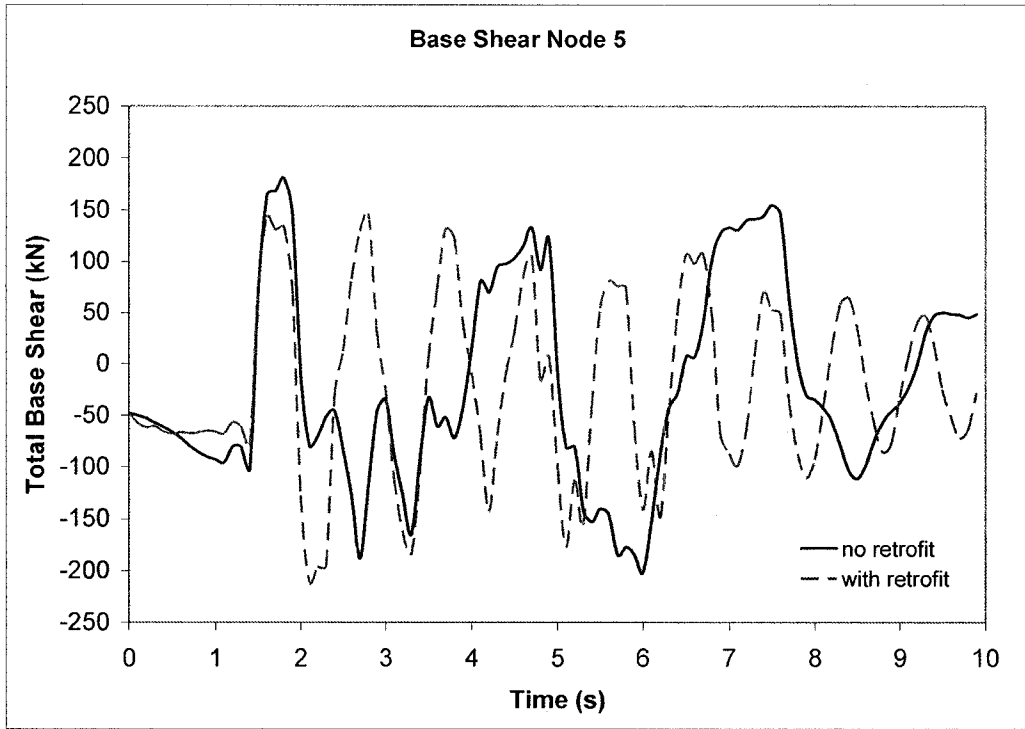
**Figure 4.58: Base Shear Contribution of Reinforced Concrete Column at Node 2**



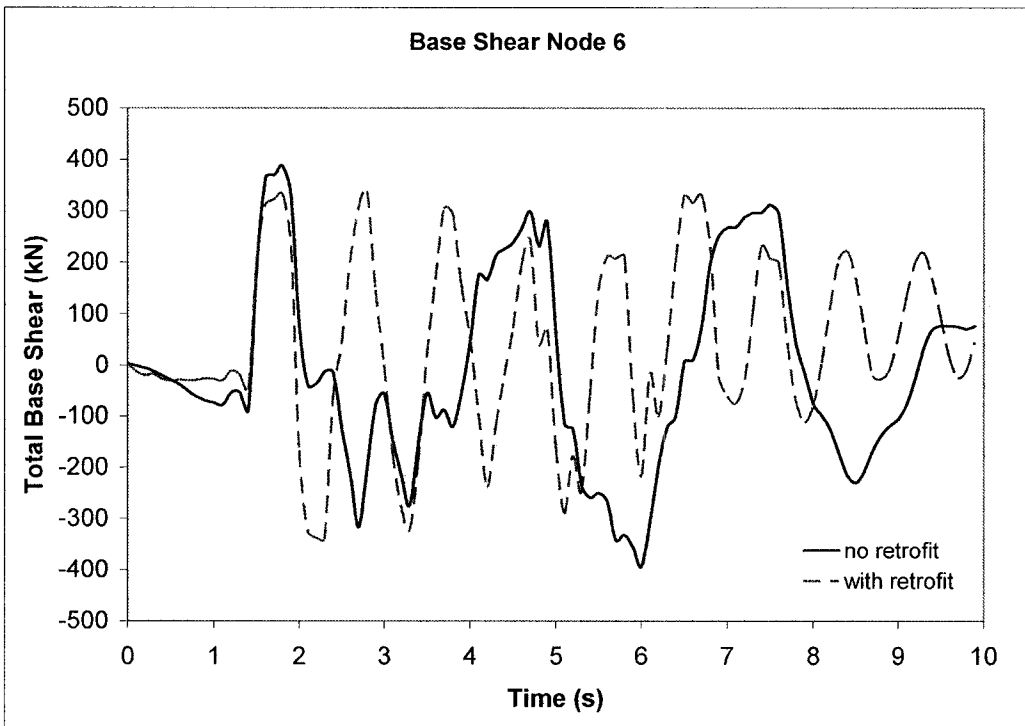
**Figure 4.59: Base Shear Contribution of Reinforced Concrete Column at Node 3**



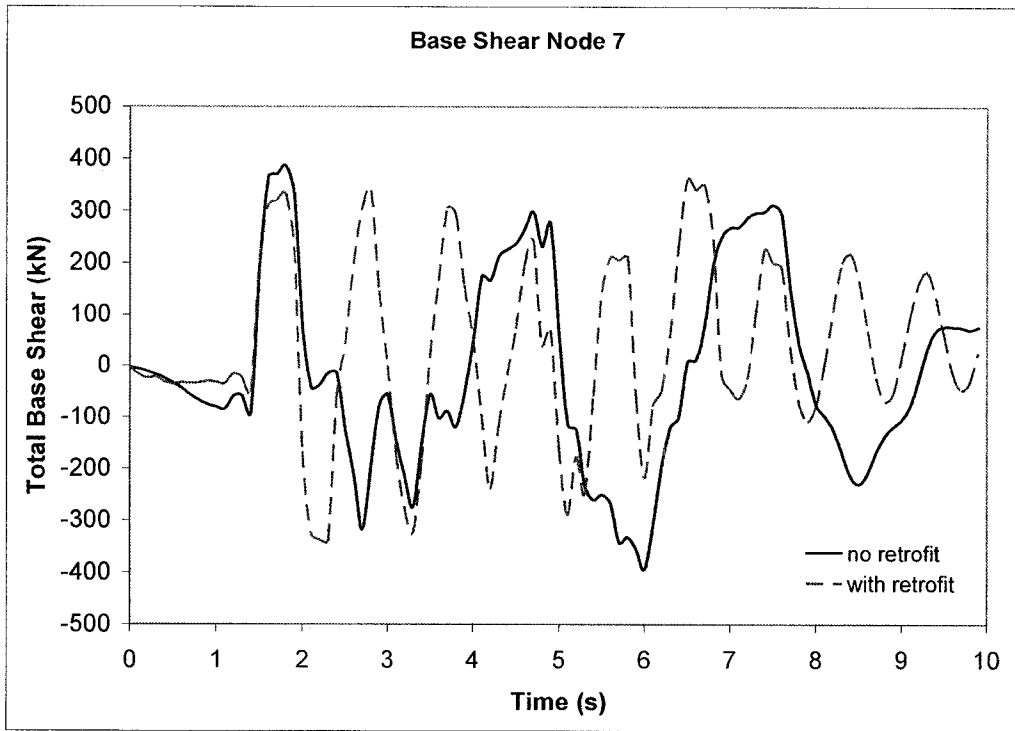
**Figure 4.60: Base Shear Contribution of Reinforced Concrete Column at Node 4**



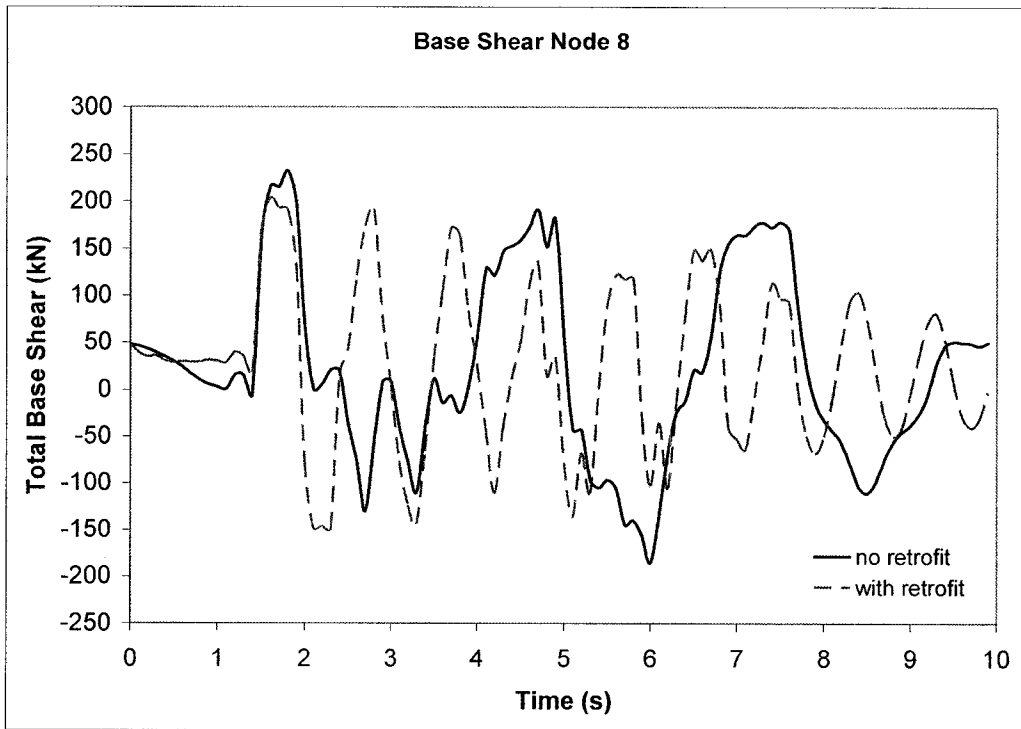
**Figure 4.61: Base Shear Contribution of Reinforced Concrete Column at Node 5**



**Figure 4.62: Base Shear Contribution of Reinforced Concrete Column at Node 6**



**Figure 4.63: Base Shear Contribution of Reinforced Concrete Column at Node 7**



**Figure 4.64: Base Shear Contribution of Reinforced Concrete Column at Node 8**