

# **Seismic Retrofitting of Conventional Reinforced Concrete Moment-Resisting Frames Using Buckling Restrained Braces**

By

Zaid Al-Sadoon

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Department of Civil Engineering  
Faculty of Engineering  
University of Ottawa



uOttawa



## **Abstract**

Reinforced concrete frame buildings designed and built prior to the enactment of modern seismic codes of the pre-1970's era are considered seismically vulnerable, particularly when they are subjected to strong ground motions. It is the objective of this research to develop a new and innovative seismic retrofit technology for seismic upgrading of non-ductile or limited ductility reinforced concrete frame buildings involving the implementation of buckling restrained braces. To achieve this objective, combined experimental and analytical research was conducted. The experimental research involved tests of large-scales reinforced concrete frames under slowly applied lateral deformation reversals, and the analytical research involved design and nonlinear analysis of laboratory specimens, as well as design and dynamic inelastic response history analysis of selected prototype buildings in eastern and western Canada.

The research project started with a comprehensive review of the building code development in Canada to assess the progression of seismic design requirements over the years, and to select a representative period within which a significant number of engineered buildings were designed and constructed with seismic deficiencies. A similar review of seismic design and detailing provisions of the Canadian Standard Association (CSA) Standard A23.3 on Design of Concrete Structures was also conducted for the same purpose. Six-storey and ten-storey prototype buildings were designed for Ottawa and Vancouver, using the seismic provisions of the 1965 National Building Code of Canada, representative of buildings in eastern and western Canadian. Preliminary static and dynamic linear elastic analyses were performed to assess the effectiveness of upgrading the ten-storey reinforced concrete building designed for Ottawa. The retrofit methods studied consisted of lateral bracing by adding reinforced concrete shear walls, diagonal steel braces, or diagonal steel cable strands. The results indicated that the retrofit techniques are effective in limiting deformations in non-ductile frame elements to the elastic range. The numerical analyses were used to demonstrate the effectiveness of Buckling Restrained Braces (BRBs) as a retrofit method for seismically deficient reinforced concrete frame buildings.

The experimental phase of research consisted of two, 2/3<sup>rd</sup> scale, single bay and single storey reinforced concrete frames, designed and constructed based on a prototype six-

storey moment resisting frame building located in Ottawa and Vancouver, following the requirements of the 1965 edition of the NBCC. One test specimen served as a bare control frame (BCF) that was first tested, repaired and retrofitted (RRF) to evaluate the effectiveness of the proposed retrofit methodology for buildings subjected to earthquakes in the City of Ottawa. The control frame was assessed to be seismically deficient. The second frame served as a companion non-damaged frame (RF) that was retrofitted with a similar retrofit concept but for buildings subjected to earthquakes in the City of Vancouver.

A new buckling restrained brace (BRB) was conceived and developed to retrofit existing sub-standard reinforced concrete frames against seismic actions. The new BRB consists of a ductile inner steel core and an outer circular sleeve that encompasses two circular steel sections of different diameters to provide lateral restraint against buckling in compression of inner steel core. Mortar is placed between the two circular sections to provide additional buckling resistance. The inner core is connected to novel end units that allow extension and contraction during tension-compression cycles under seismic loading while providing lateral restraint against buckling within the end zones. The end units constitute an original contribution to the design of Buckling Restrained Braces (BRBs), providing continuous lateral restraint along the core bar. The new technique has been verified experimentally by testing four BRBs on the two test structures under simulated seismic loading. The test results of the BRB retrofitted frames indicate promising seismic performance, with substantial increases in the lateral load and displacement ductility capacities by factors of up to 3.9 and 2.6, respectively. In addition, the test results demonstrate that the BRB technology can provide excellent drift control, increased stiffness, and significant energy dissipation, while the reinforced concrete frames continue fulfilling their function as gravity load carrying frames.

The above development was further verified by an exhaustive analytical study using SAP2000. At the onset, analyses were conducted to calibrate and verify the analytical models. Two-dimensional, one-bay, one-storey models, simulating the BCF and RRF test frames, were created. The models were subjected to incrementally increasing lateral displacement reversals in nonlinear static pushover analyses, and the results were compared with those obtained in the test program. Material nonlinearity was modeled using “Links” to incorporate all lumped linear and nonlinear properties that were defined

with moment-rotation properties for flexural frame members and with force-displacement properties for the diagonal buckling restrained braces. Comparison with test data demonstrated good agreement of the frame behaviour in the elastic and post-elastic ranges, and the loading and unloading stiffness.

The research program was further augmented with nonlinear dynamic time history analyses to verify the feasibility of the new retrofit technique in multi-storey reinforced concrete frame buildings located in Canada and their performances relative to the performance-based design objectives stated in current codes. Prior to conducting the analyses, 450 artificial earthquake records were studied to select the best matches to the Uniform Hazard Spectra (UHS) according to the 2010 edition of the NBCC for Ottawa and Vancouver. Furthermore, additional analyses were conducted on buildings for the City of Ottawa based on amplified Uniform Hazard Spectrum compatible earthquake records. The nonlinear time-history response analyses were conducted using a model that permits inelasticity in both the frame elements and the BRBs.

The results indicated that reinforced concrete buildings built before the 1970's in the City of Ottawa do not require seismic retrofitting; they remain within the elastic range under current code-compatible earthquake records. The structural building performance is within the Immediate Occupancy level, and all structural elements have capacities greater than the force demands. In the City of Vancouver, buildings in their virgin state experienced maximum interstorey drifts of 2.3%, which is within the Collapse Prevention structural performance level. Improved building performance was realized by retrofitting the exterior frames with multiple uses of the BRB developed in this research project. The seismic shear demands were reduced in the columns, while limiting the deformations in the non-ductile frame elements to the elastic range. The lateral interstorey drift was limited to 0.92%, which lies within the Life Safety structural performance level.

## Dedication

The motivation to pursue research in the field of seismic risk mitigation stems from the tremendous research needs in the area as demonstrated during past strong earthquakes worldwide. A large number of existing buildings are seismically deficient, posing a significant threat to living safety, precious human life, and economic well-being of societies. Therefore, I ask Almighty Allah to accept this research as an ongoing charity solely for him, and as a valuable knowledge to save precious lives and humanity during upcoming future earthquakes.

مِنْ أَجْلِ ذَلِكَ كَتَبْنَا عَلَى بَنِي إِسْرَائِيلَ أَنَّهُ مَنْ قَتَلَ نَفْسًا بِغَيْرِ نَفْسٍ أَوْ فَسَادٍ فِي الْأَرْضِ فَكَأَنَّمَا قَتَلَ النَّاسَ جَمِيعًا وَمَنْ أَحْيَاهَا فَكَأَنَّمَا أَحْيَا النَّاسَ جَمِيعًا .... (القرآن الكريم : 5:32)

Meaning Translation:

Because of that We ordained for the Children of Israel that if anyone killed a person not in retaliation of murder, or (and) to spread mischief in the land - it would be as if he killed all mankind, and if anyone saved a life, it would be as if he saved the life of all mankind.... [Holly Qur'an: 5:32]

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# Chapter 1

## Introduction

### 1.1 Background

Existing non-ductile or limited ductility reinforced concrete buildings built before the enhancement of modern seismic provisions in building codes pose a significant risk when subjected to strong earthquakes. These buildings were designed and built primarily to resist wind and/or gravity loads only, with less understanding of seismic demands and design detailing than we currently possess. The vulnerable stock of buildings includes: residential, commercial, schools, hospitals, and other critical facilities. Current seismic code requirements are significantly more comprehensive and stringent than those of the pre 1970's era. In Canada, seismic design force requirements for high seismic regions have increased by as much as 100% since the early 1970's (Cheung et al. 2000). In other words, ductile design and detailing requirements prescribed in newer codes to reduce seismic vulnerabilities were not implemented in the majority of existing older buildings. These vulnerabilities include lack of one or more of strength, stiffness, and ductility parameters. Therefore, a large number of existing buildings worldwide are seismically deficient, posing a significant threat to life safety and economic well-being of society.

As a result, there has been increasing interest in the last four decades, specifically after major earthquakes, to perform seismic risk mitigation by retrofitting older buildings to control lateral drifts and to reduce seismic deformation demands. An effective approach to seismic risk mitigation begins with vulnerability assessment with a view of implementing seismic retrofit techniques when appropriate. Among different retrofit techniques that have been implemented to retrofit existing buildings is the use of steel bracing. A specific form of steel bracing is buckling restrained brace, which forms the topic of current investigation. The investigation forms part of a broader, nationwide research program aimed at reducing urban seismic risk in Canada under the auspices of the Canadian Seismic Research Network (CSRN), funded by the Natural Sciences and Engineering Research Council (NSERC).

## 1.2 Problem Needs and Research Significance

Buckling restrained braces are considered a relatively new type of seismic retrofit system. It has been suggested that its seismic performance, efficiency and potential for use in practice are superior to those for conventional bracing systems (López and Sabelli 2004). Numerous types of BRBs are available for use in practice. However, most of the available literature shows that the focus has been on application to seismically deficient steel moment resisting frames. The application of BRBs to seismically deficient reinforced concrete moment resisting frames is scarce in the literature. This observation was also made by Dinu et al. (2011), and Maheri and Sahebi (1997). The application of BRBs to reinforced concrete frame structures has been assessed in the past through numerical simulations of buildings using three-dimensional models (Di Sarno and Manfredi 2012). The advancement of commercially available structural analysis software promoted the use of analytical tools in the form of nonlinear static and dynamic time-history analyses. Experimental research in the area has been very limited.

The previous experimental research involved tests of individual braces, as well as frames retrofitted with BRBs. The results indicate that the BRB failure occurs mainly in the external and internal reserve gaps of the BRB system, which is shown in Figure 1.1. These gaps are intended to eliminate direct bearing between the load carrying element and the supporting lateral restraining system of the BRB. The brace yield segment with reduced area is un-restrained within these gaps, but has a larger cross-sectional area to serve as a stiffener. Therefore, the segments in the gap are prone to failure caused by buckling. Failure of the steel core (brace yielding segment) at the ends within the gap region due to in-plane and out-of-plane bending moments have been reported by a number of researchers (Tremblay et al. 2006; Aiken et al. 2002; and Zhao et al. 2011). In addition, premature fracturing of steel core plates was reported by Mazzolani (2008) and D'Aniello et al. (2009). It was also noted that many of the previous tests were performed on small-scale samples. Therefore, there are uncertainties whether the seismic behaviour of these small-scale models are representative of full-scale structures.

The existing BRBs are predominantly pre-manufactured and installed by specialized companies and suppliers (Star Seismic, CoreBrace, Nippon Steel, etc.). This leads to additional costs for design, development, materials, transportation, installation, and quality control processes. Additionally, these commercial braces need to be inspected

and evaluated for reliability after major earthquakes. If replacement is necessary, the entire brace (the steel core and the restraining system) must be replaced since they are cast integrally as a unit.

Clearly, there remains needs to improve the current technology. The motivation for this research program is to address the above shortcomings of available BRB systems, including the un-restrained segment of the brace due to the reserve gaps. This work aims to develop an effective BRB system as a retrofit technique through combined experimental and analytical research.

### **1.3 Objectives**

The primary objective of the current research program is to develop a cost effective and structurally superior new buckling restrained brace (BRB) for retrofitting seismically deficient reinforced concrete frame buildings. The objective also includes the verification of the new technology on selected multi-storey reinforced concrete buildings and the development of a performance-based design procedure utilizing nonlinear dynamic analysis.

### **1.4 Scope**

To achieve the proposed objectives, the following experimental and analytical tasks were completed:

- Thorough review of the broader literature on buckling restrained braces, including: concept, development, types, and application and use in upgrading deficient moment-resisting reinforced concrete frame structures. Other types of bracing methods for strengthening of deficient reinforced concrete frame buildings were also reviewed.
- Review of developments in seismic design provisions of the National Building Codes of Canada (NBCC) since 1941 to determine the building code to be used in the design of test frames as well as the prototype buildings. In addition, the relevant provisions of the NBCC is used to assess the lack of strength and ductility in deficient concrete frame structures relative to comparable buildings built based on recent codes and standards with sufficient strength and ductility.

- Review of developments in seismic design and detailing provisions of the Canadian Standard Association Standard CSA A23.3 for construction of reinforced concrete buildings since its inception in 1929.
- Static and dynamic elastic analyses to examine the effectiveness of several bracing techniques to identify the most appropriate retrofit methodology.
- Design and construction of two-2/3rd scale test frames. One frame was repaired after testing followed by retrofitting, while the second frame was retrofitted in its virgin condition.
- Retrofitting of the repaired and virgin frames with buckling restrained braces followed by testing. The retrofit strategy was similar for both frames; however, the demand design forces were based on the Cities of Ottawa and Vancouver for the repaired and virgin frames, respectively.
- Analysis of test results and comparisons of the behaviour of frames to assess the effectiveness of the retrofit strategy with respect to increasing strength, stiffness, and ductility.
- Comparison between the analytical results with experimental test data under static inelastic load reversals to validate the analytical models used in the subsequent seismic analyses of the prototype buildings. The static analyses were performed under incrementally increasing lateral displacement reversals.
- Selection, design and performing dynamic response time history analyses of 6-storey reinforced concrete frame buildings for Ottawa and Vancouver. The analyses were conducted under a large number of ground motion records, compatible with UHS (Uniform Hazard Spectra) of the 2010 NBCC. The analyses were conducted on both un-retrofitted and BRB retrofitted buildings.
- Development of a performance-based design procedure for seismic retrofit of non-ductile or limited-ductility reinforced concrete frame buildings in Canada, utilizing BRBs.



## **1.5 Manuscript Organization**

This thesis is organized into nine chapters and six appendices. Chapter 1 contains the background, research needs, objectives, and scope. Chapter 2 highlights key experimental research efforts conducted by previous researchers to assess the seismic performance of conventional moment-resisting frames and seismic retrofit systems that involve various forms of lateral bracing. Buckling restrained braces concepts, components, and applications are discussed in this chapter. An overview of analytical research available in the literature is also provided. Chapter 3 describes developments of seismic design provisions of the National Building Codes of Canada (NBCC) since 1941; while Chapter 4 explains the developments of seismic design and detailing provisions of the Canadian Standard Association Standard A23.3 for construction of reinforced concrete buildings since its inception in 1929. Chapter 5 includes 2-D and 3-D analytical modelling of a ten-storey reinforced concrete frame building for SAP2000 and ETABS Software (CSI 2010) analyses. It also includes elastic static and dynamic analyses of the building with several types of retrofit techniques to assess their effectiveness in controlling lateral displacements. Chapter 6 presents details of the experimental program conducted in the structural laboratory at the University of Ottawa. The Chapter includes: test specimen design and preparations; materials used for constructing the lab specimens, including those used for the retrofit components; predicted force demands and capacities of the bare control and retrofitted frames; repairing and retrofitting methodologies; retrofitting methodology and its novelty; loading protocols, instrumentation, and test setup. The test results and discussion of results for the control bare frame and the retrofitted frames are presented in Chapter 7. Chapter 8 presents details and results of the analytical models developed to simulate the seismic response of the bare control and retrofitted frames with buckling restrained braces developed in this research program. In addition, seismic analysis of the prototype six-storey moment-resisting building is presented. Program SAP2000 (CSI 2010) was used to conduct the analysis based on incrementally increasing lateral displacements for the test frames. The same program was used to conduct nonlinear dynamic response time history analysis of the selected prototype building. Chapter 9 provides a summary of research, step-by-step design procedure for seismic retrofit of frame structures, conclusions, and recommendations for future research.

Appendices I and II present a comparison of design base shear forces based on the provisions of the NBCC from 1941 to 2010 for six- and ten-storey non-ductile concrete frame buildings, respectively. Appendix III summarizes the prototype reinforced concrete frame structure used to develop the scaled laboratory test frames and the steps considered in the design process. Appendix IV contains the material specifications data of the steel bars used in the BRBs. Appendix V presents analysis information for both bare control and retrofitted frames. These include geometric and sectional properties of frame elements, calculations of columns reinforcement development lengths, force demands, calculation of shear capacities of the frame members according to the General Method of the CSA A23.4-04, and calculation of elastic deformations of BRB brace bars. Finally, Appendix VI presents data recorded during testing by the linear variable displacement transducers LVDT, cable transducers (CT), and strain gauges for the bare control and retrofitted frames.

Figure 1.2 is a schematic that illustrates the research approach.

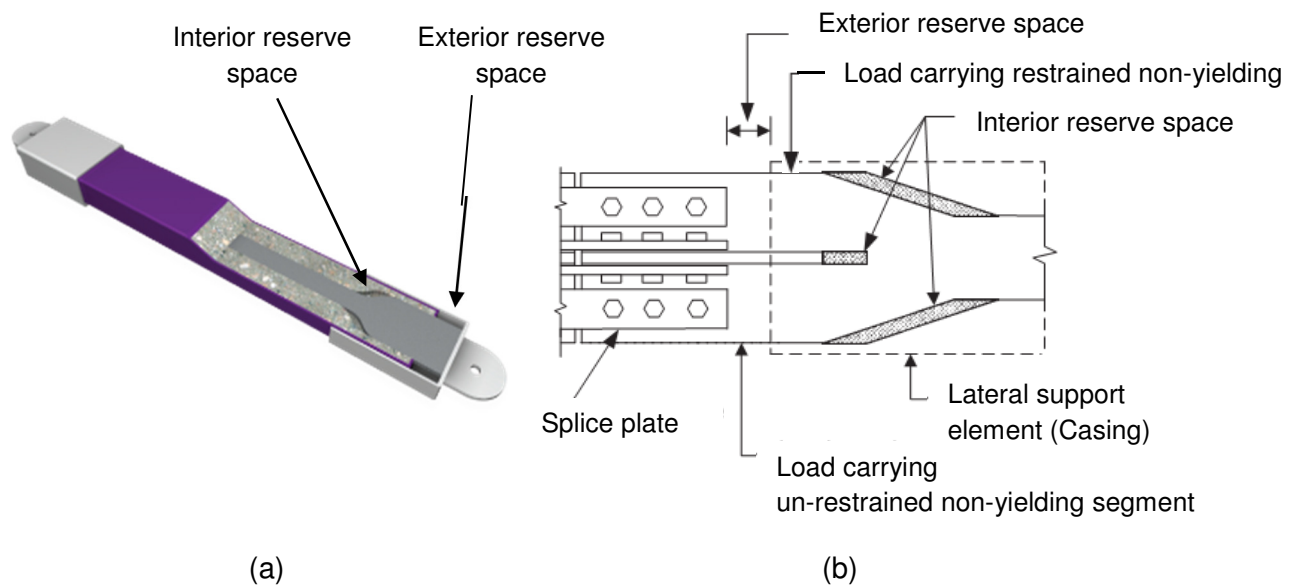


Figure 1.1: External and internal reserve spaces to prevent bearing used in buckling restrained braces (BRB): (a) 3D view of BRB manufactured by Star Seismic LLC; and (b) illustration modified from Chen et al. 2001

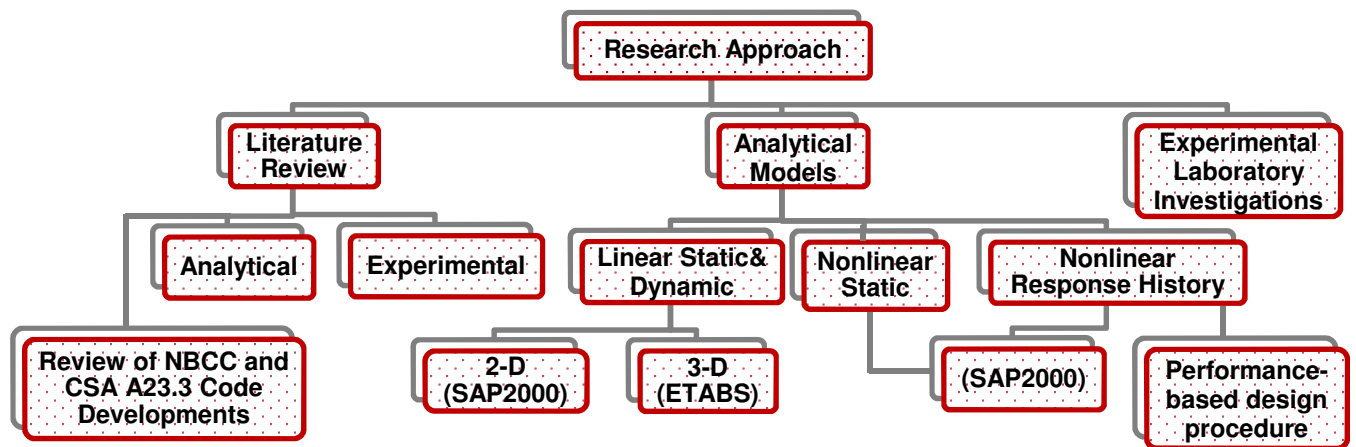


Figure 1.2: Schematic illustration of the research approach

## Chapter 2

### Literature Review

#### 2.1 Available Retrofitting Strategies

Seismic retrofit strategies can be grouped in two main categories: i) global system retrofitting and ii) local element retrofitting. Global retrofit techniques are either based on deformation control and strength enhancement, or reduction of seismic force demands. Deformation control and strength enhancement involve lateral bracing. These methods include adding diagonal steel bracing (as tension-only members, compression-only members, or tension and compression members), prestressed diagonal strands, and concrete shear walls, and implementing active force controls. These strategies may result in increased structural stiffness, reduced period, and higher seismic forces and, thus, should be thoroughly evaluated. The retrofitting techniques for reducing seismic force demands are based on mass reduction and/or the addition of supplementary energy dissipation or base isolation devices. They include: i) removal of one or more stories from the building; ii) installation of friction and viscous dampers; iii) base isolation; and iv) implementation of active and semi-active energy dissipation devices. When one or more stories are removed, the structural period is decreased, leading to higher seismic forces. This has been noted as a concern by others (Giuseppe and Massimo 2005). Damping devices are implemented often in the form of viscous, visco-elastic, friction, and metallic dampers. The effectiveness of friction dampers has been proven experimentally in laboratories and in the field during high intensity earthquakes (Foo et al. 2000). Supplementary damping devices may be more suitable for flexible (usually frame) systems, whereas base isolation may be more appropriate for rigid structures, as in the case of stone masonry heritage buildings and others. These passive control systems have the advantage of being less intrusive. However, the cost associated with the implementation is a major drawback. Active force control systems are relatively new and promising retrofit techniques, consisting of the application of external forces either on the basis of actual ground excitation (open-loop method), or structural behavior (closed-loop method), or based on both ground excitation and structural behavior (closed-open-loop method), (Shooshtari 2005). Semi-active control methods involve adjusting structural properties according to the nature of the earthquake excitation. This

can be achieved by the installation of a central processing unit in the structure (Saatcioglu 2013).

Local element retrofit techniques are based on improving strength and ductility of individual structural components. These techniques include reinforced concrete, steel and FRP column jacketing; masonry and reinforced concrete wall strengthening with steel mesh and concrete overlays, surface-bonded steel plates or FRP composite materials, and adding internal reinforcement layers to masonry. These techniques generally increase local member ductility, energy absorption capacity and lateral load resistance while attaining a more stable response at higher levels of interstorey drift.

The most dominant failure mechanisms for concrete buildings due to seismic loading are: (a) column failure due to inadequate flexural or shear strength; (b) beam-column joint failures; and (c) shear and infill wall failures due to inadequate shear strength or inadequate out-of-plane flexural strength (Dyngeland 1998). Therefore, the literature on the first two failure categories was reviewed, with emphasis on retrofitting using steel braces to be in line with the scope of the current research. The review included experimental research on moment-resisting frames and beam-column joints; while a review of analytical research focused on retrofitting of frame buildings using buckling-restrained braces. The following sections provide a summary of the experimental and analytical literature review.

## **2.2 Experimental Research**

Griffith (2008) noted that research focusing on seismic retrofitting of deficient older buildings has increased significantly. Recent earthquakes have prompted practitioners to observe earthquake-related building damages and to take precautions to protect existing deficient structures from failure in future earthquakes. Experimental research on deficient structural elements provides an opportunity to better understand their performance and to assess rehabilitation methodologies. Experimental retrofitting research on conventional moment-resisting reinforced concrete (RC) frames and beam-column joints are described hereafter.

### **2.2.1 Moment-Resisting RC Frame Retrofitting**

Moment-resisting frame buildings built prior to the enactment of modern seismic design provisions form part of the building stock that presents significant seismic risk to societies. These structures were typically constructed without the seismic detailing prescribed in recent codes. These building types, which are located in many countries, are part of the backbone of the world's infrastructure. Therefore, significant research effort has been invested to develop strategies to retrofit these existing buildings. Among retrofitting strategies is the use of diagonal bracing. Within the literature there is reasonable amount of experimental studies that discuss the use of bracing for concrete frames to increase strength and stiffness. More thorough research is available in the literature for bracing moment-resisting steel frame structures. This is attributed to the large lateral deformations associated with steel frames when subjected to strong ground motions and the ease of connection between the steel braces and the steel frames. Among past studies on concrete frames, braces were employed either in tension or compression or were utilized in both tension and compression. Different types of braces have been investigated: ordinary steel section braces; composite braces consisting of HSS sections filled with concrete; cables, braces with smart materials (SMA); braces incorporating damping devices; and buckling-restrained braces. The following sections provide a review of retrofitting with steel braces, including those incorporating dampers, and buckling-restrained braces.

#### **2.2.1.1 Retrofitting Using Steel Braces**

Youssef et al. (2007) studied experimentally the efficiency of braces for newly constructed RC frames as a means of reducing building weight and seismic demands, and increasing ductility performance. Two frames were designed and tested: the first one was a bare moment-resisting frame designed and detailed as moderately ductile according to the American Concrete Institute (ACI) 318-02 concrete standard; and the second frame was designed following a proposed design methodology for frames with concentric internal braces. The frames were 2/5 scale of an ordinary four storey building. Two, 150 × 120 × 10 mm plates were placed at each corner of the frames and anchored into the concrete with four-16 mm diameter rods. All the braces were designed according to the AISC (American Institute of Steel Construction)-LRFD (Load and Resistance Factor Design) and checked with the AISC seismic provisions for steel structures. Figure 2.1 illustrates the brace-frame connection details and the crack patterns after testing.

For the bare moment-resisting frame, the researchers indicated that plastic response initiated by yielding of the bottom bars in the bottom beam, and failure occurred as a result of plastic hinging at the ends of both the top and bottom beams. No shear cracks were visible; only flexural cracks surfaced in both the columns and beams. However, the braced frame experienced fewer cracks and plastic response was initiated through yielding of the steel brace. The sequence of failure involved buckling of the brace followed by the formation of a hinging mechanism at the ends of the beam similar to the bare moment-resisting frame failure. The hysteretic responses demonstrated that the braces increased the initial stiffness of the frame by factor of 2.0 and the lateral load capacity by a factor of 2.56. However, at failure the drift sustained by the braced frame was 4.0% compared with 5.0% for the companion bare moment-resisting frame.

Figure 2.2 provides the hysteretic behaviour of the two frames. The energy dissipation (area enclosed by the hysteretic loops) of the braced frame was approximately double that of the bare frame. This was very clear after 1.5% drift. However, for drifts less than 1%, and due to the initial higher stiffness of the braced frame, the energy dissipation of the bare moment-resisting frame was marginally higher.

The researchers also suggested that additional individual component tests, experimentally and/or numerically, on the steel connections for the braced RC frame are required. This would provide the necessary information to propose suitable seismic modification factors for design.

Maheri and Ghaffarzadeh (2008) extended the previous work of Youssef et al. (2007) by testing a third braced frame but with different brace sections. The aim was to explore the level of interaction between the strength capacities of the RC frame and the bracing system. The additional frame was of similar dimensions (referred to as FX2), and was braced with non-slender imperial channel cross section (C3 x 3.5) compared with slender double angle cross-sections (L 25 x 25 x 3.2 mm) that was used to brace the frame in the previous test as shown in Figure 2.1 (referred here as FX1). The frame was designed and detailed according to ACI 318-02 and was similar to FX1 and to another moment-resisting frame (referred here as F1). The newly constructed frame yielded and failed at 2.5% and 4.3% drifts, respectively; while FX1 had yielded and failed at 2.08% and 4.0% drifts, respectively. The frame failed at 200 kN of lateral force.

To find the corresponding forces in the braces to establish the capacities of the RC frame and the bracing system independently, the authors used the displacements recorded along the diagonal braces to estimate the forces resisted by the braces. Thereafter, force-drift relationships were established for the three frames. The researchers indicated that the actual measured capacities for both braced frames FX1 and FX2 were higher than the sum of the individual frame capacity (non-braced) and the brace only capacity. This was attributed to over-strength in the individual frame due to the reduced effective lengths of the columns and beams as a result of the brace-frame connections.. Therefore the strength and stiffness of the frames increased. The increased capacity of the braced frames FX1 and FX2 compared to the sum of the capacities of the individual systems (frame and brace independently) were 8.5% and 7%, respectively.

Maheri and Sahebi (1997) studied experimentally the behavior of diagonal tension and compression braces in four small-scale concrete frame models. The objective was to assess the effectiveness of connection details to increase the overall in-plane shear capacity of the frames. The models included: a bare non-braced frame, a frame braced with tension-only diagonal, a frame braced with compression-only diagonal, and a frame braced with an X-tension-compression diagonal brace. The frames were square in configuration with dimensions of 580 x 580 mm and thickness of 100 mm. All frames were designed based on the ACI ultimate strength method. The four frames were tested by applying compression cyclic forces in a universal testing machine. The frames were fitted and rotated such that the vertical and horizontal members of the frame were placed diagonally at  $45^{\circ}$  to the loading base. As a result, the diagonals were subjected to compression and tension. Figure 2.3 illustrates the frames along with connection details and test setup.

The results illustrate that one diagonal brace either in tension or compression increases the in-plane shear strength capacity relative to the bare frame by a factor of 2.5; while the X-braced frame resulted in an increase factor of 4 with 15% difference in ultimate load bearing capacities between the tension and compression braces (tension braces have higher capacity). Failure of the four frames initiated with flexural distress at the four corners of the frame followed by loss of the load bearing capacity. However, the initiating mode of failure resulted in the braces reaching their peak capacities. Therefore, the



brace buckled in the compression-only braced frame; while the brace failed at the welded steel connection at the frame corners in the tension-only braced frame. The X-braced frame test demonstrated that the tension brace resisted a higher portion of load compared to the compression brace. Failure was the result of plate fracture in the welded connection at the mid span of the tension brace, subsequently followed by buckling of the compression brace.

The authors conducted additional tests, using the above test setup, on full-scale connection models for new construction and existing frames. Figure 2.4 shows the arrangements in which each model was tested as a representation of each category. The tests demonstrated the capability of such connection arrangements in enhancing the capacity of the frames. However, the authors indicated the importance of extending the research to develop more appropriate anchorage systems that allow the braces to develop their full capacities prior to local failure in the connections. Extending the results of this research to full-scale frames fitted with tension and compression braces is questionable given the relatively small-scale specimens tested.

Caron (2010) investigated experimentally the benefit of utilizing compression-only X bracing to retrofit non ductile reinforced concrete repaired frames. Two damaged, half-scale frames were first repaired and then tested as shown in Figure 2.5. The original frames were designed according to the ACI 1963, representing frame buildings constructed with codes of practice before the enhancement of seismic provisions. The first frame was only repaired and served as the control frame (BR-3R), while the second frame was repaired and then strengthened with compression-only X braces (BL-3R). The braces consisted of HSS sections. Both frames were tested under reverse cyclic loads.

The tests demonstrated that the repaired control frame provided limited ductility and experienced damage at the base of the columns. The behavior was controlled by frame action. The repaired and retrofitted frame sustained damage in the foundation as well as out-of-plane-twisting. The response was dominated by truss action due to the presence of the compression braces. Although the steel braces in the retrofitted frame did not reach their load carrying capacity, the lateral load capacity increased by a factor of 5

relative to the control repaired frame and the initial stiffness increased by a factor of 7. Figure 2.6 illustrates the envelope response of the first repetition of loading.

The researcher suggested testing different sizes of steel braces and concrete filled steel tubes to increase strength and prevent buckling.

Bush et al. (1991) conducted experimental research on one-2/3 scale, two-bay and three-storey frame subjected to static cyclic lateral loads applied to the third level. The frame was strengthened by attaching exterior wide flange, X steel bracing across two floors continuously. The original frame was built with insufficient strength and ductility and was susceptible to shear failure under seismic lateral loading. The X braces were attached to a steel frame consisting of vertical steel channel chords and horizontal T-section collectors. The steel frame was attached to the columns of the concrete frame along the full height and to the diaphragm spandrel beam through the horizontal collectors using epoxy coated threaded rods. The researchers reported that the strengthening scheme resulted in an increase in both lateral strength and stiffness. The lateral strength capacity of the frame increased by a factor of 2.24 relative to the predicted design capacity and approximately 6 times relative to the calculated failure capacity. Failure of the frame initiated with buckling of the braces followed by failure of the welded connection of the brace and shear failure of the columns. The researchers emphasized the need to ensure proper quality control of the connection welding to maintain structural integrity and functionality.

Sahoo and Rai (2010) used an aluminum shear-yielding link damper (AI-SYD) in combination with steel braces as a passive energy dissipation device to upgrade seismically deficient non-ductile reinforced concrete frames. The deficient columns of the frame were strengthened by external steel cages. The intent of the research was to overcome challenges of integrating such damping devices in existing RC frames. Furthermore, the objective was to improve the lateral strength, stiffness and overall energy dissipation capacities of the entire frame. The RC frame used during testing consisted of a single storey and single bay and represented a 1:2.5 scale.

The aluminum plates were made of alloy-1100 with a thickness of 6.5 mm; while the steel cages consisted of four rolled 35 x 35 x 4 mm angles. The AI-SYD was connected

to a T-hanger using high strength bolts. Two-50 x 50 x 3 mm steel tubular sections were placed in an inverted V shape to support the T-hanger. The test setup and the sections used are shown in Figure 2.7.

The frame was simultaneously subjected to gravity loads and reversed cyclic lateral displacements as per ACI-374.1-05 loading protocol (2006). A collector beam was used to transfer a portion of frame lateral load to the passive energy dissipation device.

The load lateral carrying capacity of the strengthened frame was considered as the sum of the contributions of the AI-SYD system and the column steel encasements. The frame initially yielded at 0.5% drift and was capable of sustaining a drift of 3.5%. The AI-SYD panels demonstrated excellent dissipation through the hysteretic loops. Figure 2.8 provides the hysteretic response of the strengthened frame. The braces and collector beam remained within the elastic range in both tension and compression. No failure of the welding or pull out of anchor bolts was observed at the maximum capacity of the frame.

#### **2.2.1.2 Retrofitting Using Buckling-Restrained Braces (BRBs)**

Buckling-Restrained Brace (BRB) frames are a relatively new and special class of concentrically braced frames. They were initially developed in Japan with the work published by Atsushi Watanabe et al. (1988) and were used as hysteretic dampers for seismic strengthening extensively after the 1995 Kobe Earthquake. In the United States, the BRB system gained recognition following the 1994 Northridge Earthquake; numerous large-scale braces were tested at the University of California (Clark et al. 1999). This led to the first application of BRB in the US on 17<sup>th</sup> January, 2000 (Hussain et al. 2006). More than 250 buildings were constructed with BRBs in Japan (Uang and Nakashima 2004), while the system has been used in the United States for more than 350 Structures (Kimberley and Cameron 2011). The applications included both steel and concrete frame structures. Over the last decade, research has resulted in significant advancements. Furthermore, the implementation in practice has been aided through codification, such as the AISC Seismic Provisions ANSI/AISC in 341-05 and 341-10 editions and ASCE/SEI 7-10 (Kimberley and Cameron 2011).

The BRB concept, components, types, and previous research conducted on reinforced concrete moment-resisting frames is summarized in the following sections.

#### **2.2.1.2.1 Overview of BRBs Concept and Components**

Normal braces withstand both compressive and tensile forces and consist of different steel sections designed to avoid rupture under tensile stresses and buckling under compression. Buckling of braces is function of the slenderness ratio of the member, which is a ratio of the effective length to the least radius of gyration of the cross section. Therefore, large cross-sectional areas are usually specified to avoid buckling failure in compression and subsequent damage to the framing structural integrity. To overcome such challenges, the concept of buckling restraining was developed.

The main concept of buckling restraining is to decouple the stress resistance of the main yielding steel core from the flexural buckling resistance that is provided by the lateral casing as illustrated in Figure 2.9 (Lopez and Sabelli 2004). The relatively small intervals of lateral bracing to the compression carrying load element effectively reduce the unbraced effective length to zero (Hussain et al. 2006).

Figure 2.10 illustrates further the concept of the buckling-restrained braces and its common use. A ductile steel core is designed to yield in compression as well as in tension at their adjusted strengths (Clark et al. 1999). This behaviour is attained by placing the steel core within a casing which may be of concrete, steel, composite, or any other material. This casing will provide the required lateral stiffness when the steel core deforms laterally. A material is applied around the brace core to ensure that the axial forces are resisted only by the yielding brace. This material prevents bonding between the steel core and casing such that no axial load is transfer to the casing. Typically, mortar or concrete is used as the fill material between the un-bonding material around steel core and the casing. The fill material provides additional resistance to buckling. The steel core hysteretic behaviour is provided in Figure 2.10, which illustrates that the BRB can be viewed as a damper that dissipates significant and similar energy under tension and compression stress conditions. After major earthquake events, structures built with this type of bracing are expected to return to its original shape after the replacement of the deformed damper (Uang and Nakashima 2004). This is possible since the main

structure remains elastic and only the dampers are expected to dissipate energy (Wada et al. 1992)

Components of BRB are well explained by many researchers (Uang and Nakashima 2004; and Lopez and Sabelli, 2004). Figure 2.11 provides schematic diagrams of the five components of a commonly used BRB. The steel core is comprised of three components and the remaining two components are the un-bonding agent and the restraining mechanism of the casing. Namely they are: (1) restrained yielding segment, which can be any steel section whether single or multiple, and is laterally restrained by the surrounding filler materials and casing (Section C-C); (2) restrained non-yielding transition segment which is an extension of the restrained yielding segment placed inside the casing but with larger cross sectional area, usually by adding stiffeners, in order to ensure an elastic behaviour (Section B-B); (3) un-restrained, non-yielding segment which is an extension of the restrained non-yielding segment that extends past the casing (Section A-A). This segment provides the connection point for the brace to the frame. In steel structures, the connection can be in the form of bolting or welding or a true pin; (4) un-bonding insulator applied to the restrained yielding segment to provide a gap to prevent or minimize frictional force transfer to the surrounding mortar and casing. The literature shows that these insulators could be made of extruded polystyrene foam, silicon rubber sheets or grease, vinyl sheets, masking tape, asphalt paint, or an air gap. The size of the gap is based on the maximum strain in the yielding steel segment and lateral deformations expected while the brace is subjected to compressive stresses. Poisson's ratio of the steel core material both in the elastic and plastic ranges are used to assess these lateral deformations. Finally (5) buckling restraining system that prevents overall buckling. This includes the casing, usually HSS, and other fill materials that restrain the core.

The brace is connected to the frame building through the restrained yielding segment only; there is no connection to the supporting restraining system of the brace. This implies that the load carrying element will be shortened when the brace is subjected to compression stresses, while the length of surrounding casing remains constant. Therefore, external and internal reserve spaces are used to prevent direct bearing from the load carrying element to the supporting lateral restraining system as shown in Figure 2.12. The internal gaps contain an un-bonding material between the load carrying

element at its restrained non-yielding segment and the surrounding mortar. The exterior gaps at each end of the brace between the casing and the supporting joints (gusset plates or mechanical hinge or any other connection details) also prevent any interaction between these two components. Therefore, the presence of exterior gaps leads to shorter casing length than the length of the brace system. The yielding segment element is un-restrained in these gaps and therefore these segments are prone to buckling. Failure at the steel core ends due in-plane and out-of-plane bending were reported by number of researchers (Tremblay et al. 2006; and Aiken et al. 2002). Therefore, proper stiffening should be provided to these core segments to prevent such failure. In contrast, if no gap is provided or if it is not sufficient, the load will bear against the lateral supporting system and the brace may experience pre-mature buckling failure at its mid-length.

#### **2.2.1.2.2 Types of BRBs**

Braces restrained against buckling were first developed in Japan in two forms: (1) steel core encased and sandwiched between concrete precast panels; and (2) steel braces encased in steel tubes or reinforced concrete members. These were subsequently developed in number of other countries (Uang and Nakashima 2004).

The first type consisted of steel plates sandwiched in reinforced concrete panels serving as a restraining system as shown in Figure 2.13. A gap is typically provided between the concrete panel and the surrounding steel frame to allow for relative deformations. This type of BRB was not extensively documented relative to other types of BRBs (Escudero 2003). According to Uang and Nakashima (2004), this research was initially conducted by Wakabayashi et al. (1973) and extended further both numerically and experimentally by Inoue and Sawaisumi (1992).

The second type consists of steel cores encased in Hollow Structural Sections (HSS). The HSS are commonly of steel material and consist of one or more sections. Figure 2.14 illustrates various types of BRB cross sections. Typically, filler materials are used between the steel core and the outer casing. Steel cores confined only by the HSS casing with small void spaces to separate the two without any filler materials are referred to as, "All-Steel" buckling-restrained braces (Della Corte et al. 2005). The term unbonded brace is the current term used for BRBs that consist of a steel core that is

unbonded from the surrounding laterally restraining system. Literature on this type of brace is presented in the following section.

#### **2.2.1.2.3 Pervious Research on BRBs**

Experimental research conducted on buckling-restrained brace components and seismic performance of typical structures strengthened with these braces are categorized as follows: (a) Investigation of casing stiffness including the transfer of force between the steel core to the steel casing; (b) Assessment of brace ductility capacity under low-cycle fatigue for various loading histories; (c) Study of connection fractures and out-of-plane buckling behaviour at the joints between the braces and the adjoining frames; and (d) Investigation of the un-bonding layer and voids on the performance of the brace due to different types of interfaces between the steel core and the casing (Della Corte et al., 2011).

The literature illustrates that numerous large-scale unbonded braces have been tested in Japan since the late 80's (Atsushi Watanabe 1988) and in the United States since the late 90's, with tests conducted at the University of California in 1999 (Hussain et al. 2006). These braces were tested individually with quasi-static MTS testing machines to demonstrate the behaviour of these braces under increasing uniaxial cyclic loading histories. The literature on upgrading moment-resisting frames with such braces demonstrates reasonable effectiveness, particularly for the steel frames due to the ease of connection of the braces to the frame structure. Conversely, limited experimental research has been performed to assess the behaviour of RC moment-resisting frames strengthened with buckling-restrained braces. This has also been noted by Dinu et al. (2011), and Sarno and Manfredi (2012). The literature on seismic performance of deficient moment-resisting frames retrofitted with buckling-restrained braces is summarized in the following paragraphs.

Dinu et al. (2011) conducted an experimental test on a full-scale, one-storey, one-bay reinforced concrete frame to assess the performance of non-seismic, three-storey reinforced concrete frame buildings constructed in the 1940's and strengthened with inverted V-shaped buckling-restrained braces located in the exterior frames as shown in Figure 2.15.

Before applying the brace to the structure, 12 BRBs were tested under monotonic and cyclic loads to assess its effectiveness to rehabilitate the structure. The BRB section consisted of a steel core that was debonded with polyethylene film and inserted inside a square HSS and filled with concrete. The test setup and BRB behaviour are shown in Figure 2.16.

The frame retrofitted with BRB was tested under cyclic loading. The test setup and behavioural results are illustrated in Figure 2.17. The connection between the BRB and the frame consisted of steel plates fixed at each near and far side of the column and beam cross section. These plates were bolted with external steel ties. Testing demonstrated that the connections performed well with maximum recorded slip of 5 mm.

The hysteretic response of the frame was stable in tension and compression with a displacement ductility capacity of 2.0. The steel core reached its yield capacity in tension and failed due to rupturing. The crack pattern in the beam-column joint illustrates the progression of damage up to ultimate displacement.

Although the case study building was analyzed analytically, no experimental test was performed on a bare control frame for the purpose of comparison with the retrofitted frame. Thus, enhancements of either strength or ductility were not provided in the research.

Sarno and Manfredi (2012) carried out full-scale experimental tests on two identical reinforced concrete building frames consisting of 2 stories and 2 bays in which one of them was retrofitted with buckling-restrained braces. The buildings were designed based on 1960's design standard employing the allowable stress method and were assumed representative of typical residential buildings in moderately seismic zones with PGA of approximately 0.25 g. The frames were constructed with two bays of 2.55 m length each in the longitudinal loading direction and one bay of 4.4 m length in the transverse direction. The total building height was 7.35 m and the slabs were 0.2 and 0.25 m in thicknesses for the first and second floors, respectively, while the columns were square section with dimensions of 0.3 m. The BRB braces were installed in the longitudinal direction and parallel to the horizontal loading direction. Two MTS hydraulic jacks were connected to a steel reaction wall at one end and to the first and second floors at the



other end. The loading was applied in a displacement-controlled mode. The BRB section included 1 m long yielding steel core connected to a 101.6 x 6.3 mm circular HSS. The average length of the BRB was 4 m for the two stories. The target interstorey drift was 0.3%. Details of the test specimen and test setup are illustrated in Figure 2.18.

Both the control and retrofitted RC frames were tested under monotonic and cyclic loading. For the monotonic tests, the control building experienced yielding at the base of the columns at 0.5% building drift; cracking and damage were observed at the bottom and top of the first storey columns; and the first floor beams deformed plastically at 1.1% drift, while the second floor beams remained elastic. Failure occurred at 2.2% roof level drift with localized failure at ground floor level. The monotonic test for the braced frame revealed: no damage to the ground floor columns, while minor cracks became visible during low lateral displacement drifts; and the reinforcement in the beams and columns behaved elastically during the test as measured by the strain gauges. The test was terminated at 1% drift level which corresponded with the design axial displacements of the BRBs.

Results of the reverse cyclic loading tests for the two frame buildings are shown in Figure 2.19. The control building was pushed to roof drifts of 1.0% and 1.1%. During testing, cracking was observed at ground floor columns and deterioration in building stiffness and strength was observed at 1.0% roof drift. After the first set of reverse lateral loading, there was a reduction in the hysteretic energy dissipation of approximately 40%. On the contrary, the hysteretic behaviour of the braced building was very stable, with a significant increase in strength and energy dissipation through yielding of the BRB core steel in tension and compression. The strength of the retrofitted frame was enhanced, relative to the control building, by a factor of 1.6; while the displacement ductility increased by a factor of 4.1.

Note that there were no clear details for the brace cross sectional components that may have influenced the overall seismic performance of the braced buildings.

Mazzolani (2008) and D'Aniello et al. (2009) conducted full-scale experimental cyclic tests on a two storey building that was seismically upgraded with different bracing techniques, including the "All-Steel" buckling-restrained braces. The building was

originally constructed during the 1970's and was planned to be demolished by the local authorities. The building was divided into six separate one-bay, two-storey structures. The plan dimension of each structure was 5.3 m x 5.3 m. This was achieved by cutting the slabs and perimeter beams and removing all partitions and non-structural elements. Three tests were performed on the structures: bare control test and two additional tests on the structures retrofitted with braces. Figure 2.20 illustrates the original building and the braced structures and sections for testing. At each floor level a brace was used at both sides of the structure, but alternating in diagonal direction. Globally, the braces formed an X shape through the two floor levels.

Reverse cyclic loads were applied to the structures to failure of the BRBs by means of two hydraulic jacks working in an alternating mode in two opposite directions. The jacks were connected to an erected steel structure. The braces were built from steel sections only and consisted of a single plate (25 mm x 10 mm) that was restrained by two rectangular HSS (100 mm x 50 mm x 5 mm). Two types of BRBs were built from these section components, thus two tests were performed on the braced structures; one for each type of BRB. Gaps of 0.5 mm and 1 mm were provided between the core steel and HSS tubes for BRB type I and II, respectively. For both brace types, a 100 mm interior reserve space was provided between the core and the HSS tubes to prevent bearing on the restraining tubes. Gusset plates were used to connect the braces to the RC structure.

The test results for the BRB type I, shown in Figure 2.21, provided evidence of core yielding in both tension and compression. However, local buckling was reported at the un-restrained ends of the steel core with localized shear and bending forces resulting in separation of the steel core plate from the tapered plates. Subsequently the core plate fractured prematurely. Flexural cracks were also visible at critical sections of the first floor near the base and near the beam-column joints. The bracing system provided a reasonable upgrade in both strength and stiffness for the deficient RC frame. This is evident from the hysteretic behaviour; an over strength factor of approximately 4 relative to the control bare frame and a displacement ductility ratio of approximately 4.8.

BRB type II was designed with improved end details to enhance the global performance over BRB type I. Progress of damage observed during testing and the hysteretic

behaviour of the structure are illustrated in Figure 2.22. The inner steel core experienced inelastic buckling at one end, which was apparent at a first storey drift of 5.6%. These plastic deformations were attributed to the frictional demands between the steel core and the restraining tubes, as well as out-of-plane deformations at the transition section between the reduced core area and tapered end section. At higher drifts, large flexural cracks were visible at the first storey columns. The hysteretic behaviour of this type of BRB provided significant improvement in ductility. The maximum interstorey drift was 5.6% which corresponded to a global displacement ductility of 12.7. A strength increase of approximately 5 times the control bare frame was attained.

### **2.2.2 Beam-Column Joint Retrofitting**

Beam-column joints of older buildings are critical components ensuring integrity of reinforced concrete buildings during earthquakes. Due to its brittle behaviour, severe damages have been reported in the literature from recent earthquakes. Such damage has been responsible for either partial or total collapse of buildings (Moehle and Mahin 1991). This was the motivation to explore retrofitting strategies to rehabilitate poorly detailed beam-column joints and to avoid failure of these local components (Said and Nehdi 2008). Retrofitting strategies that can be easily accommodated include: addition of internal reinforcement, implementing steel bracing, and fixing the connections of steel structures. These methods will be further explained in the following sections.

#### **2.2.2.1 Retrofitting Using Steel Reinforcement**

Abdel-Fattah and Wight (1987) tested 12 full-size interior beam-column sub assemblages under quasi-static loading. The aim was to establish practical design recommendations of moving the plastic hinge location from the face of the columns to a distance of 1.5 effective beam depths away from the column face. The intent was to avoid stiffness and strength degradation at the joints during moderate and severe earthquakes. The strategy was achieved by adding supplementary intermediate, top and bottom longitudinal reinforcement over a specific length near the column face. The additional longitudinal reinforcement successfully distributed the flexural shear cracks over a distance within the beams rather than localized within the poorly detailed beam-column joint. The researchers recommended that the ratio of one layer of intermediate reinforcement be approximately 0.3 to 0.35 of the top tension reinforcement. The proposed methodology is schematically shown in Figure 2.23.

### 2.2.2.2 Retrofitting Using Steel Braces

Said and Nehdi (2008) conducted experimental tests on rehabilitated deficient beam-column joints by means of an external steel bracing system. Two specimens were built and tested under reverse cyclic loading, and the results were assessed against a third specimen reported in the literature. The three specimens were of similar dimensions and referred to as: non-retrofitted control specimen (J1) representing a standard joint according to modern seismic detailing practice; a retrofitted steel braced specimen (J2); and a non-retrofitted specimen reported in the literature (T0). The latter two specimens, J2 and T0, had similar reinforcement detailing representing the code of practice during the pre-1970s era. The bracing system is shown in Figure 2.24.

The researchers concluded that the rehabilitation technique was successful in improving the behaviour of deficient joints to the expected behaviour of current standards. The deficient specimen (T0) experienced slippage of beam bottom reinforcement and joint shear failure before the initiation of plastic hinge formation in the beam. The standard specimen (J1) failed at a displacement ductility factor of 6 through buckling of the compression reinforcement in the beam plastic hinge area at a distance equal to one beam depth from the face of the column. In this region, a significant portion of the cover concrete was lost. The retrofitted specimen (J2) test was terminated at a displacement ductility of 2.5 which was lower than the ductility achieved by the standard specimen. The authors attributed this to the reduction in the beam cantilever length due to the presence of the steel brace. This resulted in an increased beam tip yield capacity. The bracing members remained elastic up to joint failure. Figure 2.25 shows the crack patterns at failure and load-storey drift envelopes for the three test specimens. It is evident that the retrofitted specimen (J2) enhanced the drift capacity compared with specimen T0 by a factor of 3.0 and 5.5, and the load capacity by a factor of 2.6 and 1.7, for the pushing and pulling directions respectively. The energy dissipation, area under the hysteretic load-displacement response, was increased by a factor of 12.6.

Due to the limited test specimens, the authors suggested further testing to confirm the results, particularly specimens with deficient transverse reinforcement and no anchorage deficiencies. Additionally, it was noted that a global experimental research program involving frames rather than beam-column sub assemblage and studies to verify the results through dynamic time-history analysis be performed.

### **2.2.2.3 Retrofitting Using Steel Connections**

Maheri and Hadjipour (2003) conducted full-scale direct tensile static tests on three types of steel connections for braced RC frames. The bracing systems are applicable for seismic retrofitting of older deficient frames as well as for construction of new buildings in high seismic zones. The connection of the braces to the concrete frames were achieved with bolted plates at the frame corners rather than fully welded steel jacketing around concrete members; the latter method has been reported to lack the robustness of the bolted plate technique. Figure 2.26 shows the three connection details.

The connections were designed by combining the code provisions for steel and concrete. Mechanical strain gauges were placed on the connection elements to measure the elongation of the brace, gusset plate and the connecting plate-concrete element. Results of the three types of connections demonstrated that the gusset and connecting plates remained elastic beyond the brace yielding and rupture strength. Furthermore, connection Types (a) and (b) exhibited more robust strengths, compared to connection Type (c). However, connection Type (c) behaved in a more ductile response. The load-displacement curves of the elements of connection Type (a) and the behaviour of the three connection types are provided in Figure 2.27.

## **2.3 Analytical Research**

Literature on the analytical performance assessment of full-scale structures strengthened with BRBs can be divided in two main categories: (a) Numerical frame models to assess the overall building performance based on energy dissipation and ductility demands of the BRBs, as well as the demands of the non-dissipative force members and their connections; and (b) nonlinear finite element models to validate the experimental behaviour of these braces (Della Corte et al. 2011). The current research program includes numerical modeling to investigate retrofitting for RC frame structures using BRBs.

It is evident that the availability of commercial structural engineering software has permitted analytical investigations for both categories mentioned above to assess performance of buildings retrofitted with buckling-restrained braces. Furthermore, these tools can be used to develop design procedures to meet target displacement demands for braced buildings. The programs that have been reported in the literature include:

OpenSees, Drain-2DX, Drain-RC, SAP2000, Perform-3D, ETABS, SeismoStruct, ANSYS, RAM, Snap-2DX, amongst others. Most of these programs are capable of modelling complicated building geometries, unlimited number of floors, and different materials properties, and perform nonlinear static (pushover) and dynamic time-history analyses. For the latter, natural and artificial earthquake records can be used. In addition, inelastic response time-history analysis at different building operational states can be performed.

Di Sarno and Manfredi (2010) conducted seismic performance assessment of two storey reinforced concrete moment resisting frame structure that was designed in the early 1960 and used as a school building. The use of buckling restrained braces placed along the building perimeter frames was examined as a retrofitting strategy. The 3-D Finite Element (FE) program, SeismoStruct, was used to model the building by employing the fiber based approach to simulate the nonlinearity at both local and global frame levels. The FE models for the as-built and the retrofitted buildings are shown in Figure 2.28. Nonlinear static as well as dynamic analyses were performed on the two structures. In the latter analysis, seven natural, code-compliant earthquake records were used. The global performance of the structure was assumed to be the sum of the elastic response of the as-built structure plus the inelastic response of the energy dissipated braces (BRBs). The buckling restrained braces consisted of plates embedded in a steel tube and were modeled as inelastic truss elements. Equivalent mechanical properties were used to simulate the geometric and mechanical properties of the BRBs provided by the manufacturer and used in practice.

Results of the nonlinear analyses for the two structures revealed that BRBs were efficient to reduce both local and global structure lateral displacements and that interstorey drifts were reduced by approximately 50%. Noteworthy is that the BRBs were effective in dissipating around 60% of the input seismic energy at the ultimate limit state.

Bordea and Dubina (2009) examined the effectiveness of buckling restrained braces (BRBs) to strengthen three storey reinforced concrete frame buildings that were built in the late 1930's. Numerical models were created, using SAP2000 structural analysis program, to perform a nonlinear static pushover analysis, for the building before and after retrofitting. Inverted V BRBs were distributed along the outside perimeter in the

central bays in both directions of the building. The BRB section consisted of inner steel yielding plate core of 2.0 m length encased within an HSS steel section and filled with concrete. Three steel core cross sectional areas were selected in which larger areas were placed in the lower floors to accommodate the higher anticipated force demands. Figure 2.29 shows models of the building along with BRB brace locations and geometry.

To attain the nonlinear global behavior of the retrofitted system, nonlinearity was assigned for the materials as well as the members. Figure 2.30 illustrates nonlinear material models, plastic hinge locations in the members, and BRB force-deformation behavioral characteristics. The nonlinear model of Kent and Park was assigned to the concrete while the modified Park model was assigned to reinforcing steel. Plastic hinges (load deformation/rotation relationships) were assigned to the ends of the beams and columns, to all areas of sectional reinforcement changes, and at the intersection of the V braces in the case of beams. These were assigned based on FEMA 356 procedures. Stiffness of beams and columns were reduced by 50% and a rigid diaphragm was assumed for all three levels. The BRB was assumed pinned at both ends with identical behavior in tension and compression. Bi-linear force-deformation behavior with strain hardening was assigned to the BRBs.

Performance assessments of the original un-retrofitted building illustrated the formation of plastic hinges in columns followed by the beams. For the retrofitted building, plastic hinges generally initiated in the BRBs followed by the columns and beams. The strengthening scheme increased the strength by a factor of 2.5, while the top displacement decreased by a factor of 4. The authors suggested better sizing of the BRB yielding core area to match the predicted strength and demands, and to consider local strengthening of the columns and beams with FRP to cope with increased strength demands as a result of the newly added bracing.

Chandra and Warnitchai (2011) conducted nonlinear dynamic time history analyses to assess the seismic performance of six storey reinforced concrete moment resisting frame buildings before and after retrofitting. The "all steel" buckling restrained braces similar to the one suggested by Mazzolani (2008), with flat plates restrained by steel tubes with no mortar fillings, were considered in the retrofit. The building was designed and constructed for gravity loads only and was used for residential occupancy. The

plans and elevations of the original (ORI) and the retrofitted (BRB) buildings are illustrated in Figure 2.31. The building contains a weak/soft storey in the first floor due to the existence of a parking area; there were no masonry walls between the columns. Therefore, four buckling restrained braces were considered at first floor level to prevent soft storey failure. The base of the columns, to which the BRBs were connected to, were jacketed with reinforced concrete to increase the cross sectional areas to prevent localized failures as a result of the higher demands imposed by the BRBs.

Three dimensional modeling of the two original structures (ORI) and retrofitted (BRB) buildings were performed with Software OpenSees. Beams, columns, and beam-column joints were modeled with the capability to simulate many response characteristics including brittle behaviors, whereas infill walls were modeled as single diagonal compression struts connected to the RC frames. The slabs were assumed to behave as rigid diaphragms, while the foundations were considered fixed. Buckling restrained braces were modeled as truss elements pinned at their ends and expected to yield prior to the other elements reaching their peak strength capacities. Two ground motion sets were selected from PEER strong motion data base: San Fernando (SF 1971) and Northridge (NR 1994) earthquake records. The spectral response acceleration of these records with 5% damping was compared to the UBC 1997 spectral response. The earthquake records were then scaled based on the fundamental period of the original building to simulate moderate and strong records. Target spectral accelerations of 0.5 g and 0.7 g were selected for the moderate and strong earthquake records, respectively. The (ORI) building was analyzed under moderate earthquakes while the retrofitted (BRB) structure was analyzed under moderate and strong earthquakes. The maximum interstorey drifts are shown in Figure 2.32, as well as the sequence of plastic hinge formation within the frames.

Performance assessments demonstrated that the behaviour of the BRB building is improved relative to the ORI building for the moderate earthquake. The ORI building exhibited poor seismic performance due to the soft storey in the first storey level leading to the formation of plastic hinges and damage to the infill walls in the first storey level. Given this damage level, the building was not analyzed for strong earthquakes. Conversely, there was minor damage in the BRBs and infill walls in the BRB building. The retrofitting method limited the maximum storey drift to less than 1%. Under strong



earthquakes, maximum storey drifts were below 2% in the BRB building and minor to moderate damage was observed in the BRBs and infill wall panels in the second floor.

Oviedo et al. (2010) investigated the seismic response of a series of ten storey RC buildings with hysteretic dampers in the form of buckling restrained braces. The concept of “constant yield storey drift ratio” as a deformation–controlling approach to determine the yield deformation of the hysteretic dampers was proposed. The analysis was based on nonlinear time history analysis employing two real ground motions, El Centro NS (1940) and JMA-Kobe NS (1995), and a third synthesized record widely used in Japan. The three records were scaled to Peak Ground Velocities of 0.5, 0.75, and 1.0. The building description along with locations of the BRBs is illustrated in Figure 2.33.

Program Drain-2DX was used for 2-D numerical analysis for the building shown in Figure 2.33 (b), while Program STERA-3D was used to conduct the 3-D analysis using generic hysteretic damper models. The bilinear model and the degrading bilinear Takeda model were used to simulate the hysteretic behaviors of the columns and beams of the 2-D models, while trilinear Takeda model was used to depict the hysteresis behaviors of the columns and beams in the 3-D models.

The BRBs were modeled as single inelastic truss elements that can sustain large compressive strains without buckling. However, an idealized approach was provided for modelling the BRBs by considering three springs connected in series as shown in Figure 2.34.

A parametric study was performed with two main variable parameters:  $\beta$  (the yield strength ratio of the damper system compared to the entire system, i.e. damper and RC building); and  $v$ , the yield storey drift ratio of the damper system compared to the RC building. Analytical results revealed a relatively constant distribution of reduced floor displacements using the proposed retrofitting scheme. Moreover, the damper system dissipated hysteretic energy before the yielding of the RC main frame. This behaviour offers higher structural safety and reduced seismic damage in the RC structure.

Dinu et al. (2011) conducted nonlinear pushover analysis to assess the performance of a 3 storey, non-seismic reinforced concrete moment-resisting frame (MRF) building built

before 1963 and strengthened with inverted V-shaped buckling restrained braces. The building geometry is illustrated in Figure 2.35. The building consists of 3 bays of 4 m lengths each and is located in a seismic zone characterized by a peak ground acceleration of 0.24 g. The assessment of the retrofit was performed through three analytical models: the un-retrofitted MRF, retrofitted MRF with BRBs, and retrofitted MRF with BRBs and FRP column confinement.

The results revealed that the MRF is vulnerable and does not meet seismic requirements for moderate earthquake zones. The analysis predicted the formation of plastic hinges in the first and second floor columns and the beams of the first floor. The numerical models of the BRB retrofitted structure predicted noticeable increases in strength and stiffness, while the displacements demands at the top floor level were reduced by 50% at the ultimate limit state (ULS). The number of plastic hinges that formed was less and they were concentrated in the middle bay. The hinges initiated in the columns of the first two floors followed by the BRBs and beams. To overcome this unsatisfactory performance, the retrofitted model was modified by locally confining the columns with FRP. This resulted in the formation of localized plastic hinges in the BRBs in the first and second floors and beams of the first floor. Figure 2.36 illustrates the three numerical models including the location of the plastic hinges at the target displacement, which was beyond the ULS capacity, as well as strength enhancements at the target displacements.

Zhang and Hu (2010) idealized a multistorey RC frame building structure as a SDOF system and presented nonlinear time history analyses for two retrofitting methods: self-centering and BRBs. The prototype structure was chosen to represent a series of two to six storey RC structures found in the literature. A fundamental period of 0.6 s was set for the prototype structure. The hysteretic models employed strength and stiffness degradation to represent the hysteretic behavior of the non-ductile frame building. The hysteretic models used for the two retrofitting strategies are illustrated in Figure 2.37. Two main parameters were studied: yield strength and elastic stiffness ratios ( $r_y$  and  $r_k$ ) of the retrofitted systems relative to the original RC frame system. For the simulations, 172 historical earthquake ground motion records were selected and scaled to the design-basis earthquake of California with a probability of exceedance of 10% in 50 years.

The analysis results indicated that both retrofitting schemes achieved comparable seismic performance, while the residual displacements were negligible for the self-centering strategy. The peak displacement and energy dissipation demand ratios (ratios of maximum displacement and cumulative hysteretic dissipated energy with and without seismic retrofit, respectively) were found to generally decrease with an increase of the initial stiffness and yield strength of both retrofitted schemes.

Mazzolani (2008) validated the experimental tests of the two storey building that was seismically upgraded with two configurations of BRBs, as illustrated in Figure 2.20. The validation was conducted with nonlinear numerical models implemented in SAP2000 program. The BRBs were modeled as truss elements using a bilinear force-displacement relationship. Three models were created for the “type I” BRBs (Figure 2.20) and the assumptions that specified for each category were evaluated. The three models were referred to as: preliminary, simplified and improved. The preliminary model assigned elastic perfectly plastic behaviour to the BRBs, equal strength in tension and compression for the BRBs, and no cracked section properties for the RC frame structure. The simplified model assigned 10% over strength to the compression capacity of the BRBs relative to the tension capacities, cracked EI values to the RC structure, and 3% post elastic stiffness for the P- $\Delta$  axial relationship to the plastic hinges in the BRBs. The improved model was similar to the simplified model with the exception of increasing the post elastic stiffness from 3% to 5% of the initial brace stiffness. The three models adequately predicted the experimental response as shown in Figure 2.38(a). The BRB brace ductility was equal to 5.

The modeling of the strengthened frame structure with “Type II” BRBs (Figure 2.20) was similar to the preliminary and simplified models of the frame structure retrofitted with “Type I” BRBs. The elastic stiffness and yield strengths in both types did not differ significantly. However, the improved model was calibrated to take into account the local distortional buckling failure at one end of the BRB plates observed during testing as shown in Figure 2.22. The initial lateral deflection at the end of the brace equal to the observed experimental value was assigned at the first floor level. In addition, the post elastic stiffness of the BRBs at the first floor for the tension and compression responses was assumed to be 3% and 9% of the elastic axial stiffness, respectively. This is to take into consideration the evidence of the local buckling of the BRBs internal core occurred

during the test. The analytical results are compared to the experimental  $P-\Delta$  responses in Figure 2.38 (b). The improved model predicted a maximum ductility of 14.33.

Skokan et al. (2010) conducted a nonlinear numerical analysis to assess seismic retrofit performance of a six storey reinforced concrete MRF that was built in the early 1970's and located near the Newport-Inglewood fault. The retrofitting scheme included the addition of new exterior RC frames, each adjacent to one bay of the exterior frames of the existing building. Inverted V-shaped BRBs were placed in the new frames. The new RC frames were connected to the existing building exterior frames with epoxy dowels. The BRBs were manufactured by Nippon Steel Company in Japan. A three dimensional model was created with SAP2000 to verify the seismic behavior of the retrofitting scheme. The BRBs were modeled with nonlinear axial truss elements with specified tension and compression capacities. Figure 2.39 depicts the building retrofitting scheme. The intent of the retrofit was to provide the necessary upgrade for strength, stiffness, and ductility in the event of strong earthquake activity. Seven sets of scaled ground motion records were used in the time-history analysis for the two orthogonal building directions.

Based on analyses, the retrofitting scheme reduced the maximum storey drifts to 0.8% and the force demands imposed on the deficient MRF to 30% of the total storey shear forces.

Islam et al. (2010) conducted a case study on a 14-storey reinforced concrete MRF residential tower that was constructed in California based on 1965 edition of the UBC. The building was identified to lack the required structural life-safety performance. Upgrading included the addition of new exterior frames on each side of the existing building. Inverted V-shaped BRBs were placed within the new frames as shown in Figure 2.40. This retrofitting was chosen to minimize disturbance to the occupants. The BRBs were manufactured by Nippon Steel and had design axial capacities ranging from 1023 kN to 3114 kN and design axial strains ranging between 2.12 % - 3.04 %.

The retrofit followed the performance-based design approach and was assessed for the following two stages: (a) nonlinear dynamic response spectrum and a life-safety performance for the addition of new frames and BRBs, with a response modification

factor (R) of 8 to account for inelastic deformations; and (b) nonlinear time history analysis for the entire system including the retrofit scheme. In the latter analysis, life safety and collapse prevention performances were set as the objectives.

Three-dimensional model was created by RAM Perform-3D program. Nonlinear elements were used to simulate the existing and new structures including the BRB braces. Moment rotation hinges were assigned at the end of the beams and columns. The corner columns, which experienced significant axial loads and moments, were modeled with nonlinear fiber cross-sections at their ends. The BRBs were modeled using Perform-3D “BRBF” nonlinear elements.

The retrofitted building, subjected to seven time-history records, limited the maximum interstorey drifts to 0.8% and 1.3% for the life-safety and collapse prevention performances, respectively. The results further illustrated that the retrofiting strategy was effective in reducing drifts by 50% and the contribution to the total storey shears was 70%.

## **2.4 Summary**

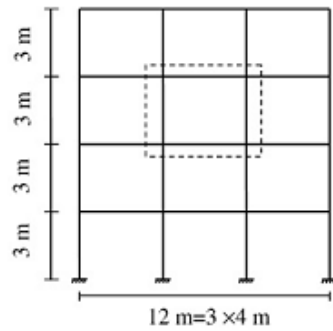
A comprehensive literature review was conducted on experimental testing on retrofitting deficient moment-resisting frames (MRFs) and beam-column joints through steel bracing. In addition, analytical research on retrofitting moment resisting frames through the employment of buckling restrained braces was thoroughly reviewed. The review of experimental work on retrofitting MRFs using steel braces included: ordinary steel sections; braces employing damping devices; and buckling restrained braces (BRBs). The literature review of deficient beam-column joints comprised of the addition of internal reinforcement, attaching tie steel braces, and modifying the connections of steel sections.

The retrofit methods presented are generally either complex, intrusive, expensive, or requires substantial time for repair and maintenance. Moreover, the review of the literature has demonstrated a need to test relatively full-scale frames rather than small-scale specimens. The scaling of the seismic behaviour of the latter to full-scale is questionable.

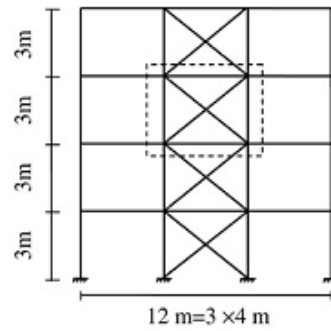
The literature on retrofitting reinforced concrete MRF buildings braced with BRBs has focused on testing individual large-scale unbonded braces under cyclic loading. There is limited research on experimental system testing of buckling restrained braces attached to reinforced concrete frames. Moreover, results that are available from individual brace testing or frames retrofitted with BRBs, indicates that the external and internal reserve gaps, that are part of the BRB systems, are locations prone to failure even though these transition sections between the reduced core area and the end tapered area are stiffened by steel plates.

The literature on analytical/numerical modeling investigating retrofitting of RC frame structures using BRBs has concentrated on assessing the overall building performance based on ductility demands and energy dissipation of the BRBs. The modelling has included nonlinear static and dynamic time-history analyses. However, the majority of this research was based on modeling the core braces of ordinary steel (hot rolled plates). Global seismic response assessments that have considered different types of core steel bars with different ductility and strength properties is limited.

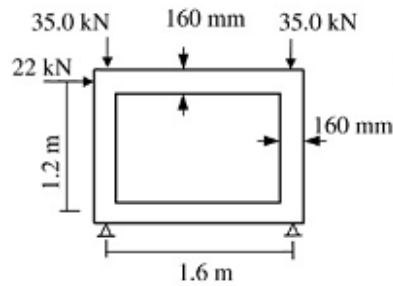
Therefore, the research presented herein addresses the lack of information in the literature and proposes a structurally sound, economically feasible, less intrusive and innovative seismic retrofit technique. This novel retrofit employs a mechanism that employs a continuously restrained core buckling restrained brace that has been tested with three different steel materials. Furthermore, the BRB system can be re-used after a seismic event. The core yielding steel component can be easily replaced. The suggested retrofit method is intended to upgrade existing repaired and virgin reinforced concrete frames structures to the state-of-the-art practice. This research will demonstrate that limited ductility reinforced concrete frames can be repaired and then retrofitted to satisfy requirements of current codes. The proposed retrofit technique is verified through large-scale testing of two 2/3 scale moment resisting frames under simulated seismic loading and verified through dynamic response time history analyses.



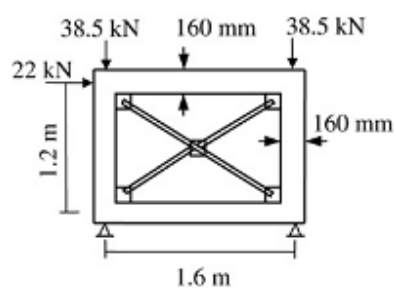
Moderately ductile RC moment frame



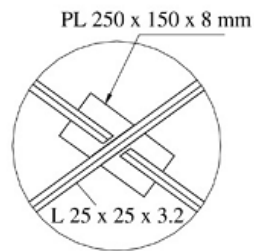
Braced RC frame



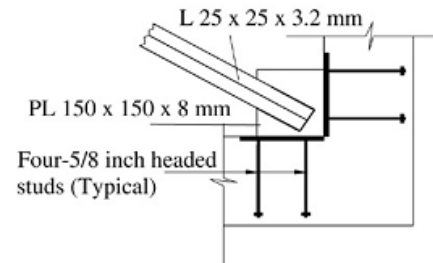
Scaled moderately ductile RC moment frame



Scaled braced frame



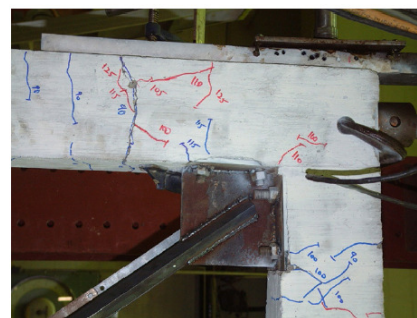
Bracing connection at the centre



Brace gusset plate connection



Test setup



Cracks in the braced frame

Figure 2.1: Lateral load resisting frame specimens with design loads, connection details, and failure crack patterns after testing (Youssef et al. 2007)

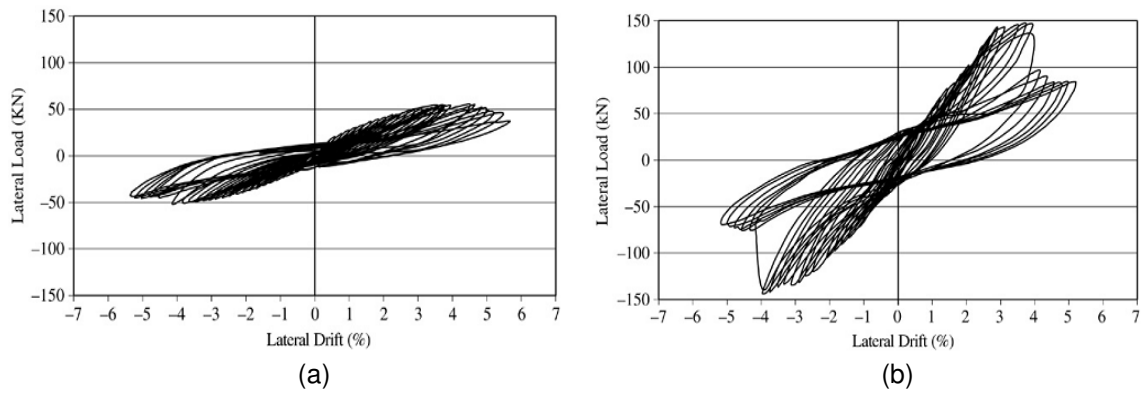


Figure 2.2: Hysteretic behaviour: (a) bare frame; and (b) braced frame (Youssef et al. 2007)

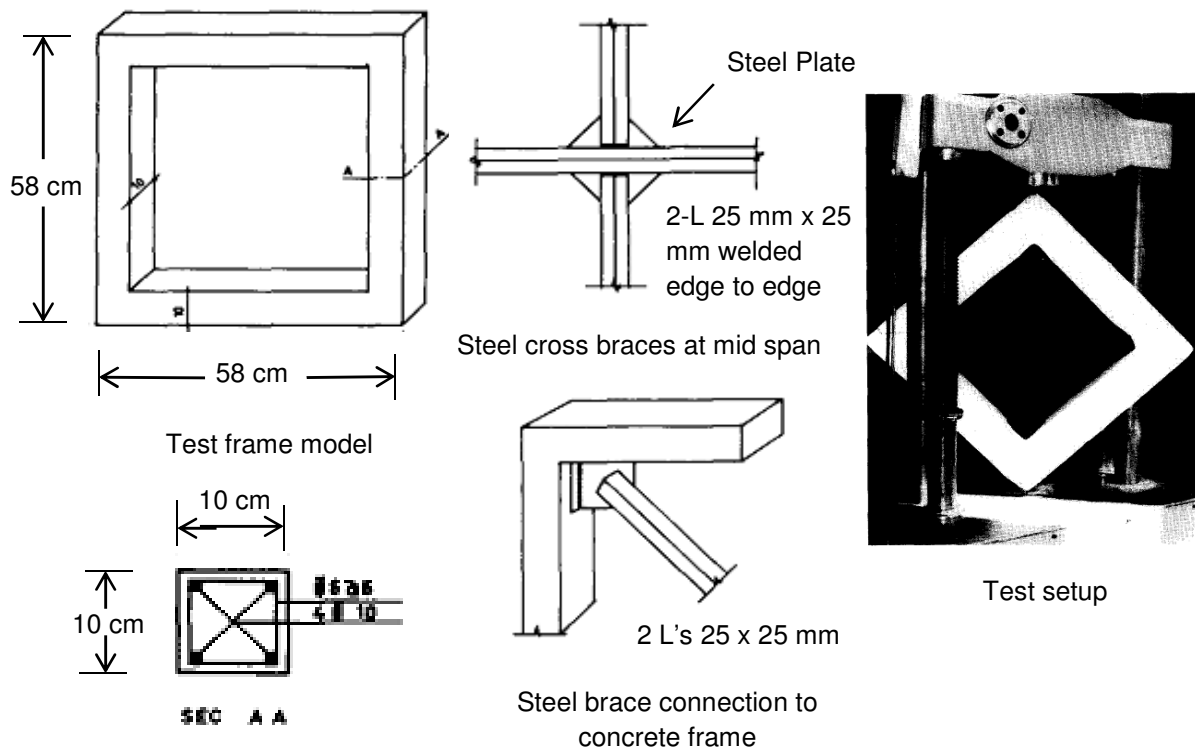


Figure 2.3: Frame configuration, connection details and test setup (Maheri and Sahebi 1997)



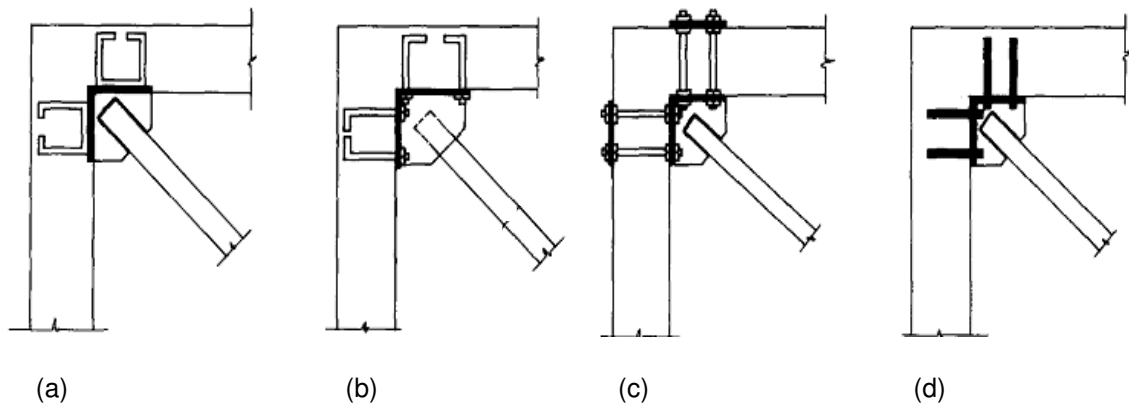


Figure 2.4: Possible brace frame connection details: (a) and (b) for frames under construction; (c) and (d) for existing frames (Maheri and Sahebi 1997)



Figure 2.5: Seismically deficient frames prior to testing: a) repaired frame BR-3R; and b) repaired and retrofitted frame BL-3R (Caron 2010)

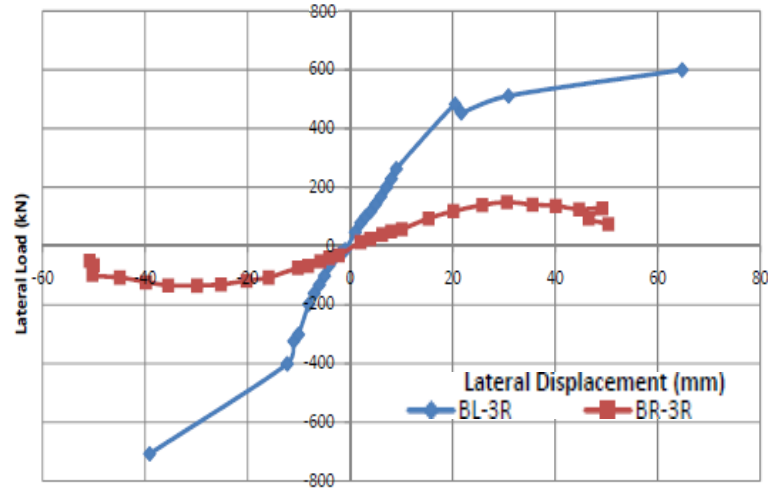


Figure 2.6: Lateral load versus lateral displacement envelope responses  
(Caron 2010)

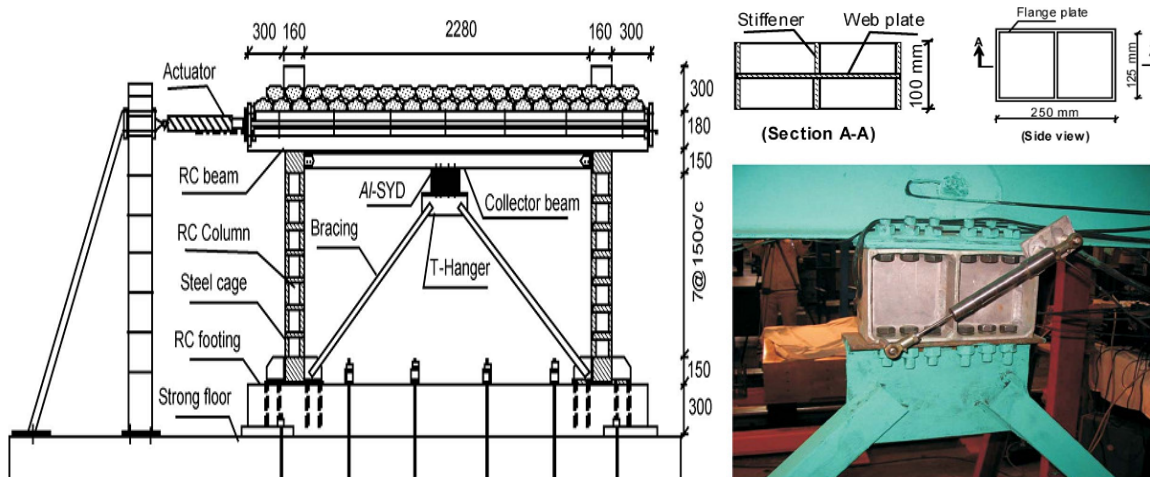


Figure 2.7: Schematic diagram of test set-up and sections, and photo of AI-SYD used to strengthen RC frame (Sahoo and Rai 2010)

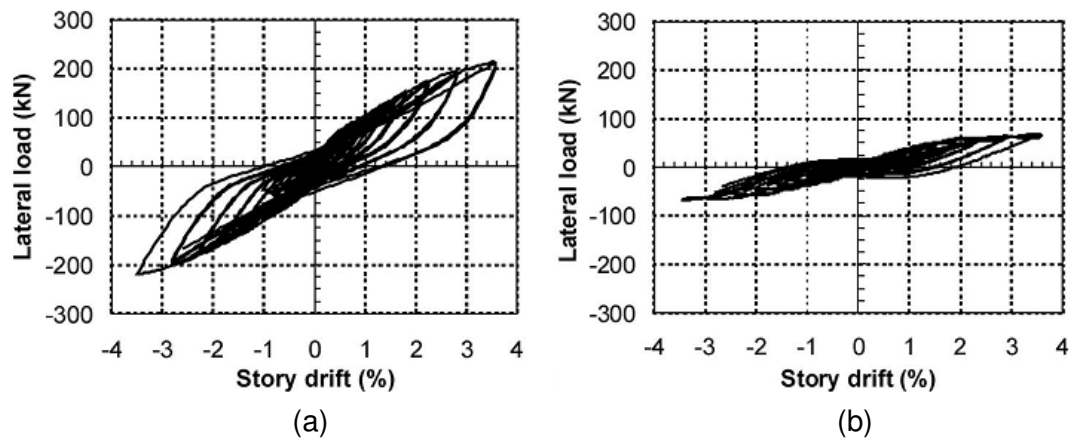


Figure 2.8: Hysteretic response of strengthened frame: (a) contributions of steel cage and AL-SYD; and (b) contribution of steel cage only (Sahoo and Rai 2010)

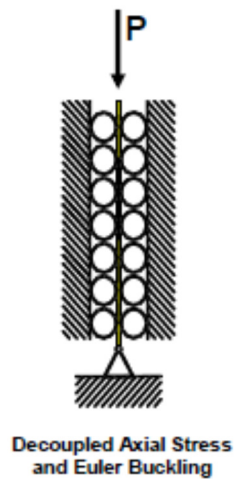


Figure 2.9: Decoupling of the axial stresses from flexural buckling stresses (Lopez and Sabelli 2004)

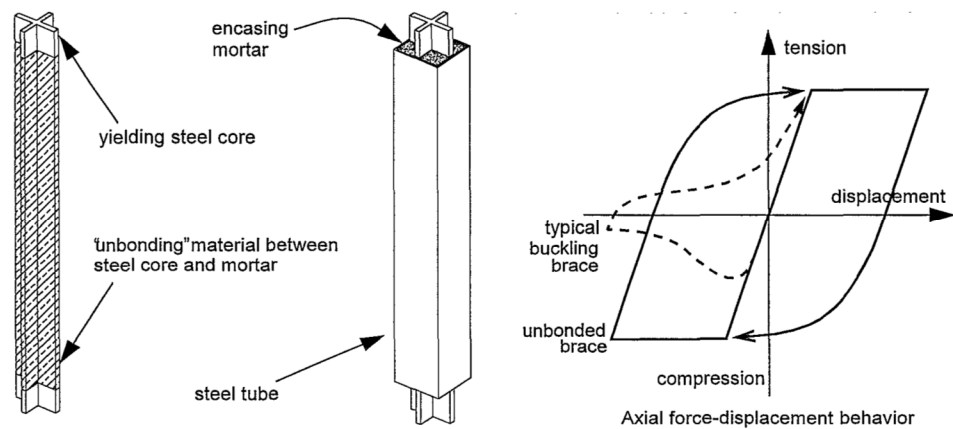


Figure 2.10: Buckling-restrained brace concept and hysteretic behaviour (Clark et al. 1999)

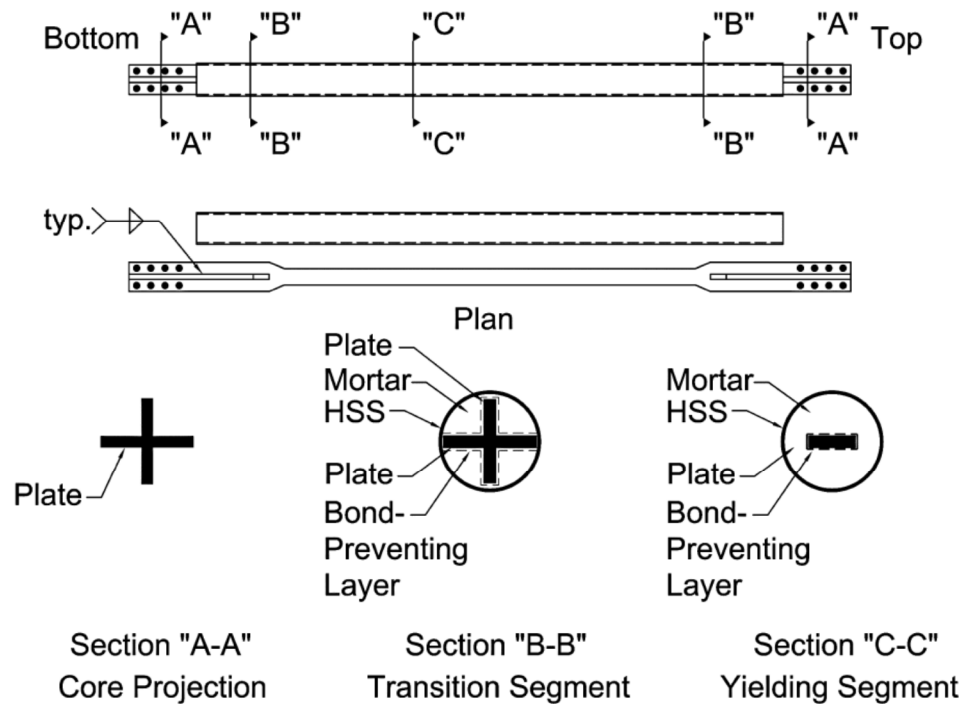


Figure 2.11: Details of buckling-restrained brace (Tremblay et al. 1999)

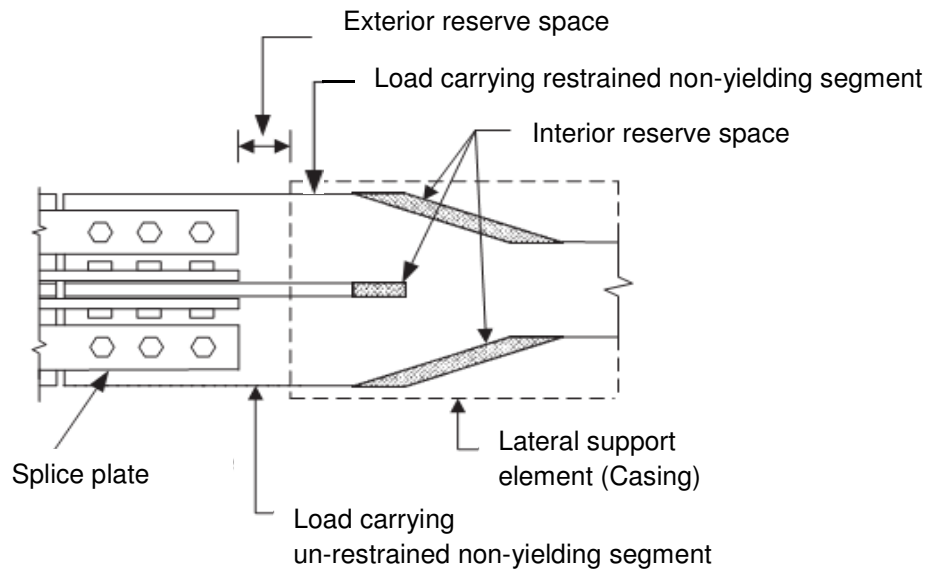
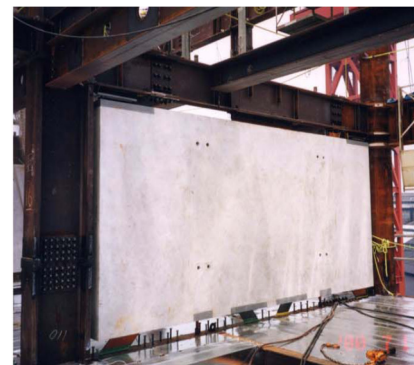
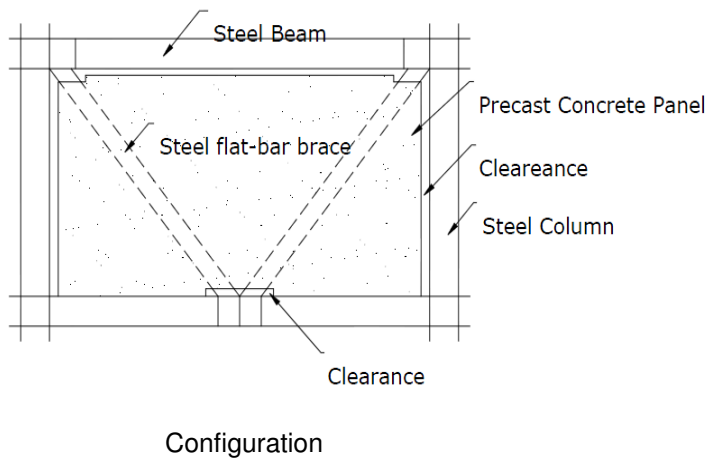


Figure 2.12: External and internal reserve spaces to prevent bearing (modified from Chen et al. 2001)



Configuration

Application

Figure 2.13: Typical configuration and application of steel brace restrained laterally against buckling by reinforced concrete panels (Escudero, 2003 and Xie 2005)

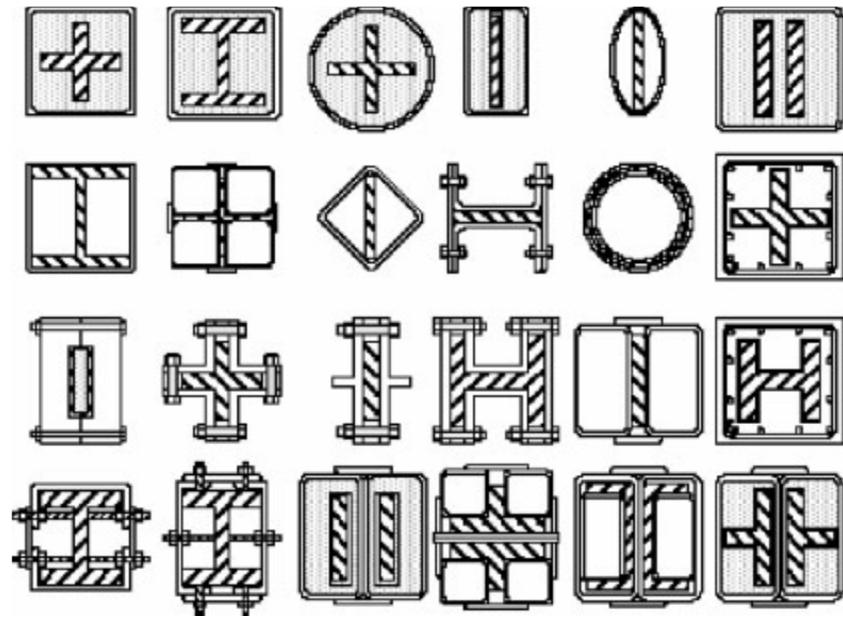


Figure 2.14: Cross sections of various types of buckling restrained braces  
(Della Corte et al. 2005)

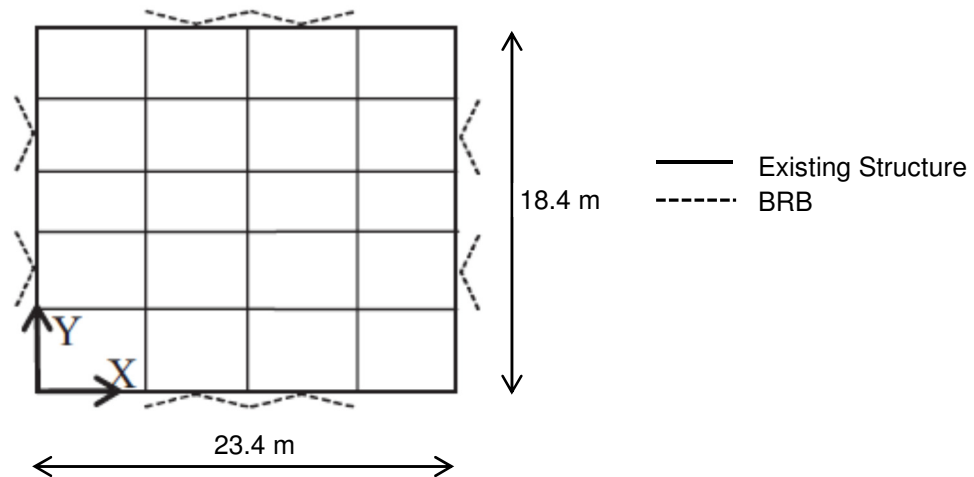
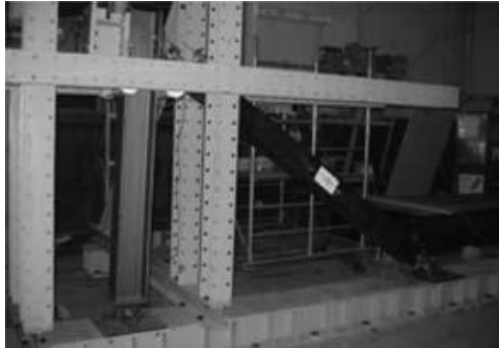
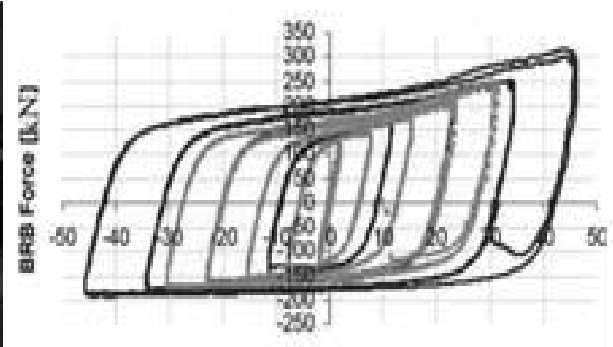


Figure 2.15: Geometry of case study building with location of the BRBs (Dinu et al. 2011)



Test setup

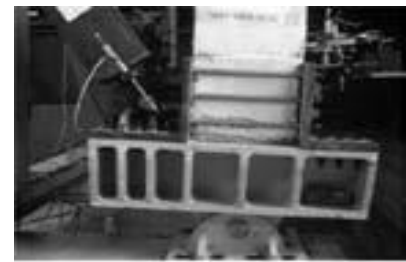


Hysteretic behavior

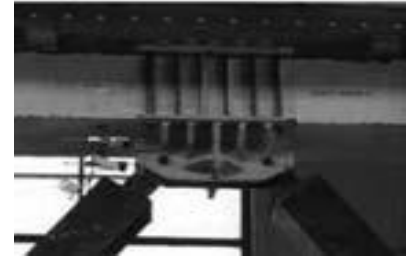
Figure 2.16: Experimental test setup and hysteretic behavior of the steel core  
(Dinu et al. 2011)



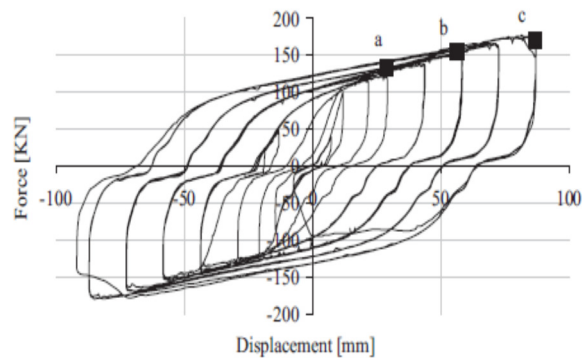
Test Setup



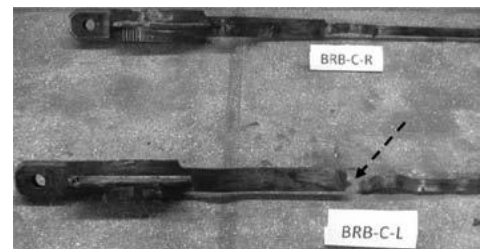
BRB connection to column



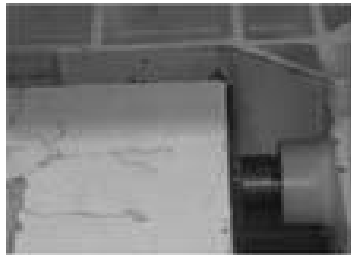
BRB connection to beam



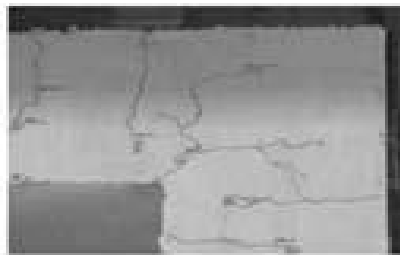
Frame hysteretic behaviour



BRB Core fracture after test



a) Cracks at yielding



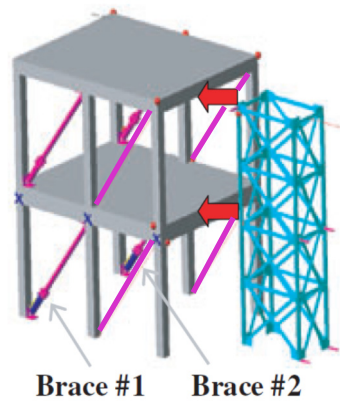
b) Cracks at 1.5 yield disp.



c) Cracks at ultimate disp.

Figure 2.17: Test setup and results (Dinu et al. 2011)





**Brace #1    Brace #2**

Test setup



Sensors at 1<sup>st</sup> floor

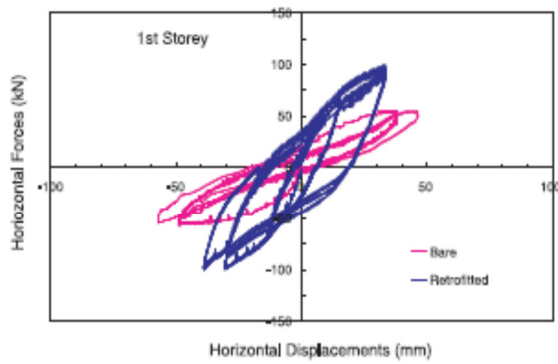


BRB connection to column

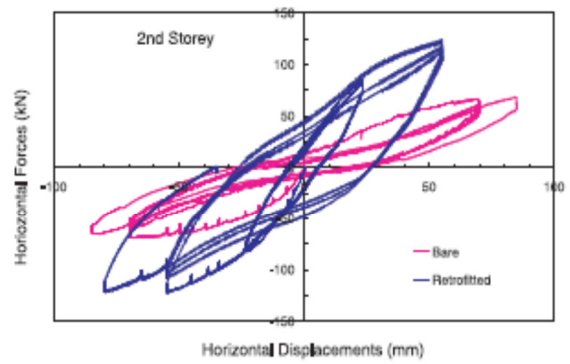


BRB end hinge

Figure 2.18: Loading setup and connection details (Sarno and Manfredi 2012)



Hysteretic force-displacement behaviour  
(1<sup>st</sup> Storey)



Hysteretic force-displacement behaviour  
(2<sup>nd</sup> Storey)



Reinforcement slip at column base:  
control building (1<sup>st</sup> Storey)



Cracking at beam-column joint:  
control building (1<sup>st</sup> Storey)

Figure 2.19: Hysteretic behaviour of the RC frame buildings and observed damage  
(Sarno and Manfredi 2012)



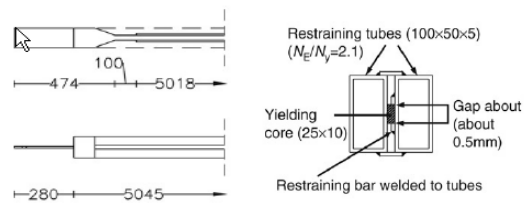
Original building



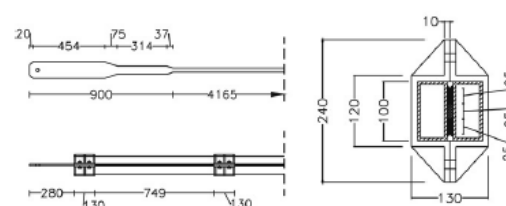
Steel loading frame



Bracing configuration



BRB section type I



BRB section type II

Figure 2.20: Building studied and configuration of the BRB retrofitting system (Mazzolani, 2008 and D'Aniello et al. 2009)

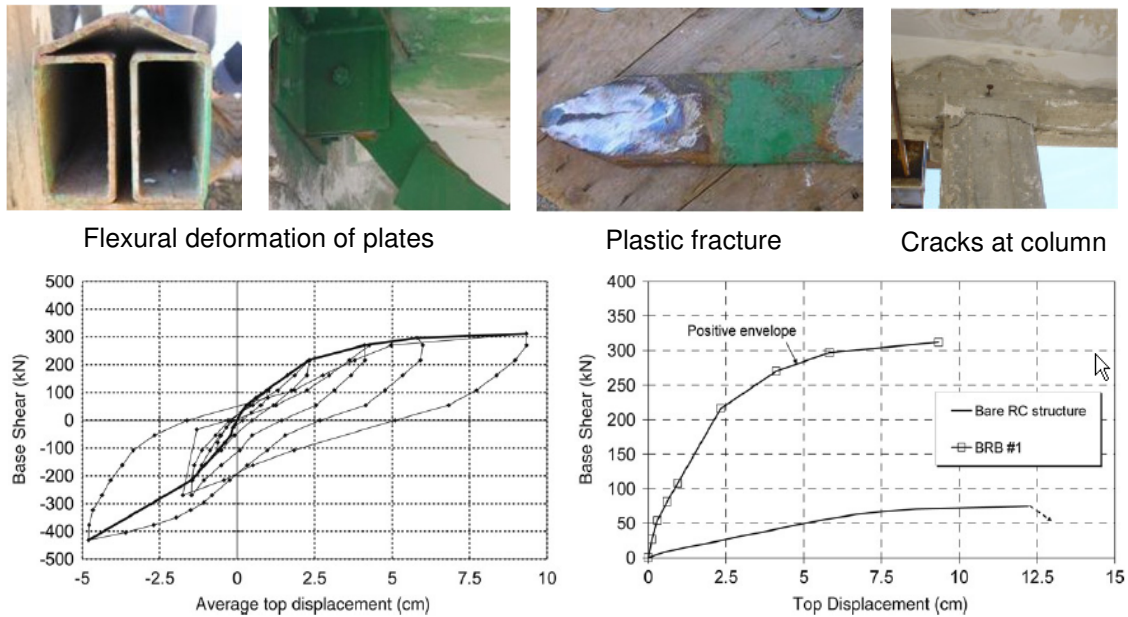


Figure 2.21: Damages and hysteretic behaviour recorded during testing structure retrofitted with BRB Type I (Mazzolani, 2008 and D'Aniello et al. 2009)

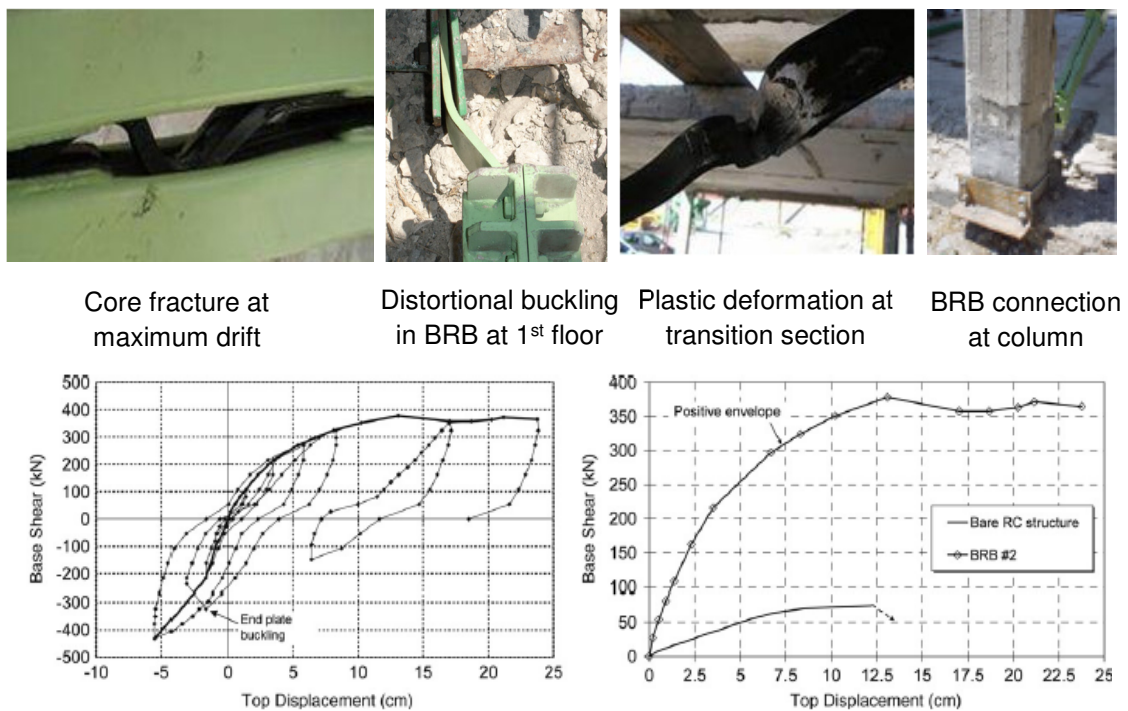


Figure 2.22: Damages and hysteretic behaviour during testing structure retrofitted with BRB Type II (Mazzolani, 2008 and D'Aniello et al. 2009)

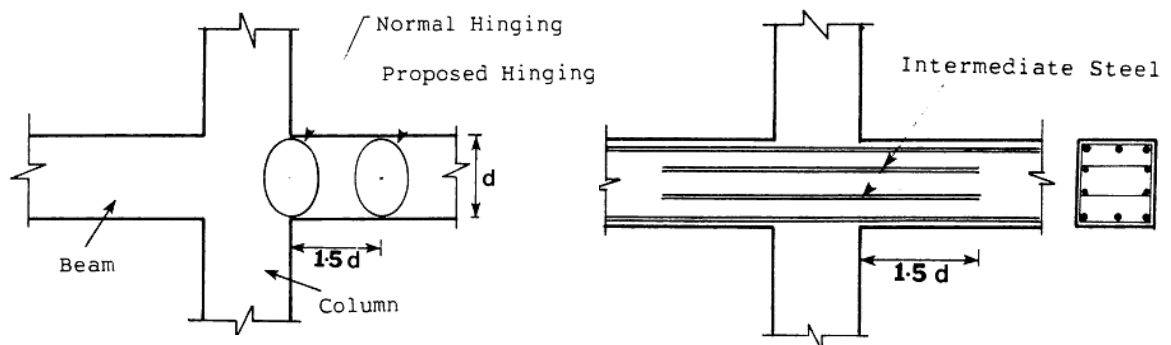


Figure 2.23: Plastic hinging relocation (Abdel-Fattah Wight 1987)

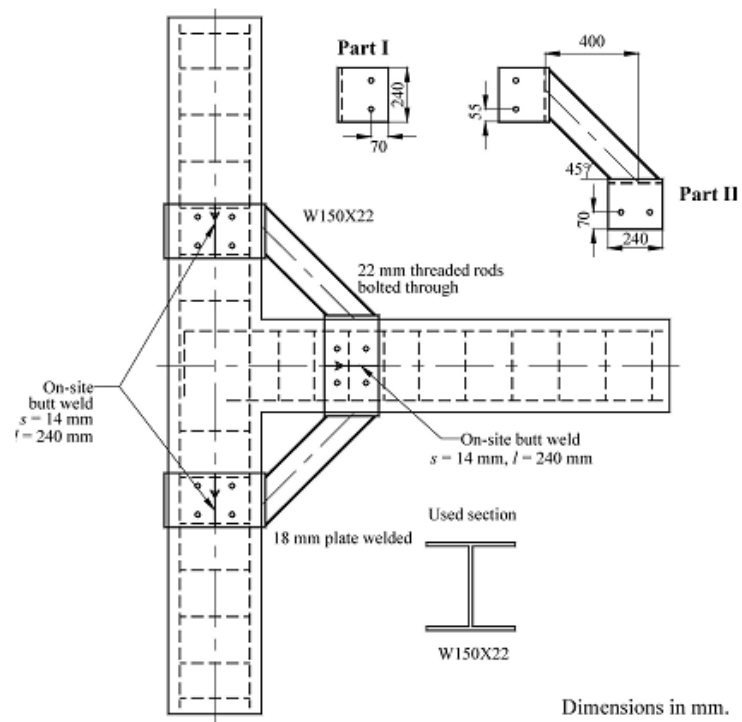


Figure 2.24: Local steel bracing retrofitting scheme (Said and Nehdi 2008)





Specimen (T0)



Specimen (J1)



Specimen (J2)

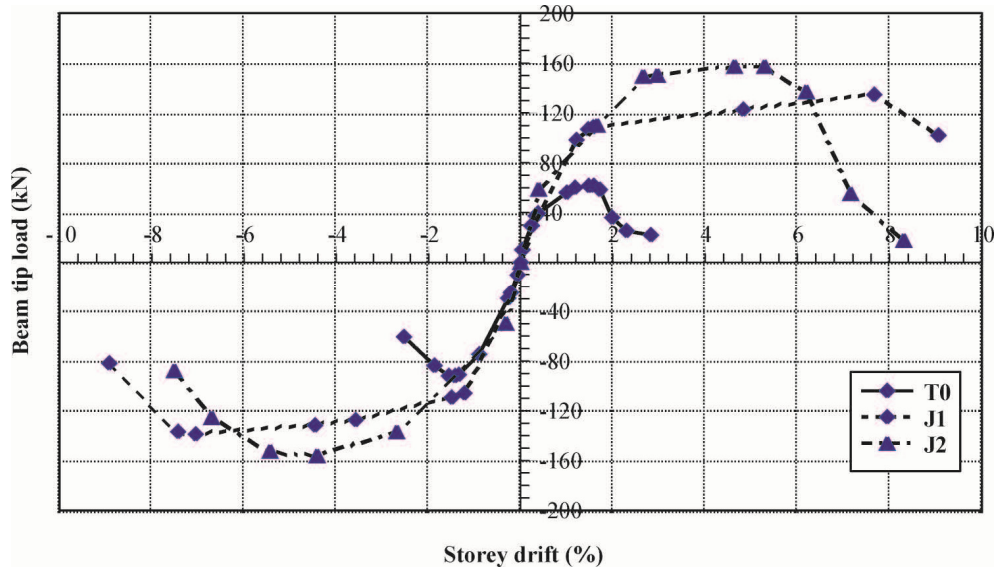


Figure 2.25: Crack patterns at failure and load-storey drift envelopes for the test (Said and Nehdi 2008)

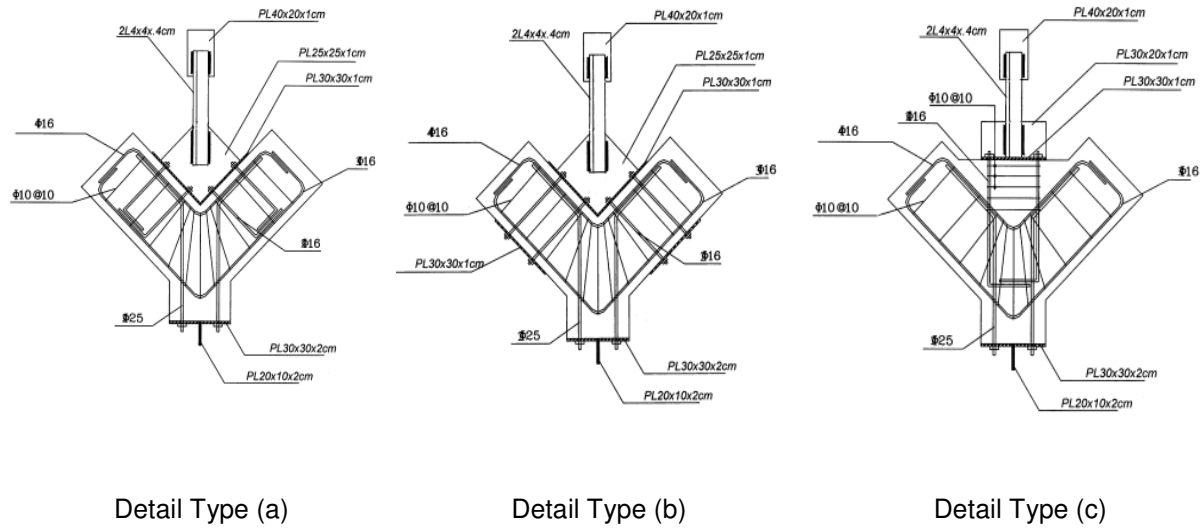


Figure 2.26: Connection details of specimens under direct tensile tests (Maheri and Hadjipour 2003)

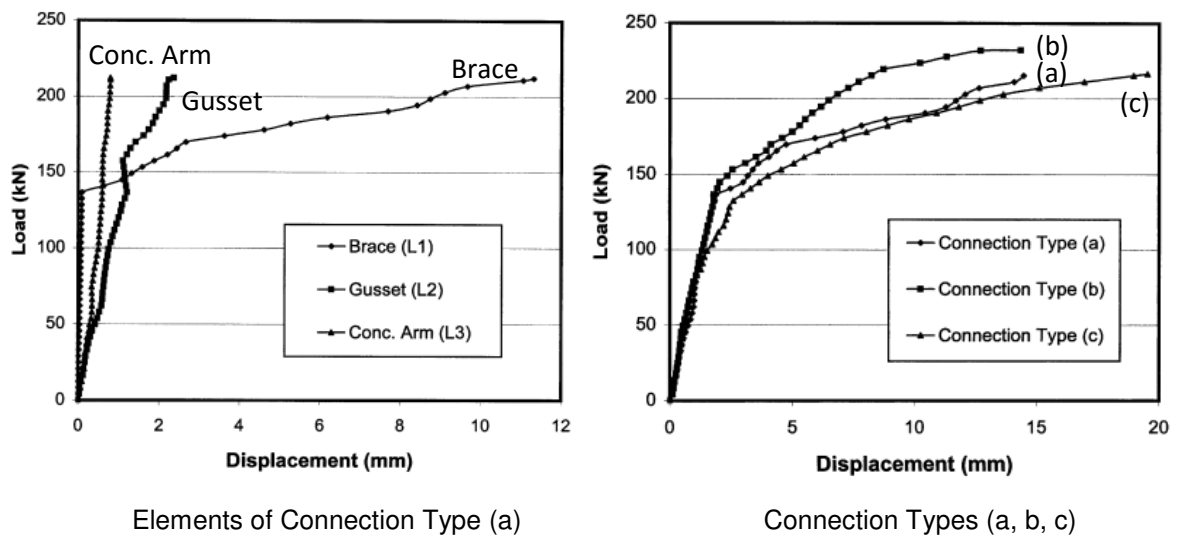


Figure 2.27: Load-displacement behaviour of the brace connections (Maheri and Hadjipour 2003)

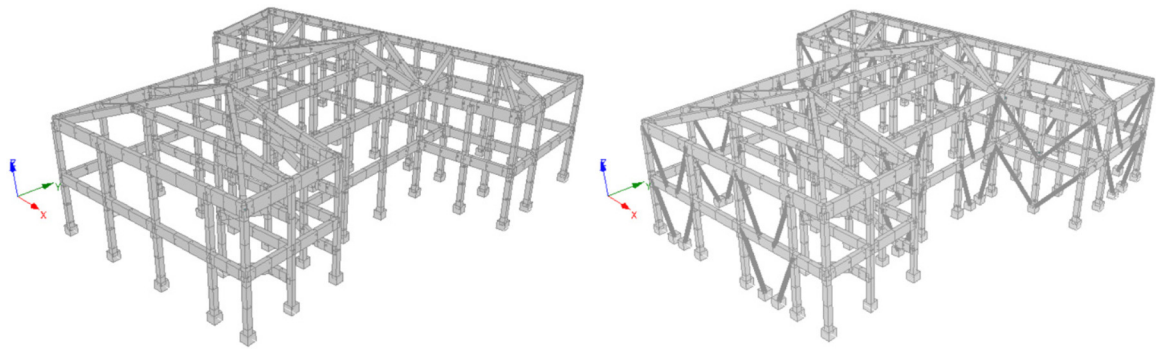


Figure 2.28: Finite element modelling of as-built and retrofitted structures (Di Sarno and Manfredi 2010)

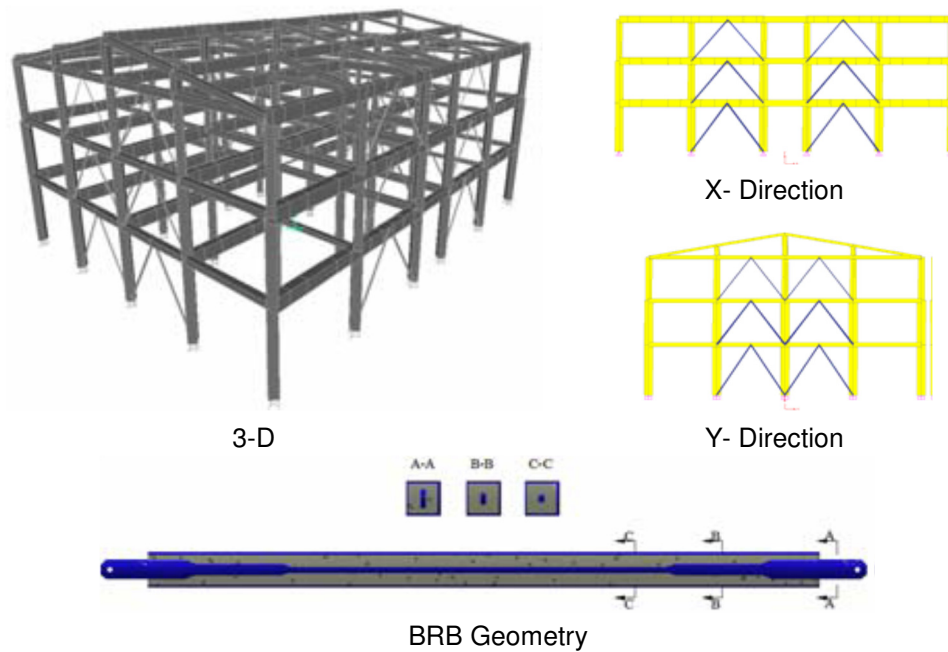
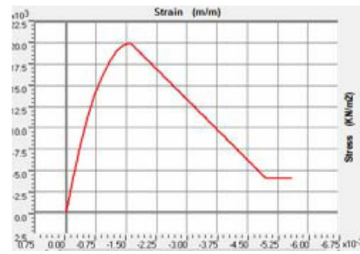
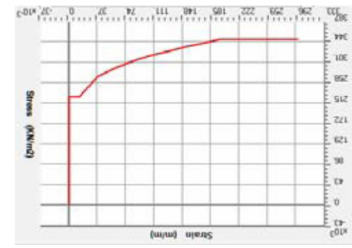


Figure 2.29: Retrofitted RC building models with locations and geometry of BRB braces (Bordea and Dubina 2009)

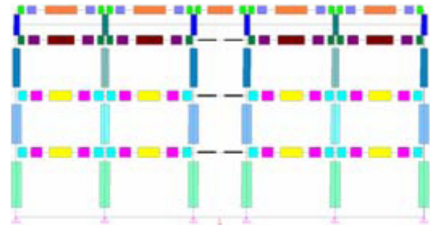




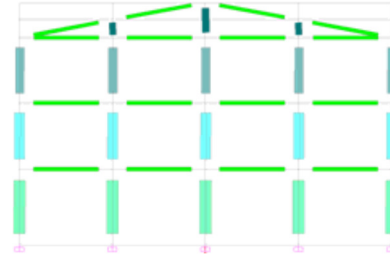
Concrete model



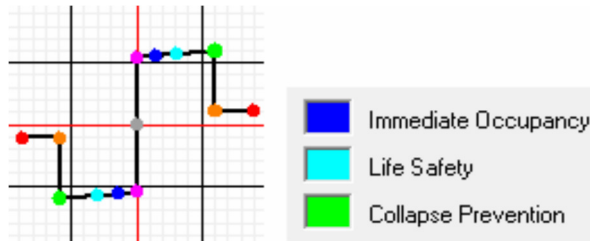
Reinforcement model



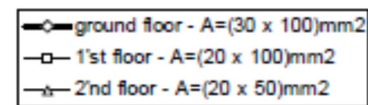
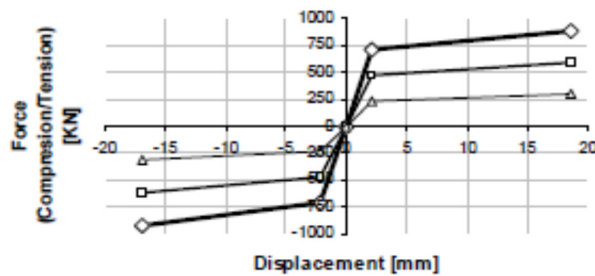
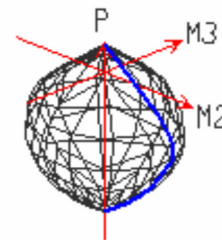
Plastic hinge locations in the X-direction



Plastic hinge locations in the Y-direction



M-θ relationship prescribed for the plastic hinges



BRB force-deformation model

Figure 2.30: Nonlinear models and plastic hinge locations for the analytical numerical models (Bordea and Dubina 2009)

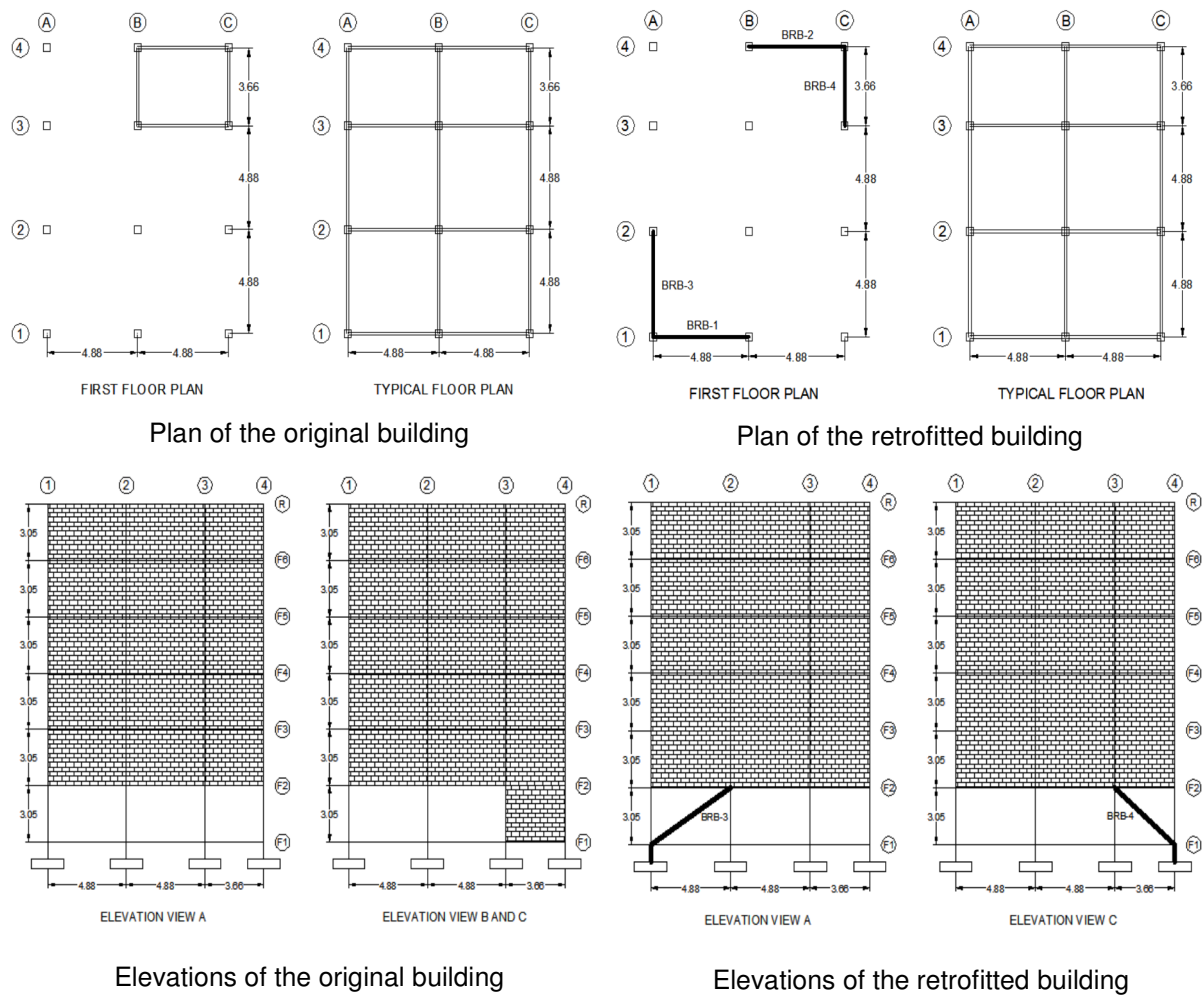
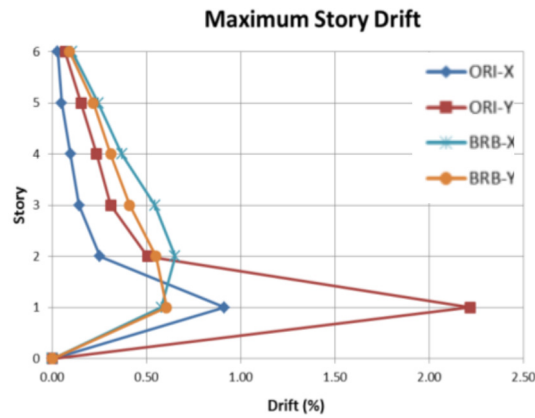
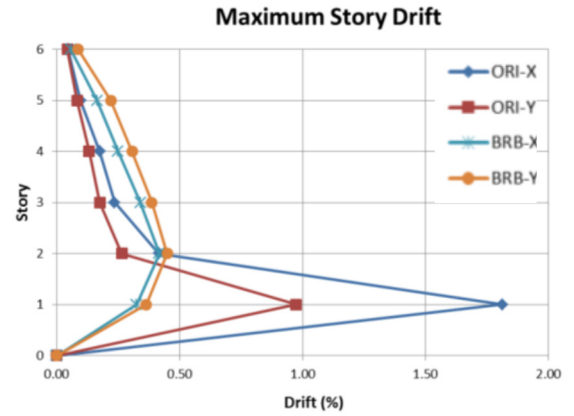


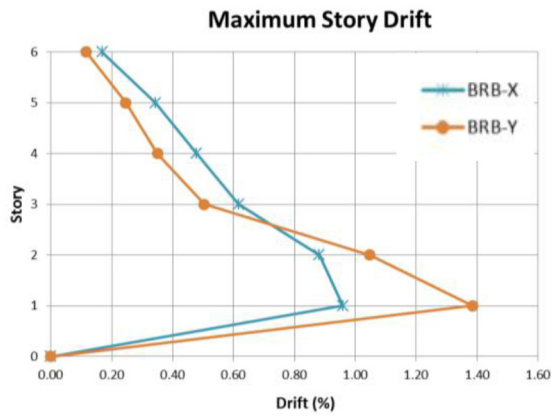
Figure 2.31: Plans and elevations of the original and retrofitted BRB buildings (Chandra, and Warnitchai 2011)



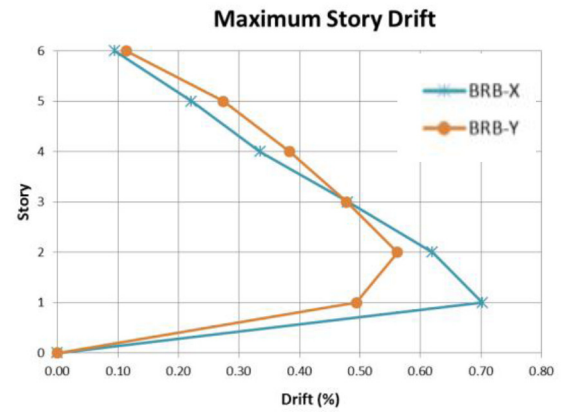
Moderate earthquake, SF 1971



Moderate earthquake, NR 1994



Strong earthquake, SF 1971



Strong earthquake, NR 1994

Figure 2.32: Comparison of storey drifts of the original (ORI) and retrofitted (BRB) buildings in the X and Y directions for selected earthquake records (Chandra and Warnitchai 2011)

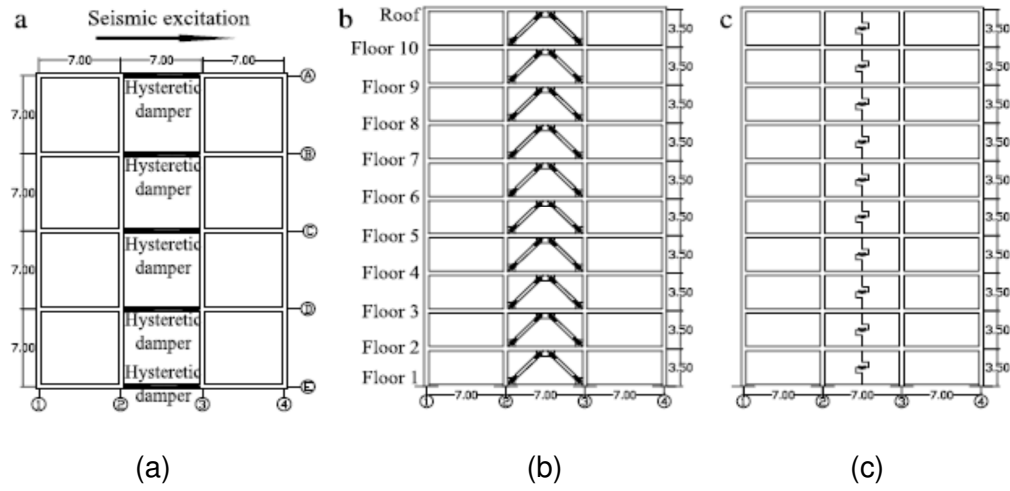


Figure 2.33: The studied building: (a) typical plan view; (b) elevation view with BRBs; and (c) elevation view with generic hysteretic dampers (Oviedo et al. 2010)

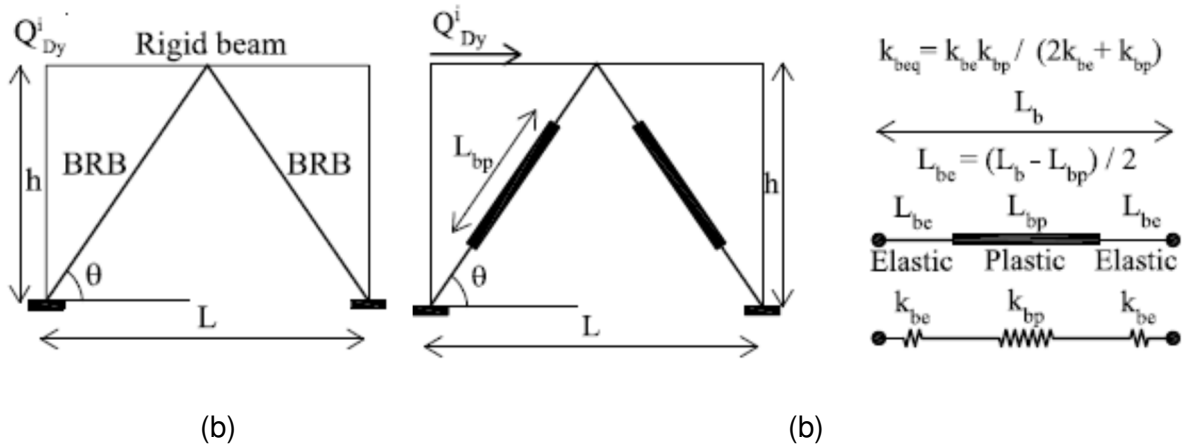


Figure 2.34: BRB modeling: (a) truss elements; and (b) suggested idealized BRB modeling (Oviedo et al. 2010)

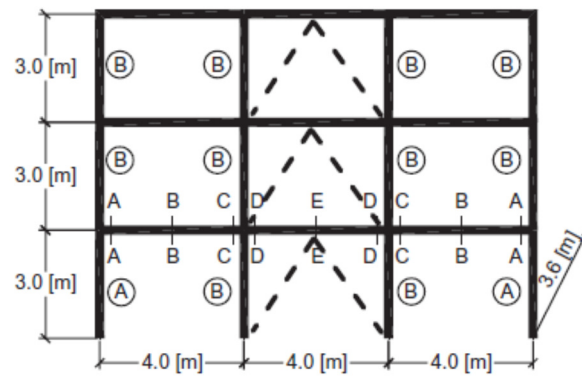


Figure 2.35: Case study building geometry (Dinu et al. 2011)

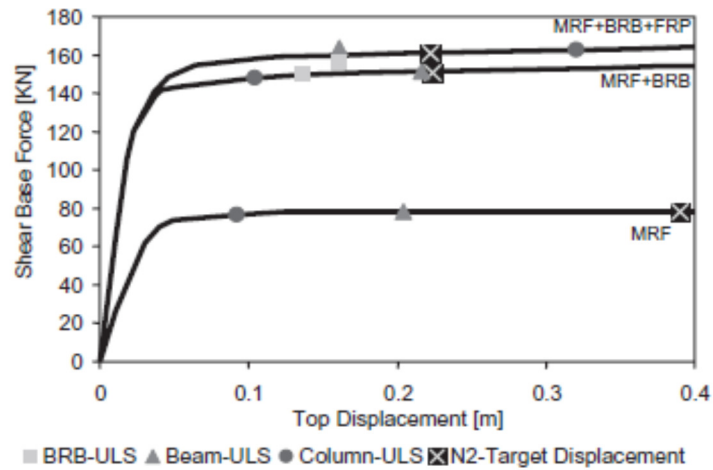
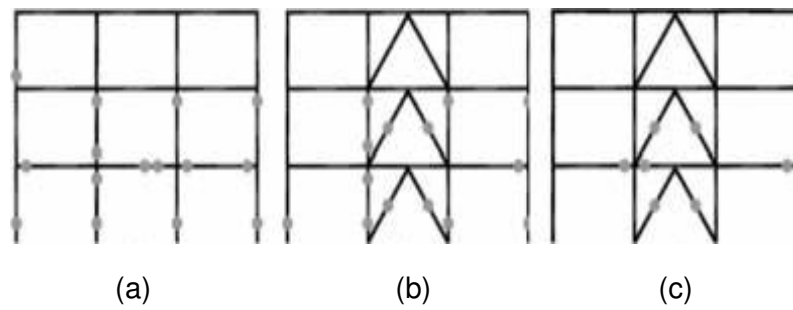


Figure 2.36: Numerical models with locations of plastic hinges and responses: a) MRF; b) MRF+ BRB; c) MRF+ BRB+ FRP; and d) load-displacement behaviours (Dinu et al. 2011)

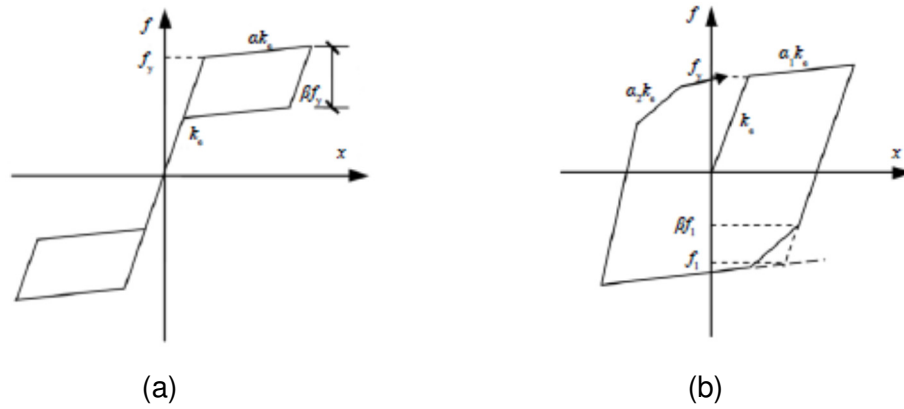


Figure 2.37: Hysteretic models employed in the analysis of the retrofitting schemes: a) self-centering flag-shaped response; and b) BRB bilinear response (Zhang and Hu 2010)

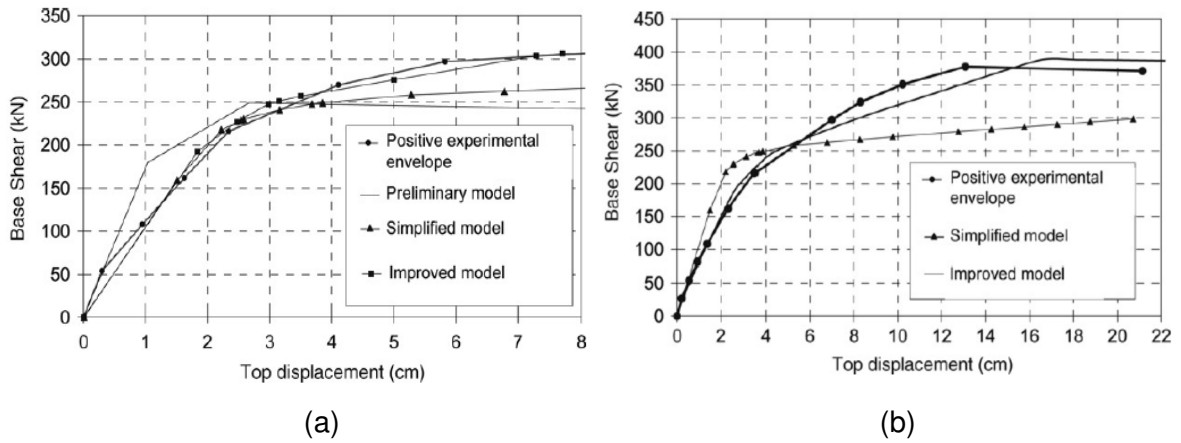
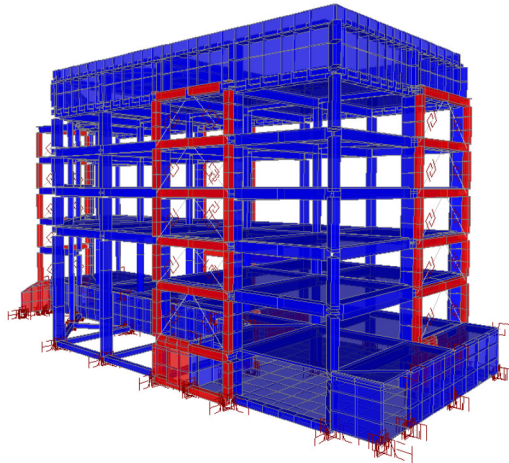


Figure 2.38: P-Δ responses of the experimental and numerical models of the BRB retrofitted frames: a) BRB type I; and b) BRB type II (Mazzolani 2008)

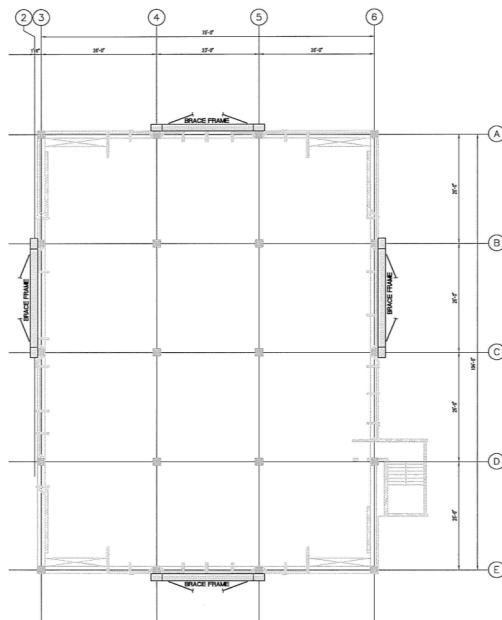


(a)



(b)

Figure 2.39: Building retrofitting scheme: a) numerical model; and b) on-site construction (Skokan et al. 2010)



(a)



(b)

Figure 2.40: Building retrofitting scheme: a) floor typical plan; and b) retrofitted building (Islam et al. 2010)



## **Chapter 3**

# **Evolution of Canadian Seismic Building Codes for Concrete Construction**

### **3.1 General**

This chapter describes the development of seismic design provisions of the National Building Code of Canada (NBCC) for the construction of buildings from 1941 to the present. Emphasis is placed on reinforced concrete buildings given their widespread use in Canada. Furthermore, older concrete buildings have been documented to be vulnerable to current seismic demands. The development of design loads for the working stress and limit states design procedures are discussed and significant factors affecting the seismic design forces are highlighted. Fundamental components to the seismic provisions discussed herein include: development of the regional seismicity maps, classification of structural systems, introduction of ductility-related provisions, and the shift of seismic analysis towards the modal dynamic method. This chapter concludes with a comparison of base shear design forces for non-ductile six and ten storey buildings based on the NBCC from 1941 to present.

This chapter serves to provide an understanding of the required increases in strength and/or ductility for upgrading a deficient concrete frame building to current seismic requirements.

### **3.2 Introduction**

Earthquake design provisions have changed significantly since the first edition of the NBCC. The updating of provisions was generally in response to major damaging earthquakes that have been experienced in high seismic hazards areas of Canada and around the world. The seismic provisions prescribing the lateral base shear equations to which buildings should be designed and detailed have been extensively modified as a result of research conducted to reduce its complexity and to account for differences in regional seismicity hazards, construction materials and structural system types.



### 3.3 NBCC Developments

A number of developments are summarized herein: seismic zoning maps, torsional effects, foundation and building importance factors, force modification factors related to ductility provisions, structural systems with irregularities, dynamic analysis, and the implementation of SI units. These developments are discussed for each version of the NBCC.

#### 3.3.1 NBC 1941

The National Building Code (NBC) was first published in Canada in 1941. It was prepared under joint sponsorship of the National Housing Administration and National Research Council of Canada.

The seismic provisions were based on the 1937 edition of the Uniform Building Code and appeared in Appendix H of the code. The seismic force was stated as provided in Equation 3.1. The horizontal force  $F$  was assumed to act at the centre of gravity of the structure and was proportional to the weight of the structure ( $W$ ), taken as the dead load plus half the live load. The proportional factor ( $C$ ) ranged between 0.02 and 0.05 depending on the allowable bearing capacity of the soil and the type of structure. There was no consideration for regional seismological intensity.

$$[3.1] \quad F = CW$$

The Canadian Standard Association (CSA) standard A23.3 for Design of Concrete Structures was first published in 1959 under the name, “Code of recommended practice for reinforced concrete design”. In NBC 1941, the design requirements of A23.3 were embedded in the NBC in Section 3.4, Reinforced Concrete Construction. The allowable stress design method was the standard practice at the time. All structural members were required to be designed, built and maintained under Dead ( $D$ ) and Live ( $L$ ) loads ensuring that member stresses were within code permitted stresses. The live loads were the sum of loads arises from occupancy (called floor loads), ceiling, roof, wind, earthquake, and other special loads. A reduction of 50% of floor loads was permitted when the load effects of wind and earthquake were combined within the live load category, provided that the stresses would not be less than the combination of dead and

live loads other than wind. The design loads were based on the combined working lateral and vertical loading combinations and were given as:

$$[3.2] \quad \text{Design Loads} = D + L$$

Note that the wind and earthquake loadings were included within the live load (L).

### **3.3.2 NBCC 1953**

The word Canada was incorporated in the second edition of NBC in 1953 (1953 NBCC). The 1949 edition of the Uniform Building Code (UBC) formed the basis of seismic provisions for the 1953 NBCC. Two new developments were introduced. The first was the introduction of the Canadian seismic zoning map that divided the country into four earthquake intensity zones (0 to 3) based on qualitative assessment of historical earthquake activity as shown in Figure 3.1. These zones remained in force until a revised zoning map of Canada was published in the 1970 NBCC. The second addition was intended to recognize the influence of building flexibility through the factor C that was set equal to  $[0.15/(N+4.5)]$  for buildings in zone 1. The two enhancements were explicitly embedded in the C factor through the modification of C by the seismic regionalization factor; i.e. multiplying the calculated C value by 2 for seismic zone 2, and by 4 for zone 3. The N factor accounted for the number of storeys above the level under consideration, which was used to incorporate the flexibility of the structure.

The lateral seismic force equation remained unchanged from the 1940 NBC; however, W was taken as the dead load plus 25% of the design snow load, instead of dead load plus half the live load as prescribed in the 1941 NBC. In addition, machinery or other fixed loads were considered part of the dead load. The soil condition factor dependency in the C factor of Equation 3.1, which appeared in the 1941 NBC, was ignored in this edition of the NBCC.

The design of concrete structures followed the allowable stress design method. However, wind and earthquake loadings were excluded from the live load category; this was a notable change relative to the 1941 NBC. These loads were considered as independent load cases.

Structures designed according to this code were required to be investigated for the combined effect of all possible vertical and lateral loadings similar to earlier codes. However, two substantial changes were incorporated: (1) an allowance of 33% for the stresses was permitted when the stresses were attributed to wind or earthquake forces; and (2) wind and earthquake loads were not to be considered simultaneously. Equation 3.3 provides the design load that account for earthquake effects:

$$[3.3] \quad \text{Design Loads} = D + L + E$$

Where: D, L and E are the load effects from dead, live, and earthquake, respectively.

### **3.3.3 NBCC 1960**

The seismic design provisions of the third edition of the NBCC (1960) remained the same as those of the 1953 NBCC. For the first time, dynamic analysis was permitted as an alternative analysis approach, provided a competent person in this field of work performed such analysis. Furthermore, tabulated design data for locations covering Canada in the climatic information supplement of the code first appeared. Earlier codes only provided charts.

### **3.3.4 NBCC 1965**

The 1965 NBCC seismic requirements were influenced by the SEAOC (Structural Engineers Association of California) and UBC. The adopted base shear equation was similar to that of the 1955 UBC, while the distribution of the horizontal forces over the building height, for the equivalent static procedure, was similar to that of the 1961 UBC (Uzumeri et al 1977).

The concept of the ductility factor was first introduced in this edition of the NBCC through the C factor; however, this was specified without any detailing for the structure. The seismic base shear, V, equation was stated as follows:

$$[3.4] \quad V = R C I F S W$$

where R = seismic zoning factor of 0, 1, 2 and 4 for seismic intensity zones 0, 1, 2 and 3, respectively; C = type of construction ( 0.75 for moment resisting frame and shear wall

buildings that are adequately reinforced to behave in a ductile fashion, and 1.25 for non-ductile structures);  $I$  = importance factor (1.3 for buildings of with large assembly occupancies and 1.0 for all other buildings);  $F$  = foundation factor (1.5 for buildings on highly compressible soil and 1.0 for other subsoil conditions);  $S$  = structural flexibility factor and determined from  $0.25/(9+N)$ , where  $N$  is the total number of storeys; and  $W$  = dead load plus 25% design snow load, plus live loads for storage areas.

The effects of torsion were first included in the 1965 NBCC seismic design provisions. The code explicitly prescribed an expression to determine the design eccentricity ( $e_x$ ) of a structure as follows:

$$[3.5] \quad e_x = 1.5 e \pm 0.05 D_n$$

Where  $e$  is the distance between centre of mass and centre of rigidity of the building, and  $D_n$  is the building plan dimension. Both  $e$  and  $D_n$  are perpendicular to the direction of the earthquake loading. If the total torsional eccentricity exceeded  $D_n/4$ , either a dynamic analysis was required or the effects of torsion would need to be doubled for static analysis.

The allowable stress design method and loading combinations generally remained similar to the 1953 and 1960 editions of NBCC. An exception was a reduction factor of 25% when earthquake loads were considered to act simultaneously with other loads. However, the design values were not to be taken less than those resulting from the combination of dead load plus earthquake loads alone in assessing structural adequacy by means of the working stress design. Equation 3.6 provides the load combinations according to the 1965 NBCC:

$$[3.6] \quad \begin{aligned} \text{Design Loads} &= D+E \\ \text{Design Loads} &= 0.75 (D+L+E) \end{aligned}$$

In this edition of the code, the ultimate strength design method was first introduced for the design of reinforced concrete structures as an alternative method. The code was designed in general agreement with American Concrete Institute ACI Standard 318-63

Building Code Requirements for Reinforced Concrete, except for revisions made by the Joint Canadian Standards Association/NBC Committee on Reinforced Concrete Design.

The ultimate load,  $U$ , with Earthquake (E) loads and earth pressure (S) was given as:

$$\begin{aligned} [3.7] \quad U &= 1.35 (D+L+ E+S); \text{ but not less than} \\ U &= 1.5 (D) + 1.8 (L+S) \end{aligned}$$

In addition, reduction factors ( $\Phi$ ) were introduced to compute the ultimate strength capacity of structural members. The factors were taken as: 0.90 for flexure; 0.85 for diagonal tension, bond and anchorage; and 0.75 for all column types.

### 3.3.5 NBCC 1970

The seismic zoning map was completely revised in this fifth edition of the NBCC. It was based on the work of Milne and Davenport (Mitchell et al. 2010) as shown in Figure 3.2. The boundaries of the four zones (0, 1, 2, and 3) were based on peak acceleration amplitudes of 0.01 annual probability of exceedance. The minimum lateral seismic base shear force includes non-dimensional multipliers (0 for zone 0, 1 for zone 1, 2 for zone 2, and 4 for zone 3). The earthquake occurrence probability of regions in British Columbia and north of Quebec City were generally comparable to that of California.

The concept of ductility was introduced in the 1965 NBCC by specifying two values in the  $C$  coefficient. Ductility was further detailed in the 1970 NBCC with the introduction of a new  $K$  factor. The previous code flexibility factor,  $S$ , was removed and the effect of building height was embedded in the new  $C$  factor equation, which appeared as a function of building period for the first time in history of the NBCC. The minimum lateral seismic base shear force,  $V$ , equation was given as:

$$[3.8] \quad V = \frac{1}{4} R K C I F W$$

Where  $R$ ,  $I$ ,  $F$ , and  $W$  remained unchanged from the 1965 NBCC;  $K$  = coefficient for construction type of buildings (0.67 for buildings with a ductile resisting space frame, 0.8 for ductile buildings with dual systems, 1.0 for other systems, and 1.33 for buildings with box systems);  $C$  = structural flexibility factor taken as 0.1 for all one and two storey

buildings and  $0.05/T^{1/3}$  (but not to exceed 1.0) for all other buildings. The building period (T) was expressed as an empirical equation, which was introduced in the 1959 edition of SEAOC as a function of building height. It was been specified as: (1) 10% of the number of storeys (N) for buildings resisting 100% of the lateral loads; or (2) 5% of building height (ft) divided by the square root of the dimension of the building (ft) parallel to the seismic force for all other buildings.

To account for higher mode effects, the 1970 NBCC required a portion of the lateral seismic force, V, to be concentrated at the top of the structure and was termed  $F_t$ . The remainder of lateral forces ( $V - F_t$ ), were required to be distributed along the height of the building.  $F_t$  was specified as:

$$\begin{aligned} [3.9] \quad F_t &= 0.004 V (h_n/D_s)^2 \leq 0.15 V, \text{ if } (h_n/D_s) > 3 \\ F_t &= 0, \text{ if } (h_n/D_s) \leq 3 \end{aligned}$$

Where  $h_n$  is the height of the structure in ft and  $D_s$  is the dimension of the lateral force resisting system, in (ft), in a direction parallel to the applied forces. These equations remain in effect until the 1995 edition of the NBCC.

New reduction factors, J and  $J_x$ , were introduced to modify the overturning moments M and  $M_x$  at the base of the structure and at any level (x), respectively, as follows:

$$\begin{aligned} [3.10] \quad J &= 0.5 + 0.25/(T^2)^{1/3} \leq 1.0 \\ J_x &= J + (1-J) (h_x/h_n)^3 \end{aligned}$$

Where  $h_x$  is the height, in (ft), of the level of the structure under consideration.

The torsional effects of the 1970 NBCC remained similar to the 1965 NBCC except that the design eccentricity ( $e_x$ ) of Equation 3.5 was stated without “the negative sign” as provided in Equation 3.11:

$$[3.11] \quad e_x = 1.5 e + 0.05 D_n$$

Additional loading combinations (Equation 3.12) were introduced for the working stress design method when combining vertical and horizontal loads with environmental effects (T). These effects could result in either contraction or expansion of structural members.

$$[3.12] \quad \text{Design Loads} = 0.66 (D + L + E + T)$$

These working stress load combinations remained valid until the 1995 NBCC where a modifier of 2/3 was applied directly to earthquake loads. Furthermore, snow loads were incorporated into live loads. This remained in effect until the the 1995 NBCC. During this code cycle, the working stress design method was no longer acknowledged.

The first reference to the stand-alone CSA A23.3-1970 (Code for the Design of Plain or Reinforced Concrete Structures) appeared in the 1970 NBCC. CSA A23.3-1970 was contained in the NBCC Supplement No. 4 of the Canadian Structural Design Manual (1970). For the ultimate strength design method, the ultimate load, U, including earthquake was slightly modified according to the following new load combinations that were specified to be the greater of:

$$\begin{aligned} [3.13] \quad U &= 1.15 D + 1.35 (L + E) \\ U &= 1.5 D + 1.8 E \\ U &= 0.9 D + 1.35 E \end{aligned}$$

The capacity reduction factors,  $\Phi$ , remained similar to the 1965 NBCC ultimate strength design approach.

### **3.3.6 NBCC 1975**

In the 1975 NBCC, the minimum design seismic base shear force, V, was given as:

$$[3.14] \quad V = A S K I F W$$

Where I and W remained unchanged from the 1965 and 1970 NBCC; A is equal to the horizontal design ground acceleration (0.0, 0.02, 0.04, and 0.08 for seismic zones 0, 1, 2 and 3, respectively, of the 1970 NBCC regional seismic zones); and S is the seismic response factor, which reflects the dependency of the acceleration on the fundamental

period of the structure ( $T$ ). The period was set equal to  $0.5/(T)^{1/3}$  with a lower limit of 1. The period calculation was similar to the 1970 NBCC. The ductility factor  $K$  depends on construction type and was expanded to 7 building system categories compared to 4 categories in the 1970 NBCC. The basic concept of this  $K$  factor was similar to that in the 1976 UBC. Essentially, it accounts for the energy-absorptive capacity of the structural system through damping and inelastic action (1970 NBCC Supplement No. 4) as provided in Table 3.1. However, the classification and values are quite different for each building category and the  $K$  values assigned to buildings are lower than those assigned to other structures. The  $K$  factors remained unchanged until the 1990 NBCC. A third intermediate value of 1.3 for the foundation factor  $F$  depending on type and depth of soil was introduced in this edition of the NBCC.

In comparison to the base shear equation of the 1970 NBCC code, application of the 1975 NBCC design base shear force to a ductile moment resisting frame building located in seismic zone 3 results in a reduction of 20% (Uzumeri et al. 1978 and Heidebrecht 2003). The combination of  $A$  and  $S$  in Equation 3.14 are 20% less relative to the term  $(RC/4)$  that appeared in the base shear equation (Equation 3.8) of the 1970 NBCC (Mitchell et al. 2010).

The distribution of the lateral seismic force following the equivalent static load procedure remained similar to that used in the 1970 NBCC, which was adopted from the 1961 UBC.

The overturning moment reduction factor  $J$  was adjusted upwards; it was realized that small values for  $J$  were not justified. Other examples that followed this trend included SEAOC, and the New Zealand Code (1970 NBCC Supplement No. 4). The  $J$  coefficient, that is used to modify the base overturning moment of the structure, was based on the fundamental period ( $T$ ) of the structure and set equal to: 1 when  $T$  is less than 0.5;  $1.1 - 0.2T$  when  $T$  is at least 0.5, but not more than 1.5; and to 0.8 where  $T$  is greater than 1.5.

The 1975 NBCC introduced an additional equation (Equation 3.15) to that previously provided in the 1970 NBCC to estimate the torsional eccentricity. The design eccentricity at each floor was taken as the greater of the two equations. The new equation



introduced 0.5 factor to the eccentricity (e) compared to 1.5 in the 1965 NBCC and the 1970 NBCC, as an intermediate value for design forces at the stiff side of the structure.

$$[3.15] \quad e_x = 0.5 e - 0.05 D_n$$

Where e is the computed eccentricity between the centre of mass and centre of rigidity of the building and  $D_n$  is the building plan dimension in the direction of the eccentricity.

Dynamic analysis was first detailed in in Commentary K of the 1975 NBCC code as the method to use in the design of complex structural configurations, for which the equivalent static procedures of the NBCC would be inadequate. For regular buildings, the equivalent static method and the recommended alternative dynamic procedure should yield similar results. A probability of exceedance of 0.01 per annum ( $A_{100}$ ) was taken as the standard hazard level for the peak ground acceleration. Damping-dependent amplification factors were used to scale the peak ground motion. A ductility factor  $\mu$  was suggested for various structures ranging from 4 for ductile moment resisting frames to 1 for structures lacking any ductility (see Table 3.2). In addition, the concept of “equal displacement theory” was employed to account for inelastic deformations. For this approach, the elastic spectral acceleration was divided by  $\mu$  for long period structures and by  $(2\mu - 1)^{1/2}$  for shorter period structures. For un-detailed analysis of the influence of soil conditions as well as accounting for the importance of the structure, the commentary suggested multiplying the average response spectrum by the foundation factor (F) and the importance factor (I) defined in the equivalent static force procedure. The commentary required dynamic design forces and inter story drifts to be calculated by the Square Root of Sum of Squares (SRSS) modal combination method. This modal combination method was removed in the 1977 NBCC.

Limit states design was first introduced in the 1975 NBCC as an alternative design approach to the working stress design method. For buildings other than concrete, load factors and combinations were stated as:

$$[3.16] \quad \begin{aligned} U &= 1.25 D + 0.7 (1.5 L + 1.5 E) \\ U &= 1.25 D + 1.5 E \\ U &= 0.85 D + 1.5 E \end{aligned}$$

These load combinations were incorporated in the CSA S16.1, Limit States Design of Steel Structures, starting in 1974. In addition, these combinations remained in effect until the 1990 NBCC where the load factor of 1.5 for earthquake (E) was reduced to 1.0.

For the design of concrete buildings, an exception to the above load combinations was provided in the 1975 NBCC and a reference was made to CSA A23.3-73. The ultimate strength load factors were given in the standard; whereas the load combinations were provided in the 1975 NBCC. The result was the following critical combinations that considered the effects of earthquake:

$$\begin{aligned}[3.17] \quad U &= 0.75 (1.4 D + 1.7 L + 1.8 E) \\ U &= 1.4 D + 1.8 E \\ U &= 0.9 D + 1.4 E\end{aligned}$$

It is apparent the limit state load factors and load combinations in use for the ultimate strength design in CSA A23.3-1973 are higher than those prescribed in NBCC (Equation 3.16) for materials other than concrete. These limit state load factors and load combinations remained unchanged until CAN-A23.3-M84 was released.

The capacity reduction factors,  $\Phi$ , are no more placed in this edition of the NBCC but are explained instead in the CSA A23.3-1973. The  $\Phi$  factor of “all column types” of 0.75 stated in 1965 and 1970 NBCC’s was updated to 0.75 for spiral columns and 0.7 for other reinforced members under axial compression.

### **3.3.7 NBCC 1977**

No major changes to the seismic provisions were incorporated in this edition of the NBCC relative to the 1975 NBCC. However, a limit on the base shear force computed through dynamic analysis was put into place. The dynamic base shear force was restricted to at least 90% of that obtained by the equivalent static force procedure. This was the result of evidence showing that for large building periods the static analysis provided higher base shear forces compared with dynamic analysis. This limit was enforced due to uncertainty in accounting for ductility requirements in the dynamic analysis and the varying assumed response spectra that can result in a greatly different design forces (1977 NBCC Supplement No. 4).

The load combinations of the working stress and limit states design methods were similar to the 1975 NBCC.

### 3.3.8 NBCC 1980

In this version of the NBCC, the zonal regional maps and the seismic base shear force equation remained the same. However, the seismic response factor,  $S$ , was modified to  $0.5/(T)^{1/2}$ , compared to  $0.5/(T)^{1/3}$  that was part of the 1975 and 1977 NBCC. In addition, as in the previous codes,  $S$  was not to exceed 1 as in the comparison of the seismic response factor in Figure 3.3. This change led to an increase in the base shear forces for stiff buildings having structural periods less than 1.0 s and to a decrease of forces for buildings with periods greater than 1.0 s. As a result, buildings of 0.5 s period experienced an increase in base shear of 12%, while buildings of 4 s period saw a decrease in base shear of 20%.

The SI units were introduced in the 1980 NBCC. The structural period equation was then modified accordingly; the 5% coefficient in previous codes (1970 NBCC) was changed to 9% as follows:

$$[3.18] \quad T = 0.09h_n / \sqrt{D}$$

Where  $h_n$  is the building height above the base in metres, and  $D$  is the dimension of the building in a direction parallel to the applied forces in metres.

Design of concrete buildings including load factors and combinations remained the same and followed CSA A23.3-M77. For CSA standards, A23.3-M77 was the first to introduce the SI units.

### 3.3.9 NBCC 1985

New seismic zoning maps were developed for the NBCC 1985 and were based on a statistical analysis of earthquakes experienced in Canada and adjacent regions. The new seismic map provided new strong ground motion attenuation relations and both peak horizontal acceleration and velocity as shown in Figure 3.4. The number of seismic zones increased to 7 from 4 that were previously in effect. The probability of exceedance of the acceleration and velocity zones increased to 0.0021 per annum (or 10 % in 50

years), compared to 0.01 per annum adopted in previous codes. Acceleration- and velocity-related seismic zones were assigned values of 0.0, 0.5, 0.1, 0.15, 0.2, 0.3, and 0.4 corresponding to the zones 0 to 6, respectively.

The minimum base shear,  $V$ , equation in this code was similar to the 1975 and 1980 NBCC except that the zonal peak acceleration coefficient,  $A$ , was superseded by the zonal velocity coefficient,  $v$ , that was normalized to a spectral velocity of 1.0 m/s. The minimum base shear,  $V$ , was given by:

$$[3.19] \quad V = v S K I F W$$

Where the seismic response factor,  $S$ , was modified to  $0.22 / (T)^{1/2}$  for structural periods  $\geq 0.5$  s and set to constant values for structural periods  $\leq 0.25$  s depending on the period of the structure, the velocity-related seismic zone,  $Z_v$ , and the acceleration-related seismic zone,  $Z_a$ , pertaining to a particular location. Specifically,  $S$  values of 0.31, 0.44, 0.62 were assigned for  $Z_a < Z_v$ ,  $Z_a = Z_v$ ,  $Z_a > Z_v$ , respectively. For structural periods  $> 0.25$  s and  $< 0.5$  s, the response factor  $S$  was permitted to be determined by linear interpolation. This change resulted in lower  $S$  values compared with the 1980 NBCC.

The equation used to estimate the fundamental period of the structure was altered compared to that prescribed in the 1980 NBCC (Equation 3.18). In particular, the parameter  $D$  previously used to represent the building dimension was changed to  $D_s$ ; the dimension of the lateral force resisting system, resulting in the following expression:

$$[3.20] \quad T = 0.09 h_n / \sqrt{D_s}$$

This reduction was based on the good agreement of vibrations of shear wall structures with this modified equation (Commentary J of the 1985 NBCC). As a result, this led to structures with longer periods and lower seismic forces. Similar to previous codes and as an exception to Equation 3.20, the fundamental period for moment resisting space frames resisting 100% of the lateral forces was specified to be 10% of the number of storeys in the building.

As an alternative, the natural period of the fundamental mode was permitted to be based on Rayleigh approximation. The NBCC limited the period calculated by this method to a maximum of 1.2 times the period based on Equation 3.20. This restriction replaced the 90% limit placed on the base shear force established by dynamic analysis in the 1980 NBCC.

Since various assumptions of structural behaviour that cannot always be substantiated can result in much lower base shear forces when relying on dynamic analysis than those prescribed by the code, a simplified procedure described in Commentary J (Dynamic Analysis for the Response of Buildings) of the 1985 NBCC replaced Commentary K of the 1980 NBCC 1980. In the 1985 NBCC, the accidental torsion was recognized by specifying an additional eccentricity of 10% of the plan dimension in the direction of the computed eccentricity; an increase from the 5% specified in the 1980 NBCC. Furthermore, the requirement for increasing the computed torsional effects by 100% when the eccentricity exceeded 25% of the dimension of the building was eliminated as it was demonstrated to be unnecessary and conservative (Commentary J of the 1985 NBCC).

Dynamic analysis was required for cases where the locations of the centre of stiffness and centre of mass varied substantially between storeys in a building. In addition, it is noted to effectively resist torsional moments, structural elements should be located near the periphery of the building.

Lateral storey deflections relative to adjacent storeys obtained by elastic analysis using the seismic loadings from equivalent static force procedure were scaled by a constant value of 3 to establish realistic deflections.

The first unification of the NBCC and CSA load factors and combinations appeared in the 1985 NBCC and CSA A23.3-M84. Thus, there was no reference in the loading section of the NBCC to the CSA standard, however a reference was provided to the code load values. In addition, these load factors and combinations were modified in this edition of the NBCC from those specified in the 1975 NBCC. The “other than concrete buildings” load factors and combinations that were initially developed for steel structures (Equation 3.16) were adopted for all building types and references for “concrete building”

and “other than concrete buildings” were removed. These loads factors and combinations were reduced in the range of 11% to 12% compared to those of the CAN-A23.3-M77 (CPCA 1985). The material resistance factors for concrete,  $\Phi_c$ , and steel,  $\Phi_s$ , were taken as 0.6 and 0.85, respectively. These values remained in effect until the CSA A23.3-04 where  $\Phi_c$  was increased to 0.65.

### 3.3.10 NBCC 1990

The seismological maps of Canada depicted by seven acceleration- and velocity- related zones remained the same to the 1990 NBCC. The minimum seismic base shear force,  $V$ , was updated by dividing the equivalent lateral seismic force representing elastic response,  $V_e$ , by a new force reduction factor,  $R$ , which replaced the previous NBCC ductility factor,  $K$ . This was then multiplied by a calibration factor  $U$  specified as 0.6.

$$\begin{aligned} [3.21] \quad V_e &= v S I F W \\ V &= (V_e / R) U \end{aligned}$$

Factors ( $v$ ,  $S$ ,  $I$ ,  $F$ ,  $W$ ) are as previously defined in the 1985 NBCC. This base shear force,  $V$ , corresponded to the ultimate limit state, where the structure is assumed to be at the point of collapse (Commentary J of the NBCC 1990).

The 1995 NBCC force modification factor,  $R$ , accounted for the energy-absorption capacity of the structural system and inelastic action through several load reversals. For reinforced concrete structures,  $R$  ranged from 4.0 for ductile moment resisting space frames to 1.0 for non-ductile systems.  $R$  values of 1.5, 2.0, and 3.5 were given to other systems having nominal ductility. The  $U$  factor was applied to calibrate the base shear forces consistent with the  $R$  factors used.

The constant factor of 3 prescribed in the 1985 NBCC that was applied to the lateral deflections obtained from an elastic analysis using the lateral seismic loadings of equivalent static force procedure was replaced by the variable ductility ( $R$ ) factors. This was to ensure more realistic calculations of anticipated deflections based on the level of ductility of the structure. In addition, limits were placed on the anticipated deflections corresponding to 0.01 and 0.02 times the inter storey height,  $h_s$ , for “post disaster buildings” and for “all other buildings”, respectively.

The seismic response factor,  $S$ , was increased as shown in Table 3.3 and set equal to  $1.5/(T)^{1/2}$  for structural periods  $\geq 0.5$  s, resulting in values of 2.1, 3, 4.2 for  $Z_a < Z_v$ ,  $Z_a = Z_v$ ,  $Z_a > Z_v$ , respectively. This was implemented such that the base shear force for highly ductile systems remained similar to that calculated in the 1985 NBCC (Heidebrecht 2003).

A fourth category related to the foundation factor,  $F$ , was included in this edition of the NBCC. The new category covered sites underlain by deposits of very soft and soft fine grained soils with depth greater than 15 m. This site classification was assigned a foundation factor  $F = 2.0$ . This was based on observation of large amplifications in the clay deposits of Mexico City during the September 19, 1985 Earthquake (Commentary J of the 1990 NBCC).

The importance factor,  $I$ , for “post disaster buildings and schools”, which was assigned a value of 1.3 in the 1985 NBCC, was divided into two separate categories in the 1990 NBCC: post disaster buildings ( $I = 1.5$ ); and schools ( $I=1.3$ ). The importance factor of 1.5 for post disaster buildings was intended to “provide a higher probability of maintaining functional requirements immediately after a major earthquake” (Commentary J of the 1990 NBCC).

The load combinations of the 1990 NBCC modified the earthquake load factor from 1.5 to 1.0. The factored loads and combinations were given as follows:

$$[3.22] \quad U = 1.25 D + 0.7 (1.5L + 1.0E)$$

$$U = 1.25 D + 1.0 E$$

$$U = 0.85 D + 1.0 E$$

### **3.3.11 NBCC 1995**

Few changes were implemented into the seismic provisions of the 1995 NBCC. A new expression for calculating the fundamental period ( $T$ ) that determines the seismic response factor,  $S$ , was added for concrete moment resisting frame assigned 100% of the seismic lateral forces. The new expression was a function of building height,  $h_n$ , in metres as follows.

$$[3.23] \quad T = 0.075 (h_n)^{0.75}$$

New force modification factors were presented in the 1995 NBCC. For “Reinforced concrete structures designed and detailed according to CSA A23.3”, a value of 4 for ductile coupled wall was specified. A new category for “nominally ductile” was added for different material types for the primary lateral force resisting system. The R value assigned for nominally ductile concrete moment resisting frames was 2.

To account for higher mode effects, the portion of the lateral seismic force concentrated at the top of structure,  $F_t$ , prescribed since the 1970 NBCC (Equation 3.9) was modified to a function of the structural period ( $T$ ). Prior to this modification, it was based on the ratio of the height of the structure to the dimension of the lateral force resisting system. Furthermore, the limit of  $F_t$  increased from 15% to 25% of the base shear force  $V$ . This new formulation was considered to better account for the effects of higher modes in tall structures than the criterion of slenderness employed in the 1985 NBC (Commentary J of the 1995 NBCC). The equation appeared in the 1995 NBCC as follows:

$$[3.24] \quad \begin{aligned} F_t &= 0.07 T V \leq 0.25 V, \text{ if } (T) > 0.7 \text{ sec} \\ F_t &= 0, \text{ if } (T) \leq 0.7 \text{ sec} \end{aligned}$$

Compared to previous code editions,  $F_t$  was removed from the torsional moments, which were computed as:

$$[3.25] \quad \begin{aligned} T_x &= F_x (1.5 e_x \pm 0.1 D_{nx}) \\ T_x &= F_x (0.5 e_x \pm 0.1 D_{nx}) \end{aligned}$$

Where  $F_x$  is lateral load applied at level  $x$ ;  $e_x$  is the distance between centre of mass and centre of rigidity measured perpendicular to the direction of seismic loading; and  $D_{nx}$  is the plan dimension of the building at level  $x$  perpendicular to the direction of seismic loading.

The normalized design distribution spectrum used in dynamic analysis derived from the response spectrum for 5% damping was enhanced by including cases for  $Z_a < Z_v$  and  $Z_a > Z_v$ , in addition to the  $Z_a = Z_v$  used in previous codes.



The earthquake load combinations (Equation 3.26) for the working stress design method were reduced to two thirds. This was the first change in the load combinations since the 1970 NBCC.

$$\begin{aligned} [3.26] \quad & \text{Design Loads} = D + 2/3 E \\ & \text{Design Loads} = 0.75 (D + L + 2/3 E) \\ & \text{Design Loads} = 0.66 (D + L + 2/3 E + T) \end{aligned}$$

The limit states load combinations were divided into two categories: (1) “Load combinations not including earthquake”; and (2) “Load combinations including earthquake”. For the latter, load combinations were formulated with only dead, live and earthquake loads. The dead load factor was reduced to 1.0 from 1.25 used in the 1990 NBCC (Equation 3.22). The amended load factors were obtained based on probabilistic analysis of 30-year life (Commentary F of the 1995 NBCC). The load combinations incorporating earthquake were as follows:

$$\begin{aligned} [3.27] \quad & U = 1.0 D + 1.0 E, \text{ and either} \\ & U = 1.0 D + 1.0 L + 1.0 E \text{ for storage and assembly occupancies, or} \\ & U = 1.0 D + 0.5 L + 1.0 E \text{ for all other occupancies.} \end{aligned}$$

### 3.3.12 NBCC 2005

Major changes were incorporated in the 2005 NBCC and were summarized in Commentary J of the 2005 NBCC. These changes included: an updated hazard map in spectral acceleration format; development of site class factors; delineation of the effects of overstrength and ductility; new expressions for the fundamental period; modifications for higher mode effects for the equivalent static force procedure; new category for importance factor; and extension of building irregularities and dynamic analysis requirements and additional special provisions.

Seismic activity after 1995 provided data that led to improvements to estimate earthquake occurrence rates as a function of earthquake magnitude (Adams and Halchuk, 2003). As a result, the Uniform Hazard Spectra (UHS) was developed for the 2005 NBCC as it provides better period dependent representation of earthquake effects on structures. The differences in spectral shape across the country were reflected

directly in the determination of design forces rather than an approximation through the amplification of zonal peak ground velocity as prescribed since the 1985 NBCC. (Commentary J of the 2005 NBCC and Heidebrecht 2003). Each ordinate of the UHS provided the same probability of exceedance of 2% in 50 years, which was lowered from 10% in 50 years used in the 1985, 1990 and 1995 NBCC.

The seismic provisions of the National Earthquake Hazards Reduction Program (NHERP) Building Seismic Safety Council were the basis for formulating the site class factors in the 2005 NBCC. It involved categorization of soil profiles using quantitative measures of soil properties and recognized the period dependence of ground motions (Commentary J of the 2005 NBCC). The site classifications for seismic response were determined using the soil properties as provided in Table 3.4. The acceleration- and velocity-based site coefficients,  $F_a$  and  $F_v$ , respectively, were established according to the site classifications as described in Table 3.5.

Due to the fact that seismic forces are reduced as structural response enters the inelastic range, the 2005 NBCC introduced two factors to account for the response: the overstrength factor,  $R_o$ ; and the ductility-related factor,  $R_d$ . Factor  $R_o$  represents the overstrength-related force modification factor that accounts for reserve of strength, of the structural members, that was not considered in earlier codes (Mitchell et al., 2003); while  $R_d$  represents the ductility capacity of the Seismic Force Resisting System (SFRS). A new category “Moderately ductile” for different types of SFRS was introduced. For most ductile steel systems, the ductility-related factor increased to 5 compared to the corresponding R factor in the 1995 NBCC that assigned a value of 4. The rationale for this increase was the development of the CSA seismic detailing provisions that was based on different levels of ductility. For concrete structures,  $R_d$  values were similar to the R values of the previous code. The only difference between the two codes was the terminology since each code descriptions was selected based on its reference to the CSA A23.3 standard. Other rationales and comparisons for these two factors were discussed in detail by Mitchell et al. (2003). The  $R_d$  and  $R_o$  factors for different concrete SFRS, with restrictions related to the maximum height of the structure, based upon the modified short and long period spectral design acceleration  $I_E F_a S_a$  (0.2) and  $I_E F_a S_a$  (1.0), are listed in Table 3.6.

The fundamental period for shear walls and braced frame buildings, based on work of Saatcioglu and Humar (2003), was simplified so that it no longer depended on the dimension of the lateral force system,  $D_s$ . This change removed confusion regarding the dimension  $D_s$ . As a result, the period was determined as:

$$[3.28] \quad T = 0.05 (h_n)^{0.75}$$

Where  $h_n$  is the building height in metres.

The 2005 NBCC permitted the calculation of the fundamental period by other established methods, with a limitation that it does not exceed 2.0 times the period calculated from Equation 3.28 for shear wall structures. For moment resisting frames and braced frames, the fundamental period established by other methods was restricted to 1.5 and 2.0, respectively, times the code prescribed fundamental period corresponding to each type of SFRF.

A new formula was introduced for braced frames (Equation 3.29); it was determined that Equation 3.28 provided conservative periods (Commentary J of the 2005 NBCC). The new expression was given as:

$$[3.29] \quad T = 0.025 h_n$$

The code-prescribed fundamental period expression for moment resisting frames remained unchanged.

The 5% damped spectral acceleration response  $S_a(T)$  was established from given values for periods of 0.2, 0.5, 1.0 and 2.0 seconds and the peak ground acceleration (PGA) was calculated from the median results; both corresponding to a 2% probability of being exceeded in 50 years. This data appeared in Appendix C of the 2005 NBCC for all areas of Canada. The design spectral acceleration,  $S(T)$ , was determined by modifying the spectral acceleration response by the acceleration-based site class coefficient,  $F_a$ , and velocity-based site class coefficient,  $F_v$ , as provided in Equation 3.30. Note that linear interpolation was permitted for intermediate values of  $T$ .

$$\begin{aligned}
[3.30] \quad S(T) &= F_a S_a(0.2) \text{ for } T \leq 0.2 \text{ s} \\
&= F_v S_a(0.5) \text{ or } F_a S_a(0.2), \text{ whichever is smaller for } T = 0.5 \text{ s} \\
&= F_v S_a(1.0) \text{ for } T = 1.0 \text{ s} \\
&= F_v S_a(2.0) \text{ for } T = 2.0 \text{ s} \\
&= F_v S_a(2.0)/2 \text{ for } T \geq 4.0 \text{ s}
\end{aligned}$$

The 1995 NBCC did not provide specific requirements for building irregularities other than analysis for torsional effects. This was an area that was improved in the 2005 NBCC with the introduction of six types of vertical irregularities and two types of horizontal irregularities as listed in Table 3.7. Specifications were provided for each category including: limitations on the use of the static analysis and where dynamic analysis was necessary; restrictions on irregularities permitted in relation to the seismic hazard; restrictions applicable to post-disaster buildings; and increases in seismic design forces and specific design requirements (Heidebrecht 2003).

The three importance factor,  $I_E$ , categories in the 1995 NBCC was expanded to four with the introduction of a new importance category of “Low” that was assigned a value of 0.8 for ultimate limit states. This category represented buildings with low risk to humans in the event of failure. The importance factors of 1.0 and 1.3 from the previous code were assigned new terminologies of “Normal” and “High”, respectively, while the 1.5 factor retained the terminology of “Post-disaster”. Buildings that fall within the “low” importance category were designed with reduced lateral earthquake forces in comparison to those in the “all other buildings” category in the 1995 NBCC.

In the 2005 NBCC provisions, higher mode effects included the same two factors that appeared in the 1995 NBCC: the portion of the base shear force applied at the roof level ( $F_t$ ), which remained unchanged; and base overturning reduction factor ( $J$ ), which was extended and modified. In addition, a new factor, “higher mode factor”,  $M_v$ , was introduced, which was applied to the base shear force. This new factor and the base overturning reduction factor ( $J$ ) were functions of the type of lateral force resisting system, fundamental period of the structure, and shape of the spectral response acceleration  $S_a(T)$  as shown in Table 3.8. The numerical reduction coefficient for overturning moment at level  $x$ ,  $J_x$ , was applicable to 60% of the building height from the

base of the structure, while the remaining 40% of the building height saw no reduction in the overturning moment as follows:

$$[3.31] \quad J_x = J + (1-J) (h_x/0.6 h_n), \text{ for } h_x < 0.6 h_n.$$

Where  $h_x$  and  $h_n$  are the heights at any level  $x$  and the top of the building, respectively, measured from the base of the structure.

A new factor  $B$  that accounts for the effects of torsion referred to, “torsional sensitivity” was introduced for structures with rigid diaphragms. The  $B$  factor was specified as the maximum of all sensitivity torsional values,  $B_x$ , calculated for each level of the structure.  $B_x$  was defined as the ratio of the maximum storey displacement at the extreme points of the structure to the average displacements at the same points. The displacements used to evaluate the torsion sensitivity corresponded to those induced by the forces determined through the equivalent static force procedure in the direction of the earthquake acting at distances of  $\pm 0.1D_{nx}$  from the centre of mass at each floor. The dimension  $D_{nx}$  was defined as the plane dimension of the building at level  $x$  perpendicular to the direction of seismic loading. The procedure to determine torsional effects, as explained by the commentary J in the 2005 NBCC, was valid only for torsionally stiff structures ( $B \leq 1.7$ ), while torsionally flexible structures ( $B > 1.7$ ) and  $I_E F_a S_a (0.2) \geq 0.35$  were required to be analyzed by dynamic analysis.

In the 2005 NBCC, the design base shear based on the equivalent static force procedure was specified as follows:

$$[3.32] \quad V = S(T_a) M_v I_E W / (R_d R_o)$$

All parameters of the base shear force expression are as previously defined. The base shear force for structures of short periods was higher than those computed with the 1995 NBCC. Rationales for this were provided in Commentary J of the 2005 NBCC. An experience-based factor of 2/3 was implemented to limit the forces corresponding to a design spectral acceleration of 0.2 s. This recognized that short period structures have inherent strength and deformation capacities that are not accounted for in analysis. Therefore, the maximum base shear ( $V_{max}$ ) was set to  $\leq [(2/3) S(0.2) I_E W / (R_d R_o)]$  for

SFRS with  $R_d \geq 1.5$ . Inherent in the equation is an  $M_v$  factor of 1.0, which was the case for all periods less than 1.0s. The code also specified a minimum on the base shear equation corresponding to periods longer than 2.0 s due to uncertainty associated with  $S(T)$  values for those periods. The minimum base shear force ( $V_{\min}$ ) was specified to be  $\geq [S(2.0) M_v I_E W / (R_d R_o)]$ .

In the 2005 NBCC, dynamic analysis was the required method of analysis. It was recognized that modal analysis had matured to the point to be readily applied to establish earthquake forces and more accurately than the Equivalent Static Force Procedure, ESFP (Commentary J of the NBCC 2005). The code permitted the ESFP for structures located in low seismic hazard area, regular structures with height less than 60 m and fundamental period less than 2 sec, or irregular structures with height less than 20 m and fundamental period less than 0.5 sec. However, the code highlighted the concern that the base shear force could be low with dynamic analysis due to analysis parameters that are not specified in the code. Therefore, a minimum dynamic base shear force of 80% and 100% of that computed with the ESFP were placed on regular and irregular structures.

The structural lateral deflections obtained from elastic analysis using either the ESFP or dynamic analysis including the effects of torsion were required to be magnified by  $R_d R_o / I_E$  to estimate realistic due to the inelastic behaviour and overstrength of the structure. The computed deflections correspond to elastic deflections based on reduced design forces (Mitchell et al., 2003). The importance factor  $I_E$ , was incorporated, as discussed by DeVall (2003), resulting in increased design loads for buildings in the high and post-disaster importance categories and a reduction of deflections to obtain realistic values. The interstorey lateral deflections limit  $0.02 h_s$  used in the 1995 NBCC for “all other buildings” was assigned to “schools” in the 2005 NBCC, while for “all other buildings”, the limit was increased to  $0.025 h_s$ , where  $h_s$  is the interstorey height in metres. For post-disaster building, the drift was limited to  $0.01 h_s$ .

The working stress design method was removed in the 2005 NBCC 2005. This method was used since it was first introduced in 1941, but was gradually phased out as an alternative procedure to the limit states design method introduced in 1975. The working

stress method is still used as the basis for some standards and specifications not directly referenced by the NBCC. (Commentary A of the 2005 NBCC).

A modification to the 1995 NBCC Ultimate Limit State earthquake load combinations incorporated the snow load,  $S$ , that included ice and associated rain as follows:

$$\begin{aligned} [3.33] \quad U &= 1.0D + 1.0E, \text{ and either} \\ U &= 1.0D + 1.0E + 1.0L + 0.25S, \text{ for storage and equipment areas and} \\ &\quad \text{service rooms} \\ U &= 1.0D + 1.0E + 0.5L + 0.25S, \text{ for all other occupancies.} \end{aligned}$$

The resistance factor of concrete,  $\Phi_c$ , was increased to 0.65 from 0.6, except for precast concrete elements produced in manufacturing plants; a factor of 0.7 was permitted. This increase was justified to achieve safety margins for components cast from concretes with compressive strengths ranging between 20 and 35 MPa. It was also consistent with the concrete standard at that time (CAC, 2006). The resistance factor of steel,  $\Phi_s$ , remained at 0.85.

### 3.3.13 NBCC 2010

The most recent NBCC was the 2010 edition. The design spectral acceleration values,  $S_a$ , were slightly adjusted in the 2010 NBCC. Comparison of  $S_a$  from 2010 and 2005 for La-Malbaie, Vancouver, Ottawa and Toronto are provided in Figure 3.5. It is evident that spectral values have been slightly lowered for very low period and stiff structures for most cities in Canada.

The minimum,  $V_{\min}$ , and maximum,  $V_{\max}$ , seismic base shear force have been slightly modified from the 2005 NBCC. In the 2005 NBCC  $V_{\min}$  was not categorized and applicable to all SFRS. In this edition, the  $V_{\min}$  from the 2005 NBCC is applicable to moment resisting frames, braced frames, and other systems; while for walls, coupled walls and wall frame systems, the design spectral acceleration response for  $V_{\min}$  is based on a larger period of 4 s, rather than 2.0 s, even though there is considerable uncertainty associated with such long periods. The spectral acceleration for periods greater than 4.0 s is equal to spectral acceleration at 4.0 s. The minimum base shear force,  $V_{\min}$ , for walls, coupled walls and wall frame systems is given as follows:

$$[3.34] \quad V_{\min} \geq [S(4.0) M_v I_E W / (R_d R_o)]$$

Limitations governing the calculation of the fundamental lateral period using established methods other than code empirical formulas remain similar to those that appeared in the 2005 NBCC. However, an exception is permitted in the calculation of the deflections; the upper limit on periods computed with other established methods is relaxed. For walls, coupled walls and wall-frame systems the period,  $T_a$ , should not exceed 4.0 s; and for moment resisting frames, braced frames, and other systems,  $T_a$  should not exceed 2.0 s.

The maximum base shear force,  $V_{\max}$ , is similar to the previous edition of the NBCC. However, this maximum is only applicable to structures located on a site class “other than class F” in addition to the SFRS with  $R_d \geq 1.5$ . The former is a new to the 2010 NBCC. The maximum base shear force is justified by the code as dynamic response of structures responding in the high frequency range typically experience small displacements and therefore minimal structural damage is expected; however, this is not applicable to structures constructed on site class F soil due to significant amplification effects.

The higher mode,  $M_v$ , and base overturning reduction,  $J$ , factors are expanded in the 2010 NBCC from those of the 2005 NBCC (Table 3.8). The three “Type of Lateral Resisting Systems” are increased to five, new case for  $M_v$  and  $J$  for  $T_a \geq 4.0$  sec is added, and the previous case for  $M_v$  and  $J$  for  $T_a \geq 2.0$  is modified to  $T_a = 2.0$ . The  $M_v$  value for long period wall-type structures is increased from 1.2 to 1.6 and from 2.5 to 3.0 for  $[S_a(0.2)/S_a(2.0)] < 8.0$  and  $\geq 8.0$ , respectively, while the  $J$  value is decreased from 0.7 to 0.5 and from 0.4 to 0.3 for  $[S_a(0.2)/S_a(2.0)] < 8.0$  and  $\geq 8.0$ , respectively.

Changes are also applicable to torsional effects. In the 2010 NBCC, structures with  $[I_E F_a S_a(0.2) < 0.35]$  shall account for torsional effects. In this case, the equivalent static force procedure is used. This torsional requirement, in the 2005 NBCC, was only valid for torsionally stiff structures with torsional sensitivity,  $B$ , less than 1.7.

For irregular structures, dynamic analysis is the default approach in the 2010 NBCC; it being understood that this method provides more realistic distribution of earthquake forces for such irregular structures than static analysis. In addition, a new reduction factor of  $[(2 S(0.2))/[3 S(T_a)]] \leq 1.0$  is applied to the elastic base shear obtained from



dynamic analysis for short period structures having an SFRS with  $R_d \geq 1.5$  and constructed on sites other than Class F to establish the design elastic base shear.. The intent of this is to limit the design spectral acceleration corresponding to the first mode period to a value equal to  $2/3 S(0.2)$  (Commentary J of the NBCC 2010).

The 2010 NBCC ultimate limit state earthquake load combinations are similar to those used in the 2005 NBCC (Equation 3.33). One modification includes the explicit consideration of crane dead and live loads. In previous codes, these loads were included in the live load. This change results in two new load combinations where earthquake is one of the principal loads in addition to the load combinations without crane loads:

$$\begin{aligned} [3.35] \quad U &= 1.0D + 1.0E + 1.0C_d + 1.0L_{xc} + 0.25S, \text{ for storage occupancies,} \\ &\quad \text{equipment areas, and service rooms} \\ U &= 1.0D + 1.0E + 1.0C_d + 0.5L_{xc} + 0.25S, \text{ for all other occupancies} \end{aligned}$$

Where  $C_d$  and  $L_{xc}$  are crane dead loads positioned for maximum effects and live loads, respectively.

### 3.4 Comparative Study of NBCC Design Base Shear Forces

A comparison study was conducted on the seismic base shear design forces of six- and ten-storey non-ductile concrete moment resisting frame buildings. This study considered all editions of the NBCC and the two structures were chosen to represent the lower and higher height range of medium-rise buildings. Furthermore, these buildings are assumed to be located in the medium and high seismic hazard regions of Ottawa and Vancouver, respectively. The six-storey building is modified from a building described in the Concrete Design Handbook (Cement Association of Canada, 2006). The modification consists of different plan dimensions in the E-W direction and interstorey heights; while the 10-storey building was modified from a building previously designed based on the 1990 NBCC (Dincer 2003). Modifications to the later building consisted of different interstorey heights and cross sectional dimensions of columns and beams to satisfy the difference in base shear demands between earlier editions of the NBCC and the 1990 NBCC of similar buildings. Plan and elevation views of these two buildings are shown in Figures 3.6 and 3.7, respectively. The assumed parameters required for calculation of the base shear were: very dense soil condition; normal importance buildings; materials

and construction type corresponding to non-ductile reinforce concrete framing systems (Conventional construction according to 2010 NBCC). Note that a factor of 2 is applied when calculating the equivalent factored design base shear forces corresponding to the 1941-1960 NBCC inclusive, since the method of analysis during that era was the working stress design method that limited the steel reinforcement strength to 50% of its yield capacity. Detailed analysis calculation for both buildings can be found in Appendix I and II.

The results of the comparative study are presented in Figure 3.8 and 3.9, respectively for the six- and ten-storey structures. The following conclusion is drawn from this study: during the era where the 1941-1965 NBCC was in force, the base shear forces for Ottawa and Vancouver were identical. This is a direct result of the formulation of base shear force that were not a function of regional seismological intensity (1941 NBCC) or with equal intensity zones in the eastern and western regions of Canada (1953, 1960 and 1965 NBCC). In addition, since the introduction of a revised seismic zoning map in the 1970 NBCC and recent uniform hazard spectrum of the 2005 and 2010 NBCC, the ratio of base shear force of Vancouver to Ottawa is approximately 2 for both buildings analyzed. This implies higher seismic hazards in the west coast regions of Canada. Furthermore, in general, the base shear forces have increased with progress in the developments of the NBCC over the years. For this type of non-ductile structure assumed parameters and the six-storeys in height located in Vancouver results in a ratio of 2010 NBCC base shear to 1941 NBCC (first edition) of 6.12; while the ratio to the 1970 NBCC (last code edition before ductility was introduced) is 4.13. Tables 3.9 and 3.10 provide other comparisons for all editions of the NBCC with respect to base shear force determined with the 2010 NBCC for the six- and ten-storey buildings, respectively. Furthermore, comparing the two buildings which are approximately of similar weight but different heights, the six-storey building exhibited higher base shear values ( $V/W$ ) than the ten-storey building for all editions of the NBCC except 1941 NBCC. For this edition, the base shear values were identical due to the absence of the building height effect in calculating the base shear. In subsequent codes, shorter structural periods, i.e. lower-rise buildings, is reflected in the higher spectral acceleration demands.

### 3.5 Summary

This chapter presented state-of-the-art developments of seismic design provisions in the National Building Code of Canada with emphasis on reinforced concrete buildings. The progressing of design load factors and load combinations were studied including the working stress and limit state design methods. Based on this study, the seismic provisions in the NBCC have experienced extensive changes based on additional seismicity data recorded after major earthquakes and advancements in knowledge through seismic research. The progression of seismic provisions have included: updated seismic hazard in spectral format, development of site and importance factors; delineation of the effects of overstrength and ductility; addition of new expressions for the calculations of the fundamental lateral period; modifications to the higher mode effects in the equivalent static force procedure; and extension of structural irregularities and dynamic analysis requirements.

The results from the NBCC comparative study of the design base shear forces for non-ductile reinforced concrete frame buildings located in moderate and high seismic zones highlights serious concern. The concrete medium-rise buildings (5 to 10 storeys) that were analyzed represent one of the more common type of structures constructed before the enactment of ductile detailing provisions were introduced in the mid-70's. Thus, these types of structures typically are considered seismically deficient and attention should be directed towards mitigating risk through retrofitting. The ductility and energy dissipating capacities of these structures should be improved by an appropriate retrofitting methodology.

In summary, this chapter provided a comprehensive review of the additional strength capacity required to retrofit deficient structures built since the 1940s and prior to seismic ductility provisions appearing in the NBCC. The NBCC base shear force comparison for the 10-storey building provided the base to select a prototype building built during a specific era. This 10-storey building also served as the prototype model for static and dynamic linear analyses to assess various retrofit strategies (Chapter 5, sec. 5.2). Furthermore, sizing of the cross-sectional areas of the buckling restrained steel braces used in the proposed retrofitting method (Chapter 6, sec. 6.7.1) for the six-storey building case study were based on the ratio of the base shear calculated with the 2010 NBCC to that of the NBCC edition in effect at the time that the prototype building was

assumed to have been designed. Thus, a non-retrofitted frame was tested as part of this research study to establish the strength and ductility capacities of non-ductile moment resisting frames built to older codes. Thereafter, retrofitted frame specimens were strengthened to meet the increased requirements of the 2010 NBCC 2010 relative to the capacities of the non-retrofitted frame.

Table 3.1: Numerical coefficient, K, factor used in lateral seismic force equation of the  
1975 NBCC (NBCC 1975)

| Case | Type or arrangement of resisting elements                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | Value of K |
|------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------|
| 1    | Buildings with a ductile moment resisting space frame with the capacity to resist the total required lateral force.                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                              | 0.7        |
| 2    | Buildings with a dual structural system consisting of a complete ductile moment resisting space frame and ductile flexural walls designed in accordance with the following criteria:<br>The frames and ductile flexural walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the flexural walls and frames. In this analysis the maximum shear in the frame must be at least 25 percent of the total base shear.                                                                                                                                                               | 0.7        |
| 3    | Buildings with a dual structural system consisting of a complete ductile moment resisting space frame and shear walls or steel bracing designed in accordance with the following criteria:<br>(a) The shear walls or steel bracing acting independently of the ductile moment resisting space frame shall resist the total required lateral force.<br>(b) The ductile moment resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force, but in no case shall the ductile moment-resisting space frame have a lower capacity than that required in accordance with the relative rigidities. | 0.8        |
| 4    | Buildings with ductile flexural walls and buildings with ductile framing systems not otherwise classified in this Table as Cases 1, 2, 3 or 5.                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   | 1.0        |
| 5    | Buildings with a dual structural system consisting of a complete ductile moment resisting space frame with masonry infilling designed in accordance with the following criteria:<br>(a) The walls system comprising the infilling and the confining elements acting independently of the ductile moment resisting space frame shall resist the total required lateral force.<br>(b) The ductile moment resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force.                                                                                                                          | 1.3        |
| 6    | Buildings other than cases 1, 2, 3, 4 and 5 of (a) continuously reinforced concrete, (b) structural steel, and (c) reinforced masonry shear walls                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                | 1.3        |
| 7    | Buildings of unreinforced masonry and all other structural systems except Cases 1 to 6 inclusive.                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                | 2.0        |
| 8    | Elevated tanks plus full contents, on 4 or more cross-braced legs and not supported by a building designed in accordance with the following criteria:<br>(a) The minimum and maximum value of the product SK1 shall be taken as 1.2 and 2.5 respectively.<br>(b) For overturning, the factor J as set forth in Clause 4.1.9.1.(14) shall be 1.0<br>(c) The torsional requirements of Sentence 4.1.9.1.(15) shall apply                                                                                                                                                                                                                           | 3.0        |

Table 3.2: Maximum structural ductility factor,  $\mu$ , for various building types (Commentary K of the 1975 NBCC)

| Building type                                                                                 | Structural ductility factor |
|-----------------------------------------------------------------------------------------------|-----------------------------|
| Ductile moment resisting space frame                                                          | 4                           |
| Combined system of 25 percent ductile moment resisting space frame and ductile flexural walls | 3                           |
| Ductile reinforced concrete flexural walls                                                    | 3                           |
| Regular reinforced concrete structures, cross-braced frame structures and reinforced masonry  | 2                           |
| Structures having no ductility, plain masonry                                                 | 1                           |

Table 3.3: The seismic response factor,  $S$ , of the 1990 NBCC (Commentary J of the NBCC1990)

| Seismic response factor |            |                    |
|-------------------------|------------|--------------------|
| T                       | $Z_a/Z_v$  | S                  |
| < 0.25                  | > 1.0      | 4.2                |
|                         | 1.0        | 3.0                |
|                         | < 1.0      | 2.1                |
| > 0.25 but < 0.5        | > 1.0      | 4.2 - 8.4 (T-0.25) |
|                         | 1.0        | 3.0 - 3.6 (T-0.25) |
|                         | < 1.0      | 2.1                |
| $\geq 0.5$              | All values | $1.5/(T)^{1/2}$    |

Table 3.4: Site classification for seismic site response (NBCC 2005)

| Site Class | Ground profile name           | Average properties in top 30 m                                                                                                                                                                                                                                                                        |                                                         |                                            |
|------------|-------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------|--------------------------------------------|
|            |                               | Average shear wave velocity, $\bar{V}_s$ (m/s)                                                                                                                                                                                                                                                        | Average standard penetration resistance, $\bar{N}_{60}$ | Soil undrained shear strength, $S_u$ (kPa) |
| A          | Hard rock                     | $\bar{V}_s > 1500$                                                                                                                                                                                                                                                                                    | n/a                                                     | n/a                                        |
| B          | Rock                          | $760 < \bar{V}_s \leq 1500$                                                                                                                                                                                                                                                                           | n/a                                                     | n/a                                        |
| C          | Very dense soil and soft rock | $360 < \bar{V}_s < 760$                                                                                                                                                                                                                                                                               | $\bar{N}_{60} > 50$                                     | $S_u > 100$                                |
| D          | Stiff soil                    | $180 < \bar{V}_s < 360$                                                                                                                                                                                                                                                                               | $15 \leq \bar{N}_{60} \leq 50$                          | $50 \leq S_u \leq 100$                     |
| E          | Soft soil                     | $\bar{V}_s < 180$                                                                                                                                                                                                                                                                                     | $\bar{N}_{60} < 15$                                     | $S_u < 50$                                 |
|            |                               | Any profile with more than 3 m of soil with the following characteristics: <ul style="list-style-type: none"> <li>• Plasticity index: <math>PI &gt; 20</math></li> <li>• Moisture content: <math>w \geq 40\%</math>, and</li> <li>• Undrained shear strength: <math>S_u &lt; 25</math> kPa</li> </ul> |                                                         |                                            |
| F          | Other soils                   | Site specific evaluation required                                                                                                                                                                                                                                                                     |                                                         |                                            |

Table 3.5: Values of acceleration ( $F_a$ ) and velocity ( $F_v$ ) based site coefficients as a function of site class and  $S_a$  (NBCC 2005)

| Site class |                               | Values of $F_a$                                                              |          |          |         |             | Values of $F_v$                                                              |          |          |          |             |
|------------|-------------------------------|------------------------------------------------------------------------------|----------|----------|---------|-------------|------------------------------------------------------------------------------|----------|----------|----------|-------------|
|            |                               | $S_a(0.2)$                                                                   |          |          |         |             | $S_a(1.0)$                                                                   |          |          |          |             |
|            |                               | $\leq 0.25$                                                                  | $= 0.50$ | $= 0.75$ | $= 1.0$ | $\geq 1.25$ | $\leq 0.10$                                                                  | $= 0.20$ | $= 0.30$ | $= 0.40$ | $\geq 0.50$ |
| A          | Hard rock                     | 0.7                                                                          | 0.7      | 0.8      | 0.8     | 0.8         | 0.5                                                                          | 0.5      | 0.5      | 0.6      | 0.6         |
| B          | Rock                          | 0.8                                                                          | 0.8      | 0.9      | 1.0     | 1.0         | 0.6                                                                          | 0.7      | 0.7      | 0.8      | 0.8         |
| C          | Very dense soil and soft rock | 1.0                                                                          | 1.0      | 1.0      | 1.0     | 1.0         | 1.0                                                                          | 1.0      | 1.0      | 1.0      | 1.0         |
| D          | Stiff soil                    | 1.3                                                                          | 1.2      | 1.1      | 1.1     | 1.0         | 1.4                                                                          | 1.3      | 1.2      | 1.1      | 1.1         |
| E          | Soft soil                     | 2.1                                                                          | 1.4      | 1.1      | 0.9     | 0.9         | 2.1                                                                          | 2.0      | 1.9      | 1.7      | 1.7         |
| F          | Other soils                   | Specific geotechnical investigations and dynamic analysis shall be performed |          |          |         |             | Specific geotechnical investigations and dynamic analysis shall be performed |          |          |          |             |

Table 3.6: SFRS ductility-related force modification factor,  $R_d$ , overstrength-related force modification factor ( $R_o$ ) of concrete structures and general restrictions (NBCC 2005)

| Type of SFRS                                                     | R <sub>d</sub> | R <sub>o</sub> | Restrictions (maximum height limits)               |                 |                  |        |                                                    |
|------------------------------------------------------------------|----------------|----------------|----------------------------------------------------|-----------------|------------------|--------|----------------------------------------------------|
|                                                                  |                |                | I <sub>E</sub> F <sub>a</sub> S <sub>a</sub> (0.2) |                 |                  |        | I <sub>E</sub> F <sub>a</sub> S <sub>a</sub> (1.0) |
|                                                                  |                |                | < 0.2                                              | ≥ 0.2 to < 0.35 | ≥ 0.35 to < 0.75 | > 0.75 | > 0.3                                              |
| Concrete structures designed and detailed according to CSA A23.3 |                |                |                                                    |                 |                  |        |                                                    |
| Ductile moment resisting frames                                  | 4.0            | 1.7            | NL                                                 | NL              | NL               | NL     | NL                                                 |
| Moderately ductile moment resisting frames                       | 2.5            | 1.4            | NL                                                 | NL              | 60               | 40     | 40                                                 |
| Ductile coupled walls                                            | 4.0            | 1.7            | NL                                                 | NL              | NL               | NL     | NL                                                 |
| Ductile partially coupled walls                                  | 3.5            | 1.7            | NL                                                 | NL              | NL               | NL     | NL                                                 |
| Ductile shear walls                                              | 3.5            | 1.6            | NL                                                 | NL              | NL               | NL     | NL                                                 |
| Moderately ductile shear walls                                   | 2.0            | 1.4            | NL                                                 | NL              | NL               | 60     | 60                                                 |
| Conventional construction                                        |                |                |                                                    |                 |                  |        |                                                    |
| Moment resisting frames                                          | 1.5            | 1.3            | NL                                                 | NL              | 15               | NP     | NP                                                 |
| Shear walls                                                      | 1.5            | 1.3            | NL                                                 | NL              | 40               | 30     | 30                                                 |
| Other concrete SFRS(s)<br>Not listed above                       | 1.0            | 1.0            | 15                                                 | 15              | NP               | NP     | NP                                                 |



Table 3.7: Structural irregularities (NBCC 2005)

| Type | Irregularity type and definition                                                                                                                                                                                        |
|------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 1    | Vertical Stiffness Irregularity<br>If the lateral stiffness of SFRS in a storey is less than 70% of the stiffness of any adjacent storey, or less than 80% of average stiffness of three stories above or below.        |
| 2    | Weight (mass) Irregularity<br>If the weight of any storey, $W_i$ , is more than 150% of the weight of an adjacent storey. A roof that is lighter than the floor below need not to be considered.                        |
| 3    | Vertical geometry Irregularity<br>If the horizontal dimension of the SFRS in a storey is more than 130% of that in an adjacent storey.                                                                                  |
| 4    | In-plane discontinuity in Vertical Lateral-Force-Resisting Element<br>An in-plane offset of a lateral-force-resisting element of the SFRS or a reduction in lateral stiffness of resisting element in the storey below. |
| 5    | Out-of-Plane Offsets<br>Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS.                                                                                     |
| 6    | Discontinuity in Capacity- Weak storey<br>If the storey shear strength is less than that in the storey above.                                                                                                           |
| 7    | Torsional Sensitivity<br>If the ratio $B$ , maximum value of torsional sensitivity, exceeds 1.7.                                                                                                                        |
| 8    | Non-orthogonal Systems<br>If the SFRS is not oriented along a set of orthogonal axes.                                                                                                                                   |

Table 3.8: Higher mode factor,  $M_v$ , and base overturning reduction factor,  $J$  (NBCC 2005)

| $S_a(0.2)/S_a(2.0)$ | Type of lateral resisting systems          | $M_v$ for $T_a \leq 1.0$ | $M_v$ for $T_a \geq 2.0$ | $J$ for $T_a \leq 0.5$ | $J$ for $T_a \geq 2.0$ |
|---------------------|--------------------------------------------|--------------------------|--------------------------|------------------------|------------------------|
| < 8.0               | Moment- resisting frames or coupled walls. | 1.0                      | 1.0                      | 1.0                    | 1.0                    |
|                     | Braced frames                              | 1.0                      | 1.0                      | 1.0                    | 1.0                    |
|                     | Walls, wall-frame systems, other systems   | 1.0                      | 1.2                      | 1.0                    | 1.2                    |
| $\geq 8.0$          | Moment resisting frames or coupled walls.  | 1.0                      | 1.2                      | 1.0                    | 1.2                    |
|                     | Braced frames                              | 1.0                      | 1.5                      | 1.0                    | 1.5                    |
|                     | Walls, wall-frame systems, other systems.  | 1.0                      | 2.5                      | 1.0                    | 2.5                    |

Table 3.9: Ratio of base shear design force of NBCC 2010 and other NBCC editions for non-ductile six storey concrete frame building located in Ottawa and Vancouver

| NBCC       | NBCC 2010/ NBCC (Year) |           |
|------------|------------------------|-----------|
|            | Ottawa                 | Vancouver |
| 1941       | 2.83                   | 6.12      |
| 1953, 1960 | 0.99                   | 2.14      |
| 1965       | 1.36                   | 2.94      |
| 1970       | 3.82                   | 4.13      |
| 1975, 1977 | 3.68                   | 3.97      |
| 1980       | 3.38                   | 3.65      |
| 1985       | 3.07                   | 3.32      |
| 1990       | 1.46                   | 1.58      |
| 1995       | 1.65                   | 1.78      |
| 2005       | 1.00                   | 0.98      |
| 2010       | 1.00                   | 1.00      |

Table 3.10: Ratio of base shear design force of NBCC 2010 and other NBCC editions for non-ductile ten storey concrete frame building located in Ottawa and Vancouver

| NBCC       | NBCC 2010/ NBCC (Year) |           |
|------------|------------------------|-----------|
|            | Ottawa                 | Vancouver |
| 1941       | 1.733                  | 4.068     |
| 1953, 1960 | 0.838                  | 1.966     |
| 1965       | 1.054                  | 2.473     |
| 1970       | 2.773                  | 3.255     |
| 1975, 1977 | 2.667                  | 3.129     |
| 1980       | 2.667                  | 3.129     |
| 1985       | 2.424                  | 2.845     |
| 1990       | 1.156                  | 1.356     |
| 1995       | 1.200                  | 1.356     |
| 2005       | 1.080                  | 0.972     |
| 2010       | 1.0                    | 1.0       |

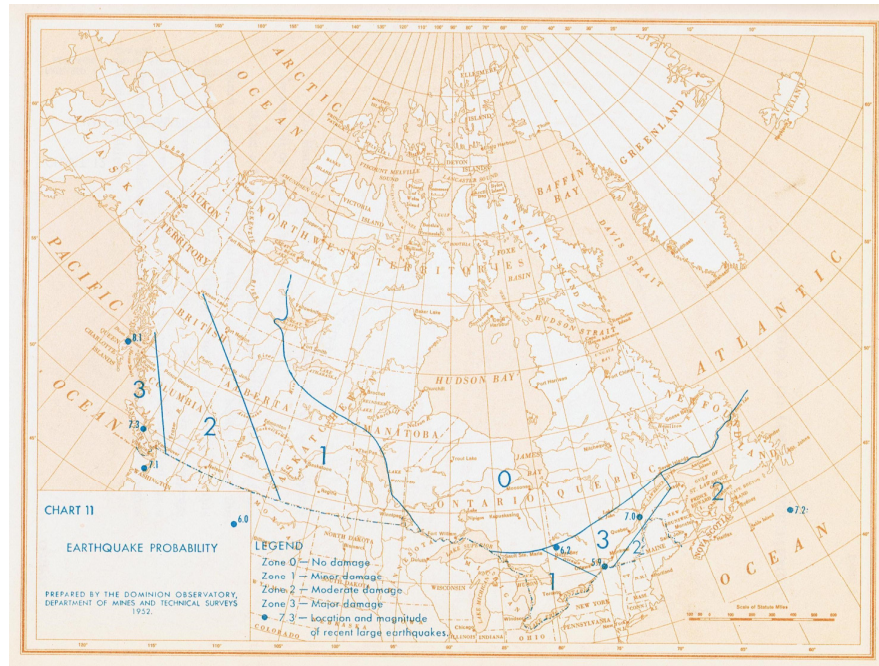


Figure 3.1: Earthquake probability zones of Canada (NBCC 1953)

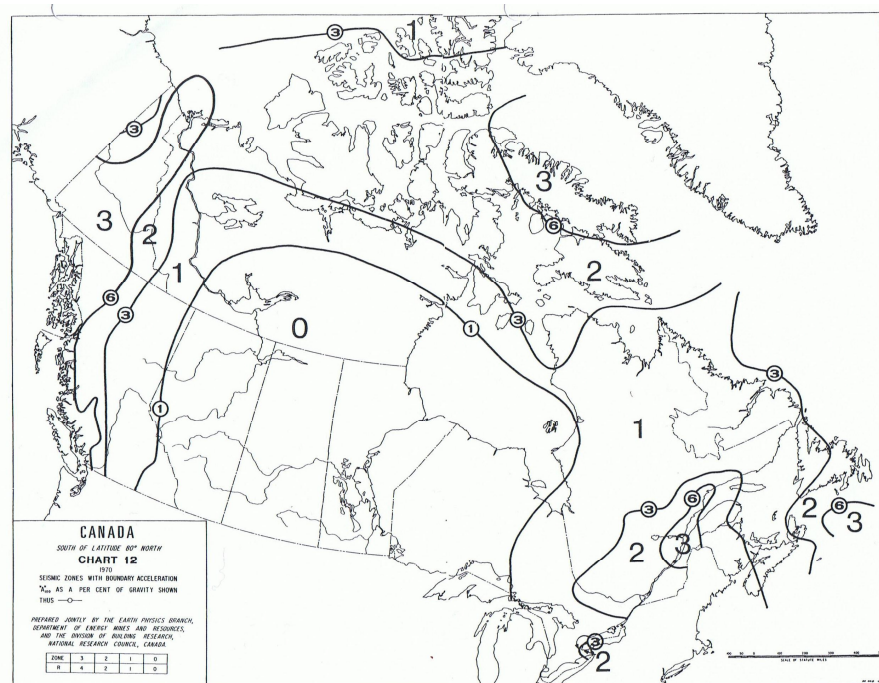


Figure 3.2: Seismic zones “A<sub>100</sub>” as percent of gravity (NBCC 1970)

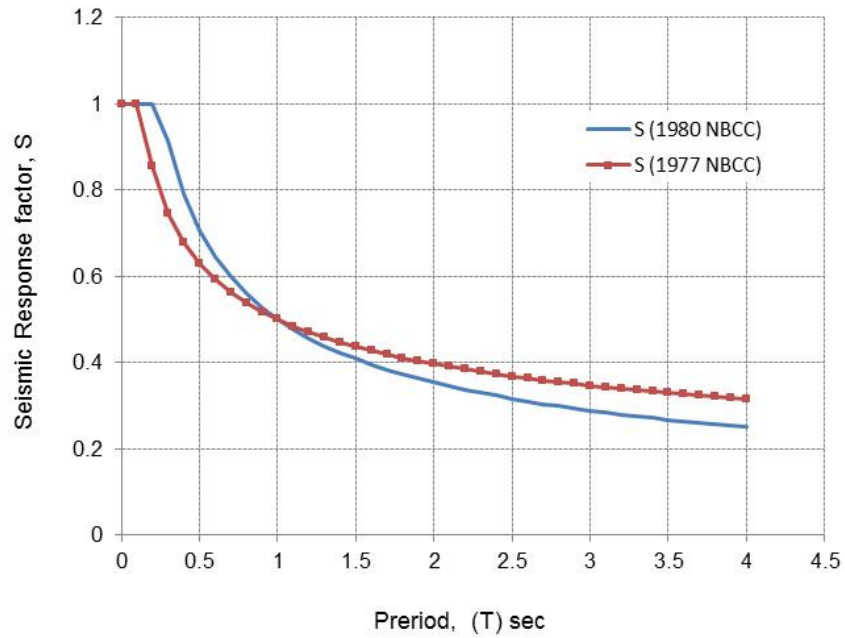


Figure 3.3 Comparison of seismic response factor,  $S$ , of NBCC 1980 and 1977

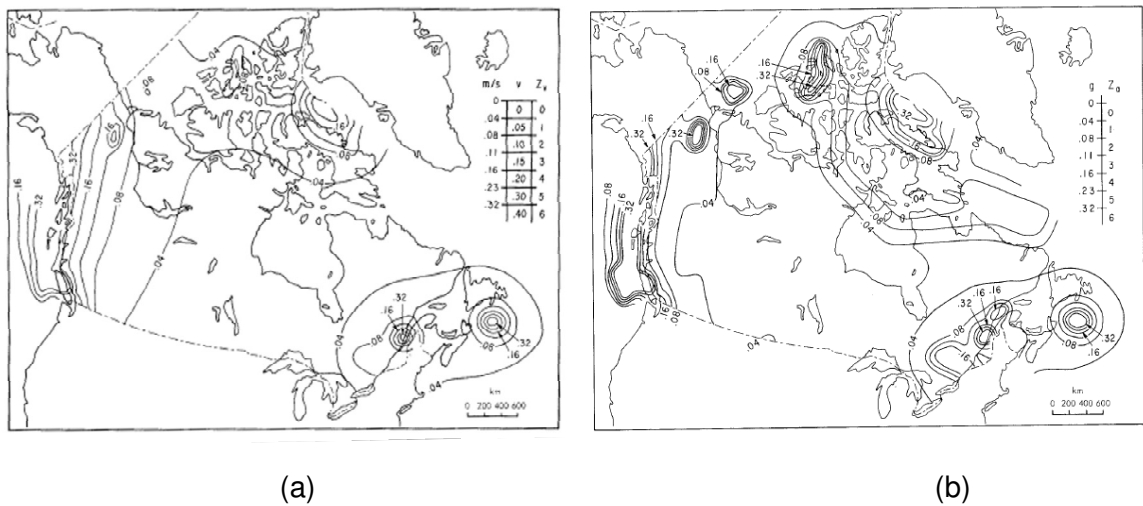


Figure 3.4: Contours of peak of acceleration and velocities: (a) peak horizontal ground acceleration, in units of  $g$ ; and (b) peak horizontal ground velocities, in m/s (NBCC 1985)

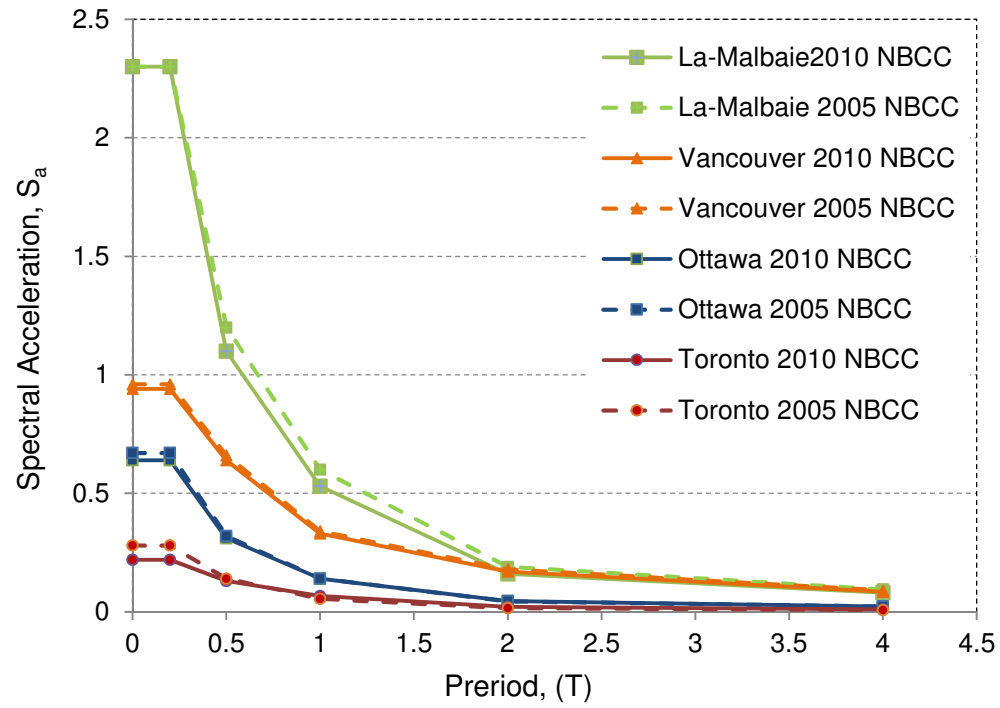
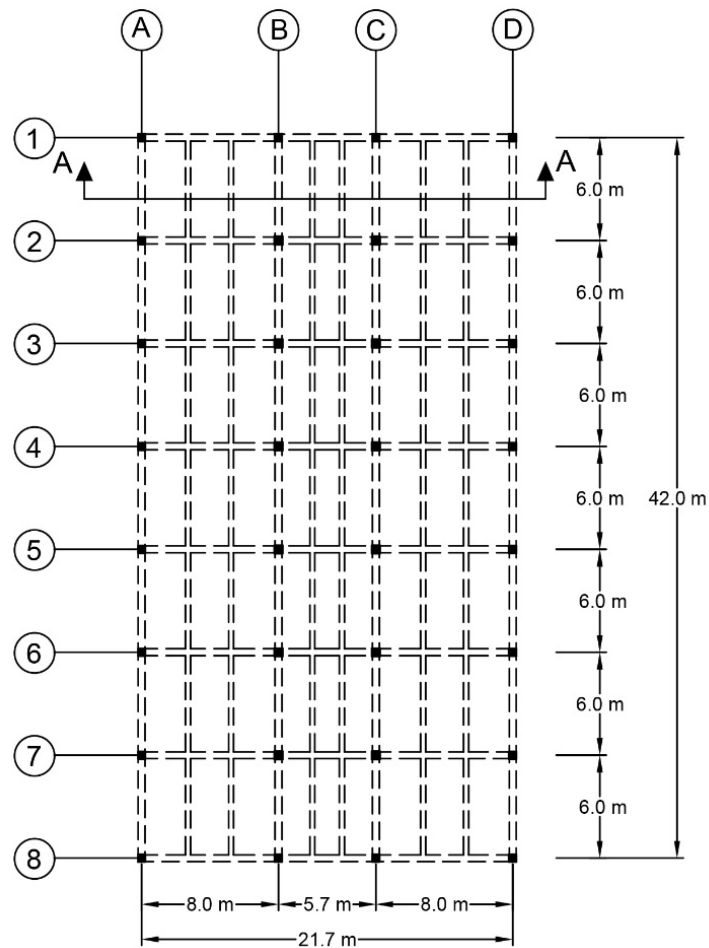
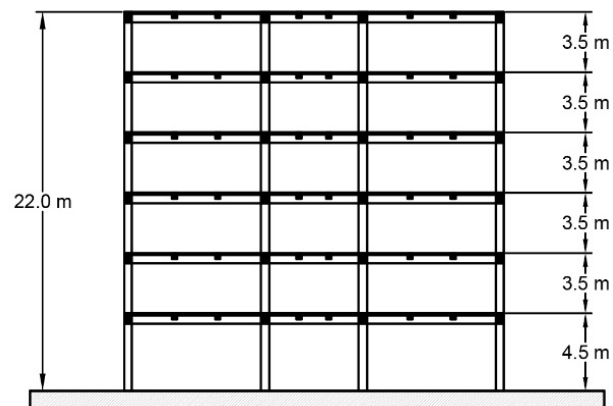


Figure 3.5: Comparisons of 5% spectral acceleration  $S_a$  values of 2005 and 2010 NBCC for La-Malbaie, Vancouver, Ottawa and Toronto



Plan View



Section A-A

Figure 3.6: Studied non-ductile six-storey concrete frame building located in Ottawa and Vancouver (modified from Cement Association of Canada 2006)

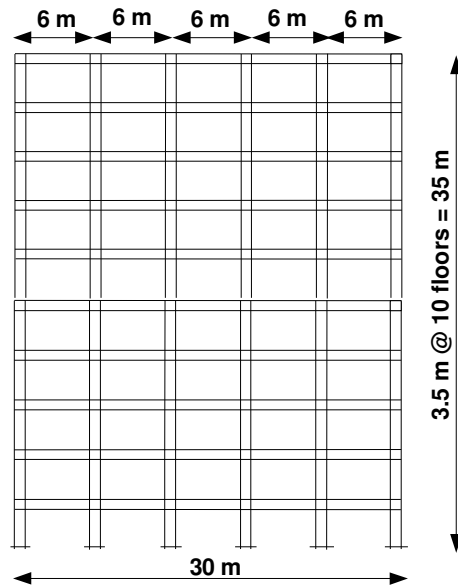
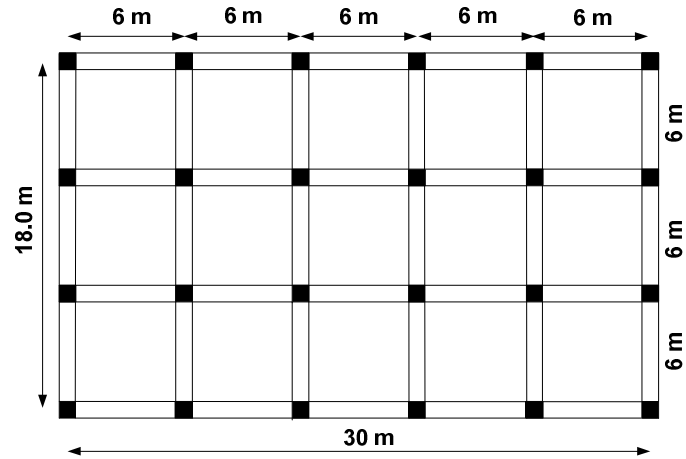


Figure 3.7: Studied non-ductile ten-storey concrete frame building located in Ottawa and Vancouver (modified from Dincer 2003)

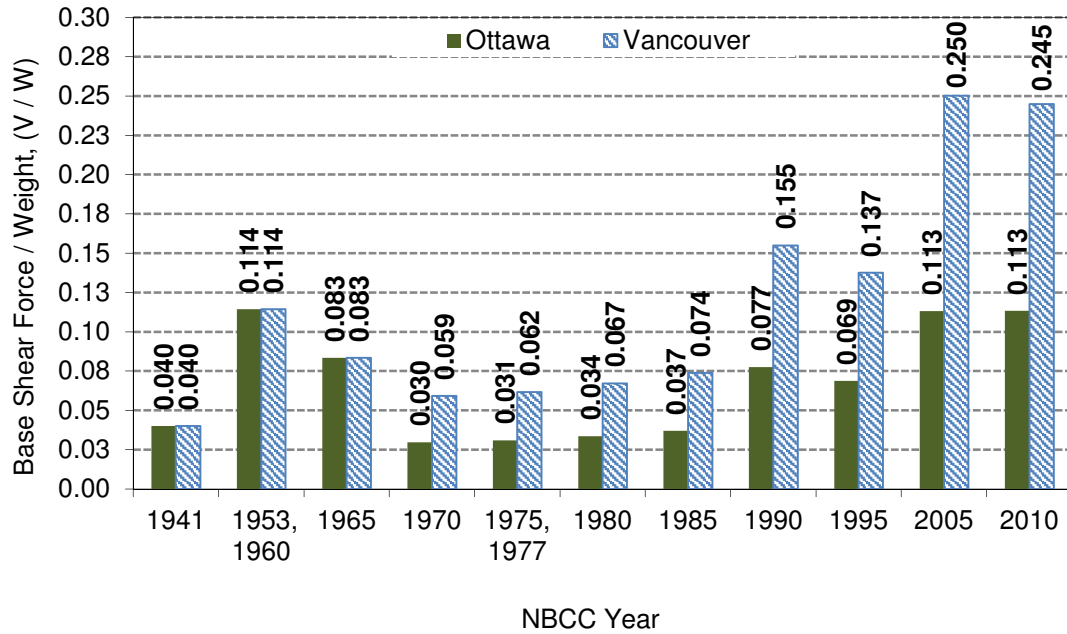


Figure 3.8: Comparison of NBCC base shear design force for non-ductile six-storey concrete frame building located in Ottawa and Vancouver

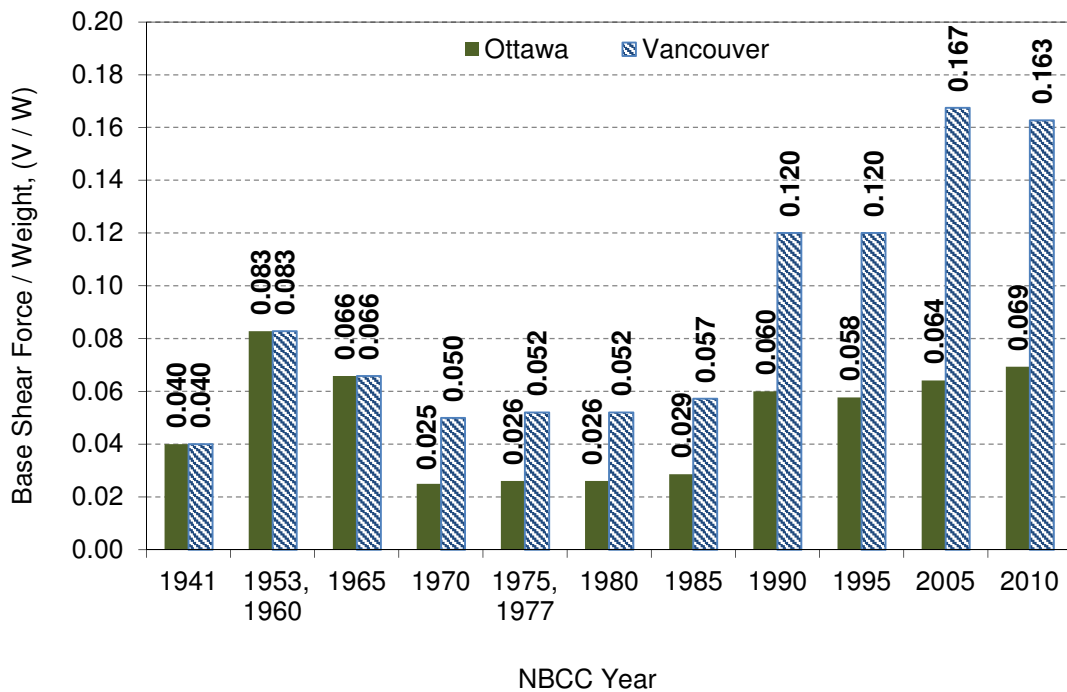


Figure 3.9: Comparison of NBCC base shear design force for non-ductile ten-storey concrete frame building located in Ottawa and Vancouver



## **Chapter 4**

# **Evolution of Canadian Seismic Detailing Requirements of Monolithic Reinforced Concrete Structures**

### **4.1 General**

This Chapter describes the development of seismic design and detailing provisions of the Canadian Standard CSA A23.3, “Design of Concrete Structures,” since its inception in 1929. This review was necessary for designing the prototype building frames used in the current investigation. The two-third scale test frames employed in the investigation were designed based on the 1965 NBCC as representative of seismically deficient buildings over five decades ago. The frame design details are further explained in Chapter 6 and Appendix III.

The introduction of improved force modification factors in the National Building Code of Canada (NBCC) over the years made it necessary to improve and expand related design and ductile detailing requirements in relevant CSA Standards. This chapter provides a review of the seismic provisions of A23.3 for ductile, moderately ductile, and conventional frame elements, including beam column Joints, individual shear walls, coupled walls and partially coupled walls.

### **4.2 Introduction**

Many existing reinforced concrete frame structures built prior to the enactment of modern building codes are seismically deficient. These buildings were designed primarily for gravity and wind loads. Building Code requirements for earthquake effects have been extensively modified as a result of research and knowledge gained over the years, often making these requirements more stringent. However, the majority of existing infrastructure remains to be seismically deficient and pose significant seismic risk. The best approach to improve their seismic performance is to retrofit them. This requires the assessment of their existing strength and ductility, and the required enhancements so that an appropriate retrofit methodology can be implemented. Therefore, the review of CSA A23.3 seismic provisions over the years is necessary. The review is done by first

providing an overview of pre-seismic era (prior to the introduction of ductility requirements), followed by the era covering the evolution of modern seismic provisions with the introduction of ductility, continuity and drift control requirements. These provisions include design for flexure (strength; percentage, cut-off points, splices, development lengths and anchorage of longitudinal reinforcement); shear (strength; stirrup and tie requirements); ductility (concrete confinement and detailing of transverse reinforcement); capacity design requirements (hierarchy of yielding and relative strengths); as well as geometric constraints.

### **4.3 CSA A23 Prior to the Ductile Design Requirements of 1973**

The Canadian Standards Association has published two series of CSA A23 Standard on concrete and reinforced concrete in 1929 and 1942 under the name of “Standard Specification for Concrete and Reinforced Concrete”. They covered all aspects of concrete and reinforced concrete before they were split into four parts as CSA Standards A23.1 to A23.4, covering concrete materials and methods of concrete construction, methods of test for concrete, design of reinforced concrete structures and precast concrete – materials and construction, respectively.

The first published CSA A23.3 Standard was in 1959 under the name of “Code of Recommended Practice for Reinforced Concrete Design.” This edition of conformed to the requirements of the American Concrete Institute’s ACI 318-51. The design requirements were based on allowable concrete and reinforcement stresses for dead (D) and live (L) loads, which were defined as safe design loads. The method, referred as “Allowable Stress Method,” was used for design, construction and serviceability requirements. The 1960 and 1965 editions of the National Building Code (NBC) did not reference this standard. Instead, the 1960 NBC and 1965 NBC required reinforced concrete structures to conform to the ACI 318-56 and ACI 318-63, respectively.

The CSA A23.3 was renamed in 1970 as “Code for the Design of Plain or Reinforced Concrete Structures.” This edition of the standard was included in the NBC Supplement No. 4 of the Canadian Structural Design Manual (1970). The ultimate strength design method was included for the first time as an alternative design approach. The capacity reduction factors,  $\Phi$ , to be applied to ultimate member strengths, were introduced as

0.90 for flexure; 0.85 for diagonal tension, bond and anchorage; and 0.75 for all types of columns.

None of the above editions of reinforced concrete design standards included seismic design provisions. This remained unchanged until the introduction of ductility requirements in Chapter 19 of the CSA A23.3-1973, which was referenced by the 1975 NBCC.

#### **4.4 CSA A23.3 after the Introduction of Ductile Design Requirements in 1973**

##### **4.4.1 CSA A23.3-73**

The 1975 NBCC is the first edition of the code that emphasised ductility by expanding the base shear parameter ( $K$ ) to seven building systems based on the ductility of the structural system, ranging from ductile moment resisting frames to non-ductile frames and unreinforced masonry buildings. The ductility requirements in NBCC were ensured through the seismic design and detailing requirements specified in CSA A23.3-73 "Design of Concrete Structures for Buildings," and CSA S16-1969 "Steel Structures for Buildings".

The special provisions for seismic design in CSA A23.3-73 were intended for buildings designed to resist earthquake forces in a ductile manner. The provisions included general requirements and assumptions, special design and detailing requirements for frames, beam column connections, and walls. The Standard emphasized reinforcement detailing for concrete confinement; prevention of longitudinal reinforcement buckling; minimum percentages for flexural reinforcement; longitudinal reinforcement detailing (cut-off points, splices and development lengths, reinforcement anchorage); joint shear; and prevention of brittle shear failure. In addition, the design would promote the desired sequence of energy dissipation among members (Collins and Uzumeri 1981). These requirements were essentially the same as those in ACI 318-71 building code in the US.

The design of flexural members included requirements for ductility and continuity of longitudinal reinforcement. The maximum and minimum percentages of flexural reinforcement remained as 50% of that corresponding to balanced section and  $200/f_y$ ,

respectively, where  $f_y$  was in psi (or  $1.4/f_y$  when  $f_y$  was in MPa) as in the case of non-seismic design. However, because of the possibility of the occurrence of both negative and positive moments in any section along the length of the member under reversed cyclic loading, some minimum design capacity was required in both the negative and positive bending regions. Accordingly, at least one third of the negative beam moment reinforcement at the face of the column would extend towards the beam positive moment region beyond the inflection point, by development length but not less than one quarter of the clear span length at either end of the beam. In addition, one quarter of the larger top tension reinforcement at beam ends would be continues throughout the beam length. As for the bottom beam positive reinforcement, they would continue and extend into the columns at both ends to provide a positive moment capacity greater than 50% of the larger of the two end negative moment capacities.

Web shear reinforcement was required throughout the entire length of beams to resist shear forces resulting from gravity loads and nominal yield moments at member ends associated with the formation of plastic hinges. The minimum stirrup size was set to be #3US (with 29% less area than 10M) and a maximum spacing was specified to be  $d/2$  (where  $d$  is equal to the effective beam depth). In addition, the stirrup spacing was limited to 16 times the longitudinal bar diameter or 300 mm to control compression bar buckling. The potential plastic hinge regions at the ends of beams were required to be confined by closely spaced stirrups. The confined end region was specified as a distance of at least twice the effective depth from the column face. The stirrups shall be provided of an amount of not less than 15% of the greater of the tension and compression reinforcement and shall be spaced of a distance not exceeding  $d/4$ . The first stirrups shall be located within 75 mm from the column face.

The CSA Standard also set specific requirements for splices of ductile flexural members. At least two stirrup ties would be provided at all lap splices, which would be of at least 24 bar diameter or 12 inches (300 mm) in length. Welded splices were prohibited within a distance equal to plastic hinge length plus the effective depth  $d$ . Furthermore, no lap splicing was permitted in tension region or regions subjected to stress reversals unless seismic web shear reinforcement requirements were satisfied.

The requirements for columns under axial loads and bending included limits for longitudinal reinforcement ratio,  $\rho$ , between minimum and maximum values of 1% and 6%, respectively. The concrete column core was required to be confined with special transverse reinforcement when the maximum design axial compressive load during an earthquake,  $P_e$ , was larger than 40% of the balanced axial load,  $P_b$ . Otherwise the confinement reinforcement intended for flexural members was adequate. No detailing requirement was indicated for the confinement reinforcement, leaving it up to the engineering judgement of the designer.

Column confinement with spirals or hoops was required to be provided above and below beam-column connections for a distance from the face of the connection equal to at least the longer dimension of the column or column diameter, or 18 inches (450 mm), or 1/6 of the clear height of the column, whichever was less. The hoop spacing, measured centre-to-centre, was required not to exceed 4 inches (100 mm) and the minimum transverse re-bar size was required to be the same as that for non-seismic columns. In addition, supplementary crossties of the same tie diameter might be used to minimize the unsupported length of hoops. The minimum cover for the supplementary crossties was set to be 1/2 inch (13 mm). The Standard also specified the volumetric ratio of spiral steel not to be less than that required for non-seismic columns, provided that it was not less than  $0.12 f'_c/f_y$ . If rectangular hoop reinforcement was to be used, the total required area of hoop steel perpendicular to the direction of seismic load,  $A_{sh}$ , was to be computed as:

$$[4.1] \quad A_{sh} = \frac{1}{2} (l_n \rho_n S_h)$$

Where  $l_n$  is the maximum unsupported length of hoop,  $\rho_n$  is the volumetric ratio of spirally used hoops and  $S_h$  is the centre to centre hoop spacing.

The shear design requirements for columns were intended to prevent brittle shear failure by ensuring column shear capacity to be at least equal to shear corresponding to the formation of plastic hinges at member ends. Thus, supplementary ties in addition to the transverse reinforcement along the column height would be provided if needed. The maximum spacing of the shear reinforcement in columns was set to be 1/2 the effective depth.

The splices of vertical column reinforcement was required to be not less than 30 bar diameters or 16 inches (400 mm) whichever was greater. In addition, the Standard specified that, if continuity were established by welding or the use of mechanical devices, not more than  $\frac{1}{4}$  of the bars would be spliced at any level. A distance of 12 inch (300 mm) would be the minimum distance between the splice locations.

Seismic requirements for beam-column connections in ductile frames were specified to implement strong-column weak beam concept. Accordingly, at any beam-column connection, the sum of column moment strengths would be greater than the sum of beam moment strengths along each principle plane, unless moment capacities of confined column cores were sufficient to resist the applied design loads. An exception to this requirement occurred if the remaining columns and flexural members in the frame were capable of resisting the entire storey shear at that level, including the effect of torsion. The design shear force would be computed as the maximum of shear computed by analysis, or that computed by assuming the yielding of beams.

Beam-column connections were required to be reinforced against joint shear by continuing the column transverse reinforcement into the joint. If a joint was confined by four beams framing into the joint with each beam having at least a width of not less than the column width, and a depth of no less than  $\frac{3}{4}$  of the deepest beam framing into the joint, then the joint transverse reinforcement could be reduced by one half.

The requirements for ductile flexural walls were explained in detail in the Commentary on CSA Standard A23.3-1973 and the provisions in the 1975 NBCC were more comprehensive than the corresponding ACI code provisions for “special shear walls”, (Uzumeri et al. 1978). A design procedure was given in Appendix A of the commentary as a mean to achieve sufficient ductility in a flexural wall for which the sectional ductility would be calculated and would be required to be at least three folds. Wall ductility detailing provisions covered a number of topics, including; minimum amount of distributed vertical and horizontal reinforcement, concentrated vertical reinforcement at wall ends within potential plastic hinge regions, reinforcement splices at wall ends, and transverse buckling restraining ties in the end boundary regions to prevent bar buckling. To ensure adequate protection against web distress during formation of plastic hinges, minimum areas of distributed horizontal and vertical reinforcements of 0.0025 and

0.0015 times the wall cross sectional area, respectively were specified. In addition, the maximum spacing of distributed web steel was specified as 18 inches (450 mm) except for the lower half of the structure where the value would be 12 inches (300 mm).

The requirement for the boundary element reinforcement was intended to provide sufficient strength and ductility. The area of this reinforcement was specified to be the greater of; (i) tension area of steel required to resist factored axial loads and associated moments, (ii) tension area of steel required to resist service axial loads and associated moments that crack the wall, and (iii) tension area of steel of  $0.0018 b_w d$  for grade 60 steel and of  $0.002 b_w d$  for intermediate and hard grade steels. A reduction in this reinforcement at upper levels was allowed if the plastic hinge was not expected to propagate to upper floors. The reduced area of reinforcement in this case would be  $0.001 b_w d$ .

No more than 50% of the main tension reinforcement at wall ends was permitted to be spliced at the same location. The splice length would be at least 1.7 times the calculated development length.

The ties for the concentrated boundary element reinforcement was required to be spaced in accordance with the requirements for columns. An exception was made for regions above and below the expected plastic hinge regions to limit the tie spacing to 8 times the vertical bar diameter.

The seismic provisions also provide requirements for flange width dimension at wall ends whether it was designed as I, L, C, or T shaped sections. It was specified not to extend further from the face of the web  $\frac{1}{2}$  the distance to an adjacent shear wall web or 10% of the wall height.

Wall shear design requirements were intended to prevent shear failure in plastic hinge regions. Wall sections were required to have a minimum shear capacity,  $V_{uc}$ , equal to 1.1 times the shear force associated with wall moment capacity,  $M_{uc}$ .

$$[4.2] \quad V_{uc} = 1.1 F V_u$$

Where  $V_u$  is the factored shear obtained from seismic analysis and  $F$  is the ratio of flexural capacity to the factored moment at the wall base. The maximum design shear stress  $V_{uc}$  was given as  $0.83 \sqrt{f'_c}$ .

The standard provided requirements for shear friction to ensure adequate protection against sliding shear at construction joints. Accordingly, the total amount of vertical reinforcement crossing the joint was required to be not less than  $V_{uc} / (0.85 f_y)$ , where  $f_y$  is the specified yield strength of steel.

Additional general design requirements and assumptions were specified as part of the special provision for seismic design. The minimum specified concrete strength,  $f'_c$ , was specified to be 3000 psi (20 MPa), while the maximum specified strength for structural lightweight concrete was 4000 psi (30 MPa). The maximum specified yield strength of reinforcement was given to be 60000 psi (414 MPa). The substitution of higher grades was not permitted and the strength established by tests was not to exceed the specified value by more than 18000 psi (124 MPa). The interaction of all structural and non-structural elements contributing to seismic mass was to be considered in the analysis. Non-continuous walls, stiff partitions from storey to storey, and the columns supporting these walls and partitions were required to be designed to carry maximum forces associated with moment capacity, including gravity loads. The overall structural design was based on the requirement that a ductile structure would be designed consisting of flexural members and columns that could sustain reversible inelastic lateral deformations through plastic hinging of critical regions when subjected to major earthquakes.

The capacity reduction factors,  $\Phi$ , previously specified in CSA A23.3-70 remained the same except for columns. This edition of the standard specified capacity reduction factors of 0.75 and 0.7 for spirally reinforced columns and for all other columns under compression, respectively, as opposed to 0.75 used earlier for all column types.

#### **4.4.2 CSA A23.3-M77**

This edition of the CSA Standard was based on the SI system of units, which was done for the first time. However, the “Special Provisions for Seismic Design” in this edition



remained identical to those in CSA A23.3-73. This Standard was referenced in the 1980 NBCC.

#### **4.4.3 CSA A23.3-M84**

This edition of the Standard adopted new strength reduction factors, applied to materials, rather than member strengths under different stress conditions. These material resistance factors were applied to material strengths for computing member resistances. They were defined to be  $\phi_c = 0.6$  for concrete,  $\phi_s = 0.85$  for reinforcing bars and  $\phi_p = 0.9$  for prestressing tendons. The Standard included expanded seismic design provisions in Chapter 21, essentially following the requirements of Appendix A of ACI 318-83. It was referenced by the 1985 NBCC for conformity of ductility and structural detailing. The new standard included seismic design provisions for frames, columns, beam column connections, and walls. In addition, new clauses were provided for frame members not considered as part of the lateral force resisting system, as well as members requiring nominal ductility. A new set of requirements were added for earthquake resistant coupled walls.

The seismic base shear in 1985 NBCC was a function of the energy dissipation capacity of the structure as characterized by the coefficient K. The CSA A23.3-M84 indicated that structures designated and detailed as a ductile moment resisting frames needed only to be designed for a base shear of about 1/3 of that required if the structure were to remain elastic (i.e.  $K = 2.0$ ). Therefore, the seismic provisions of CSA A23.3 were completely revised to accommodate design and detailing requirements for structures designed using K value of 1.3 or less (CPCA 1985). In addition, the stability of the lateral load resisting system at inelastic displacements, larger than those computed based on elastic analysis using equivalent static load procedure, needed to be investigated. Inelastic displacements could be estimated by multiplying the elastic displacements calculated from factored forces using linear analysis by a factor of  $2.0/K$  as stated in the Explanatory notes on CSA Standard A23.3-M84 (CPCA 1985). For frame members that are part of the lateral force resisting system subjected to flexure, the CSA Standard added two new requirements related to axial force and geometry. Accordingly, the axial load was limited to 10% of  $(A_g f'_c)$ ; the clear span had to be greater than 4 times the effective member depth; and the width of the section had to be greater than 30% of its depth and 250 mm, and not greater than the width of the supporting member. These

requirements were intended to allow for unexpected deformations and moment redistributions from severe earthquake loadings.

The maximum limit for beam longitudinal reinforcement was redefined as  $\rho = 0.0025$  to avoid excessive steel congestions and excessive joint shear stresses. (CPCA 1985). Furthermore, lap splicing of longitudinal reinforcement was disallowed unless hoop or spiral reinforcement was provided over the lap length. In addition, lap splices were not permitted within column-beam joints, or within a distance  $(2d)$  and  $(d)$  from the face of the joint and from any expected plastic hinge region, respectively. The maximum spacing of transverse reinforcement enclosing lapped bars was specified not to exceed  $d/4$  or 100 mm.

The requirements for beam transverse reinforcement remained essentially the same as those specified by the previous edition of the Standard, with little changes. The hoops were designed to satisfy shear requirements and be provided over a length equal to  $2d$  measured from the face of the joint and over the expected plastic regions for a distance of effective depth,  $d$  on either side of the hinge. The first hoop was to be located not more than 50 mm away from the face of the support, compared to 75 mm in 1977 CSA A23.3. The hoop spacing was revised from not exceeding 16 bar diameter or 300 mm in the earlier Standard to not exceeding  $d/4$  or 8 times the smallest longitudinal bar or 24 times the hoop bar diameter or 300 in CSA A23.3-M84. Hoop reinforcement for intentionally allocated plastic hinge detailing was also specified. Figure 4.1 illustrates these detailing requirements.

The CSA A23.3-M84 defined ductile frame members subjected to flexure and axial loads as columns if subjected to factored axial compressive loads in excess of 10% of  $(A_g f'_c)$ . Geometric constraints were defined for column cross-sections. The shortest sectional dimension was required to be greater than 250 mm and greater than 40% of the sectional dimension in the orthogonal direction.

The concept of strong columns and weak beams was also employed in this edition for energy dissipation through yielding of flexure-dominant members. The formation of ductile plastic hinges was promoted to occur in beams, tolerating larger rotations, rather than in columns, which are, members subjected to axial compression. This was

achieved by computing the total factored resistance of columns based on  $\Phi_c=0.6$  and  $\Phi_s= 0.85$ , and ensuring that this value would be at least 10% higher than the total nominal moment resistance of the framing beams, based on  $\Phi_c=1.0$  and  $\Phi_s= 1.0$ . This requirement is illustrated below:

$$[4.3] \quad \sum M_{rc} \geq 1.1 \sum M_{nb}$$

Where  $\sum M_{rc}$  is the sum of moments at the centre of the joint corresponding to the factored resistance of the columns framing into the joint, and  $\sum M_{nb}$  is the sum of moments at the centre of the joint corresponding to the nominal resistance of the beams and girders framing into the joint.

Lap splices of longitudinal column bars were specified to be proportioned as tensile splices, and were permitted to be located in only within the centre half of the member length unless they had a lap length of 1.3 times that required for non-seismic columns. When the latter requirement governed, the reinforcement splices could be used at any section except within the joint.

CSA A23.3-M84 revised some of the transverse steel requirements relative to those contained in CSA A23.3-M77. The volumetric reinforcement ratio,  $\rho$ , for spirals remained unchanged. However, the total cross sectional area of rectangular hoop reinforcement was increased since rectangular hoops are less effective in confining concrete. The required hoop cross sectional area,  $A_{sh}$  in each cross-sectional direction was given to be more than the larger of the following two quantities:

$$[4.4] \quad A_{sh} = 0.3 \frac{s h_c f'_c}{f_{yh}} \left( \frac{A_g}{A_{ch}} - 1 \right)$$

$$A_{sh} = 0.12 \frac{s h_c f'_c}{f_{yh}}$$

Where  $(s)$  is the centre-to-centre spacing of transverse reinforcement in mm,  $(h_c)$  is the cross sectional dimension of the column core in mm,  $(A_g)$  and  $(A_{ch})$  are the gross and core areas respectively of the section in (mm<sup>2</sup>).

The spacing of transverse reinforcement has been revised from not exceeding 100 mm in CSA A23.3-M77, to not exceeding the following quantities: (i)  $\frac{1}{4}$  the member dimension, (ii) 100 mm, and (iii) 6 times the diameter of the smallest longitudinal bar. In both Standards, the requirements for non-seismic transverse reinforcements were expected to be met. The A23.3-M84 specified also that cross ties or legs of overlapping hoops, in the cross-sectional plane could not exceed the greater of 200 mm or  $\frac{1}{3}$  of the core dimension or 300 mm. The column length,  $l_o$ , over which the confinement reinforcement was to be placed remain unchanged in this edition. Figure 4.2 shows the requirements for these reinforcement details.

The requirements for beam-column joints of ductile frames were further explained in this Standard. The tensile stress in longitudinal beam reinforcement at the face of the joint was set to be  $1.25 f_y$  for the purpose of joint shear calculation. This 1.25 factor was intended to account for the likely increase in bar stress when the plastic hinge has formed in the beams. The joint transverse reinforcement requirements remained the same as those specified in A23.3 1973 and 1977. The longitudinal column reinforcement was specified to have well distribution around the column core with centre-to-centre spacing not exceeding the larger of 200 mm or  $\frac{1}{3}$  of the core dimension. With the compliment of the transverse and longitudinal reinforcement in the joint, this edition of the Standard assumed the joints to have specific magnitude of factored shear resistance. The factored shear resistance in the joint was specified not to exceed  $(2.4 \lambda \Phi_c \sqrt{f'_c} A_j)$  for joints externally confined by framing beams, and  $(1.8 \lambda \Phi_c \sqrt{f'_c} A_j)$  for others, where  $A_j$  is the joint area. The factors  $\lambda$  and  $\Phi_c$  accounts for the type of concrete and the concrete material resistance, respectively while  $A_j$  is the minimum cross sectional area within a joint ( $\text{mm}^2$ ).

Development length provisions were introduced for the joints. The development length,  $l_{dh}$ , for a bar with  $90^\circ$  hook and a diameter greater than 35 mm used in normal density concrete was set to be greater than 8 bar diameter,  $d_b$ , or 150 mm, while for bar sizes of No.35 mm and smaller, the length was given by:

$$[4.5] \quad l_{dh} = (f_y d_b) / (5.4 \sqrt{f'_c})$$

If straight bars of No.35 mm and smaller were being used, the development length,  $l_d$ , inside the joint was specified not to be less than 2.5 and 3.5 times the length required by the hooked bar if the depth of concrete cast in one lift beneath the bar did not exceed 300 mm or it did exceed 300 mm, respectively. In addition, terminated straight bars at a joint were required to pass through the confined core of column. Lastly, there was a limitation to the bar diameter to be less than or equal  $1/24$  of the joint length to have some control on bond stresses.

New provisions for ductile flexural walls were included based on comparable provisions in the New Zealand Standard NZS3101 Part 1, 1982, "Code of Practice for the design of Concrete Structures" (CPCA 1985). New provisions were also included for earthquake resistant coupled walls.

Dimensional stability of ductile walls in potential plastic hinge regions were ensured by geometric constraints. The wall thickness within a plastic hinge region was required to be not less than 10% of the clear distance between the floors with some exception for simple rectangular walls and others sections shown in Figure 4.3. Additional requirements were implemented to avoid excessive congestion in wall boundary elements. The maximum reinforcement ratio for concentrated longitudinal steel in boundary elements was set at 6%. In addition, the diameter of the bars in the same region was required not to exceed 10% of the wall thickness.

Wall horizontal reinforcement was specified to be extended to the boundary elements at the ends of the walls, well anchored to develop the yielding stress  $f_y$ . The web reinforcement was to be placed in at least two curtains when the factored shear force exceeds  $(0.2 \lambda \Phi_c \sqrt{f'_c} A_{cv})$  where  $A_{cv}$  is the net area of concrete section in  $\text{mm}^2$  bounded by web thickness and length of section in the direction of lateral force considered.

The concentrated reinforcement requirements in regions of plastic hinges were also modified. The minimum area of the reinforcement in boundary elements was increased to  $(0.002 b_w l_w)$  using full horizontal length of the wall,  $l_w$ , instead of the previously used value of  $(0.002 b_w d)$ . The tie spacing in boundary elements was specified not to exceed the least of 6 longitudinal bar diameters; 24 tie diameters;  $1/2$  the wall thickness,  $b_w$  or that required for concrete confinement.

Additional wall design requirements were introduced relevant to wall ductility. It was specified that the depth of the compression zone,  $C_c$ , could not exceed  $0.4 l_w$  (in which case the wall will be considered to be too brittle). When  $C_c$  exceeds  $0.1 l_w \gamma_w$  (in which case the wall requires additional measures to improve ductility), the compression region of the wall would be required to be confined as a column over a distance of not less than  $C_c (0.4 + 1.5 C_c/l_w)$  from the compression wall face.  $C_c$  can be computed either by using the approximate expression given in Eq. 4.6, or by calculating the flexural wall resistance under the action of axial load,  $P_s$ , resulting from dead and live loads, and earthquake induced transfer force,  $P_q$ .

$$[4.6] \quad C_c = \frac{P_s + P_q - \phi_c f_c A_f}{\phi_c f_c b_w}$$

Where  $A_f$  is the flange area in ( $\text{mm}^2$ ),  $l_w$  is the horizontal length of the wall,  $\gamma_w$  is the wall overstrength factor,  $\phi_c$  is the concrete resistance factor, and  $b_w$  is the wall thickness.

A minimum reinforcement ratio of 0.005 was specified for any part of this confined region. Different requirements for distributed and concentrated reinforcement in ductile flexural walls are summarized in Table 4.1.

The requirements for coupled walls were specified briefly with detailing of coupling beams as primary energy dissipating systems. The coupled walls were required to be proportioned effectively so that significant amount of overturning moments could be resisted by the axial loads resulted from coupling members. In the absence of this effective proportioning,  $C_c$  would comply with Eq. (4.6), using the length of individual wall elements. Diagonal reinforcement was specified for coupling beams. The diagonal reinforcement was required to be enclosed by hoops or spirals whose spacing could not exceed the smaller of 6 diagonal bar diameters or 24 tie diameters or 100 mm. Concentrated vertical reinforcement was also required at the end of coupled walls, similar to isolated ductile walls.

The shear strength requirements were specified to ensure the development of flexural hinging prior to shear failure. For ductile frame members subjected to flexure only, as in the case of beams, members were required to resist factored shear forces associated

with the development of probable moments on the opposite faces of columns plus the moments resulting from factored tributary gravity load along the span. For ductile frame members subjected to flexure and significant axial load, as in the case of columns, the factored shear resistance was required to exceed the greater of the forces resulted from the development of beam probable moment resistances or shear forces due to factored loads, while for ductile flexural wall, the designed shear resistance would be greater than the shear associated with the development of wall plastic hinge mechanism. The shear reinforcement in members would be determined based on the procedure specified for shear design, following either the simplified or the general method of design.

The A23.3-M84 introduced for the first time design clauses for “Building members requiring nominal ductility.” These clauses consisted of design and detailing requirements for beams, columns, and walls with nominal ductility. The stringency of seismic detailing was reduced compared to that for fully ductile members. For beams, the positive moment resistance at the face of the joint was reduced from  $1/2$  to  $1/3$  of the negative moment resistance at the same joint. This amount was specified to ensure reinforcement continuity and some positive and negative moment capacities throughout the beam to allow for unexpected deformations during severe earthquake loading (CPCA 1985). The first stirrup location was also reduced from 70 mm to 50 mm from the supporting member face, and the spacing requirements was doubled relative to fully ductile beams except for the 300 mm maximum limit which was kept the same with an addition requirement of 24 times the stirrup bar diameter. Aside from the critical column end regions, the max stirrups spacing of not more than  $d/2$  was kept the same throughout the entire beam length.

Column and wall detailing requirements for nominally ductile members remained the same as those for ductile members. This was due to the uncertainty in relying on fewer ductility provisions in regions of potential inelastic actions. An additional requirement for locating the first tie in the column was added and set to be not more than  $1/2$  tie spacing from the joint face. However with respect to walls of nominal ductility, the depth of the compression block  $C_c$ , determined in accordance with Eq. 4.6, was limited to  $0.25 l_w \gamma_w$  or approximately  $0.3 l_w$ , ( $\gamma_w$  is 1.18), compared with the fully ductile wall requirement of  $0.4 l_w$ . An exception was stated to this limit if concentrated vertical reinforcement with a

minimum reinforcement ratio of 0.005, tied with non-seismic ties and provided over the outer half of the compression block.

A23.3-M84 also introduced requirements for members that are not satisfying the dimensional limits of ductile frames and labelled as “Frame members not considered as part of the lateral force resisting system”. The intent of this clause was to ensure that these structural elements, designed for gravity loading only, could continue to fulfill their gravity load carrying functions lateral displacements expected during earthquakes. They were therefore required to be designed for minimum level of ductility. Where plastic hinges were expected in these non-resisting flexural members, horizontal and transverse reinforcement similar to those specified for ductile flexural members would be provided. On the other hand, if the plastic hinges occurred in columns, hoop reinforcement would be provided over the entire length of the column.

#### **4.4.4 CSA A23.3-1994**

The title of CSA A23.3-1994 was revised to become “Design of Concrete Structures.” The introduction of new force modification factors,  $R$ , to the 1995 NBCC for ductile coupled walls and the new category for “nominally ductile”  $R$  values necessitated the expansions of relevant detailing requirements.

This edition of the Standard specified design and detailing requirements for the elements of moment resisting frames, walls, and coupled walls for the three categories of ductility levels indicated in the NBCC; ductile, nominally ductility, and non-ductile. These systems had force modification factors,  $R$ , of 3.5 to 4.0; 2.0, and 1.5, respectively.

The Standard required the use of reduced effective rigidities for elastic analysis to account for concrete cracking, but did not specify the amount of reductions to be employed. The Explanatory Notes on CSA Standard A23.3-94 (CAC 1995) provided recommended values for effective stiffness. Accordingly, beam were assigned 40% of their rigidities computed on the basis of gross cross sectional properties. Columns and walls as axially loaded members were assigned 70% of their gross properties. Diagonally and traditionally reinforced coupling beams were assigned effective properties of  $0.4 k I_g$  and  $0.2 k I_g$  respectively.  $k$  factor is defined below:



$$[4.7] \quad k = \frac{1}{[1 + 3(h / l_n)^2]}$$

Where  $h$  is the beam depth and  $l_n$  is the clear span.

A minor modification was introduced to the confinement steel requirements for columns. The rectangular hoop cross sectional area,  $A_{sh}$ , to confine ductile frame members subjected to flexure and axial loads was increased to be the larger of the amounts given below:

$$[4.8] \quad A_{sh} = 0.3 \frac{s \cdot h_c f'_c}{f_{yh}} \left( \frac{A_g}{A_{ch}} - 1 \right)$$

$$A_{sh} = 0.09 \frac{s \cdot h_c f'_c}{f_{yh}}$$

Where  $s$ ,  $h_c$ ,  $A_g$  and  $A_{ch}$  are as defined previously.

A new requirement was introduced for confinement of regions outside the potential hinge regions. If the confinement steel was not continuous throughout the length of the column, then the remainder of the column was required to be provided with centre to centre spacing of spirals or hoop reinforcement to be less than or equal to 6 times the longitudinal bar diameter or 150 mm whichever was smaller.

For ductile flexural walls, new lap splice requirements were specified to have a minimum length of 1.5 times the development length,  $l_d$ . As for the ductility calculations in plastic hinge regions, the confinement requirements were slightly modified for walls with intermediate values of compression zone depth,  $C_c$ . The minimum and maximum limits of the compression zone were increased to  $0.14 l_w$  and  $0.55 l_w$  respectively while the boundary elements that required confinement as a column, was revised to be a distance of not less than  $C_c (0.25 + C_c / l_w)$  from the compression wall face. The terminologies and parameters of the empirical equation calculating  $C_c$  of the previous code were modified as presented below:

$$[4.9] \quad C_c = \frac{P_s + P_n + P_{ns} - \alpha_1 \phi_c f_c A_f}{\alpha_1 \beta_1 \phi_c f_c b_w}$$

Where  $P_n$  is earthquake induced transfer shear forces corresponding to the nominal flexural resistance in the coupling beams above the section and  $P_{ns}$  is the nominal net force on a cross section for the direction being considered during plastic hinge formation,  $\beta_1$  and  $\alpha_1$  are the rectangular stress block parameters. All other parameters were previously defined in Eq. (4.6).  $\gamma_w$  is the individual wall over strength except for coupled walls where the combined over strengths is used. This supersedes the consideration of  $\gamma_w$  for the entire building walls in the previous edition of CSA A23.3.

The requirements of ductile coupled and partially coupled walls were further detailed in A23.3-1994. The maximum compression zone  $C_c$  was set not to exceed  $0.5 l_w \gamma_w$ , which is slightly higher than the comparable value for non-coupled walls. This requirement did not apply to coupled walls confined as ductile columns over the length of the outmost compression segment. In addition, the factored moment resistance of the wall at the end of a coupling beam was required to exceed the nominal resistance of the framing coupling beam.

The Standard increased the required spacing of transverse reinforcement in beam-column joints of ductile frames not to exceed  $\frac{1}{4}$  of the minimum member dimension; 150 mm; 6 times the diameter of the smallest longitudinal bar; or the requirement of shear transverse reinforcement. The beams framing into a joint were considered to confine the joint externally if the framing beams cover at least  $\frac{3}{4}$  column widths. The factored shear resistance at the joint was expanded from two cases in the previous Standard to three cases;  $(2.4 \lambda \Phi_c \sqrt{f_c'} A_j)$  for joints confined on all four sides;  $(1.8 \lambda \Phi_c \sqrt{f_c'} A_j)$  for joints confined on three faces or on two opposite faces; and  $(1.5 \lambda \Phi_c \sqrt{f_c'} A_j)$  for all others, where  $A_j$  is the joint area. Development length provisions for reinforcement in beam-column joints were revised. The new requirements specified the development length,  $l_{dh}$ , for bars with  $90^\circ$  hooks, having a diameter of 35 mm and smaller the requirement increased from  $(0.185 f_y d_b / \sqrt{f_c'})$  in the previous Standard to  $(0.2 f_y d_b / \sqrt{f_c'})$  for normal density concrete.

The shear design provisions were expanded in A23.3-94. Tabulated values of shear resistance of cracked concrete,  $\beta$ , for different values of strains and factored shear stresses were introduced. Also, in addition to the previous shear provisions, special requirements were introduced for shear forces within the plastic hinge region for design with Strut and Tie method.

The requirements for designing members with nominal ductility were expanded in A23.3-94. New shear design provisions were introduced for both frames and walls. For walls, shear design requirements were similar to those for ductile members, but were less stringent. Wall thickness limit within plastic hinges was introduced to be equal or greater than 10% of the clear distance between the floors, while the limit to neutral axis,  $C_c$ , determined by Eq. (4.9) was increased to be less than  $0.33 l_w \gamma_w$ , as opposed to  $0.25 l_w \gamma_w$  in the previous Standard.

Requirements for frame members that are not part of the lateral force resisting system were separated into three categories in this edition of the standard depending on the level of ductility demand. The expected displacement of the structure played a pivotal role on the stringency of ductile detailing to be implemented.

In recognition of the increase in use of high-strength concretes in the construction industry, an upper limit was introduced for the specified concrete strength,  $f'_c$ . This limit was set to be 55 MPa because of the brittle nature of very high strength concretes, and lack of research on ductile design of high strength concretes at the time.

#### **4.4.5 CSA A23.3-2004**

In 2004, new seismic design provisions were added to A23.3-2004 to address the introduction of “Moderately ductile” category and newly added seismic force resisting systems (SFRS) in NBCC 2005. The Standard included provisions for six ductile systems and two conventional construction systems. The previous clauses were rearranged to accommodate these new categories. Furthermore, the effective elastic sectional rigidities recommended in the Explanatory Notes on CSA Standard A23.3-94 (CAC 1995) were included in the main body of CSA A23.3-04. New reduction factors for effective elastic rigidities of columns and walls,  $\alpha_c$  and  $\alpha_w$ , were introduced. The reduction factors were expressed as a function of the member axial stress ratio while the

(k) factor recommended in the earlier document (Explanatory Notes on CSA Standard A23.3-94) disappeared.

The requirements for beam design remained essentially unchanged. For ductile moment resisting frames subjected to flexure and significant axial loads, as in the case of columns, the calculation of minimum resistance of columns to ensure the strong column weak beam concept, was changed. The resistances of columns and beams were revised to include nominal and probable resistances, respectively, compared with the A23.3-94 where element resistances would be computed nominal quantities for both columns and beams. A23.3-04 required that the total nominal resistances of columns would be greater than the total probable resistances of adjoining beams and girders. This requirement is shown below:

$$[4.10] \quad \sum M_{nc} \geq \sum M_{pb}$$

Where:  $\sum M_{nc}$  and  $\sum M_{pb}$  are the sum of moments at the centre of the joint, corresponding to the nominal resistance of the columns and probable resistance of the beams and girders framing into the joint, respectively.

The transverse confinement reinforcement requirements for ductile frame members subjected to flexure and axial forces were changed and made as a function of axial load and reinforcement arrangement. The volumetric ratio,  $\rho$ , for spirals was given as:

$$[4.11] \quad \rho_s = 0.4 k_p \frac{f'_c}{f_{yh}}, \quad f_{yh} \leq 500 MPa$$

Where  $k_p$  is the ratio of factored axial load,  $P_f$  divided by the nominal axial resistance at zero eccentricity,  $P_o$ .

For rectangular column hoop reinforcement, the first equation in previous standard, shown in Eq. 4.8, was modified while the second equation remained the same. The modified expression is shown below:

$$[4.12] \quad A_{sh} = 0.2 k_n k_p \cdot \frac{A_g}{A_{ch}} \cdot \frac{f'_c}{f_{yh}} \cdot s \cdot h_c, \quad k_n = \frac{n_l}{n_l - 2}, \quad f_{yh} \leq 500 MPa$$

Where  $n_l$  is total number of longitudinal bars that are laterally supported by the corner of hoops or by hooks of seismic crossties. All other parameters in the equation are as previously defined.

Additional transverse reinforcement was also specified if concrete cover outside the confining reinforcement exceeded 100 mm. This additional reinforcement would be included in the cover with a spacing of 300 mm or less.

The maximum spacing limits for transverse reinforcement was revised. The previous limit of 100 mm was replaced with the equation given below:

$$[4.13] \quad S_x = 100 + \left( \frac{350 - h_x}{3} \right)$$

Where  $h_x$  is the maximum horizontal c/c of longitudinal bars that are laterally supported by seismic ties. The limit for distance  $h_x$  was increased from 300 mm in earlier standards (Figure 4.2), to 350 mm in A23.3-04 as illustrated in Figure 4.4. The length over which transverse reinforcement was to be provided,  $l_o$ , was amended and given as a function of factored axial loads, provided that it would not be less than 1/6 the clear column height;

$$[4.14] \quad \begin{aligned} l_o &\geq 1.5 \text{ Col}_{\max} \text{ if } P_f \leq 0.5 \Phi_c f'_c A_g \\ l_o &\geq 2.0 \text{ Col}_{\max} \text{ if } P_f > 0.5 \Phi_c f'_c A_g \end{aligned}$$

Where  $\text{Col}_{\max}$  is the largest dimension of column cross-section.

The requirements for joint shear was revised for joints with rectangular columns. The geometric limits for joint area used in the earlier edition of A23.3 (limits of 200 mm and 1/3 the core dimension in the direction considered) was replaced with new limits for joint

shear. The new limits were lowered as a function of  $(\lambda \Phi_c \sqrt{f'_c} A_j)$  from 2.4 to 2.2, 1.8 to 1.6, and 1.5 to 1.3 for joint externally confined with four framing beams, confined on three or two opposite faces, and for other joints, respectively. In addition, the development of longitudinal reinforcement in joints was revised to include the coating factor,  $k_2$ . Also, specifications for epoxy coated reinforcement were introduced.

New ductility provisions that were introduced to the NBCC 2005 resulted in corresponding revisions in A23.3-04, which was published in 2005 with the code. The new ductility related force modification factors for limits for ductile walls with and without openings, coupled or partially coupled shear walls, and coupling beams were incorporated in the standard. The requirements for squat shear walls with height-to-width ratios of  $h_w/l_w \leq 2.0$  were revised to recognize that these walls could develop plastic shear deformations rather than plastic flexural rotations. Furthermore, the new structural irregularity types of the NBCC 2005 were used for wall ductility provisions. Buildings of substantially uniform systems required special detailing within plastic hinging lengths of 1.5 times the length of longest wall above the design critical section. Vertical and horizontal reinforcements calculated at critical sections were required to be extended over these plastic hinge regions, while buildings having stiffness or geometric irregularities were required to have seismic detailing within 1.5 times the length of the longest wall above and below each of these irregularities.

The wall flange width limitation for shaped sections of I, L, C, or T, used since A23.3-1973, of not extending farther from the face of the web more than 10% of the wall height, was increased to 25% of the wall height. In addition, tie requirements for vertical distributed reinforcement has been introduced and were made to be similar to those for non-seismic provision of compression members. These ties were to be omitted if the area of vertical steel was less than  $0.005 A_g$  and the max bar size was 20M or smaller. In addition to the 300 mm maximum spacing of distributed reinforcement in regions of plastic hinging, buckling prevention provisions were introduced due to anticipated reverse cycling yielding when the area of vertical steel was greater than  $0.005 A_g$  and the max bar size was greater than 15M. The standard specified at least two curtains of reinforcement when factored shear forces at hinging regions exceeded  $(0.18 \lambda \Phi_c \sqrt{f'_c})$

$A_{cv}$ ). The steel stress to be developed in regions of plastic hinging was increased from  $f_y$  to  $1.25 f_y$ .

The minimum area of concentrated vertical reinforcement at each wall end in plastic hinge regions was reduced from  $0.002 b_w l_w$  to  $0.0015 b_w l_w$ . Concentrated reinforcement was also required for flanged walls.

A23.3-04 introduced completely new ductility provisions for ductile individual shear walls, coupled walls and partially coupled walls. These provisions superseded previous editions of A23.3, which followed the recommendations of Paulay and Uzumeri (1975). The new provisions required an estimate of the displacement demand of the seismic force resisting system due to the design earthquake, and found to be less restrictive than the old provisions for ductile walls, but considerably more restrictive for moderately (nominally) ductile walls (Adebar, P, .et. al. 2005). The concept of limit state was introduced as inelastic rotational capacities of both walls and coupling beams,  $\theta_{ic}$ , must be greater than their respective demands,  $\theta_{id}$ . The inelastic rotations were given as:

$$\begin{aligned}
 [4.15] \quad \theta_{ic} &= \left( \frac{\epsilon_{cu} l_w}{2c} - 0.002 \right) \leq 0.025 \\
 \theta_{id} &= \frac{\Delta_f R_0 R_d - \Delta_f \gamma_w}{h_w} \geq 0.004, \text{ for ductile individual walls} \\
 \theta_{id} &= \frac{\Delta_f R_0 R_d}{h_w} \geq 0.004, \text{ for ductile coupled and partially coupled walls} \\
 \theta_{id} &= \left( \frac{\Delta_f R_0 R_d}{h_w} \right) \frac{l_{cg}}{l_u}, \text{ for coupling beams}
 \end{aligned}$$

Where  $l_w$  is the length of the longest wall in the direction being considered for individual wall; lengths of the two individual wall segments for partially coupled walls; or the length of the system for coupled walls.  $l_{cg}$  is the horizontal distance between centroids of walls on either side of the coupling beam,  $l_u$  is the unsupported length between floors,  $(\Delta_f R_0 R_d)$  is the design displacement and  $(\Delta_f \gamma_w)$  is the elastic displacement. The

plastic hinge length for more conservative values was taken as  $0.5l_w$  for rotational capacity and  $l_w$  for rotational demand. The limiting value of 0.025 was stated as the upper limit of inelastic rotation capacity governed by tension steel strain, and 0.004 limit was assigned as the minimum inelastic rotational demand for minimum level of ductility. The computation of the distance to the neutral axis,  $C$ , was kept the same as before ( $C_c$ , Eq. 4.9). New requirements were introduced for coupling beams without diagonal reinforcement, and new detailing was specified for coupling beams with diagonal reinforcement. Coupling beam inelastic rotational capacity,  $\theta_{ic}$ , was specified to be 0.04 and 0.02 for coupling beams with and without diagonal reinforcement, respectively.

Factored shear resistance calculations for ductile shear walls were revised and new requirements were introduced on the basis of the limiting values of shear demands and capacities in regions of plastic hinging. The standard also introduced new requirements for detailing of moderately ductile columns of moment-resisting frames. The requirements were similar to those required for columns of ductile moment resisting frames. The coefficients for calculating transverse reinforcement of 0.4 in Eq. 4.11 and 0.2 in Eq. 4.12 were reduced to 0.3 and 0.15, respectively. New joint shear expressions were introduced for moderately ductile frame joints, and made similar to comparable requirements for joints of ductile moment resisting frames.

The requirements for moderately ductile shear walls were similar, but less stringent than those for fully ductile walls. The requirements were specified for walls having a ratio of vertical over horizontal dimensions of greater than 2.0. A limitation was added for the wall thickness in plastic hinge regions to be less than 5% of the wall unsupported length. The rotational capacity and demand calculations were those of ductile walls except that the minimum rotational demand limit of 0.004, specified in Eq. 4.15, was lowered to 0.003. These rotational calculations became redundant if the distance to the neutral axis,  $C$  computed in Eq. 4.9, was less than 15% of the horizontal length of the wall. The rotational limits were to be neglected also when the distance to the neutral axis was less than 33% of the horizontal wall length and the top wall displacement under factored loads did not exceed  $h_w/350$ .



Shear strength requirements of moderately ductile shear wall were similar to those for ductile walls, with some new shear force limitations and parameters specified for design. Squat shear walls had new requirements ensuring that factored resistance of foundations and diaphragms of the SFRS were greater than the nominal resistances of the walls. All other requirements for moderately ductile shear walls, including those for effective flange widths, maximum concentrated reinforcement ratios and bar diameters, splices, and development lengths of ductile walls were the same as those for moderately ductile Squat walls. However, additional distributed vertical and horizontal reinforcement ratios, to control the widths of diagonal cracks, were imposed and specified to be 0.003, with reinforcement spacing not exceeding 300 mm compared with the ratio of 0.0025 and a spacing of 450 mm for ductile walls. Additional requirements were specified for vertical concentrated and distribution reinforcements. For shear design requirements of Squat walls, A23.3-04 specified that design should satisfy comparable ductile walls requirements plus additional new requirements for shear force limitations, shear parameters, and amount of distributed horizontal reinforcement.

Detailing requirements for conventional construction structures, with  $R_d = 1.5$ , were introduced in A23.3-04 for the first time. The requirements covered frames, walls, and two-way slabs without beams. Those for frames were specified to prevent column hinging. The standard specified that the frame members would comply with general guidelines of non-seismic detailing of beam-column connections. In addition, requirements for buckling prevention ties were specified for columns unless the columns met seismic design requirements. Additional design requirements were specified for walls to prevent shear failures by ensuring factored shear resistances exceed factored shear loads by a specified minimum margin. Additional provisions were included for two-way slabs without beams based on the recommendations of ACI 318-02 Commentary. Topics covered included; amounts and locations of reinforcement in slab column strips; detailing of reinforcement; and shear design requirements at column faces.

The requirements for frame members not considered part of the seismic force resisting system were specified to ensure that these gravity load resisting elements continue functioning at anticipated displacement levels during earthquakes (CAC 2006). These elements were to be treated as non-structural elements without the need for any special design requirements provided that their effects on forces and deformations of structural

elements are calculated and the factored capacities of these structural elements would be sufficient to resist these effects. If considered as non-structural elements, they would be anchored properly to the rest of the structure. The members of the gravity load frames were required to be checked against the effects of combined gravity loads and earthquake induced lateral deformations to ensure sufficient nominal resistances. Otherwise, they were required to be detailed in their plastic hinge regions depending on the significance of forces generated by lateral displacements. If forces generated by  $1/2.5$  of design displacement exceeded the nominal resistance, then the beams would have to conform to the longitudinal and shear reinforcement requirements of ductile beams with stirrups spaced with a maximum spacing of  $d/2$ . Similarly, the columns would be required to be confined and designed for shear as if they were fully ductile columns, provided that their axial loads remained below 35% of the column concentric capacity. Beyond this level of axial compression the columns were required to have nominal resistance greater than that induced by lateral deformations.

The design requirements for slab-column connections were added as new provisions to prevent punching shear failures around columns during significant earthquake excitations. A reduction factor ( $R_E$ ) was specified to reduce the contribution of concrete to two-way shear. Furthermore, new transverse shear reinforcement requirements were specified for slabs.

A23.3-04 specified new upper limit for the specified concrete strength,  $f'_c$ , increasing the previous limit of 55 MPa to 80 MPa. This was concluded through research that showed a more confident prediction of ductile behaviour of high-strength concrete members when appropriately detailed (Paultre and Mitchell 2004). In addition, the resistance factor for concrete,  $\Phi_c$ , was increased from 0.6, to 0.65 with the exception of precast concrete elements produced in manufacturing plants, which had a value of 0.7. This increase was justified to achieve the same margin of safety as concrete components with compressive strengths ranging between 20 MPa and 35 MPa (CAC 2006). The resistance factor for steel,  $\Phi_s$ , remained unchanged at 0.85.

This edition of the standard also introduced new requirements for precast concrete, diaphragm systems, and foundations. The ACI 318-02 and its commentary provided basis for these requirements.

#### 4.4.6 CSA A23.3-2014

Chapter 21 of CSA A23.3 has been revised significantly in 2014. The chapter has been reorganized to group the provisions for frames first, consisting of the requirements for ductile and moderately ductile elements in two separate subsections, followed by the requirements for ductile and moderately ductile walls in one section and conventional construction in the next. Some of the geometric requirements that also pertain to non-seismic buildings have been moved to Chapter 14 where the provisions for wall design are included.

The effective section properties specified for structural analysis, reflecting the effects of concrete cracking and associated softening in the structure remained the same as before, except for the reduction factor  $\alpha_w$  for walls. The inelastic displacements are to be calculated by multiplying the elastic analysis results by  $R_d R_o / I_E$ .

Material strength limits for concrete and reinforcing steel remained the same as those in the previous edition. Special provisions for lap splices are spelled out in sections that pertain to the design of different types of elements, which are included in the relevant design clauses for these elements. Mechanical splice provisions remained essentially unchanged, with minor modifications. The provisions for special ties used in compression members for concrete confinement and reinforcement buckling prevention have been moved forward in the standard as these requirements are common for members with different ductility requirements. The confinement reinforcement requirement remained the same except for the introduction of a new coefficient defining the amount of confinement steel required as a function of the ductility level (i.e., whether  $R_d = 2.0$  or  $2.5$ ; or  $R_d > 2.5$ ). The buckling restraining reinforcement also remained the same, except it was moved from the clause for shear wall boundary elements to the general transverse reinforcement requirements at the beginning of the Chapter.

The dimensional, longitudinal steel and lap-splice requirements for ductile flexure-dominant elements (as in the case of beams) remained unchanged. Similarly, the shear design requirements for flexural members remained unchanged, except for shear design force level, which was increased from that obtained under factored load combinations using  $R_d R_o$  equals 1.0 to that obtained with  $R_d R_o$  equals 1.3. The requirements for ductile members subjected to combined flexure and axial load (as in the case of columns) and

beam-column joint shear capacities remained unchanged as well. Transverse joint reinforcement equal to that placed in columns is required in the joints unless the joint is confined by four framing beam elements.

The requirements for moderately ductile frame elements were rearranged and had minor changes. Dimensional limitations were added and made similar to the ones specified for ductile moment-resisting frames, but with reduced limits. For beams, the clear span of the member was specified to be not less than three times its effective depth (compared to four times the effective depth for ductile frame elements). For columns, the shortest cross-sectional dimension was specified to be not less than 250 mm (compared to 300 mm for ductile frame elements). The flexural design requirements remained unchanged while the shear design force level was increased from that obtained under factored load combinations using  $R_d R_o$  equals 1.0 to that obtained with  $R_d R_o$  equals 1.3. Similarly, the joint shear design force level was increased from that computed using  $R_d R_o$  equals 1.0 to that using  $R_d R_o$  equals 1.3. Joint shear capacity was also changed from  $(2.2 \lambda \Phi_c \sqrt{f_c'} A_j)$  to  $(1.7 \lambda \Phi_c \sqrt{f_c'} A_j)$  for confined joints; from  $(1.6 \lambda \Phi_c \sqrt{f_c'} A_j)$  to  $(1.2 \lambda \Phi_c \sqrt{f_c'} A_j)$  for jointly confined on three faces or two opposite faces; and from  $(1.3 \lambda \Phi_c \sqrt{f_c'} A_j)$  to  $(1.0 \lambda \Phi_c \sqrt{f_c'} A_j)$ . The joint shear reinforcement will consist of the larger of the transverse reinforcement in columns above and below. Alternatively, this requirement can be met by providing half the joint shear reinforcement provided in ductile frame joints. The spacing limits for joint reinforcement remained the same as those of the previous standard.

A23.3-14 makes a clear distinction between flexure dominant shear walls with aspect ratios of 2.0 or more and squat (low-rise) walls with aspect ratios of less than 2.0. Some of the dimensional requirements have been moved to Chapter 14 as they apply to walls in general. The plastic hinge length is defined to extend a minimum distance of half the wall length (of the longest wall or overall length of the coupled shear wall) plus 10% of the wall height above the critical wall section where the first yielding of longitudinal reinforcement is expected. For regular buildings with a clearly defined critical section at wall base, the walls may be designed for a single plastic hinge region. For irregular buildings, additional plastic hinges may have to be considered.

The new standard clearly states requirements for flexural design at critical sections, as well as above and below plastic hinge regions. The regions above and below the plastic hinge region will be protected against yielding by providing excess capacity, with a simplification of linear moment variation above the hinging region when equivalent static load method is used. The regions below the plastic hinge region may be protected against yielding by providing 20% additional tension reinforcement. Similarly, the plastic hinge regions are required to be designed to have shear resistances higher than the factored shear forces calculated using  $R_d R_o = 1.3$ , as in the case of the previous requirement. The standard provides a factor by which the higher mode effects are introduced to factored design forces for isolated shear walls. In addition, wall thickness requirements are specified based on two limits as a function of the ductility level (i.e., whether  $R_d \leq 2.5$ ; or  $R_d > 3.5$ ). Limit A was set to be  $1/14$  of the clear spans  $\ell_u$  and  $1/10 \ell_u$  for  $R_d \leq 2.5$ ; and  $R_d > 3.5$ , respectively. Limit B was set to be  $1/20 \ell_u$  and  $1/14 \ell_u$  for  $R_d \leq 2.5$ ; and  $R_d > 3.5$ , respectively. Wall thickness for plastic hinge regions should not be less than Limit A but in no case should be less than Limit B, while the wall thickness for segments outside these regions should not be less than Limit B but in no case should be less than  $1/20 \ell_u$ . Some exceptional conditions for these two limits were also specified in the Standard.

The requirements for longitudinal wall reinforcement are spelled out for concentrated and distributed wall reinforcement with different spacing limits for ductile and moderately ductile wall elements. The maximum percentage of concentrated reinforcement remained at 6% in walls designed with  $R_d \geq 3.5$ . New limits are defined for minimum percentage of concentrated vertical reinforcement as -0.05% and 0.1% outside the plastic hinge region for moderately ductile and ductile walls, respectively, with 50% increase in these limits for the plastic hinge region. All concentrated reinforcement, as well as those distributed longitudinal reinforcement that has larger than 0.5% steel area or having bar sizes greater than 20M should be protected against compression buckling by means of buckling restraining ties. New maximum horizontal reinforcement spacing was specified (400 mm) for structures having  $R_d \leq 2.5$ . The standard also provides additional detailing requirements for anchoring of horizontal wall reinforcement into the end regions, specified as three permitted anchorage types as illustrated in Figure 4.5, for different  $R_d$  values. Anchorage Types 1, 2, and 3 are permitted for wall systems designed using  $R_d = 1.5$ , while anchorage Types 2, and 3 are permitted for those

designed using  $R_d = 2.0$  or  $2.5$ , and finally anchorage Type 3, is detailed for wall systems designed using  $R_d = 3.5$  or  $4.0$ .

The new standard provided expanded versions of the previous requirements for wall ductility. The ductility of regions outside the plastic hinge region is checked against limited yielding caused by higher mode effects by ensuring that the neutral axis depth “c” does not exceed 50% and 40% of wall length for moderately ductile and ductile walls, respectively. Within the plastic hinge region, inelastic rotational demands are computed and made sure not to exceed corresponding capacities. The expressions given in CSA A23.3-04 for rotational demands and capacities remained the same, except for presenting the minimum rotational demands for ductile and moderately ductile walls together in the same clause. The ultimate concrete strain limits used in the expression for rotational demand also remained unchanged for both unconfined and confined wall boundary elements. A new clause was added as a simplified procedure for checking ductility of plastic hinges for moderately ductile walls through imposing new limits for the neutral axis depth “c”.

The clauses for the design of coupling beams with and without diagonal reinforcement have been revised and expressed more clearly in the new edition of the standard with an introduction of two new types of systems; moderately ductile coupled and partially coupled systems, while the requirements essentially remained the same as those in the earlier edition. Similarly, the requirements for wall piers in coupled shear walls remained essentially unchanged in terms of concentrated vertical reinforcement, flexural design and ductility requirements, except for being reorganized to reflect the design process more clearly. The expressions for inelastic wall and coupling beam plastic hinge rotational demands also remained unchanged, except for merging the limits for ductile and moderately ductile coupled walls in the same clause.

Shear design for shear walls have been rearranged substantially in the new edition of the standard. The shear strength requirement remained essentially the same, with the clause that addressed flexural overstrength in the earlier edition being presented more explicitly for ductile and moderately ductile walls as probable and nominal bending capacities. Shear design method is specified for ductile and nominally ductile walls as general method and simplified method, respectively. New limits are specified for factor  $\beta$ ,

accounting for shear resistance of cracked concrete, and the angle of inclination of compressive stresses  $\theta$  for plastic hinge regions, as well as for wall segments outside these regions. The requirements for squat walls, with an aspect ratio of 2.0 or less remained the same, except for the minimum shear capacity being changed to  $0.25\sqrt{f'_c}b_w\ell_w$ . A new minimum wall thickness limit of  $\ell_u/20$  has been specified for squat walls. The distributed reinforcement requirements remained the same as those for the earlier edition except for the minimum spacing being increased from 300 mm to 400mm, with the new anchorage provisions provided for horizontal reinforcement extending into the concentrated reinforcement regions of ductile walls being also applicable to squat shear walls. While the percentage of vertical distributed reinforcement required for squat walls remained the same as before, a new limit was specified for the horizontal reinforcement as a function of the shear angle,  $\theta$ .

A23.3-14 substantially expanded the seismic requirements for conventional construction forming part of the SFRS. The columns are required to have buckling restraining ties unless the structure is located in low seismic regions ( $I_E F_a S_a(0.2) < 0.2$ ) or the factored resistances are greater than the effects of factored loads using  $R_d R_o = 1.3$ . In these columns, the ties are required to continue over the storey height if the strong-column weak-beam concept is not implemented. Frame members meeting these requirements are also required to be design for shear for the higher of the shear force associated with the development of factor flexural resistances at the ends of the members or shear obtained under factor loads with  $R_d R_o = 1.3$ . Similarly, more stringent requirements are included for shear walls in conventional construction to ensure reasonably ductile response with brittle shear failures prevented. The shear design requirements of conventional squat shear walls are made similar to the requirements of moderately ductile squat shear walls, except for increasing the maximum shear stress to  $0.2 \Phi_c f'_c$ . For flexure dominant conventional shear walls with aspect ratios of 2.0 or more, A23.3-14 specified maximum shear resistance similar to that specified for ductile flexural shear walls outside the plastic hinging regions. The height (length) of the potential plastic hinge region is specified to be the horizontal length of wall above the critical section for vertical reinforcement where the first yielding is expected. The wall thickness over the plastic hinge length is specified to be not less than  $1/20$  of the clear spans  $\ell_u$ , but in no case shall be less than  $1/25 \ell_u$ , unless meeting specified conditions for reduced wall thickness. Furthermore, ductility requirements were specified for locations of the potential plastic



hinge region as well as at locations above. The ductility is checked by ensuring that the neutral axis depth “c” does not exceed 50% and 60% of the wall horizontal length for regions within and above the plastic hinge length, respectively.

As in the case of the above requirements for conventional construction, gravity load frames not forming part of the SFRS, but merely experiencing the same deformations as the SFRS, need to be designed to ensure sufficient ductility and gravity load carrying capacity during seismic response. A23.2-14 has new suggested analysis and design requirements specified for this category of construction, with significantly more details than those included in the earlier edition of the standard.

#### **4.5 Summary**

This chapter provides an overview of the evolution of seismic design and detailing requirements for reinforced concrete structures according to CSA Standard A23.3 since its inception in 1929. This review was felt essential as part of the overall scope of work in the current research project, because the selected buildings was representative of an era where the ductile design and capacity design principles of modern seismic codes had not been implemented. The level of seismic deficiency in the selected prototype building can be appreciated by reviewing the progress made in the Canadian seismic design and detailing practices. The retrofitted structure is expected to meet the requirements of the more recent codes and standards.

The review of seismic design and detailing provisions of CSA A23.3 presented in this Chapter covers the requirements of ductile, moderately ductile, nominally ductile, and conventional construction for frame elements, beam column Joints, individual walls, coupled walls and partially coupled walls. Though the current research project deals specifically with reinforced concrete frames and frame elements, the review is extended to include shear walls for completeness, as well as to draw parallel between shear walls and the retrofit system developed as a buckling restrained bracing elements, both used for lateral strength and displacement control. The extensive design and detailing requirements for shear walls provide insight into the stringency of design and the efforts involved in building such a system, in comparison with the use of buckling restrained braces developed in the experimental phase of this investigation.



The main seismic design and detailing provisions studied include the requirements for; flexural reinforcement, shear design and transverse reinforcement, beam stirrup sizes and their locations, beam-column connection details and joint shear strengths, column confinement, reinforcement anchorage splicing details, proportioning of structural members, and other general seismic design requirements, including material strength limits and geometric restrictions. This study illustrates the essential needs to assess strength and ductility of existing, potentially vulnerable and seismically deficient buildings. It also provides a feel for the mitigation requirements to attain a similar performance level as newer buildings that conform to the requirements of modern seismic codes.

Table 4.1: Reinforcement requirement for ductile flexural walls (CSA A23.3 M84)

| Region                     |                                    | Region of plastic Hinging                                                                              | Other regions                                                                                         |
|----------------------------|------------------------------------|--------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------|
| Distributed reinforcement  | Amount                             | $\rho \geq 0.0025$                                                                                     | $\rho \geq 0.0025$                                                                                    |
|                            | Spacing                            | $\leq 300 \text{ mm}$                                                                                  | $\leq 450 \text{ mm}$                                                                                 |
|                            | Horizontal reinforcement anchorage | develop $f_y$ within region of concentrated reinforcement                                              | extend into region of concentrated reinforcement                                                      |
| Concentrated reinforcement | Where required                     | At ends, corners, and junctions of walls                                                               | At ends of walls                                                                                      |
|                            | Amount (at least 4 bars)           | $\rho \geq 0.0025 b_w l_w$<br>$\rho \leq 0.06 \times \text{area of concentrated reinforcement region}$ | $\rho \geq 0.001 b_w l_w$<br>$\rho \leq 0.06 \times \text{area of concentrated reinforcement region}$ |
|                            | Hoop requirements                  | must satisfy Clause 7.6 and 21.5.6.5                                                                   | Hoop spacing according to Clause 7.6                                                                  |
|                            | Splice requirements                | Not more than 50% spliced at same location                                                             | 100% may be lap spliced                                                                               |

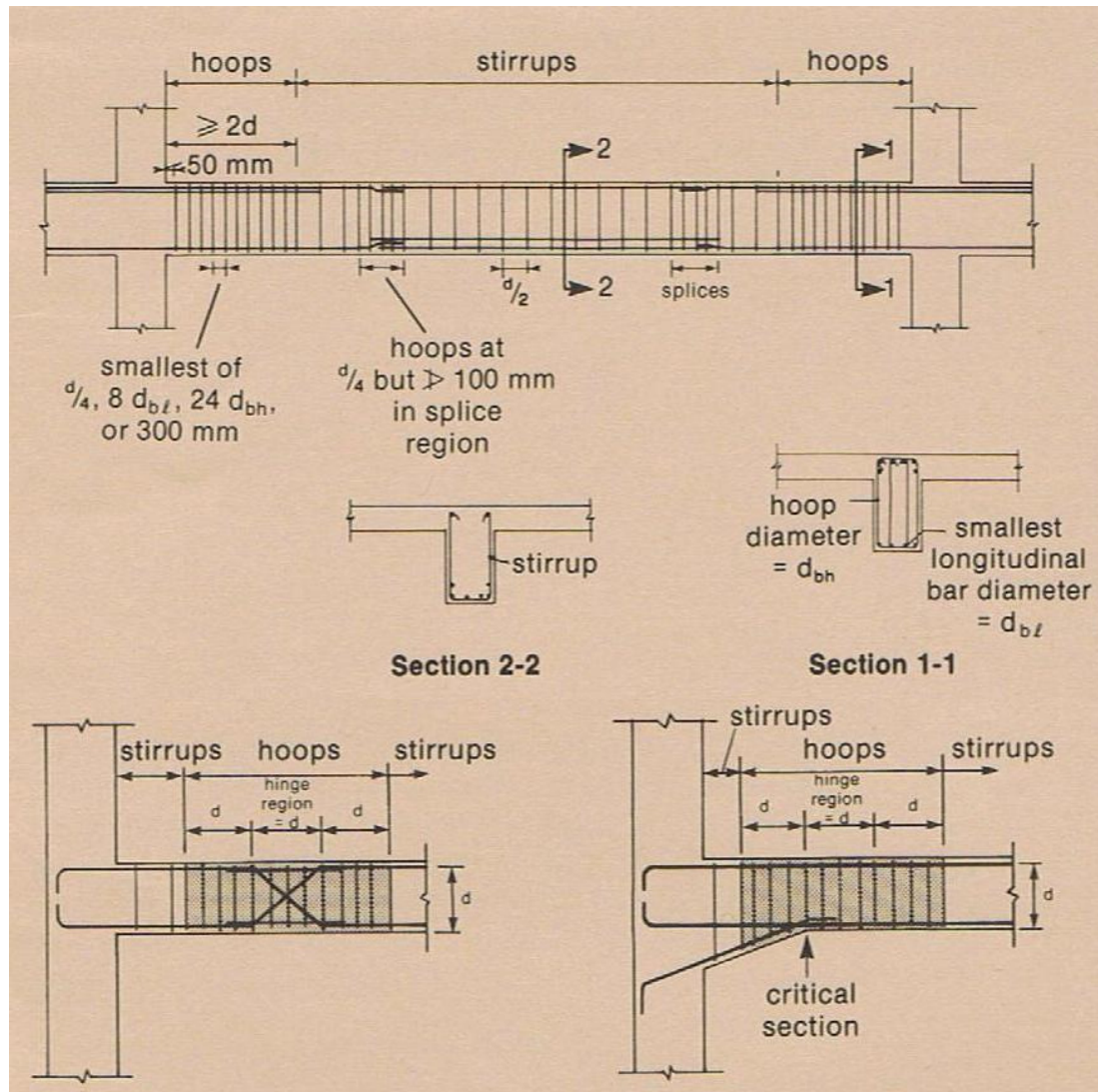


Figure 4.1: Transverse reinforcement requirements for beams and plastic hinge detailing in ductile moment resisting frames (CPCA 1985)

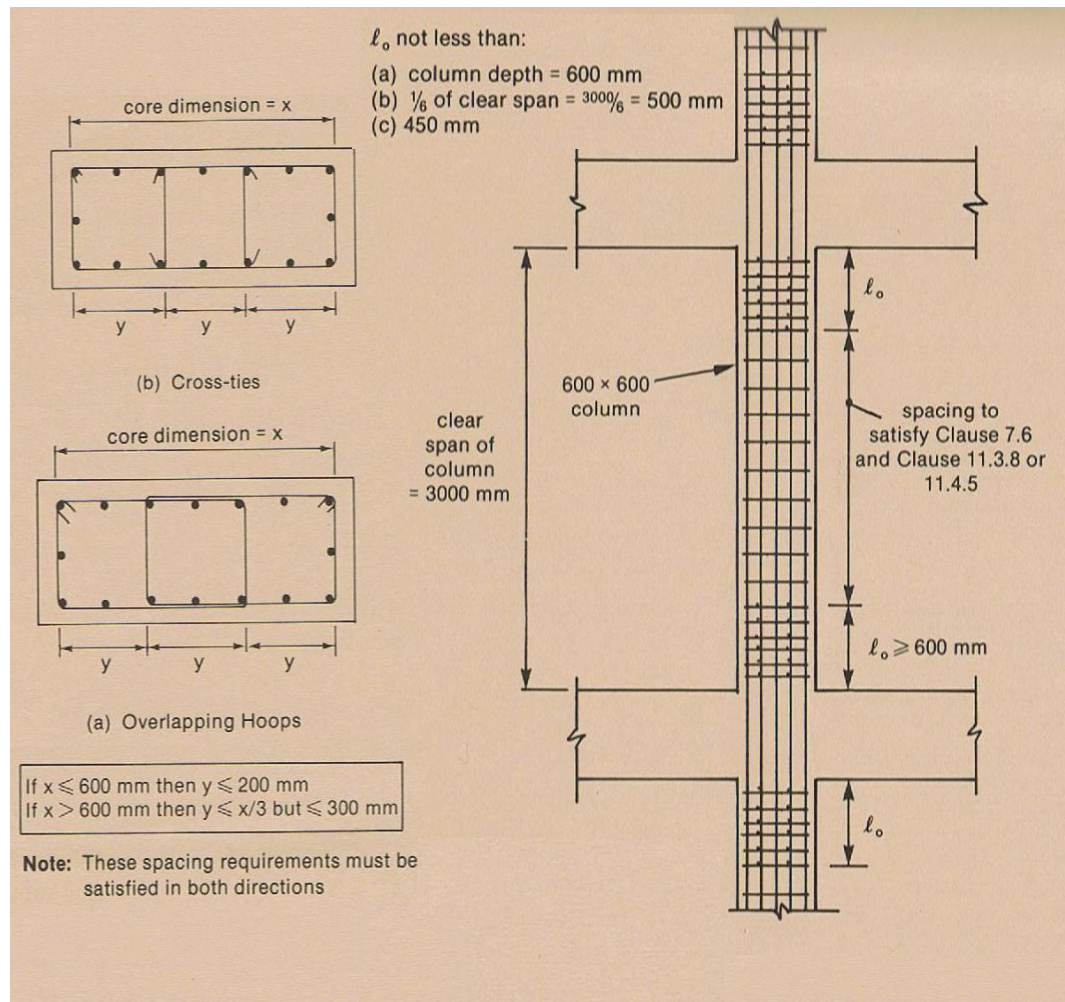


Figure 4.2: Column detailing, overlapping hoops and cross-tie specifications (CPCA 1985)

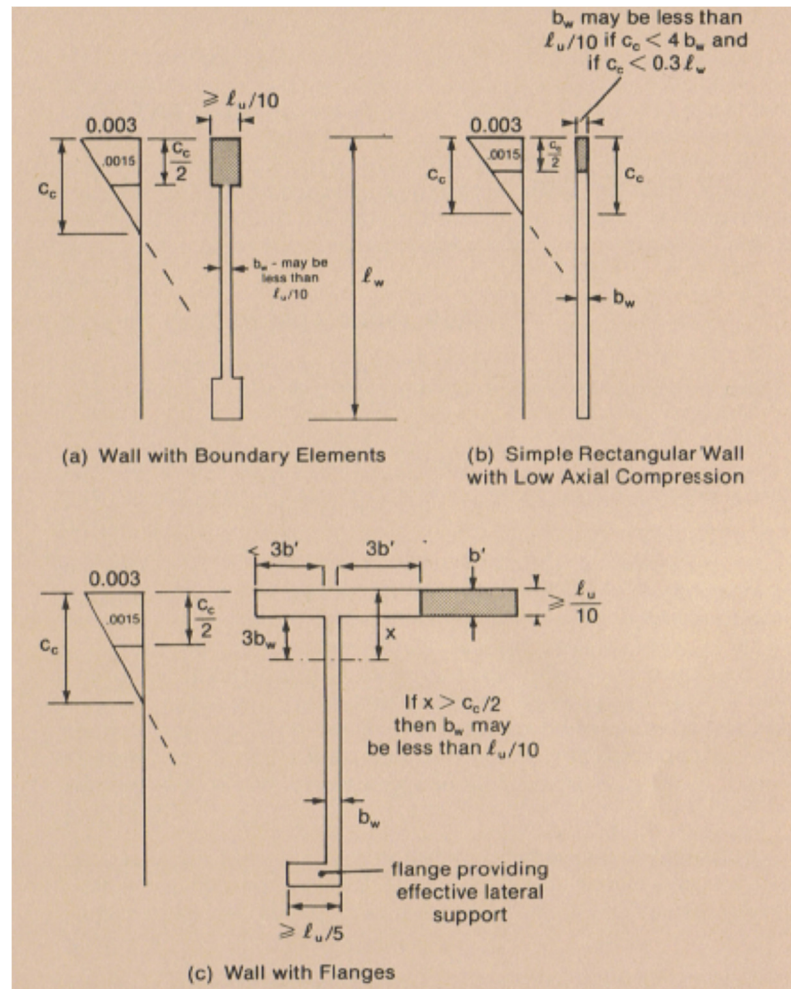


Figure 4.3: Minimum ductile wall thickness in plastic hinge regions (CPCA 1985)



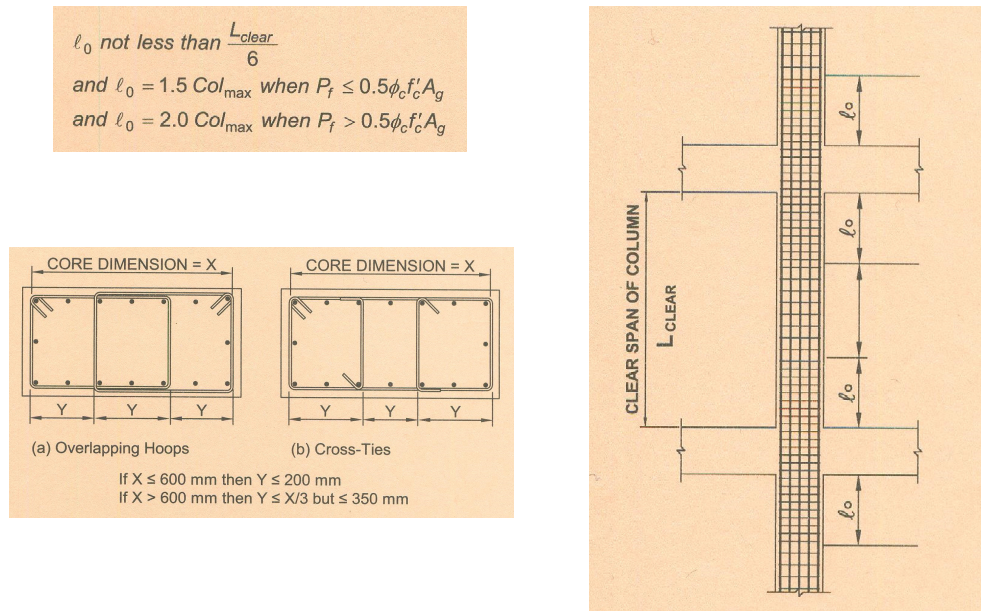


Figure 4.4: Column detailing, overlapping hoops and cross-tie specifications (CAC 2006)

| Type | Brief description                                                                                    | Figure |
|------|------------------------------------------------------------------------------------------------------|--------|
| 1    | Standard 90° hook around vertical end bar                                                            |        |
| 2    | U-bar around vertical end bars; staggered lap splices minimum five times wall thick from end of wall |        |
| 3    | Anchored within tied vertical reinforcement                                                          |        |

Figure 4.5: Anchorage types of horizontal reinforcement at the ends of ductile and moderately ductile walls (CSA A23.3 2014)

## Chapter 5

# Static and Dynamic Linear Analysis of Reinforced Concrete Frame Structures

### 5.1 General

Many reinforced concrete frame structures built before the 1970's can be classified as non-ductile and seismically deficient. The objective of this chapter is to investigate innovative, yet cost-effective seismic retrofit methodologies for seismic upgrading of non-ductile or limited ductility reinforced concrete frame buildings. A 10-storey reinforced concrete moment-resisting frame building selected for Ottawa based on design practice according during the time that the 1965 NBCC was in effect. This design is representative of seismically deficient older buildings. The building was then investigated analytically. Three retrofit schemes were considered, including the addition of: shear walls, diagonal steel braces, and diagonal prestressing cables. The analyses were conducted with 2-D models using SAP2000 and 3-D models using ETABS (CSI 2010). Both the Equivalent Static Force Procedure (ESFP) and dynamic analyses were investigated. The dynamic analyses were based on the modal response spectrum method using Ritz vector analysis. The numerical results demonstrated that seismic forces imposed on the structure computed by dynamic analysis is consistent with the ESFP; while the displacements predicted by ESFP were overestimated and hence conservative. The retrofitting schemes considered were effective in reducing roof drift demands up to 37% of that for the original as-built building. The steel bracing technique proved to be effective; while adding shear walls resulted in substantially more structural stiffness, leading to a decrease in building period, thus attracting higher seismic forces. The analysis results from the 2-D and 3-D models were found to be in good agreement.

The numerical analyses described herein were used to develop the proposed retrofit strategy of this research program, which was then verified through large-scale testing under simulated seismic loading. The results of this chapter provided the motivation to investigate experimentally and analytically steel braces as a retrofit strategy to upgrade

non-ductile and limited ductility reinforced concrete moment-resisting frames that were built according to provisions in older building codes.

## **5.2 Description of the Prototype Building**

The building investigated herein is a 10-storey reinforced concrete moment-resisting frame structure that is assumed to be located in Ottawa and represent a typical medium-rise building constructed during the era spanning the late 1960's and early 1970's. The building floor plan dimensions are similar to a 10-storey building of 40 m height that was previously designed based on the 1990 NBCC (Dincer 2003). However, the interstorey floor heights for the building investigated in this present study were modified to 3.5 m. This was judged to be more representative of the practice during the period of the 1960's and early 1970's. The cross sectional dimensions of columns and beams of the original building design were increased by approximately 10% for this study to satisfy the difference in base shear demands between the 1965 NBCC and 1990 NBCC for similar buildings. This modification is based on the comparison of all editions of the NBCC seismic base shear design force for this non-ductile ten-storey concrete frame building shown in Figures 3.7 and 3.9 of Chapter 3. The sectional dimensions of the building are acceptable since the actual design member capacities can be easily defined through the exact calculation of steel reinforcement ratios required for these proposed sections. The effect of the reinforcement ratios in the calculation of the modulus of elasticity of the concrete members were assumed to be negligible and therefore ignored in the linear static and dynamic analyses. The building details and geometry are shown in Figure 5.1.

The floor plan of the building is 18 m x 30 m and is divided into 3 bays of 6 m each in the E-W direction and 5 bays of 6 m each in the N-S direction. A symmetrical floor plan was selected to eliminate torsional effects. The structural elements consist of 550 mm square columns and 350 mm x 500 mm rectangular beams. The flooring system consists of a two-way reinforced concrete slab with a 200 mm slab thickness.

The building was analyzed under the following loads: i) roof snow load of  $2.4 \text{ kN/m}^2$ ; ii) roof mechanical load of  $2.0 \text{ kN/m}^2$  over the middle bay in the E-W direction; iii) typical floor dead load of  $1.5 \text{ kN/m}^2$ ; iv) typical roof dead load of  $1.0 \text{ kN/m}^2$ ; and v) typical floor live load of  $2.4 \text{ kN/m}^2$ . Normal density concrete was assumed with a mass density of  $2400 \text{ kg/m}^3$ .



### 5.3 Structural Analysis

The building was analyzed for two loading conditions: equivalent static seismic loads, and dynamic loading. The analyses were performed with and without retrofitting using two- and three-dimensional (2-D or 3-D) models. Computer software SAP2000 and ETABS (CSI 2010) were employed for 2-D and 3-D analyses, respectively. The different cases considered include: i) elastic static analysis under code-specified lateral forces (2-D and 3-D); and ii) 2-D elastic dynamic analysis using the mode superposition technique with the Uniform Hazard Spectrum (UHS) for the City of Ottawa. The latter analysis also provided vibration frequencies and mode shapes. Damping was considered to be 5% of critical damping, and contributions from 10 modes were included accounting for more than 90% of the total mass. The Root Sum Square (RSS) method was used to compute maximum storey shears.

#### 5.3.1 Structural Modelling

The four interior frames were lumped together and the two exterior frames were lumped together. Thereafter the lumped interior frames were connected to the lumped exterior frames to capture the entire structural stiffness and strength. Rigid links with infinite axial rigidity and very small flexural rigidity were used to connect the lumped interior frames to the lumped exterior frames. The links simulated rigid floor diaphragms ensuring equal displacements at each floor level without transferring moments. The mass of each floor was lumped and assigned to the floor joints. Figure 5.2 illustrates the 2-D model used in the analyses.

The selected retrofit schemes were only placed in the central bay of each exterior frame. The stiffness of non-structural elements, including masonry walls, was neglected. However, their weights were included in the lumped floor masses. The models of three retrofit techniques considered are shown in Figure 5.3.

These techniques included the addition of: i) reinforced concrete shear walls; ii) diagonal steel braces; and iii) diagonal prestressing cables. The shear wall thickness was 200 mm, while the braces were of Class H Square Hollow Structural Sections (HSS) of 304.8 x 304.8 x 12.7 mm. The prestressing steel cables used were high-strength, 7-wire strands of 15.47 mm diameter. In the latter technique, 4 strands were used in each diagonal without prestressing. The resulting area of prestressing steel was sufficient to

brace the building. Reduced section properties based on the 2010 NBCC were considered to account for cracking in concrete, resulting in 40%, 65% and 85% of gross moment of inertia for beams, columns and shear wall, respectively.

### **5.3.2 Uniform Hazard Spectrum (UHS)**

The UHS specified in the 2010 NBCC for the City of Ottawa was used to represent the most recent seismic hazard. Equivalent Static Force Procedure (ESFP) was employed to establish elastic seismic forces for both 2-D and 3-D analyses. This was achieved by distributing the base shear computed as per NBCC 2010 along the height of the structure following an inverted triangular force distribution. The soil condition selected was equivalent to the reference Site Class C (very dense soil and soft rock) given in NBCC 2010. There was no torsion present in the structural layout of the building, and accidental torsion as prescribed by NBCC 2010 was not considered.

## **5.4 Analysis Results**

The analysis results are compared to assess: i) the change in seismic force demands between 1970 and 2010 practice in Canada; ii) the significance of 2-D versus 3-D analysis; iii) the significance of conducting dynamic analysis relative to the equivalent static force procedure; and iv) the impact of using different retrofit techniques on structural response.

NBCC 2010 equivalent static forces were computed using two different procedures to determine the fundamental period: i) the empirical expressions provided in NBCC 2010; and ii) applying permitted increases in periods computed by other methods relative to the empirical values (up to 50% increase in the period of frame buildings and up to 100% increase in the period of shear wall and braced frame buildings when warranted by period calculations conducted on the basis of accepted procedures of mechanics). The former procedure to establish the periods resulted in higher seismic forces, whereas the latter procedure resulted in reduced forces due to reductions in accelerations for increased structural periods. Table 5.1 lists elastic base shear forces obtained from each analysis.

#### **5.4.1 Seismic Force Demands and Capacities**

Seismic force demands specified in the National Building Code of Canada have evolved over the years along with knowledge on earthquake engineering and Canadian seismicity. Therefore, the analysis of the as-built building representing conventional construction designed on the basis of the 1970 NBCC resulted in substantially different seismic force demands than those under the 2010 NBCC. The comparison indicates increase factors of 1.9 to 2.8 in base shear forces from 1970 to 2010 editions of NBCC, depending on the approach used with the 2010 NBCC. This implies that the elastic strength capacities of older frame buildings may be approximately 36% to 53% of those of newer structures. Therefore, the building considered in this analysis is expected to yield and suffer structural damage under the 2010 NBCC seismic forces. The first step in any retrofit assessment project should involve the computation of seismic force and deformation demands, while also assessing force and deformation capacities.

#### **5.4.2 Two-Dimensional Versus Three-Dimensional Analysis**

The as-built, 10-storey frame structure was analyzed using the 2-D and 3-D models shown in Figure 5.3. SAP2000 computer software (CSI 2010) was used to conduct the 2-D analysis, whereas ETABS software (CSI 2010) was employed for the 3-D analysis. The NBCC 2010 static lateral forces were applied in both cases. The results indicate nearly identical axial and shear forces in the columns as shown in Figure 5.4. These results confirm, as expected, the negligible difference in computed forces between 2-D and 3-D analysis for such buildings with symmetrical structural layouts.

The horizontal displacement at the roof level obtained by 3-D analysis was approximately 80% of that computed by the 2-D analysis. Comparison of horizontal displacements at the roof level and the critical 4<sup>th</sup> floor level, where the highest interstorey drift was observed, are shown in Figure 5.5. The interstorey drifts at the roof level were 0.45% and 0.38%, whereas they were 1.36% and 1.08% at the 4<sup>th</sup> floor level for the 2D and 3D models, respectively. The minor differences observed in computed displacements are attributed to the simplifying assumptions used in lumping the frames in the 2-D model. However, the 2-D model sufficiently represented the 3-D structure.

### 5.4.3 Equivalent Static Force Procedure Versus Dynamic Analysis

Equivalent static forces in the NBCC 2010 are generated from the UHS based on the assumption that the first mode response governs overall structural behaviour. For a single-degree-of-freedom structure, the two approaches are expected to generate very similar results. The 10-storey building considered in the current project may be affected by higher modes. Furthermore, the empirical period expression forming the basis for equivalent static forces may result in smaller than the actual period, resulting in higher base shear forces for the same design spectrum. The significance of potential differences between static and dynamic analyses is assessed in Table 5.1, which provides a summary of base shear forces obtained for each building from the 2-D elastic static and dynamic analyses. It is evident that the fundamental period computed with the empirical expressions provided in the 2010 NBCC are significantly smaller than those computed by dynamic analysis. The table also includes upper bound permitted fundamental periods and associated base shear forces according to the NBCC 2010. These permissible values indicate significantly higher period, but lower base shear force values. Dynamic base shear forces, on the contrary, are higher than those permitted by the 2010 NBCC for the retrofitted schemes and with approximate equal base shear for the As-built building. Dynamic analysis results are generally regarded as more representative of actual structural response for the buildings analyzed.

Figures 5.6 and 5.7 illustrate the variation of Eigenvalues as frequencies and the modal mass participation ratios as functions of modes of vibration. It is evident that the natural frequencies of the building increased due to retrofitting with steel braces or shear wall, and the increase is more pronounced after Mode 6. Steel bracing resulted in an increase of frequency of approximately 2.3 times for Mode 10 compared with the as-built building. Conversely, the modal mass participation ratio for the four buildings reached the 90% target as required by the NBCC during Mode 3. The cable bracing does not provide any significant increase in stiffness and hence the frequencies and modal mass participation ratios are similar.

### 5.4.4 Structural Response of Retrofitted Buildings

The seismically deficient as-built frame structure designed on the basis of the 1970 NBCC is expected to yield when subjected to the 2010 NBCC seismic loads. The ratio of base shears between the two editions of the NBCC indicates an approximate

displacement ductility demand of 2.77, which may not be available as a result of the non-ductile detailing practice of the era with the 1970 NBCC. The response of structural members of all building types were assessed based on permissible 2010 NBCC upper bound fundamental periods and associated base shear forces. The static analysis results indicate that exterior frames retrofitted with lateral bracing elements attracted significantly higher seismic shear forces, as well as axial forces in the vertical elements, relative to the as-built building. This signifies that the adequacy of these exterior frame elements should be checked, including the potential uplift of foundations and excessive horizontal shear force demands.

Figures 5.8 and 5.9 show the results of first floor column axial and shear forces at the foundation level, respectively. It is evident that the middle bay (columns 2 and 3) of the exterior frame attracted a higher portion of the total seismic loads compared to those for the outer bays (columns 1 and 4). This observation is true for all the retrofit schemes considered. However, the trend was different for the interior frames with respect to axial forces where larger forces were observed in outer bays. The added stiffness due to retrofitting played an important role in the distribution of frame forces. The highest increase or decrease in column axial and shear forces were obtained in the exterior frames of the shear wall building. Conversely, forces and moments in framing elements of the upper storeys were reduced by about 25%, as forces are transferred to the shear walls. In contrast to the shear wall building, the building retrofitted with cable bracing indicated reduced change in frame member forces.

Elastic deflections computed by static and dynamic analyses, for all models, were multiplied by  $(R_d R_o / I_E)$  to estimate the realistic displacements of structures in the inelastic range of deformations.  $R_d$  and  $R_o$  are ductility and over strength-related force modification factors. According to the 2010 NBCC these values are 1.5 and 1.3, respectively, for conventional buildings.  $I_E$  is the importance factor and is equal to 1.0. The dynamic linear analysis deflection results were scaled further, as per requirements of NBCC 2010, to a ratio of  $(V_d / V_e)$ , where  $V_d$  is the design base shear and  $V_e$  is the base shear representing elastic response, to obtain design values that account for inelastic effects. Table 5.2 lists realistic displacements and drift ratios obtained from each analysis.

Figure 5.10 shows graphically the realistic displacements at the roof level for the four buildings. The retrofitting techniques reduced the roof displacement demands by approximately 37% to 84% when applying the equivalent static force procedure and 36% to 66% with dynamic analysis, while the maximum interstorey drifts were reduced up to 33% and 46% when static and dynamic approaches were employed, respectively. Retrofitting by structural steel bracing demonstrated substantially higher reduction in drift demands than those attained by the other two retrofit methods.

The roof displacements computed through dynamic analysis were lower than those computed by static analysis for all retrofitted buildings, ranging from 33% to 60% of the static values. This can be attributed to the conservative and approximate nature of equivalent static force procedure.

## **5.5 Summary**

This chapter presented an overview of three seismic retrofit techniques that were investigated for seismically deficient reinforced concrete frame buildings. A literature review was included followed by an analytical investigation to assess the effectiveness of upgrading an existing 10-storey reinforced concrete building assumed to be located in Ottawa. The retrofit methods consisted of lateral bracing by adding reinforced concrete shear walls, diagonal steel bracings, and diagonal cable strands. The retrofit elements were all implemented in the central bay of the exterior building frames. Two- and three-dimensional analytical models were used to conduct linear static and dynamic analyses.

The results indicate that the selected retrofit techniques are effective in reducing roof displacements to 37% of that for the original as-built building, limiting deformations in non-ductile frame elements to elastic ranges. The drift control upgrades are generally accompanied by increased building stiffness and frequency, which in turn may attract higher seismic forces. Therefore, a thorough investigation of the entire building system is recommended to assess structural capacities and demands before an appropriate retrofit scheme is selected. This study demonstrated that linear elastic static analysis produces conservative results in predicting roof displacements, as compared with linear elastic dynamic analysis. The analytical results further indicate that the use of structural steel bracing results in the highest reduction in deformations. However, this system induces high foundation forces, potentially necessitating foundation upgrades. Similarly

effective drift control can be attained by shear walls, with appropriate attention paid to the foundation force demands. The cable bracing technique appears to provide sufficient bracing action without shortening the building period significantly and without imposing high force demands on the foundation.

Table 5.1: Comparison of elastic base shear forces: ESFP vs dynamic modal analysis

| Building Type               | NBCC | Equivalent Static Force Approach (NBCC 2010) |        |                  |        | Dynamic Modal Analysis |                     |                     |                                              |
|-----------------------------|------|----------------------------------------------|--------|------------------|--------|------------------------|---------------------|---------------------|----------------------------------------------|
|                             |      | NBCC Empirical                               |        | NBCC Permissible |        | Based on 2010 UHS      |                     |                     |                                              |
|                             |      | T (Sec)                                      | V (kN) | T (sec)          | V (kN) | T (Sec)                | V <sub>e</sub> (kN) | V <sub>d</sub> (kN) | (V <sub>d</sub> ) <sub>min</sub> = 0.8V (kN) |
| As-built                    | 2010 | 1.08                                         | 3217   | 1.62             | 2198   | 3                      | 2179                | 1117                | 1758                                         |
|                             | 1970 | 1.0                                          | 1162   | -                | -      | -                      | -                   | -                   | -                                            |
| Retro. by Shear Walls       | 2010 | 0.72                                         | 5620   | 1.44             | 3913   | 1.89                   | 4678                | 2399                | 3130                                         |
| Retro. by Steel Braces      | 2010 | 0.88                                         | 4320   | 1.49             | 2795   | 1.49                   | 5341                | 2739                | 2235                                         |
| Retro. by P/S Strand Braces | 2010 | 0.88                                         | 4320   | 1.76             | 2283   | 2.62                   | 2329                | 1194                | 1827                                         |

Table 5.2: Comparison of roof displacements: ESFP vs dynamic modal analysis (mm)

| Building Type           | NBCC 2010 Based on ESFP Permissible |                    |                                                             |                        | Dynamic Analysis                 |           |                                                             |                        |
|-------------------------|-------------------------------------|--------------------|-------------------------------------------------------------|------------------------|----------------------------------|-----------|-------------------------------------------------------------|------------------------|
|                         | Roof                                |                    |                                                             | Any Floor              | Roof                             |           |                                                             | Any Floor              |
|                         | $\Delta_{\text{Realistic}}$ (mm)    | Building Drift (%) | Ratio ( $\Delta_{\text{retro.}}/\Delta_{\text{As-built}}$ ) | Inter-storey Drift (%) | $\Delta_{\text{Realistic}}$ (mm) | Drift (%) | Ratio ( $\Delta_{\text{Retro.}}/\Delta_{\text{As-built}}$ ) | Inter-storey Drift (%) |
| As-built                | 348                                 | 0.99               | 1.00                                                        | 0.70                   | 210                              | 0.38      | 1.00                                                        | 0.55                   |
| Retro. by Shear Walls   | 294                                 | 0.84               | 0.85                                                        | 0.52                   | 96                               | 0.21      | 0.46                                                        | 0.25                   |
| Retro. by Steel Braces  | 130                                 | 0.37               | 0.37                                                        | 0.23                   | 76                               | 0.22      | 0.36                                                        | 0.25                   |
| Retro. by Strand Braces | 243                                 | 0.70               | 0.70                                                        | 0.48                   | 138                              | 0.26      | 0.66                                                        | 0.37                   |



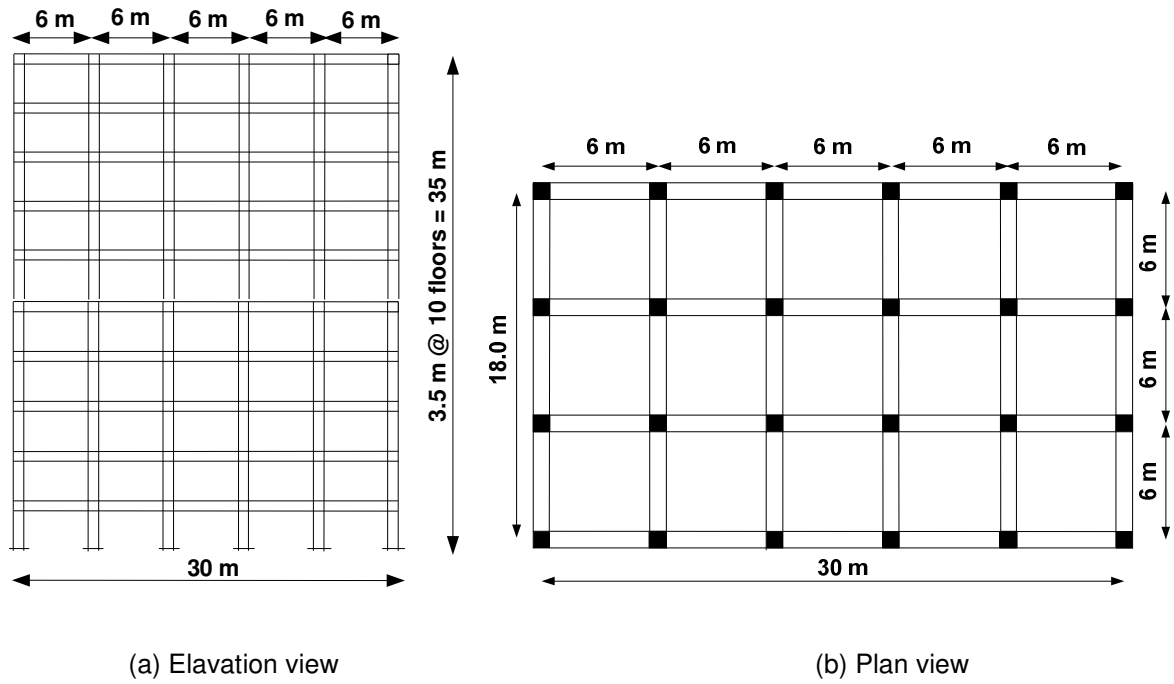


Figure 5.1: Plan and elevation views of the as-built building (modified from Dincer 2003)

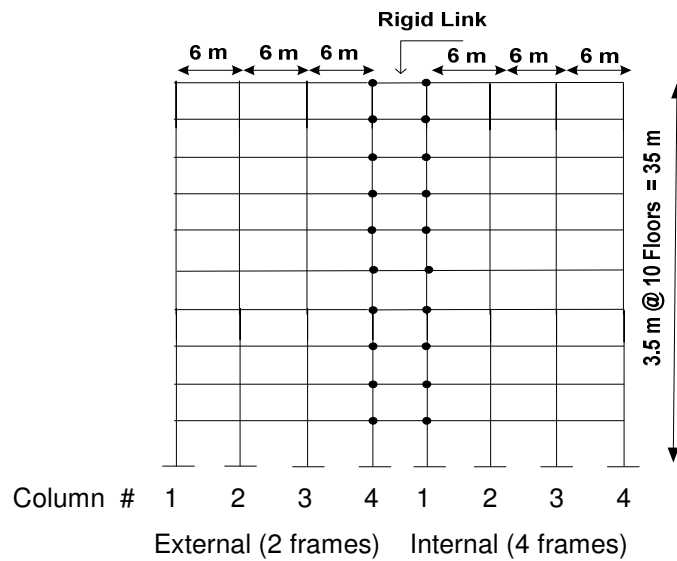


Figure 5.2: Lumped frames in the 2-D model

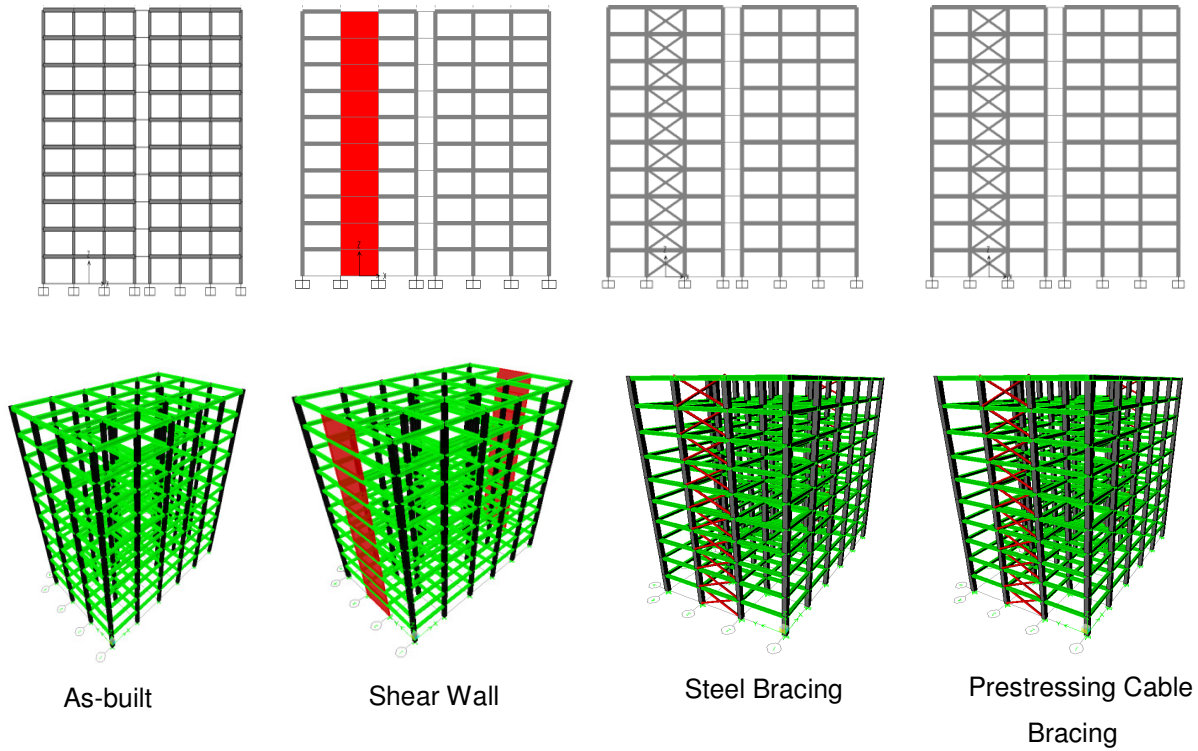


Figure 5.3: 2-D and 3-D models of the buildings analyzed

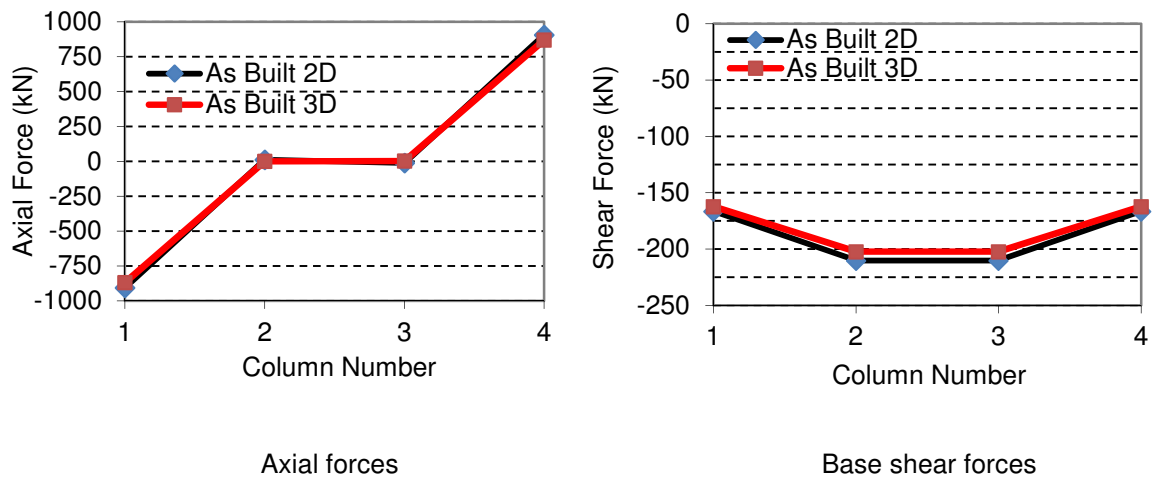


Figure 5.4: Ground floor column forces at the foundation level of exterior frames

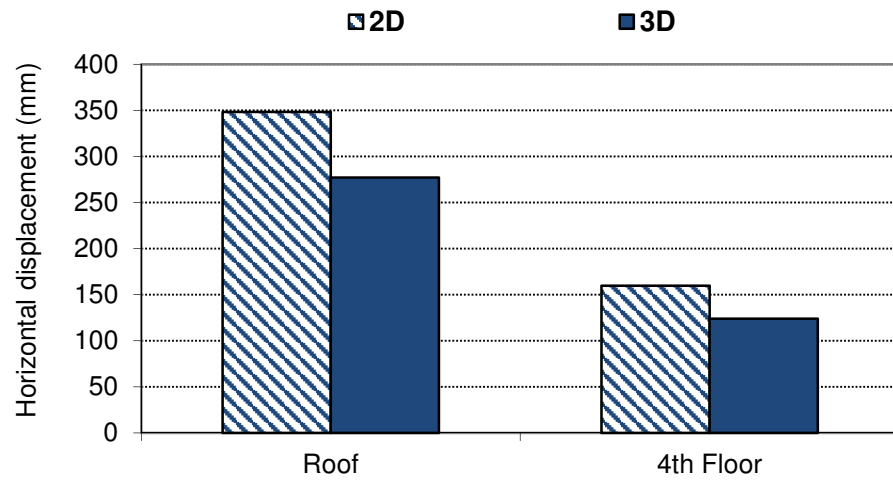


Figure 5.5: Comparison of horizontal displacements of the as-built model:  
2-D and 3-D analysis

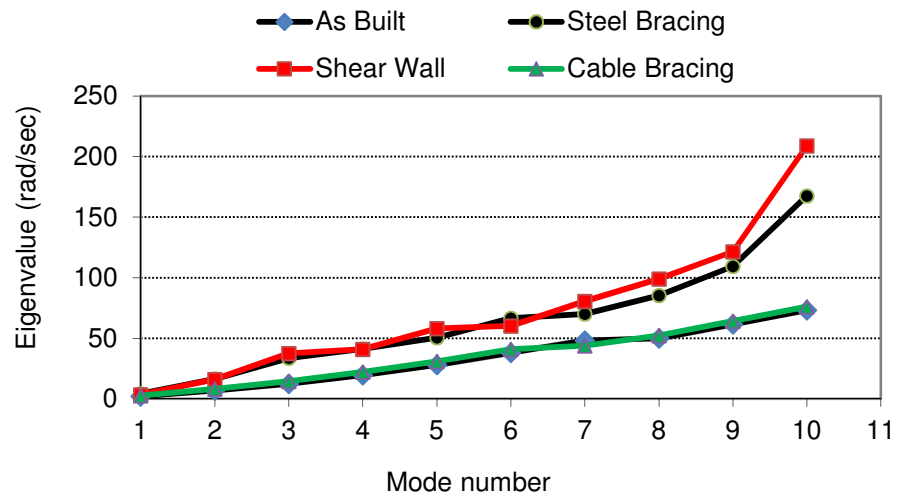


Figure 5.6: Eigenvalues from dynamic analysis

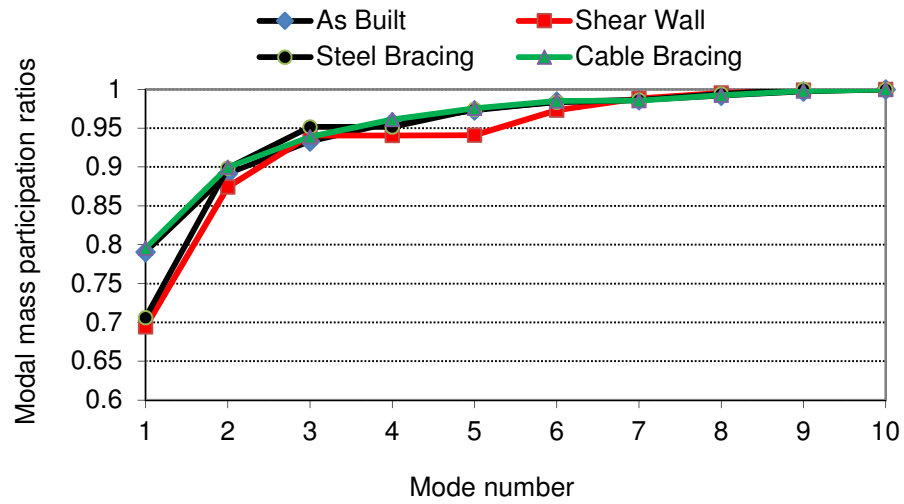
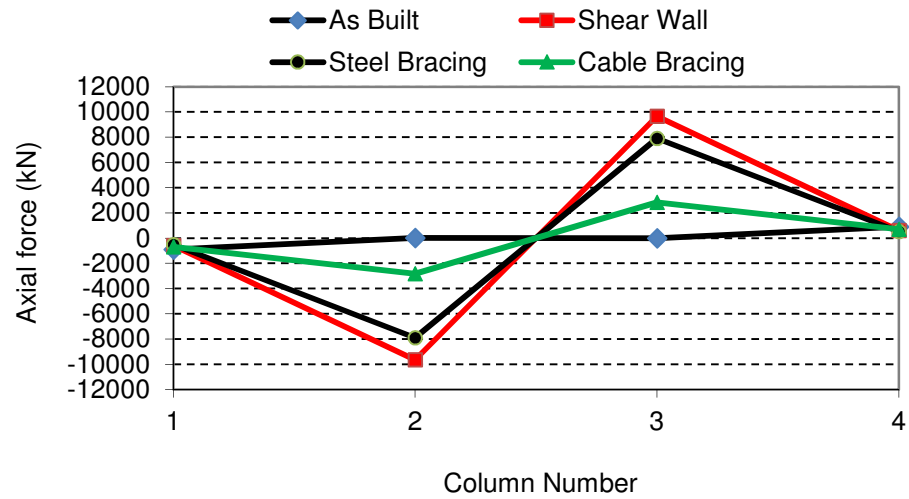
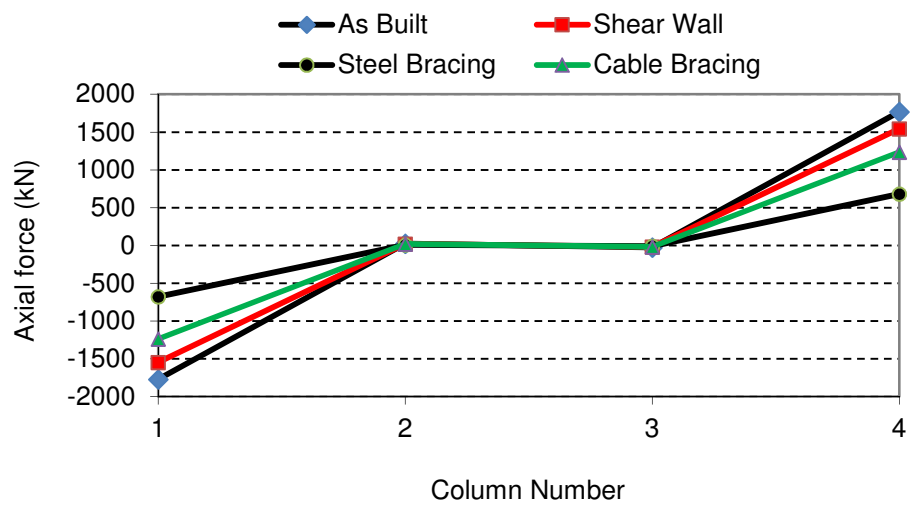


Figure 5.7: Modal mass participation ratios

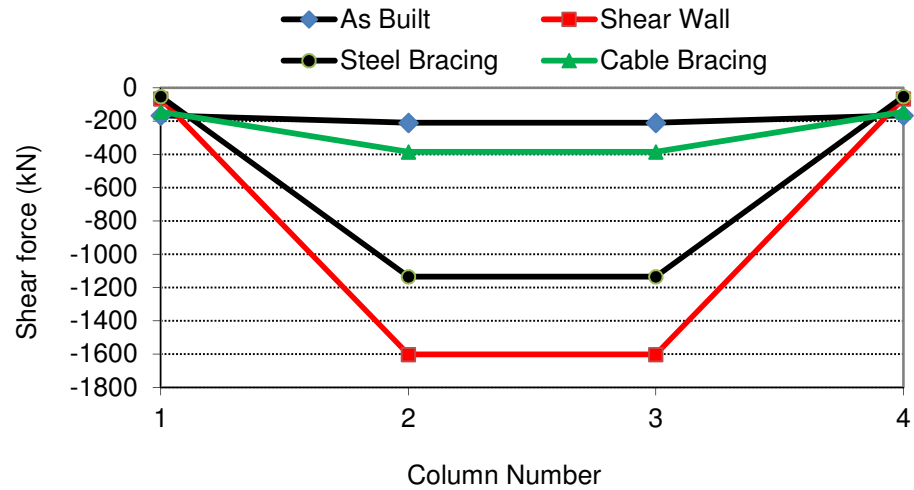


Exterior frames

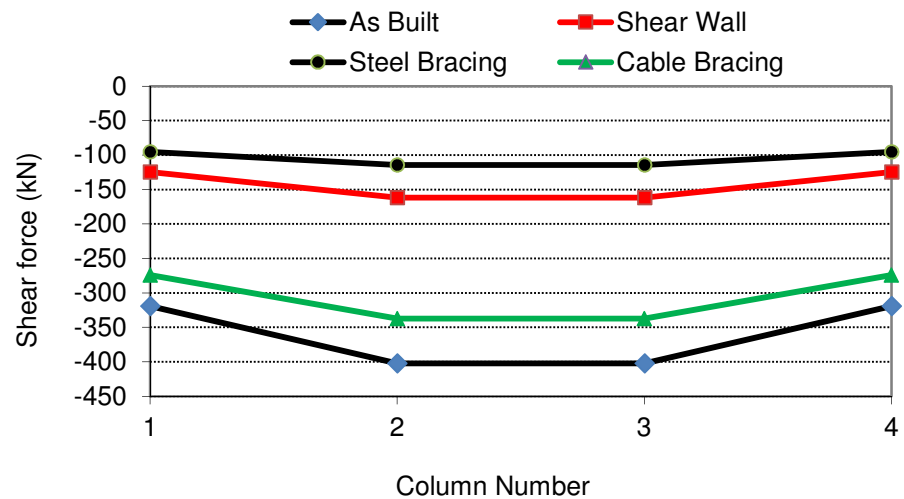


Interior frames

Figure 5.8: Effects of retrofitting on first-storey columns: axial forces at foundation level



Exterior Frames



Interior Frames

Figure 5.9: Effects of retrofitting on first-storey columns: shear forces at foundation level

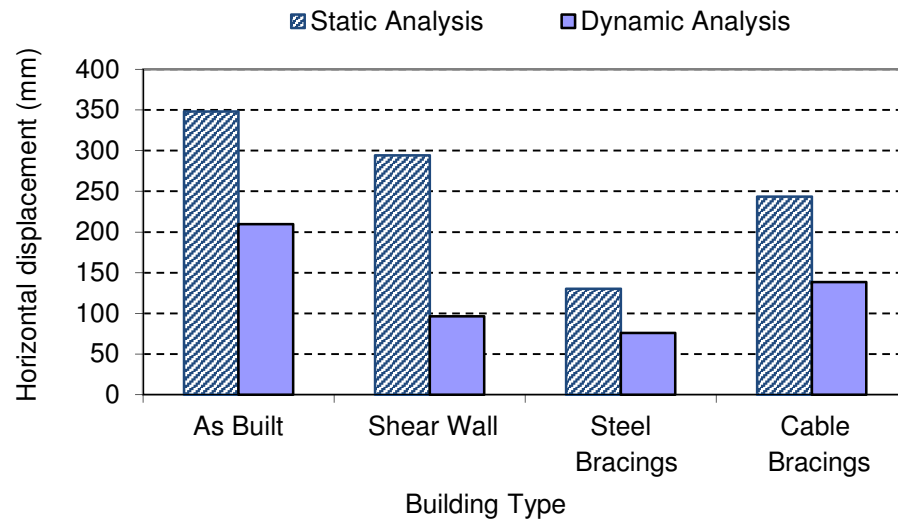


Figure 5.10: Effect of retrofitting on roof displacement demands:  
Static vs. dynamic analyses

## Chapter 6

### Experimental Research

#### 6.1 Introduction

The primary objective of the research program is to assess the hysteretic behaviour of seismically deficient reinforced concrete frame structures built prior to the enactment of seismic detailing provisions prescribed in recent building codes and to develop a new seismic retrofit system to improve their seismic performance effectively and economically. This objective is fulfilled through experimental research with analytical verifications. Two, 2/3<sup>rd</sup> scale test frames were identically designed and built for this purpose, based on the 1965 NBCC to represent an interior bay of the second storey or an exterior bay of the ground floor level of a six storey building located in Vancouver and Ottawa. [Note that there is no difference in design for Vancouver and Ottawa according to the 1965 NBCC.] The first served as a bare control frame that was first tested and then repaired and retrofitted to evaluate the effectiveness of the proposed retrofitting methodology for buildings subjected to earthquakes in the City of Ottawa. The second frame served as a companion non-damaged frame that was retrofitted with a similar retrofitting concept as the first frame but for buildings subjected to earthquakes in the City of Vancouver. The proposed retrofitting technology consisted of Buckling Restrained Braces (BRB). Two tests were performed to retrofit the repaired bare control frame using two types of BRB steel core bars: AISI 12L14 carbon and AISI Type 304 stainless steel; while AISI 4140 chrome-molybdenum high-tensile steel bar was used as the steel core bar to retrofit the second non-damaged frame.

A description of the experimental research program is presented in the following sections.

#### 6.2 Lab Specimen Design

The building investigated is a 6-storey reinforced concrete moment-resisting frame structure located in Ottawa and Vancouver representing a typical seismically deficient older building of medium-rise constructed during the 1960's and early 1970's as shown in Figure III.1 of Appendix III. The building is modified from a building described in the Concrete Design Handbook (Cement Association of Canada 2006). The modifications



include changes to the plan dimensions in the E-W direction and the interstorey heights. The interstorey height is 3.5 m with a first storey of 4.5 m. The floor plan of 21.7 m x 42m is divided into 3 bays in the short direction in which the two end bays are 8 m in width and the centre bay is 5.7 m, while in the long direction there are 7 bays of 6 m widths each. The slab thickness is 110 mm and the interior and exterior columns are 500 x 500 mm and 450 x 450 mm, respectively. The main and secondary beams are 400 mm wide and 600 mm deep and 300 mm wide and 350 mm deep, respectively. The building is proposed for office use occupancy.

The two test frames of this research represent 2/3<sup>rd</sup> scale dimensions of the 5.7 m-wide centre bay. All dimensions were scaled by 2/3 and, therefore, the applied loads were scaled by  $(2/3)^2$ . The frames were designed according to the 1965 NBCC load combinations for dead, live, wind, and earthquake. Snow loads were considered part of the dead loads as prescribed in the code. Detailed calculations and analysis results are presented in Appendix III.

The 1965 NBCC assigned the identical seismic zone factor of 4 and zone intensity of 3 for both Ottawa and Vancouver. The base shear force was calculated based on the assumption of non-ductile construction type C, equal to 1.25, and building of normal importance not designed for post disasters corresponding to an importance factor of 1.0. The foundation factor F was assumed to equal 1 for non-highly compressible soil conditions. Detailed calculations are presented in Appendix I.

The loading combinations for the 2<sup>nd</sup> storey interior frame (Grid lines 7B-7C) and for the 1<sup>st</sup> storey exterior frame (Grid lines 8B-8C) of Figure III.1 in Appendix III were simulated in the SAP2000 models for the scaled single storey, single bay lab test frame. The governing ultimate forces were then used to design the frames. The maximum moments in the columns were 111 kN.m, while the maximum axial and shear forces were 1340 kN and 64 kN, respectively. The negative and positive moments in the beam were 111 kN.m and 89 kN.m, respectively, and the maximum shear force was 120 kN.

### **6.3 Lab Specimen Description and Preparations**

Elevation, sectional and top views of the frame test specimens illustrating dimensions and reinforcement details are illustrated in Figures 6.1 and 6.2, while cross sectional details of

the columns and beams are shown in Figure 6.3. The frame specimens were 4250 mm in height and 5100 mm in length with 3000 mm clear height and 3500 mm clear spacing (length between columns). The centre-to-centre frame height and length were 3425 mm and 3800 mm, respectively. The columns were 300 mm square, and the beams were 300 mm wide and 350 mm deep. The frames were built on a rigid I-shaped foundation of 500 mm depth. The foundations at the location of the columns were 1500 mm wide, while the width of the foundation between the two columns was 500 mm. One of the frame foundations was re-used from a parallel research program that focused on the effect of diagonal prestressing cables to upgrade deficient moment-resisting frames (Molai 2014). That study adopted the identical frame design, dimensions, and reinforcement details of this research program.

Four plastic tubes were inserted in each foundation to create 75 mm-diameter voids. Steel anchor bolts were later threaded through the voids to secure the foundation to the laboratory strong floor. In addition, 12 high-strength threaded bars were embedded in the foundation on the two longitudinal sides of each column to accommodate prestressing cables, which were used to impose gravity loads on the columns. Furthermore, three square HSS (102 x 102 x 9.5 mm) were imbedded near the base of the foundation between the two columns. Prestressing cables were attached to the HSS and the top beam to simulate the imposed gravity load on the frame beam.

As stated in the 1965 edition of the NBCC, the code was designed to be in general agreement with ACI standard 318-63 except for revisions made by the Joint Canadian Standards Association/NBC Committee on Reinforced Concrete Design (NBCC 1965). The reinforcement details for the two frames are shown in Figures 6.1 through 6.3. The beam longitudinal reinforcement at the supports consisted of 4-20M and 2-15M bars at the top, which resulted in reinforcement ratio of 1.67%, and 2-20M bars at the bottom. The beam longitudinal reinforcement at mid span consisted of 2-20M and 2-15M bars at the bottom, representing 1.04% reinforcement ratio and 4-20M at the top. The 20M longitudinal bars were continuous throughout the length of the beam. The transverse shear reinforcement (stirrups) consisted of 10M bars spaced at 150 mm c/c along the length of the beam. There were no ties in the beam-column joints; they were not required according to the 1965 edition of the NBCC. The transverse ties (stirrups) in the columns were spaced at 200 mm c/c. The beam stirrups and column ties were closed by 90° hooks

with an extension of 64 mm (the greater of  $6 d_b$  or 2 ½ inches) as shown in Figure 6.3. The foundations were heavily reinforced to prevent any damage. Column starter bars were placed and cast within the foundation as illustrated in Figures 6.4 and 6.5. Figure 6.6 shows the foundation after casting. The column longitudinal bars were then spliced with the starter bars over a length equal to 24 times the bar diameter, resulting in 480 mm lap length above the foundation level as shown in Figures 6.7 and 6.8. Inadequate lap splices at probable plastic hinge locations of columns is not permitted according to current codes of practice. For ductile behaviour in moment-resisting frames, lap splices should be located away from the plastic hinge region. The beam reinforcement was prepared, assembled and instrumented with strain gauges before placing into the formwork and connecting to the columns. Tokyo Sokki FLA-10-11 strain gauges with a gauge length of 10 mm were used to record reinforcement strains during testing. Eight strain gauges were used in the beams; four at each beam end adjacent to the beam-column joint. The gauges were placed on the top and bottom corner reinforcing bars. Figure 6.9 provides the layout of strain gauges for each frame. The longitudinal reinforcement and starter bars near the base of each column were instrumented with 10 strain gauges. These gauges were distributed at the foundation level interface, and 125 mm and 250 mm above the foundation level as illustrated in Figure 6.10. In addition, 2 strain gauges were used at the top of each column adjacent to the beam-column joint; one on each middle longitudinal reinforcing bar.

Once the formwork for the frame, which was mainly built from 20 mm-thick plywood, was erected as shown in Figure 6.11, the beam reinforcement cage was placed into the formwork and connected to the column longitudinal bars as illustrated in Figure 6.12. The columns and beam were cast integrally. Each frame was cured for one week after casting followed by removal of the formwork. Figure 6.13 illustrates the constructed frames after the formwork removal.

The first frame tested was the Bare Control Frame (BCF). After repairing and retrofitting this frame, it was renamed (RRF). The second non-damaged, retrofitted frame was named (RF). The design force and ductility demands for Frame RRF are based on the requirements for the City of Ottawa, while the retrofit design for Frame RF is based on the required capacities of buildings for Vancouver.

## **6.4 Material Properties**

Materials used in the construction of the test frames and the BRB retrofitting components were assessed to establish their mechanical properties. The following sections provide details of the testing and mechanical properties.

### **6.4.1 Construction of Frames**

Mechanical properties of the materials used in the construction of the reinforced concrete test frames were established through a series of standard tests. Concrete cylinders and reinforcing steel coupons were tested to assess the compressive strength of concrete and the stress-strain relationships of the reinforcing steel and the prestressing strands, respectively.

#### **6.4.1.1 Concrete**

The two frame specimens were cast using ready-mix concrete with 10 mm maximum aggregate size and 120 mm slump; one batch was used for each frame. Standard concrete cylinders of 100 mm diameter and 200 mm height were cast on the same date as the frame specimens and tested according to the procedures prescribed in CSA A23.2-C as shown in Figure 6.14. These cylinders were cured under similar environmental conditions as the frame specimens. The average concrete compressive strengths at 28 days and on the day of testing of Frames BCF and RF, including the repaired concrete used for Frame BCF, are summarized in Table 6.1.

#### **6.4.1.2 Reinforcing Steel**

Standard tension coupon tests were conducted to establish mechanical properties of the deformed reinforcing bars used in the frame specimens. The reinforcement was ordered from a local supplier. Three random coupons were prepared for each reinforcing bar type used in the frames. They were tested with a 600 kN capacity Galdabini Universal Testing Machine (UTM). A 50 mm long extensometer was connected to each test bar to measure the strains during testing. Figure 6.15 shows one of the coupons after testing. The average yield strengths for 10M, 15M, and 20M reinforcing bars were 481 MPa, 450 MPa, and 430 MPa, respectively. Figure 6.16 provides typical stress-strain relationships for each type of reinforcing bar.

#### **6.4.1.3 Prestressing Strands**

Standard size Grade 15, 1860 MPa seven-wire strands were used to impose the gravity loads that were established and scaled from the prototype frame building. The nominal diameter of the strands was 15.24 mm with 140 mm<sup>2</sup> nominal cross-sectional area. Three strand coupons were tested under increasing tension loads to failure of at least one of the wires within the strands. The coupons were tested in the Galdabini UTM. Standard anchors with conical threaded wedges were connected at each end of the coupons to transfer the force from the wedge connections in the testing machine to the strands. The average yield and ultimate rupture strengths for the strands were 1650 MPa and 1857 MPa, respectively, as shown in Figure 6.17. Two of the coupons ruptured at one end of the anchor connections where high stress concentrations were the probable cause of failure.

#### **6.4.2 BRB Retrofit Components**

The BRB retrofit bracing components consist of a steel core and a restraining system that prevents the steel core from buckling. Round steel bars were used as the steel core; whereas the restraining system consisted of a circular HSS casing that was filled with mortar. The mechanical properties of these components were established through a series of standard tests, including coupon tensile tests and concrete compressive cylinders tests. These are discussed in the following sections.

##### **6.4.2.1 Steel Core Bars**

Coupons were tested to assess the average stress-strain relationships for each steel core bar used in the buckling restrained braces. Three bars were used, namely: AISI 12L14 carbon steel; AISI Type 304 stainless steel; and AISI 4140 chrome-molybdenum high tensile steel bars. Three coupons, with a 300 mm length and a 10 mm diameter, were drawn from each steel bar. The nominal cross-sectional area of these coupons was 78 mm<sup>2</sup>. The material specifications data of BRB steel core bars, as prescribed in product data sheet, are provided in Appendix IV.

The stress-strain relationship for the AISI 12L14 carbon steel bar is illustrated in Figure 6.18. The yield strength of 480 MPa was recorded at a corresponding strain of 0.24%, and an ultimate tensile strength of 535 MPa at a corresponding strain of 4.84%. The modulus of elasticity was 200.5 GPa.

AISI Type 304 stainless steel and AISI 4140 chrome-molybdenum high tensile steel bars exhibited larger ductility, as shown in Figures 6.19 and 6.20, respectively. The graphical 0.2% offset method was used to define the yield strength and corresponding strain for the AISI 4140 chrome-molybdenum high tensile steel bar.

AISI Type 304 stainless steel yielded at 303 MPa with a corresponding strain of 0.16%. The ultimate tensile strength was 601 MPa at a corresponding strain of 45%. The modulus of elasticity was 199.2 GPa. The AISI 4140 chrome-molybdenum high tensile steel bar yielded at 390 MPa corresponding strain of 0.38%. The ultimate tensile strength was 740 MPa corresponding to a strain of 10.8%. The modulus of elasticity was 190.5 GPa.

#### **6.4.2.2 HSS Round Casing**

The BRB system includes an outer steel round casing that consists of an HSS 168 x 8 mm that serves to provide outer confinement for the steel core bar of the BRB system. Three rectangular coupons of 500 mm lengths were machined from the HSS section. The cross-sectional dimensions of each coupon were 19.6 mm x 6.2 mm with a 121.5 mm<sup>2</sup> nominal cross-sectional area. Figure 6.21 illustrates a typical tensile stress-strain relationship of these coupons. The average yield strength was 390 MPa corresponding to a strain of 0.17% while the ultimate tensile strength was 425 MPa corresponding to a strain of 11%. The modulus of elasticity was 225.8 GPa.

#### **6.4.2.3 Filling Mortar**

Sikacrete-08 SCC (Self-Consolidating Concrete), a product of Sika Canada Inc., was used as a filler material between the steel circular round surrounding the steel core bars and the outer HSS casing. The material is characterized as a high flow cement-based concrete and is used for concrete thickness between 25 and 450 mm. Standard concrete cylinders were cast and tested to assess the average concrete compressive strengths at 28 days and on the average day of the three retrofitted frame tests. The resulting strengths were 57.7 MPa and 67.4 MPa, respectively.

### **6.5 Demands and Capacities of Test Frame Members**

Comprehensive analyses were conducted on the individual members of the bare control, BCF, and retrofitted RRF/RF frames. The analyses included the sectional properties of the frame members, reinforcing bar splices and development lengths at the base of the

columns, imposed force demands, moment and axial load interaction, calculation of shear capacities according to the General Method of the CSA A23.4-04, and calculations of elastic deformations of the BRB steel core bars. Appendix V contains detailed calculations.

Cracked section properties were assigned to the concrete frame members according to Table 21.1 of CSA A23.3-04. Factors of 0.40, and 0.68 were applied to the moment of inertia to simulate cracked properties of the beam and columns, respectively. The yield strength ( $f_y$ ) of column longitudinal steel bars at the foundation level was factored by 0.73, to take into account the ratio of the splice length required by the 1965 NBCC, which was used in designing the frame, and the splice length required by the 2010 NBCC. Calculations of the cracked section properties and yield strengths of the column starter bars are provided in V.1 and V.2 of Appendix V, respectively.

Force demands for each member of the frames were established to predict the sequence of failure of the frame based on the calculated strength capacities. Axial, moment and shear force demands were determined from the gravity loading and a unit lateral load based on an elastic analysis using SAP2000 software. The unit lateral load represented the application of the lateral load in the lab. The analytical models of frames corresponded to the centre-to-centre dimensions of the lab frame specimens.

Figures V.2 to V.6 of Appendix V provide the member forces determined from the elastic analysis due to the gravity and unit lateral loads, separately, for both the bare control frame and the retrofitted frames. The force demands imposed on the frame members were established as a function of the applied horizontal load ( $H$ ), which represented the increasing lateral load applied during testing. These load demands are summarized in Tables 6.2 and 6.3, respectively for the bare control frame and retrofitted frames. The “Near” refers to the column and beam end adjacent to the actuator through which the horizontal load was applied, while the “Far” refers to column or beam end away from the actuator.

Computer program Response 2000 (Bentz 2001) was used to assess the moment and axial load sectional capacities of the frame members. Interaction diagrams for the columns and beam of Frame BCF are illustrated in Figures 6.22 and 6.23, respectively; while

Figures 6.24 and 6.25 illustrate the interaction diagrams of the columns and beams of Frames RRF/RF, respectively.

The sectional capacities of the columns were calculated at the top, adjacent to the beam-column joint, and at the base. The moment and axial force demands on the columns due to the gravity and lateral loading were superimposed on the moment-axial load interaction diagrams of the columns. The intersection between the loading lines (force demands) and the moment-axial load interaction diagrams provide a probable lateral failure load. Similarly, interaction diagrams were established for the beams. Two beam sections were analyzed: at the supports and the mid span. The loading lines (moment and axial force demands) were drawn on the interaction diagrams. Note that although beams are typically understood to carry negligible axial loading, the test specimens were loaded at the ends by hydraulic jacks, which imposed axial loading into the beams, especially when BRB was used.

Collins and Mitchell (1987) and Collins et al. (1996) developed and introduced the General Method for shear which was based on compression field theory developed by Vecchio and Collins (1986) and depends on the state of stress and strain in the member (Rahal and Collins, 1999 and Brzev and Pao 2009). According to the commentary of CSA A23.3-04, this method is applicable to members that are subjected to significant axial tension forces that either increases the tension induced stresses of the longitudinal tension reinforcement at crack locations of more than 50 MPa or is large enough to crack the flexural compression face of the member (CSA A23.3-04 – N11.3.6.1). Although this was not the case for the columns and beam members of the frames under investigation due to the compressive forces from external loading, the General Method was, nonetheless used in the analysis rather than the simplified method. The General Method provides more accurate estimate of the sectional shear capacities. Section V.4 of Appendix V provides detailed calculations of the shear capacities for the bare control and retrofitted frames.

Based on the elastic analysis described above and the sectional capacities of the bare control and retrofitted frames, the predicted sequence of damage due to the lateral loading was established and summarized in Table 6.4 and Table 6.5, respectively. For the bare control frame, damage was predicted to initiate at the base of the columns due to the combined effects of axial loading and flexure, then subsequently in the beam near the



beam-column joint due to the same combined effects. Shear failure then followed in the ensuing order: at the base of the columns, at the top of the columns, and at the ends of the beam at a distance  $d_v$  from the faces of the supports. The force demand imposed on the BRB was substantially higher than the forces experienced by the frame members. This is attributed to the truss action that took place in the retrofitted frames. The analytically computed force demands are summarized in Table 6.3. Accordingly, the fracture of the brace in tension was predicted to initiate prior to damaging the frame columns. This would exceed the capacity of the hydraulic actuator used during testing. Following fracture of the brace bar, the lateral load capacity of the retrofitted frame would be controlled by the capacity of the frame alone.

### **6.6 Repair of the Bare Control Frame**

The bare control frame was repaired after testing so that it could be retrofitted with the proposed BRB system. At the maximum drift of 4%, the longitudinal and transverse tie reinforcing bars were visible at the base of the column and top of the columns adjacent to the beam-column joint due to excessive spalling of concrete cover. In addition, the longitudinal reinforcing bars in the beam near the beam-column joint experienced permanent buckling deformations. A number of cracks of less than 25 mm depth were visible throughout the frame. The first step in repairing the bare control frame was to remove any loose concrete with the aid of a pneumatic chisel gun and to prepare all longitudinal and transverse steel bars for the new repair concrete. Reinforcing bars that experienced some bending were straightened, while bars that buckled were cut over a distance of 100 mm on either side of the buckled length and replaced with new bars. The new bar segments were connected to the original remaining bars with screw-lock steel couplers. Two couplers were used for the bottom 20M longitudinal reinforcement at one end of the beam. Figure 6.26 illustrates the various steps taken to repair the bare control frame. Visible strain gauges that were damaged during the bare control frame test were replaced with new gauges. Figure 6.27 displays the bare control frame at the end of the repair preparation phase; while Figure 6.28 illustrates placement of frame formwork prior to casting the concrete for the repair work.

Self-Compacting Concrete (SCC) based on Sika Viscocrete Technology was used to replace the removed concrete. As prescribed in product data sheet, it is a ready-to-use high flow cement-based concrete applicable for concrete thickness between 25 and 450

mm and has a compressive strength of 45 MPa at 28 days (The actual compressive strengths attained at 28 days and on the day of testing are summarized in Table 6.1). Figure 6.29 shows the SCC used with other Sika products that are subsequently discussed.

Figure 6.30 shows the frame after casting the self-compacting concrete at four locations in the frame: at the base of the columns and at the ends of the beam near the beam-column joints. Curing of the SSC commenced immediately after placing and finishing. These areas were covered with wet burlap for 14 days to cure the new concrete. Figure 6.31 shows the frame after curing.

Visible cracks of less than 25 mm in depth into the concrete were first widened and prepared prior to the application of a bonding agent. Thereafter, a cementitious mortar was applied to close the cracks. SikaTop Armatec 110 EpoCem, a three-component water-based epoxy resin (left side of Figure 6.29), was used as the bonding agent to bond the new mortar to existing concrete. Figure 6.32 shows the application of the bonding agent on one of the columns near the base. SikaTop 123 plus mortar, a polymer modified cementitious mortar (as shown in Figure 6.29), was then used as the patching repair material. It was applied with a trowel over the bonding agent within the applicable contact time of 8 hrs (for 30 C°). The mortar had a compressive strength of 28 MPa at 50 days. The curing, as prescribed by the manufacturer, started immediately after placing and was maintained for the first 24 hours only. Figure 6.33 shows the application of the SikaTop 123 plus mortar at the columns and areas adjacent to the beam-column joints. The frame at the end of the repairing process is illustrated in Figure 6.34 prior to painting.

## **6.7 Buckling Restrained Brace Retrofitting**

The new retrofit technology involved the implementation of buckling restrained braces in two frames: a bare control frame that was tested, repaired, and retrofitted to evaluate the effectiveness of the retrofitting methodology for buildings subjected to earthquakes in Ottawa; while the second frame served as a non-damaged frame building that was retrofitted and tested with similar retrofitting concepts but based on higher earthquake demands to represent buildings located in Vancouver.

The lateral base shear design force for the non-ductile, six-storey RC moment-resisting frame building (Figure AIII.1 of Appendix III) according to the 1965 and 2010 editions of NBCC was investigated. The study revealed that the strength capacity of buildings constructed in Ottawa and Vancouver based on NBCC 1965 should be upgraded by a factor 1.36 and 2.95 respectively, to satisfy the strength requirements of recent codes. These increased factors were based on a ductility force-modification factor ( $R_d$ ) of 1.5 and the over strength force-modification factor ( $R_o$ ) of 1.3 as prescribed in NBCC 2010 for conventional and un-retrofitted MRF concrete buildings. However, when the conventional building is retrofitted with a system that provides some ductility, such as the buckling restrained braces used in this study, the  $R_d$  and  $R_o$  factors should be revised. There is no such category in the NBCC 2010 and, thus,  $R_d$  of 2.0 and  $R_o$  of 1.4 were selected based on engineering judgement and by examining the other categories described in the code. The selected modification factors are similar to those assigned for moderately ductile concrete shear walls and close to those assigned for limited ductility moment-resisting steel frames ( $R_d$  of 2.0 and  $R_o$  of 1.3). Bracing conventional moment-resisting frame buildings leads to stiffening of the structure and, therefore, shortening of the building fundamental period. Empirical equations for the fundamental lateral period prescribed in the 2010 NBCC for braced structures results in a period of 0.55 s compared to 0.76 s for moment-resisting frames. With the former building period and modified  $R_d$  and  $R_o$  factors, the strength of buildings for Ottawa and Vancouver should be upgraded by a factor 1.25 and 2.6, respectively. This is based on the change in building category from moment-resisting frame to braced frame. Care should be exercised in retrofitting non-ductile buildings, as these buildings should go for the ride during the earthquake and experience deformations as the Seismic Force-Resisting System (SFRS) provide resistance to earthquake forces.

The design of the BRB components, displacements sustained by the steel core bar, and the description of the BRB system employed are discussed in detail in the following sections.

#### **6.7.1 Design of BRB system**

The BRB system consists of a core steel bar, restraining HSS casing, and steel hinge joints. In this research, three different steel core bars were investigated through three individual frame tests. These bars were selected to satisfy the seismic provisions of the

2010 NBCC and included: AISI 12L14 carbon steel, AISI Type 304 stainless steel, and AISI 4140 chrome-molybdenum high tensile steel. The first two bar types were used to retrofit the repaired frame BCF representing retrofitting for buildings in the City of Ottawa, while the third bar type was used to retrofit the non-damaged frame representing retrofitted buildings in Vancouver.

An analytical model of the retrofitted frame specimen was analyzed with SAP2000 to assess the design forces for the BRB steel core bars (Section V.3.2 of Appendix V). The design was based on S16-09 Limit Steel Design of Steel Structures (CSA 2009). The steel core bars were protected against buckling in compression, therefore the tensile and compressive strengths are based on similar sectional capacities. However, the bar is expected to rupture in tension. The cross sectional areas ( $A$ ) of the steel core bars was determined from the following:

$$[6.1] \quad T_r = \Phi A F_y$$

Where  $T_r$  is the tensile resistance of the brace member,  $\Phi$  is the resistance factor equal to 0.9,  $F_y$  is the specified minimum yield strength. For the test frames, the  $\Phi$  factor was selected to be 1.0.

Based on the results from the frame analysis (Figures V.4 and V.5 of Appendix V), the total axial load sustained by the core steel bar is equal to the sum of forces due to the gravity loads and the unit lateral load. The gravity loads impose a compressive force of 6.6 kN, while the unit lateral load results in a tensile force of 1.24 kN. Therefore, the brace bar was designed for an axial tension force equal to 1.24 times the target NBCC required horizontal force less the compressive axial force of 6.6 kN. Figure V.6 (Appendix V) represents the analysis results of the retrofitted frame in the pull direction (brace in compression). This analysis was not vital in assessing the sequence of failure in the frame since the brace was restrained from buckling. Essentially the strength capacity of the brace in tension and compression were equal.

The three steel core bars were 2700 mm in length, including 220 mm of threaded sections at the ends to accommodate the connection to the joint steel plates at either side of the brace. For the three bars, the original diameter size was 1  $\frac{3}{4}$ " (44.5 mm), while the

diameter of the threaded sections was 38.3 mm. The first bar, AISI 12L14 carbon steel, was implemented in its virgin condition with the threaded ends, while the other two bars, AISI Type 304 stainless steel and AISI 4140 chrome-molybdenum high tensile steel, had reduced sectional area around the mid-length to promote yielding of the bar at this location. Figure 6.35 illustrates details of the three different steel core bars used in the construction of the BRBs.

Based on a numerical study, 617 kN of axial tension force is required to rupture the AISI 12L14 carbon steel brace bar. The corresponding lateral load imposed on the frame to initiate failure of the brace is 503 kN. This represents an over-strength factor of 2.2 relative to the capacity of the bare control frame. An axial tension force of 477 kN is required to fracture the AISI Type 304 stainless steel brace bar at the reduced section, corresponding to a lateral load of 390 kN and over strength factor of 1.67 relative to the capacity of the bare control frame. The second un-damaged frame was tested with AISI 4140 chrome-molybdenum high tensile steel brace bar. An axial tension force of 588 kN is required to fracture the brace, which corresponds to a lateral load of 479 kN and over strength factor of 2.1 relative to the capacity of the bare control frame.

The steel casing should be designed for sufficient flexural stiffness that exceeds the yield strength of the restrained yielding core segment. Watanabe et al. (1988) suggested a strength ratio of 1.5 to avoid elastic global buckling due to initial geometric imperfections. The steel casing was designed according to S16-09 (Section 13.2 and 13.3; CSA 2009) and an HSS 168 x 8 mm was selected. The section has an area of 4000 mm<sup>2</sup>, radius of gyration of 56.8 mm, and moment of inertia of 12.9 x 10<sup>6</sup> mm<sup>4</sup>. The axial tension resistance is based on Equation 6.1 with  $\Phi$  factor of 1.0, while the axial compression resistance is calculated as follows:

$$[6.2] \quad C_r = \Phi A F_y (1 + \lambda^{2n})^{-1/n}$$

Where  $n$  is a parameter equal to 1.34 for hot rolled sections, while  $\lambda$  is a non-dimensional slenderness parameter equal to:

$$[6.3] \quad \lambda = (KL/r) (f_y/\pi^2 E)^{1/2}$$

The tensile force of the HSS section is 1560 kN, while the compressive flexural buckling resistance is 1393 kN. This provides a tensile axial strength increase factor of 2.8, 6.5, and 5 relative to the yield strength for the 3 core steel bars used for retrofitting (i.e. AISI 12L14 carbon steel, AISI Type 304 stainless steel, and AISI 4140 chrome-molybdenum high tensile steel, respectively). Similarly, the compressive flexural buckling resistance of the casing to the restrained core bar yield strength is 2.5, 5.8, and 4.5, respectively for the same core bars.

CSA S16-09 standard was used to design the steel joints of the BRB system to sustain the forces that the BRB steel core bars were intended to experience. The joints consisted of steel plates, bolts, HSS sections, and connecting bars. The steel plates are components of the joint that contained a through hole to simulate a hinged connection to which the brace was attached (shown in Figures 6.48 to 6.51). These plates were checked against axial tensile forces, shear block failure, and load bearing; while the bolts connecting these hinge plates to the BRB were checked for shear strength capacity. HSS sections and steel plates used to connect these HSS sections within the test frames were checked for bending strength capacity and maximum width-to-thickness ratios. The steel bars connecting the HSS sections and the steel plates were checked for axial tensile strengths. Finally, groove welding used to connect and assemble the components above was checked for tension, compression, and shear resistances.

### 6.7.2 Displacement of BRB Steel Core Bar

Total displacement of the BRB steel core bar sustained during testing consisted of elastic and plastic deformations. The elastic deformations,  $\Delta L_e$ , of a bar with a constant cross sectional area along its length, such as the AISI 12L14 carbon steel bar used in the first retrofitted frame test, is calculated as follows:

$$[6.4] \quad \Delta L_e = NL_t / AE = \sigma L_t / E$$

Where  $N$  is the axial force in the brace bar,  $L_t$  is the length of the brace bar,  $A$  is the cross sectional area of the brace bar, and  $E$  is the Modulus of Elasticity. The strain experienced by the bar is the ratio of the change in length ( $\Delta L$ ) to the original length of the brace bar ( $L$ ). The axial stress ( $\sigma$ ) in the bar is determined from the axial force in the bar ( $N$ ) divided

by the cross sectional area ( $A$ ) of the bar. The axial stress in the bar is limited to the steel yield strength,  $f_y$ , in evaluating the maximum elastic deformations.

Conversely, when the brace bar is detailed with different sectional areas along the length, such as the AISI Type 304 stainless and AISI 4140 chrome-molybdenum bar types, the elastic deformation,  $\Delta L_e$ , is proportional to the axial stresses and corresponding lengths of the reduced and original areas. For this case, the elastic deformation is calculated as follows:

$$[6.5] \quad \Delta L_e = (f_y L_r / E) + (f_y L_t / E)(\sigma_{unr} / \sigma_r) - (f_y L_t / E)(\sigma_{unr} / \sigma_r)(L_r / L_t)$$

Where  $L_r$  is the length of the reduced area of the bar,  $L_t$  is the total length of the brace bar between the two securing nuts (including the reduced and un-reduced lengths),  $\sigma_{unr}$  is the average axial stress in the un-reduced area, and  $\sigma_r$  is the average axial stress in the reduced area. The first term is the elastic deformation of the reduced bar section, while the summation of the second and third terms represent the elastic deformation of the un-reduced section of the bar. Equation 6.5 could be simplified as:

$$[6.6] \quad \Delta L_e = (f_y L_t / E) [\lambda + \eta (1 - \lambda)]$$

Where  $\lambda$  is the ratio of the length of the reduced bar area to the length of the un-reduced bar area while  $\eta$  is the ratio of the average axial stress in the un-reduced bar area to the stress of the reduced bar area. Detailed calculations of elastic deformations of the BRB bars are presented in Section V.5 of Appendix V. The total elastic deformations calculated for the three steel core bars (AISI 12L14 carbon steel, AISI Type 304 stainless steel, and AISI 4140 chrome-molybdenum high tensile steel) were 5.8 mm, 2.9 mm, and 3.9 mm respectively.

Using geometry of the displaced frame along with the data recorded from instrumentation, the total (elastic and plastic) deformations of the BRB steel core bar were estimated while the BRB was subjected to tensile loading as illustrated in Figure 6.36. The lateral drift of the frame is evaluated from the horizontal displacement of the frame ( $\delta$ ) to the column height ( $H$ ), which is measured from the base of the column to the centre line of the beam. The change in length of the steel core bar ( $\Delta L$ ) is, therefore, calculated as follows:

$$[6.7] \quad \Delta L = \delta \cos \Theta$$

Where  $\Theta$  is the angle of inclination of the BRB relative to the horizontal. Table 6.6 provides elongation of the BRB steel core bar relative to frame lateral displacement,  $\delta$ .

### 6.7.3 BRB System Description

The proposed retrofit methodology consists of a single diagonal buckling restrained brace that utilizes the full compression and tension strength capacities. The brace is connected to steel joints surrounding the beam and columns near the frame joints. Yielding in tension and compression provides a system with approximately the same lateral strength, stiffness and energy dissipation capacities during reverse cycling loading.

The steel core bar is connected to the frame only through the end threaded sections of 44.5 mm diameter buckling restrained steel core bar. It is secured at each end to a steel plate that acts as a hinge when connected to the frame, as illustrated in Figure 6.37. The steel brace consists of a core segment (1565 mm in length) and steel core restraining end elements (430 mm in length), allowing relative movements between parts of the end elements. The total brace length of 2439 mm incorporates these two components (2425 mm) plus thicknesses of the two plate hinges (57 mm each at either side). The lengths of the brace components were based on the geometry of the frame at 3% lateral drift. At this drift, the predicted elongation of the steel bar (Table 6.6) is 70.5 mm, corresponding to a global bar strain of 2.9%; approximately 14.5 times the bar yielding strain of 0.2%.

Details of the BRB system configuration using AISI 12L14 carbon steel core bar is shown in Figure 6.38. The steel bar cross section consists of four 6.35 mm diameter spacers welded to the 44.5 mm diameter solid steel bar along the core length to accommodate lead wires of 12 strain gauges placed on the core bar; 4 at the mid length and 4 at each end. The bar is inserted into a steel pipe of 59 mm inner diameter, which is encased by mortar and HSS casing in the core segment (1565 mm in length). At mid length of the HSS casing, 8 strain gauges were placed; 4 in the longitudinal direction and 4 in the circumferential direction. Figure 6.39 a) and d) illustrate the locations of the strain gauges on the steel core bar, while Figure 6.39 c) and e) provide the locations on the HSS casing. Sikacrete-08 SCC (self-consolidating concrete), a product of Sika Canada Inc., was used



as a filler material between the outer HSS casing and the inner 59 mm diameter steel pipe of the core segment as shown in Figure 6.40.

The BRB system configuration for the AISI Type 304 stainless steel and AISI 4140 chrome-molybdenum high tensile steel core bars (Figure 6.41 and Figure 6.42, respectively) were similar to the earlier system except that the area of these two core bars was reduced around the mid length. In addition, four 12.7 mm diameter spacer bars were required due to the reduced area rather than the 6.35 mm diameter spaces. The spacers were welded in the reduced area section. The main function was to accommodate the lead wires of a similar number and arrangement of strain gauges placed on the reduced area as shown in Figure 6.39 (b) and (d). To prevent any contribution of these spacers to the axial load carrying capacity of the brace, the spacers were welded only at the middle and were free to slide in steel sleeves that were welded to the ends of the reduced areas of the core bars as illustrated in Figures 6.41 and 6.42. Finally, to avoid any misalignment of these spacers during testing, transparent adhesive tape was wrapped around the AISI 4140 chrome-molybdenum high tensile steel bar and spacers followed by placing a mix of epoxy and sand in the gaps between these spacers as shown in Figure 6.42 (d) and (e).

The 0.9 mm clearance between each spacer bar and the 59 mm diameter steel casing for the three BRB core bars permits free lateral expansion of the steel core bar under compressive stresses, and minimizes/eliminates the transfer of axial force between the core steel bar and the surrounding mortar and steel casing. The clearance was based on an assumed maximum longitudinal strain of the steel core bar and Poisson's ratios of 0.3 and 0.5 in the elastic and inelastic ranges, respectively, according to the material properties of the steel bar. Therefore, the total transverse strain of the steel core bar would be equal to the sum of the products of the longitudinal strains and the Poisson's ratios in the elastic and inelastic ranges. At 3% drift, the maximum transverse strain in the core bars is calculated as  $0.3\varepsilon_y + 0.5 \times 14.5\varepsilon_y$ , resulting in a total transverse strain of 1.51% and corresponding to a lateral expansion of 0.7 mm. However, the 0.9 mm gap was selected to account for larger strains in the reduced area section of the core bars and to accommodate the standard size of the 59 mm diameter steel pipe.

The steel core restraining end elements provide a transition mechanism between the brace and the supporting frame members. It typically provides an adequate gap between

the brace ends and the frame joints to allow interstorey deformations without interference from the brace. Researchers have reported local buckling failure at these regions and, therefore, these regions are critical design areas. The steel core restraining end elements of this research are described in Figure 6.43 and consist of a solid steel circular section with 150 mm and 46.5 mm outer and inner diameters, respectively. The 44.5 mm diameter core bar penetrates through these steel ends. The core bars are threaded at the ends and then bolted to the joint plates at either side of the brace. The internal gap of 60 mm near the ends of the brace are designed to accommodate 70.5 mm (35.3 mm at each brace end) of compressive axial deformations that was calculated for 3% drift, but still provide full restraining for the steel core yielding bar along its entire length and within these end elements. This full restraint prevents end buckling failures that have been reported by other researchers. These steel core restraining end elements are inserted inside a similar size HSS of 370 mm in length that is welded to the 1565 mm long HSS casing (Figure 6.39 c)). This assembly is executed at both ends of the brace, resulting in an external longitudinal gap length of 60 mm between the end of the HSS and the steel plates. This prevents bearing between the HSS and the steel plates.

The steel core restraining end elements are further illustrated with two sections as illustrated in Figure 6.44, detailing the restraining mechanism when the steel core bar experiences tension and compression during the cyclic testing. Each end element has two components (Part 1 and 2) consisting of a cored end section that permits relative movement between the two parts. The end section of Part 2 is 60 mm thick and is placed inside the HSS casing. It is secured to the HSS casing near the steel core with four perimeter screws. The end section of Part 1 is 170 mm thick and is connected to the 57 mm thick joint steel plate by four perimeter screws as shown in Figure 6.43. As illustrated in Figure 6.44, Part 1 and 2 each contain three, 140 mm long wedge segments (sum of 60 mm and 80 mm), which extend from the cored end sections and are distributed around the perimeter. These segments are offset between Part 1 and 2, such that they can slide between each other to form a closed hollow cored section. The distance between the cored end sections of Part 1 and 2 is 200 mm. This section has been divided into three transition lengths: 60 mm at each end to permit axial deformations when the core is subjected to compression; and 80 mm interlocking zone that prevents the two parts from separating when the steel core bar is in tension. Before the brace is loaded, the core bar is restrained by the three solid wedge segments that extend from the cored end section of

Part 1 and 2 over the 60 mm distance from the cored end sections. Within the 80 mm distance region, the steel core bar is restrained by interlocking of the three wedge segments from Part 1 and 2. Therefore, based on the mechanism devised in this research program, the yielding steel core bar is fully restrained against buckling over its entire length, from joint to joint. This is believed to be an original contribution to the practice of the Buckling Restrained Braces (BRBs) and is intended to eliminate local buckling deficiencies reported in the literature. The 5 mm gap between the interlocking wedge segments of Part 1 and 2, and the 1 mm gap around the perimeter between these parts and the HSS casing are greased to minimize friction and permit free translational movement during testing. The fabrication process of the interlocking steel wedges of Part 1 and 2 is illustrated in Figure 6.45. As previously stated, a core is first drilled into a solid steel cylinder, after which it is segmented into two parts (Part 1 and 2), each containing a cored end and three wedge segments. Figure 6.46 shows the brace end elements after fabrication, while Figure 6.47 shows these brace restraining end elements after they are placed within the end HSS section that were welded to the main HSS casing.

The steel hinge connecting the BRB to the concrete frame were fabricated at the University of Ottawa and consisted of two parts (a and b) as shown in Figure 6.48. A single 44.5 mm diameter high strength bolt was used to attach the two components of the hinge together to form a pinned-end connection. The purpose of this pin connection was to prevent secondary moments at the ends of the buckling restrained brace bar. The first part of the connection (a) consisted of a thick steel plate (406.4 x 406.4 x 57.2 mm) that secured the steel core yielding bar at both ends. Two plates of 25.4 mm thickness were welded to the 57.2 mm end plate. The plates were fabricated with a bolt hole of 44.5 mm diameter to connect to Part b of the connection assembly, which in turn was connected to the concrete frame. This bolt which connected the two parts (a and b) provided the hinging mechanism. A rectangular cut out of 127 x 254 mm was provided in the two steel plates for the purpose of providing sufficient space to secure the nuts of the steel core yielding bar. Part b of the pin connection consisted of a 50.8 mm thick steel plate which contained a 44.5 mm diameter bolt hole to connect with Part a. The plate was welded to an end plate (254 x 254 x 38.1 mm). An elevation view of the two parts of the pin connection joined by the 44.5 mm diameter bolt is illustrated in Figure 6.48 (c). Part b of the pin connection assembly was welded to an HSS 254 x 254 x 13 mm at either ends of the brace inside the frame. To accommodate this, “steel chairs” as illustrated in Figure 6.49 (a) were used to

transfer the axial forces from the 38.1 mm thick end plates of Part b of the connection assembly (Figure 6.48 (b)) to the HSS 254 x 254 x 13 mm. These steel chairs were welded to the end plates of Part b and the HSS. Figure 6.50 shows the two manufactured brace hinge joint parts along with the HSS section that Part b of the connection was welded to.

The proposed retrofit configuration is illustrated in Figure 6.51. The brace is placed diagonally at an angle of  $41^\circ$  degrees with horizontal at the foundation level. The steel brace was assembled first and then connected to Part a of the brace hinge joint (Figure 6.50 (a)) by securing nuts at the threaded ends of the steel core yielding bar, as shown in Figure 6.37. In addition, the steel core restraining end element of Part 1 was connected to the 57 mm thick joint steel plate by the four perimeter bolts as shown in Figure 6.52 (a and b). Two springs were fastened on opposites sides of the HSS at the end of the lower joint (Figure 6.52 (a)) to maintain the 60 mm external gap between the HSS casing and the 57 mm thick joint steel plate (Figure 6.43). The springs prevented the HSS casing from sliding. The brace, thereafter, was ready for installation as shown in Figure 6.52 (c). The brace was then inserted into the concrete frame and secured by a bolt to Part b of the joint connection. As previously described, Part b was welded to a steel bearing assembly that, in turn, was connected to the joints of the frame. Two additional HSS 254 x 254 x 13 mm were placed on the outer faces of the frame: one at the top corner of the beam-column joint and the other at the intersection of the column and foundation. Two, 38.1 mm-diameter Dywidag bars ran along each side of the frame at top and lower corners and were used to secure the inner and outer HSS to the frame as shown in Figure 6.53. Holes were drilled into the inner and outer HSS to accommodate the Dywidag bars. At the lower joint, two additional Dywidag bars were placed vertically and anchored the HSS placed within the frame to the rigid foundation. Furthermore, the outer HSS was fixed to the foundation with two anchor bolts that were anchored to the lab strong floor (Figure 6.53 (a)). At the upper corner of the beam-column joint, the Dywidag bars were positioned at an angle of  $49^\circ$  to accommodate the geometry of the frame and brace connection assembly. Steel chairs as illustrated in Figure 6.49 (b) were used to secure the ends of the Dywidag bars.

Stiffeners were also added in the joint connection of the inner HSS to the concrete frame. This was necessitated due to observed relative rotations at the joints at higher drift demands during the first cyclic test on the Frame RRF that incorporated the AISI 12L14

carbon steel core bar. At the lower brace joint, two 25.4 mm threaded high tensile bars were placed on the top of the inner and outer HSS and were secured inside 100 mm steel round sleeves of 25.4 mm inner diameter as shown in Figures 6.51 and 6.54 a) and b). At the upper brace joint, one-25.4 mm thick steel plate was placed on the other face of the column and another was placed on top of the beam. These plates were secured to the inner HSS of the brace joint through similar mechanism used at the lower brace joint as shown in Figures 6.51 and 6.54 c). Figure 6.55 shows the retrofitted frame after the BRB was connected to the frame.

### **6.8 Novelty of the New BRB System**

The research presented herein proposes a structurally sound, new, and economically feasible, less intrusive and innovative seismic retrofit technique that is intended to upgrade existing deficient reinforced concrete frame structures to the state-of-the-art practice. In addition, the BRB can be used as bracing for lateral strength and stiffness in new construction. This novel retrofit method was verified in this research through three large-scale experimental tests that employed three different steel materials for the core bar. Complementary dynamic time history analyses were also conducted. The proposed retrofit method has been patented by the United States Patent and Trademark Office. The following sections provide further descriptions of the novelty of the system, including: 1) buckling characteristics; 2) brace end secondary moments; and 3) system economic feasibility.

- 1) Based on the mechanism devised in this research program, the yielding core steel bar is fully restrained against buckling over its entire length, from joint to joint. This is believed to be an original contribution to the practice of the Buckling Restrained Braces (BRBs) and to be the first BRB system mechanism that employs a continuously restrained core. Additionally, the circular cross sectional bar shape offers greater moment of inertia ( $I$ ), therefore offering more resistance to buckling than the commonly used rectangular cross sectional plate of similar area. In comparison, for similar sectional areas (reduced and un-reduced) that were used for the brace bars in retrofitting the frames, an equivalent steel brace plate of 115 mm width ( $\approx 75\%$  of the HSS casing inner diameter) would result in a moment of inertia ratio of the bar to the plate of 8.2 and 15.3 for the un-reduced and reduced areas, respectively.

- 2) The hinge connections at the ends of the BRB were designed to eliminate any non-axial loads, and secondary moments.
- 3) As for economic feasibility of the proposed BRB, the system can be assembled locally on construction sites from materials readily available. This reduces costs for design, development, materials, transportation, installation, and quality control. Furthermore, the system allows BRB brace bar installation tolerance through adjustments of the nuts securing the brace bar to joint steel plates. Finally, after major earthquakes, damage will mostly likely be confined to the core bar of the BRB. The BRB system has been detailed such that the steel core yielding bar can be replaced while the other components remain intact. This will also reduce the post-earthquake maintenance. In this research, the same BRB system was used for three tests while only changing the steel core yielding bar.

## 6.9 Test Setup

The test setup was the same for all frames. The frames were secured to the laboratory strong floor by means of four high tensile strength anchor bolts to provide fixity at the foundation level as shown in Figure 6.56. Seven-wire prestressing strands (size 15), with  $140 \text{ mm}^2$  cross sectional area, were used to impose the gravity loading on the columns and the beam. Strain gauges were bonded to each cable (prestressing strands) and a hand-held hydraulic jack was then used to apply post-tensioning stresses to the strands. The strain gauges were used to track the strains that corresponded to the gravity loading required on the columns and beam. The relation between the strains and loading was based on the stress-strain response of the strands recorded from coupon tests (Figure 6.17). Six strands were used for each column to apply 800 kN of compression loading per column, corresponding to 25% of the column axial compressive resistance; while four strands were used on the beam, each providing 31 kN of loading to simulate the scaled gravity load of 124 kN calculated for the prototype frame building. At each column, the cables were tensioned between steel supports placed on the top of the column and to high tensile threaded bars at the bottom of the column that were embedded in the foundations before casting. Each steel support at the top of the column was built from three HSS (102 x 102 x 9.5 mm) welded together with a steel plate. The cables were secured at both ends via conical wedges that were secured by barrel-shaped anchors. The cables used to load the beam were tensioned with similar mechanism. The only differences were that the strands were anchored to a single HSS (102 x 102 x 9.5 mm) for each loading point on

top of the beam and to a second HSS of similar section that was embedded in the foundation.

The test frames were subjected to in-plane reverse cyclic loading by an MTS hydraulic actuator of 1000 kN capacity. This actuator imposed the prescribed incrementally increasing lateral displacements to simulate seismic loading. The actuator has a stroke capacity of 500 mm; therefore the frame can be pushed or pulled to a maximum displacement of 250 mm in each direction. The actuator was set parallel to the frame at the height of the beam and was fixed at one end to a steel reaction frame and to the frame beam at the other end. The actuator was connected to the frame with two steel plates; one at either end of the extension of the beam. The two plates were secured to the frame by means of two Dywidag bars (1069 MPa ultimate strength) running along each side of the beam. The function of the four Dywidag bars was to enable the actuator to pull the frame from the far end, while the pushing mode of the frame was achieved through bearing of the actuator against the steel plate at the near side of the actuator. The dywidag bars were not prestressed, and hence were not expected to impose additional axial force on the beam during testing. The test frames were painted white prior to testing to enable visual observation of cracking during testing. Figure 6.57 illustrates the frame setup before testing.

## **6.10 Instrumentations and Data Acquisition System**

Four tests were performed: 1) one Bare Control Frame (BCF) test; 2) two tests on a repaired and retrofitted frame (RRF); and 3) one test on a retrofitted frame (RF) that was in its undamaged state. The frames were instrumented with Displacement Cable Transducers (DCT's); Linear Variable Displacement Transducers (LVDT's); and electrical resistance strain gauges that were placed on the internal reinforcing steel of the frame, the prestressing strands, and BRB steel core bars. All instrumentations were connected to a data acquisition system; while the MTS actuator was calibrated for data collection. Additional details of the instrumentations used during testing are described in the following sections.

### **6.10.1 Bare Control Frame**

The lateral displacements of the Bare Control Frame (BCF) along the centreline of the applied lateral load were recorded using two displacement cable transducers. These



cables were connected to a steel frame at the far end of the concrete test frame and to a 2" x 4" timber section near the actuator. The two supporting systems were mounted to the laboratory strong floor as shown in Figure 6.58. Two additional displacement cable transducers were connected to the foundation of the frame to record displacements due to slip or rocking of the foundation during testing as shown in Figure 6.59. The lateral displacements of the frame relative to the foundation were calculated from the difference between the total displacements of the test frame and foundation, both relative to the strong floor.

LVDT's were used to measure the vertical displacements at each face of the two columns as illustrated in Figure 6.56 and Figure 6.60. Two LVDT's were connected to each column at 25 mm above the foundation; while two additional LVDT's were positioned at a height of 300 mm (column cross section width) above the foundation level. The 25 mm-height LVDT's were used to measure base anchorage slip of the column reinforcement; while the 300 mm-height LVDT's recorded the column vertical displacements within the plastic hinge region.

#### **6.10.2 Retrofitted Frames**

Bare Control Frame (BCF), after repair, was retrofitted and subjected to two tests using two different BRB steel core bars (AISI 12L14 carbon and AISI Type 304 stainless steel). Instrumentations for both tests were the same as those used to test Frame BCF, with the exception that some instrumentation were removed and others were added to measure the displacements of the frame and BRB system. LVDT's 3, 4, 7 and 8 located at the near column (Figure 6.56) were removed to accommodate the lower hinge joint steel sections for the BRB. Therefore, column anchorage slip and vertical displacements within the plastic hinge region were recorded only by LVDT's connected to the far column (LVDT's 1, 2, 5 and 6). An additional LVDT was connected to the side of the foundation to measure uplift displacements during testing as shown in Figure 6.61.

Additional instrumentations were used during testing of Frame RRF that incorporated the AISI 12L14 carbon steel BRB steel core bar. This included two DCT's that were connected to the steel plates of the upper and lower hinge joint assemblies at one end and to the HSS round casing at the other end. The instrumentation measured the longitudinal



displacements of the brace along the steel core bar near the upper and lower hinge joints. Figure 6.62 illustrates the DCT's and the retrofitted frame setup before testing.

Further instrumentations were employed during testing of Frame RRF that used the AISI Type 304 stainless steel BRB brace bar. This consisted of one DCT to measure the lateral displacements at the centreline of the applied lateral load relative to the foundation. This was intended to eliminate any contribution of foundation slip/rocking on the lateral displacements experienced by the concrete frame. The DCT was attached to a light steel frame that was connected to the foundation of the test frame as shown in Figure 6.63. The arrangement of displacement cable transducers used to measure the BRB steel core bar longitudinal displacements was similar to that of the previous retrofitting test (Figures 6.62 (a) and (b)) with the exception that the DCT connected to the upper hinge joint plate at one end was connected to the lower hinge joint plate at the other end. This modification was intended to measure the total displacements between the joint plates (i.e. total steel core bar displacements). In addition, adjacent to the ends of the reduced section of the core steel bar (Figures 6.35 (b) and (c)), two wires were connected before inserting the core bar into the BRB casing as shown in Figure 6.64. These steel wires were then drawn out of the HSS casing through a drilled hole and connected to two DCT's that were connected to the lower hinge joint plate as shown in Figures 6.65 (a) and (b). Two pulleys were secured to the HSS casing to hold the steel wires. The lower hinge joint plate also held the other two DCT's: one that was connected to the HSS casing (Figure 6.65 (b)), and another that was connected to the upper hinge joint plate (Figure 6.65 (c)). Figure 6.66 illustrates the repaired retrofitted frame before testing.

The instrumentation used during the testing of Frame RF (retrofitted virgin frame), incorporating the AISI 4140 chrome-molybdenum high tensile steel core bar was identical to Frame RRF that used the AISI Type 304 stainless steel core BRB bar.

All three BRB steel core bars used in retrofitting the concrete frames were instrumented with strain gauges as illustrated in Figures 6.39 (a) and (b). Each of the three steel core bars was set within the BRB system such that S.G. # 1-4 were adjacent to the beam-column joint, while S.G. # 9-12 were near the joint of the frame column and the foundation.

### 6.11 Loading Program

Lateral loading, simulating earthquake actions, was applied in a displacement-controlled mode and consisted of three cycles of incrementally increasing displacement reversals. The lateral displacements of the frames were measured by DCTs connected at the mid height of the loading beam.

Two different loading programs were used to test the Bare Control Frame (BCF) and the retrofitted frames (RRF and RF). This was due to the higher stiffness of the retrofitted frames. The loading protocol selected included more cycles at each deformation level than what is suggested by existing loading protocols (ACI 2013), to closely monitor the performance of BRB during testing, with amplified potential effects of cycling. The bare control frame was loaded in drift increments of 0.25% until 1% drift and then in increments of 0.5% until 3% drift, followed by a final drift of 4%. The retrofitted frames were loaded in drift increments of 0.125% until 0.75% drift and then in increments of 0.5% from 1% drift until 3% drift. Herein failure was defined as the first cycle to experience a 20% reduction of lateral load strength capacity relative to the peak recorded strength. The loading programs are summarized for the bare control frame and for the retrofitted frames in Tables 6.7 and 6.8, respectively, and are further illustrated with the imposed number of cycles at each drift in Figures 6.67 and 6.68, respectively.

Table 6.1: Average Compressive Strengths of Concrete in Frames  
BCF and RF (MPa)

| Frame Reference                                                                                                                           | 28 Days    | Day of Test |
|-------------------------------------------------------------------------------------------------------------------------------------------|------------|-------------|
| Bare Control Frame (BCF)                                                                                                                  | 27.6       | 30          |
| Repaired Retrofitted Frame (RRF) <ul style="list-style-type: none"> <li>Columns and Beam Repair Self-Compacting Concrete (SCC)</li> </ul> | 27.6<br>49 | 31<br>61    |
| Retrofitted Frame (RF)                                                                                                                    | 28.2       | 31          |

Table 6.2: Frame member moment (M), axial force (P) and shear force (V) due to gravity and unit horizontal loads for the bare control frame

| Member      | Location          | Forces | Forces due to gravity load (kN·m & kN) | Forces due to unit horiz. load (kN·m & kN) | Total forces due to gravity and unit horiz. loads (kN·m & kN) |
|-------------|-------------------|--------|----------------------------------------|--------------------------------------------|---------------------------------------------------------------|
| Near Column | Beam-column joint | M      | 32.07                                  | -0.71                                      | 32.07 - 0.71 (H)                                              |
|             |                   | V      | -13.96                                 | 0.5                                        | - 13.96 + 0.5 (H)                                             |
|             | Base              | M      | -15.74                                 | 1                                          | - 15.74 + 1.0 (H)                                             |
|             |                   | V      | -13.96                                 | 0.5                                        | - 13.96 + 0.5 (H)                                             |
|             | Entire span       | P      | -862.57                                | 0.37                                       | - 862.57 + 0.37 (H)                                           |
| Far Column  | Beam-column joint | M      | -32.07                                 | -0.71                                      | - 32.07 - 0.71 (H)                                            |
|             |                   | V      | -13.96                                 | -0.5                                       | - 13.96 - 0.5 (H)                                             |
|             | Base              | M      | 15.74                                  | 1                                          | 15.74 + 1.0 (H)                                               |
|             |                   | V      | -13.96                                 | -0.5                                       | - 13.96 - 0.5 (H)                                             |
|             | Entire span       | P      | -862.57                                | -0.37                                      | - 862.57 - 0.37 (H)                                           |
| Beam        | Near support      | M      | -32.07                                 | 0.71                                       | - 32.07 + 0.71 (H)                                            |
|             |                   | V      | -62.26                                 | 0.37                                       | - 62.26 + 0.37 (H)                                            |
|             | Far support       | M      | -32.07                                 | -0.71                                      | - 32.07 - 0.71 (H)                                            |
|             |                   | V      | 62.26                                  | 0.37                                       | 62.26 + 0.37 (H)                                              |
|             | Mid span          | M      | 30.22                                  | 0                                          | 30.22 + 0 (H)                                                 |
|             |                   | V      | 0                                      | 0.37                                       | 0 + 0.37 (H)                                                  |
|             | Entire span       | P      | -13.96                                 | -0.5                                       | - 13.96 - 0.5 (H)                                             |

Table 6.3: Frame member moment (M), axial force (P) and shear force (V) due to gravity and unit horizontal loads for the retrofitted frames

| Member             | Location          | Forces | Forces due to gravity load (kN·m & kN) | Forces due to unit horiz. load (kN·m & kN) | Total forces due to gravity and unit horiz. loads (kN·m & kN) |
|--------------------|-------------------|--------|----------------------------------------|--------------------------------------------|---------------------------------------------------------------|
| BRB steel core bar | Entire span       | P      | -6.61                                  | 1.24                                       | - 6.6 + 1.24 (H)                                              |
| Near Column        | Beam-column joint | M      | 28.6                                   | -0.06                                      | 28.6 - 0.06 (H)                                               |
|                    |                   | V      | 11.5                                   | -0.04                                      | 11.5 - 0.04 (H)                                               |
|                    | Base              | M      | -10.8                                  | 0.09                                       | - 10.8 + 0.09 (H)                                             |
|                    |                   | V      | 11.5                                   | -0.04                                      | 11.5 - 0.04 (H)                                               |
|                    | Entire span       | P      | -861                                   | 0.03                                       | - 861 + 0.03 (H)                                              |
| Far Column         | Beam-column joint | M      | -35.6                                  | -0.06                                      | - 35.6 - 0.06 (H)                                             |
|                    |                   | V      | -16.4                                  | -0.04                                      | - 16.4 - 0.04 (H)                                             |
|                    | Base              | M      | 20.6                                   | 0.08                                       | 20.6 + 0.08 (H)                                               |
|                    |                   | V      | -16.4                                  | -0.04                                      | - 16.4 - 0.04 (H)                                             |
|                    | Entire span       | P      | -861                                   | -0.86                                      | - 861 - 0.86 (H)                                              |
| Beam               | Near support      | M      | -28.6                                  | 0.06                                       | - 28.6 + 0.06 (H)                                             |
|                    |                   | V      | -60.4                                  | 0.03                                       | - 60.4 + 0.03 (H)                                             |
|                    | Far support       | M      | -35.6                                  | -0.06                                      | - 35.6 - 0.06 (H)                                             |
|                    |                   | V      | 64                                     | 0.03                                       | 64 + 0.03 (H)                                                 |
|                    | Mid span          | M      | 31.8                                   | 0                                          | 31.8 + 0 (H)                                                  |
|                    |                   | V      | 1.9                                    | 0.03                                       | 1.9 + 0.03 (H)                                                |
|                    | Entire span       | P      | -11.5                                  | -0.96                                      | - 11.5 - 0.96 (H)                                             |

Table 6.4: Sequence and location of damage due to lateral force on the bare control frame

| Lateral force, H (kN) | Member        | Position from actuator (Push and Pull) | Location of failure             | Damage mode       |
|-----------------------|---------------|----------------------------------------|---------------------------------|-------------------|
| 138                   | Column        | Far                                    | Base                            | Axial and flexure |
| 165                   | Column        | Near                                   | Base                            | Axial and flexure |
| 182                   | Column        | Far                                    | Beam-column joint               | Axial and flexure |
| 200                   | Beam support  | Near                                   | Bottom of cross section (+ve M) | Axial and flexure |
| 235                   | Beam support  | Far                                    | Top of cross section (-ve M)    | Axial and flexure |
| 270                   | Column        | Near                                   | Beam-column joint               | Axial and flexure |
| 314                   | Column        | Far                                    | Base                            | Shear             |
| 314                   | Column        | Near                                   | Base                            | Shear             |
| 354                   | Column        | Far                                    | Beam-column joint               | Shear             |
| 370                   | Column        | Near                                   | Beam-column joint               | Shear             |
| 478                   | Beam support  | Far                                    | $d_v$ from support              | Shear             |
| 505                   | Beam support  | Near                                   | $d_v$ from support              | Shear             |
| 1000                  | Beam mid span | Near/Far                               | Mid span                        | Shear             |
| 7500                  | Beam mid span | Near/Far                               | Bottom of cross section (+ve M) | Axial and flexure |

Table 6.5: Sequence and location of damage due to lateral force on the retrofitted frames

| Lateral force, H (kN) | Member                                                      | Position from actuator (Push and Pull) | Location of failure             | Damage mode        |
|-----------------------|-------------------------------------------------------------|----------------------------------------|---------------------------------|--------------------|
| 503                   | AISI 12L14 Carbon BRB steel core bar                        | Diagonal                               | Entire span                     | Fracture (tension) |
| 390                   | AISI Type 304 Stainless BRB steel core bar                  | Diagonal                               | Entire span                     | Fracture (tension) |
| 479                   | AISI 4140 Chrome-Molybdenum High Tensile BRB steel core bar | Diagonal                               | Entire span                     | Fracture (tension) |
| 1280                  | Column                                                      | Far                                    | Base                            | Axial and flexure  |
| 1480                  | Column                                                      | Far                                    | Beam-column joint               | Axial and flexure  |
| 1541                  | HSS - Round                                                 | Diagonal                               | Mid length                      | Buckling           |
| 1800                  | Column                                                      | Near                                   | Base                            | Axial and flexure  |
| 2100                  | Beam support                                                | Far                                    | Top of cross section (-ve M)    | Axial and flexure  |
| 3130                  | Beam support                                                | Near                                   | Bottom of cross section (+ve M) | Axial and flexure  |
| 3150                  | Column                                                      | Near                                   | Beam-column joint               | Axial and flexure  |
| 3359                  | Column                                                      | Near                                   | Base                            | Shear              |
| 3900                  | Beam mid span                                               | Near/Far                               | Bottom of cross section (+ve M) | Axial and flexure  |
| 4146                  | Column                                                      | Near                                   | Beam-column joint               | Shear              |
| 8300                  | Column                                                      | Far                                    | Base                            | Shear              |
| 8300                  | Column                                                      | Far                                    | Beam-column joint               | Shear              |
| 16820                 | Beam support                                                | Far                                    | $d_v$ from support              | Shear              |
| 18770                 | Beam mid span                                               | Near/Far                               | Mid span                        | Shear              |
| 20792                 | Beam support                                                | Near                                   | $d_v$ from support              | Shear              |

Table 6.6: Elongation of BRB steel core bar relative to frame lateral displacement

| Frame lateral displacement,<br>$\delta$ (mm) | Frame lateral drift (%) | Brace inclination angle to horizontal,<br>$\Theta$ (rad) | Elongation of steel core bar,<br>$\Delta L$ (mm) |
|----------------------------------------------|-------------------------|----------------------------------------------------------|--------------------------------------------------|
| 4                                            | 0.125                   | 40.95                                                    | 3                                                |
| 8                                            | 0.25                    | 41.01                                                    | 6                                                |
| 12                                           | 0.375                   | 41.06                                                    | 9                                                |
| 16                                           | 0.5                     | 41.12                                                    | 12                                               |
| 20                                           | 0.625                   | 41.18                                                    | 15                                               |
| 24                                           | 0.75                    | 41.23                                                    | 18                                               |
| 32                                           | 1.0                     | 41.34                                                    | 24                                               |
| 48                                           | 1.5                     | 41.56                                                    | 36                                               |
| 63.5                                         | 2.0                     | 41.77                                                    | 47.5                                             |
| 79                                           | 2.5                     | 41.98                                                    | 59                                               |
| 95                                           | 3.0                     | 42.19                                                    | 70.5                                             |



Table 6.7: Displacement loading cycles of the bare control frame

| Load cycle | Frame lateral displacement (mm) | Lateral drift (%) |
|------------|---------------------------------|-------------------|
| 1          | 8                               | 0.25              |
| 2          | 16                              | 0.5               |
| 3          | 24                              | 0.75              |
| 4          | 32                              | 1                 |
| 5          | 48                              | 1.5               |
| 6          | 63.5                            | 2                 |
| 7          | 79                              | 2.5               |
| 8          | 95                              | 3                 |
| 9          | 127                             | 4                 |

Table 6.8: Displacement loading cycles of the retrofitted frames

| Load cycle | Frame lateral displacement (mm) | Lateral drift (%) |
|------------|---------------------------------|-------------------|
| 1          | 4                               | 0.125             |
| 2          | 8                               | 0.25              |
| 3          | 12                              | 0.375             |
| 4          | 16                              | 0.5               |
| 5          | 20                              | 0.625             |
| 6          | 24                              | 0.75              |
| 7          | 32                              | 1                 |
| 8          | 48                              | 1.5               |
| 9          | 63.5                            | 2                 |
| 10         | 79                              | 2.5               |
| 11         | 95                              | 3                 |

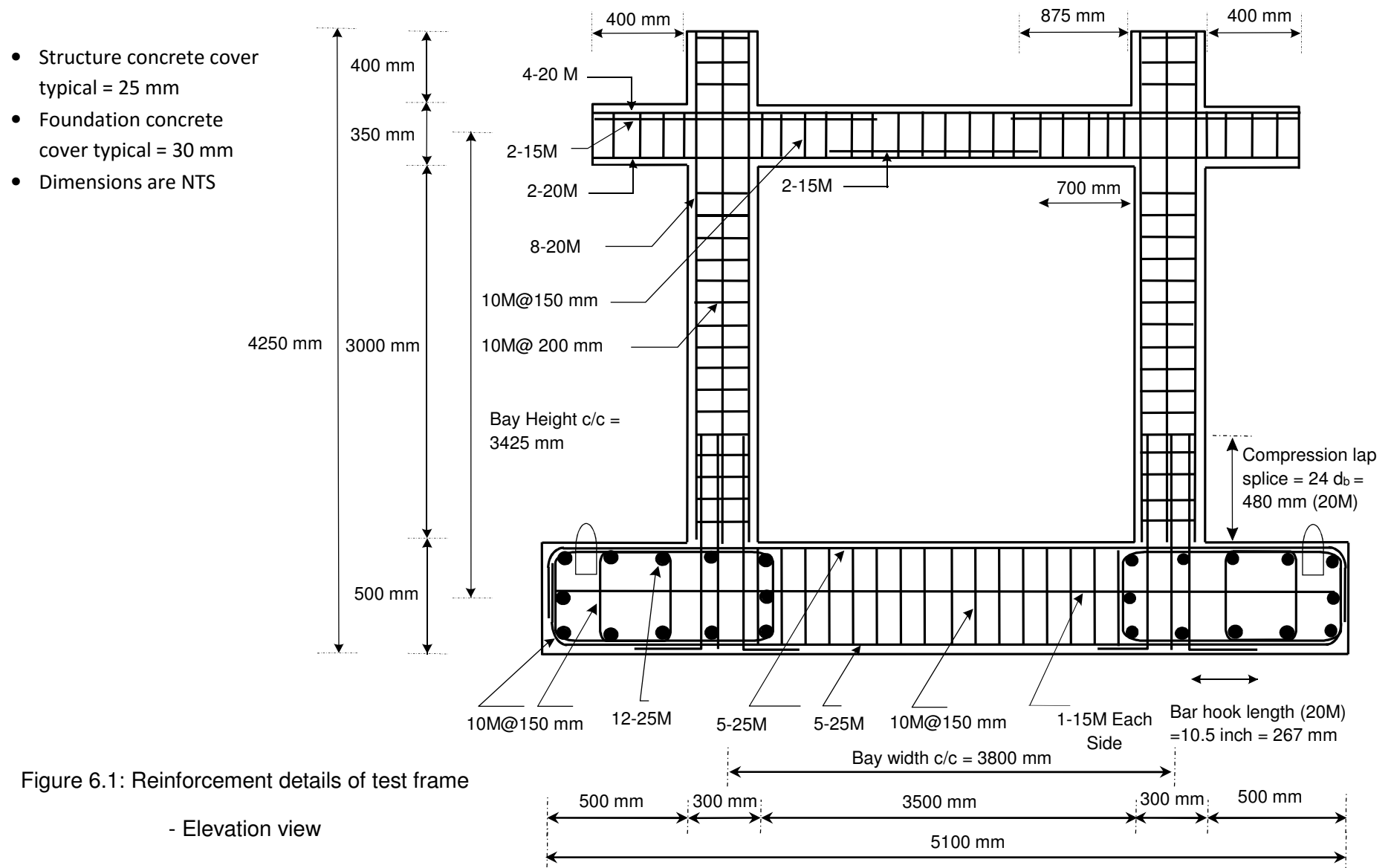
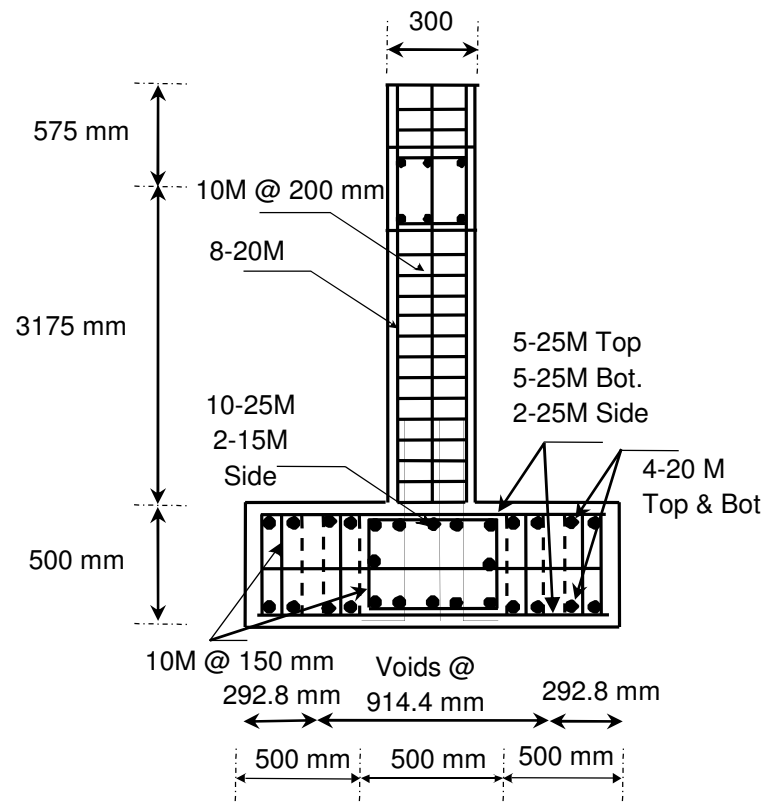
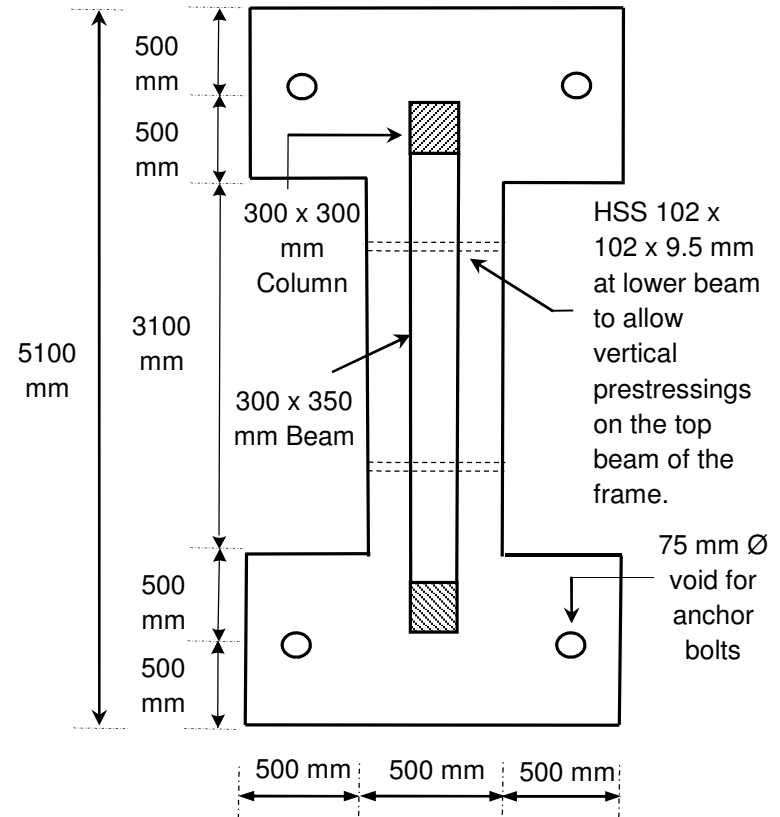


Figure 6.1: Reinforcement details of test frame

- Elevation view



Frame Sectional View –Reinforcement Details



Frame Plan View - Layout

Figure 6.2: Frame sectional and plan views

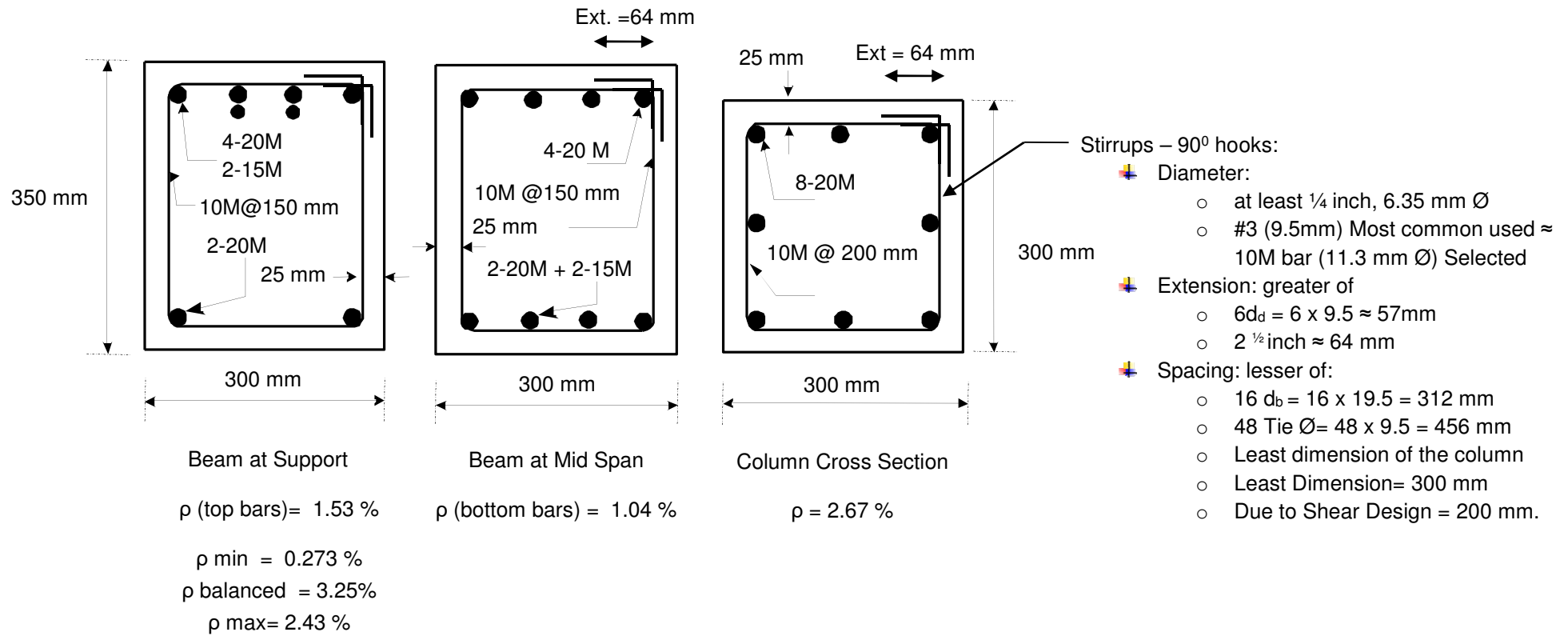


Figure 6.3: Frame column and beam cross-section details



Figure 6.4: Construction of foundation reinforcement cage



Figure 6.5: Foundation formwork and reinforcement cage with embedded column starter bars





Figure 6.6: Concrete casting of the fame foundation

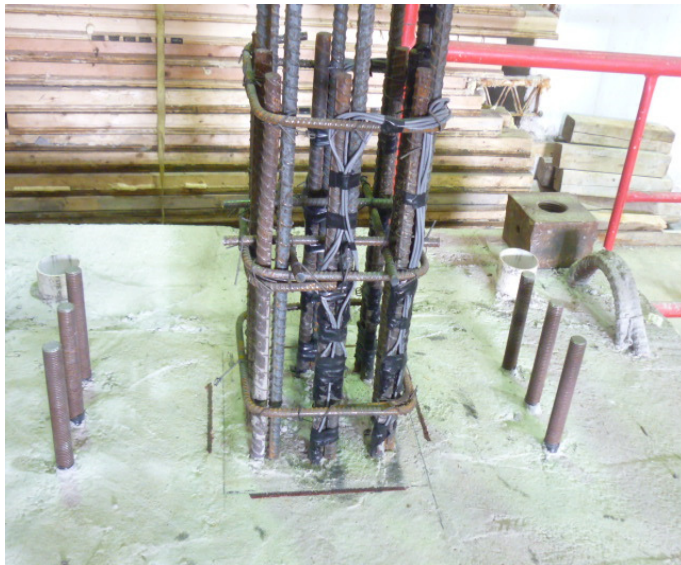


Figure 6.7: Placement of column longitudinal reinforcement and strain gauges



Figure 6.8: Placement of column longitudinal and transverse tie reinforcements

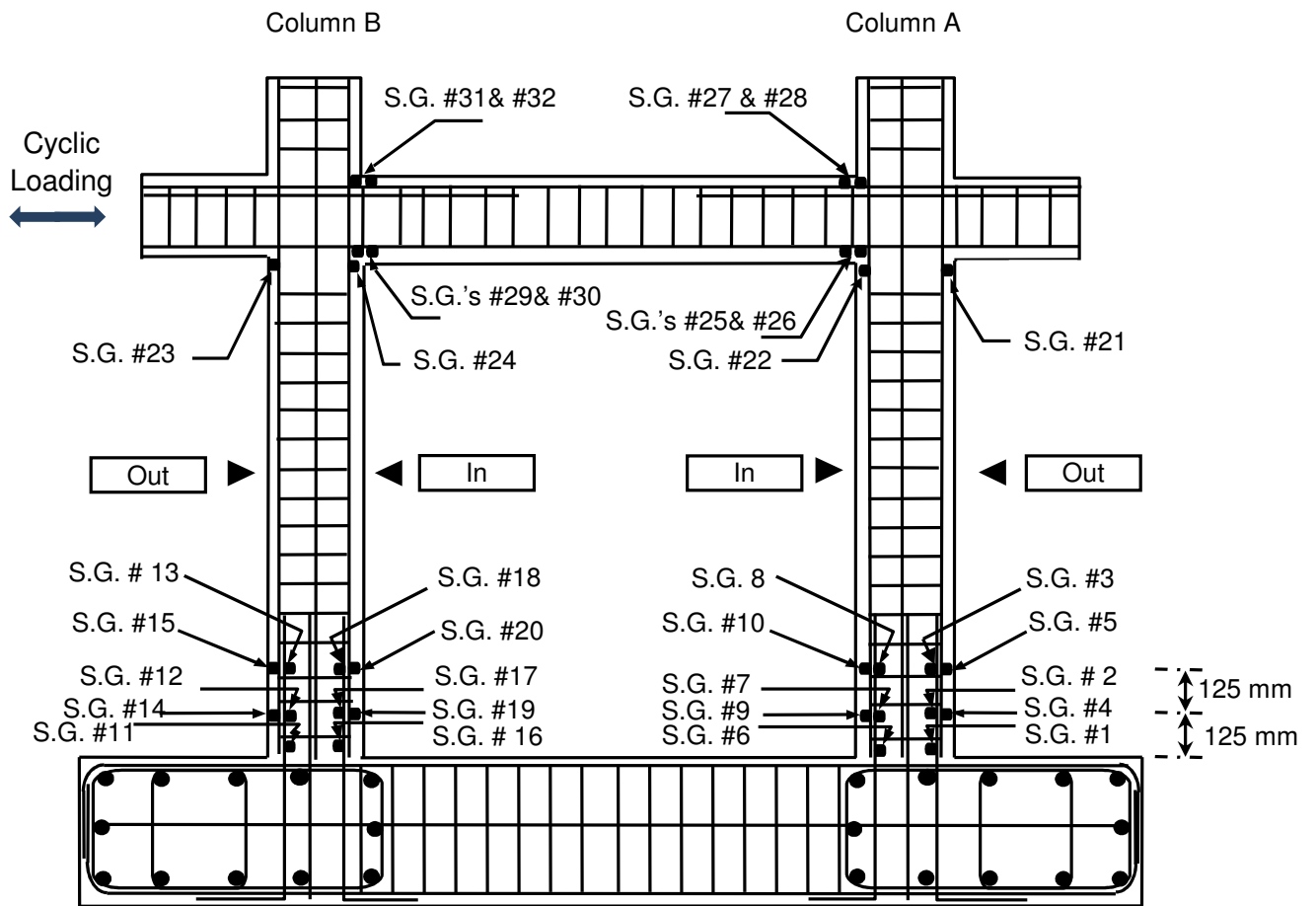


Figure 6.9: Strain gauge locations on the frame reinforcement



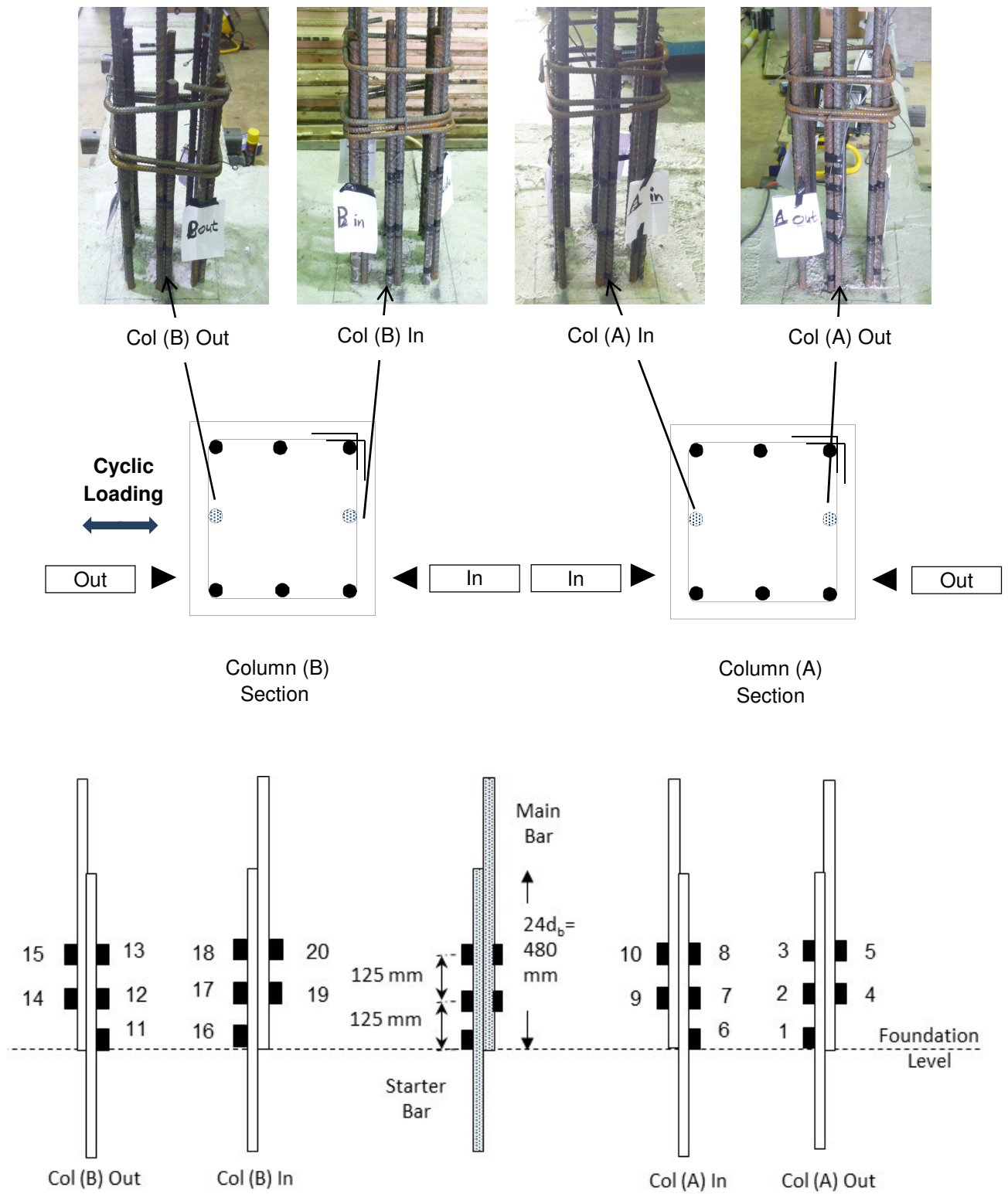


Figure 6.10: Strain gauge numbering at the base of the columns



Figure 6.11: Construction of formwork prior to placement of beam reinforcement and concrete casting



Figure 6.12: Placement of frame beam reinforcement



Figure 6.13: Frame specimen after concrete casting



Figure 6.14: Compressive strength testing of concrete cylinders

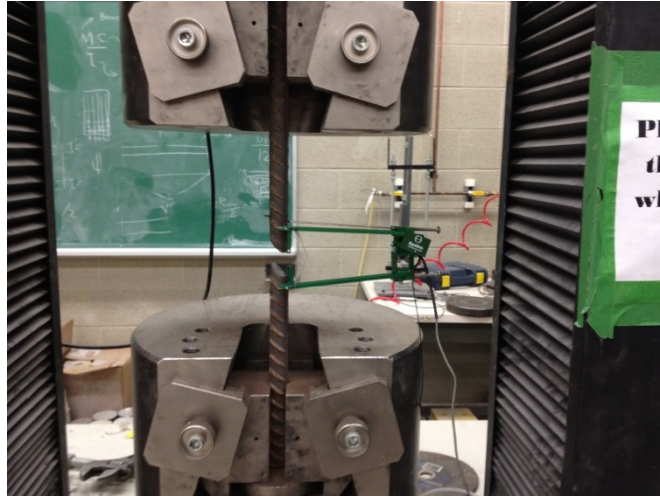


Figure 6.15: Testing of steel coupon bars in Galdabini UTM

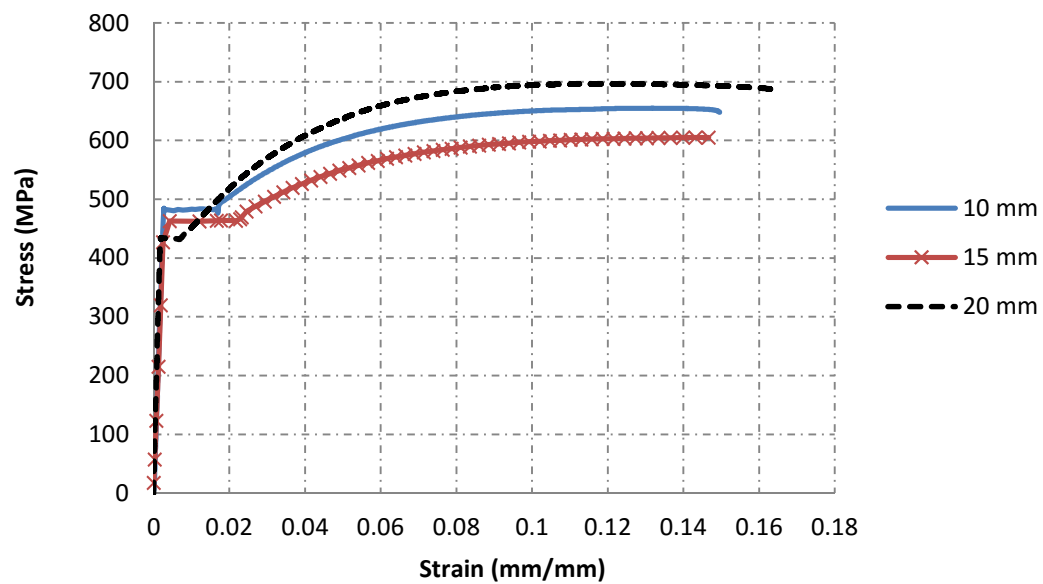


Figure 6.16: Typical stress-strain relationships of the reinforcing bars used in the frames



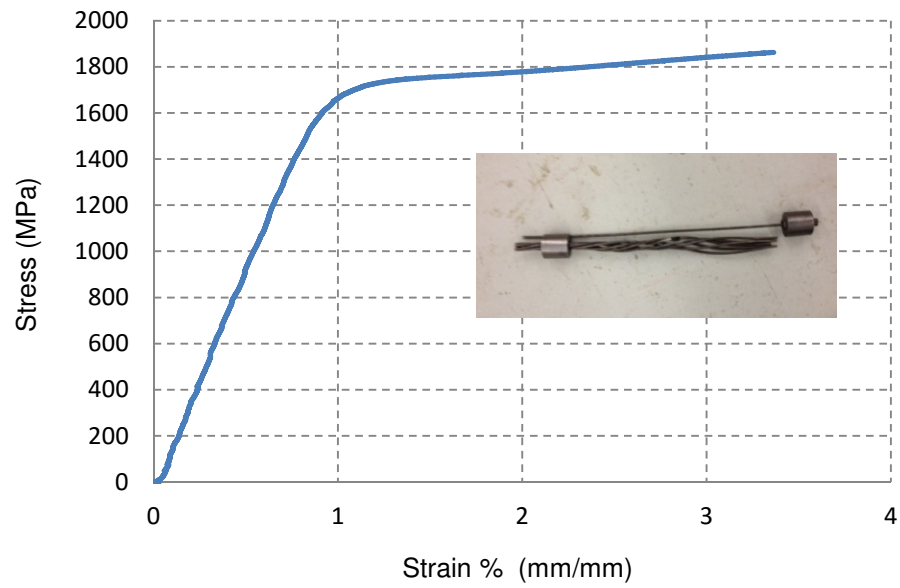


Figure 6.17: Typical stress-strain relationship for the 15.6 mm prestressing 7 wire strands

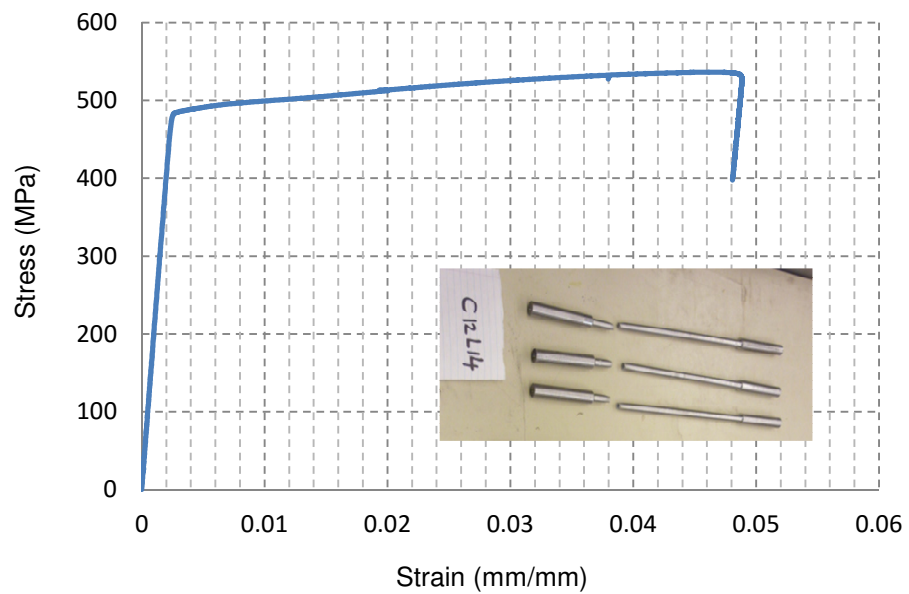


Figure 6.18: Typical stress-strain relationship for the AISI 12L14 carbon steel core bar

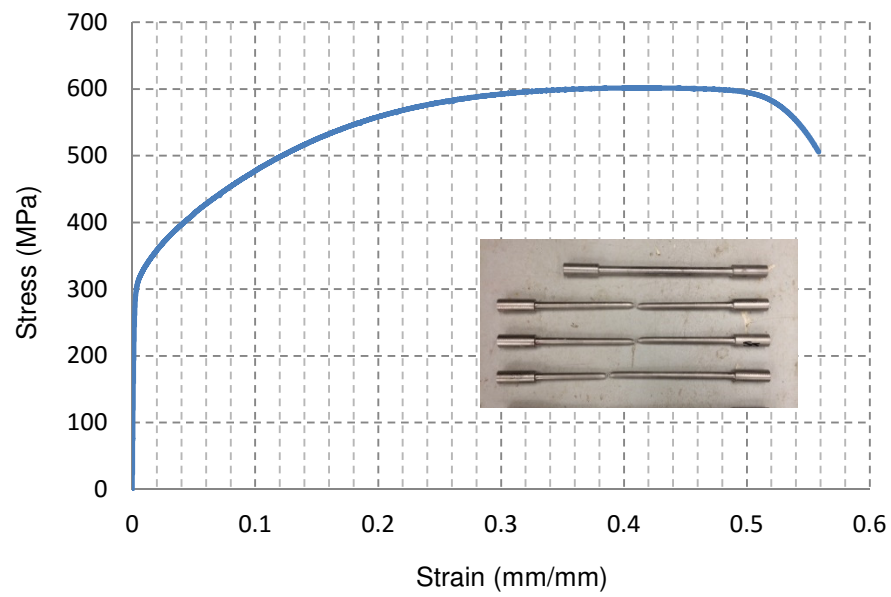


Figure 6.19: Typical stress-strain relationship for the AISI Type 304 stainless steel core bar

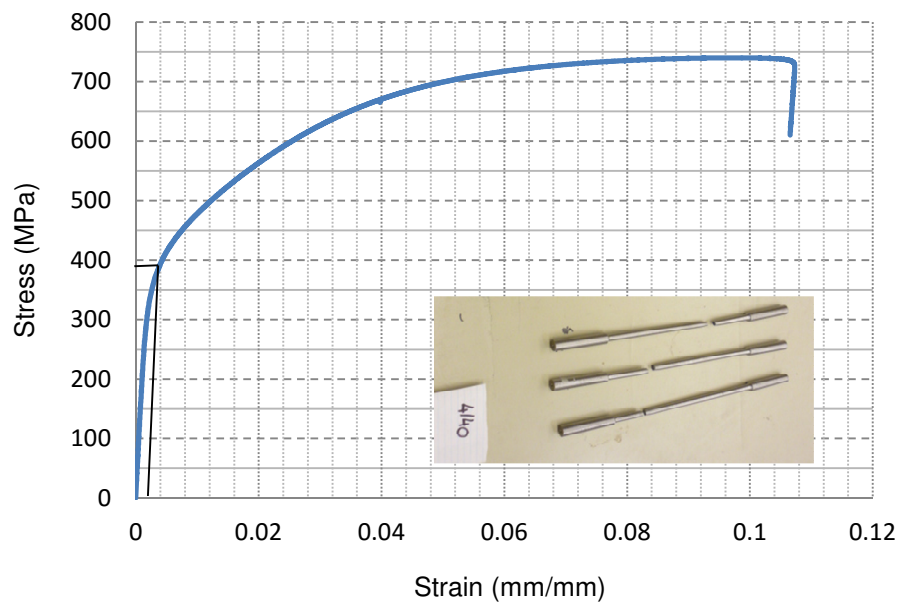


Figure 6.20: Typical stress-strain relationship for the AISI 4140 chrome-molybdenum high tensile steel core bar

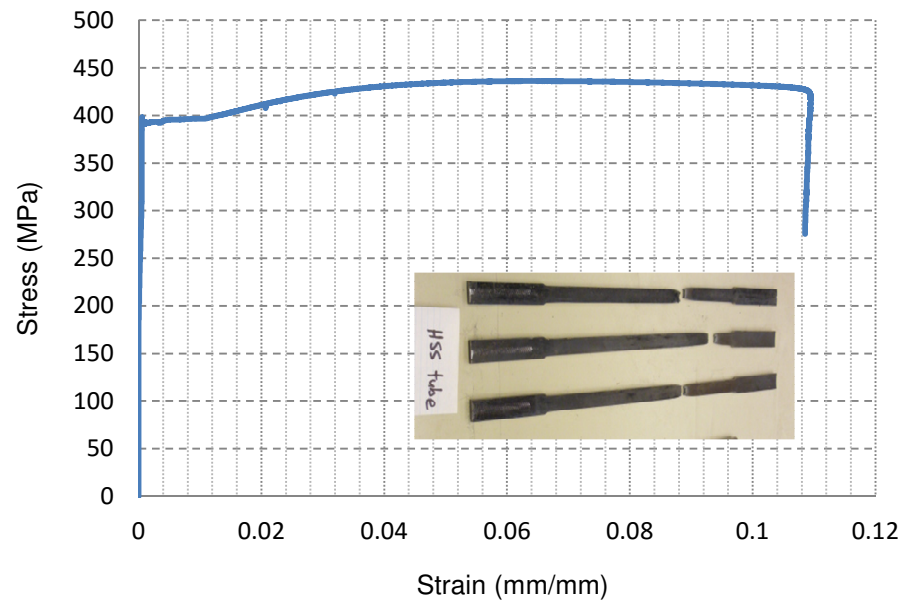
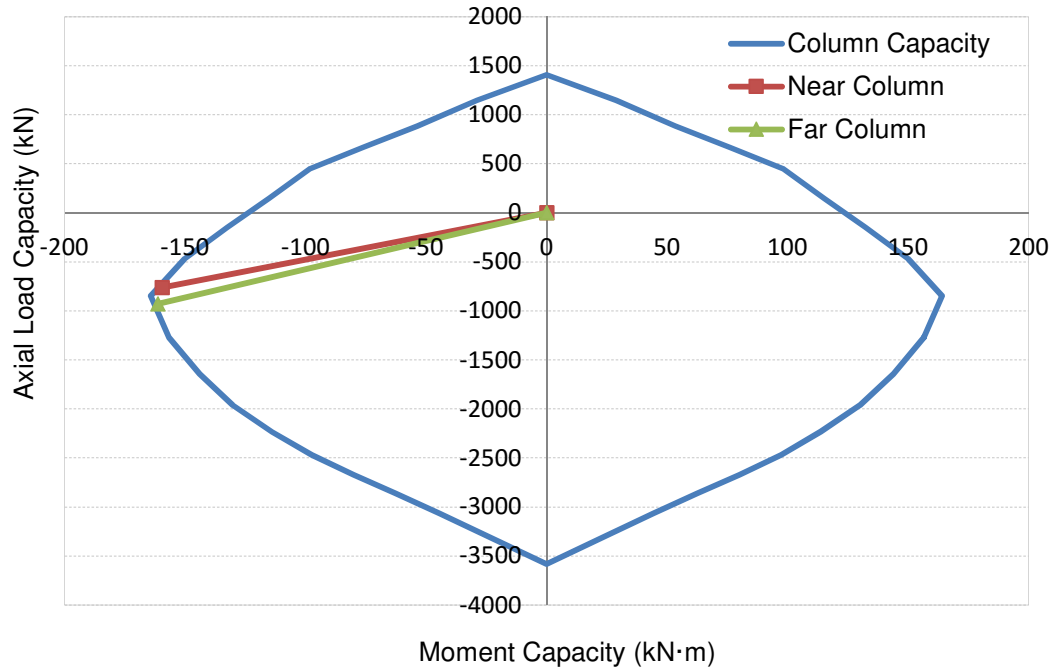
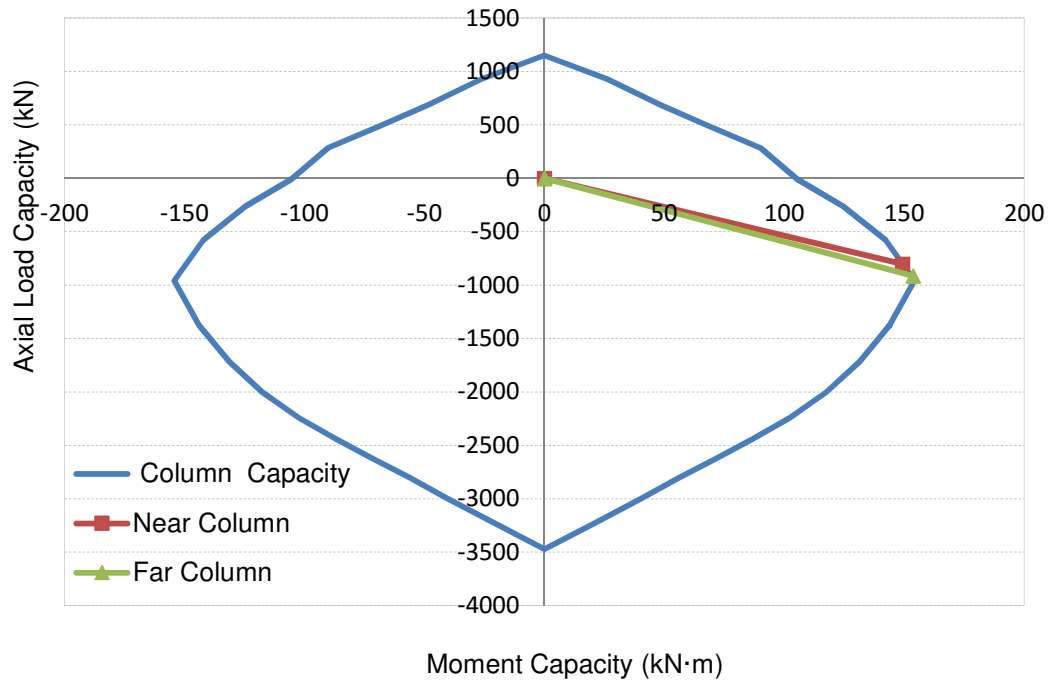


Figure 6.21: Typical stress-strain relationship for the HSS Round Casing



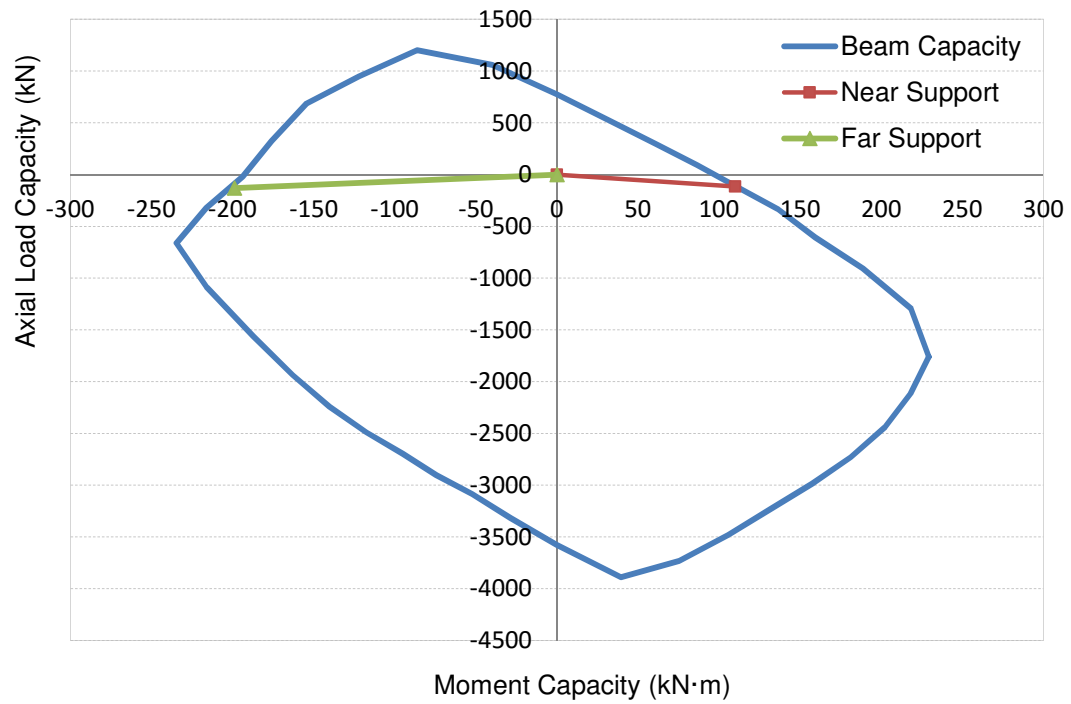
(a)



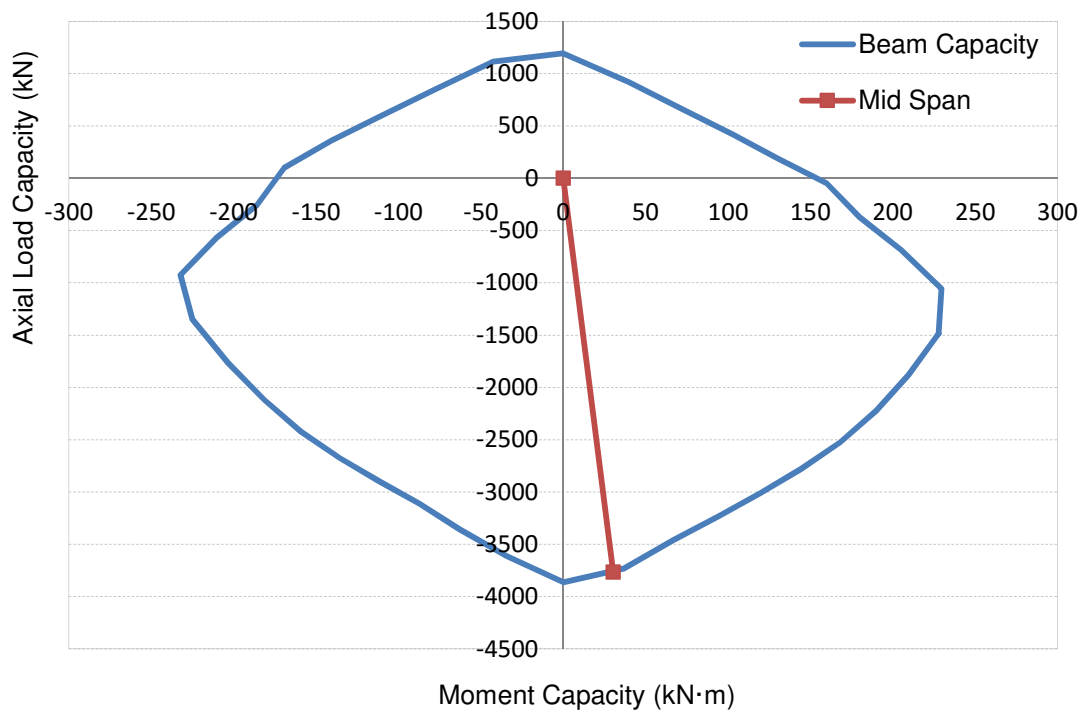
(b)

Figure 6.22: Interaction diagram of columns and applied loading for the near and far columns of the bare control frame: a) at beam-column joint ( $f_y = 430$  MPa); and b) at foundation level ( $f_y = 311$  MPa)



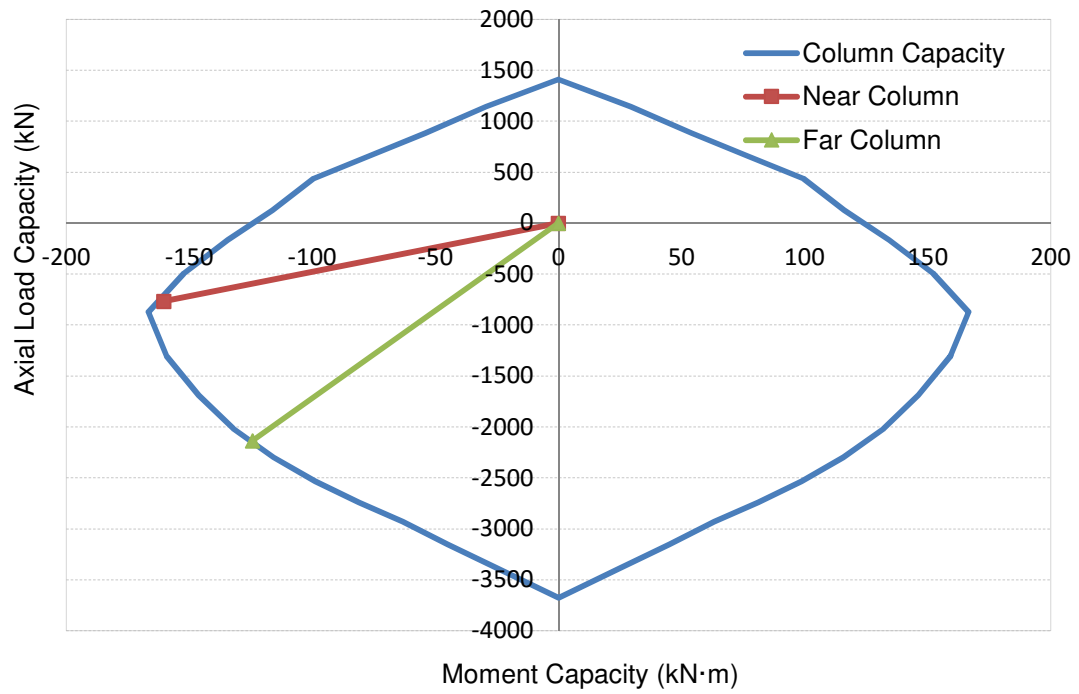


(a)

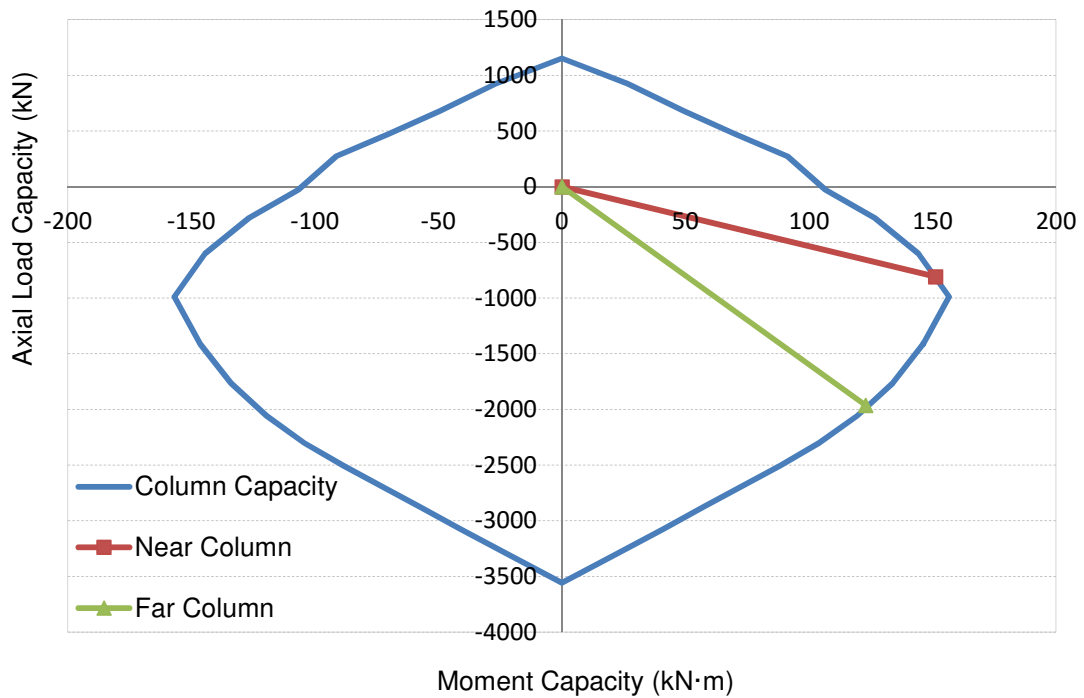


(b)

Figure 6.23: Interaction diagram of beam of the bare control frame: a) at supports; and b) at mid span

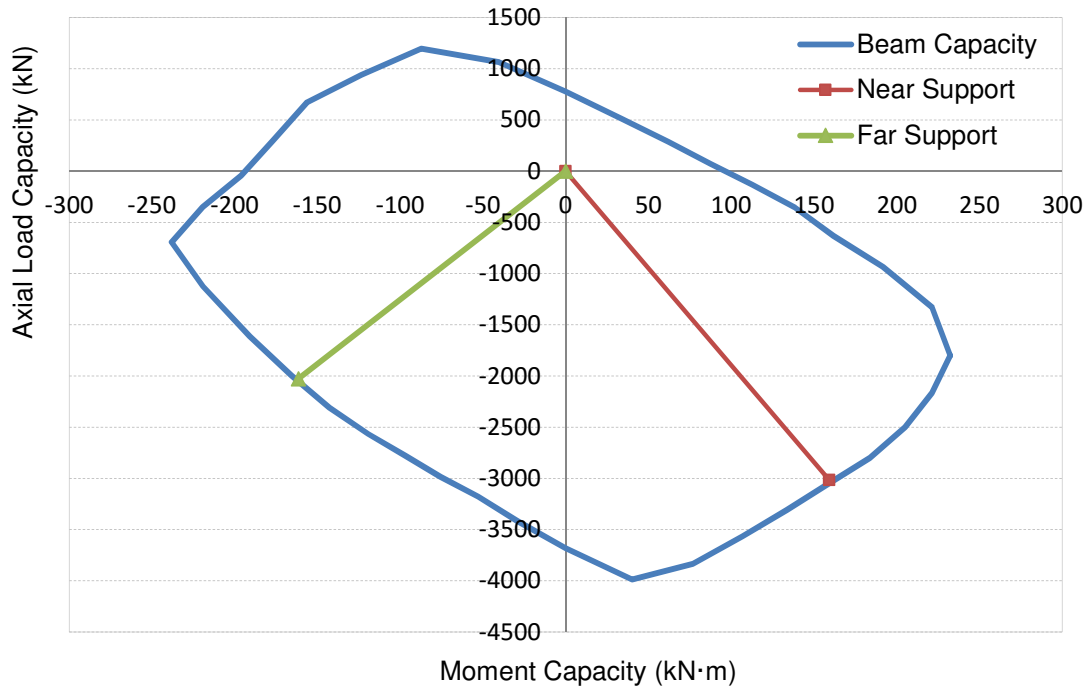


(a)

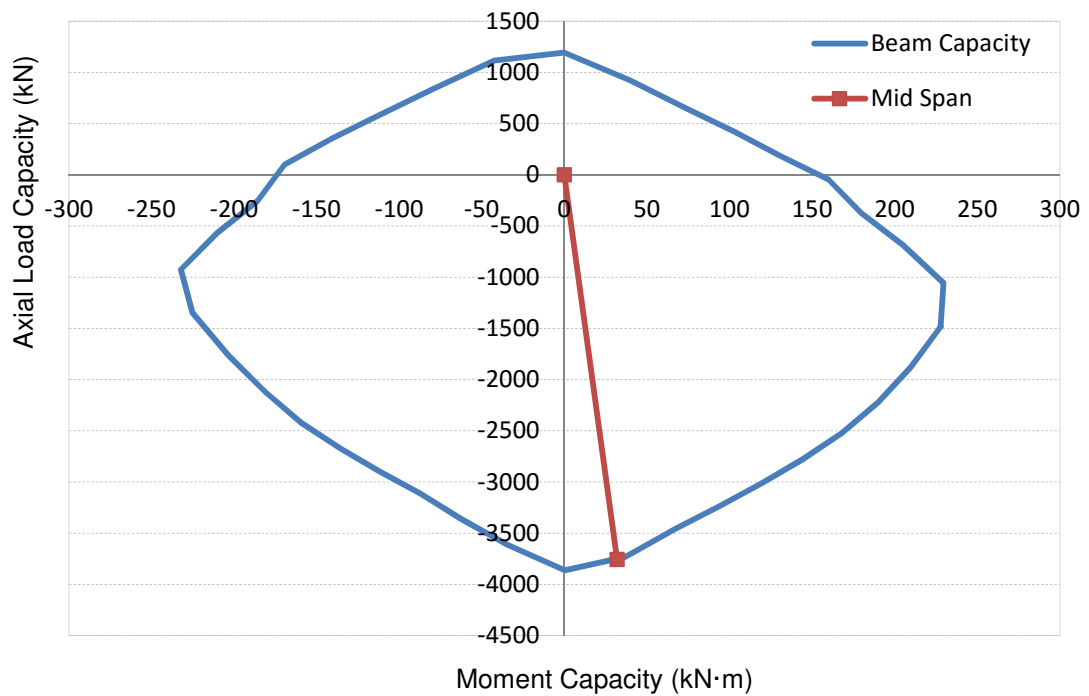


(b)

Figure 6.24: Interaction diagram of columns and applied loading for the near and far columns of the retrofitted frames: a) at beam-column joint ( $f_y = 430$  MPa); and b) at foundation level ( $f_y = 311$  MPa)



(a)



(b)

Figure 6.25: Interaction diagram of beam of the retrofitted frames: a) at supports; and b) at mid span



(a)



(b)



(c)

Figure 6.26: Removal of loose concrete and buckled steel bar segments of the bare control frame: a) columns; b) beam-column joint; and c) connecting new steel bar segments with couplers in beam





Figure 6.27: Bare control frame before placing repair formwork



Figure 6.28: Formwork before casting the new self-compacting concrete (Sikacrete-08 SCC)



Figure 6.29: SIKA products used in the repairing process





(a)



(b)

Figure 6.30: Casting of Self-Compacting Concrete (SCC) to the frame: a) columns; and b) beam-column joints

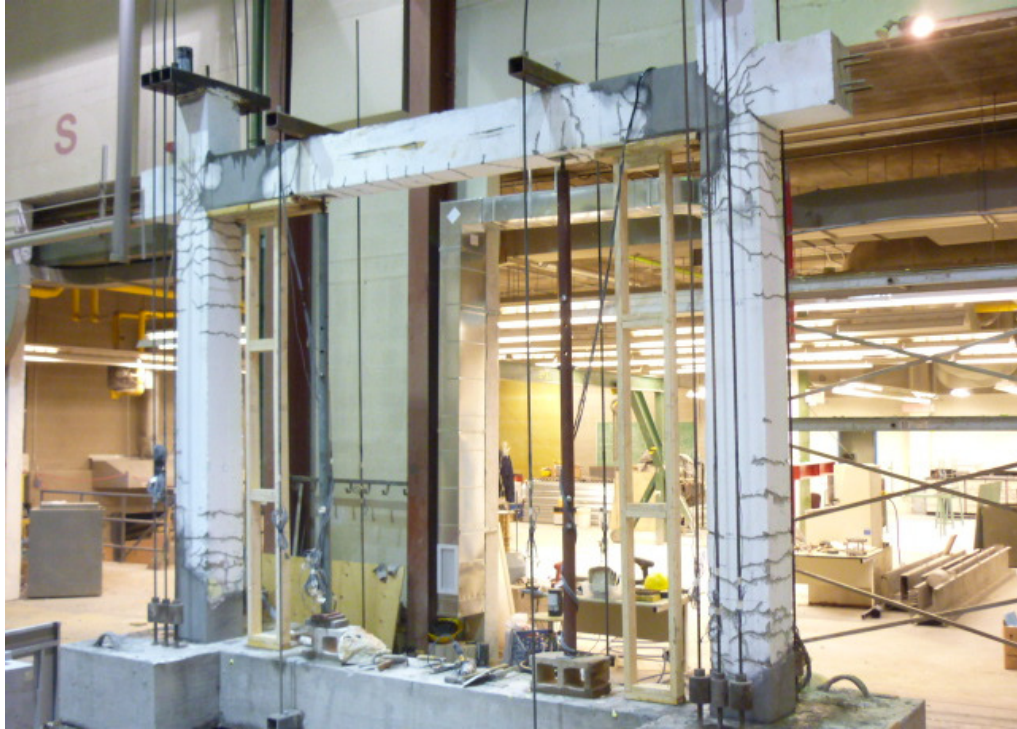


Figure 6.31: Bare control frame after casting SCC



Figure 6.32: Application of bonding agent (SikaTop Armatec 110 EpoCem) prior to patching with mortar





(a)



(b)

Figure 6.33: Application of SikaTop 123 plus mortar to the frame: a) at columns; and b) adjacent to beam-column joint



Figure 6.34: Bare control frame at end of repairing

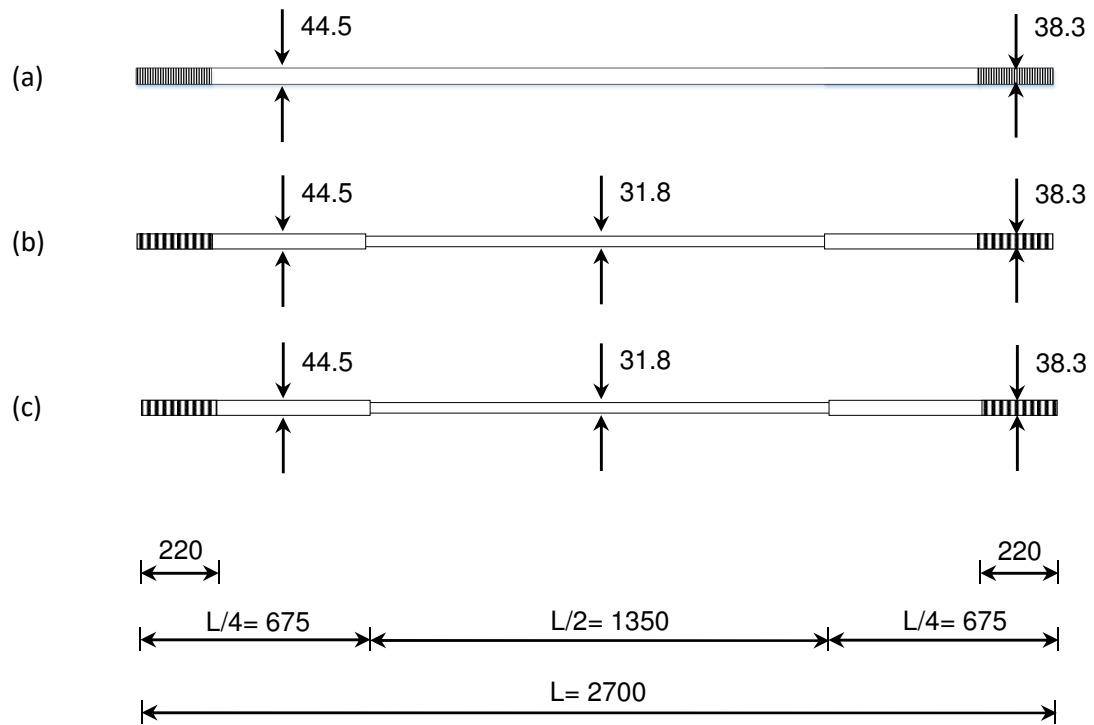


Figure 6.35: Details of the BRB steel core bars: a) AISI 12L14 carbon steel; b) AISI Type 304 stainless steel; and c) AISI 4140 chrome-molybdenum high tensile steel (All dimensions in mm)

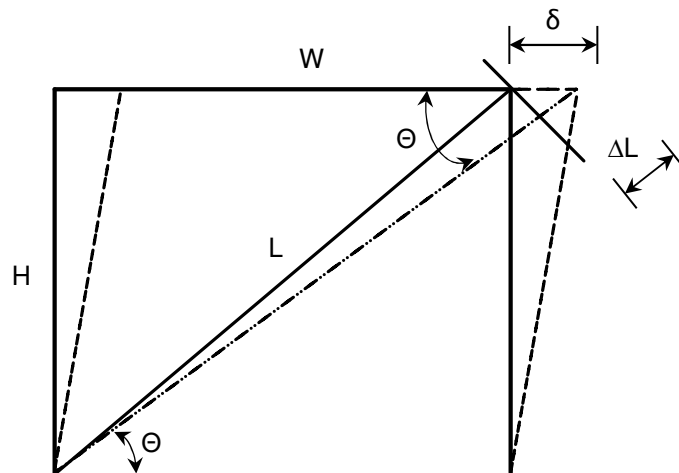


Figure 6.36: BRB bar strain demand calculations with respect to angle of inclination and horizontal displacements of the frame

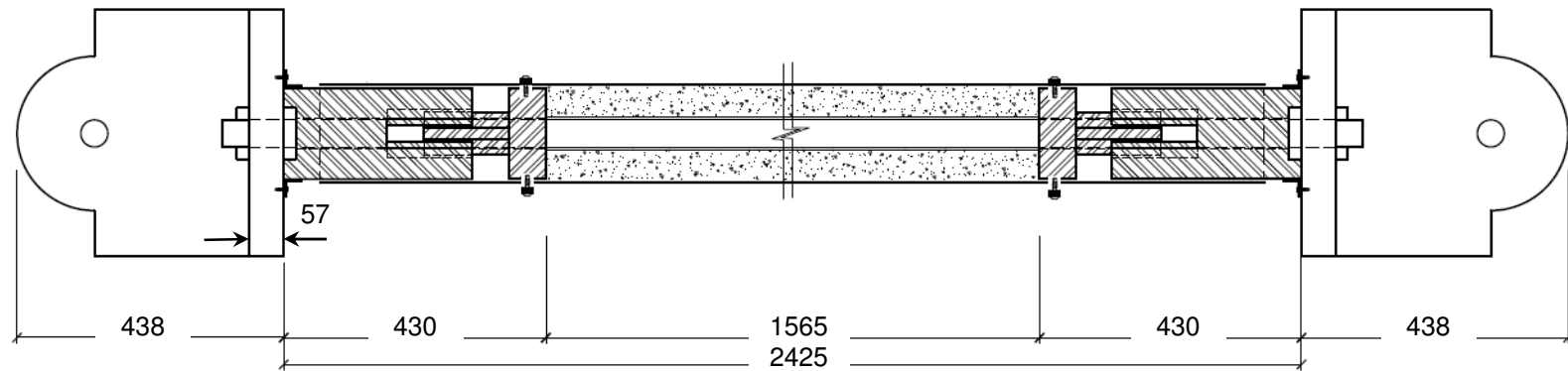
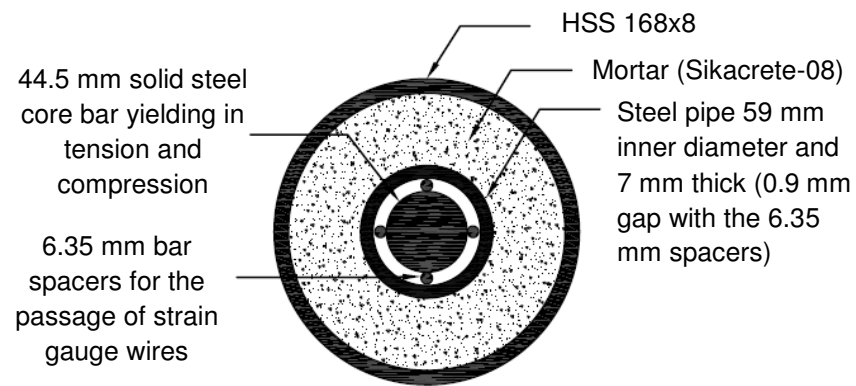


Figure 6.37: Side view of buckling restrained brace showing the steel core bar and end restraining elements  
(All dimensions in mm)



(a)



(b)



(c)

Figure 6.38: BRB system configuration using AISI 12L14 carbon steel core bar:  
a) cross section; b) welding of steel spacers; and c) strain gauge placement on core bar

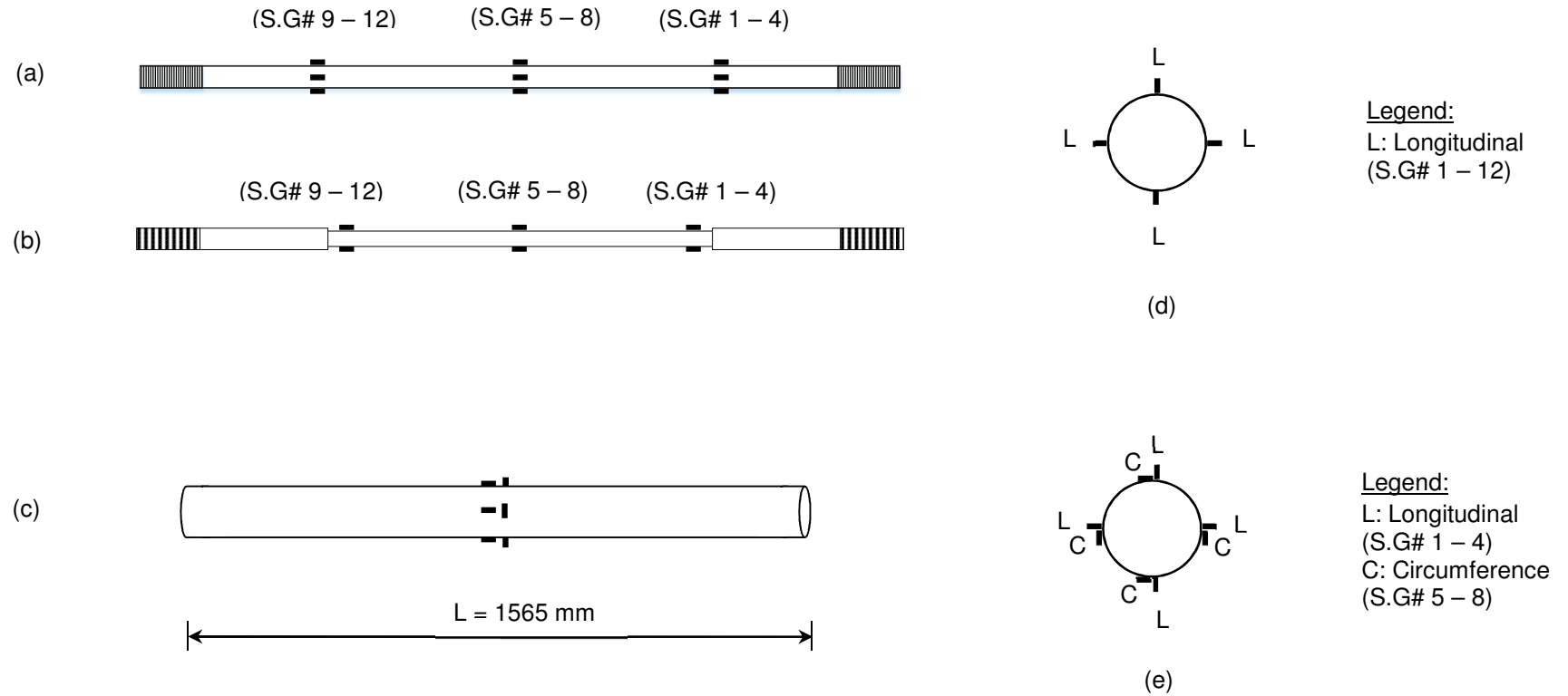


Figure 6.39: Strain gauge locations on the steel core bars and HSS round casing: a) AISI 12L14 carbon steel bar; b) AISI Type 304 stainless steel and AISI 4140 chrome-molybdenum high tensile steel bars; c) HSS round casing; d) cross section of bars at strain gauge locations; and e) cross section of HSS round casing at strain gauge locations





(a)



(b)

Figure 6.40: Preparation of BRB casing and concrete filler: a) top view of HSS round casing and inner pipe; and b) casting of SCC filler material

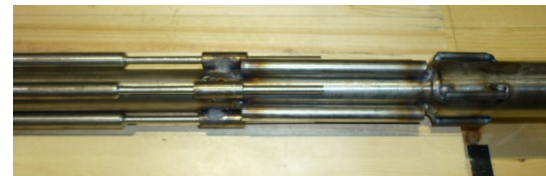
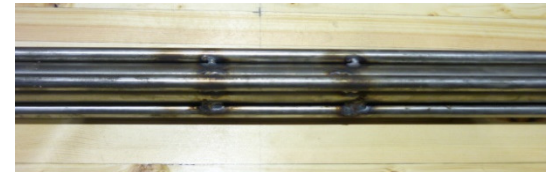
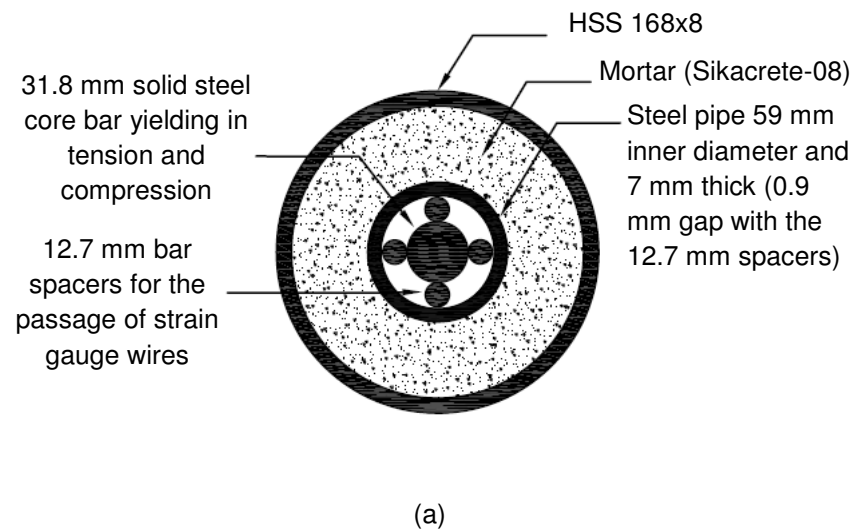
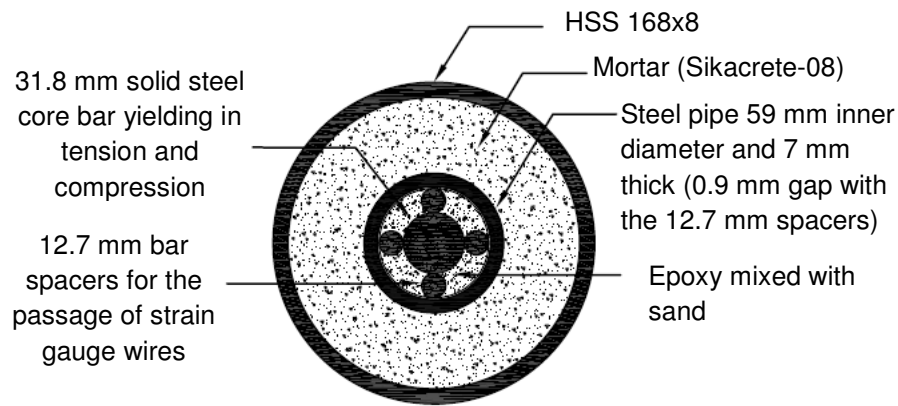


Figure 6.41: BRB system configuration with AISI Type 304 stainless steel core bar: a) cross section; b) reduced section at mid length of bar; c) welding of steel spacers; d) spacer sleeves at left end; e) spacer sleeves at right end; and f) strain gauges placed on core bar





(a)



(b)



(c)



(d)



(e)

Figure 6.42: BRB system configuration with AISI 4140 chrome-molybdenum high tensile steel core bar: a) cross section; b) reduced section at mid length of bar; c) welding of steel spacers and placement of strain gauges on core bar; d) wrapping of adhesive tape; and e) epoxy/sand filler between spacers

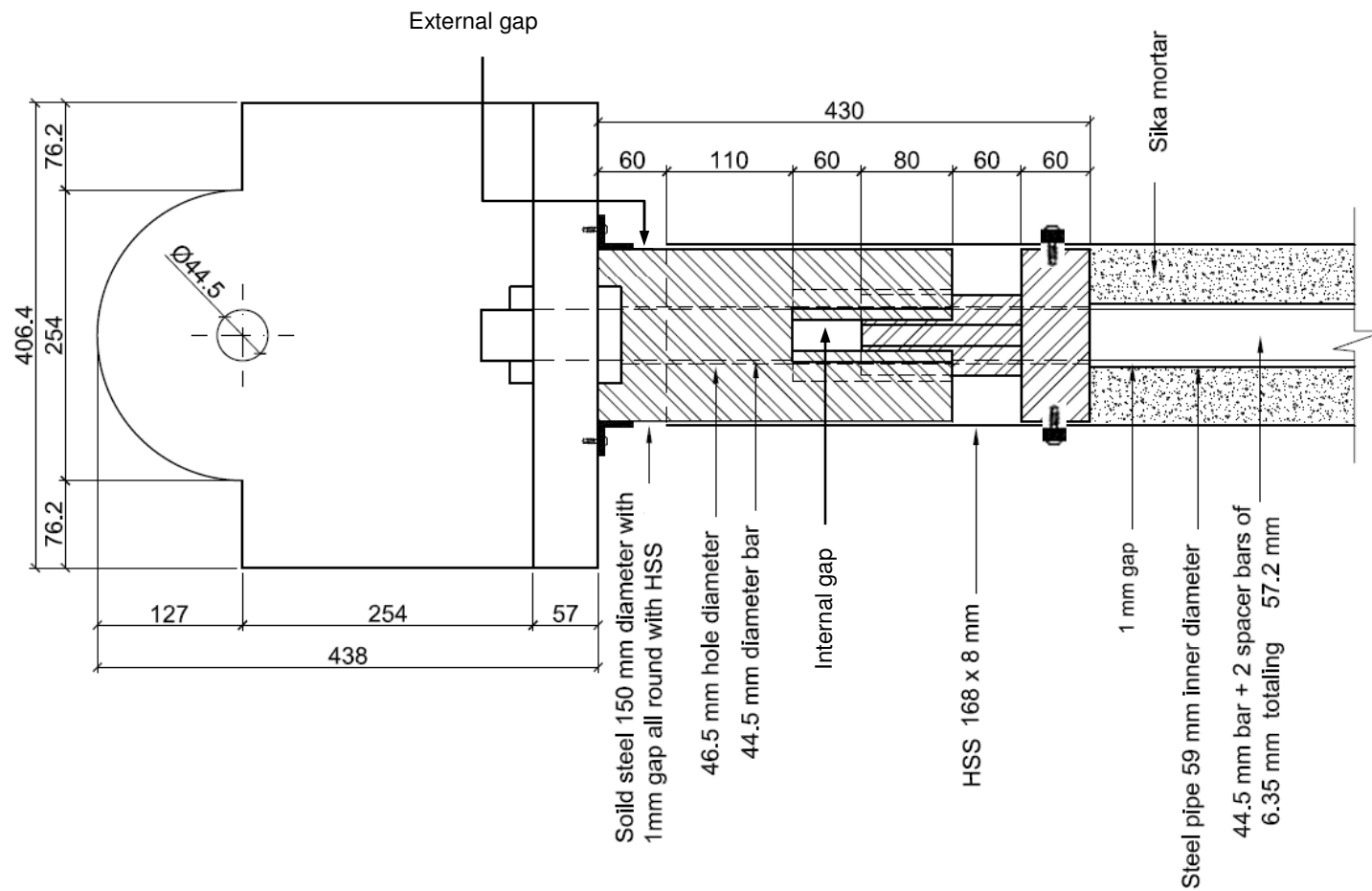


Figure 6.43: Side view of brace steel core restraining end element details (all dimensions in mm)

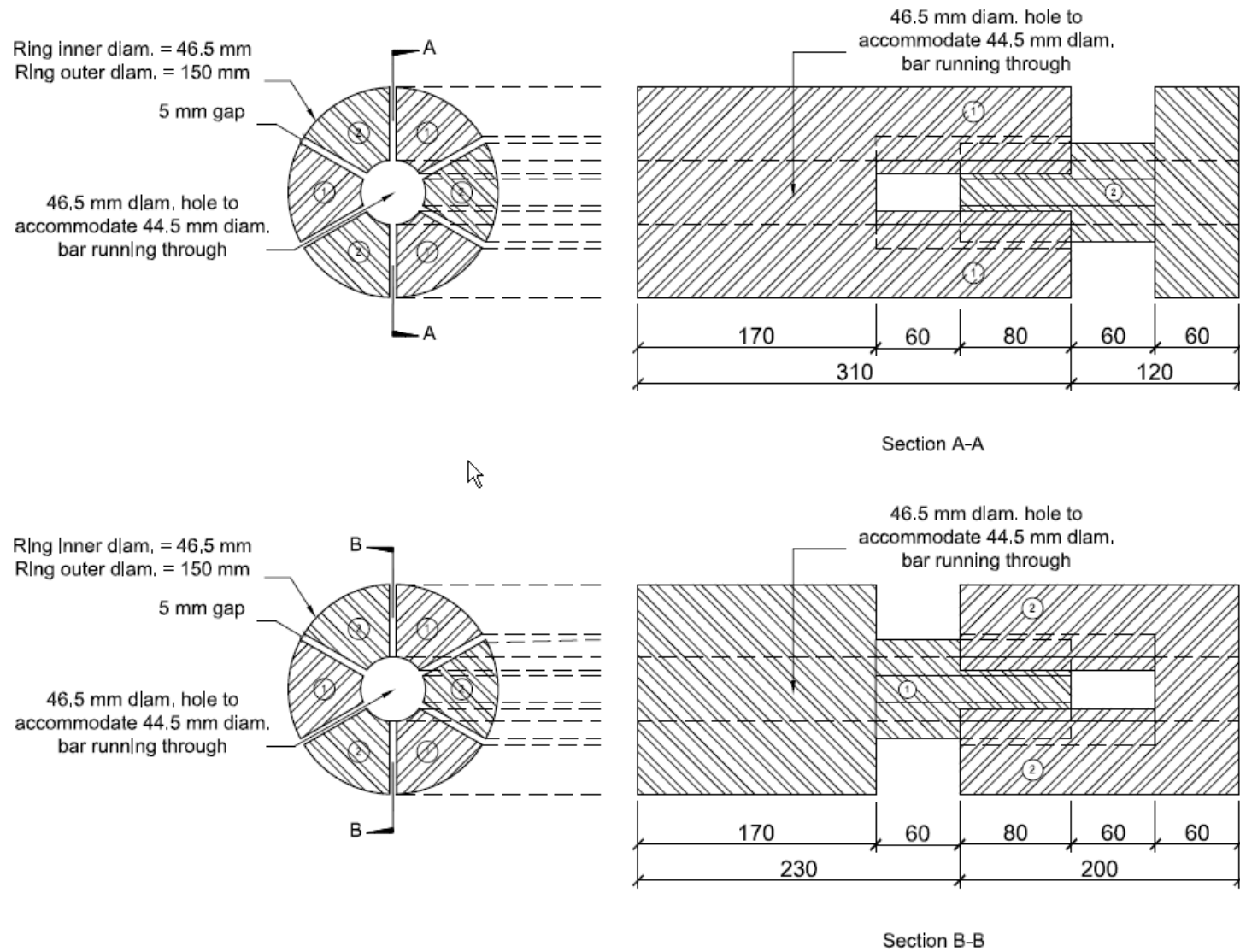


Figure 6.44: Cross sections of steel core restraining end elements (all dimensions in mm)

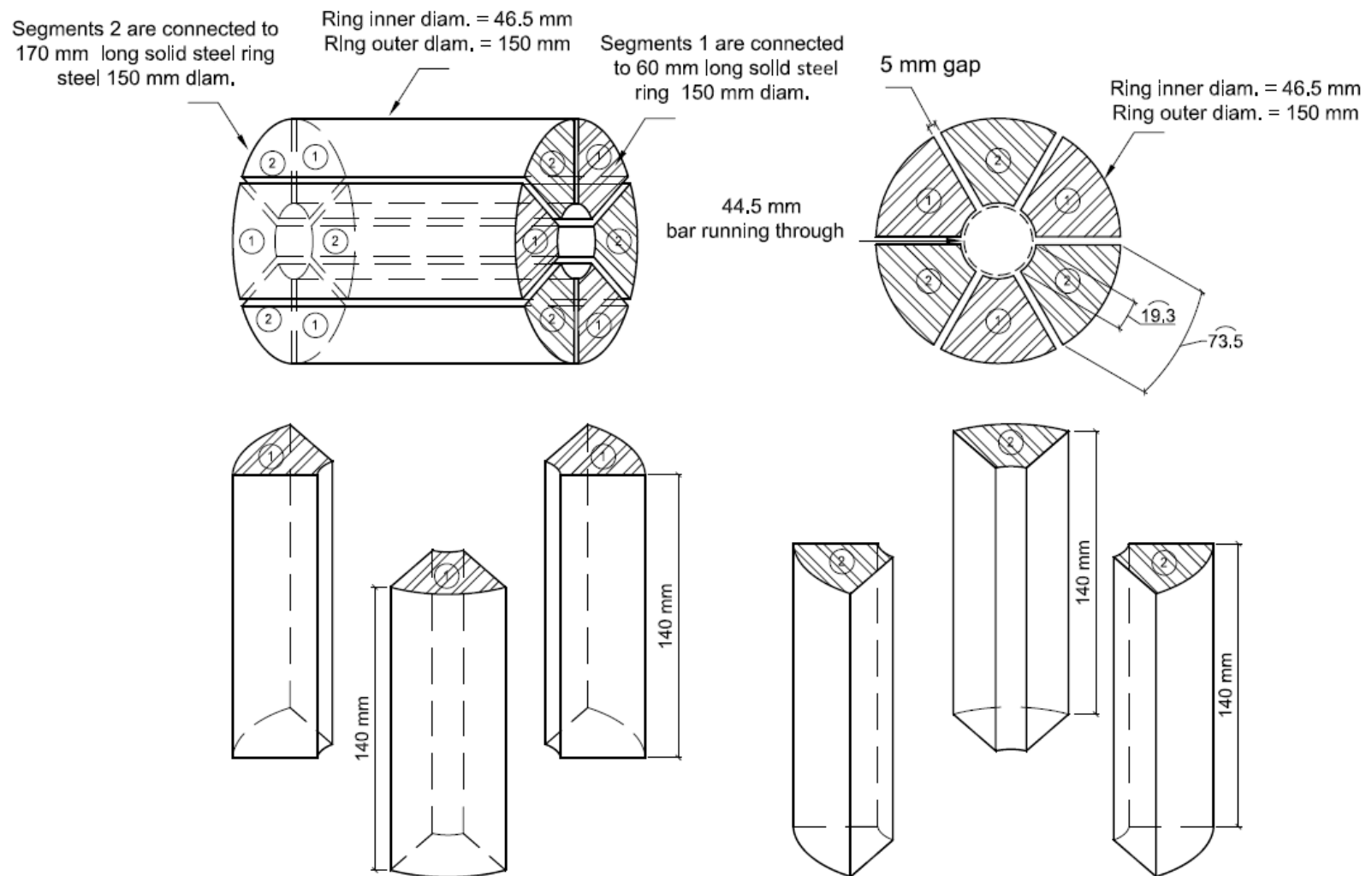


Figure 6.45: Fabrication details of the steel core restraining end elements (all dimensions in mm)



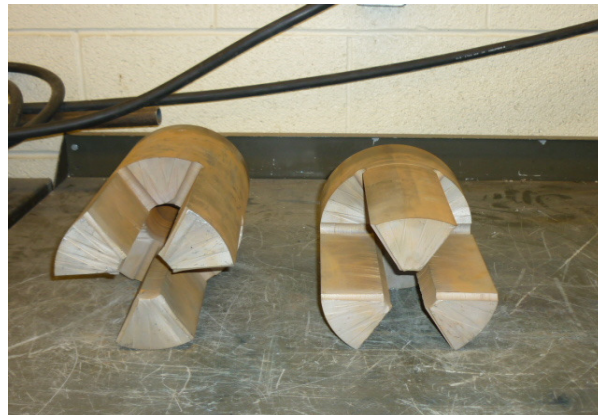
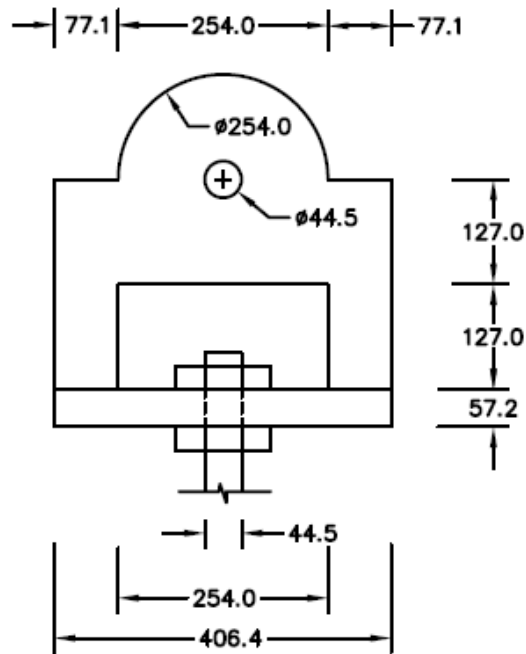


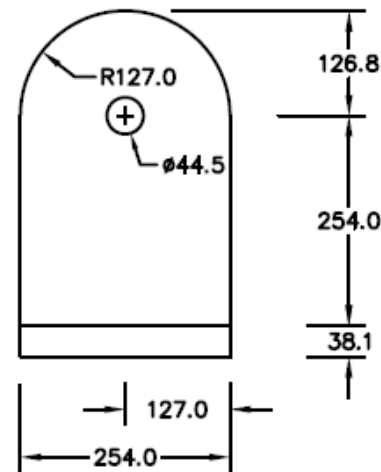
Figure 6.46: Fabricated steel core restraining end elements



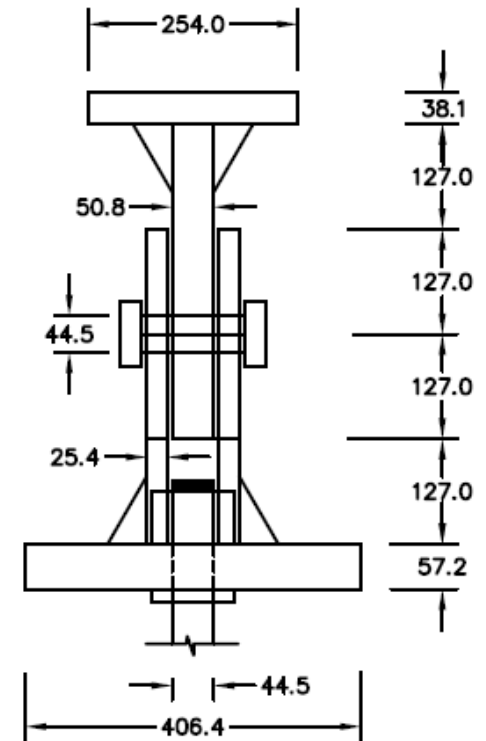
Figure 6.47: Welding of end HSS sections and insertion of steel core restraining end elements



(a)

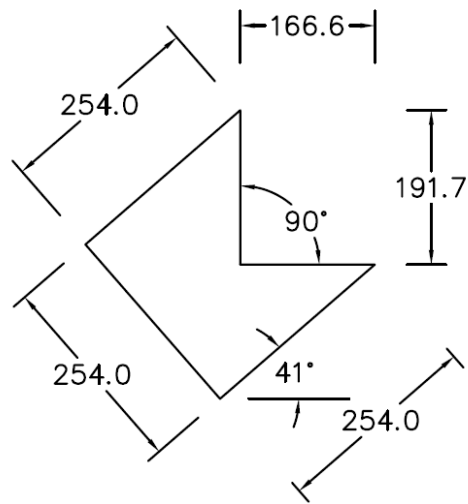


(b)

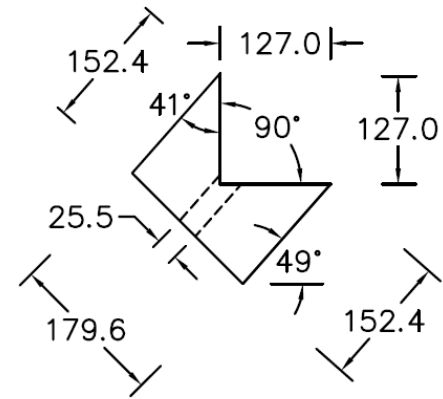


(c)

Figure 6.48: Elevation views of brace hinge connection details: a) hinge component connected to brace steel core bar; b) hinge component connected to concrete frame; and c) assembled hinge connection (all dimensions in mm)



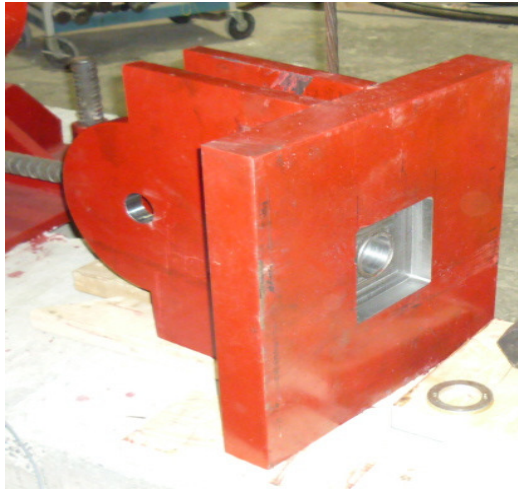
(a)



(b)

Figure 6.49: Steel chair supports details: a) support for brace pin hinge connection; and b) support for dywidag bar (all dimensions in mm)





(a)



(b)

Figure 6.50: Manufactured brace joint hinge components: a) hinge component connected to the brace steel core bar; and b) hinge component connected to the HSS section through three steel chairs

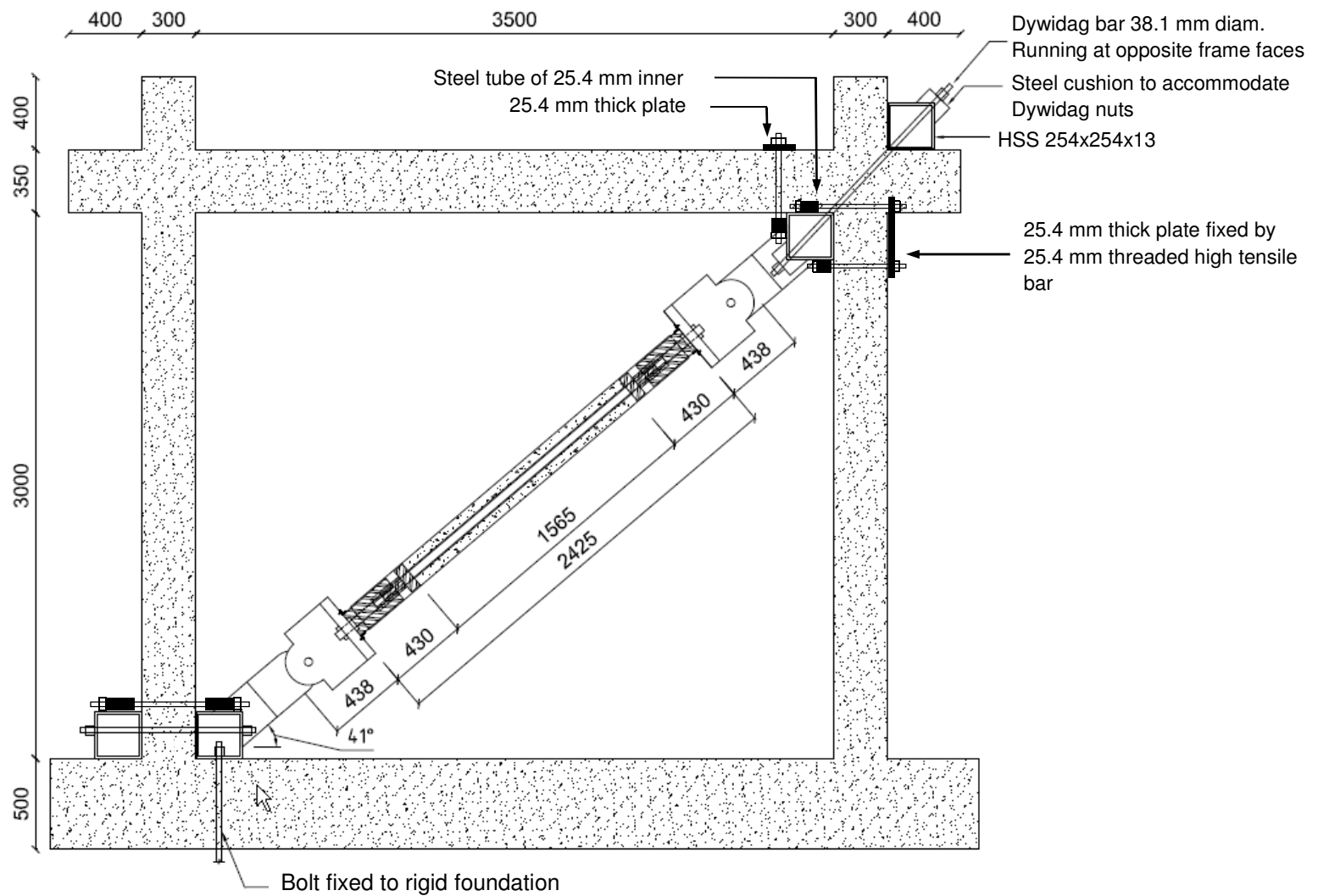
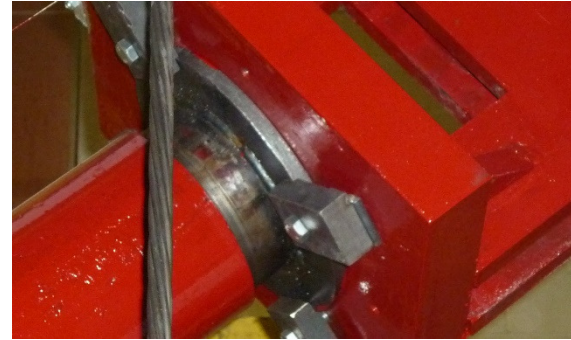


Figure 6.51: Buckling restrained brace configuration within the testing frame (all dimensions in mm)



(a)



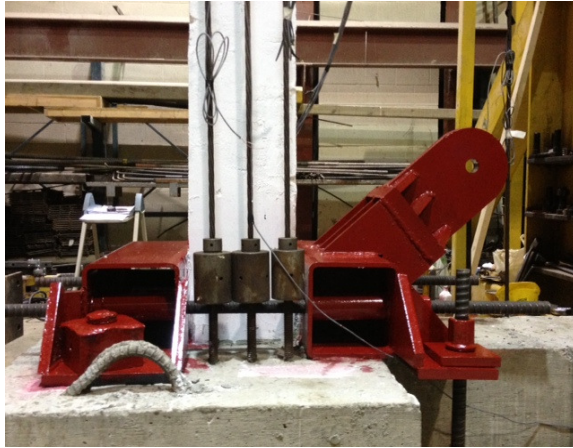
(b)



(c)

Figure 6.52: Brace steel core restraining end elements connected to the joint hinge plates: a) lower hinge plate; b) upper hinge plate; and c) lower and upper hinge plates

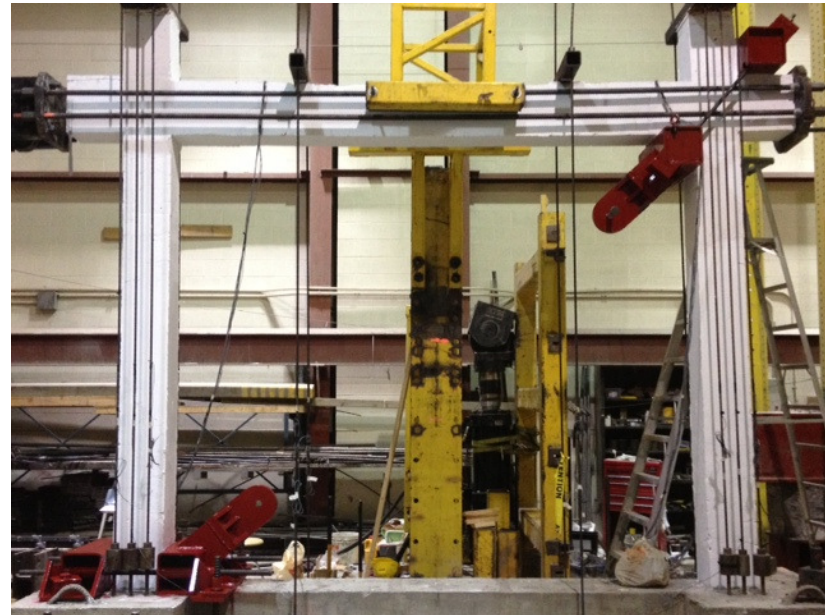




(a)

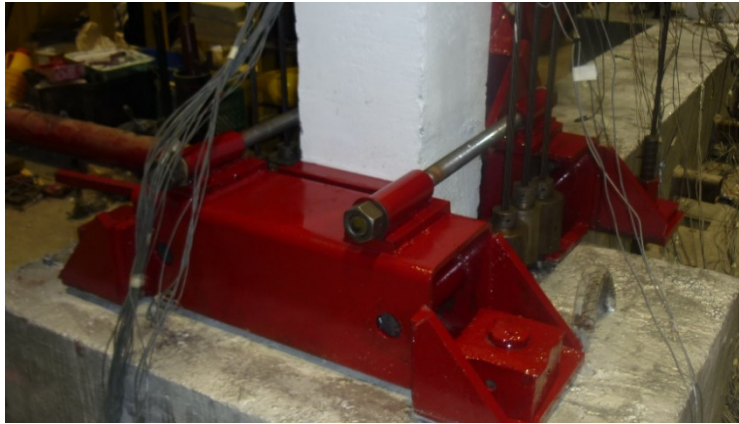


(b)

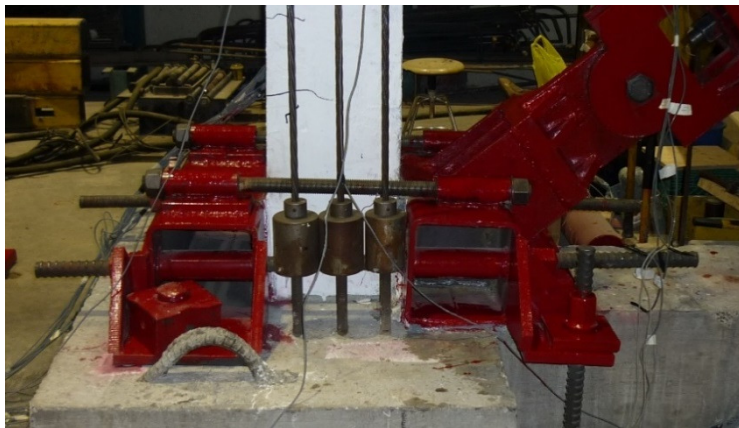


(c)

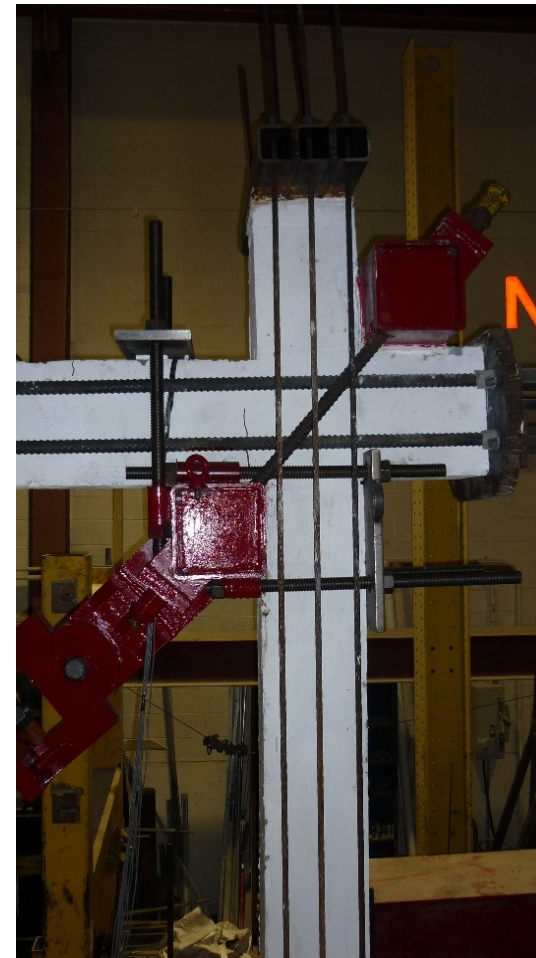
Figure 6.53: Installation of brace joints inside the concrete frame: a) lower joint; b) upper joint; and c) lower and upper joints



(a)



(b)



(c)

Figure 6.54: Stiffening of brace joints to the frame: a) lower joint isometric view; b) lower joint side view; and c) upper joint



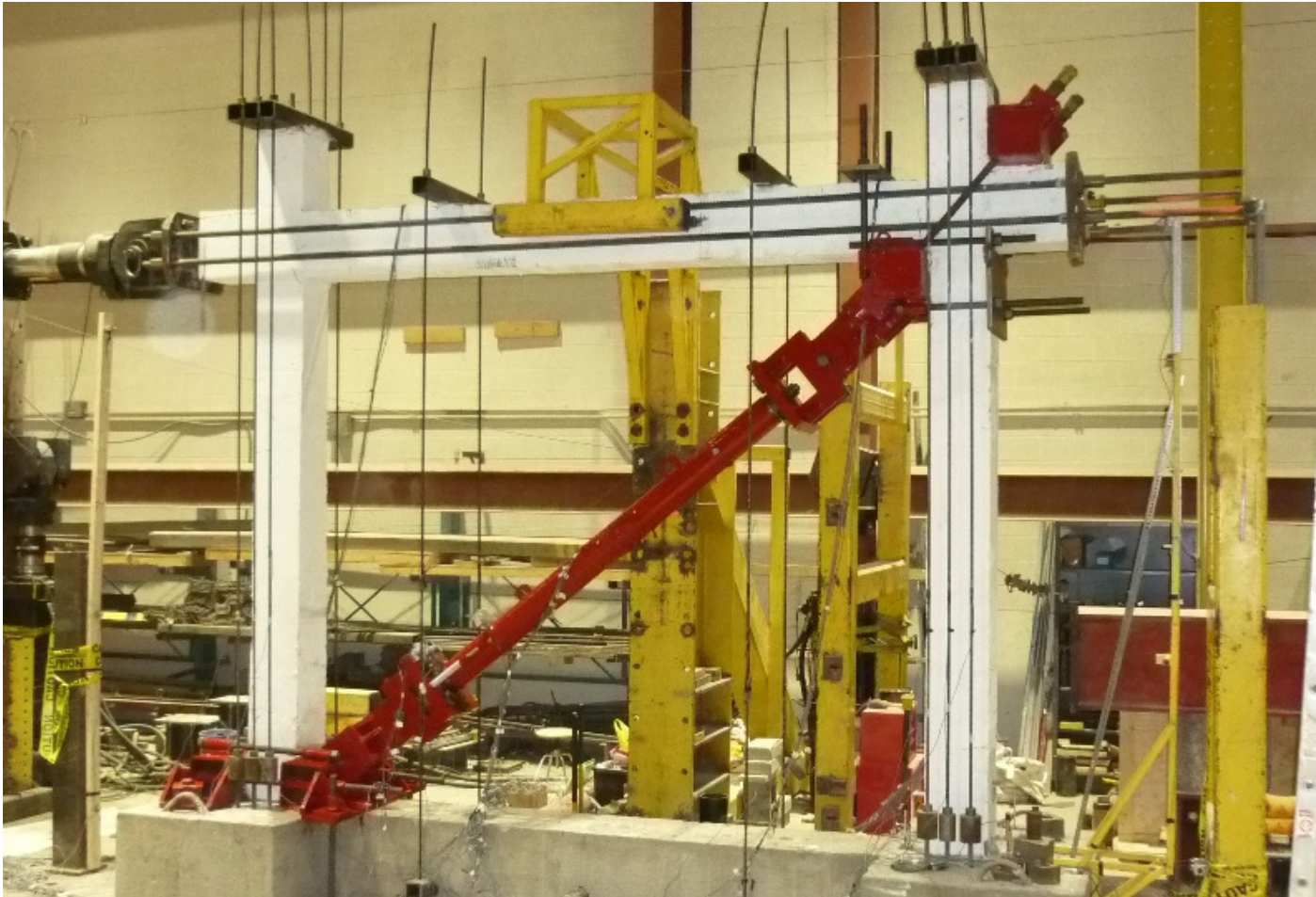


Figure 6.55: Retrofitted frame after installation of the BRB

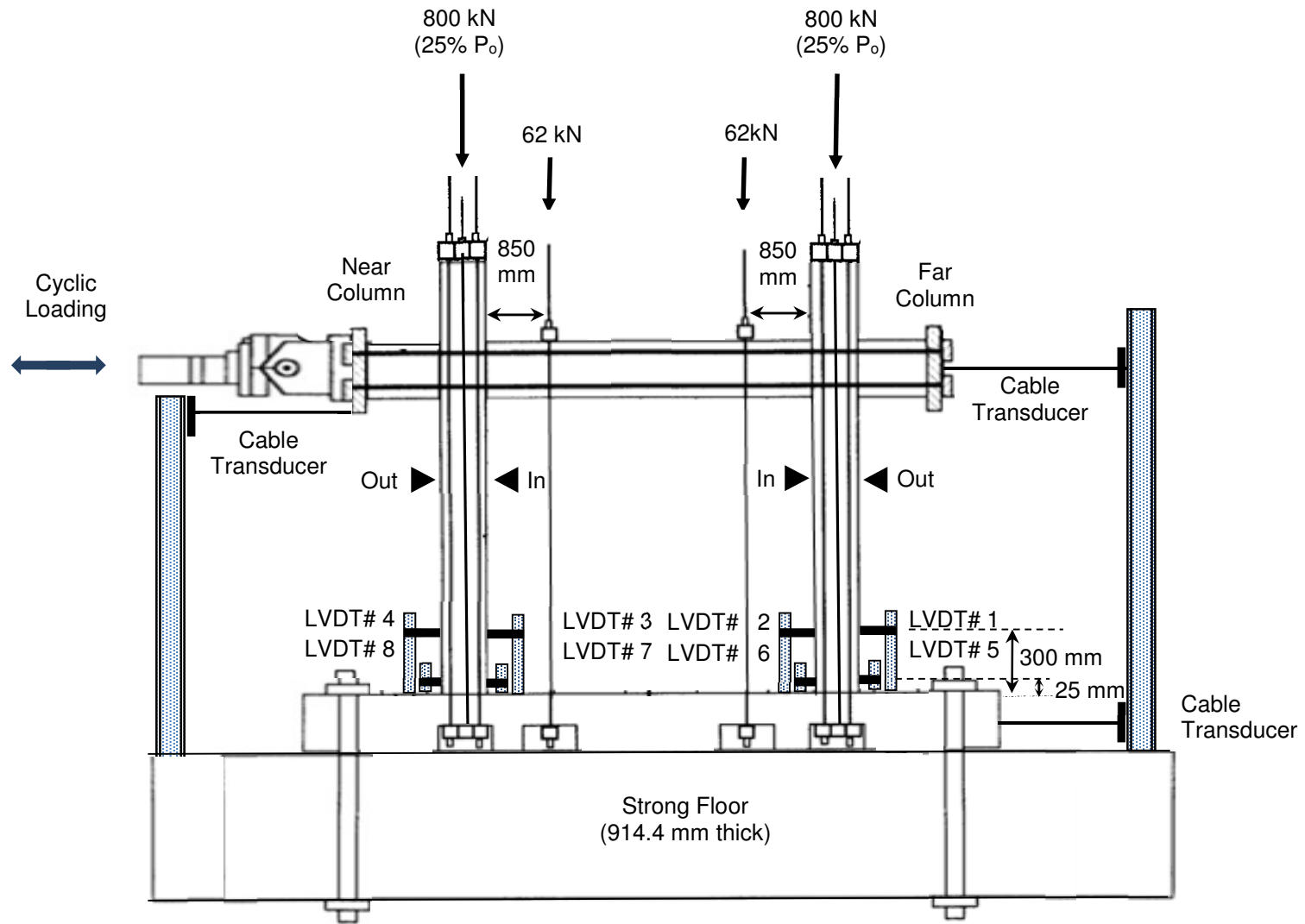


Figure 6.56: Frame loading setup and instrumentations

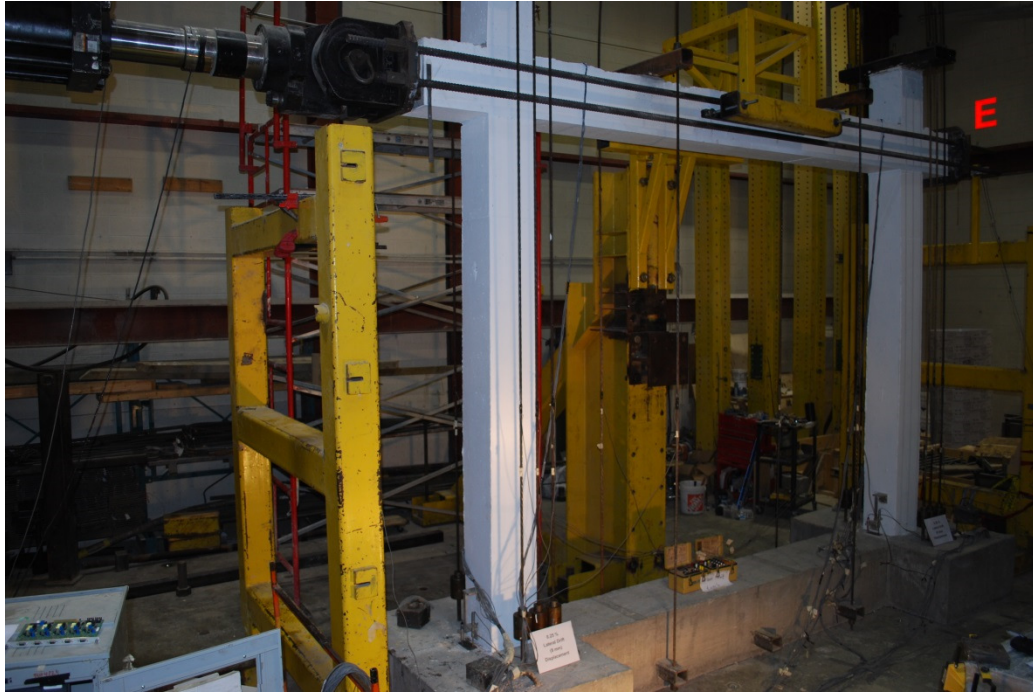
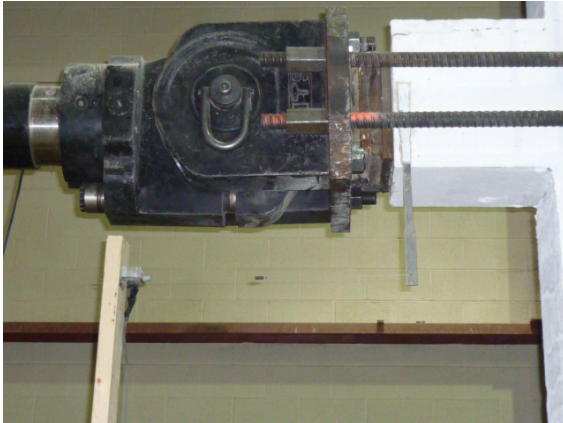
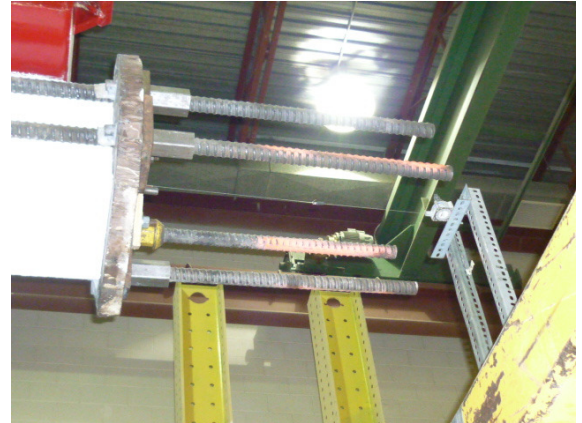


Figure 6.57: Actuator setup and frame before testing





(a)



(b)

Figure 6.58: Displacement cable transducers to measure frame lateral displacements: a) frame near end; and b) frame far end



Figure 6.59: Displacement cable transducers used to measure foundation lateral displacements



Figure 6.60: LVDT's at 25 mm and 300 mm height from foundation level to measure vertical displacements near the base



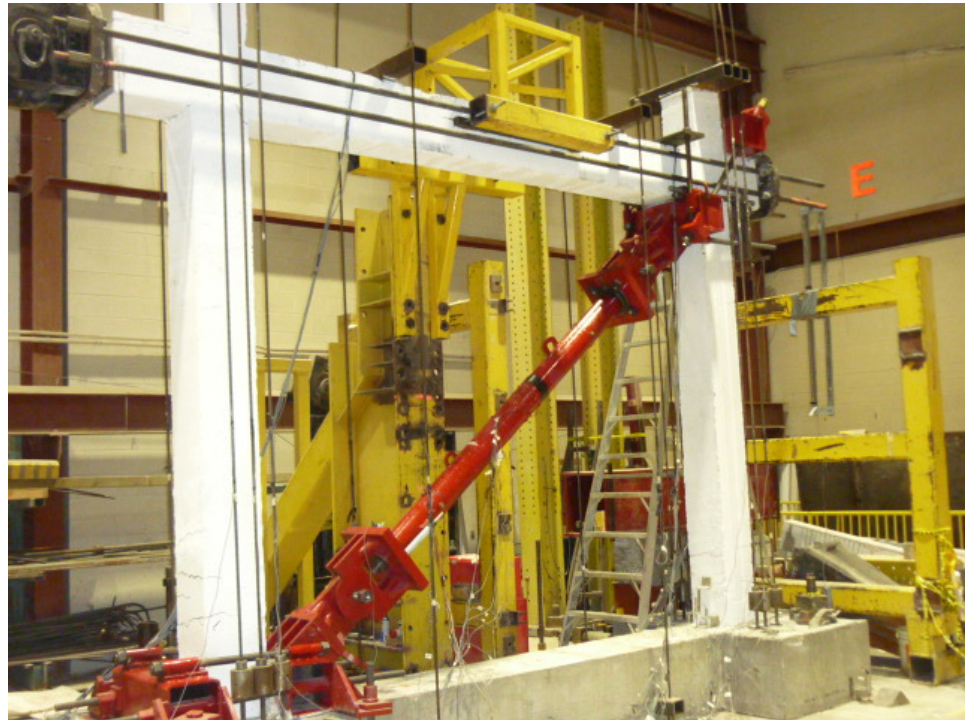
Figure 6.61: LVDT to measure uplift of foundation



(a)



(b)



(c)

Figure 6.62: Displacement cable transducers to measure steel core bar longitudinal displacements for Frame RRF (AISI 12L14 carbon steel core bar) : a) upper hinge; b) lower hinge; and (c) frame before testing



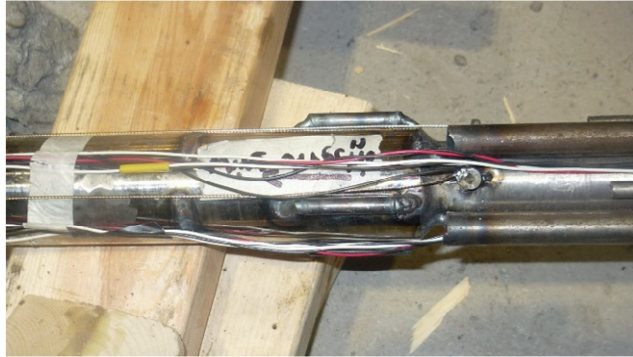


(a)



(b)

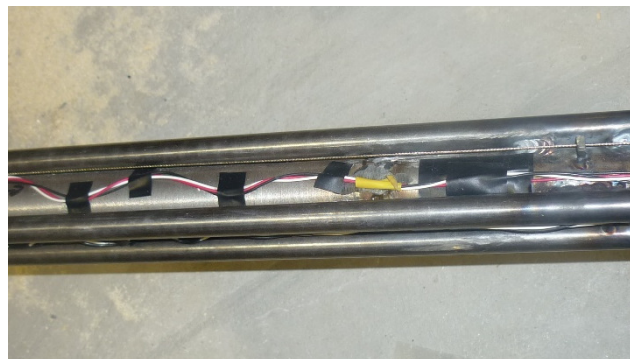
Figure 6.63: Displacement cable transducer to measure frame lateral displacements: a) connection to the test frame; and b) light steel frame support



(a)

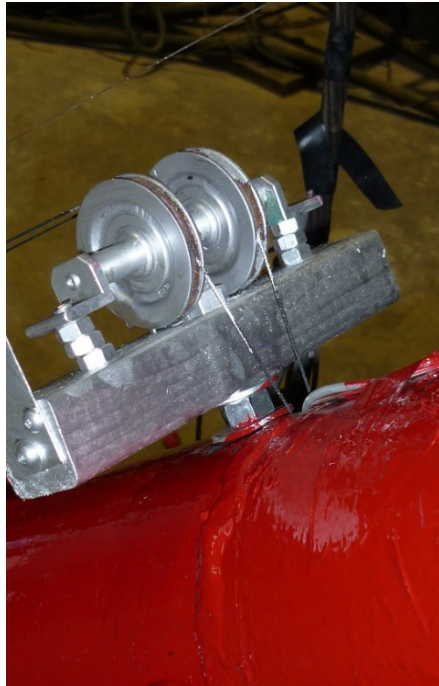


(b)

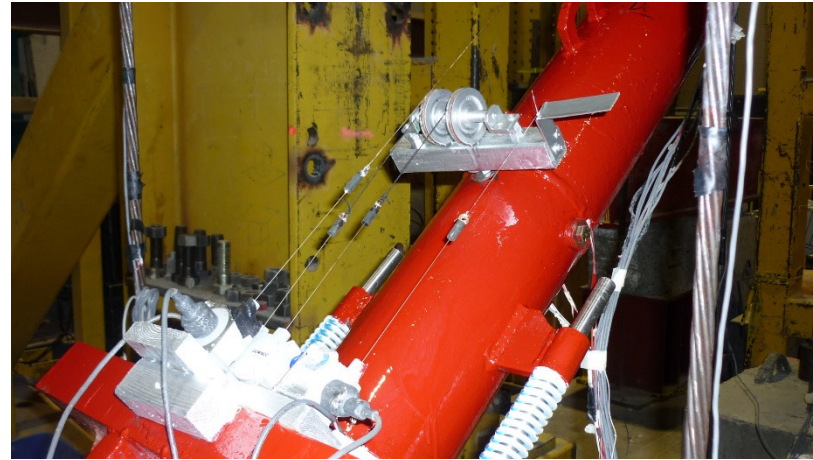


(c)

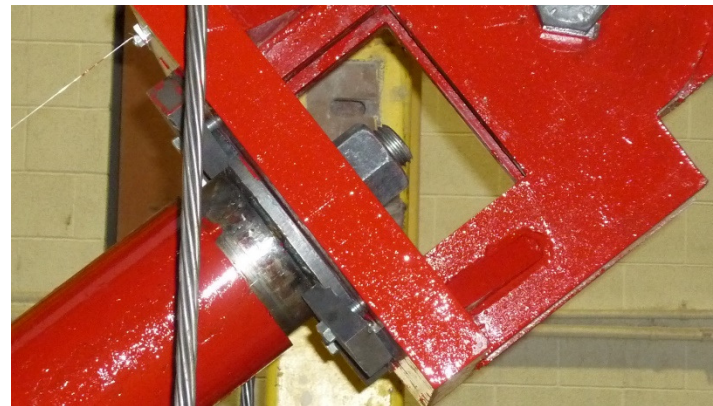
Figure 6.64: Steel wires connected adjacent to the ends of the steel core bar reduced section to measure bar longitudinal displacements in Frame RRF: a) lower reduced section; b) upper reduced section; and (c) middle of reduced section



(a)



(b)



(c)

Figure 6.65: Displacement cable transducers to measure brace bar longitudinal displacements for Frame RRF using AISI Type 304 stainless steel core bar: a) pulleys holding two steel wires connected to the reduced section of the core bar; b) DCT's connected to the lower hinge plate; and c) DCT connected to upper hinge plate



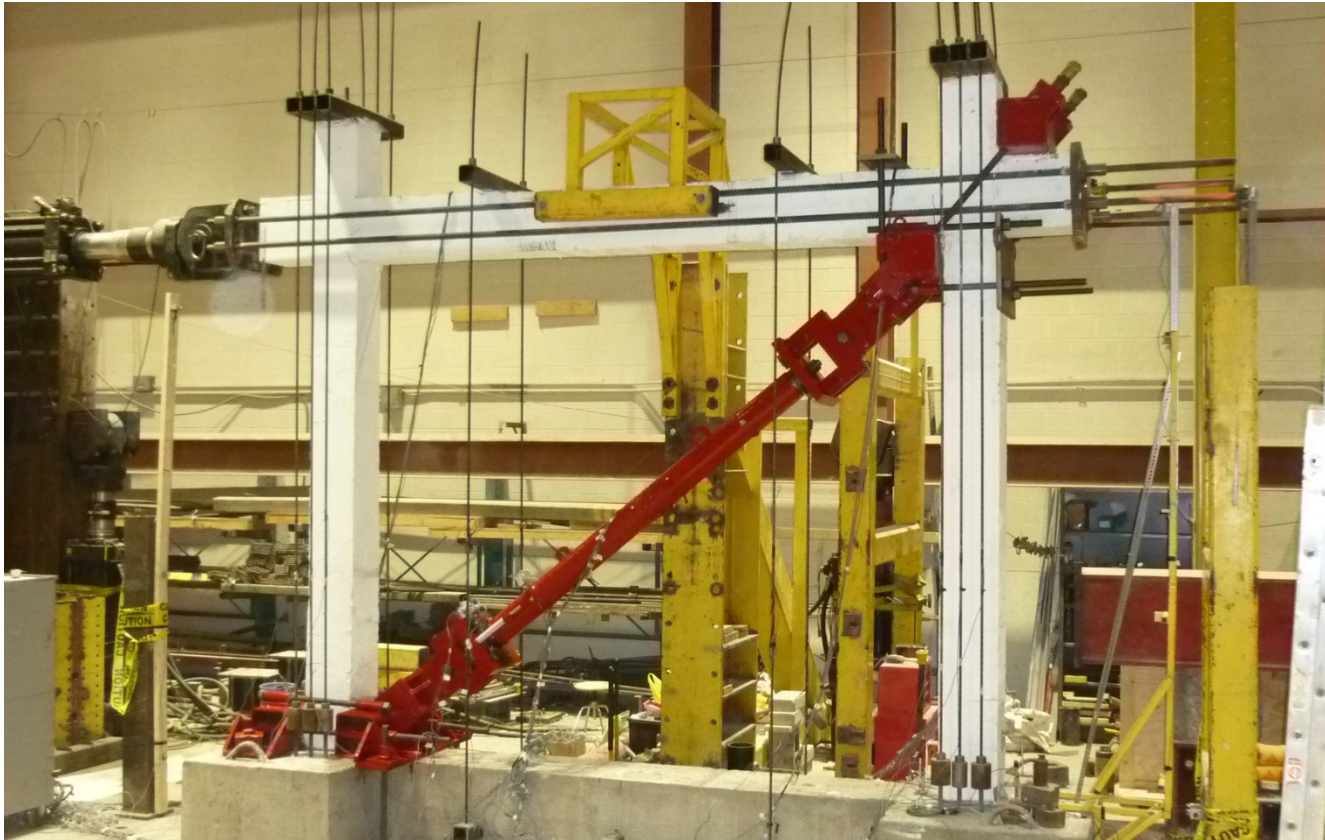


Figure 6.66: Frame RRF using AISI Type 304 stainless steel core bar before testing



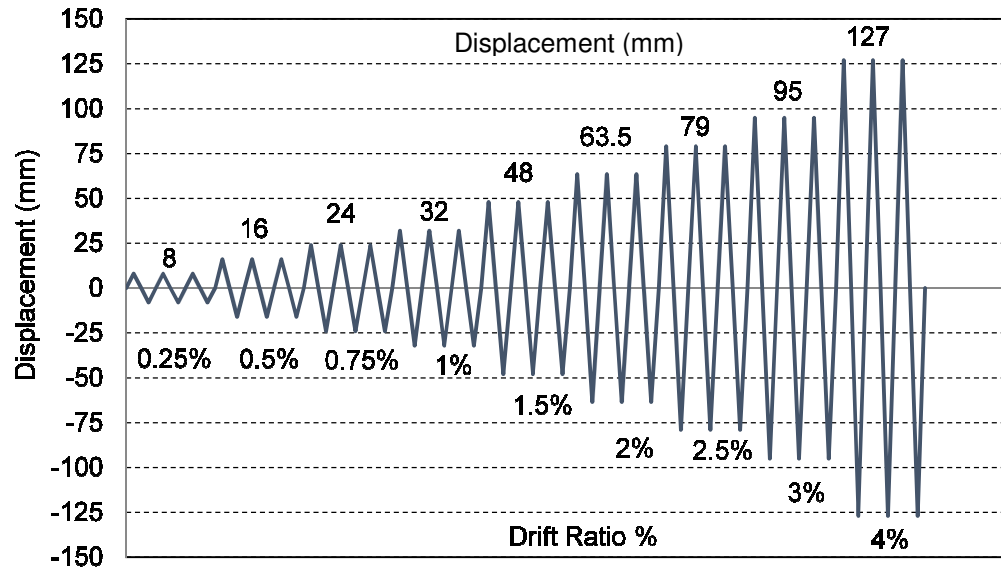


Figure 6.67: Loading protocol of the bare control frame

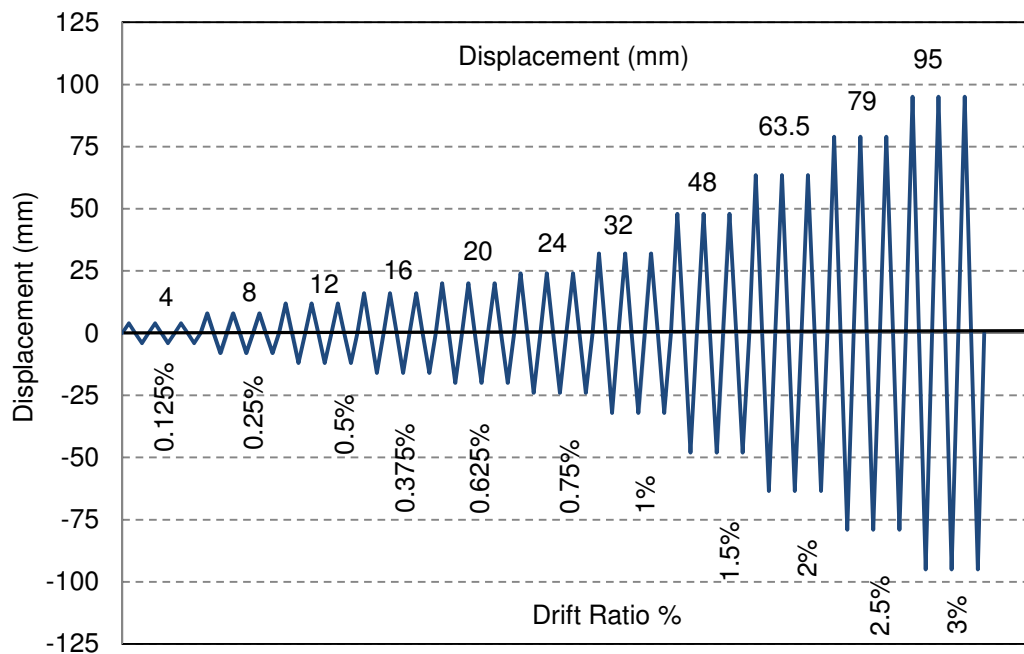


Figure 6.68: Loading protocol of the retrofitted frames

## Chapter 7

### Experimental Test Results and Discussion

#### 7.1 Introduction

This chapter includes a discussion of observations and recorded data during testing of the seismically deficient reinforced concrete bare control frame (BCF), repaired and retrofitted frame (RRF), and non-damaged retrofitted frame (RF). The results are presented in the form of a table and figures and discussed at the end of the chapter. The results of Frame BCF are discussed and explained with reference to frame behaviour, while results of the retrofitted frames are explained with reference to frame and retrofitting system behaviours, all at each test drift ratios. Instrumentation, including those placed on the BRB system, consisted of strain gauges, linear variable displacement transducers (LVDTs), and displacement cable transducers (DCTs). The data recorded are discussed in this chapter. Additional information is provided in Appendix VI.

The lateral force resistance of the frames recorded during testing included the component provided by the concrete frame and the retrofit system, as well as the horizontal component of the prestressing strands that were used to impose gravity loading. The experimental load-displacement hysteretic relationships were adjusted to include the contribution of the frame and the retrofit system only.

#### 7.2 Experimental Results of the Bare Control Frame (BCF)

Frame BCF was designed and built based on the 1965 NBCC. The test served as a reference for the retrofit methodology developed herein for damaged and non-damaged frames. Figure 7.1 illustrates the experimental hysteretic lateral load-lateral displacement relationship for the bare control frame (3 cycles for each drift level). The hysteretic behaviour reveals that the frame experienced a gradual decrease of lateral stiffness during the elastic range of loading due to the cracking of concrete. Yielding initiated at a drift ratio of 1.3% (41 mm). The frame attained its maximum lateral strength of 233 kN and 219 kN at 2.5% drift ratio during the first push and pull cycles, respectively. As the drift increased to 3% (41 mm), the lateral strengths of 219 kN and 205 kN were recorded

during the first push and pull cycles, respectively. During the 3<sup>rd</sup> cycle, lateral resistances of the frame dropped to 82% and 81% of the peak resistances in the push and pull cycles, respectively. At 4% drift, the lateral resistance recorded during the first cycle in the push and pull loadings were 78% and 81%, and during the 3<sup>rd</sup> cycle they were 60% and 53% of the peak resistance, respectively. Therefore, the frame was considered to have reached its maximum drift capacity of 3% in both push and pull modes, based on a drop in lateral load capacity exceeding 20% of the maximum lateral resistance. The experimental results were in agreement with the analytical sectional calculations both in terms of predicted sequence of damage and member capacity (Table 6.4 of Chapter 6). The bare control frame sequence of damage with reference to the lateral load and drifts is provided in Table 7.1. Both columns reached their flexural capacities at the base at a drift limit of 0.75%; while the far column, at the beam-column joint, reached its flexural capacity at 1% drift. Subsequently, the flexural capacities of the beam near the supports and of the near column adjacent to the beam-column joint were reached at 1.5% and 2.5% drift, respectively.

Plots of the experimental data recorded by instrumentation (Section 6.10.1 of Chapter 6) are available in Appendix VI Sec. (A). This includes: strain gauge readings from the steel reinforcement and the lateral force and displacements experienced by the frame. In addition, vertical displacements of the column within the plastic hinge region recorded by LVDT No. 1-4 (Figure 6.56) and base anchorage slip of the column reinforcement captured by LVDT No. 5-8 (Figure 6.56) are also included. The LVDT readings were either in tension or in compression depending on the lateral force push/pull modes. Maximum tensile straining recorded by these LVDTs are discussed hereafter. The observations made during testing, in terms of crack patterns and recorded data are discussed in the following paragraphs.

The formation of cracks were noticed at 0.25% drift ratio. Cracks initiated in the beam and more noticeably at regions near the applied gravity loading. No visible flexural cracks were evident on the column far from the actuator; however, few cracks surfaced at the base of the near column. The cracking is illustrated in Figure 7.2 corresponding to a lateral displacement of 8 mm and a lateral load capacity of 73 kN. The maximum base anchorage slip and the vertical displacement of the columns were 0.16 mm and 0.35 mm, respectively.

The flexural cracks further propagated at 0.5% drift ratio (16 mm) along the entire length of the beam from the soffit up to the mid-depth of the section. Shear cracking initiated at this drift level near the ends of the beam. Moreover, flexural cracks began to surface at the base of the far column in the plastic hinge region, and additional cracking became visible near the base of the near column as illustrated in Figure 7.3. The lateral load capacity at this drift level was 116 kN. The maximum base anchorage slip and vertical displacement of the columns were 0.41 mm and 0.8 mm, respectively.

As the drift ratio increased to 0.75% (24 mm), shear cracks surfaced in beam-column joints. The shear and flexural cracks widened along the length of the beam and throughout the beam depth. In addition, the bottom reinforcing bars in the beam began yielding at the far end of the beam adjacent to the beam-column joint according to strain gauges S.G. #25 and #26. The reinforcing starter bars at the base of both columns were also yielding as recorded by strain gauges S.G. #6 and #11. Cracking propagated from the base of the column up to a height of 900 mm from the foundation level. For this drift level, the peak lateral load resistance was 151 kN, and the maximum base anchorage slip and vertical displacement of the columns were 0.62 mm and 1.12 mm, respectively. The observed cracking at 0.75% drift is shown in Figure 7.4.

At 1% drift ratio (32 mm) an increase in shear cracks at beam-column joints and widening of flexural and shear cracks in the beam was observed. The columns near the beam-column joints developed the first signs of flexural cracking. Furthermore, the flexural capacity of the far column near the beam soffit was realized. The maximum base anchorage slip and vertical displacement of the columns were 0.86 mm and 1.5 mm, respectively. The starter bars of both columns yielded at a height of 25 mm and 125 mm from the foundation level. The latter was confirmed by the readings of S.G. #1 at the far column and S.G. #12 at the near column. Flexural cracking at the bottom of both columns surfaced to a height of 1.0 m from the foundation level. The lateral load capacity at this drift was 175 kN. Figures 7.5 and 7.6 illustrate the crack patterns and damage of the frame at this drift level.

Based on the hysteretic lateral load-lateral displacement response, the global yielding of the frame developed at a drift ratio of 1.3% (41 mm) which corresponded to a lateral load resistance of 200 kN as shown in Figure 7.1. This yield displacement was calculated

based on the equivalent elasto-plastic system with a secant stiffness passing through the lateral load-displacement response at 75% of the ultimate lateral load. This is based on the definition of yielding for reinforced concrete members established by Park (1988).

As the imposed drift increased to 1.5% (48 mm), the beam reached its flexural capacity at both supports and for both positive and negative bending. Additional flexural and shear cracking was visible at these locations. Yielding was confirmed by examining the readings of strain gauge S.G. #29 and #30 located on the beam longitudinal reinforcement near the beam-column joint. Flexural cracking in the columns was visible to a distance of 700 mm from the beam soffit and to a height of 1.2 m from the foundation level as shown in Figure 7.7. The column starter bars continued to yield at 125 mm above the foundation level as recorded by S.G. #7 and #17. The maximum base anchorage slip and the vertical displacement of the columns were 1.78 mm and 2.2 mm, respectively. This level of drift corresponded to a lateral load 208 kN. The cracking patterns of the frame at this drift level are shown in Figure 7.8.

At 2.0% drift ratio (63.5 mm), additional shear cracks became visible in the beam and in the beam-column joints as illustrated in Figure 7.9. At this drift level, and at the column-foundation interfaces, vertical cracks became visible due to yield penetration of the reinforcement extending into the foundation and the initiation of anchorage slip of reinforcing bars. The maximum base anchorage slip and vertical displacement of the columns were 3.7 mm and 2.5 mm, respectively. The lateral strength capacity of the frame at this drift was 226 kN. The corresponding observed frame cracking is shown in Figure 7.9.

The peak lateral resistance of Frame BCF was attained at 2.5% drift (79 mm) corresponding to 233 kN and 219 kN in push and pull cycles, respectively. Damage in the form of concrete cover spalling became visible at this drift level at the near column adjacent to the beam soffit. This is attributed to the flexural capacity being exceeded in this region. However, no exposure of the main reinforcing steel bars or stirrups was evident. Shear and flexural cracking increased along the beam and in the beam-column joints as illustrated in Figures 7.10 and 7.11. All the individual frame members exceeded their flexural capacities at this drift level. The far column starter bars experienced yielding at a height of 250 mm from the foundation level according to S.G. #8, as well as

the middle longitudinal bar near the beam-column joint as recorded by S.G. #21. The maximum base anchorage slip and vertical displacement of the columns were 6.8 mm and 3.4 mm, respectively.

As the drift ratio increased to 3% (95 mm), significant concrete spalling at the base of both columns was observed which resulted in exposure of the column reinforcement as illustrated in Figure 7.12. As a result of the bars being exposed, LVDT # 5, 6, 7, and 8 (see Figure 6.56), located on the column 25 mm above the foundation, were removed. Shear tension cracks extended and widened at the critical regions of the beam, as well as at the beam-column joints. This resulted in significant softening in the frame response accompanied with inelastic energy dissipation. The hysteretic response indicated that the frame, at this drift level, experienced strength decay and continuous stiffness degradation. The maximum lateral load capacity attained was 219 kN and 206 kN in push and pull cycles, respectively. The maximum base anchorage slip and vertical displacement of the columns were 11.6 mm and 5.2 mm, respectively. Figure 7.13 shows the cracking pattern of the frame at this drift.

The frame deteriorated and was deemed to have failed at 4.0% drift during the first push and second pull cycles as the lateral resistances were 182 kN and 156 kN, which corresponded to 78% and 71% of the peak lateral strength capacity, respectively. At this drift ratio, the frame was considered to have exceeded its maximum drift capacities, based on a drop in lateral load capacity exceeding 20% of the maximum. Therefore, the frame is considered to have 3% drift capacities in both push and pull modes. The longitudinal reinforcing bars of the columns and the beam near the supports were exposed after significant spalling of concrete cover as illustrated in Figures 7.14 and 7.15. Moreover, the bottom longitudinal bars in the beam experienced permanent buckling deformations. At 3% drift, the frame attained a ductility of 2.3. This was based on the initiation of yielding coinciding with 1.3% drift as determined from the hysteretic response. The hinges that formed at the ends of the beam provided energy dissipation. This frame which was designed according to earlier code provisions was expected to sustain a smaller drift capacity at failure. The expected behaviour for frames designed to earlier codes is the “weak column-strong beam” mechanism. However, this is true only if the beam moment capacity ( $M_b$ ) exceeds the flexural capacities of the connecting columns ( $M_c$ ). For this frame  $M_b/M_c = 160/165 = 0.97$  which verifies the formation of

beam hinges prior to any column hinges as observed during testing. The maximum base anchorage slip and vertical displacement in the columns were 14.0 mm and 10.3 mm, respectively.

### **7.3 Experimental Results for the Retrofitted Frames**

The experimental data of the retrofitted frames includes the results of three tests: two tests on the repaired bare control frame; referred to as RRF, that incorporated two BRB steel core bars (AISI 12L14 carbon and AISI Type 304 stainless steel); and a third test on a non-damaged frame, referred to as RF, with an AISI 4140 chrome-molybdenum high tensile steel core bar within the BRB system.

The lateral force resistance of these retrofitted frames consisted of the component provided by the frame and the component provided by the BRB retrofitting system. Therefore, the hysteretic response of these frames were plotted with reference to the lateral resistance of Frame BCF (dotted curves in Figure 7.17, Figure 7.36, and Figure 7.62). Note that there was very good agreement between the response of Frame BCF and the lateral resistance of the retrofitted frames after excluding the BRB steel core contribution. The forces sustained by the BRB steel core bars were determined from the strains recorded by the strain gauges mounted on the steel core bars within their strain limits; otherwise, the forces were calculated from the axial deformations of the BRB steel core bar captured by the DCT's. Yielding of these composite structures (concrete frames and steel retrofitting bars) were established graphically by intersecting two lines on the envelope of the hysteretic response: one parallel to the initial stiffness of the composite structure and the other parallel to the ascending curve of the hysteretic response. This is believed to be the most reliable method for assessing yielding of such composite structures since the hysteretic response curves were rounded and did not exhibit a well-defined yield plateau. As stated in Table 6.5 of Chapter 6, fracture of the steel core brace bars in tension was expected to control the response of the three tests on the retrofitted frames.

The instrumentation data of the experimental test results (Section 6.10.2 of Chapter 6) are available in Appendix VI. Section VI.B provides data for the two tests conducted on Frame RRF; while Section VI.C provides data for Frame RF. Some strain gauges located on the internal steel reinforcement in the frames were defective and only those



that were functional are presented in the appendices.

A description of these test results are discussed in the following sections.

### **7.3.1 RRF Test Using AISI 12L14 Carbon Steel Core Bar**

Frame RRF was tested with AISI 12L14 carbon steel core bar (Figure 6.35 (a) and Figure 6.38) according to the BRB configuration shown in Figure 6.53. The retrofitted frame was tested under cyclic loading until 1.5 % lateral drift. At this stage of loading it was noticed that the recorded displacement of the BRB was approximately half of what was expected based on the geometry of the deformed shape of the frame. Therefore, the test was stopped and the observed looseness in the hinge connections was removed by tightening the end hinges. The subsequent testing resulted in the rupturing of the BRB core bar during the initial cycle, hence giving a more stiff response (due to the tighter hinge connections) and a monotonically increasing force-displacement relationship. The experimental hysteretic lateral load-lateral displacement response (3 cycles for each drift level), as well as the monotonic curve of the second test are illustrated in Figure 7.17.

The hysteretic response under cyclic loading demonstrated rigid elastic behaviour until the onset of yielding of the BRB steel core at 20 mm lateral displacement (drift ratio of 0.625%), showing softer response thereafter. The second softening was also observed at a point corresponding to a lateral displacement of 40 mm (drift ratio of 1.25%) caused by the initiation of frame yielding. Both brace core bar yielding as well as frame yielding resulted in the softening of the global system behaviour. Lateral strength capacities of 309 kN and 525 kN were recorded during the first push cycles at 20 mm and 40 mm lateral displacements, respectively. Testing continued until a lateral displacement of 48 mm (1.5% drift), which corresponded to lateral resistances of 578 kN and 621 kN in push and pull modes, respectively. The subsequent testing after the tightening of the hinge joints aimed at achieving the previous maximum frame displacement of 48 mm. However, it ruptured at a displacement of 36 mm (1.13% drift) and a corresponding lateral strength of 577 kN. This monotonic loading behaviour demonstrated increased elastic stiffness as well as earlier initiation of yielding at a lateral displacement of 16 mm (0.5% drift), which corresponded to 389 kN of lateral strength. The BRB steel core bar fractured at the threaded end where the bar connected to the plate of the lower hinge

joint. It is evident that the stiffening of the frame with the BRB system resulted in a more rigid and less ductile structure. The retrofitted frame attained a ductility ratio of 2.4 under the cyclic loading and 2.25 under the subsequent monotonic loading. The experimental results confirm the analytical sectional calculations for predicted sequence of damage in individual frame members (Table 6.5 of Chapter 6). However, None of the frame members reached their flexural capacities. Strain gauge readings indicated yielding of the upper reinforcement in the beam near the left side of the beam-column joint at 0.5% drift, and at the starter bars of the near column reinforcement at 1.25% drift.

Figure VI.14 of Appendix VI illustrates the strain data from gauges located on the steel core bar of the BRB system (Configuration shown in Figure 6.39 (a) of Chapter 6). The data shows that none of the strain gauges recorded yielding when the bar was under axial tension (push frame mode) and, therefore, the bar remained in the elastic range below the yielding strain of 0.24%. However, when the bar was under compression (pull frame mode), yielding was recorded at 1.5% lateral drift. Placement of strain gauges was not possible at this location. Therefore, the strains recorded by the strain gauges were not representative of the strains experienced by the steel bar at the location of failure. However, the axial force sustained by the steel core was constant along the entire length. The elongation of the BRB steel core recorded by the DCTs (Figure VI.15 of Appendix VI) was approximately half of the elongation calculated from the geometry of the displaced frame (Table 6.6 of Chapter 6). This was attributed to the looseness of the BRB system hinge joints before tightening them. Strains transferred from the steel core bar to the outer HSS casing were negligible. This was confirmed from the strain readings recorded by the strain gauges located on the outer HSS round casing (Figure 6.39 c)) as illustrated in Figure VI.16 and Figure VI.17 of Appendix VI. Maximum recorded strain was 0.032%, which was less than yielding strain of 0.24% obtained from coupon testing of the HSS round casing.

Figure VI.8 and Figure VI.9 of Appendix VI provide readings of the LVDTs that measured vertical displacements: LVDTs No. 1 and 2 recorded displacements within the plastic hinge region; while LVDTs No. 5 and 6 measured base anchorage slip of the far column. The experimental displacements from LVDTs No. 1 and 2 indicate that the response was symmetrical, with maximum displacements of 4 mm and 2 mm in the push and pull modes, respectively. Similar trends were captured by LVDTs No. 5 and 6 with maximum

displacements of 2.9 mm and 1.9 mm in the push and pull modes, respectively. Rocking of the foundation of the concrete frame during loading was negligible. Figure VI.18 indicates maximum rocking displacements of 1.1 mm and 1.0 mm in the push and pull modes, respectively.

No cracks were visible on the frame at the end of the first three imposed lateral drifts: 0.125% (4 mm); 0.25% (8 mm); and 0.375% (12 mm). The lateral load capacities of the frame in the push mode, at these drifts levels, were 50 kN, 118 kN, and 182 kN, respectively; while the lateral load capacities in the pull mode were 37 kN, 121 kN, and 182 kN, respectively. The average axial tensile forces sustained by the BRB steel core based on the strain gauge readings, at these three lateral drift levels, were 38 kN, 64 kN, and 122 kN, corresponding to average strains of 0.0123%, 0.0204% and 0.0393%, respectively. The average axial compression forces experienced by the steel core bar were 29 kN, 125 kN, and 128 kN, corresponding to average strains of 0.0092%, 0.0402%, and 0.0411%, respectively. Figures 7.18 – 7.23 illustrate the frame behaviour at these three lateral drift levels. There were no noticeable change in the external gaps between the outer HSS round casing and the plates of the upper and lower hinge joints (see figures corresponding to (e) and (f) of Figures 7.18, 7.20, and 7.21). At 0.375% lateral drift in the push mode, uplift of the HSS of the BRB hinge at the near column was observed (Figure 7.22 (g)).

The first cracks became visible at 0.5% drift ratio (16 mm). Shear cracks initiated at the beam-column joint of the near column; otherwise, no other cracks were observed in the frame as shown in Figures 7.24 and 7.25. At the location of the crack, the top reinforcing bars in the beam began yielding according to S.G. #31. The lateral load capacities of the frame in the push and pull modes at this drift were 243 kN and 251 kN, respectively. The average axial tension and compression forces of the BRB steel core bar were 148 kN corresponding to an average strain of 0.0475% and 352 kN corresponding to an average strain of 0.113%, respectively.

Shear cracks further propagated and surfaced at beam-column joints of the near and far columns at 0.625% drift ratio (20 mm). In addition, flexural cracks surfaced at the base of the near and far columns. Figures 7.26 and 7.27 depict the frame behaviour at this drift ratio. The lateral load capacities of the frame at this drift were 308 kN and 327 kN in the

push and pull modes, respectively. Furthermore, the hysteretic response illustrates the first softening of the global system behaviour at this drift level (Figure 7.17). The average axial tension and compression forces experienced by the BRB steel core bar were 204 kN which corresponded to an average strain of 0.0654% and 452 kN corresponding to an average strain of 0.147%, respectively. Although these forces and corresponding strains were less than the yield force of 553 kN and yield strain of 0.24% determined from coupon testing, it is probable that yielding occurred at the reduced sectional area where the BRB steel core bar was threaded and bolted to the plate of the lower hinge joint. At these threads, placement of strain gauges was not possible.

No additional cracks were observed at 0.75% drift ratio (24 mm) as illustrated in Figures 7.28 and 7.29. The lateral load capacities of the frame at this drift level were 354 kN and 397 kN in the push and pull modes, respectively; while the average axial tension and compression forces sustained by the BRB steel core bar were 292 kN, which corresponded to an average strain of 0.0939%; and 519 kN corresponding to an average strain of 0.171%, respectively.

The flexural cracks further propagated at 1.0% drift ratio (32 mm) at the base of the near column and began to surface at the base of the far column in the plastic hinge region and up to the height of 1.0 m from the foundation level as illustrated in Figure 7.30. No additional cracking became visible in the beam and beam-column joints. Figure 7.31 illustrates the global view of the frame at this drift limit. The lateral load capacities of the frame at this drift were 458 kN and 467 kN in the push and pull modes, respectively. The average axial tension and compression forces sustained by the BRB steel core bar were 337 kN corresponding to an average strain of 0.108% and 578 kN corresponding to an average strain of 0.189%, respectively.

As the drift ratio increased to 1.25% (40 mm), the hysteretic response (Figure 7.17) experienced a second softening point. At this drift level, the reinforcing starter bars at the base of the near column at the outside edge of the column began yielding according to strain gauges S.G. #12 and #15.

As the imposed drift increased to 1.5% (48 mm), no further cracking surfaced during the push and pull cycles, as shown in Figure 7.32. However, noticeable relative rotations

during the pushing cycles were observed at the BRB joints of the inner HSS that were connected to the concrete frame as illustrated in Figure 7.32 (g) and (h). As a result the testing of this frame was paused. The maximum lateral load capacities attained were 578 kN and 621 kN in the push and pull modes, respectively. At this drift level, the BRB steel core was yielding in compression (the pull mode). The axial force in the bar was 757 kN which corresponded to an average strain of 0.402%; while in tension (the push mode) bar remained elastic with an axial force of 495 kN corresponding to an average strain of 0.159%. The axial capacity of the BRB steel core bar was higher in compression (frame pull mode) than the axial capacity in tension (frame push mode). This is attributed to the snug-tight fit of the BRB system during the frame pull mode. Conversely, during the frame push mode some slack between the BRB system and the concrete frame was present. Figure 7.33 provides a photo of the frame at the end of these drift cycles. The BRB brace system was then dismantled and subsequently placed back in the frame after the addition of stiffeners for the connection of the BRB hinge joints, as illustrated in Figure 7.34.

Upon retesting, Frame RRF was pushed monotonically to attain the peak displacement before the test was paused. The monotonic behaviour is illustrated in Figure 7.17, which illustrates the effect of securing the BRB hinge connections in terms of increasing the elastic stiffness and experiencing earlier yielding compared to the system behaviour prior to the securement. The system yielded at 16 mm (0.5% drift) and fractured at 36 mm (1.13% drift), which corresponded to 389 kN and 577 kN of lateral load, respectively. The analytical sectional calculations predicted a lateral force of 503 kN, which would cause the BRB steel core to fracture (Table 6.5 of Chapter 6). This corresponds to 87% of the experimentally recorded force. Based on the strain gauge readings, the BRB steel core was subjected to an average axial tension force of 294 kN corresponding to an average strain of 0.1253% at 16 mm lateral displacement. These strain gauges became defective after the frame was pushed to a lateral displacement of 32 mm. At lateral displacement of 32 mm, the BRB steel core bar sustained an axial tension force of 439 kN corresponding to an average strain of 0.188%. The bar fractured at the reduced threaded area where the bar was bolted to the plate of the lower hinge joint as illustrated in Figures 7.35 (a) and 7.35 (b). The bar fracture was accompanied by twisting of the 6.35 mm thick end plate that secured the steel core restraining end elements to the 57 mm thick joint steel plate as illustrated in Figures 7.35 (c) and 7.35 (d). The twisted plate

was removed and replaced by a 12.7 mm thick plate in subsequent testing.

The frame was laterally loaded back to its initial displacement (zero lateral displacement). The BRB assembly was then prepared for the second type of steel core bar described in the following section. The marked flexural and shear cracks that surfaced during testing remained and were not painted over.

### **7.3.2 RRF Test Using AISI Type 304 Stainless Steel Core Bar**

Frame RRF was re-tested with the AISI Type 304 stainless steel core bar placed within the BRB system (Figures 6.35 (b) and Figure 6.41) as illustrated in the BRB configuration shown in Figure 6.55. Figure 7.36 illustrates the experimental hysteretic lateral load-lateral displacement response based on 3 cycles for each drift level. The hysteretic response revealed that the system remained elastic until it yielded at a lateral displacement of 16 mm (drift ratio of 0.5%), and further softened at a lateral displacement of 75 mm (drift ratio of 2.3%). It finally attained its maximum lateral strength capacity at a lateral displacement of 95 mm (drift ratio of 3%). The system yielding was controlled by the yielding of the BRB steel core at a lateral displacement of 12 mm (drift ratio of 0.375%) during the push frame mode, while the second softening of the system was due to yielding of the concrete frame. The change of slope of the hysteretic response (stiffness) was significant after the system yielded and was characterized with linear ascending slope rather than a flat plateau. The stress-strain behaviour of the BRB steel core bar (Figure 6.19 of Chapter 6) with curved post yield plateau, was an essential contribution to the positive ascending hysteretic slope. The lateral load capacities of the frame at lateral displacements of 16 mm and 75 mm were 249 kN and 560 kN, respectively during the push cycles. The lateral resistance of the frame continued to increase until a lateral displacement of 95 mm (3.0% drift), at which point the BRB steel core fractured in the middle section of the reduced area during the second push cycle. The lateral strength capacity at 95 mm lateral displacement was 600 kN. This confirmed the analytically predicted sequence of damage for the individual frame members due (Table 6.5 of Chapter 6). The experimental lateral drift capacities of the frame were 2.5% in push and 3.0% in pull modes. Therefore, the frame attained a displacement ductility ratio of 5.0 and 6.0 in the push and pull modes, respectively, based on the initiation of yielding coinciding with 0.5% drift as determined from the hysteretic response. The BRB steel core bar yielded at an axial tension force of 247 kN,

which was determined from correlating the strain gauge readings located on the bar with the stress-strain response attained from coupon testing. This experimental axial tension force was in good agreement with the analytical force of 241 kN calculated based on the yield force of the reduced sectional area of the BRB. Furthermore, based on the frame analysis (Appendix V, Section V.3.2 and Table 6.3 of Chapter 6), this BRB steel core axial load of 241 kN corresponds to 199 kN lateral loading on the frame. This represents approximately 80% of the experimental lateral load capacity sustained at this drift level (0.5%).

None of the individual frame members reached their flexural capacities. Strain gauges readings indicated yielding of the starter bars of the near column at 75 mm (2.3% drift). It was expected that the frame would yield at larger drifts compared to the earlier two tests performed on the same frame (BCF and RRF (AISI 12L14)). This was the result of softening in the frame due to the residual damage from the earlier tests. Strain gauges situated on the steel reinforcement of the beam and columns near the beam-column joints were defective and the results were omitted.

Figure VI.23 of Appendix VI provides strain data from the strain gauges that were placed on the steel core bar (configuration shown in Figure 6.39 (b) of Chapter 6). The yielding first initiated in the bar under axial tension force (push frame mode) at a lateral displacement of 12 mm (0.375% drift), exceeding the yielding strain 0.16% obtained from coupon tests. Under compressive stresses (pull frame mode), yielding was realized at a lateral displacement of 16 mm (0.5% drift). At lateral displacements greater than 16 mm, the strain gauges became defective and, therefore, the elongation of the BRB steel core was based on the data recorded from the DCTs (Figure VI.24 of Appendix VI). The elongations based on the DCTs were in good agreement with the elongations calculated from the geometry of the displaced frame (Table 6.6 of Chapter 6). These DCT elongation readings were recorded between the plates of upper and lower joints (Figure 6.65 of Chapter 6) and those calculated from the difference of the DCTs that were connected to the upper and lower reduced areas of the core bar (Figure 6.64 of Chapter 6). At 3% drift, the axial elongation of the BRB steel core bar recorded by the DCT was 63 mm; while the displacement calculated from geometry was 70.5 mm. The difference is attributed to the rotation of the BRB hinge joints during testing.



Figure VI.25 and Figure VI.26 of Appendix VI illustrates longitudinal and circumference strains, respectively, recorded by strain gauges located on the outer HSS round casing (Figure 6.39 c)). The data indicates that strains of the outer HSS round casing were negligible and within the elastic range; the maximum recorded strain was 0.036%, which was significantly lower than yielding strain of 0.24% obtained from coupon testing of the HSS round casing.

Data readings of the 300 mm-height LVDTs (No.1 and 2) measuring vertical displacements within the plastic hinge region of the far column are illustrated in Figure VI.19; while Figure VI.20 illustrates data readings of the 25 mm-height LVDTs (No.5 and 6) used to measure base anchorage slip of the column reinforcement. Symmetrical responses were captured for the LVDTs on the opposite faces of the far column. Maximum-recorded displacements were 10.5 mm and 4 mm in the push and pull modes, respectively for LVDT No.1 and 2; while maximum displacements were 7.3 mm and 8.1 mm in the push and pull modes, respectively for LVDT No.5 and 6. Measuring of foundation rocking of the concrete frame during loading was not possible due to instrumentation problems during testing. However, based on the results of the previous test, uplift of the foundation during testing was assumed to be trivial.

No cracks surfaced on the frame at the end of 0.125% drift (4 mm). The lateral load capacities at this drift were 116 kN in the push mode and 42 kN in the pull mode. The average axial force experienced by the BRB steel core in tension, determined from the strains recorded by strain gauges, was 50 kN, which corresponded to an average strain of 0.032%; while the axial compression force was 18 kN, which corresponded to an average strain of 0.011%. Figures 7.37 and 7.38 illustrate the frame behaviour at this lateral drift level.

The formation of new shear cracks became visible at 0.25% drift ratio (8 mm). Shear cracks initiated on the beam near the actuator plate; otherwise no other cracks were observed in the frame as shown in Figure 7.39 and Figure 7.40. The lateral force resistances of the frame at this drift were 182 kN and 132 kN in push and pull cycles, respectively. The average axial force sustained by the BRB steel core bar in tension was 67 kN, which corresponded to an average strain of 0.043%; while the axial force in compression was 26 kN corresponding to an average strain of 0.016%.

Shear cracking further propagated and surfaced at the beam-column joint of the near column at 0.375% drift ratio (12 mm). In addition, new flexural cracks surfaced at the soffit of the beam near the left beam-column joint while no additional cracks were observed in the columns. Figure 7.41 and Figure 7.42 depict the frame behaviour at this drift ratio. The lateral load capacity in the push mode was 226 kN while the lateral load capacity in the pull mode was 190 kN. The BRB steel core bar yielded at this drift ratio, which corresponded to the yield point of the global hysteric behaviour of the frame. The average axial tension force in the BRB steel core bar was 247 kN which corresponded to an average strain of 0.264%; while the axial compression force was 33 kN corresponding to an average strain of 0.021%. The lower axial forces in compression compared to tension in the BRB steel core bar up to this drift level were apparent from the strain gauge readings. This probable cause of this discrepancy was the slackness between the BRB system and the frame.

Flexural cracking further propagated at 0.5% drift ratio (16 mm) along the entire half-length of the left end of the beam from the soffit up to the two-thirds of the depth of the cross section. Shear cracking also propagated at this drift level at the beam-column joint of the near column. Moreover, new flexural cracking began to surface at the base of both columns. Figure 7.43 and Figure 7.44 illustrate the crack pattern and behaviour of the frame at this drift level. Peak lateral strength resistances of the frame at this drift were 249 kN and 230 kN in push and pull cycles, respectively. The average tension axial force experienced by the BRB steel core bar was 266 kN which corresponded to an average strain of 0.712%; while the axial compression force was 244 kN which corresponding to an average strain of 0.24%.

No additional cracks surfaced during push and pull cycles at 0.625% drift ratio (20 mm) as shown in Figures 7.45 and 7.46. The corresponding lateral load force resistances were 280 kN and 275 kN in push and pull cycles, respectively. The capacity of the strain gauges located on the BRB steel core (Figure VI.23 of Appendix VI) were exceeded. Beyond this drift level, these gauges were no longer functional. Therefore, strains and corresponding forces experienced along the length of the BRB steel bar with reduced area were calculated based on average axial longitudinal displacements of the steel core bar (Figure VI.24 of Appendix VI). These average longitudinal displacements were based on recorded data from the DCT that was connected between the plates of upper

and lower joints, and based on the difference of displacements between the two DCTs that were connected at both ends of the reduced area of the core bar. The average axial force sustained by the BRB steel core bar in tension and compression were similar with a value of 256 kN corresponding to an average strain of 0.707%.

Flexural and shear cracks further widened and propagated at 0.75% drift ratio (24 mm) at beam-column joint of the near column and at the bases of the columns as illustrated in Figure 7.47 and Figure 7.48. The lateral load capacities at this drift were 326 kN and 303 during push and pull cycles, respectively. Axial tension and compression forces in the BRB steel core were 262 kN corresponding to an average strain of 1.037% and 259 kN corresponding to an average strain of 0.87%, respectively.

As the drift ratio increased to 1.0% (32 mm), shear and flexural cracks widened along the half-length of the left end of the beam and throughout the beam height. Shear cracks also widened at both beam-column joints as illustrated in Figures 7.49 and 7.50. This level of drift corresponded to a lateral load 370 kN and 355 kN in push and pull cycles, respectively. Average axial forces experienced by the BRB steel core bar were 265 kN corresponding to an average strain of 1.26% in tension and 285 kN corresponding to an average strain of 1.97% in compression.

At 1.5% drift ratio (48 mm), additional shear cracks were visible in the beam and in the beam-column joint of the far column. New flexural cracks formed at the base of the near column as illustrated in Figures 7.51 and 7.52. The displacements recorded by LVDTs # 1 and # 5 (see Figure 6.56), located on the column at 25 mm and 300 mm above the foundation, respectively, were inconsistent and not reliable. Therefore, from this drift level and beyond, the data from these LVDTs were omitted. The lateral load capacities of the frame in the push and pull modes at this drift were 460 kN and 462 kN, respectively. The average axial tension and compression forces in the BRB steel core bar were 269 kN corresponding to an average strain 1.69% and 290 kN corresponding to an average strain of 2.33%, respectively.

The lateral resistances of the frame at 2.0% drift (63.5 mm) were to 524 kN and 570 kN in push and pull cycles, respectively. Observed damage in the form of concrete cover spalling became visible at the base of the far column at this drift level. However, no

exposure of main reinforcing steel bars or stirrups was evident. Shear and flexural cracking increased along the beam and in the beam-column joints, as illustrated in Figures 7.53 and 7.54. The average axial tension and compression forces sustained by the BRB steel core bar were 287 kN corresponding to an average strain of 2.06% and 300 kN corresponding to an average strain of 2.92%, respectively.

The hysteretic response indicated that the frame experienced strength decay and continuous stiffness degradation at lateral a displacement of 75 mm (drift ratio of 2.3%) as shown in Figure 7.36. This softening resulted from cracks propagating along the frame as well as from yielding of the column reinforcement as confirmed by the strains recorded by Strain Gauge #16 located on the column longitudinal reinforcement near the column base.

As the drift ratio increased to 2.5% (79 mm), shear and flexural cracks extended and widened at the critical regions of the beam and at the beam-column joints. Furthermore, significant concrete spalling at the base of both columns was observed, as illustrated in Figures 7.55 and 7.56. The lower external gap between the HSS round casing and the plate of the lower joint was nearly exhausted as shown in Figure 7.55 (e). However, there was no direct bearing between the HSS round casing and the plates at the lower and upper joints of the BRB system; therefore, no axial loads were expected to be transferred to the HSS round casing. The maximum lateral load capacities were 577 kN and 730 kN in push and pull cycles, respectively, at this drift level. The average axial tension and compression forces experienced by the BRB steel core bar were 290 kN corresponding to an average strain of 2.28% and 306 kN corresponding to average strain of 3.3%, respectively.

The peak lateral load resistance of frame was attained at 3.0% drift (95 mm) corresponding to 600 kN and 850 kN in push and pull cycles, respectively. At this drift ratio, significant concrete spalling at the base of the far column was observed. Furthermore, shear tension and flexural cracks propagated and widened at the critical regions of the frame: beam-column joints; beam soffit; and column bases as illustrated in Figure 7.57 and Figure 7.58. During the second pushing cycle of the frame towards this drift ratio, the BRB steel core bar fractured at the middle section of the reduced area. Buckling of the steel core bar was also evident as illustrated in Figure 7.59. Furthermore,

and after dismantling the BRB system, the four-12.7 mm bar spacers (Figure 6.41 of Chapter 6) that were welded to the reduced middle section of the steel core bar were observed to have buckled and displaced from their steel sleeves at their free ends. This was the probable cause for the buckling of the bar. Therefore, more stress concentrations were experienced at certain segments of the bar during compression. At this drift ratio, the frame reached its maximum drift capacity during the push cycle, based on the drop in lateral load capacity due to the rupturing of the BRB steel core. Therefore, the lateral drift capacity of this frame was 2.5% in the push mode and 3.0% in the pull mode. Accordingly, the frame attained ductility ratios of 5.0 and 6.0 in the push and pull modes, respectively, based on the initiation of yielding coinciding with 0.5% drift as determined from the hysteretic response. At 3% drift, average axial tension and compression forces sustained by the BRB steel core bar were 313 kN corresponding to an average strain of 3.82% and 310 kN corresponding to an average strain of 3.58%, respectively.

Although the maximum axial tension force of the BRB steel core bar was less than the analytically predicted fracture force of 477 kN, it is apparent that the localized distorted segments of the bar, illustrated in Figure 7.59 (c), resulted in higher stress concentrations causing the bar to fracture prematurely.

Figure 7.60 and Figure 7.61 illustrates frame-crack patterns after the removal of the BRB system.

### **7.3.3 RF Test Using AISI 4140 Chrome-Molybdenum Steel Core Bar**

Un-damaged Frame RF was tested using AISI 4140 chrome-molybdenum high tensile steel for the BRB core bar (configuration shown in Figures 6.35 (c) and 6.42 of Chapter 6) as illustrated in Figure 6.55. The experimental hysteretic lateral load-lateral displacement response of the frame (3 cycles at each drift level) is illustrated in Figure 7.62. Yielding of the BRB steel core and yielding of the concrete frame dictated the softening response of the global behaviour. The hysteretic response illustrates rigid elastic behaviour until yielding at 16 mm lateral displacement (drift ratio of 0.5%), and further softening was observed at 38 mm (drift ratio of 1.2%) in which yielding of the column longitudinal reinforcement near the far column base was confirmed from the strain gauges. The positive ascending slope of the stress-strain behaviour of the BRB

steel core bar (Figure 6.20 of Chapter 6) clearly influenced the global hysteric response of the frame. In the push mode cycles, the lateral load capacities at lateral displacements of 16 mm and 38 mm were 291 kN and 483 kN, respectively. The frame attained its maximum displacement level during the first pushing cycle at 48 mm (1.5% drift), which corresponded to a lateral resistance of 529 kN. During the second push cycle towards the maximum drift, the BRB steel core bar fractured at the middle section of the reduced area at a displacement of 42 mm corresponding to 1.33% drift and 510 kN of lateral load capacity. Therefore, the lateral drift capacities of the frame were 1.33% and 1.5% in the push and pull modes, respectively. The analytical sectional calculations predicted that a lateral force of 479 kN would cause fracture of the BRB steel core bar (Table 6.5 of Chapter 6), which is slightly lower than the experimental force. The frame attained ductility ratios of 2.6 and 3.0 in push and pull modes, respectively, based on the initiation of yielding coinciding with 0.5% drift as determined from the hysteretic response. The experimental results were in good agreement with the analytical sectional calculations for the predicted sequence of damage of the frame members (Table 6.5 of Chapter 6). None of the frame members reached their flexural capacities. Strain gauge readings indicated yielding of starter bars of the far column reinforcement at 1.2% drift, as well as the middle longitudinal column bar and the beam lower reinforcement near the left beam-column joint, both at 1.5% drift.

Figure Column 3 of Appendix VI provides data of the strain gauges located on the steel core bar of the BRB system (configuration shown in Figure 6.39 (b) of Chapter 6). The BRB steel core bar yielded at a lateral displacement of 24 mm (0.75% drift ratio) based on an axial tension force of 377 kN. The latter was attained from correlating the strains recorded by the strain gauges located on the bar with the stress-strain behaviour based on coupon testing. Yielding of the bar initiated at a lateral displacement of 22 mm (0.7% drift ratio) which corresponded to an axial tension force of 328 kN; approximately equal to the analytical yield force of 310 kN calculated based on yielding of the reduced sectional area of the BRB steel core. During the frame pull cycles, the strain gauge readings indicated bar yielding at a lateral displacement of 16 mm (0.5% drift ratio) corresponding to an axial compression force of 419 kN. Yielding of the bar is a localized phenomenon. Therefore, the strains recorded by the strain gauges located on the BRB steel bar were generally smaller than those based on the elongation of the BRB steel bar recorded from the DCTs (Figure VI.34 of Appendix VI) and the elongation calculated

from the geometry of the displaced frame (Table 6.6 of Chapter 6). The latter two types of measurements were in very good agreement. At 1.5% drift, the total axial deformation of the BRB steel core bar was 23 mm according to the readings of the strain gauge; while the axial deformation based on the DCT that was fixed between the plates of upper and lower joints (Figure 6.65 of Chapter 6), and the axial displacement calculated from geometry (Table 6.6 of Chapter 6) were 32.5 mm and 36 mm, respectively. Strains transferred from the steel core to the outer HSS casing were negligible. This was confirmed from the strain gauges located on the outer HSS round casing (Figure 6.39 (c)) as illustrated in Figure VI.35 and Figure VI.36 of Appendix VI. The maximum recorded strain was 0.025%; less than the yield strain of 0.24% obtained from coupon testing of the HSS round casing.

The vertical displacements within the plastic hinge region of the far column at a height of the 300 mm from the foundation (LVDT No.1 and 2) are illustrated in Figure VI.27; while Figure VI.28 illustrates recorded data of the 25 mm-height LVDTs (No. 5 and 6) used to measure base anchorage slip of the column reinforcement. Symmetrical responses were observed for the LVDTs on opposite faces of the far column. The maximum displacements were 3.5 mm and 2.6 mm in the push and pull modes, respectively, for LVDTs No.1 and 2; while the maximum displacements were 2.2 mm and 2.1 mm in the push and pull modes, respectively for LVDTs No. 5 and 6. Rocking of the foundation of the concrete frame during loading was negligible. This was evident from Figure VI.37 of Appendix VI, which indicates maximum displacements of 3 mm and 1.3 mm in the pushing and pulling modes, respectively.

No cracks surfaced on the frame at the end of 0.125% drift (4 mm) and 0.25% drift (8 mm) cycles. The lateral load capacities at these drifts were 83 kN and 165 kN in the push mode, respectively, and 134 kN and 235 kN in the pull mode, respectively. The average axial force in the BRB steel core bar in tension based on the strain gauge readings were 29 kN and 106 kN corresponding to average strains of 0.02% and 0.07%, respectively at 0.125% and 0.25% drifts. The average axial compression forces in the steel core bar were 111 kN and 293 kN corresponding to average strains of 0.073% and 0.194%, respectively at the above drift levels. Figures 7.63 to 7.66 illustrate the frame behaviour at these two lateral drifts.



The first flexural cracks became visible at 0.375% drift ratio (12 mm). The cracks initiated at the beam soffit and more noticeably at the left half span. No other cracks were observed in the frame as shown in Figures 7.67 and 7.68. The lateral load capacities of the frame in the push and pull modes at this drift were 233 kN and 302 kN, respectively. The average axial tension and compression forces sustained by the BRB steel core bar were 113 kN corresponding to an average strain of 0.075% and 298 kN corresponding to an average strain of 0.33%, respectively.

Flexural cracking further propagated at 0.5% drift ratio (16 mm) along the entire length of the beam, extending to both beam-column joints and from the soffit of the beam up to the one-third of the depth of the cross section. No other cracks were observed elsewhere in the frame as illustrated in Figures 7.69 and 7.70. The hysteretic lateral load-lateral displacement response at this drift level demonstrated the first softening of the global behaviour corresponding to lateral load capacities of 291 kN and 339 during the push and pull cycles, respectively. Based on the strains recorded by the strain gauges located on the BRB steel core, a reduction in force was experienced at this drift in the push cycles relative to the previous drift level. The probable cause is the slackness in the BRB system; there was no evidence for other causes of softening, such as frame reinforcement yielding. However, the strain gauges indicated bar yielding in compression. Axial tension and compression forces in the BRB steel core bar were 89 kN corresponding to an average strain of 0.059% and 419 kN corresponding to an average strain of 0.56%, respectively.

No additional cracks were observed at 0.625% drift ratio (20 mm) and at 0.75% drift ratio (24 mm) as illustrated in Figures 7.71 to 7.74. The lateral load capacities of the frame at these two drifts were 341 kN and 379 kN in the push cycles, respectively, and 360 kN and 396 kN in the pull cycles, respectively. Based on the strains recorded by the strain gauges, the BRB steel core bar yielded in tension at 24 mm (0.75% drift ratio). The average axial tension forces at 0.625% and 0.75% drift ratios were 222 kN and 377 kN corresponding to average strains of 0.147% and 0.41%, respectively. The average axial compression forces of the steel core bar were 363 kN and 371 kN corresponding to average strains of 0.83% and 0.91%, respectively at these two drift levels.

The first flexural cracks in the columns became visible at 1.0% drift ratio (32 mm).

Cracks initiated in the columns and more noticeably at the base of the far column up to a height of 1000 mm from the foundation level. No other cracks were observed in the rest of the frame as illustrated in the frame crack patterns shown in Figures 7.75 and 7.76. Frame lateral load capacities were 437 kN and 451 kN in the push and pull cycles, respectively. The average axial tension force experienced by the BRB steel core bar was 352 kN, corresponding to an average strain of 0.663%; while the axial compression force was 397 kN, corresponding to an average strain of 1.23%.

As the drift ratio increased to 1.2% (38 mm), the hysteretic behaviour (Figure 7.62) depicted the second softening point. This softening was due to the yielding of the starter bars at the base of the far column as recorded by Strain Gauge S.G. #1. This is illustrated in Figure VI.29 of Appendix VI.

Peak lateral load resistances of 529 kN and 543 kN for the frame were attained at 1.5% drift (48 mm) during the first push and pull cycles, respectively. The average axial forces sustained by the BRB steel core bar were nearly identical in tension and compression at 314 kN corresponding to an average strain of 1.5%. Flexural cracking widened and extended along both columns, and extended in the far column up to the mid height, as illustrated in Figure 7.77. No further cracking in the beam was observed during the push and pull cycles, as illustrated in the crack pattern of Figure 7.78. However, at the near beam-column joint, the strain gauges on the middle longitudinal column bar (S.G. #23) and strain gauges on the bottom reinforcing bars of the beam (S.G. #29 and S.G. #30) recorded strains in excess of yielding as illustrated in Figure VI.31 and Figure VI.32 of Appendix VI, respectively. As the frame was pushed to the second cycle, the BRB steel core bar fractured at the middle section of the reduced area at a lateral displacement of 42 mm, corresponding to 1.33% drift and 510 kN lateral load capacity. Figures 7.80 (a), (b), and (c) illustrate the BRB steel core bar at failure. The un-bonded epoxy sand filler served as a viable method to restrain the steel core bar from buckling under compressive forces. These segments of the epoxy sand fillers and the steel spacer bars, that were welded to the mid length of the BRB steel core bar, were intact after dismantling the BRB system at the end of testing as shown in Figure 7.80 (d) and (e).

#### **7.4 Summary and Discussion of Experimental Results**

This section discusses a summary of the experimental force-deformation envelopes

showing the results of the four tests to assess the contribution of the buckling restrained brace retrofit to the overall frame behaviour. Frame BCF served as a reference for the retrofit methodology. Frame RRF, retrofitted with AISI 12L14 carbon BRB steel core bar was tested twice: first, under cyclic loading prior to securing the hinge connections to the concrete frame; and second, monotonically after securing the BRB hinges on the joints rigidly. Therefore, a softer response was observed initially due to the slack in the system. The other two types of BRB steel core (AISI Type 304 stainless steel bar and AISI 4140 chrome-molybdenum high tensile bar) were tested using similar setup with stiff BRB hinge connections. However, the AISI Type 304 stainless steel bar was tested within the same Frame RRF that had been tested earlier with the AISI 12L14 carbon steel bar; while the AISI 4140 chrome-molybdenum high tensile was tested within the un-damaged Frame RF.

The comparison includes global load–displacement responses and the ductility attained; concrete frame lateral resistance; overall stiffness; energy dissipation; and column moment-rotation responses. These will be discussed in the following sections.

#### **7.4.1 Load–Displacement Response and Ductility**

The retrofitted frames were tested using the same loading protocol. Their lateral load–lateral displacement envelope responses are provided in Figure 7.81 with key values summarized in Tables 7.2 and 7.3. The retrofitted frames were characterized by more rigid behaviour than the bare control frame. Retrofitting substantially increased the stiffness of the frames. Furthermore, the load–displacement response of the retrofitted frames were dominated by the stress-strain response of the steel core bar used in the buckling restrained brace (Figures 6.18 to 6.20 of Chapter 6). Generally, the envelope response showed elastic behaviour until the onset of the BRB steel core yield. The steel core mechanical properties: modulus of elasticity; yield strength; yield strain; strain-hardening plateau; ultimate strength; and ultimate strain were significant contributors to the overall frame envelope response. Furthermore, the additional softening observed in envelope responses was a result of slackness in the bracing system; yielding of the frame reinforcing bars, particularly yielding of the columns; and the propagation of concrete cracking.

The bare control frame (BCF) experienced a gradual decrease in lateral stiffness during

the elastic range of loading due to the cracking of concrete. Yielding initiated at a drift ratio of 1.3% (41 mm). The maximum lateral strength was sustained at 2.5% drift ratio, which corresponded to 233 kN and 219 kN during first push and pull cycles, respectively. The frame failed at 4.0% drift during the first push and second pull cycles as the lateral resistances were 78% and 71% of peak strengths, respectively. Therefore the drift capacities of the frame were 3% in the push and pull modes with a corresponding displacement ductility capacity of 2.3.

The modulus of elasticity of the three types of BRB steel core bars were similar, ranging from 190.5 GPa to 200.5 GPa. Therefore, the retrofitted frames were expected to respond with similar rigid elastic responses until the onset of yielding. This was confirmed by the responses shown in Figure 7.81. All the retrofitted frames exhibited similar response in the elastic range with yielding at 16 mm (0.5% drift ratio), with the exception of Frame RRF that was retrofitted with AISI 12L14 and tested before the BRB hinge joints rigidly secured. For this frame, yielding occurred at a displacement of 20 mm (0.625% drift ratio). Frame RRF tested with AISI 12L14 steel core provided the largest lateral load capacity at yield. The yield loads were 389 kN and 309 kN after and before the BRB hinge joint connections were improved, respectively. Frame RRF tested using the AISI Type 304 stainless steel core yielded at a lateral yield load of 305 kN; while Frame RF tested using AISI 4140 chrome-molybdenum high tensile steel bar experienced a lateral yield load of 300 kN. These lateral yield capacities followed the order of yield strengths attained in respective coupon tests; the higher the yield strength of the bar was, the higher the yield capacity of the frame was. Furthermore, the AISI 12L14 steel core bar had the largest bar cross sectional area compared with the other two bars, which were prepared with reduced sectional areas around the mid-length. Therefore, the frame with this bar type experienced higher lateral loads.

The post-yield of the stress-strain behaviour, particularly the strain hardening plateau and the rupturing stress and strain of the steel core bars, dominated the post-yield lateral load-lateral displacement envelope responses of retrofitted frames. The frame with AISI 12L14 carbon steel core bar experienced a linear response after yielding in the tests conducted before and after rigidly securing the hinge connections on the frame. This response was similar to the post-yield behaviour of the stress-strain behaviour established from coupon tests. The frame attained a maximum displacement of 48 mm

(1.5% drift) which corresponded to a lateral strength resistance of 578 kN and 621 kN in the push and pull modes, respectively prior to securing the BRB hinges rigidly. The AISI 12L14 carbon steel core bar fractured at approximately the same lateral load of 577 kN but at a lower lateral displacement of 36 mm (1.13% drift) after the BRB hinge joints were rigidly secured. The maximum lateral load capacity of Frame RRF with this bar increased by a factor of 2.48 and 2.84 in the push and pull modes, respectively, relative to the companion bare Frame BCF. The frame ductility ratios attained were 2.4 and 2.25 before and after the improvement of the rigid connection of the BRB hinge joints, respectively, with almost equal ductility values to those attained by the bare control frame.

Frame RRF with the AISI Type 304 stainless steel and Frame RF with the AISI 4140 chrome-molybdenum high tensile steel core bars demonstrated a softer response with curved post-yield plateau. This trend was similar to the corresponding post-yield coupon behavior. Note that the rupturing strain of the AISI Type 304 stainless steel bar was significantly larger than that of the AISI 12L14 carbon steel and the AISI 4140 chrome-molybdenum high tensile steel bars; therefore, a larger lateral displacement capacity was experienced by Frame RRF with the AISI Type 304 stainless steel core bar. The frame sustained a lateral force capacity of 600 kN, corresponding to a displacement of 95 mm. However, the BRB steel core bar ruptured at the middle section of the reduced bar area during the second push cycle to a displacement of 79 mm; therefore, the frame attained lateral drift capacities of 2.5% corresponding to 577 kN of lateral load in the push mode, and 3.0% corresponding to a lateral load resistance of 850 kN in the pull mode. At these two drift levels, the frame ductility ratios were 5 and 6 in the push and pull modes, respectively. Furthermore, the lateral load capacity increased by factors of 2.48 and 3.65 and the displacement ductility factors increased by factors of 2.13 and 2.56 relative to Frame BCF, respectively in the push and pull modes at maximum drift levels.

The load–displacement envelope response of the un-damaged Frame RF tested with AISI 4140 chrome-molybdenum high tensile steel core bar demonstrated that the frame experienced its maximum displacement earlier compared to Frame RRF with the AISI Type 304 stainless steel core bar and Frame RRF with AISI 12L14 carbon steel core bar after the BRB hinges were rigidly secured. This was consistent with the rupturing strains

of the steel core bars established from the stress-strain coupon response. The AISI 4140 chrome-molybdenum high tensile steel bar had a smaller rupturing strain capacity compared to that for the AISI Type 304 stainless steel core bar, but greater than that for the AISI 12L14 carbon steel bar. Frame RF experienced a maximum displacement capacity of 48 mm (1.5% drift) and lateral force capacities of 529 kN and 543 kN during the push and pull cycles, respectively. The BRB steel core ruptured at the middle section of the reduced area at a displacement of 42 mm corresponding to 1.33% drift and 510 kN of lateral load capacity during the second push cycle. The lateral drift capacities of the frame were 1.33% and 1.5% in the push and pull modes, respectively; while the lateral load capacities increased by factors of 2.2 and 2.48 relative to Frame BCF. The frame attained ductility ratios of 2.6 and 3.0 in the push and pull modes, respectively, an increase in ductility ratios of 1.13 and 1.29 relative to Frame BCF in the push and pull modes, respectively.

#### **7.4.2 Concrete Frame Lateral Force Resisting Capacity**

The bare control frame (BCF) was first tested and then repaired for retrofitting. The retrofitting concept was similar for all tests and incorporated buckling restrained braces (BRBs). The main difference was the types of steel core bar placed within the BRB. AISI 12L14 carbon steel and AISI Type 304 stainless steel core bars were investigated first in two separate tests with the repaired bare control frame (RRF); while the AISI 4140 chrome-molybdenum steel core bar was tested with a second un-damaged frame (RF) that was similar in design to the first frame (BCF). The effect of the horizontal component of the prestressing strands that were used to impose the gravity loading on the frames was excluded from load-displacement responses. The net result provided the lateral force resistance of the retrofitted frames that consisted of the sum of components of the concrete frame and the horizontal component of the BRB steel core bars. The axial force component of the BRB steel core was established by using the average strains recorded by the strain gauges located on the bar with the stress-strain response attained from coupon tests. However, DCTs were used in lieu of the strain gauges once the gauges became defective. Figure 7.82 provides the lateral force component provided by the frame alone for the retrofitted frames. The axial force component of the BRB was first resolved into its horizontal component based on frame geometry at each lateral displacement i.e. based on the angle of inclination of the bar relative to the horizontal (Table 6.6 of Chapter 6). The horizontal force components of the BRB steel core bar

were then subtracted from the net lateral load (concrete frame with BRB contributions). The concrete frame contribution to the resulting lateral force-lateral displacement envelope response is provided in Figure 7.82 along with the response of the bare control frame (BCF). To provide a more comprehensive understanding of the behaviour, axial forces sustained by the BRB steel were compared against the axial force component of the difference between the lateral load resistance of the retrofitted frames and that of Frame BCF, as illustrated in Figure 7.83. The results in Figures 7.82 and 7.83 are from the push cycles, as the strain gauge data of the steel core bars were more consistent when the bar was in tension (rather than compression during the pull mode. The latter local buckling of the steel core affected the strain data. Both comparisons illustrate reasonable agreement of the repaired and un-damaged frames, illustrating the effectiveness of the repair technique used. However, some discrepancies are evident that are attributed to localized data provided by strain gauges, possible slackness in the BRB system, and softening of the frames due to multiple testing and pre-existing damage to the frame. The lateral force resistance component provided by Frame RRF using AISI 12L14 carbon steel core bar before the BRB hinge joint connection improvement was in very good agreement with the lateral load resistance of the bare Frame BCF. This is illustrated in Figures 7.82 (a) and 7.83 (a). After the BRB hinge joint connection was improved a more rigid system was present. The concrete frame contributed a smaller share to the lateral strength, as illustrated in Figure 7.82 (b) and, subsequently, higher axial forces were sustained by the BRB steel core as demonstrated in Figure 7.83 (b). The lateral force resisting component provided by Frame RRF using AISI Type 304 stainless steel core was characterized by increasing and decreasing fluctuations of up to 25 mm of lateral displacement and an increase beyond 35 mm lateral displacement relative to Frame BCF. This is illustrated in Figure 7.82 (c). This was reflected accordingly by the axial force resistance of the BRB steel core based on the instrumentation as illustrated in Figure 7.83 (c). The slack in the BRB system at initial drift levels, as confirmed from the strain gauge data, resulted in lower stresses in the steel core up to 10 mm of lateral displacement. This was followed by a significant increase in stress up to a lateral displacement of 16 mm. Thereafter the strain gauges became defective and the DCTs measurements were used to determine the strain and stresses sustained by the BRB steel bar. The lateral strength of the frame component provided by Frame RF using AISI 4140 chrome-molybdenum steel core bar was in good agreement with Frame BCF at displacements smaller than 10 mm and greater than 30



mm. Both frames were tested in their virgin conditions, free from residual damage. However, in the 10 to 20 mm displacement range, there was an increase in the force contribution of Frame RF; while in the 20 to 30 mm range there was a noticeable decrease as illustrated in Figure 7.82 9d). Conversely, the axial strength contribution of the BRB steel bar, provided in Figure 7.83 (d), was in contrast to the frame contributions for the 10 to 20 mm and 20 to 30 mm ranges.

### 7.4.3 Stiffness

The secant stiffness-lateral displacement envelope response for all the frames is illustrated in Figure 7.84 and further summarized in Table 7.4. The results are based on the push mode cycles for clarity, and the stiffness was calculated by dividing the lateral strength by the corresponding lateral displacement at each drift level. The initial stiffness was calculated at a lateral displacement of 4 mm, the displacement imposed on the retrofitted frames during the first load cycles. The table also provides the secant stiffness at the yield point for each frame (illustrated in Table 7.2). The yield point was based on recommendations reported by ACI Committee (2013) and the Eurocode 8 (2004). At 4 mm lateral displacement, all frames (after the BRB joints were rigidly secured) provided the largest stiffness. Frame RRF using AISI Type 304 stainless steel and AISI 12L14 carbon steel core bars experienced initial stiffnesses of 29 kN/mm and 28.5 kN/mm, respectively, and secant stiffnesses at yield of 19.1 kN/mm and 24.3 kN/mm, respectively. The initial stiffnesses were nearly identical due to the similarity in modulus of elasticity of the two BRB steel core bars (199.2 GPa and 200.5 GPa, respectively) and due to the similarity of the test setup. Frame RF with AISI 4140 chrome-molybdenum steel core bar experienced an initial stiffness of 20.8 kN/mm and a secant stiffness at yielding of 18.8 kN/mm. Note that the modulus of elasticity (190.5 GPa) was lower than those of the other core bars. The lower initial and secant stiffnesses at yield, and softer response was experienced by Frame RRF when the AISI 12L14 carbon steel bar was used. The initial and secant stiffnesses at yield were 12.5 kN/mm and 15.4 kN/mm at a lateral displacement of 4 mm and 20 mm, respectively. This test was conducted before the BRB hinge joints were secured rigidly. Lastly, Frame BCF experienced an initial stiffness of 11.7 kN/mm at 4 mm lateral displacement, and secant stiffness at yield of 4.9 kN/mm at 41 mm lateral displacement. The retrofitted frames, after the BRB hinge joints were secured rigidly, provided increased initial stiffness to that of Frame BCF by factors of 2.5, 2.4, and 1.8, respectively for the frames with AISI Type 304 stainless steel, AISI

12L14 carbon steel, and AISI 4140 chrome-molybdenum high tensile steel. The increased factors for secant stiffness at yield, for the same bars to that of Frame BCF, were of 3.9, 5.0, and 3.8, respectively.

#### **7.4.4 Energy Dissipation**

When concrete structures move from elastic to plastic response, energy is dissipated. Figure 7.85 provides the strain energy-lateral displacement envelope response experienced by all the frames. The strain energy was calculated from the area of the closed loop of the first cycles from the hysteretic lateral load-lateral displacement response. Frame RRF using AISI 12L14 carbon steel bar, after the BRB hinge joints were rigidly connected, was not included since this test was conducted under monotonic loading. An increase in energy dissipation was experienced by Frame RRF using the AISI Type 304 stainless steel bar, while Frame RF using the AISI 4140 chrome-molybdenum high tensile steel bar, and Frame RRF with AISI 12L14 carbon steel bar experienced smaller increases. The latter two frames were capable of dissipating 26027 kN.mm and 13914 kN.mm of energy at a lateral displacement of 48 mm (1.5% drift ratio), respectively. This corresponded to increase factors of 6.6 and 3.5, respectively, relative to Frame BCF (3937 kN.mm) at the same lateral displacement. At the lateral displacement of 95 mm (3.0% drift ratio), Frame RRF using the AISI Type 304 stainless steel bar experienced maximum energy dissipation of 108619 kN.mm, corresponding to an increase of 8.2 times the energy dissipated by Frame BCF.

#### **7.4.5 Moment- Rotation Response**

The moment-rotation response of structural elements play an essential role in the post-elastic performance of reinforced concrete structures during earthquakes. Therefore, it is crucial to properly assess the inelastic response for the purpose of accurate analytical modelling under seismic loading. The flexural rotations within the plastic hinge region of the far column base were computed for all the frames as illustrated in Figure 7.86. The flexural rotations were calculated from the vertical displacements recorded by LVDT No. 1 and 2 (Figure 6.56 of Chapter 6) within the plastic hinge region. This data is available in Appendix VI. The difference of the vertical displacements at the ends of the column was divided by the distance between the two LVDTs to establish rotations. The effect of vertical anchorage slip displacements, recorded by LVDT Nos. 5 and 6 (Figure 6.56, in Appendix VI), were not considered as these displacements captured flexural rotations in

addition to anchorage slip rotations at the base. The moment-rotation responses of the far column for all frames were established using the moment-rotation response calculated for the far column of Frame BCF. Note that the virgin frame (BCF) was assumed to reasonably represent the concrete frame lateral strength component (excluding the BRB steel core contribution) of the retrofitted frames, as discussed in Section 7.4.1.1. For each lateral displacement level, the far column moments for each frame were based on the moments calculated for Frame BCF (Figure V.3 of Appendix V and as summarized in Table 6.2 of Chapter 6). The corresponding rotations for the far columns for each frame were then determined using the LVDT data as described above. It is apparent from Figure 7.86 that for the same base moments, higher rotations were experienced for the frames that attained larger lateral displacements. Larger rotations were also induced due to the softening and cracking of concrete frames after multiple testing. Slackness of the BRB system was also another factor contributing to increases in flexural rotations relative to the virgin Frame BCF. The retrofitted frames experienced similar flexural rotations up to a base moment of 150 KN·m, corresponding a rotation of  $4 \times 10^{-3}$  rad. Thereafter the responses deviated. Flexural rotations of the retrofitted frames were calculated up to a lateral displacement of 48 mm (1.5% drift ratio); the displacement at which either the peak lateral displacement was experienced or the displacement at which the LVDTs were no longer reliable due to the concrete cover cracking. The only exception was Frame RRF, using AISI 12L14 carbon steel core bar with the BRB hinges were rigidly secured, which experienced fracturing of the steel core bar at a lateral displacement of 32 mm. Lateral displacements of 32 mm and 48 mm corresponded to base moments of 190 KN.m and 227 KN.m, respectively. Both Frame BCF and Frame RF using the AISI 4140 chrome-molybdenum high tensile steel core bar were tested in their virgin conditions. Therefore, similar responses were expected. However, due to the slack in the BRB system during the testing of Frame RF in the range of 10 to 20 mm of lateral displacement, a larger lateral strength component was contributed by the concrete frame. Therefore, greater flexural rotations were experienced for the same base moments relative to Frame BCF. Figures 7.82 (d) and Figure 7.83 (d) illustrate this behaviour. The maximum flexural rotation sustained by Frame RF was  $11 \times 10^{-3}$  rad, compared to  $9.7 \times 10^{-3}$  rad experienced by Frame BCF. The BRB joints of Frame RRF with AISI 12L14 carbon steel bar, before the rigid connection of the hinges, provides further evidence of the effect of the slack in the system. The frame experienced a maximum flexural rotation of  $12.1 \times 10^{-3}$  rad. After the

BRB joints were properly secured, retrofitted Frame RRF experienced limited rotations. This was attributed to a higher contribution of the BRB steel bar to the overall lateral load capacity (Figure 7.83 (b)) and, subsequently, a smaller contribution from the concrete frame (Figure 7.82 (b)). The frame experienced a maximum flexural rotation of  $6.5 \times 10^{-3}$  rad. Finally, Frame RRF using the AISI Type 304 stainless steel bar sustained the largest flexural rotations among all frames; the maximum rotation was  $13.5 \times 10^{-3}$  rad. This softer response was expected due to the multiple testing and the residual damage present in Frame RRF prior to testing with the AISI Type 304 stainless steel core bar.

Table 7.1: Sequence of Damage in Frame BCF

| Load Cycle | Lateral displacement (mm) | Lateral drift (%) | Lateral load capacity (kN) | Damage mode                                             |
|------------|---------------------------|-------------------|----------------------------|---------------------------------------------------------|
| 1          | 8                         | 0.25              | 83                         | -                                                       |
| 2          | 16                        | 0.5               | 125                        | -                                                       |
| 3          | 24                        | 0.75              | 165                        | Flexure - both column bases                             |
| 4          | 32                        | 1                 | 187                        | Flexure - far column adjacent to beam-column joint      |
| 5          | 48                        | 1.5               | 225                        | Flexure - beam supports                                 |
| 6          | 63.5                      | 2                 | 250                        | -                                                       |
| 7          | 79                        | 2.5               | 270                        | Flexure - near column adjacent to the beam-column joint |
| 8          | 95                        | 3                 | 250                        | -                                                       |
| 9          | 127                       | 4                 | 215                        | -                                                       |

Table 7.2: Lateral load- lateral displacement responses for all frame tests

| Test                           | Yield displacement (mm) | Lateral frame capacity at yield (kN) | Displacement capacity (mm) |      | Drift capacity (%) |      | Lateral load capacity, $F_u$ (kN) |      | Ductility capacity |      |
|--------------------------------|-------------------------|--------------------------------------|----------------------------|------|--------------------|------|-----------------------------------|------|--------------------|------|
|                                |                         |                                      | Push                       | Pull | Push               | Pull | Push                              | Pull | Push               | Pull |
| BCF                            | 41                      | 200                                  | 95                         | 95   | 3.0                | 3.0  | 233                               | 219  | 2.3                | 2.3  |
| RRF- C12L14 Cyclic             | 20                      | 309                                  | 48                         | 48   | 1.5                | 1.5  | 578                               | 621  | 2.4                | 2.4  |
| RRF- C12L14 Monotonic          | 16                      | 389                                  | 36                         | -    | 1.1                | -    | 577                               | -    | 2.3                | -    |
| RRF - AISI 304 Stainless Steel | 16                      | 305                                  | 79                         | 95   | 2.5                | 3.0  | 577                               | 850  | 4.9                | 5.9  |
| RF - AISI 4140                 | 16                      | 300                                  | 42                         | 48   | 1.3                | 1.5  | 510                               | 543  | 2.6                | 3.0  |

Table 7.3: Increase factors of lateral load capacity and ductility of retrofitted frames relative to Frame BCF

| Test                           | Ratio of $F_u$ (RRF, RF/ BCF) |      | Ratio of ductility (RRF, RF/ BCF) |      |
|--------------------------------|-------------------------------|------|-----------------------------------|------|
|                                | Push                          | Pull | Push                              | Pull |
| BCF                            | 1.00                          | 1.00 | 1.00                              | 1.00 |
| RRF- C12L14 Cyclic             | 2.48                          | 2.84 | 1.04                              | 1.04 |
| RRF- C12L14 Monotonic          | 2.48                          | -    | 0.97                              | -    |
| RRF - AISI 304 Stainless Steel | 2.48                          | 3.88 | 2.13                              | 2.56 |
| RF - AISI 4140                 | 2.19                          | 2.48 | 1.13                              | 1.29 |

Table 7.4: Initial and secant stiffness for all frame tests and the increase factors of retrofitted frames relative to frame BCF

| Test                           | Initial stiffness (kN/mm) | Secant stiffness at yielding (kN/mm) | Ratio of initial stiffness (RRF, RF/BCF) | Ratio of secant stiffness (RRF, RF/BCF) |
|--------------------------------|---------------------------|--------------------------------------|------------------------------------------|-----------------------------------------|
| BCF                            | 11.7                      | 4.9                                  | 1                                        | 1.0                                     |
| RRF- C12L14 Cyclic             | 12.5                      | 15.4                                 | 1.1                                      | 3.2                                     |
| RRF- C12L14 Monotonic          | 28.5                      | 24.3                                 | 2.4                                      | 5.0                                     |
| RRF - AISI 304 Stainless Steel | 29.0                      | 19.1                                 | 2.5                                      | 3.9                                     |
| RF - AISI 4140                 | 20.8                      | 18.8                                 | 1.8                                      | 3.8                                     |



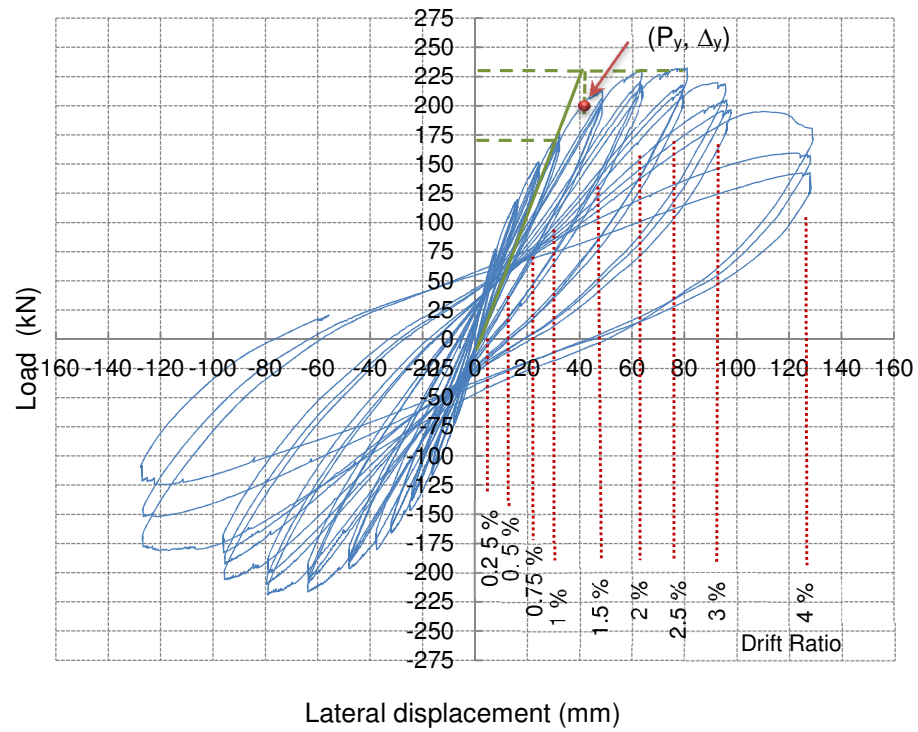
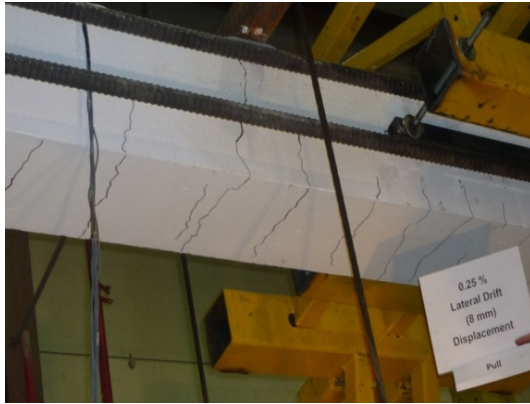


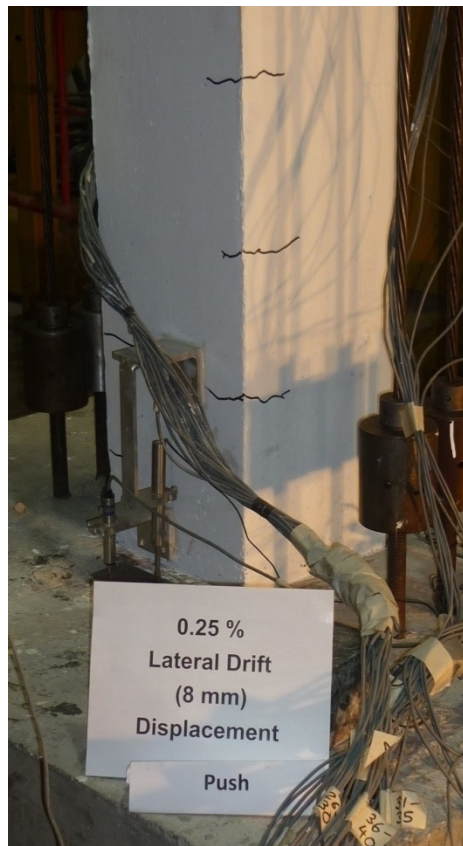
Figure 7.1: Hysteretic lateral load-lateral displacement response of Frame BCF (3 cycles for each drift level)



(a)



(b)

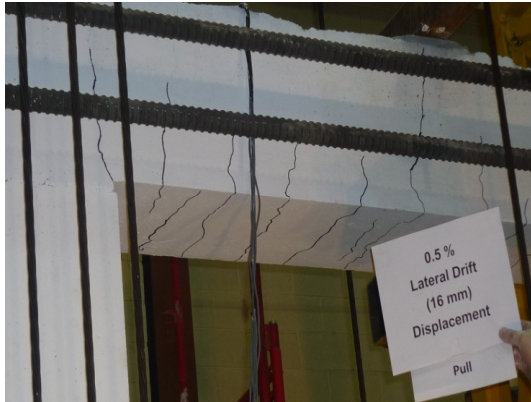


(c)



(d)

Figure 7.2: Flexural cracking at 0.25% lateral drift for Frame BCF: a) beam left side; b) beam right side; c) near column; and d) far column



(a)



(b)



(c)



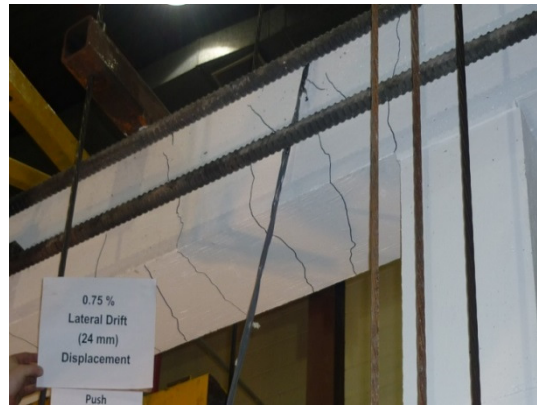
(d)

Figure 7.3: Propagation of flexural cracking and initiation of shear cracking at 0.5% lateral drift for Frame BCF: a) beam left side; b) beam right side; c) near column; and d) far column

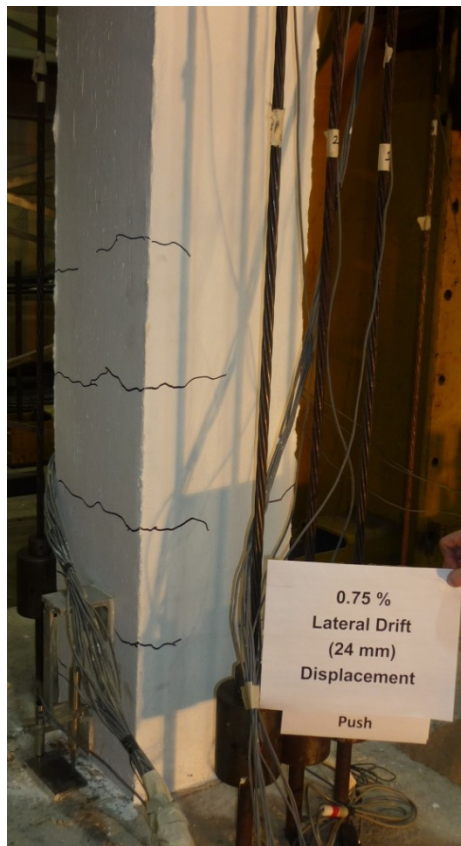




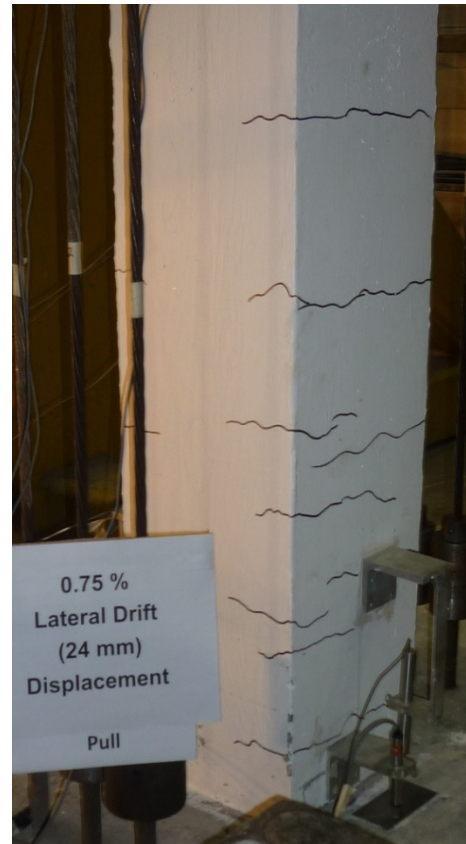
(a)



(b)

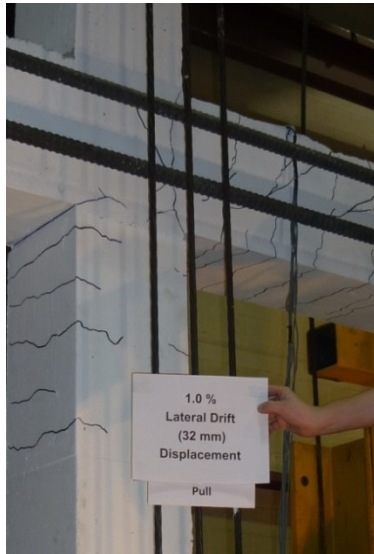


(c)

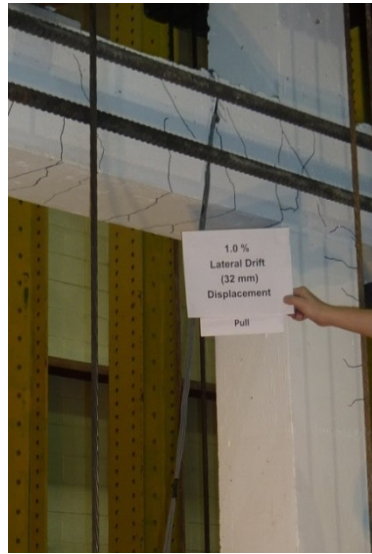


(d)

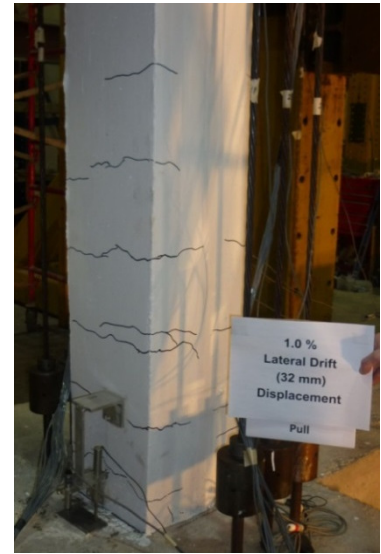
Figure 7.4: Flexural and shear cracking patterns at 0.75% lateral drift for Frame BCF:  
a) beam left side; b) beam right side; c) near column; and d) far column



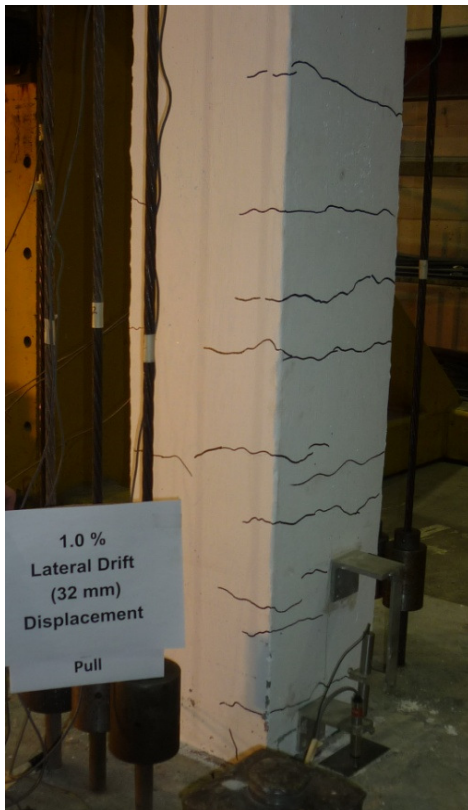
(a)



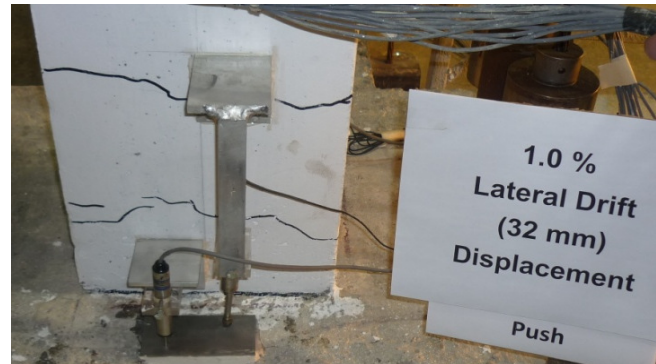
(b)



(c)



(d)



(e)



(f)

Figure 7.5: Initiation of shear cracking in the joints, flexural cracking at the top of the columns and foundation interface at 1.0% lateral drift for Frame BCF: a) frame joint left side; b) frame joint right side; c) near column; d) far column; e) base of near column; and f) base of far column





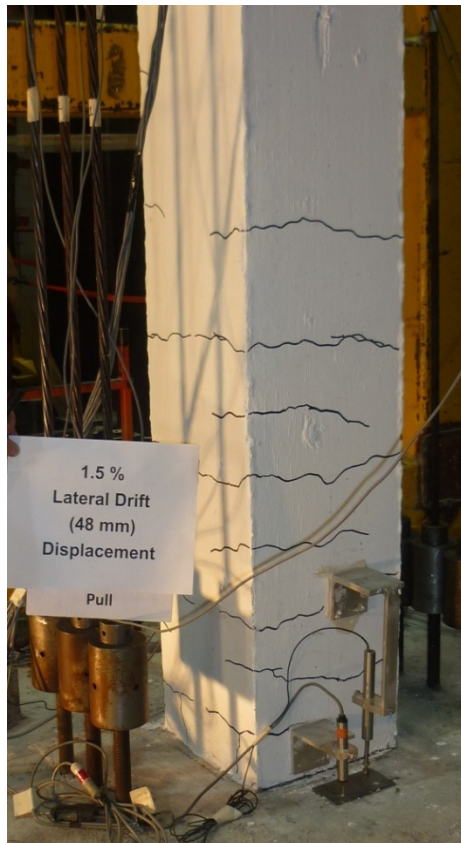
Figure 7.6: Global cracking pattern at 1.0% lateral drift for Frame BCF



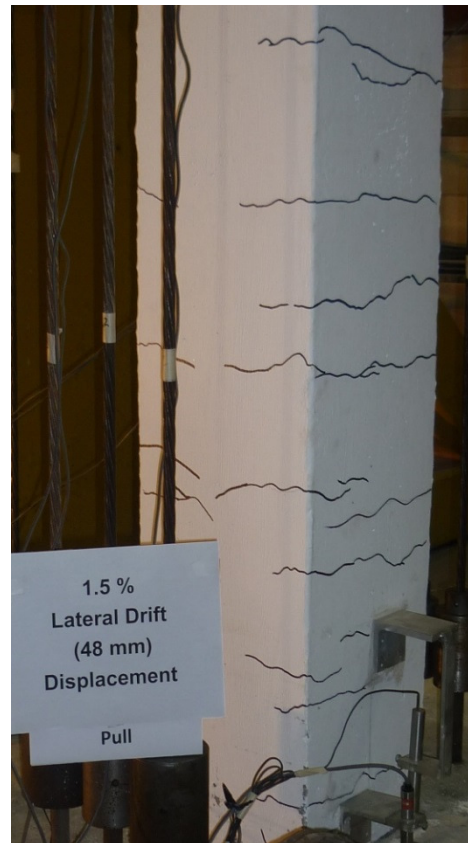
(a)



(b)



(c)



(d)

Figure 7.7: Widening of flexural and shear cracking at 1.5% lateral drift for Frame BCF:  
a) beam left side; b) beam right side; c) near column; and d) far column





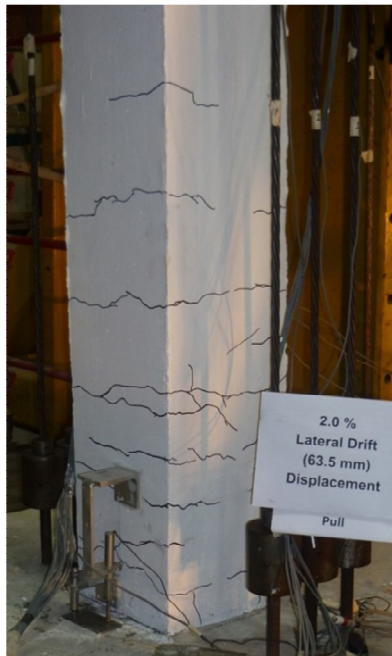
Figure 7.8: Cracking pattern at 1.5% lateral drift for Frame BCF



(a)



(b)



(c)



(d)

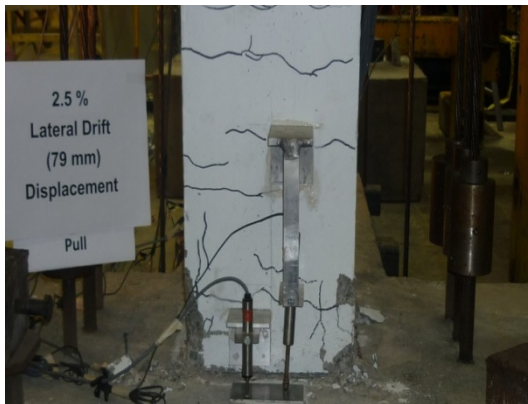
Figure 7.9: Flexural and shear cracking patterns at 2.0% lateral drift including initiation of vertical cracking at column bases for Frame BCF: a) frame joint left side; b) frame joint right side; c) near column; and d) far column



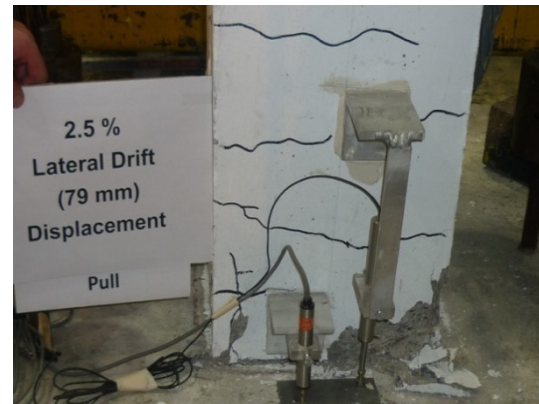
(a)



(b)



(c)



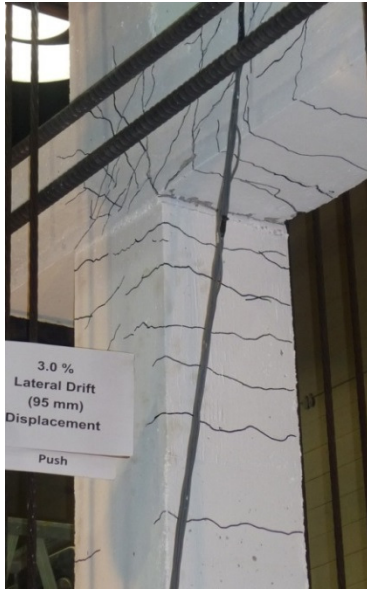
(d)

Figure 7.10: Propagation of flexural and shear cracking, and initiation of column concrete cover spalling at 2.5% lateral drift for Frame BCF: a) frame joint left side; b) frame joint right side; c) near column base; and d) far column base





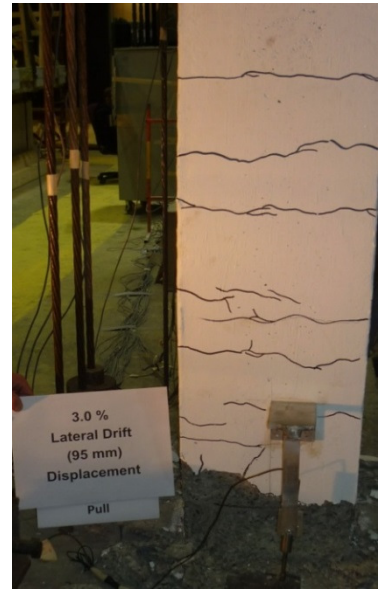
Figure 7.11: Cracking pattern at 2.5% lateral drift for Frame BCF



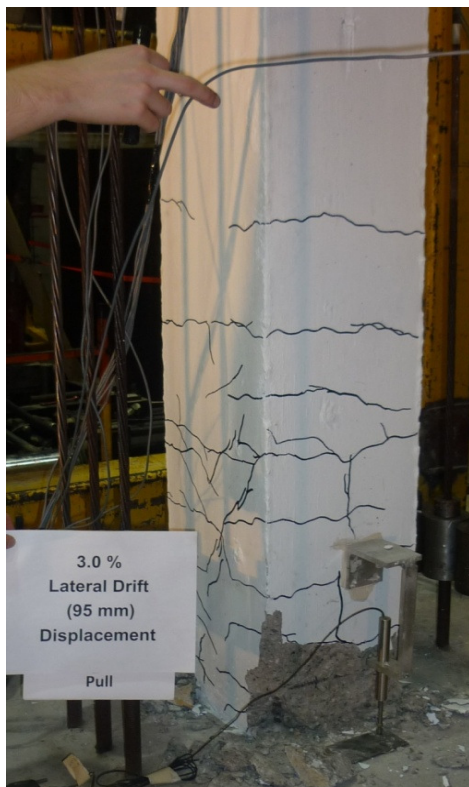
(a)



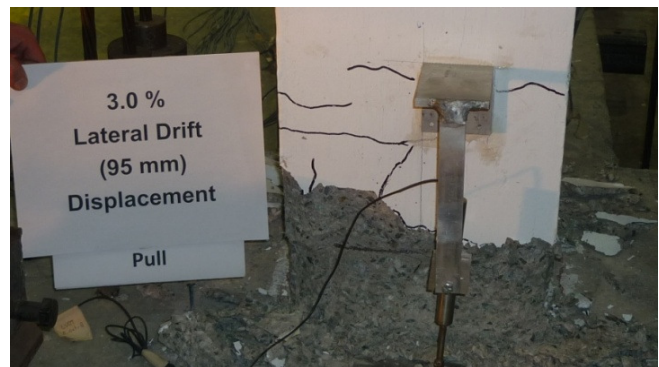
(b)



(c)



(d)



(e)



(f)

Figure 7.12: Widening of beam shear cracking and exposure of reinforcement at base of columns at 3.0% lateral drift for Frame BCF: a) frame joint left side; b) frame joint right side; c) near column; d) far column; e) base of near column; and f) base of far column





Figure 7.13: Cracking pattern at 3.0% lateral drift for Frame BCF



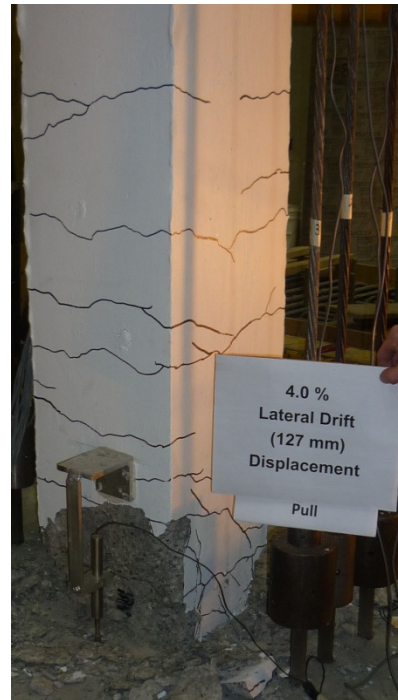
(a)



(b)



(c)



(d)

Figure 7.14: Damaged frame highlighting concrete cover spalling at 4.0% drift for Frame BCF: a) frame joint left side; b) frame joint right side; c) near column; and d) far column





(a)



(b)



(c)

Figure 7.15: Shear cracking in beam and exposed longitudinal bars in columns at 4.0% lateral drift for Frame BCF: a) frame joint left side; b) base of near column; and c) base of far column



Figure 7.16: Cracking pattern and damage at 4.0% lateral drift for Frame BCF

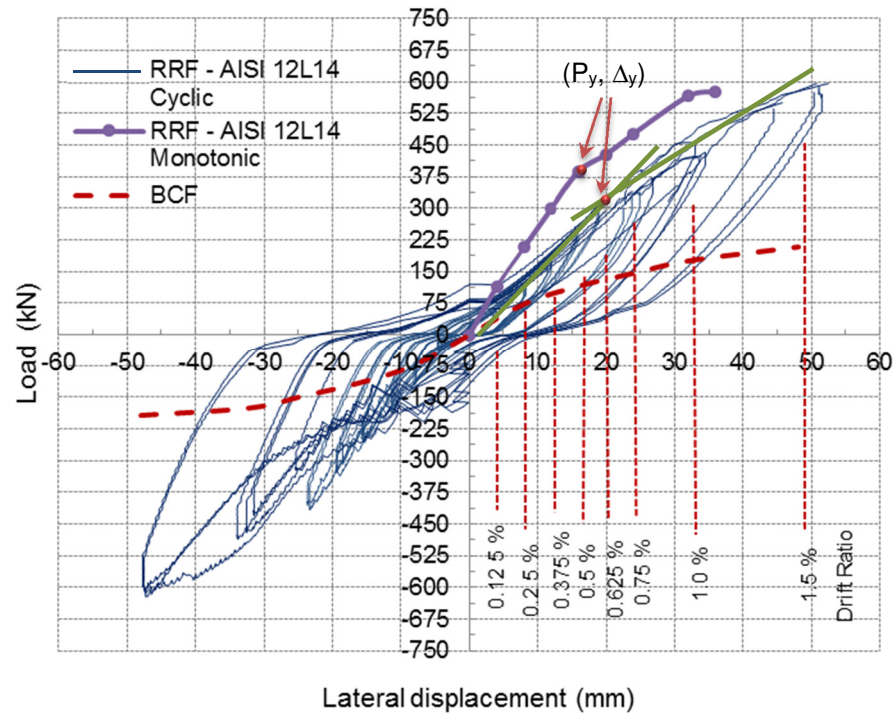


Figure 7.17: Hysteretic lateral load-lateral displacement response for repaired retrofitted frame with AISI 12L14 carbon steel BRB core bar (3 cycles for each drift level)





(a)



(b)



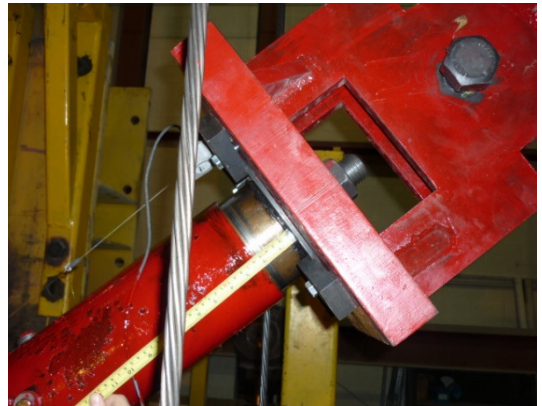
(c)



(d)



(e)



(f)

Figure 7.18: Frame RRF (AISI 12L14) at 0.125% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



Fig. 7.19: Global view at 0.125% lateral drift for Frame RRF (AISI 12L14)

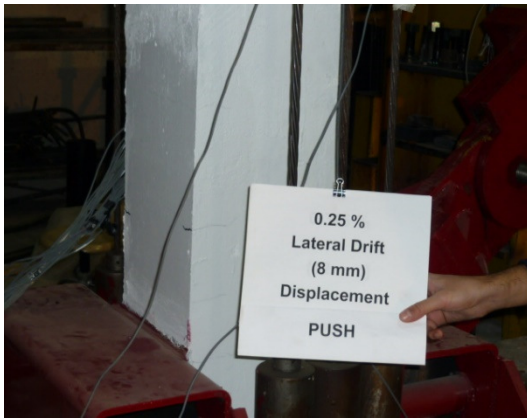




(a)



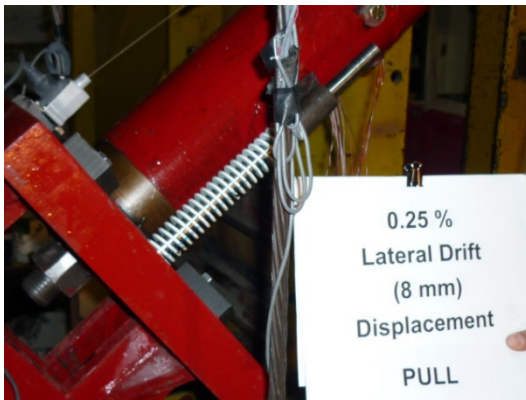
(b)



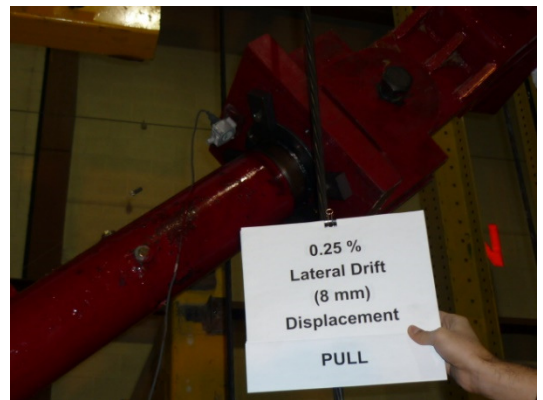
(c)



(d)



(e)



(f)

Figure 7.20: Frame RRF (AISI 12L14) at 0.25% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

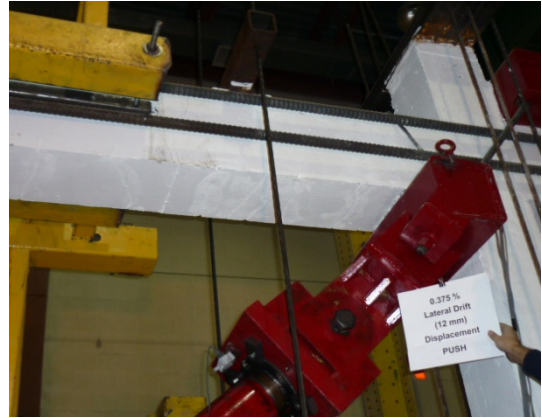


Figure 7.21: Global view at 0.25% lateral drift for Frame RRF (AISI 12L14)





(a)



(b)

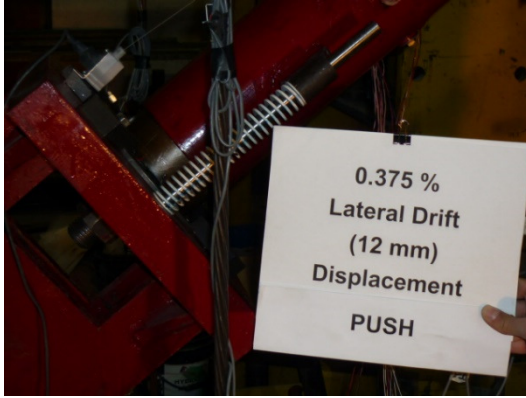


(c)

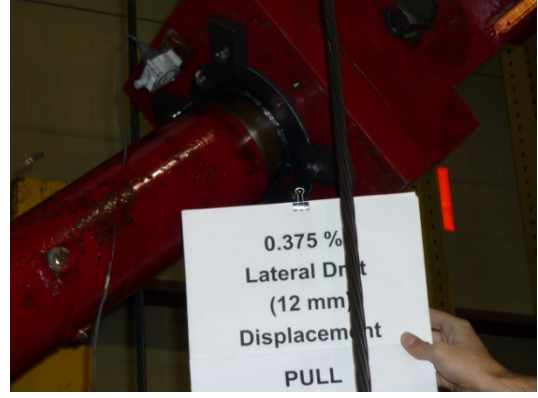


(d)

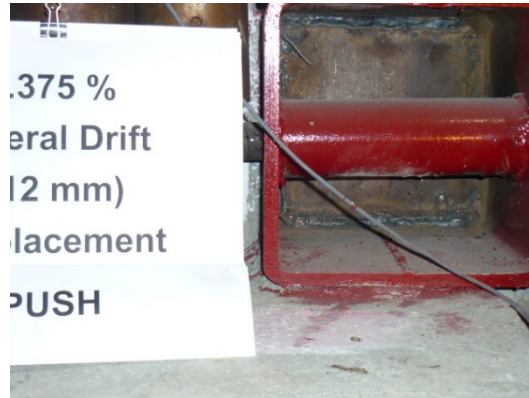
Figure 7.22: Frame RRF (AISI 12L14) at 0.375% lateral drift: a) beam left side; b) beam right side; c) near column; and d) far column



(e)



(f)



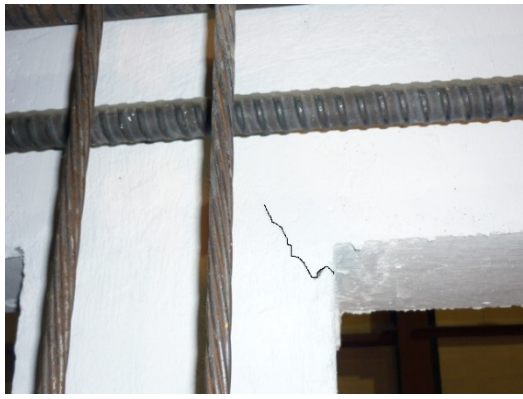
(g)

Figure 7.22 (Cont'd): Frame RRF (AISI 12L14) at 0.375% lateral drift: e) external gap at lower joint; f) external gap at upper joint; and g) gap opening between HSS of the BRB hinge and near column



Figure 7.23: Global view at 0.375% lateral drift for Frame RRF (AISI 12L14)





(a)



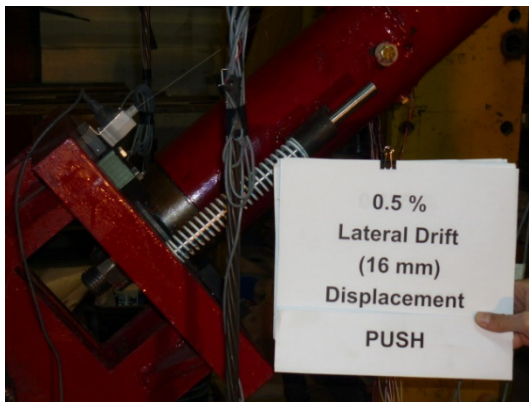
(b)



(c)



(d)



(e)



(f)

Figure 7.24: Initiation of shear cracking, cracking pattern of Frame RRF (AISI 12L14), and BRB external gaps at 0.5% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

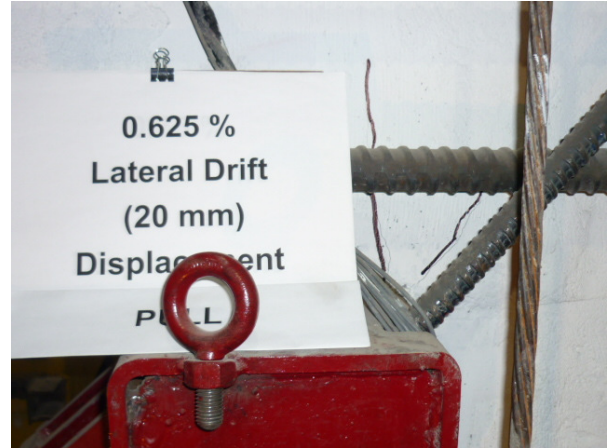


Figure 7.25: Global view at 0.5% lateral drift for Frame RRF (AISI 12L14)

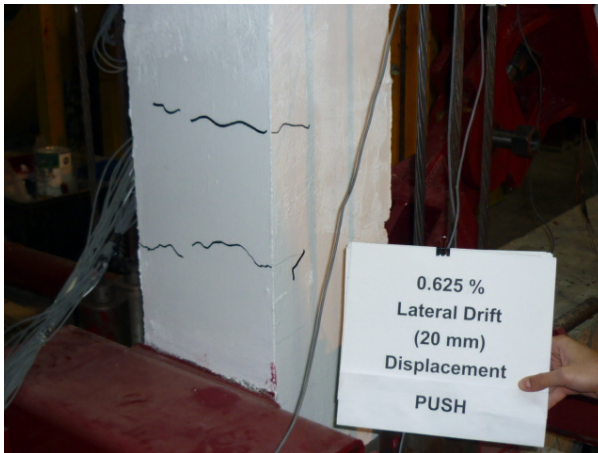




(a)



(b)



(c)



(d)

Figure 7.26: Initiation of shear cracking at far beam-column joints and flexural cracking at column bases of Frame RRF (AISI 12L14) at 0.625% lateral drift: a) beam left side; b) beam right side; c) near column; and d) far column



Figure 7.27: Global view at 0.625% lateral drift for Frame RRF (AISI 12L14)

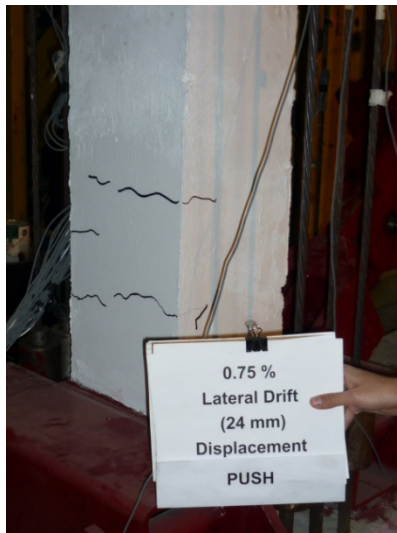




(a)



(b)



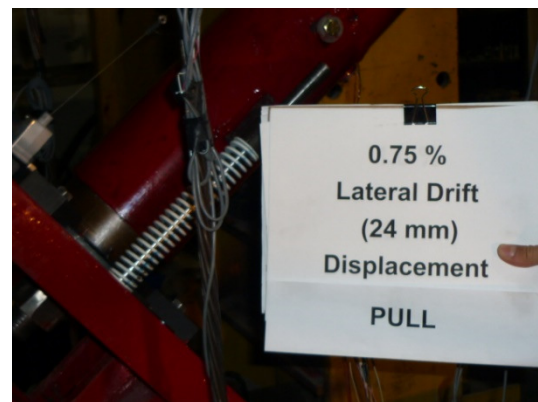
(c)



(d)



(e)



(f)

Figure 7.28: Frame RRF (AISI 12L14) at 0.75% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



Figure 7.29: Global view at 0.75% lateral drift for Frame RRF (AISI 12L14)



(a)



(b)



(c)



(d)



(e)



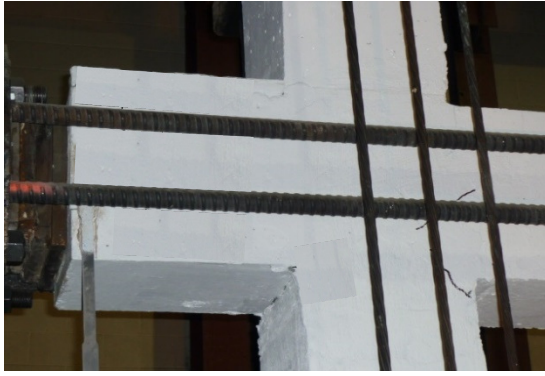
(f)

Figure 7.30: Flexural and shear cracking patterns and BRB external gaps at 1.0% lateral drift for Frame RRF (AISI 12L14): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

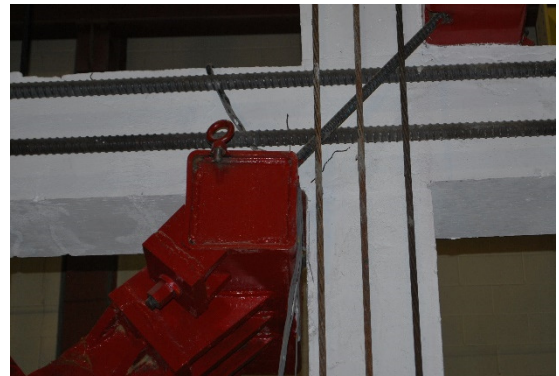




Figure 7.31: Global view at 1.0% lateral drift for Frame RRF (AISI 12L14)



(a)



(b)



(c)



(d)

Figure 7.32: Frame RRF (AISI 12L14) at 1.5% lateral drift: a) beam left side; b) beam right side; c) near column; and d) far column



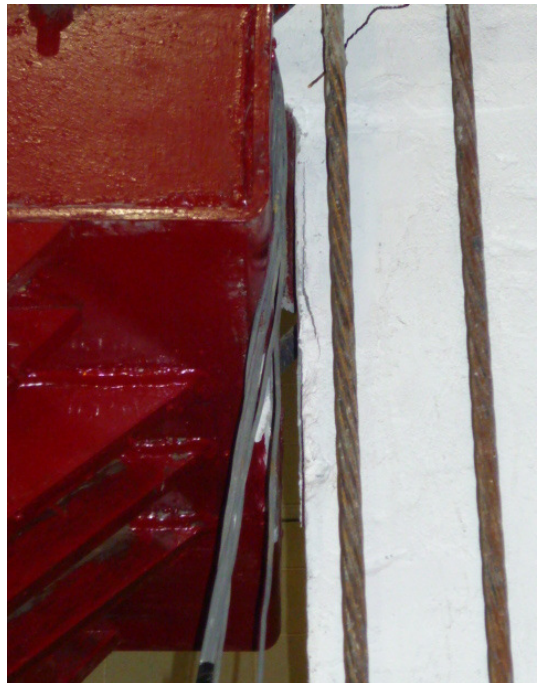
(e)



(f)



(g)



(h)

Figure 7.32 (Cont'd): Frame RRF (AISI 12L14) at 1.5% lateral drift: e) external gap at lower joint; f) external gap at upper joint; g) uplift of lower HSS joint; and h) gap between the upper joint and beam-column joint

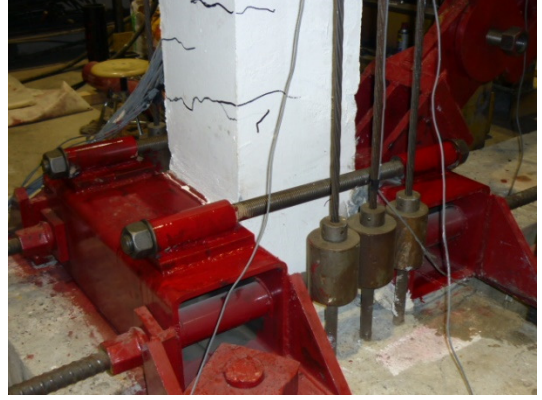




Figure 7.33: Global view at 1.5% lateral drift for Frame RRF (AISI 12L14)



(a)



(b)

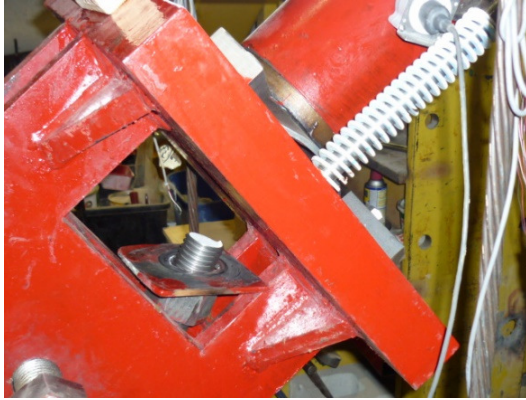


(c)



(d)

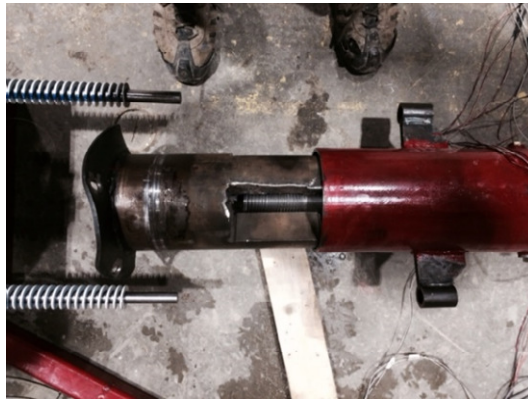
Figure 7.34: Dismantling and stiffening process of the BRB system of Frame RRF (AISI 12L14): a) dismantled BRB components; b) stiffening of lower joint connection; c) stiffening of upper joint connection; and d) stiffened BRB inside the frame



(a)



(b)



(c)



(d)

Figure 7.35: Fracture of the BRB steel core bar of Frame RRF (AISI 12L14): a) bar fracture during testing; b) sectional view of the fractured bar; c) twisted end plate of steel core restraining end element; and d) isometric view of the twisted plate of steel core restraining end element



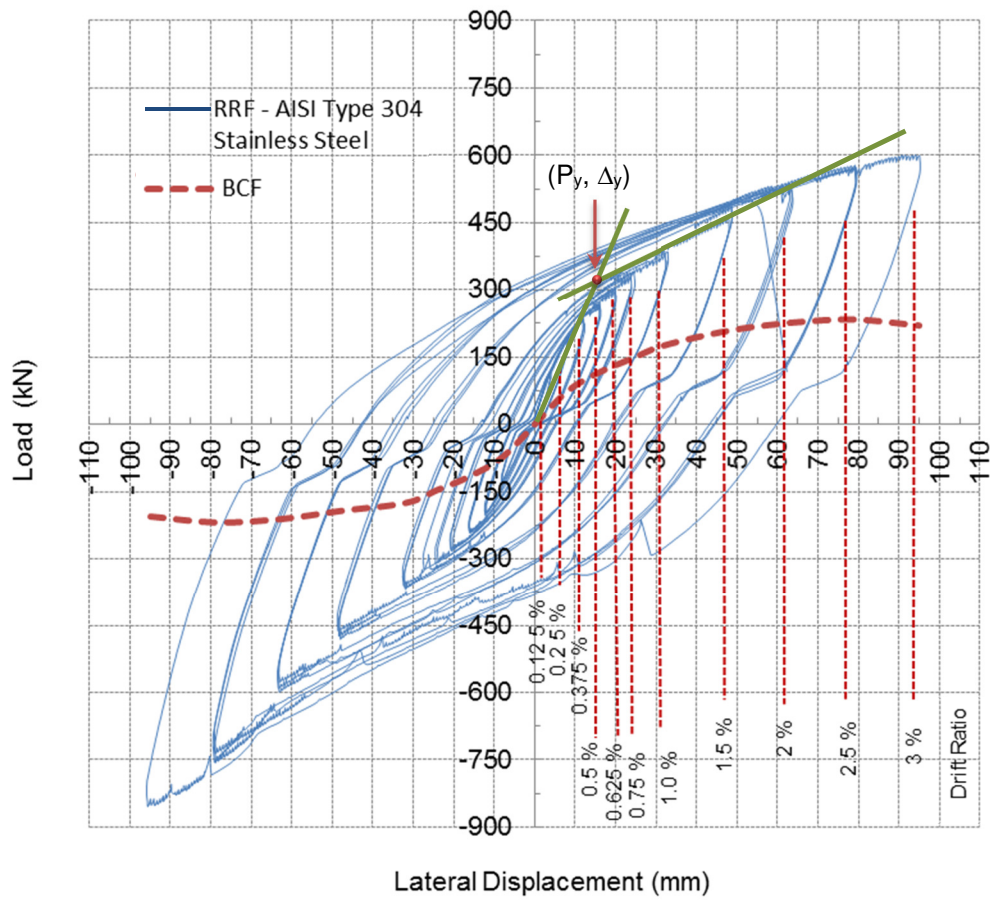


Figure 7.36: Hysteretic lateral load-lateral displacement response for repaired retrofitted frame using AISI Type 304 stainless steel BRB core bar (3 cycles for each drift level)



(a)



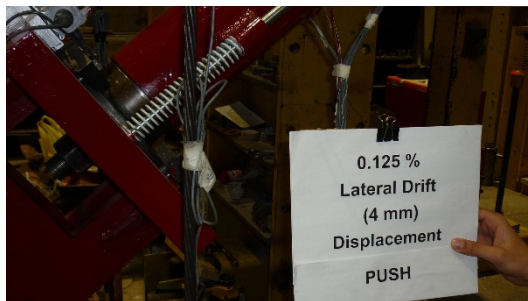
(b)



(c)



(d)



(e)



(f)

Figure 7.37: Frame RRF (AISI 304 stainless steel) at 0.125% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



Figure 7.38: Global view at 0.125% lateral drift for Frame RRF (AISI 304 stainless steel)





(a)



(b)



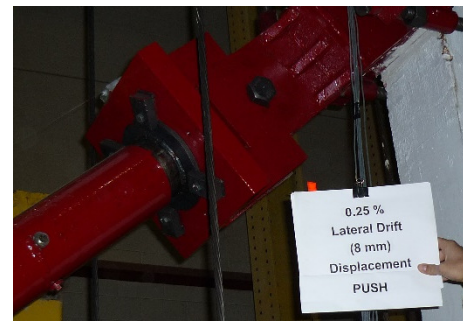
(c)



(d)



(e)



(f)

Figure 7.39: Initiation of shear cracking at the near joint and frame behaviour of Frame RRF (AISI 304 stainless steel) at 0.25% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



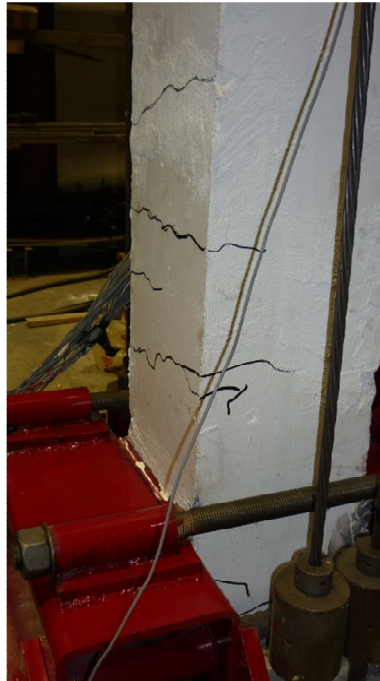
Figure 7.40: Global view at 0.25% lateral drift for Frame RRF (AISI 304 stainless steel)



(a)



(b)



(c)



(d)

Figure 7.41: Initiation of flexural and shear cracking at the beam left side and frame cracking pattern of Frame RRF (AISI 304 stainless steel) at 0.375% lateral drift: a) beam left side; b) beam right side; c) near column; and d) far column





Figure 7.42: Cracking pattern at 0.375% lateral drift for Frame RRF (AISI 304 stainless steel)



(a)



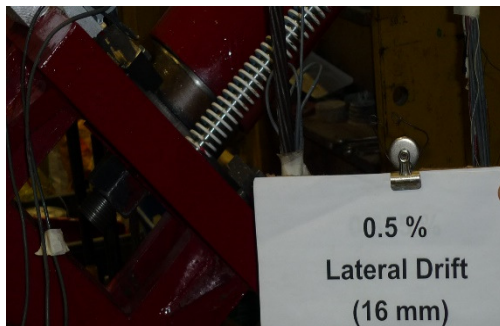
(b)



(c)



(d)



(e)



(f)

Figure 7.43: Propagation of flexural and shear cracking at the near joint and initiation of flexural cracking at column bases and BRB external gaps at 0.5% lateral drift of Frame RRF (AISI 304 stainless steel): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

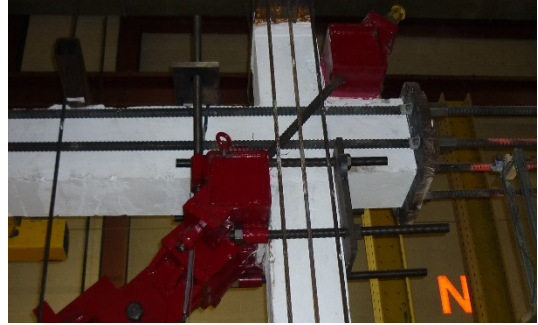


Figure 7.44: Cracking pattern at 0.5% lateral drift for Frame RRF (AISI 304 stainless steel)

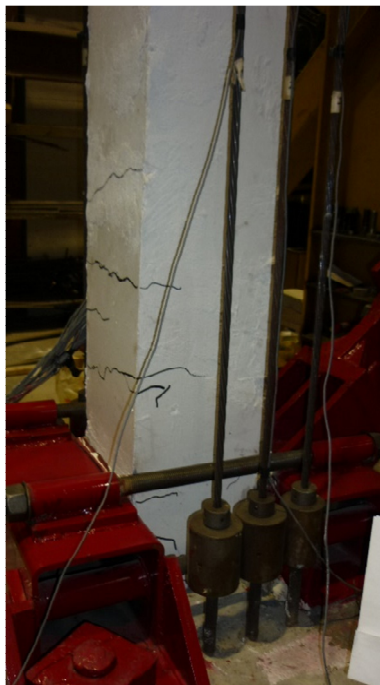




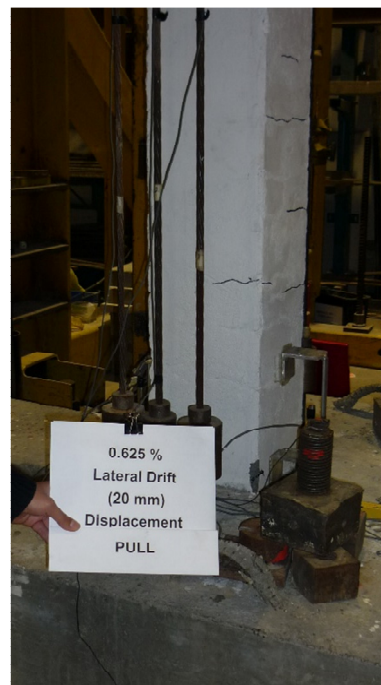
(a)



(b)



(c)



(d)

Figure 7.45: Frame RRF (AISI 304 stainless steel) at 0.625% lateral drift: a) beam left side; b) beam right side; c) near column; and d) far column

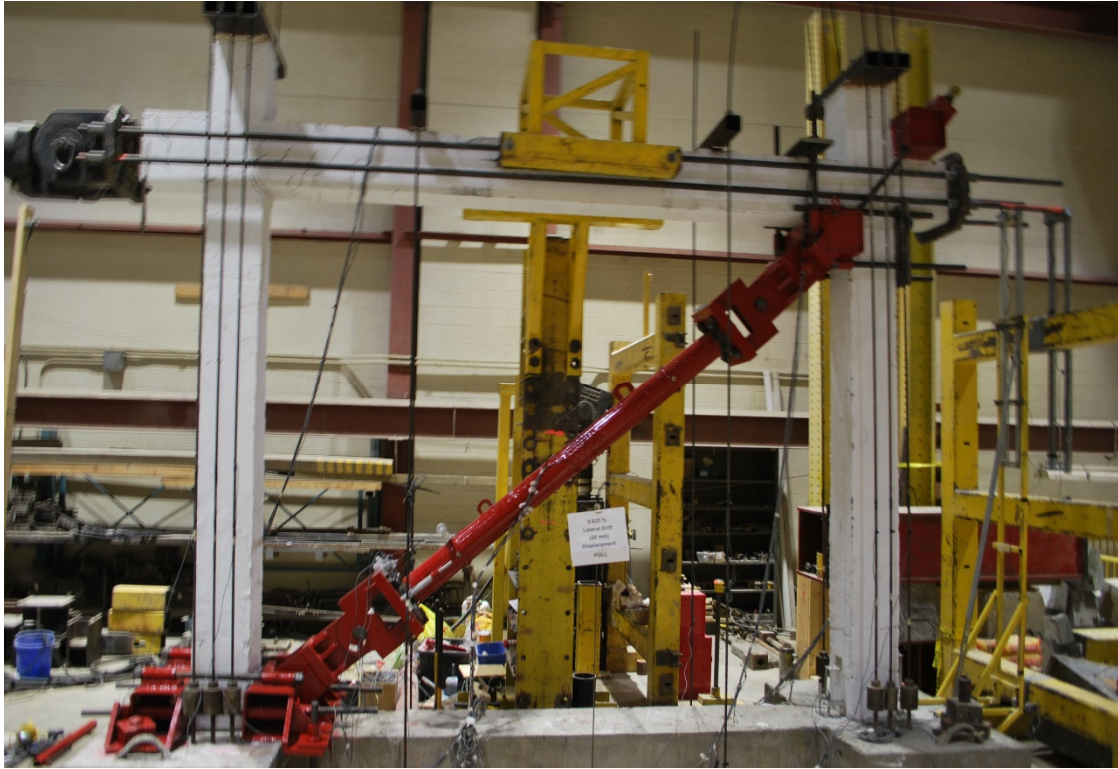


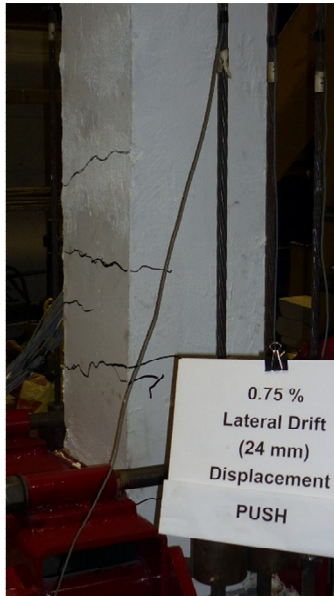
Figure 7.46: Cracking pattern at 0.625% lateral drift for Frame RRF (AISI 304 stainless steel)



(a)



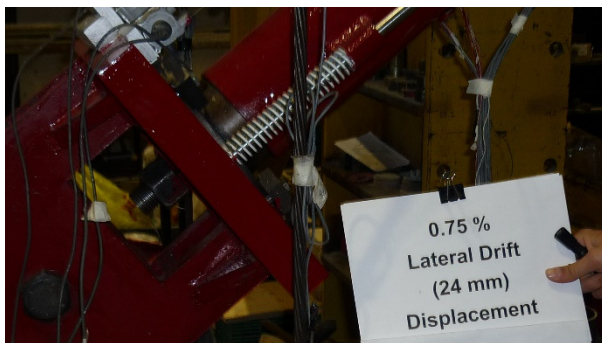
(b)



(c)



(d)



(e)



(f)

Figure 7.47: Propagation of flexural and shear cracking and BRB external gaps at 0.75% lateral drift of Frame RRF (AISI 304 stainless steel): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



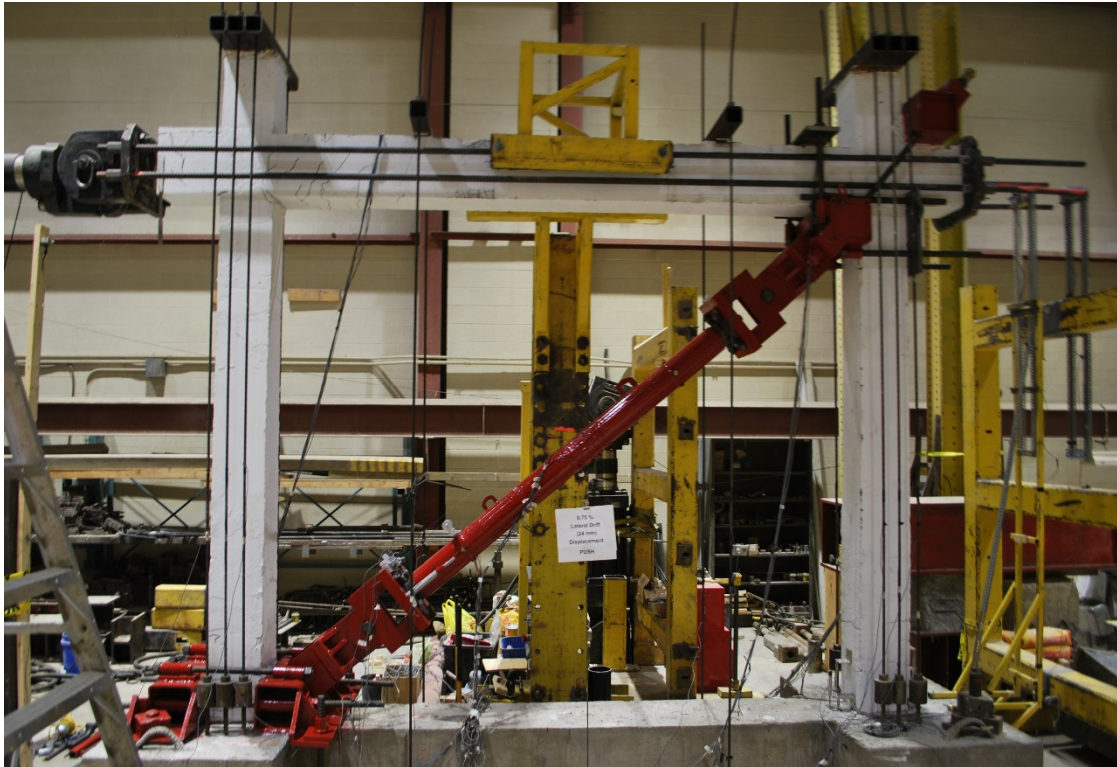
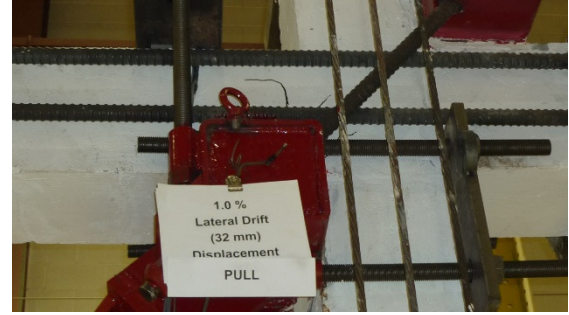


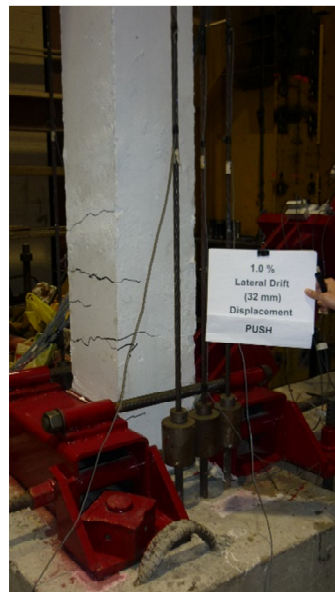
Figure 7.48: Cracking pattern at 0.75% lateral drift for Frame RRF (AISI 304 stainless steel)



(a)



(b)



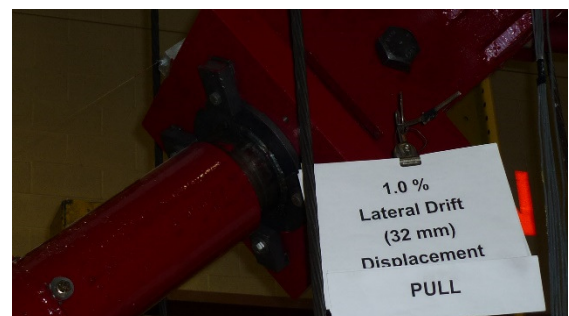
(c)



(d)



(e)



(f)

Figure 7.49: Widening of flexural and shear cracks at beam joints and BRB external gaps at 1.0% lateral drift of Frame RRF (AISI 304 stainless steel): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

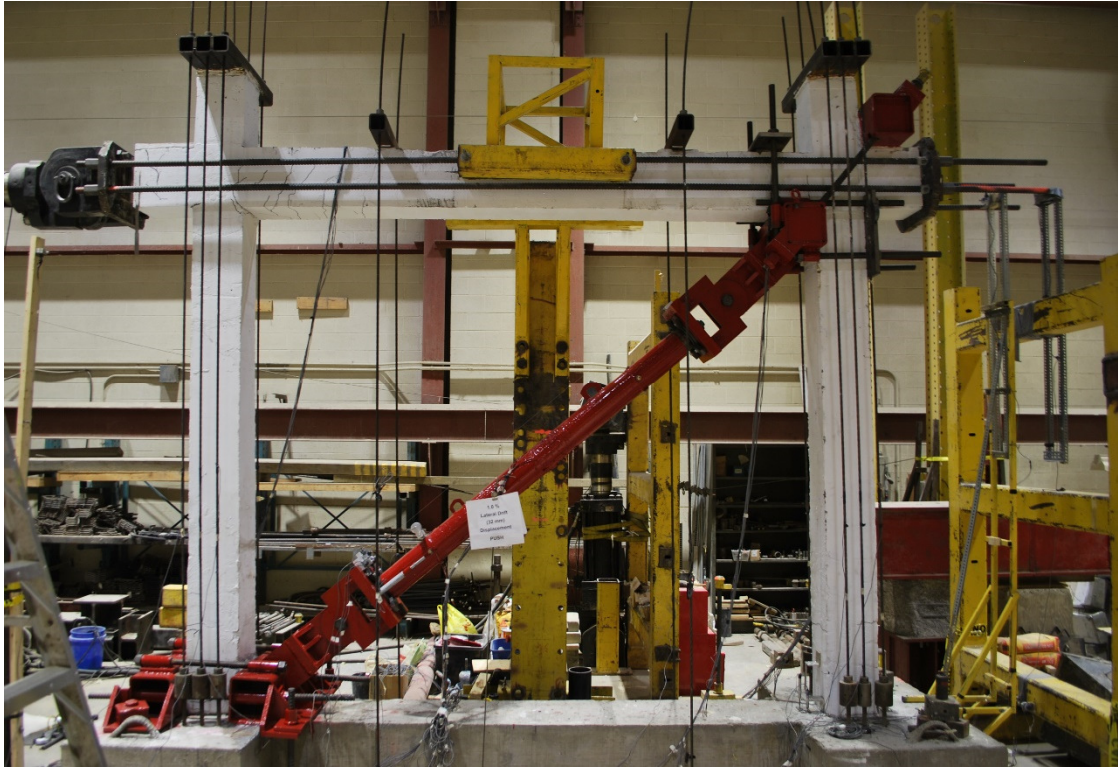


Figure 7.50: Cracking pattern at 1.0% lateral drift for Frame RRF (AISI 304 stainless steel)





(a)



(b)



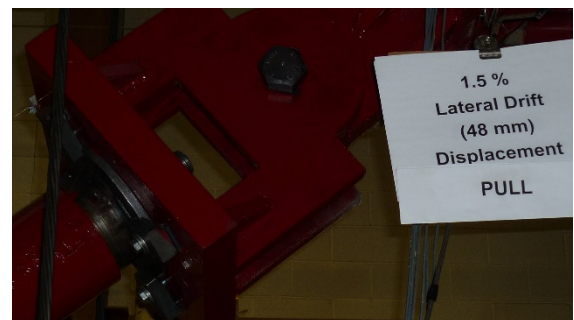
(c)



(d)



(e)



(f)

Figure 7.51: Increase of flexural cracking at near column, shear cracking at right beam joint, and BRB external gaps at 1.5% lateral drift of Frame RRF (AISI 304 stainless steel): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

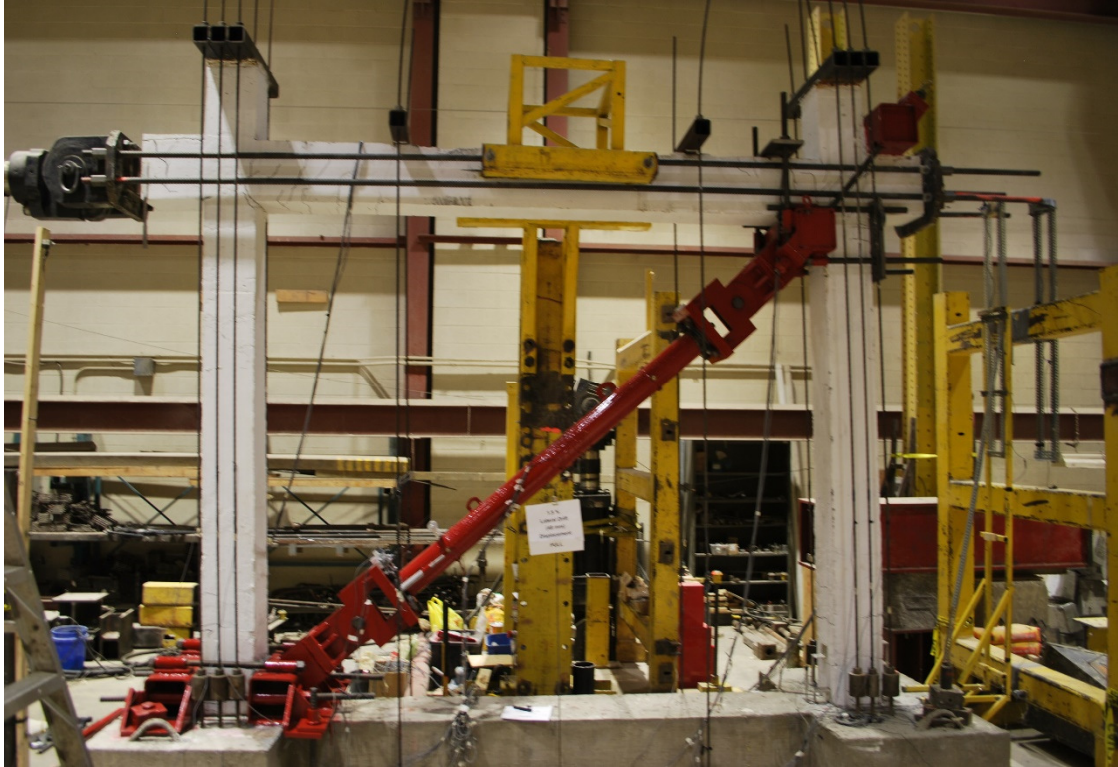


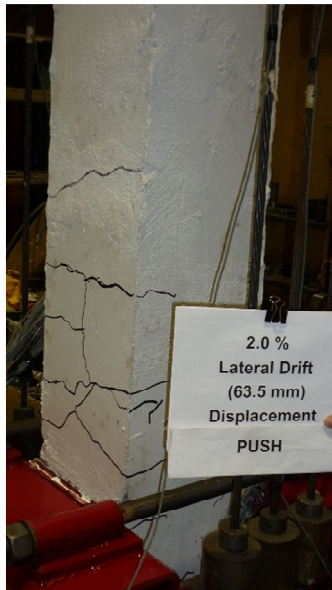
Figure 7.52: Cracking pattern at 1.5% lateral drift for Frame RRF (AISI 304 stainless steel)



(a)



(b)



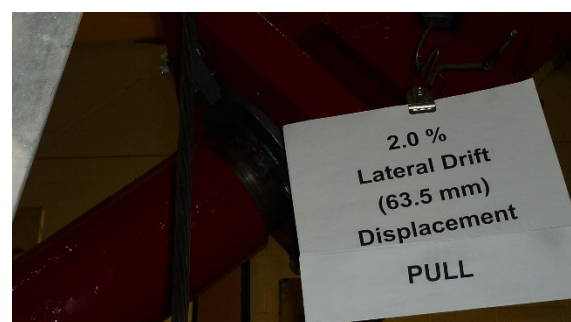
(c)



(d)



(e)



(f)

Figure 7.53: Propagation of flexural and shear cracking, initiation of column concrete cover spalling, and BRB external gaps at 2.0% lateral drift of Frame RRF (AISI 304 stainless steel): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint





Figure 7.54: Cracking pattern at 2% lateral drift for Frame RRF (AISI 304 stainless steel)



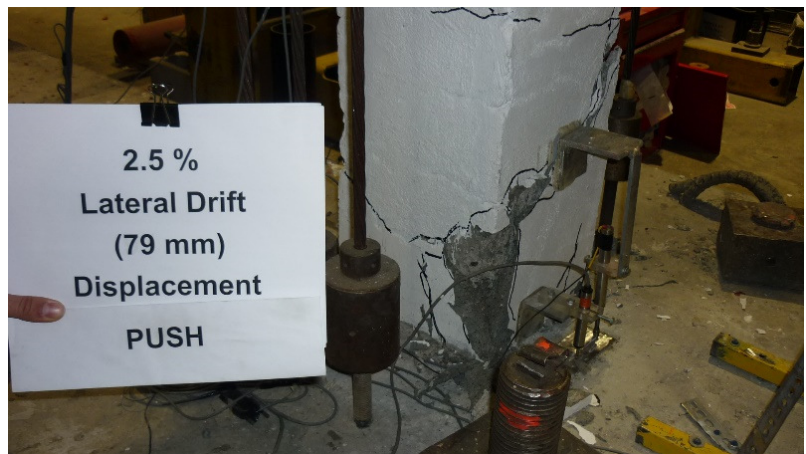
(a)



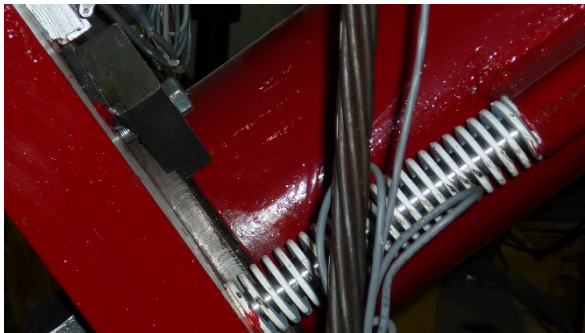
(b)



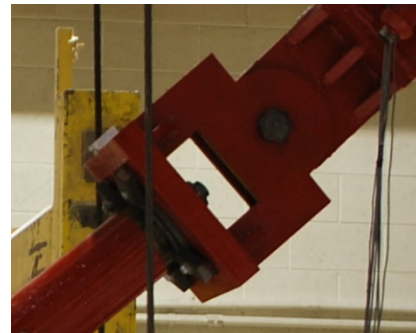
(c)



(d)



(e)



(f)

Figure 7.55: Widening of flexural and shear cracking, propagation of column concrete cover spalling, and BRB external gaps at 2.5% lateral drift of Frame RRF (AISI 304 stainless steel): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



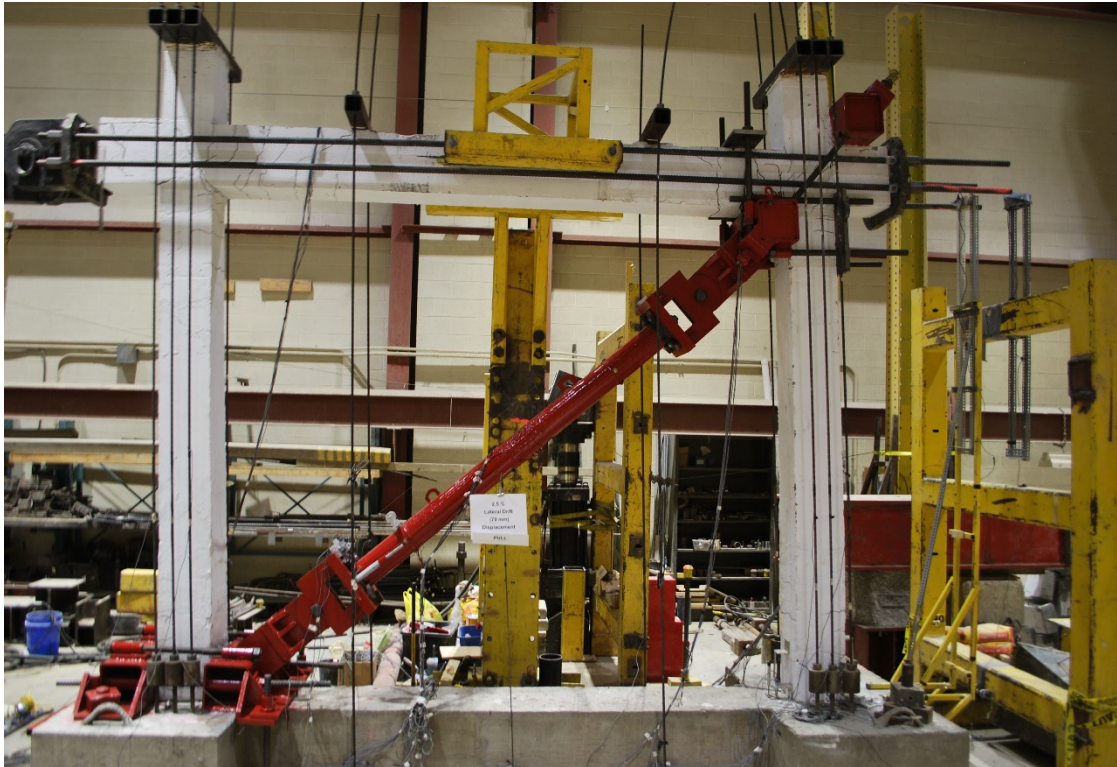


Figure 7.56: Cracking pattern at 2.5% lateral drift for Frame RRF (AISI 304 stainless steel)





(a)



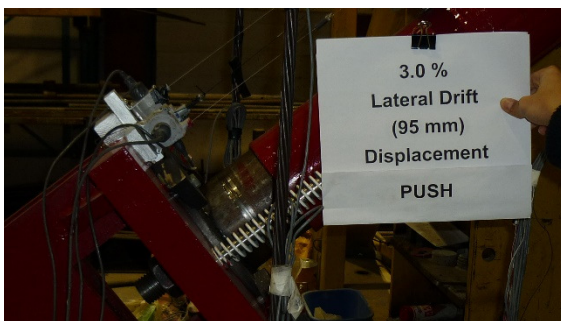
(b)



(c)



(d)



(e)

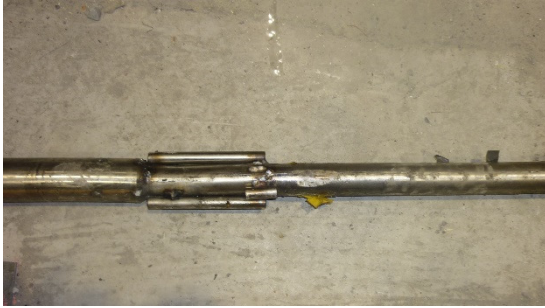


(f)

Figure 7.57: Propagation of flexural and shear cracking and extensive column concrete cover spalling and BRB external gaps at 3.0% lateral drift of Frame RRF (AISI 304 stainless steel): a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



Figure 7.58: Cracking pattern at 3.0% lateral drift for Frame RRF (AISI 304 stainless steel)



(a)



(b)



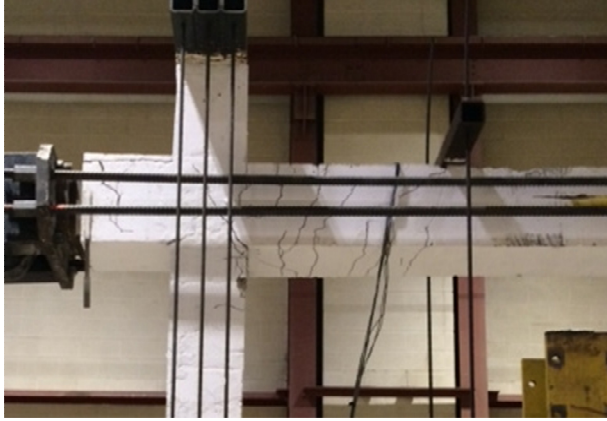
(c)



(d)

Figure 7.59: BRB core bar of Frame RRF (AISI 304 stainless steel) at failure: a) BRB core bar left side; b) BRB core bar right side; c) rupture of BRB core bar; and d) BRB core bar and the spacers





(a)



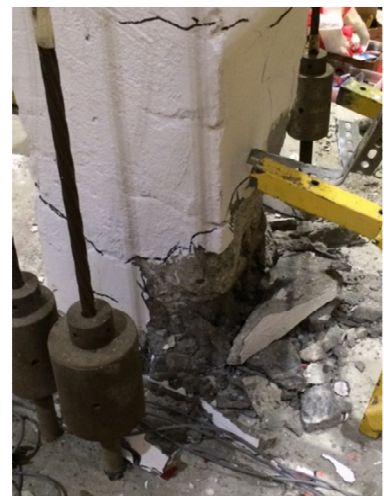
(b)



(c)



(d)



(e)

Figure 7.60: Cracking pattern for Frame RRF (AISI 304 stainless steel) after BRB removal: a) beam left side; b) beam right side; c) near column outer face; d) near column inner face; and e) far column outer face



Figure 7.61: Global view of Frame RRF (AISI 304 stainless steel) after BRB removal

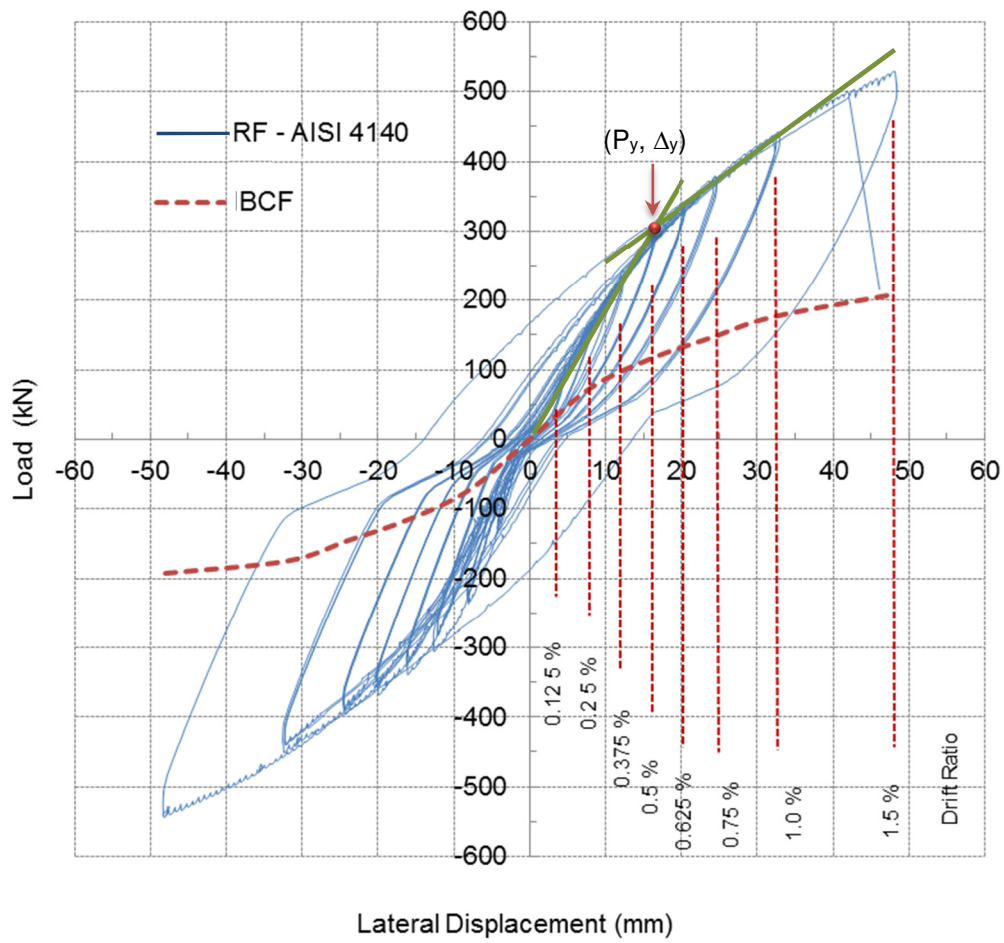


Figure 7.62: Hysteretic lateral load-lateral displacement response for the retrofitted frame using AISI 4140 chrome-molybdenum high tensile steel BRB (3 cycles for each drift level)





(a)



(b)



(c)



(d)



(e)

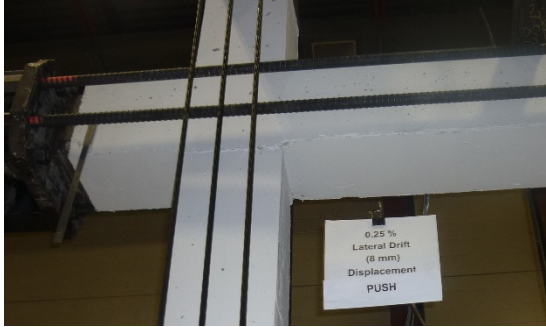


(f)

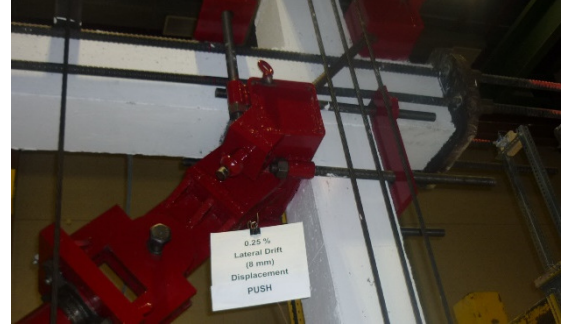
Figure 7.63: Frame RF behaviour at 0.125% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



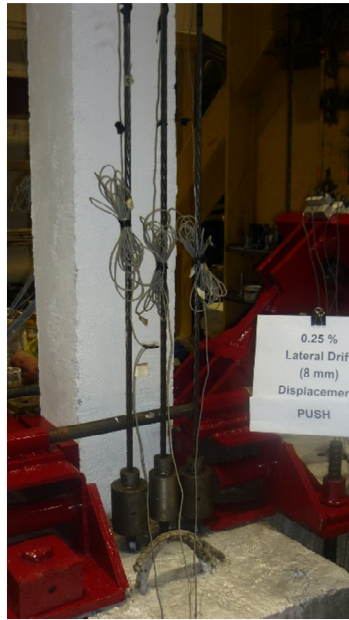
Figure 7.64: Global view at 0.125% lateral drift for Frame RF



(a)



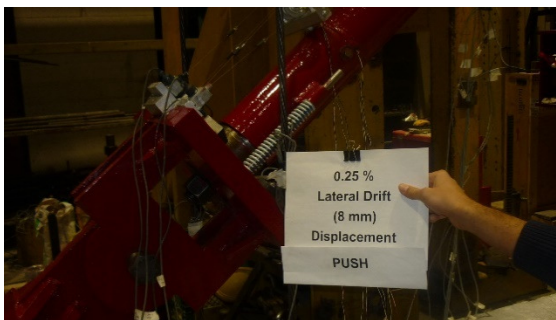
(b)



(c)



(d)



(e)



(f)

Figure 7.65: Frame RF behaviour at 0.25% lateral drift: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

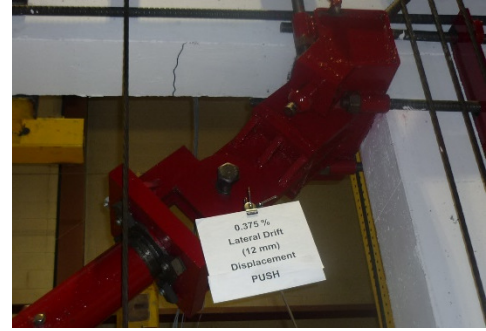




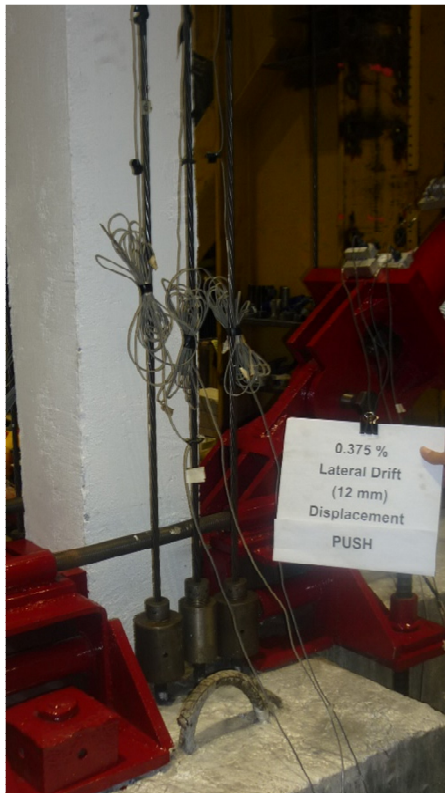
Figure 7.66: Global view at 0.25% lateral drift for Frame RF



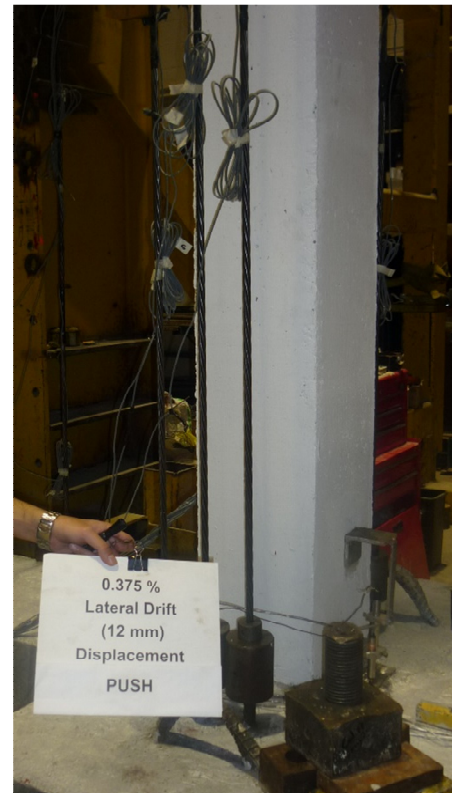
(a)



(b)



(c)



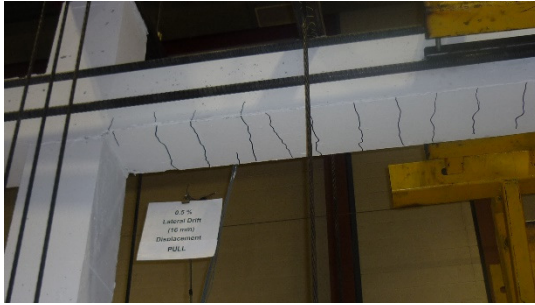
(d)

Figure 7.67: Initiation of flexural cracking at the beam soffit and frame crack pattern at 0.375% lateral drift for Frame RF: a) beam left side; b) beam right side; c) near column; and d) far column

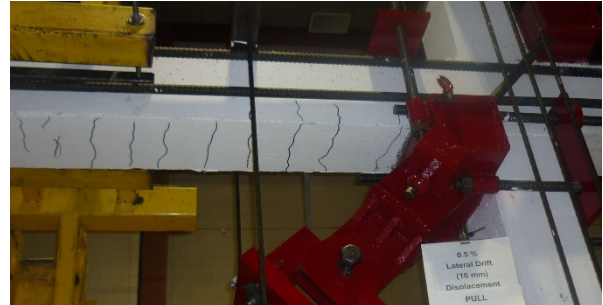


Figure 7.68: Cracking pattern at 0.375% lateral drift for Frame RF





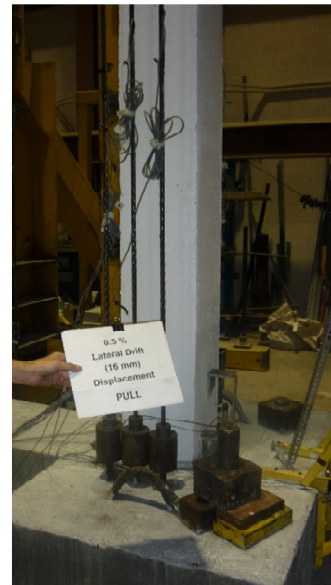
(a)



(b)



(c)



(d)



(e)



(f)

Figure 7.69: Propagation of flexural cracking at the beam and BRB external gaps at 0.5% lateral drift of Frame RF: a) beam left side; b) beam right side; c) near column; d) far column; (e) external gap at lower joint; and f) external gap at upper joint

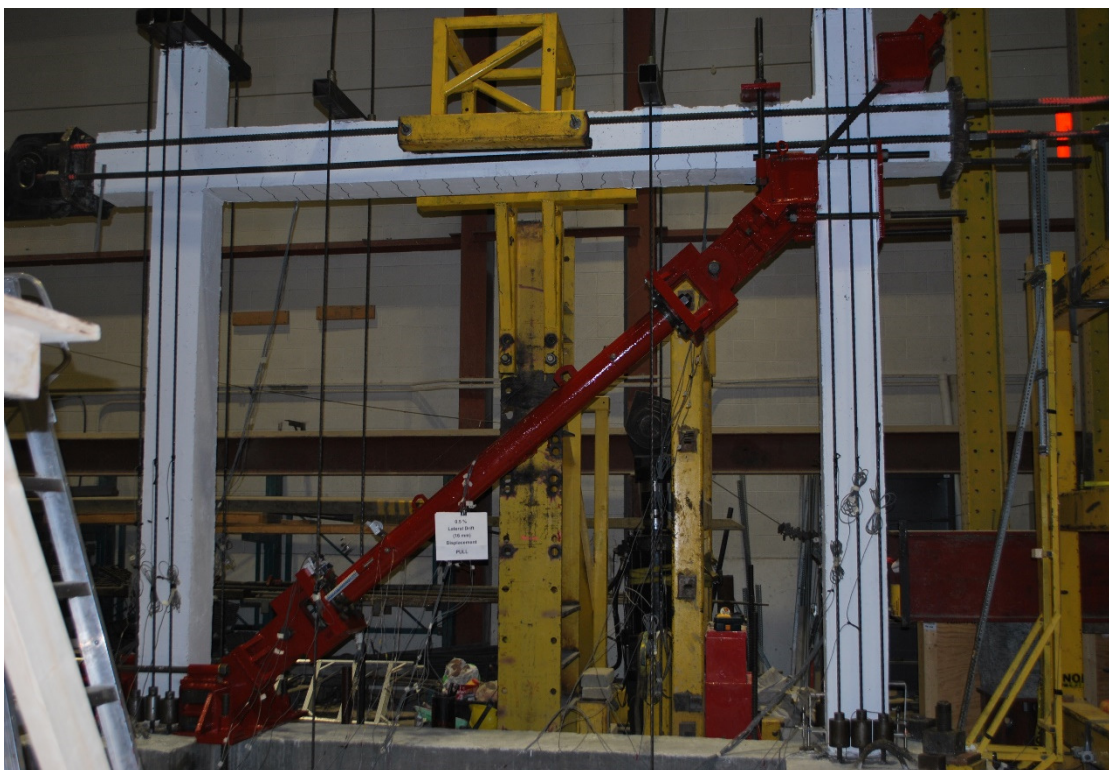
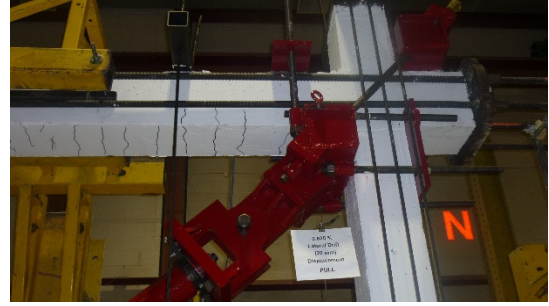


Figure 7.70: Cracking pattern at 0.5% lateral drift for Frame RF



(a)



(b)



(c)



(d)

Figure 7.71: Frame RF cracking pattern at 0.625% lateral drift: a) beam left side; b) beam right side; c) near column; and d) far column



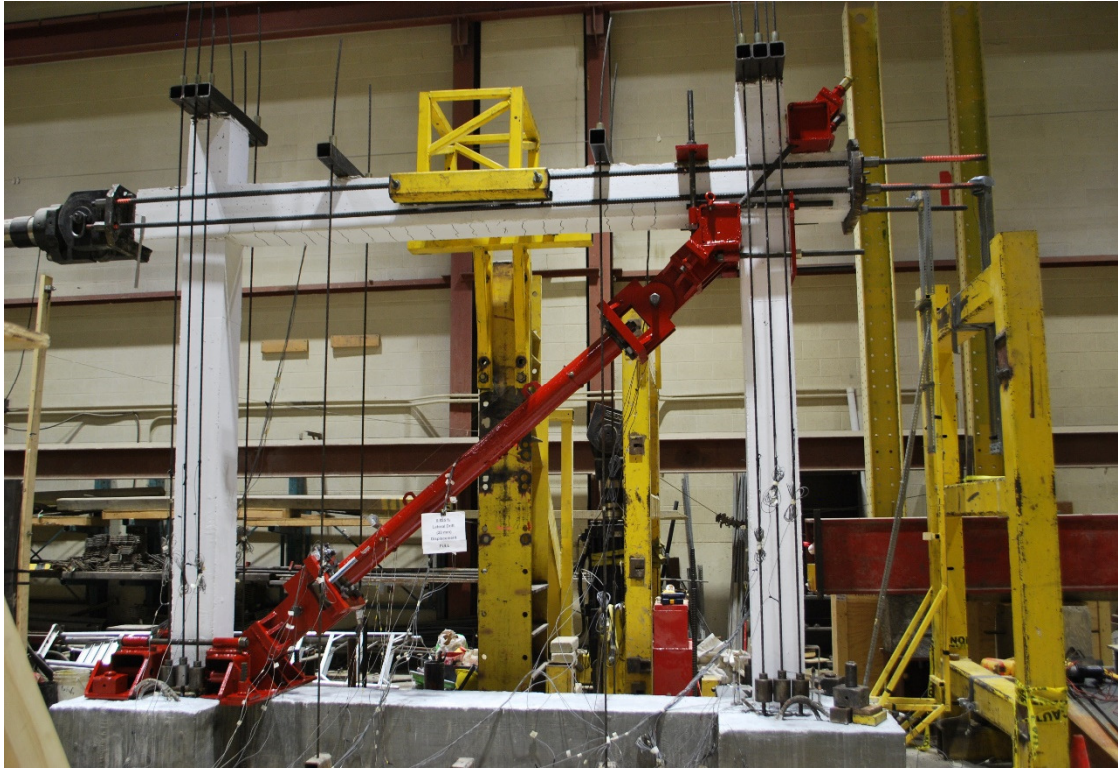
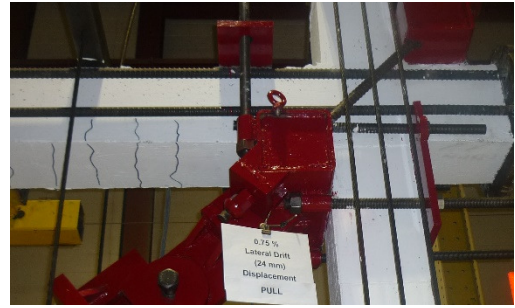


Figure 7.72: Cracking pattern at 0.625% lateral drift for Frame RF



(a)



(b)



(c)



(d)



(e)



(f)

Figure 7.73: Cracking pattern and BRB external gaps at 0.75% lateral drift of Frame RF:  
a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint

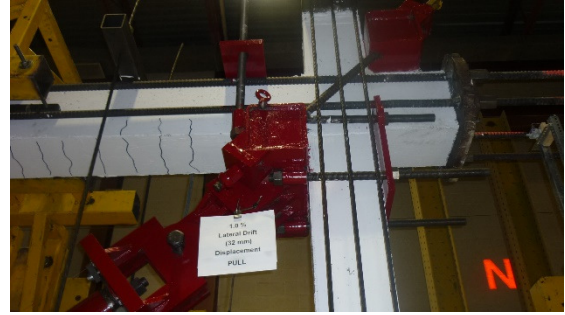


Figure 7.74: Cracking pattern at 0.75% lateral drift for Frame RF

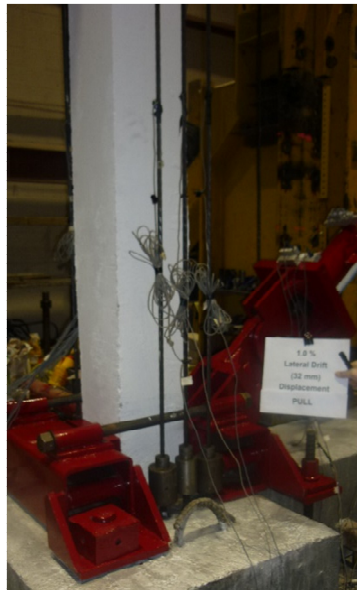




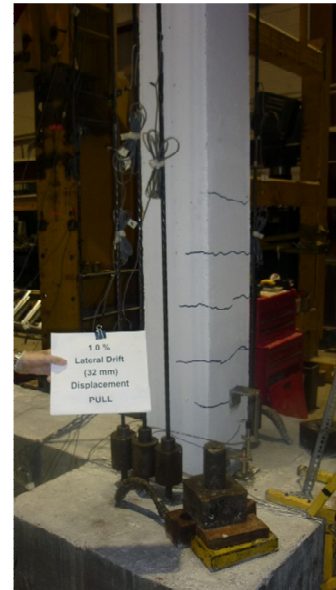
(a)



(b)



(c)



(d)



(e)



(f)

Figure 7.75: Initiation of flexural cracking at far column base and behaviour at 1.0% lateral drift for Frame RF: a) beam left side; b) beam right side; c) near column; d) far column; e) external gap at lower joint; and f) external gap at upper joint



Figure 7.76: Cracking pattern at 1.0% lateral drift for Frame RF



(e)



(f)



(g)



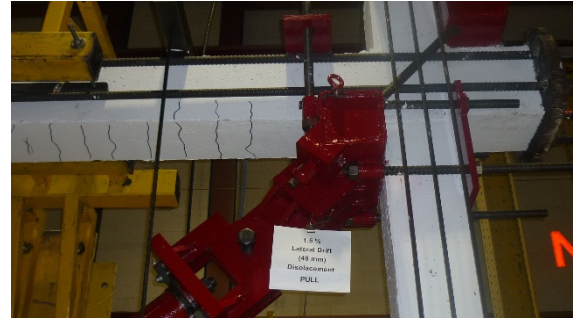
(h)

Figure 7.77: Propagation of flexural cracking at columns and frame behavior at 1.5% lateral drift for Frame RF: e) near column outer face; f) near column inner face; g) far column inner face; and h) far column outer face





(a)



(b)



(c)

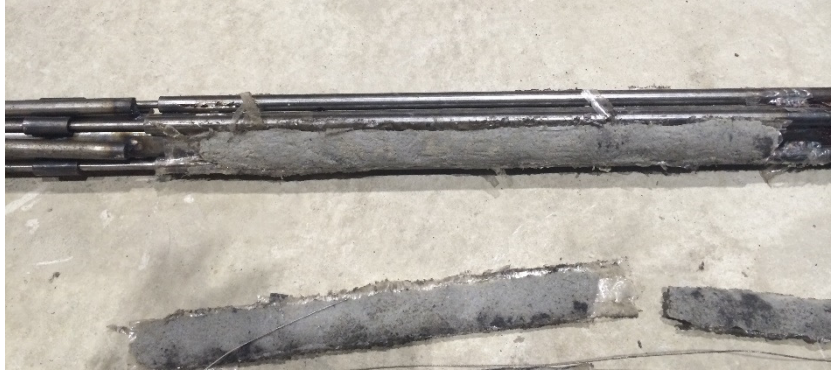


(d)

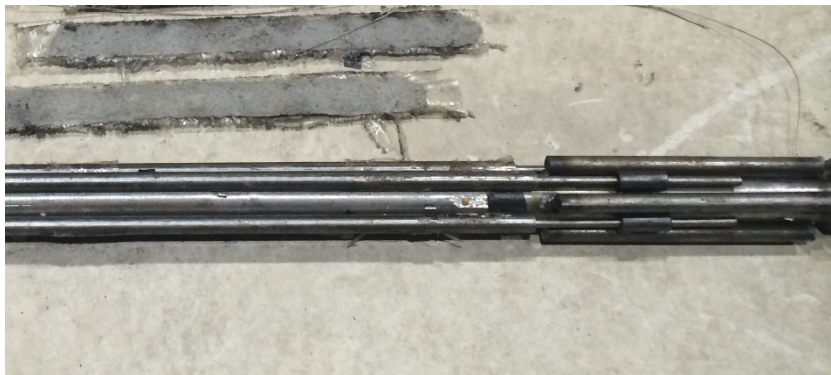
Figure 7.78: Propagation of flexural cracking at beam and behavior at 1.5% lateral drift for Frame RF: a) beam left side; b) beam right side; c) external gap at lower joint; and d) external gap at upper joint



Figure 7.79: Cracking pattern at 1.5% lateral drift for Frame RF



(a)



(b)



(c)

Figure 7.80: BRB core bar of Frame RF at failure: a) BRB core bar left side; b) BRB core bar right side; and c) rupture of BRB core bar





(d)



(e)

Figure 7.80 (Cont'd): BRB core bar of Frame RF at failure: d) BRB core bar with the unbonded Epoxy/sand filler; and e) complete BRB components after dismantling

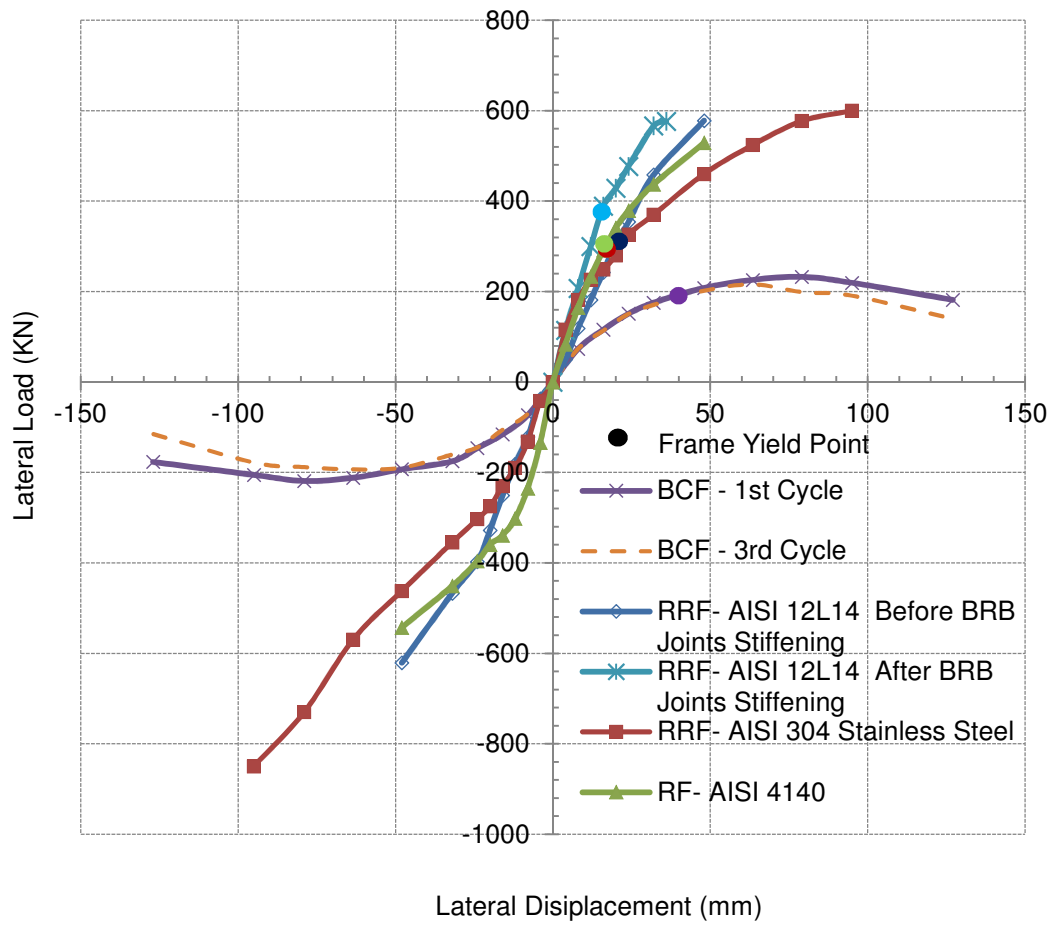
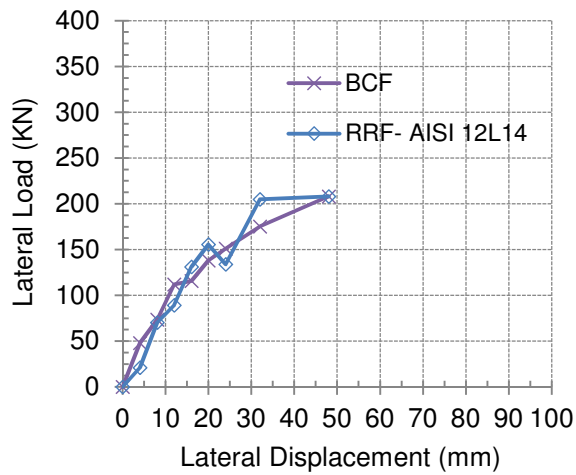
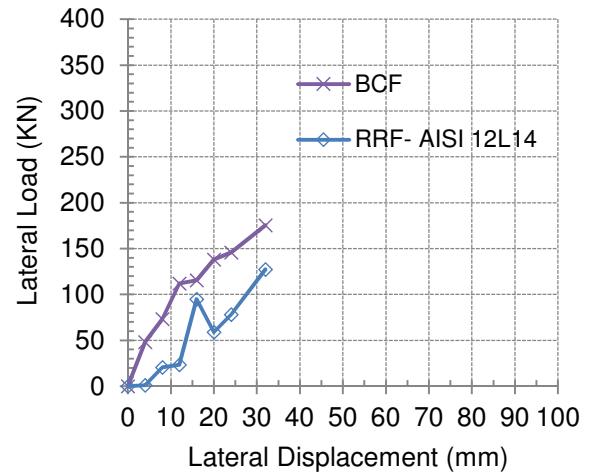


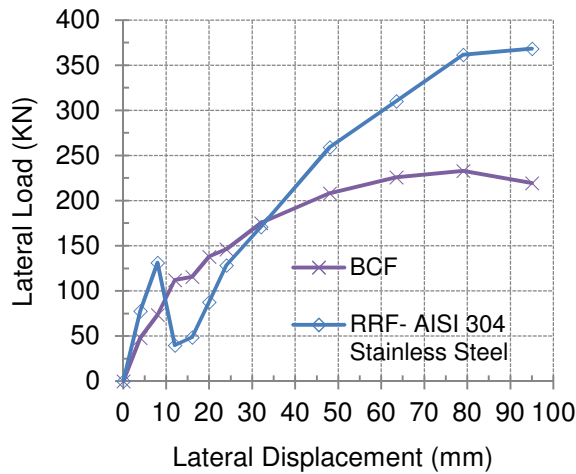
Figure 7.81: Lateral load- lateral displacement envelop responses for all frame tests



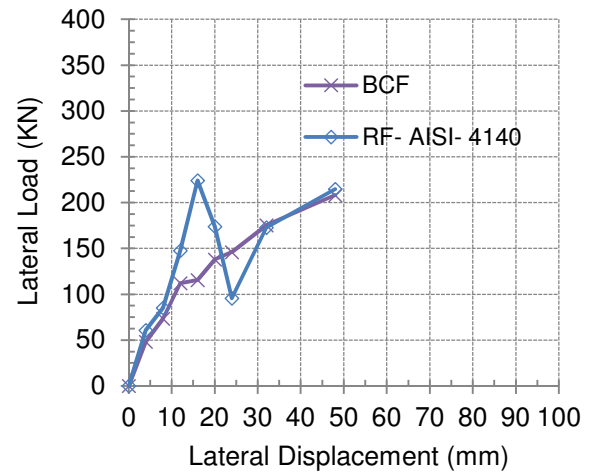
(a)



(b)

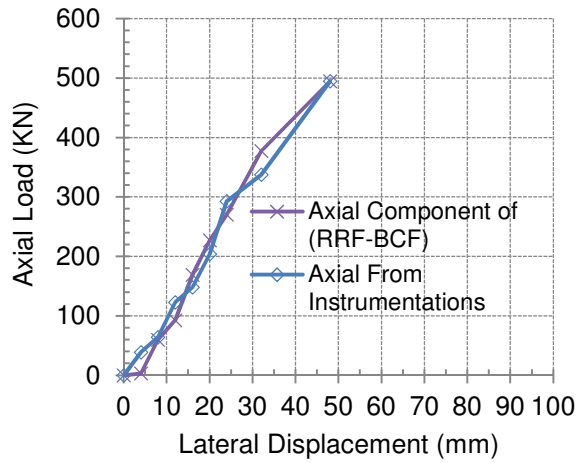


(c)

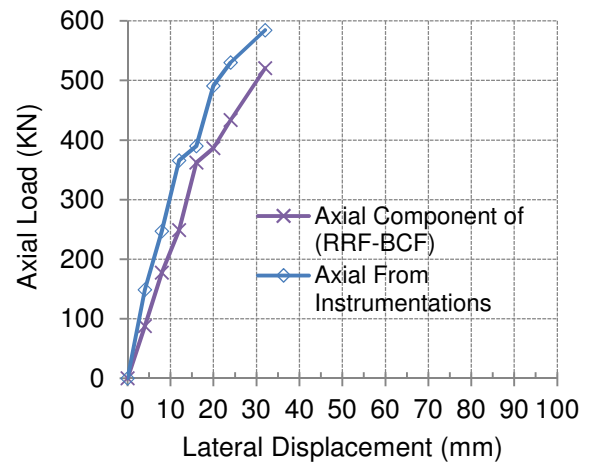


(d)

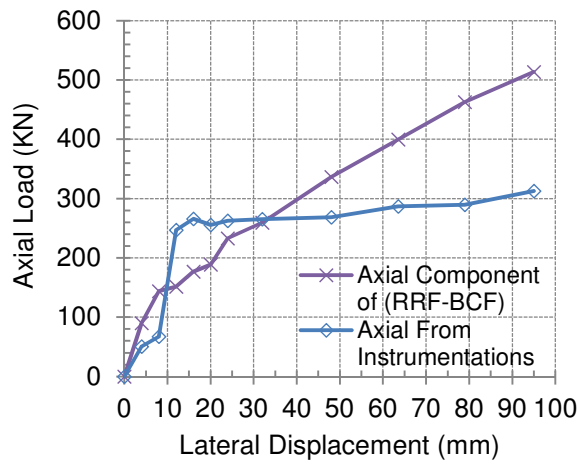
Figure 7.82: Comparison of lateral strength component provided by the concrete frames:  
a) Frame RRF (AISI 12L14) before BRB joint stiffening; b) Frame RRF (AISI 12L14) after BRB joint stiffening; c) Frame RRF (AISI 304 stainless steel); and d) Frame RF



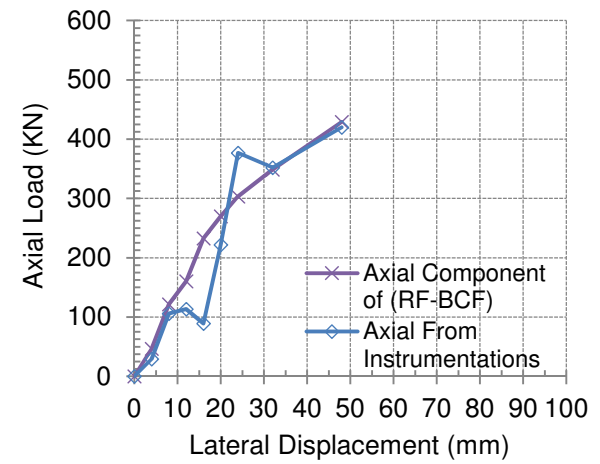
(a)



(b)



(c)



(d)

Figure 7.83: Comparison of axial strength of the BRB steel core bars used in the retrofitted frames: a) Frame RRF (AISI 12L14) before BRB joint stiffening; b) Frame RRF (AISI 12L14) after BRB joint stiffening c) Frame RRF (AISI 304 stainless steel); and d) Frame RF

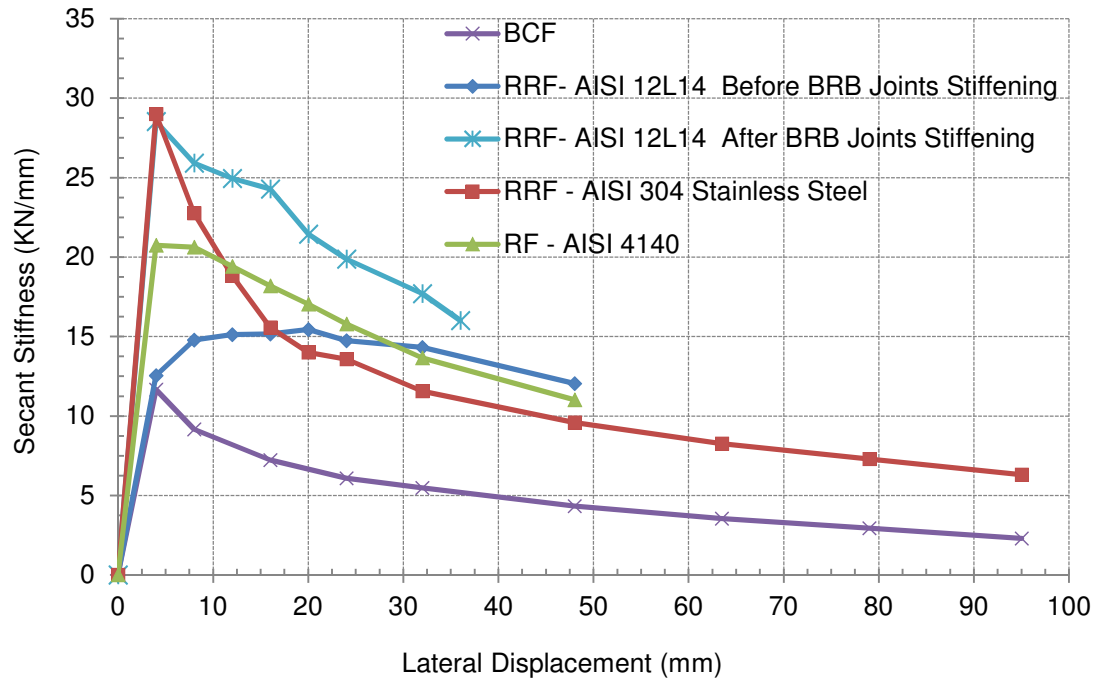


Figure 7.84: Secant stiffness-lateral displacement envelope responses for all tested frames

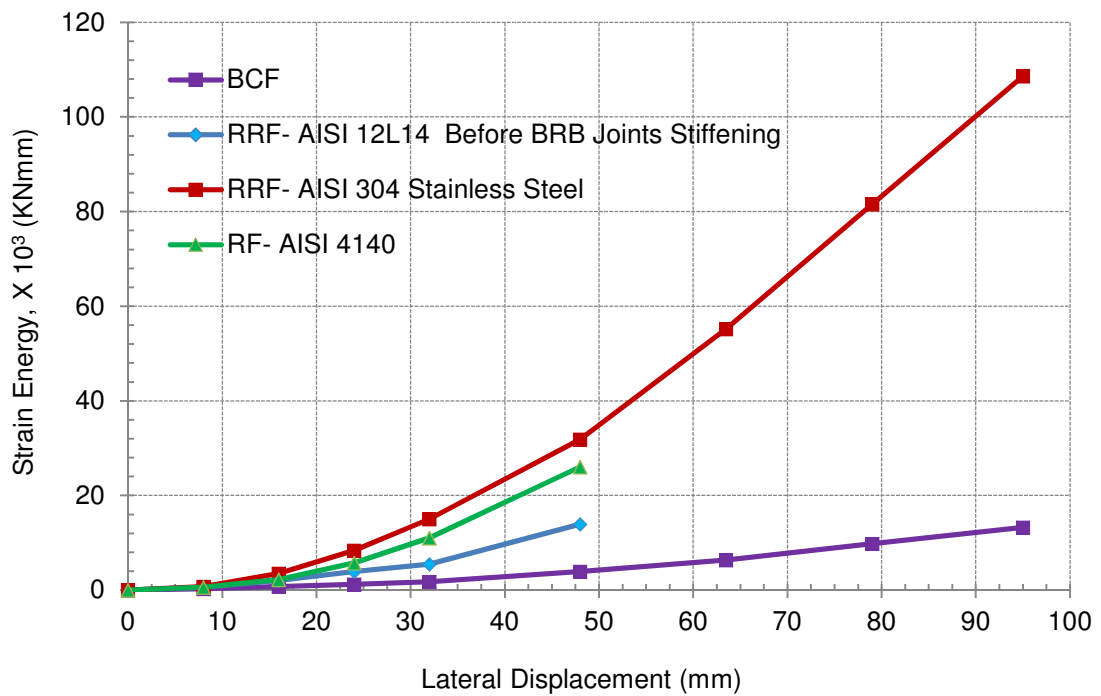


Figure 7.85: Energy dissipation-lateral displacement responses for all tested frames



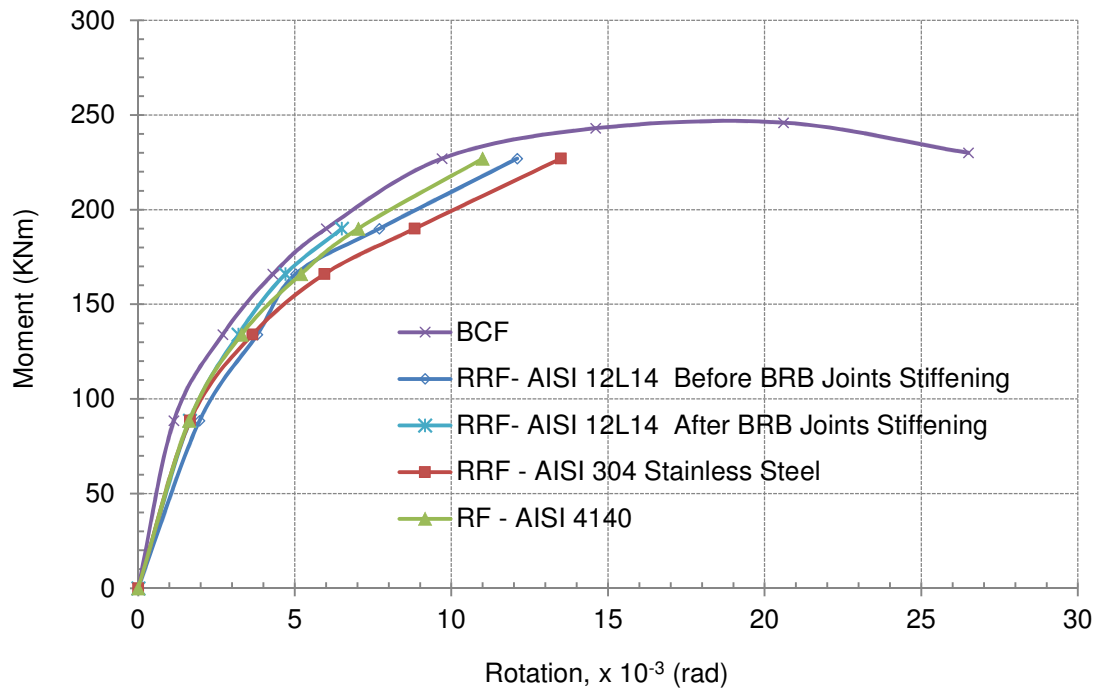


Figure 7.86: Moment-rotation response for all tested frames

## Chapter 8

# Nonlinear Static and Dynamic Analyses

### 8.1 Introduction

This chapter reports on analytical investigations. Its primary aim is to assess the effectiveness of using buckling restrained braces in seismic response of non-ductile reinforced concrete moment-resisting frames. The investigations were carried on the experimentally tested bare control and retrofitted frames, as well as on the prototype six-storey moment resisting frame building, details of which are discussed in Chapter 6. SAP2000 (CSI 2010) software was used to perform the analyses for both nonlinear static and nonlinear dynamic analyses. The static analysis was conducted under incrementally increasing lateral loads to verify the test results obtained in the experimental phase of research in the form of lateral force-lateral deformation relationships. Nonlinear dynamic response time history analyses were conducted using the prototype building to explore the effectiveness of using the BRB system developed in the current research project. The latter analyses were carried out using Uniform Hazard Spectra compatible earthquake records for the Cities of Ottawa and Vancouver in Canada under different intensity levels.

### 8.2 SAP2000 Software

SAP2000 is a general-purpose computer program that can be used for predicting response of structures to various types of static and dynamic loading. Developed by Computers and Structures Inc. (CSI 2010), SAP2000 is a powerful tool for use in current research, providing options to simulate nonlinear incremental static load analysis in displacement or force control modes, as well as nonlinear dynamic time history analyses under seismic excitations.

Nonlinearity in SAP2000 includes and not limited to the following; (i) initial structure condition, (ii) material nonlinearity, and (ii) geometric nonlinearity. The initial structure condition represents the state of the structure at the start of the subsequent analysis. In other words, whether the structure has previously been subjected to history of stresses and deformations or not, i.e. whether the analysis starts from unstressed state, or

continues from previous nonlinear loading with all previous stresses and deformations carried forward. The second type of nonlinearity, i.e., material nonlinearity, can be modeled by using lumped discrete elements called “links,” placed at critical regions. These links contain linear and nonlinear member properties, and are defined either with moment-rotation (for flexural members) or with force-displacement relationships (for axial members) for use in the current set of analyses. These links consist of either two joints or a single joint. The two-joint links can include finite lengths, representing linear and nonlinear properties of the link lengths; or can have approximately zero length between the two joints. In the latter case, the lumped linear and nonlinear properties of the links represent member lengths that are specified as infinitely rigid segments by the user. This latter approach was found to be more accurate when compared with experimental results. The single joint links are used when connecting the structure to the ground. The software also has the capability to model P-Delta effects representing geometric nonlinearities. This effect is significant especially when large stresses and deformations are exerted on the structure during nonlinear response.

### **8.3 Modelling of the Laboratory Frames for Nonlinear Static Analyses**

A numerical simulation study was performed to analyse the hysteretic response of the two large-scale reinforced concrete frames, representative of older buildings in Ottawa, tested under slowly applied lateral deformation reversals. These were, the bare control frame (BCF), and the repaired and retrofitted frame (RRF) in which AISI Type 304 stainless steel buckling restrained brace system (BRB) was used. The aim of this study was to have reliable analytical models that could reasonably represent the linear and nonlinear behaviour of the frame members as well as of the retrofitting elements for use in the subsequent phase of the analytical studies where full-size buildings are analyzed.

#### **8.3.1 Bare Control Frame Model**

A two-dimensional one bay one-storey analytical model was created by SAP2000 as shown in Figure 8.1. The model represents the bare control frame (BCF) geometry illustrated in Figures 6.1 through 6.3. The frame members were modeled with linear elastic elements while inelastic deformations were concentrated at plastic hinge regions at member ends. The columns were fixed at the base, and the beam-column joints were considered infinitely rigid. Characteristic strengths of concrete and steel were based on standard cylinder tests performed on the day of frame testing and standard steel tension

coupon tests, respectively (Table 6.1 and Figure 6.16 of Chapter 6). Factors of 0.4 and 0.68 were applied to the moment of inertia of the beams and columns, respectively, to simulate cracked concrete properties, as per CSA A23.3-04.

The frame model was subjected to gravity loads that were applied on the test frames, as illustrated in Figure 6.56. The gravity load on each column was 800 kN, and the two-point loads on the beam at 0.85 m distance away from beam-column joints, each was 62 kN. The seismic loading was simulated analytically through incrementally increasing in-plane displacement reversals. Three cycles of lateral displacement were applied at each deformation level following the loading protocol used during the BCF Frame test (Table 6.7). The nonlinear static analysis results obtained under lateral deformation reversals were superimposed on the results of the gravity load analysis. P-Delta effects were included to account for geometric nonlinearities. The following modelling features had to be implemented in the analysis as precisely as possible for improved accuracy of results:

- i. Sectional moment-curvature and member moment-rotation or force-deformation relationships.
- ii. Lengths of plastic hinge regions at member ends.
- iii. Locations of link hinges with zero link length within each plastic hinge region at member ends,
- iv. A hysteretic model that could trace the load reversal paths within each hysteresis loop.

Sectional capacities of frame members were determined using computer software RcSection 1.3 (Pikaso Software Inc. 1997). Beam capacities were calculated at column faces for negative and positive bending under load reversals for input into the hysteretic model. Column capacities were computed to be uniform and symmetric along the column height, assuming sufficient splice length at the column base.

Plastic hinge lengths for frame members,  $l_p$ , were calculated based on the suggestion of Paulay and Priestley (1992) as indicated in the following equation:

$$[8.1] \quad l_p = 0.08L + 0.022d_b f_y$$

Where  $L$  is the member length between the critical section and the point of contra-flexure (shear span),  $d_b$  is the diameter of the longitudinal reinforcement in m,  $f_y$  is the yield strength of reinforcement in MPa. The point of contra-flexure was assumed to occur at mid-length of clear span. The plastic hinges were defined at both ends of linear elastic frame elements, as shown in Figure 8.1. Plastic hinge properties attained from the moment-curvature analyses of frame members are summarized in Table 8.1.

Links, consisting of two joints connected by a near-zero length, were used to lump the elastic and plastic deformations within the plastic hinge region of each member. Therefore, the segment corresponding to the hinging region was made infinitely rigid for flexure with links positioned in the middle of each plastic hinge length, as illustrated in Figure 8.1. This approach is believed to be more realistic than placing the links at locations of highest flexural demand. i.e. at the column face in the case of beam links; and at the base and the beam soffit level in the case of column links. Placing the links in the middle of plastic hinge lengths were also used by other researchers (Kaushik et. al. 2009 and Deka et. al. 2014).

Dowell et al. (1998) introduced a new hysteresis model called “Pivot Hysteresis Model” which is similar to the one introduced by Takeda et.al (1970) but has additional parameters that control the degrading stiffness of hysteresis loops. This model accurately captures the nonlinear behaviour of reinforced concrete members and is useful for modeling asymmetric sections and/or sections with different amounts of reinforcement in top and bottom layers (Dowell et al.,1998). The model requires input values for three pivot parameters that can be calibrated using the hysteretic relationships that represent the behaviour of actual members in the structure being analysed. This information in the current study was obtained from the hysteretic relationships generated in the laboratory. The parameters of the model include  $\alpha$ ,  $\beta$ , and  $\eta$ . They are illustrated in Figure 8.2 with further details provided in Figure 8.3. Parameters  $\alpha_1$  and  $\alpha_2$  represent the points towards which the unloading positive and negative forces are oriented up to the zero force level, respectively While  $\beta_1$  and  $\beta_2$  represent the points towards which the reverse loadings from zero are oriented towards the positive and negative forces, respectively. The  $\eta$  parameter represents the degradation amount of the elastic slopes in the post-elastic region. After calibration with the experimental hysteretic response behaviour of the Frame BCF, these analytical parameters were integrated in the



SAP2000 link models as  $\alpha_1 = 1$ ,  $\alpha_2 = 1$ ,  $\beta_1 = 0.25$ ,  $\beta_2 = 0.18$ , and  $\eta = 9$ .

### 8.3.2 Repaired Retrofitted Frame Model

The bare control frame was tested, then repaired, and retrofitted using the newly developed buckling restrained brace (BRB). Therefore, the analytical model of the bare control frame with reduced initial rigidities and an additional element for the BRB should represent the analytical model of the retrofitted repaired frame.

The BRB configuration used in the test frame is illustrated in Figure 6.51 of Chapter 6. Its analytical model is shown in Figure 8.4 with geometric and mechanical properties incorporated. The brace consists of a restrained brace element (steel core bar), connected to the test frame through hinged end connections as illustrated in Figures 6.37 and 6.48. These hinge connections were modeled as frame elements with high flexural rigidities. They were assigned moment releases at their connections to the brace element. The restrained brace element, specifically the steel core bar illustrated in Figure 6.35 (b), was modeled by three link elements, each with two joints, connected in series to represent the reduced and un-reduced sections of the steel core bar. These link joints were locked against five degrees of freedom, leaving the two translational and three rotational components. The uniaxial directional components were assigned nonlinear force-deformation properties of the steel bar. The hysteretic behaviour suggested by Wen (1979), as illustrated in Figure 8.5, expressing axial forces and deformations relative to the yield deformation, was adopted for the links properties. This was done in SAP2000 by specifying four parameters; (i) stiffness during loading and unloading, (b) yield strength, (c) post-yield stiffness ratio, and (d) yielding exponent, which controls the sharpness of the yielding curve, and has a range of values between 1.0 for a very curved response, and infinity for a perfectly elastic-plastic behaviour. The results of tension coupon tests were used to construct the nonlinear axial force-axial deformation relationship of the steel core bar. This was done by using the coupon area and the extensometer length used to measure the deformations. The graphical 0.2% offset method and the equivalent elasto-plastic relationship were used to capture the Wen model parameters as shown in Figure 8.6. The tensile behaviour of the coupon was then replicated for compression to have a full cyclic hysteresis curve as illustrated in Figure 8.7.

The following equations were used to find the BRB link element nonlinear properties:

$$[8.2] \quad \Delta L = \Delta_c (L/L_c)$$

$$[8.3] \quad P = P_c (A/A_c)$$

$$[8.4] \quad K = (P_c / \Delta_c) [(A/A_c)/(L/L_c)] = K_c [(A/A_c)/(L/L_c)]$$

Where  $\Delta L$ ,  $L$ ,  $P$ ,  $A$ , and  $K$  are the axial deformation, original length, axial load, cross-sectional area, and stiffness of the BRB steel core bar, respectively, whereas  $\Delta_c$ ,  $L_c$ ,  $P_c$ ,  $A_c$ ,  $K_c$  are the axial deformation, extensometer length, axial load, area, and stiffness of the bar, respectively. Table 8.2 summarizes the Wen plasticity properties used for the BRB steel core link element.

The loading protocol used was similar to that used for Frame BCF adapted to the loading protocol used for Frame RRF during testing (Table 6.8).

#### **8.4 Analytical Results of Laboratory Frames under Reverse Cyclic Loading**

The validity of the analysis procedure and the hysteretic models was assessed by comparing the results of static inelastic analysis under reversed cyclic loading with experimental results obtained by testing the bare control frame (BCF) and the repaired and retrofitted frame (RRF). These comparisons are crucial before conducting dynamic inelastic response history analysis of prototype structures. Geometric nonlinearity was introduced through P-Delta effect. Two aspects of observed behavior were neglected in the analytical model; i) the variation of axial load during response to lateral forces and ii) any potential bar slip within the lap splice region, as well as any anchorage slip that may take place within the adjoining member. The impact of axial loads is to change flexural resistances, as well as stiffnesses. As the column axial loads vary due to the lateral sway of the frame, sectional moment capacities also vary with changing axial loads. Figure 8.8 (a) illustrates the axial force-moment interaction diagram for a column.

The percentage of axial load variation due to the lateral loads on BCF and RRF Frame members are shown in Figure V.3 (b), and V.5/V.6 (b) of Appendix V, respectively. It is evident that the axial load has an effect on stiffness and strength. Column moment capacities used in the analytical model were calculated based on a constant axial compressive force equal to 25% of the maximum concentric axial capacity ( $P_o$ ), as

presented in Appendix III. Hence, any increase in axial load would increase the moment capacity up to the balanced section at around 40% of  $P_o$  as depicted in Figure 8.8 (a). The column axial load variation during testing was monitored by strain gauges placed on prestressing cables used to apply the gravity loads. The variation was found to be  $\pm 8.3\%$  at 2.5% drift and  $\pm 12.2\%$  at 3% drift in the near-end and far-end columns relative to the loading end. Also, with increased lateral deformations, the columns develop extra moments due to the P-Delta effect, further increasing the significance of axial force variation on response.

The analytical model also neglects the potential effects of reinforcement bond-slip within inadequate splice regions, as well as anchorage slip that may take place in adjoining members. Furthermore, inelastic shear deformations are neglected and not modeled in the analysis. These aspects of analytical modelling may result in slight discrepancies between the analytical and experimental results.

#### **8.4.1 Bare Control Frame Model**

Figure 8.9 illustrates the hysteretic lateral load-lateral displacement relationship for the bare control frame (3 cycles at each drift level). The frame experienced symmetrical elastic response until the onset of yielding (Table 8.1). The first yield was initiated at a drift of 0.8% (25 mm) near the column base, corresponding to a lateral load of 145 kN. A second yield point was observed at a drift of 1% (34 mm) when the columns experienced yielding at beam-column joints, corresponding to a lateral load of 165 kN. The hysteretic behaviour showed a gradual increase of lateral resistance at each drift level with the post-yield slope of the hysteretic response (stiffness) developing 4% and 5% of initial stiffness in columns and beams, respectively. The frame reached its maximum lateral resistance of 185 kN at 2.7% drift (86 mm). As the drift increased to 4% (127 mm), the lateral resistance of Frame BCF decayed to 168 kN.

The analytical and experimental hysteretic lateral load-lateral displacement relationships for Frame BCF are compared in Figure 8.10. The comparison reveals very good agreement. The frame lateral load resistance indicates slightly higher values in the experimental relationship. This is especially true during the push cycles, which are denoted on the upper right quadrant of the relationship. The push cycles experienced new levels of deformation for the first time with increased strains in steel, as opposed to

the cycles in the opposite direction which experienced the same level of deformations with steel that had been stressed to higher levels in the opposite direction. This results in the baushinger effect and the associated permanent softening of steel in the opposite direction with slightly reduced strength. Similarly slight differences occur in damaged concrete between the initial damage imposed in one direction and the following damage in the opposite direction. These may result in a slight increase in initial force resistance in the direction where the first damage is imposed, which is not accounted for in the analytical model, in addition to the unavoidable experimental imperfections that may have contributed to the observed slight asymmetry. Furthermore, the beam is subjected to small axial loads during the test, equal to the shear force in the column opposite to the loading side. This axial load, whose effect is not included in the analytical model, may have increased the beam flexural resistance, which in turn may have contributed to the overall increase in frame resistance relative to the analytically computed resistance.

Frame BCF moment-rotation responses are illustrated in Figures 8.11, 8.12, and 8.13 for column rotations at the base and at beam-column joint; and for beam-end rotations, respectively. The column response at the base is symmetrical in the push and pull directions. Yielding in the positive rotation direction (clockwise moment) was initiated at 32 mm (1% Drift). The yielding initiated in the negative rotation direction (counter-clockwise moment) at 24 mm (0.75% Drift). The column exceeded its capacity of 185 KN·m at 63.5 mm (2.0% drift) at rotations of 0.0143 rad and 0.0036 rad in the positive and negative rotations, respectively. The experimental behaviour also indicated yielding of reinforcing starter bars at the base of both columns at these drifts as recorded by strain gauges S.G. #1, #6, #11, and #12 (shown in Figure 6.9 of Chapter 6). At 2% drift, horizontal flexural cracks became wider at the base of the column.

At beam-column joints, different levels of bending moments are transferred from the beam ends to the columns, as beam positive and negative moment capacities are different. This behaviour is clearly observed in the moment-rotation response of the column at the beam-column joint, and of the beam ends as shown in Figure 8.12, and Figure 8.13, respectively. The column response indicated yield initiation near the beam at a lateral displacement of 32 mm (1% drift) and exceeded the ultimate capacity of 185 KN·m at a lateral displacement of 63.5 mm (2.0% drift), corresponding to a rotation of 0.0153 rad in the positive rotation direction. In the negative rotation direction, the column

behaved elastically and developed a maximum moment demand of 56 KN·m at a lateral displacement of 24 mm (0.75% drift), corresponding to a rotation of 0.00124 rad. Experimentally, strain gauges located on the column bars near the beam-column joint did not all function. Strain gauges S.G. #21, #24 located at the middle longitudinal bar near the beam-column joint, locations of which are shown in Figure 6.9, indicated yield initiation at higher drifts of 2.5% and 3.0%, respectively. On the other hand, the moment-rotation response of the beam showed yield initiation at a lateral displacement of 24 mm (0.75% drift) and at 95 mm (3% drift) under positive and negative rotations, respectively. The beam developed its ultimate capacity of 81 KN·m in the positive rotation direction at a lateral displacement of 127 mm (4% drift) corresponding to a rotation of 0.051 rad, while the maximum moment of 192 KN·m was attained in the negative rotation direction at the same drift level, corresponding to a rotation of 0.019 rad. The strain gauges located on the bottom beam bars near the far-end beam-column joint (away from the actuator) indicated onset of yielding at 0.75% drift (S.G. #25, and #26). The gauges located on the same bars but at the near-end beam-column joint (near the actuator) indicated yielding at a drift of 1.25% (S.G. #30) and a drift of 1.5% (S.G. #29). The small discrepancy observed in yield drifts between the analysis and the test can be attributed to the difference between localized bar yielding as indicated by strain gauges and the yielding of the entire section observed in the test. At 4% drift, the bottom longitudinal bars in the beam experienced buckling, which was followed by a drop in load resistance to more than 20% of the peak capacity. The latter behaviour is in good agreement with the analytical model at ultimate.

#### **8.4.2 Repaired Retrofitted Frame Model**

The analytical lateral load-lateral displacement hysteretic relationship of the repaired retrofitted frame RRF is illustrated in Figure 8.14. As in the case of the previous frame analyzed, the hysteretic relationship is based on three cycles at each of the incrementally increasing drift level. The frame, including the BRB, remained elastic in both loading directions until the onset of yielding at a lateral displacement of 10 mm (0.32% drift), corresponding to a lateral load resistance of 300 kN. The overall system yielding was controlled by the yielding of the BRB steel core bar at the aforementioned lateral displacement exceeding the bar axial force yield capacity of 354 kN (Table 8.2). The system further softened at a lateral displacement of 24 mm (0.75% drift) corresponding to lateral load capacities of 400 kN and 450 kN in the push and pull

directions, respectively. The second softening of the system was due to the yield initiation of the reinforced concrete frame at critical beam sections. In the post yield region, the change of slope of the hysteretic response, indicating overall stiffness, is significant and is characterized by a linear ascending slope up to the maximum ultimate lateral load capacities in both directions. The retrofitted frame system attained its maximum lateral force resistance of 410 kN at a lateral displacement of 48 mm (1.5% drift) in the push direction. Subsequently, the lateral force resistance degraded to 298 kN at a lateral displacement of 95 mm (3% drift). In contrast, the hysteretic response of the system in the pull direction showed a continuous increase of lateral load resistance up to a lateral displacement of 95 mm (3% drift). At this drift level, the lateral resistance of Frame RRF was 744 kN. It is evident that the nonlinear properties of the BRB steel core bar, as specified in Table 8.2, including the post-yield stiffness ratio, had a significant effect on the overall lateral load capacity of the frame.

The analytical lateral force-lateral displacement hysteretic relationship of Frame RRF is compared with the experimental relationship in Figure 8.15. The comparison reveals good agreement in the elastic range of deformations with approximately 10% higher initial stiffness obtained analytically. This may be attributed to the initially cracked rigidity of the repaired test frame. In the post-elastic region, the lateral resistance recorded experimentally show continuous increase with increasing inelastic deformations, unlike the analytically computed force resistances. This is especially prevalent in the push direction. The difference in experimental and analytical resistances is primarily attributed to the effect of variable axial load during response, which becomes especially more pronounced in the retrofitted frame. The frame columns are subjected to variable axial loads as a result of lateral sway, as also discussed previously for Frame BCF. In the retrofitted frame (RRF), significant axial forces are imposed on the far-end column due to the lateral brace (BRB). During the push cycles, the far end column is subjected to an additional axial compression, which is equal to the vertical component of the diagonal force in the brace. The analysis of frames, presented in Appendix V, shows the variation in axial forces in the frame elements. The axial compression on the far-end column increases column capacity, and hence the lateral resistance of the frame. Furthermore, the beam also develops axial compression during the push cycles as it is compressed between the applied horizontal load and the horizontal component of the diagonal brace force. Figure V.5 (b) of Appendix V indicates that the beam takes 96% of the applied



lateral load in the push direction. The axial compressive force on the beam increases beam bending capacity, as illustrated in the beam interaction diagrams shown in Figure 8.8 (b) and (c). When the experimental lateral force resistance reached 600 kN in the push mode, the beam developed an axial compression force of 576 kN. This axial load resulted in an increase in the beam negative bending capacity to a value of 160 kN·m. This moment resistance corresponds to a factor of 2.0 increase relative to the moment capacity in pure bending. The axial force resulted in an increase in the beam positive bending capacity as well, to 248 kN·m. This increase corresponds to an increase factor of 1.26 relative to pure bending. These increased bending capacities were not simulated in the analytical model thus resulting in frame response with reduced stiffness and strength when compared with the experimental frame response. It is also clear from the comparison shown in Figure 8.15 that the frame experienced higher lateral load capacity on the pull side, when the BRB developed compression forces, than on the push side when the BRB was in tension. This is true for both experimental and analytical responses and is in agreement with the experimental findings of other researchers (Uang and Nakashima 2004 and Tremblay et al. 2006). These researchers reported higher brace capacity in compression than in tension. This increase was attributed to the frictional forces that develop between the steel core and the surrounding restraining element, and the strain hardening of steel core. Another factor that is believed to contribute to the reduction in the brace capacity when in tension is the reduced cross-sectional area of steel compared to the increased cross-sectional area in compression due to the Poisson's effect, result in reduced ultimate capacity when the frame is in the push mode.

The P-Delta effect was accounted for both experimentally and analytically and played a dominant role in reducing the frame lateral force capacity during push. The columns resist moments that consist of two components; those caused by lateral loads and the remaining caused by the P-Delta effect. When the column starts yielding, its capacity cannot increase significantly and the resistance remains more or less constant. However, when inelastic displacements increase, the P-Delta component of applied moment increases as the component due to lateral forces decreases. This implies that the lateral force resistance starts decreasing as the frame displacements increase. During the push cycles, the far-end column moment capacity increases due to the accompanying axial compression, but the moment component associated with lateral

forces decreases. These two effects show opposite trends when the hysteretic relationship is shown as a function of lateral force rather than applied moment (which includes the P-Delta moment). In contrast, when the frame is in the pull mode, the same column is subjected to increasing tensile forces, as shown in Figure V.6 (b) of Appendix V, offsetting gravity loads, causing reduction in the initially applied compressive loads due to gravity. Therefore, in the pull mode, the effect of P-Delta is not dominant.

Computed moment-rotation relationships of frame members for Frame RRF are illustrated in Figures 8.16 through 8.18. During testing, the column response at the base indicated yield initiation at lateral displacements of 32 mm (1.0% Drift) and 48 mm (1.5% Drift) in positive rotation and negative rotation directions, respectively. The column exceeded its computed ultimate capacity at lateral displacements of 79 mm (2.5% drift) and 63.5 mm (2.0% drift) in positive and negative directions, respectively. The analytical results are in good agreement with the experimental strain gauge readings as recorded by S.G.# 16, indicating yielding of the starter bars of the near-end column at a lateral displacement of 75 mm (2.3% drift). At higher drifts of 2.5% and 3%, the columns suffered significant concrete spalling at the base, and developed propagation of flexural cracks. The moment-rotation response of the column at the beam-column joint, depicted in Figure 8.16, indicates yield initiation at a lateral displacement of 48 mm (1.5% drift). The sectional ultimate moment capacity was exceeded at a lateral displacement of 79 mm (2.5% drift) in the positive rotation direction. The column behaved elastically in the negative rotation direction and developed a maximum moment demand of 56 KN·m at a lateral displacement of 24 mm (0.75% drift), corresponding to a rotation of 0.00124 rad. The moment-rotation response of the beam, shown in Figure 8.18, indicates yield initiation at lateral displacements of 24 mm (0.75% drift) and 79 mm (2.5% drift) in positive and negative rotations, respectively. The beam did not exceed its ultimate capacity in both directions. Moment demands of 79 KN·m and 188 KN·m were recorded both at a lateral displacement of 94 mm (3% drift) corresponding to rotations of 0.0356 rad and 0.0084 rad for positive and negative rotations, respectively. Experimentally, strain gauges situated on the steel reinforcement of the columns near the beam-column joints, as well as on the beam, were defective, and no comparison could be made regarding the strains on the reinforcement at these locations. Nevertheless, at a drift range of 2% to 3%, shear cracks and flexural cracks widened and increased extensively along the beam and at beam ends near the beam-column joints. The same were also

noticed regarding flexural cracking at columns near the beam-column joints.

The axial deformation response of the BRB steel core bar at reduced and un-reduced sections are illustrated in Figures 8.19 and 8.20, respectively. The reduced section deformation response indicate bar yielding at an axial deformation of 10 mm (13.3 mm lateral deformation), which corresponds to 0.43% drift as listed in Table 6.6. This is an excellent agreement with the experimental BRB steel core axial deformation response that indicated yielding at a lateral displacement at 12 mm (0.375% drift) during the push mode. However, the experimental axial yield force of 247 kN in the BRB steel core, which was determined by correlating strain gauge readings located on the core with the stress-strain response attained from coupon testing, was lower than the yield load of 354 kN assigned to the analytical model, as indicated in Table 8.2. With an assigned post-yield ascending stiffness ratio of 0.2%, the core bar continued sustaining increased axial tension and compression loads, and attained a maximum axial load of 369 kN at an axial displacement of 70 mm (corresponding to 95 mm lateral deformation and 3% drift as listed in Table 6.6). Experimentally at 3% drift, the BRB steel core fractured at the middle section of the reduced area during the second push. The average axial tension and compression forces sustained by the BRB steel core during testing were 313 kN and 310 kN, respectively. The analytical value of 369 kN was obtained at the same drift level, which is 18% higher than the measured values. It is believed that the strain gauge readings reflect localized stress conditions on the bar rather than capturing the entire member performance. The analytical model for the BRB steel core was based on coupon tests. Improved analytical models could be employed in the analysis if test results for full-size brace elements were available. The un-reduced sections of BRB steel core bar demonstrated elastic response during the analysis because of the larger cross-sectional area assigned to these segments. This is illustrated in Figure 8.20.

### **8.5 Design of the Prototype Building for Nonlinear Seismic Analysis**

The six-storey moment-resisting frame prototype building, described in Section 6.2 of Chapter 6 and shown in Figure III.1 of Appendix III, representing a typical seismically deficient older medium-rise building, was considered for nonlinear response history analyses. Such buildings were constructed during the 1960's and early 1970's. The building consists of two exterior and six interior frames of three bays in the short direction. Ottawa and Vancouver were selected as major centers representing the

seismicity of eastern and western Canada for the design of the building. However, the provisions of NBCC 1965, based on which the building was designed, had the same seismic zoning factor for both cities. Therefore, the design base shear for the building was almost identical when designed for these two cities (see calculations in Appendix I). SAP2000 software was used to conduct linear elastic static analyses for design purposes. The building model consisted of individual two-dimensional frames as external and internal plane frames (detailed calculations are presented in Appendix III). Loads were applied in the short plan dimension of the building. The analyses were conducted based on preliminary member dimensions and the load combinations of NBCC 1965. Cracked section properties of frame members were used for the columns and beams, as per the calculations presented in Section V.1 of Appendix V. Design moments, axial forces and shear forces were computed for the beams and columns, as summarized in Tables 8.3 and 8.4, respectively. The summary of sectional sizes and reinforcement details are shown in Tables 8.5 and 8.6, respectively.

### **8.6 Structural Modeling for Nonlinear Analyses**

A two-dimensional lumped frame model, shown in Figure 8.21, was used for seismic analysis of the building with Ottawa and Vancouver as site locations. The 2-D analysis was considered sufficiently accurate since the buildings considered were regular buildings without torsional sensitivity. The modeling of the building in SAP2000 consists of two stages; the first stage involves modeling the entire frame with linear elastic elements, while the second stage involves modeling nonlinearities within each frame element. The first stage was achieved through lumping the properties of identical frames in one frame. In other words, through strength and stiffness modifiers, the two exterior frames were lumped into one frame and the six interior frames were lumped into another frame (Figure 8.21). These two frames were then connected through rigid beam elements having infinite axial rigidity and very small flexural rigidity. These rigid beam elements were employed to ensure equal lateral displacements without transferring moments between the two lumped frames. Rigid diaphragms were assigned to all building floors. Static gravity loads, as calculated in Appendix III, were applied as distributed loads on floor beams. The load factors and load combinations specified in NBCC 2010 were used for each floor, which were then converted into masses, and assigned to each node. Effects of potential cracking of concrete were taken into account through the reduced flexural rigidities as defined in CSA A23.4 (2004) and as per the

calculations in Section V.1 of Appendix V. Accordingly,  $0.4 E I_g$  and  $0.67 E I_g$  were used for beams and columns. The span lengths and storey heights were specified based on centre-to-centre (joint-to-joint) dimensions. Automatic rigid offset option of the software was assigned at the end of each element to account for finite widths of elements. The columns were fully fixed at the base.

The second stage of member modeling for inelastic deformations is discussed in the following sections.

### **8.6.1 Nonlinear Properties of Un-retrofitted Buildings**

The nominal axial, bending, and shear capacities of the prototype building were determined by performing sectional analyses using RcSection software, version 1.3, developed by Pikaso Software Inc. (1997). Tables 8.7 and 8.8 illustrate nominal capacities of beams and columns, respectively. The sectional moment-curvature relationship for each frame member was established under unfactored axial loads using the Software. The analyses indicated well-defined yield curvatures for beam sections under zero axial loads. The yield curvatures for columns were determined graphically with the equivalent elasto-plastic energy principle suggested by Park (1988), as the yield region was curved. The curved behaviour is attributed to the gradual yielding of column bars, which were well distributed around the section perimeter. The ultimate curvature is defined as the section capacity when extreme fiber compression strain reaches a value of 0.6%. This value was decided based on analytical expressions of strains corresponding to 85% strength level beyond the peak ( $\epsilon_{85}$ ) suggested by Sheikh and Uzumeri (1980). At the same strain level,  $\epsilon_{85}$ , Saatcioglu and Razvi (1992) proposed a confined analytical model that resulted in a strain value of 0.74%. The lesser value between the two models, i.e., 0.6%, was used in this study. It is suspected that at this strain level the unconfined concrete crushes and the longitudinal bars become susceptible to buckling.

The equivalent lengths of plastic hinges of frame members ( $l_p$ ) were calculated based on the suggestion of Paulay and Priestley (1992) as expressed in Equation 8.1. Accordingly, moment-rotation ( $M-\theta$ ) relationships were calculated for these plastic hinge lengths based on the linear idealization suggested by Park and Paulay (1975). The following expressions were used to compute the moment-rotation relationships, which

were then assigned to each link.

$$[8.5] \quad \theta_e = \phi_y I_p$$

$$[8.6] \quad \phi_p = (\phi_u - \phi_y) I_p$$

$$[8.7] \quad \theta_u = \theta_e + \theta_p$$

Where  $\phi_y$  and  $\phi_u$  are rotations of the members at yield and at ultimate, respectively;  $\theta_e$  is the elastic rotation,  $\theta_p$  is the plastic rotation, and  $\theta_u$  is the ultimate rotation. The ratio of ultimate moment ( $M_u$ ) to yield moment ( $M_y$ ) was 1.05 for all frame members while the ratio of the post-yield stiffness ( $K_p$ ) to the elastic stiffness ( $K_e$ ) were ranging between 0.73% and 1.88% for columns, and between 0.5% and 1.0% for beams. Table 8.9 summarizes nominal strengths and plastic hinge properties for the columns, while Table 8.10 summarizes the same for the beams of the prototype building.

The un-retrofitted building model, illustrated in Figure 8.21, was modeled in the same manner as the idealized bare control test frame model presented earlier (Figure 8.1). Each plastic hinge region was modelled by two equal length rigid elastic beam elements with an inelastic link placed in between these rigid beam elements. The two rigid beam elements and the in-between link together represent the overall nonlinear response of the hinging region. The rigid beam elements would not deform, but the link between the two with assigned inelastic properties deforms and simulates both the elastic and inelastic deformations that take place within the entire hinging region. This is illustrated in Figure 8.22. SAP2000 calculates elastic rotations of the frame members between the plastic hinge regions. Each link element consists of two joints to connect the link to the nearby elements, separated by finite length, which was taken as 0.001 m (creating virtually a zero length link) in the current model. These links were modeled to simulate moment-rotation,  $M-\Theta$ , relationships of plastic hinges. The properties of the links for the columns and the beams are summarized in Tables 8.9 and 8.10, respectively. All other degrees of freedom (three translations and two rotations) were fixed. The “Pivot Hysteresis Model” suggested by Dowell et al. (1998) was assigned to each link and used as the numerical model for the hysteretic relationship. The hysteretic model had three pivot parameters that were calibrated from the overall hysteretic relationship of the frame obtained experimentally from the bare control frame test. These parameters ( $\alpha$  pivot,  $\beta$  pivot, and  $\eta$ ) were described previously in Section 8.3.1.



### 8.6.2 Nonlinear Properties of Retrofitted Buildings

The nonlinear properties of the retrofitted buildings are similar to those of the unretrofitted buildings, except for adding diagonal buckling restrained braces (BRBs) in exterior frames. Figures 8.23 through Figure 8.26 illustrate the configurations of the BRBs schematically. The retrofitted building for Ottawa is illustrated in Figure 8.23 schematically. The retrofitted buildings for Vancouver are illustrated schematically in four different configurations shown in Figures 8.23 through 8.26.

The retrofitted building models were modeled in the same manner as the idealized retrofitted test frame model illustrated in Figure 8.4. Each BRB steel core bar, illustrated schematically in Figure 6.35 (b) of Chapter 6, was modeled with three link elements connected in series. They represented the reduced middle portion (yielding segment representing  $\frac{1}{2}$  of the bar length), and two un-reduced portions (elastic segments each representing  $\frac{1}{4}$  of the bar length). These link properties were representative of two lumped BRB braces, one at each exterior frame. Links were fixed at their joints for translational and rotational displacements except for the uniaxial deformation direction, which was assigned the nonlinear force-deformation relationship. Hinges connecting the BRB core bars to building frame elements were modelled using frame elements of 0.7 m length, representing the hinge length used in the experimental program with a high flexural rigidity. Moment releases were also specified at joints connecting these rigid frame elements with BRB core link elements. This was done to simulate hinge connections and eliminate any secondary moments at BRB bar ends.

The test frames were designed as a  $\frac{2}{3}$  scale model of a six-storey prototype building located in Ottawa. They represented the central bay of a first-storey exterior frame. The building was designed according to the provisions of NBCC 1965. Hence it was deficient in resisting seismic forces associated with the 2010 NBCC hazard values. The retrofitted test frames had BRBs with sufficient areas of core steel to upgrade the frames to the NBCC 2010 seismic requirements. The retrofit design for prototype buildings involved finding the areas of BRB steel cores, which were needed to develop similar improvements in the retrofitted prototype buildings. This was done by scaling up the BRBs used in the laboratory specimens. Full scaling up of the size used in the laboratory test frames would imply that the area of the same type of bar (AISI Type 304 stainless steel in this case) had to be increased by a factor of  $(\frac{3}{2})^2 = 2.25$ . While this was done

for one of the retrofitted buildings in Vancouver (Configuration IV), a smaller total area of steel bars was used in other configurations corresponding to a scale factor of 1.5, to explore the effectiveness of selected BRB arrangements while promoting more yielding and energy dissipation with somewhat reduced areas of BRBs. BRB areas of upper floors were made in groups (having the same core area) for every couple of floors, and they were proportioned to the percentages of base shears acting on individual floor levels relative to the base shear as described in Appendix III. This implies that the BRB's were sized for every two floors of the building with higher steel areas used at lower floors to maintain similar demand-capacity ratios. The diameter of the steel bars used as the BRB core for the building in Vancouver, schematically represented by Configuration I in Figure 8.23, was proportionally higher than those used for Ottawa, with the ratio being representative of seismic demand ratios based on NBCC 2010. This ratio was calculated to be 2.4 based on the selected ductility force-modification factor of  $R_d = 2.0$  and the over strength force-modification factor of  $R_o = 1.4$ . These values, as explained in Section 6.7 of Chapter 6, are similar to those assigned to moderately ductile shear walls in NBCC 2010. The fundamental period of the structure was calculated using the empirical expression prescribed in the 2010 NBCC with the applicable increase factor of up to 2.0. This approach resulted in a period for braced frames equal to 1.1 sec compared to 0.55 sec found by the empirical code equation. The rationale for selecting the former  $R_d$  and  $R_o$  factors and the expression for the period calculation was based on the fact that the BRBs would provide some ductility to the non-ductile conventional building system, and the building would perform as a braced frame building. Retrofitted building models of Configuration II and III for Vancouver, as illustrated in Figures 8.24 and 8.25, represent configurations where the BRBs are placed at all floor levels of both exterior frames either in each of the two exterior bays or the adjacent two bays, respectively. The areas of BRB steel cores used in these configurations had approximately one half the area used in Configuration I, which had single braces used in the middle bay, corresponding to the nearest commercially available bar size. The BRB configuration IV shown in Figure 8.26, had BRBs in all three bays of both exterior frames, and is intended for the building in Vancouver. The BRB steel core area in this case was approximately  $\frac{1}{2}$  the area of the BRB in Configuration I consisting of bars with nearest commercially available size. Table 8.11 provides the sizes of steel bars used in each retrofitted building with different BRB configuration.

The nonlinear axial-force axial-deformation relationship assigned to the BRB model was similar to that used for retrofitting the laboratory repaired frame. Wen plasticity model

(1979) was used, with axial-deformation properties calculated by proportioning the coupon test values of the BRB steel bar that was used in the frame test (Figure 8.7). For all BRB models, the post-yield stiffness ratio was 0.2% and the yielding exponent, which controls the sharpness of the yielding segment of the curve, was equal to 1.0. Equations 8.2 to 8.4 were used with appropriate BRB lengths and proper magnification factors. Table 8.11 represents the mechanical properties of all BRBs used to retrofit the buildings in Ottawa and Vancouver. The areas and stiffnesses of BRBs were lumped in one diagonal element in the model as illustrated in Figures 8.23 through 8.26.

### **8.7 Response History Analyses and Dynamic Properties of Prototype Buildings**

The selected buildings for seismic analysis were analyzed first under gravity loads based on the provisions of NBCC 2010. The analysis results were used as initial conditions for the subsequent seismic response history analysis. The response history analysis in SAP2000 was conducted by utilizing a step-by-step numerical integration method. This method can be used to evaluate structural response due to arbitrary loading at defined time intervals through solving the equation of motion of the multi-degree-of-freedom (MDOF) system. This equation of motion is given by:

$$[8.8] \quad M \ddot{u}(t) + C \dot{u}(t) + K u(t) = F(t)$$

Where  $M$ ,  $C$ , and  $K$  are mass, damping, and stiffness matrices of the system, whereas  $\ddot{u}$ ,  $\dot{u}$ , and  $u$  are accelerations, velocities and displacements of the structure measured relative to the ground under the specified ground motion, respectively.  $F(t)$  is the force vector for ground excitation.

Geometric nonlinearities were introduced in both the nonlinear static analysis and the response history analysis through P-Delta effects. P-Delta effect is important for structures experiencing significant seismic lateral forces and deformations. The incorporation of this effect is a requirement when gravity loads are acted in conjunction with lateral seismic forces (FEMA Sec. 3.2.5).

During response to seismic excitations, structures dissipate energy through damping, which is often characterized by the mechanism of viscous behaviour. Therefore, damping of structures are often referred to as “viscous damping”. The characteristics of

damping can be established experimentally for single-degree-of-freedom (SDOF) systems. However, in MDOF this property is relatively difficult to measure due to the complexity of the damping phenomenon, since the primary energy-loss mechanisms are seldom fully understood (Clough and Penzien 2003). Damping in direct integration time-history analysis can be modeled using a full damping matrix  $[C]$  that allows coupling of the effects of different modes (CSI 2010). The damping matrix is calculated as a linear combination of the mass matrix  $[M]$  multiplied by the mass coefficient  $C_m$  and the stiffness matrix  $[K]$  multiplied by the stiffness coefficient  $C_k$ . These two coefficients can be calculated because of orthogonality of the undamped mode shapes with respect to each of these matrices, as given in below:

$$[8.9] \quad C_m = \frac{4\pi(T_i \zeta_i - T_j \zeta_j)}{T_i^2 - T_j^2}$$

$$[8.10] \quad C_k = \frac{T_i T_j (T_j \zeta_i - T_i \zeta_j)}{\pi (T_j^2 - T_i^2)}$$

Where  $T_i$  and  $T_j$  are periods of vibration of the  $i^{th}$  and  $j^{th}$  modes, whereas  $\zeta_i$  and  $\zeta_j$  are the critical damping ratios for modes  $i$  and  $j$ , respectively.

The two damping coefficients can be specified directly in SAP2000, or can be computed by the program by defining the damping coefficient  $\zeta$  (here assumed to be 5% of critical) and the fundamental structural periods of the structural models for the first and second vibration modes. Structural periods were calculated by running modal analysis for each building investigated. Table 8.12 summarizes structural periods for the first and second modes, along with the calculated mass and stiffness damping coefficients for the un-retrofitted and retrofitted buildings selected in this research program.

## 8.8 Selection of Earthquake Ground Motion Records

The primary objective of this segment of the analytical investigation was to assess the deficiency of older buildings that were designed prior to the enactment of recent seismic provisions of modern building codes and to evaluate the effectiveness of the newly developed seismic retrofit methodology in the form of the use of BRBs. This objective has been fulfilled by conducting response history analyses of un-retrofitted and retrofitted buildings under the seismic hazard values specified in the 2010 National

Building of Canada (NBCC). The Code prescribes spectral accelerations defined by the uniform hazard spectrum (UHS) which was derived based on the probability of exceedance of 2% in 50 years. The code provides 5% damped spectral acceleration values at periods of 0.2, 0.5, 1.0, and 2.0 seconds for different municipalities across the country, and allows linear interpolation for in-between values. These spectral acceleration values were obtained by considering a range of earthquake records, where their magnitudes and distances contribute most strongly to the hazards. Cities of Ottawa and Vancouver were selected to reflect eastern and western Canadian seismicity. The UHS values corresponding to the periods indicated previously, denoted here as  $S_{a_{targ}}$  are 0.64g, 0.31g, 0.14g, 0.046g for Ottawa and 0.94g, 0.64g, 0.33g, 0.17g for Vancouver.

UHS-compatible ground motion records, generated synthetically by Atkinson (2009a) were used in the dynamic analyses. These artificial records were provided as sets of 90 records for each Site Class (A, C, D and E in NBCC-2010) and for different moment magnitudes (M) and fault distances (R). For eastern Canada, ground motions were simulated by two sets; M6 for R-values ranging between 10 km and 30 km, matching the short-period range of the UHS; and M7 for R-values ranging between 15 km and 100 km, matching the long-period range of the UHS. For western Canada, on the other hand, the ground motions were simulated by three sets; M6.5 for R-values ranging between 10 km and 15 km; and between 20 km and 30 km, matching the short-period range of the UHS, and M7.5 for R-values ranging between 15 km and 25 km and between 50 km and 100 km, matching the long-period range of the UHS. Additionally, to simulate significant future earthquakes that may result from the megathrust event of long durations on the Cascadia subduction zone, another set was used with M9 for R-values ranging between 100 km and 200 km.

A total number of 450 records were considered to select the best matching records to the target uniform hazard spectrum specified in the 2010 NBCC, for both Cities of Ottawa and Vancouver. Elastic spectral values,  $S_{a_{sim}}$  for each record was computed by PRISM Program (Seong-Hoon et al. 2010), a program that was developed for seismic response analysis of structures idealized as SDOF systems. The selection process among those 450 records was based on the procedure suggested by Atkinson (2009b). Two-period ranges of interest were selected; one for the short period range (0.1 sec to

0.5 sec), and the other for the long period range (0.5 sec to 2 sec). The ratio of the  $Sa_{targ}/Sa_{sim}$  was computed for each record and mean and standard deviation values were calculated over the selected period ranges of interest. Records of lowest standard deviation but having a mean of approximate range of 0.5 sec to 2.0 sec were chosen as candidate records to be used in the dynamic analysis. The mean value of the  $Sa_{targ}/Sa_{sim}$  of each of these selected records was used afterwards as a scale factor for every point on the accelerogram. Eight earthquake records were selected for the City of Ottawa, which consisted of four short duration records, as shown in Figure 8.27, and four long duration records, as shown in Figure 8.28. For the City of Vancouver, a total of twelve earthquake records were selected which consisted of four short duration records, as shown in Figure 8.29, four long duration records, as shown in Figure 8.30, and four Cascadia records, as shown in Figure 8.31. Table 8.13 summarizes the characteristics of the artificial earthquake records used in dynamic analysis.

Spectral accelerations of the selected records were plotted and compared with the 2010 NBCC Uniform Hazard Spectra for Ottawa and Vancouver. Figures 8.32 and 8.33 illustrate the acceleration response spectra of four short and four long events, respectively, for Ottawa. Average values of each set were calculated and then compared to the UHS as shown in Figure 8.34. The comparison revealed excellent agreement in the short period range of interest from 0.2 sec to 0.5 sec, while the long period range of interest showed lower spectral acceleration values between 0.5 sec and 0.8 sec. Excellent agreement was attained between 0.8 sec to 4 sec. Figures 8.35, through 8.37 illustrate acceleration response spectra of four short, four long, and four Cascadia events for Vancouver. Similarly, average values of four records in each set were calculated and then compared with the UHS as shown in Figure 8.38. The comparison revealed a very good agreement for the short period records from 0.2 sec to 1.2 sec, and excellent agreement for the long period records from 0.4 sec to 4 sec. For the Cascadia record events, the comparison indicated a reasonable agreement of spectral acceleration average values in a range of structural periods between 1 sec and 2 sec. However, the comparison revealed that the spectral values are lower than the UHS when the period is less than 1.0 s and higher when greater than 2 sec.

## 8.9 Analysis Results of Prototype Buildings

This section presents the results of dynamic analysis for unretrofitted and retrofitted



prototype buildings designed for Ottawa and Vancouver. The building frames analyzed are illustrated in Figure 8.21 and Figures 8.23 to 8.28. The objective and scope of the dynamic analysis of prototype buildings are explained in the “Introduction” sub-section, along with a discussion of the structural performance levels suggested by ASCE-41 (2006 and 2014), which are used to assess the performance of buildings analyzed.

### **8.9.1 Introduction**

One of the primary objectives of this study was to assess the seismic response of un-retrofitted and retrofitted prototype buildings, originally designed and built based on the 1965 NBCC, under the 2010 NBCC compatible earthquake intensity for eastern and western Canada. Buckling restrained braces were used in different configurations as the retrofit technique for strength and ductility enhancements. This objective was achieved by conducting a comprehensive investigation, through inelastic response time history analyses. The analyses were carried out using Uniform Hazard Spectrum compatible earthquake records for the cities of Ottawa and Vancouver. In addition, a particular set of analyses was conducted for the buildings in Ottawa under amplified Uniform Hazard Spectrum compatible earthquake records to assess the residual strength and ductility in this medium-seismic region. The governing critical earthquake records, among the 450 considered, were selected for the analyses. The results were assessed in terms of drift response, storey base shears, as well as member force and ductility demands.

The lateral drift response was recorded for both interstorey and overall roof drift ratios. The drift ratios were recorded in both positive and negative directions. The performance levels and damage indices used were consistent with those specified in ACI 374.2R-13 (ACI 2013), and ASCE 41-06 (ASCE 2006), and were based on interstorey drift ratios. Accordingly, the structural target performance levels were defined as four discrete limits of damage states and two intermediate performance ranges. The performance levels were specified for primary structural elements; which include those that are part of the lateral-force-resisting system; and secondary elements, which include structural components other than those for seismic resistance. The frame elements investigated in the current investigation fall in the “primary structural elements” category.

The four discrete performance levels specified in ASCE-41 (2006) are; Immediate Occupancy (S-1), Life Safety (S-3), Collapse Prevention (S-5), and Not Considered (S-

6). The two intermediate structural performance ranges are Damage Control range (S-2), which is defined as the continuous range of damage states between Life Safety (S-3) and Immediate Occupancy (S-1) performance levels; and Limited Safety range (S-4) which is defined as the continuous range of damage states between Life Safety (S-3) and Collapse Prevention (S-5) performance levels. Not Considered (S-6) performance level is defined for structural assessment that does not involve any structural component (e.g. bracing parapets or anchoring hazardous materials and materials storage containers, etc.).

Immediate Occupancy (S-1) performance level is defined as “post-earthquake damage state in which a structure remains safe to occupy.” The structure essentially retains its pre-earthquake design strength and stiffness. Life Safety performance level (S-3) is defined as “post-earthquake damage state in which a structure has damaged components but retains a margin against onset of partial or total collapse”. Collapse Prevention performance level (S-5) is defined as “post-earthquake damage state in which a structure has damaged components and continues to support gravity loads but retains no margin against collapse” (ASCE 2006). The three discrete structural performance levels were illustrated as a function of damage states of primary and secondary components as well as storey-drift ratios.

The 2006 ASCE-41 performance levels were quantified for different types of structural systems either for overall building response or for individual element response. The overall building response was specified as approximate indicators of performance levels, and were expressed in terms of interstorey drift ratios. Individual member performance was quantified in terms of individual deformation limits assigned to each performance level as acceptance criteria for that specific level of performance. This implies that a member meeting the acceptance criterion of a specific deformation level for a given performance level would be deemed to have satisfied this specific structural performance criterion.

For overall response of concrete frame buildings, the storey-drift ratio for Immediate Occupancy structural performance level (S-1) is not to exceed 1% transient drift and having a negligible permanent drift. For Life Safety structural performance level (S-3), the storey-drift ratio is limited to 2% and 1% for transient and permanent drifts,

respectively. For Collapse Prevention structural performance level (S-5), the storey-drift ratio is limited to 4% of transient or (S-5) permanent drifts.

ASCE 41-06 (ASCE 2006) does not specify performance levels for concrete frames retrofitted with steel braces. Instead, performance levels for structural wall buildings and braced steel frames are provided. Both of these systems that have bracing elements share the same structural performance level for Immediate Occupancy (S-1) with a transient drift ratio not to exceed 0.5% with a negligible permanent drift. The Collapse Prevention (S-5) level is also specified for both braced systems as 2% of transient or permanent storey drifts. The transient drift limit for Life Safety (S-3) is 1% for structural wall buildings and 1.5% for braced steel frames, while both systems having the same permanent drift limit of 0.5%. The performance assessment of BRB retrofitted reinforced concrete frames of the current investigation were made by adopting the criteria given for braced steel frames.

The 2013 edition of ASCE 41 revised target performance levels slightly and changed some of the terminology. The two previous intermediate structural performance ranges; Damage Control range (S-2) and Limited Safety range (S-4) were made performance levels instead. The structural performance level of Damage Control (S-2) was defined to be taken halfway between Immediate Occupancy (S-1) and Life Safety (S-3); and the structural performance level of Limited Safety (S-4) was defined to be halfway between Life Safety (S-3) and Collapse Prevention (S-5). Therefore, target performance levels were defined as six discrete limits of damage states S (1-6). Consequently, two new intermediate ranges were defined as Enhanced Safety Range, which was defined as the continuous range of damage states between the performance levels of Life Safety (S-3) and Immediate Occupancy (S-1), while the Reduced Safety Range was defined as the continuous range of damage states between performance levels of Life Safety (S-3) and Collapse Prevention (S-5).

ASCE 41 (ASCE 2013) does not provide quantifiable limits for overall building response, but provides descriptive definitions of each performance level based on observed damage states. Drift levels are defined qualitatively as a function of expected damage to non-structural damage. Accordingly, Immediate Occupancy (S-1) is defined as overall response with little yielding and transient drift that causes minor or no non-structural

damage. Life Safety (S-3) is associated with extensive damage to beams some spalling of cover concrete and shear cracking with transient drift sufficient to cause non-structural damage and noticeable permanent drift. Collapse prevention corresponds to excessive cracking and hinge formation in ductile elements with severe damage in short columns, and transient drift sufficient to cause extensive non-structural damage and extensive permanent drift. The ASCE-41 (2013) standard does, however provide revised deformation limits for individual elements as acceptance criteria for different performance limits. Hence, quantification is to be done at the element level. Plastic chord rotation limits are specified for frame elements to identify the level of performance met. Acceptance criteria for inelastic actions demonstrated by concrete braced frames are stated for frames that are restrained by braces (called braced frames) and for those that have no braces (called isolated frames). The braced frame acceptance criteria are set similar to reinforced concrete infilled frames criteria having numerical limit values for tension and compression strain capacities. For columns that are not confined along their entire length, these numerical values were 0.002 for Immediate Occupancy (S-1) and 0.01 for both Life Safety (S-3) and Collapse Prevention (S-5) structural performance levels.

ASCE 41-13 (ASCE 2014) stated new performance criteria for BRBs added to an existing building system. Structural performance levels for buckling restrained braces used in braced steel frames are defined for: Immediate Occupancy (S-1) which refers to “minor yielding or buckling of braces”, Life Safety (S-3) which refers to “many braces yield or buckle but do not totally fail, many connections may fail.” Collapse Prevention (S-5) refers to “extensive yielding and buckling of braces, many braces and their connections may fail”. For these performance levels, BRB axial deformations are limited to 3, 10, and 13.3 times the yield capacity at the expected brace capacity, respectively. Similarly, structural performance levels of Damage Control (S-2) and Limited Safety (S-4) would be limited to 6.5 and 11.7 times the yield deformation at the expected capacity, based on the performance definitions stated earlier. The BRBs axial deformation performance levels were set solely if BRBs were tested to comply with cyclic qualification tests prescribed in a related standard ANSI/AISC 341-10 (AISC 2010). Otherwise, these acceptance criteria shall be multiplied by a factor of 0.7. In addition, the maximum strain of the BRB core is limited not to exceed a value of 2.5%.

Most recent Canadian Steel Standard, S16-14 (CSA 2014), focuses on “general performance based design requirements together with qualification testing requirements for the bracing members”. These requirements are found to agree with the BRB requirements stated in ANSI/AISC 341-10. However, neither of these standards has specifications for structural performance levels of BRBs used in reinforced concrete buildings.

Dynamic base shear demand was another response quantity that was important in assessing the effectiveness of BRB retrofitting. Therefore, it was calculated for each building, under each earthquake record. Base shear was also calculated using the Equivalent Static Load Approach three times; i) using the 1965 NBCC as the code that was in effect when the buildings were designed, ii) using the 2010 NBCC and the applicable empirical equations for period calculations, and iii) using the 2010 NBCC with 50% and 100% elongation of periods permitted over the empirical period calculations for the unretrofitted and braced frames, respectively. The 2010 NBCC Equivalent Static Analysis values were computed to fulfill the code requirements of dynamic base shears not to be lower than those computed based on the static shears. It should be noted that the static base shears were obtained by applying the ductility related force modification factor ( $R_d$ ) and the overstrength related force modification factor ( $R_o$ ). These factors were taken as ( $R_d$ ) = 2.0 and ( $R_o$ ) = 1.5 for unretrofitted buildings and ( $R_d$ ) = 1.5 and ( $R_o$ ) = 1.3 for the retrofitted buildings. Furthermore, localized critical elements of buildings and building columns were assessed under earthquake records of critical total base shears. Force demands were also recorded for shears, moments, and axial forces and are compared with nominal sectional capacities listed in Table 8.8. The demand values were taken as the maximum over the entire time history.

It is noted that shear and flexural response under nonlinear dynamic analyses are not in phase. Therefore, their ratios, commonly referred to as shear amplification factors, are not equal to the ratios of shear to flexure computed from static linear analysis or linear dynamic response spectrum analysis (Rad and Adebar 2008). The amplifications observed in dynamic analyses were attributed to the higher mode effects as well as the hinging of structural elements (Rad and Adebar 2008).

Moment-cord rotation relationships were examined for selected critical elements of un-

retrofitted and retrofitted buildings, under governing earthquake records, to assess the hysteretic characteristics of elements. The critical elements selected for this purpose were one column and one beam at the first-floor level of each of the exterior and interior frame. The analytical model for the frames, also showing column lines, is schematically represented in Figure 8.22. The first storey columns were examined at the link element at column base on Column Line 2 while the first-floor beams between Column Lines 1 and 2 were examined at links representing plastic hinges adjacent to the central columns.

In the current investigation, the ductility provided by BRBs were examined at all floor levels, under the governing earthquake motion. These axial-force deformation responses were investigated by examining the behaviour of link elements in the structural model representing the reduced section of BRBs in the middle of the steel core. The un-reduced sections of BRB cores were investigated only at the first-floor level, as they were expected to behave elastically elsewhere.

Further description of the analyses results of the prototype buildings are presented in the following sections. The comparisons of analysis results are presented at the end of this Chapter.

### **8.9.2 Analysis of Un-retrofitted Building in Eastern Canada**

The un-retrofitted building in eastern Canada, illustrated in Figure 8.21, was first analyzed using two sets of ground motions, each consisting of four records matching the short and long period ranges of the 2010 NBCC UHS for Ottawa. The interstorey drift ratios along the height of the building are shown in Figure 8.39. The maximum interstorey drift ratio was 0.47% recorded at the second floor under the Long Event # 2 earthquake record. This brings the building to Immediate Occupancy (S-1) structural performance level. The short event records did not govern the response. The same record was critical for the top lateral displacement with a magnitude of 73 mm, as well as the overall drift ratio, which was 0.33%. Figures 8.40 and 8.41 show the top lateral displacement response of the building under short and long period events, respectively.

Dynamic base shear demands were calculated for all the records employed and then compared with those obtained from the Equivalent Static Force Procedure according to



NBCC 1965 and 2010, as illustrated in Figures 8.42 to 8.49. Long Event # 4 generated maximum total base shear of 3747 kN. The total base shear value that the building was designed for, based on NBCC 1965 (3403 kN), was found to be 90% of the maximum dynamic base shear value. Furthermore, the maximum dynamic base shear was found to be lower than the NBCC 2010 base shear value calculated based on the empirical period expression that resulted in  $T = 0.76$  sec, which was equal to 4630 kN. This was greater than that based on the permissible increased period of 1.14 sec, which was equal to 2652 kN.

Localized shear, bending moment, and axial force for building columns were investigated under response history analysis for the critical earthquake record, i.e., Long Event # 4. These demand forces are shown in Figure 8.50. They were then compared with nominal sectional capacities. The comparison revealed that none of the columns under dynamic analysis exceeded its nominal capacity.

Moment-chord rotation hysteretic relationships for critical beams and columns of the first storey were investigated under the governing UHS compatible Long Event # 4. All the members remained elastic as illustrated in Figure 8.51. The critical exterior and interior beams developed a maximum of 47% and 57% of their yield moments, respectively, while the critical exterior and interior columns developed a maximum of 50% and 58% of their yield moments, respectively.

It is evident from these analyses results that the selected building in Ottawa does not require seismic retrofiting, remaining elastic under the code compatible earthquake record. The structural performance level was found to be within the Immediate Occupancy (S-1) limit, with all the elements having sufficient capacities. Carriere, 2007, also arrived at the same conclusion when he analyzed regular buildings in Ottawa, ranging between 5 and 15 storeys. This finding could be attributed to the structural overstrength, and especially to the higher load factors prescribed in the 1965 NBCC (see Equation 3.7 of Chapter 3).

The above conclusion was drawn for a building that was regular in plan and elevation, and subjected to earthquake records that were compatible with the 2010 NBCC UHS, which was based on ground motions with a 2% probability of exceedance in 50 years.

These UHS spectral accelerations are the mean values of accelerations obtained from the use of earthquake records of different moment magnitudes (M) and fault distances (R) that contribute most strongly to the hazard. However, greater hazard levels with different random intensities might occur based on lower probability of exceedance. Therefore, the critical earthquake records, Long Events # 2 and 4, were amplified by 100% and used to investigate the seismic response of the building using dynamic inelastic response history analysis.

Figures 8.52 and Figure 8.53 illustrate the maximum interstorey drift ratios and top lateral displacement response of the un-retrofitted building in Ottawa, respectively, analyzed under the governing UHS code compatible and amplified Long Event #2. The maximum interstorey drift ratio was limited to 0.94% at the second floor level while the top lateral displacement response of the building and the overall drift ratio were 0.149 m and 0.68%, respectively. The structural performance level of the building remained within the Immediate Occupancy (S-1) level.

Maximum base shear obtained from the dynamic analysis under the amplified shear-critical Long Event # 4 was found to be 6965 kN. This was almost 1.86 times the base shear obtained under the un-magnified record. This dynamic base shear was compared with those obtained from the Equivalent Static Force Procedure of NBCC 1965 and 2010 as illustrated in Figure 8.54. The NBCC 1965 base shear was found to be 48% of the maximum dynamic base shear. The static base shear of NBCC 2010 based on the empirical period expression was 66%, while that obtained based on the permissible increase in period was 38% of the maximum dynamic base shear developed under the governing UHS code amplified Long Event #-4.

Column force demands under the amplified shear-critical Long Event #4 are provided in Figure 8.55. These demands were compared with nominal column capacities. Axial force and bending moment demands were found to remain below the column sectional capacities while the nominal shear capacities were exceeded in 6 first-storey columns of the central bay at the first storey level. This is illustrated in Figure 8.55.

Moment-chord rotation hysteretic relationships for critical columns and beams were investigated at the first storey level under the amplified earthquake record, as illustrated

in Figure 8.56. The beams remained elastic during response, attaining maximum moments of 74% and 82% of their yield moments in exterior and interior frames, respectively. The columns of the interior frames, however, experienced inelasticity with a maximum ductility ratio of 1.28, while the columns of the exterior frames remained elastic with a maximum of 82% of their yield moments.

### **8.9.3 Analysis of Un-retrofitted Building in Western Canada**

Three different sets of earthquake records, matching the UHS of NBCC 2010 for Vancouver, were used to analyse the un-retrofitted building in Western Canada illustrated in Figure 8.21. These sets of records were four short, four long, and four Cascadia events. The results with respect to interstorey drift ratios along the height of the building are plotted in Figure 8.57 and show a maximum drift ratio of 2.3% recorded at the first storey when subjected to the Cascadia Event #1. The building is therefore considered to be within the Collapse Prevention (S-5) structural performance level. Top lateral displacement response histories were plotted under the same three sets of records in Figures 8.58, 8.59, and 8.60. Maximum top lateral displacement was 0.264 m (1.2% overall drift ratio of the building) recorded under the Cascadia Event # 1.

Figures 8.61 to 8.72 illustrate the plots of base shear time histories under the same three sets of earthquake records. The base shears are compared with those obtained from the Equivalent Static Force Procedures of NBCC 1965 and 2010. Maximum total base shear was 9113 kN recorded under the Cascadia Event #1. The original design base shear value of 3384 kN (based on NBCC 1965) was found to be 37% of the maximum dynamic base shear. The maximum dynamic base shear was found to be lower than the NBCC 2010 base shear value calculated based on the empirical period expression ( $T = 0.76$  s), which was equal to 9992 kN, and greater than that based on the permissible elongation of period ( $T = 1.14$  s), which was equal to 6426 kN.

Columns of the building were investigated for their shear, bending moment, and axial force response histories under the shear-critical Cascadia Event # 1. They are compared with nominal sectional capacities. Deficient column members were identified schematically in Figure 8.73. Column shear capacities of the exterior frames were exceeded in 14 columns at the first three stories of the building. In addition 30 columns of the interior frames exceeded their shear capacities at the first and second stories.

This brings the total number of shear deficient columns to 44. Bending moments and axial forces imposed on all building columns were within their capacity limits.

To assess the level of inelasticity developed in critical beams and columns of the first storey, moment-chord rotation hysteretic relationships are plotted in Figure 8.74, obtained from the analysis under the governing UHS code compatible Cascadia Event #1. The beams of the exterior and interior frames both developed 97% of their failure moments and maximum rotational ductility ratios of 2.7 and 3.1, respectively. The columns of the exterior and interior frames slightly exceeded their moment capacities and developed 102% and 105% of their failure moments, and 127% and 178% of their ultimate rotations, respectively. Columns developed maximum ductility ratios of 5.4 and 6.6, for the exterior and interior frames, respectively.

#### **8.9.4 Analysis of Retrofitted Building in Eastern Canada**

The retrofitted building in eastern Canada, illustrated in Figure 8.23, was analyzed using two sets of earthquake records, each consisting of four ground motions, matching the short and long period ranges of the 2010 NBCC UHS for Ottawa. Interstorey drift ratios along the height are shown in Figure 8.75. The maximum interstorey drift ratio was 0.43% recorded at the first storey level by the Long Event # 4. Similar to the response of the un-retrofitted building, the short event records did not govern response. The critical record generated the top maximum lateral displacement of 0.071 m and overall drift ratio of 0.32%. Figures 8.76 and 8.77 show the top lateral displacement response of the building under the short and long period events, respectively. As illustrated, the retrofitting has marginal effects on interstorey and overall drift ratios. The structural performance level of the building is judged to be at the Immediate Occupancy (S-1) level. Figures 8.78 and 8.79 illustrate the maximum interstorey drift ratios and top lateral displacement response of the retrofitted building. The maximum interstorey drift ratio was limited to 0.85% at the second floor level. The maximum top displacement was 0.138 m and the overall drift ratio was 0.63%.

Dynamic base shear time histories are compared with static base shear values in Figures 8.80 to 8.87. The Long Event # 4 generated the maximum total base shear of 4284 kN. The original design base shear (based on NBCC 1965) was 3403 kN, which is equal to 79% of the maximum dynamic base shear computed. Furthermore, the

maximum dynamic base shear was found to be almost equal to the NBCC 2010 base shear value of 4275 kN calculated based on the empirical period expression with  $T = 0.55$  s.

The retrofitted building was also analyzed for dynamic base shear under the amplified shear-critical Long Event # 4. The dynamic base shear demand was found to be 7444 kN under this record. This value is almost 1.74 times the base shear obtained under the same record without the amplification. The dynamic base shear was also compared with those obtained from the Equivalent Static Force Procedure of NBCC 1965 and 2010, as illustrated in Figure 8.88. NBCC 1965 base shear was found to be 46% of the maximum dynamic base shear. The static base shear of NBCC 2010 based on the empirical period expression was 57% of the dynamic base shear, while that obtained based on the elongated permissible period was equal to 26% of the maximum dynamic base shear.

Localized shear force, bending moment, and axial force demands in columns under the amplified earthquake record are shown in Figure 8.89. These force demands were compared with column nominal capacities. The comparison revealed that none of the dynamic force demands for columns exceed their nominal capacities.

Figure 8.90 illustrates hysteretic relationships for selected first-storey beams and columns, obtained under the amplified shear-critical earthquake record. The critical exterior and interior beams developed 70% and 78% of their yield moments, respectively, while the critical exterior and interior columns developed 90% and 100% of their yield moments, respectively. In other words, all the frame elements behaved elastically.

Finally, axial force-axial deformation response of the BRBs used in the building were investigated at each floor level under the amplified earthquake record. The hysteretic relationships are shown in Figure 8.91. The BRBs showed stable hysteretic behavior at all building floor levels. The un-reduced sections of BRBs, outside the critical reduced portion, developed a maximum ductility demand ratio of 1.28 at the first-floor level, corresponding to a maximum strain of 0.29%. The maximum ductility demand in the reduced sections were 3.55; 3.44; 3.14; 2.43; 1.82; and 1.0 at the first through sixth floor

levels, respectively. BRBs of the first storey developed the maximum strain of 0.79% while BRB strains of upper stories progressively decreased up to a strain value of 0.22% at the sixth storey. All BRB strains were less than the maximum strain value of 2.5%. Also, all BRBs were within the Enhanced Safety structural performance range and never exceeded the Damage Control (S-2) performance level.

### **8.9.5 Analysis of Retrofitted Building in Western Canada**

The retrofitted building located in Vancouver was assessed for its seismic response in four retrofit configurations illustrated in Figures 8.23 to 8.26. Retrofitted building configuration (I) was analyzed under three sets of earthquake records, consisting of short period, long period, and Cascadia events. Each set had four records matching the NBCC 2010 UHS spectral accelerations for Vancouver. The other three retrofitted building configurations (II to IV) were analyzed only for the critical earthquake record that governed response for the retrofitted building configuration (I). This was justified because all retrofitted buildings had relatively close fundamental periods (Table 8.12) with approximately the same spectral acceleration demands. The response of each retrofit configuration is discussed separately in the following sections with comparisons and conclusions presented at the end of the Chapter.

#### **8.9.5.1 Retrofitted Building in Western Canada - Configuration I**

Figure 8.92 illustrates the maximum interstorey drift ratios along the height of the retrofitted building in Vancouver (Configuration I) under 12 earthquake records that match the NBCC 2010 UHS spectral accelerations for Vancouver. The maximum interstorey drift ratio was limited to 1.22%, recorded at the first storey level under the Cascadia Event #1. This maximum drift ratio brings building structural performance level to the Life Safety (S-3) range. Figures 8.93 to 8.95 show the top lateral displacement response resulting from the use of the same 12 earthquake records. Cascadia Event #3 was found to be the critical record developing a maximum displacement of 0.179 m and an overall drift ratio of 0.81%.

Dynamic base shear demands were calculated for 12 records and then compared with those obtained from the Equivalent Static Force Procedure of NBCC 1965 and 2010. This is illustrated in Figures 8.96 to 8.107. The Cascadia Event #1 generated maximum total base shear of 10350 kN. The total base shear value that the building was designed



for, based on the NBCC 1965, was 3384 kN, which was found to be 33% of the maximum dynamic base shear demand. The maximum dynamic base shear was also found to be 116% of the NBCC 2010 base shear value of 8873 kN, calculated based on the empirical period expression with  $T = 0.55$  s, and 226% of that based on the elongated permissible period with  $T = 1.1$  s, which was equal to 4575 kN.

Columns of the prototype building were investigated for their shear, bending moment, and axial force response time histories under the shear-critical Cascadia Event #1. The analysis results are compared with nominal sectional capacities, and deficient column members are identified in Figure 8.108. Bending moments and axial forces of all building members were within their capacity limits. However, column shear capacities of the exterior frames were exceeded in 6 columns at the first and second stories; and for the interior frames, 12 columns exceeded their shear capacities at the central bay of the first storey. This brings the total number of shear deficient columns of the building to 18.

Moment-chord rotation hysteretic relationships for critical beams and columns are plotted in Figure 8.109 as obtained from the analysis under the governing earthquake record. The exterior and interior beams remained elastic during response with maximum moments of 78% and 84% of their yield moments, respectively. The columns of the exterior and interior frames, however, experienced inelasticity with maximum rotational ductility ratios of 1.84 and 2.56, respectively.

The axial force-axial deformation response for the BRBs were investigated at each floor under the governing earthquake record, as illustrated in Figure 8.110. The un-reduced sections of BRBs at the first storey level showed a maximum ductility demand ratio of 2.0 and a maximum strain of 0.45%. The maximum ductility demand ratios of BRBs in reduced sections were 5.0, 3.3, 2.52, 1.73, 1.0, and 0.36 in the first to sixth stories, respectively. The BRBs of the first storey developed the maximum strain of 1.1%, while the BRB strains at upper stories progressively decreased to 0.08% at the sixth storey. None of the BRBs exceeded the maximum strain value of 2.5%. All BRB responses were within the Enhanced Safety structural performance range and they never exceeded the Damage Control (S-2) performance level.

#### 8.9.5.2 Retrofitted Building in Western Canada - Configuration II

The building in Vancouver, retrofitted with Configuration II BRBs, was analyzed under Cascadia Events #1 and #3, which were critical in drift response. The analysis results are shown in Figure 8.111 with respect to interstorey drift ratios. The maximum drift ratio was recorded to be 1.16% at the first storey level under the Cascadia Event # 1. The structure satisfied the Life Safety (S-3) performance level. The top lateral displacement response was computed under the Cascadia Event # 3 to be 0.176 m, corresponding to 0.8% lateral drift. This is illustrated in Figure 8.112.

Figure 8.113 shows dynamic base shear demands under the shear-critical Cascadia Event # 1. The comparison of dynamic base shear demands with that obtained from the Equivalent Static Force Procedure of NBCC 1965 indicates that the initial design base shear of 3384 kN is equal to 33 % of the maximum dynamic base shear demand. When compared with the static base shear of NBCC 2010, the maximum dynamic base shear was found to be equal to 115% of the code base shear of 8873 kN based on the empirical period expression of  $T = 0.55$  s, and 224% of 4575 kN based on the permissible elongation of the period to  $T = 1.1$  s.

The analysis results were compared with nominal capacities of members. It was found that flexural and axial force capacities were not exceeded, but four first-storey columns of the exterior frames were found to be deficient in shear at the interior bay. The shear-deficient columns are identified and depicted schematically in Figure 8.114. In addition, 12 columns of the interior frames exceeded their shear capacities at the first storey level. This brings the total number of shear deficient columns to 18.

Hysteretic moment-chord rotation relationships for first-storey critical beams and columns are plotted in Figure 8.115. The Figure indicates that the beams remained elastic with maximum moments of 72% and 82% of yield moments in exterior and interior frames, respectively. The columns of exterior and interior frames experienced inelasticity with maximum ductility ratios of 1.85 and 2.32, respectively.

The axial force-axial deformation response for each BRB used to retrofit the building was investigated under the governing earthquake record. The BRB response in the left and right retrofitted bays of exterior frames showed almost identical response as depicted in

Figures 8.116 and 8.117, respectively. For the retrofitted left bay, the un-reduced sections of the first storey BRBs showed a maximum ductility demand ratio of 1.5 and a maximum strain of 0.34%. The maximum ductility demand ratios for the reduced BRB sections in the first to sixth stories were 4.1, 2.8, 2.4, 1.82, 1.3, and 0.66, respectively. BRBs of the first storey developed a maximum strain of 0.15%. All BRB strains were less than the maximum strain of 2.5%. Furthermore, all BRB responses were within the Enhanced Safety structural performance range, and they never exceeded the Damage Control (S-2) performance level.

### **8.9.5.3 Retrofitted Building in Western Canada - Configuration III**

Figure 8.118 illustrates the maximum interstorey drift ratios for the building in Vancouver, retrofitted with Configuration III. These drift ratios were obtained using the drift-critical earthquake records of Cascadia Events #1 and #3. The maximum interstorey drift ratio was limited to 1.13% recorded at the first storey level by the Cascadia Event #1, while the maximum top lateral displacement response was 0.173 m, recorded when the Cascadia Event #3 was used. This is illustrated in Figure 8.119. The overall drift ratio was recorded to be 0.79%. The structural performance level was at the Life Safety (S-3) level.

Dynamic base shear demands were calculated for the shear-critical Cascadia Events #1 and then compared with those obtained from the Equivalent Static Force Procedures of NBCC's 1965 and 2010 in Figures 8.120. The original design base shear based on the 1965 NBCC was 3384 kN, which was found to be 34% of the maximum dynamic base shear of 10110 kN. The maximum dynamic base shear was also found to be 114% of the NBCC 2010 base shear calculated based on the empirical period expression with  $T = 0.55$  s, which was equal to 8873 kN. It was equal to 221% of the base shear when the period was elongated to the permissible period of 1.1 s, which was equal to 4575 kN.

Columns of the prototype building were investigated for their shear, moment, and axial force demands under the shear-critical earthquake record, i.e., the Cascadia Event # 1. The analysis results were compared with nominal member capacities. The shear-deficient columns were identified as illustrated schematically in Figure 8.121. Moments and axial forces for all members were within their capacity limits. However, column shear capacities of the exterior frames were exceeded in six columns at the first storey level.

In addition, twelve first-storey columns exceeded their shear capacities at the central bay of interior frames. This brought the total number of deficient columns for the building to 18 columns.

Moment-chord rotation hysteretic relationships for critical beams and columns, located at the first storey level of exterior and interior frames are plotted in Figure 8.122. The beams remained elastic during response with maximum moments of 73% and 80% of their yield moments in the exterior and interior frames, respectively. The columns, however, experienced inelasticity with maximum ductility ratios of 1.6 and 2.2 for exterior and interior frames, respectively.

The axial force-axial deformation response for the BRBs were investigated at each storey. The hysteretic relationships recorded are illustrated in Figures 8.123 and 8.124. The first-storey BRB in the central bay developed a maximum ductility demand ratio of 1.56 and a maximum strain of 0.35% in un-reduced sections. The maximum BRB ductility demand ratios were 5.15, 3.85, 3.22, 2.33, 1.56, and 0.7 in the reduced sections for the first to sixth stories, respectively. BRBs of the first storey developed a maximum strain of 1.15% while the BRB strains of upper stories progressively decreased to a strain value of 0.16% at the sixth storey level. None of the BRBs exceeded the maximum strain value of 2.5%. All BRB responses were within the Enhanced Safety structural performance range and never exceeded the Damage Control (S-2) performance level. The retrofitted right bay showed slightly lower ductility demands and strain values compared to the central bay. This is due to the asymmetric loading path in the bracing arrangements within the exterior frames, resulting in greater BRB demand forces at the retrofitted central bay.

#### **8.9.5.4 Retrofitted Building in Western Canada - Configuration IV**

The retrofitted building located in Vancouver with Configuration IV BRBs, was analyzed for interstorey and overall building drift ratios under drift-critical earthquake records, i.e., Cascadia Events #1 and #3. The interstorey drift ratios are illustrated in Figure 8.125, showing a maximum ratio of 0.92% recorded at the first storey level. The structural performance level of the building at the maximum drift response was at the Life Safety (S-3) level. Top lateral displacement response of the building was investigated as well, and presented in Figure 8.126. The maximum top lateral displacement was 0.143 m

corresponding to an overall drift of 0.65%.

Figure 8.127 illustrates dynamic base shear demands under the shear-critical Cascadia Event # 1. The dynamic base shear of 9130 kN, relative to the original design base shear of 3384 kN, obtained from the Equivalent Static Force Procedure of NBCC 1965, was significantly in excess of the design value, with the design base shear forming only 37% of the dynamic shear demand. A similar comparison was made with the NBCC 2010 base shears. The maximum dynamic base shear was found to be 103% of the NBCC 2010 base shear when calculated based on the empirical period expression with  $T = 0.55$  s, which was equal to 8873 kN, and 200% of that computed based on the elongated permissible period of 1.1 s, which was equal to 4575 kN.

The analysis results, in terms of maximum shear forces, bending moments and axial forces, are shown in Figure 8.128. When compared with nominal member capacities, all the force capacities indicate higher values than the demands. Therefore, none of the building columns has become deficient after retrofitting.

The moment-chord rotation hysteretic relationships for critical first-storey beams and columns are shown in Figure 8.129 obtained under the same shear-critical Cascadia Event # 1. The beams of the exterior and interior frames remained elastic during response with maximum moments of 63% and 73% of their yield moments, respectively. The columns of the exterior and interior frames also remained elastic with maximum moments of 80% and 90% of their yield moments, respectively.

The axial-force axial-deformation response for each BRB was investigated under the governing Cascadia Event #1 to assess their performance. The BRB response for the exterior frames are illustrated in Figures 8.130 to 8.132. The left and right retrofitted bays showed approximate similar response due to symmetry, with lower demands than that for the central bay. For the retrofitted central bay at the first-storey level, the un-reduced sections of BRB showed a maximum ductility demand ratio of 1.47 and a maximum strain of 0.33%. The reduced BRB sections indicated maximum ductility demand ratios of 2.95, 2.57, 2.49, 1.82, 1.12, and 0.53 in the first through sixth floors, respectively. BRBs of the first storey developed a maximum strain of 0.66% while the BRB strains of upper stories progressively decreased to a strain value of 0.12% at the sixth storey. None of

the BRBs exceeded the maximum strain value of 2.5%. Furthermore, all BRB responses were within the Enhanced Safety structural performance range, and never exceeded the Damage Control (S-2) performance level.

#### **8.9.6 Comparison and Summary of Buildings Performances**

Performance of un-retrofitted and retrofitted buildings in eastern and western Canada are summarized in this Section. Comparisons are made between the maximum response quantities obtained from dynamic response time history analyses of buildings. Table 8.14 summarizes interstorey drifts and overall drift ratios obtained at the building roof level. The maximum interstorey drift ratio for un-retrofitted buildings in eastern Canada is less than 1% and the structural performance level is within the Immediate Occupancy (S-1) level when the 2010 NBCC UHS compatible earthquake records are used. The results also show that retrofitting has a marginal effect on drift demands of these buildings. In western Canada, un-retrofitted buildings showed a maximum interstorey drift ratio of 2.3%, which is within the Collapse Prevention (S-5) structural performance level. By retrofitting exterior frames in four different BRB configurations (I – IV), maximum lateral interstorey drift ratios were reduced to 0.92% to 1.22% placing the building in the Life Safety (S-3) structural performance level. The overall drift demands were less than those for interstorey drifts. The retrofit configurations I to III indicated approximately similar interstorey drift demands as illustrated in Figures 8.133 and 8.134. The configuration IV resulted in the most drift control.

Table 8.15 summarizes base shear values based on the NBCC Equivalent Static Force Procedure, computed using the 1965 and 2010 editions of the code. Static base shears of NBCC 2010 based on the empirical period expressions were higher than those based on the elongated periods permitted in the code. The NBCC 1965 values were 70% and 80% of the NBCC 2010 values computed using empirically calculated periods for eastern un-retrofitted and retrofitted buildings, respectively. The same comparison for western un-retrofitted and retrofitted buildings indicated 30% and 40%, respectively. It is noteworthy that, although the retrofitted structures had shorter periods because of the inherent rigidity of braced structures, the base shear was reduced in these buildings because of the effective transfer of seismic forces to the foundation by the first storey BRBs. Therefore, the foundation design of retrofitted structures requires special attention. Another factor for the reduction in base shear is the use of higher  $R_d = 2.0$  and



$R_o = 1.4$  factors for seismic resistant braced frames compared to  $R_d = 1.3$  and  $R_o = 1.5$  used for un-retrofitted conventional frames, implying that the ductility and energy dissipation introduced by BRBs improve seismic response.

The maximum dynamic base shear demands obtained from the analyses are summarized in Table 8.16. The total dynamic base shear for the un-retrofitted building in eastern Canada, originally designed following the requirements of NBCC 1965, was found to be only 10% under-designed. However, for the un-retrofitted building in western Canada, the design base shear force was about 37% of the dynamic base shear demand. The dynamic base shear values for un-retrofitted buildings were greater than those obtained from the Equivalent Static Force Procedure based on the empirical period expressions. When retrofitted, the dynamic base shear increased by 14.3% for the building in eastern Canada, and by 13.6%, 12.8%, 10.9%, 0.2 % for the buildings in western Canada with retrofit configurations of I through IV, respectively.

Table 8.17 shows the number of columns that exceeded shear capacities of un-retrofitted and retrofitted prototype buildings in eastern and western Canada. It is evident from the results that reinforced concrete frame buildings similar to that designed in the current investigation for Ottawa, following the 1965 NBCC requirements, do not require seismic retrofitting, and they remain elastic under the 2010 NBCC compatible earthquake records. If however, similar buildings were subjected to a higher intensity earthquake, with a hazard level 100% higher than the code anticipated level, they then would be vulnerable, and would develop base shear demands approximately equal to twice the base shear capacities. In this case, the analysis results indicated that six internal columns would exceed their nominal shear capacities and hence would require retrofitting. For buildings in western Canada, a total of 44 columns exceeded their nominal shear capacities. The columns of exterior and interior frames indicated failure and developed maximum ductility ratios of 5.4 and 6.6, respectively. Through retrofitting, the number of shear deficient columns reduced to 18, 16, 18, and zero with maximum column ductility ratios of 2.6, 2.3, 2.2, and 0.9 for configurations I, II, III, IV, respectively. The performance level for columns and beams of the frame with configuration IV retrofit was Immediate Occupancy (S-1).

The ductility demands of BRBs, used to retrofit the buildings considered in the current

investigation, are summarized in Table 8.18. The retrofitted building in Ottawa was analyzed under 100% amplified Long Event # 4, while the buildings in Vancouver, retrofitted with configurations I through IV, were analyzed under the UHS code compatible Cascadia Event #1. The BRBs showed stable hysteretic behaviour in all building types and at floor levels. All the BRBs performed within the Enhanced Safety structural performance range, never exceeding the Damage Control (S-2) structural performance level.

The BRB steel core dimensions were determined to promote yielding and energy dissipation through significant inelastic deformations during seismic response while the frame elements remain essentially elastic at expected BRB capacity. This was achieved in prototype buildings by proportioning BRBs to resist the same percentage of base shear as that in the experimental repaired and retrofitted frame (RRF). This calculation was based on static analyses. However, these static base shear percentages may not reflect the actual dynamic force distribution at each floor. Therefore, yielding may occur in the frame members. BRB design, like any other design, involves a trial and error process. Using larger areas of BRBs results in stiffer structural systems with increased seismic force demands. Conversely, reduced sections of BRBs result in softer structural systems with higher interstorey drifts, increased force demands in frame elements and increased ductility demands in BRBs. A recommended BRB retrofit design procedure is provided in Chapter 9.

Table 8.1: Plastic hinge properties used for the bare control frame elements

| Flexural hinge properties    | Columns | Beam                 |                         |
|------------------------------|---------|----------------------|-------------------------|
|                              |         | Compression side top | Compression side bottom |
| $\phi_y \cdot 10^{-6}$ (rad) | 12      | 13.8                 | 11                      |
| $\phi_u \cdot 10^{-6}$ (rad) | 45      | 170                  | 195                     |
| $\Theta_y$ (rad)             | 0.0037  | 0.0045               | 0.0036                  |
| $\Theta_u$ (rad)             | 0.0139  | 0.056                | 0.0313                  |
| $M_y$ (KN·m)                 | 167     | 77                   | 186                     |
| $M_u$ (KN·m)                 | 185     | 81                   | 196                     |

Table 8.2: Wen plasticity properties used for BRB steel core bar elements

| Properties                     | Middle reduced bar section | Edges un-reduced bar sections |
|--------------------------------|----------------------------|-------------------------------|
| Diameter (mm)                  | 31.8                       | 44.5                          |
| Stiffness (KN·m)               | 117068                     | 458540                        |
| Yield force (kN)               | 354                        | 693                           |
| Post yield stiffness ratio (%) | 0.2                        | 0.2                           |
| Yielding exponent              | 1                          | 1                             |

Table 8.3: Beam design forces for the prototype building

| Frame (Grid)   | Beam (Grid)      | Floor Level | Moment for top reinforcements (KN·m) | Moment for bottom reinforcements (KN·m) | Shear (kN) |
|----------------|------------------|-------------|--------------------------------------|-----------------------------------------|------------|
| Interior (2-7) | Interior B-C     | 1           | 573                                  | 120                                     | 382        |
|                |                  | 2-6         | 500                                  | 104                                     | 324        |
|                | Exterior A-B/C-D | 1           | 705                                  | 356                                     | 447        |
|                |                  | 2-6         | 595                                  | 257                                     | 360        |
| Exterior (1&8) | Interior B-C     | 1           | 434                                  | 195                                     | 244        |
|                |                  | 2-6         | 394                                  | 214                                     | 215        |
|                | Exterior A-B/C-D | 1           | 499                                  | 238                                     | 260        |
|                |                  | 2-6         | 418                                  | 191                                     | 214        |

Table 8.4: Column design forces for the prototype building

| Frame (Grid)   | Column (Grid) | Floor level | Moment (KN·m) | Axial load (kN) | Shear (kN) |
|----------------|---------------|-------------|---------------|-----------------|------------|
| Interior (2-7) | Interior B&C  | 1           | 492           | 3793            | 223        |
|                |               | 2-6         | 581           | 2989            | 270        |
|                | Exterior A&D  | 1           | 410           | 2131            | 165        |
|                |               | 2-6         | 399           | 1686            | 216        |
| Exterior (1&8) | Interior B&C  | 1           | 446           | 1880            | 176        |
|                |               | 2-6         | 385           | 1486            | 217        |
|                | Exterior A&D  | 1           | 330           | 1227            | 153        |
|                |               | 2-6         | 153           | 966             | 151        |

Table 8.5: Beam sectional sizes and reinforcement arrangements for the prototype building

| Frame (Grid)   | Beam (Grid) | Floor level | Beam section (mm) and reinforcement |           |            |                        |
|----------------|-------------|-------------|-------------------------------------|-----------|------------|------------------------|
|                |             |             | Section (mm)                        | Top steel | Bot. steel | Stirrups (Bar No. @mm) |
| Interior (2-7) | A-D         | 1           | 400x 600                            | 8-25M     | 7-20M      | 10M@ 75                |
|                | A-D         | 2-6         | 400x 600                            | 7-25M     | 5-20M      | 10M@ 100               |
| Exterior (1&8) | A-D         | 1           | 400x 600                            | 6-25M     | 5-20M      | 10M@ 150               |
|                | A-D         | 2-6         | 400x 600                            | 5-25M     | 4-20M      | 10M@ 175               |

Table 8.6: Column sectional sizes and reinforcement arrangements for the prototype building

| Frame (Grid)   | Floor level | Column section (mm) and reinforcement |                   |                        |                      |                   |                        |
|----------------|-------------|---------------------------------------|-------------------|------------------------|----------------------|-------------------|------------------------|
|                |             | Interior columns B&C                  |                   |                        | Exterior columns A&D |                   |                        |
|                |             | Section (mm)                          | Longitudinal bars | Stirrups (Bar No. @mm) | Section (mm)         | Longitudinal bars | Stirrups (Bar No. @mm) |
| Interior (2-7) | 1           | 500x 500                              | 12-25M            | 10M@ 125               | 450x 450             | 8-25M             | 10M@ 150               |
|                | 2-6         | 500x 500                              | 8-25M             | 10M@ 100               | 450x 450             | 8-25M             | 10M@ 125               |
| Exterior (1&8) | 1           | 450x 450                              | 12-25M            | 10M@ 150               | 450x 450             | 8-25M             | 10M@ 175               |
|                | 2-6         | 450x 450                              | 8-25M             | 10M@ 125               | 450x 450             | 8-25M             | 10M@ 150               |

Table 8.7: Beam flexural and shear nominal capacities for the prototype building

| Frame (Grid)   | Beam (Grid) | Floor level | Moment for top reinforcements (KN·m) | Moment for bottom reinforcements (KN·m) | Shear (kN) |
|----------------|-------------|-------------|--------------------------------------|-----------------------------------------|------------|
| Interior (2-7) | A-D         | 1           | 838                                  | 466                                     | 608        |
|                | A-D         | 2-6         | 739                                  | 336                                     | 456        |
| Exterior (1&8) | A-D         | 1           | 638                                  | 334                                     | 304        |
|                | A-D         | 2-6         | 542                                  | 265                                     | 260        |

Table 8.8: Columns axial, flexural, and shear nominal capacities for the prototype building

| Frame (Grid)   | Column       | Floor level | Bending moment at balanced point (KN·m) | Axial force at balanced point (kN) | Pure bending moment (KN·m) | Pure axial load (kN) | Shear (kN) |
|----------------|--------------|-------------|-----------------------------------------|------------------------------------|----------------------------|----------------------|------------|
| Interior (2-7) | Interior B&C | 1           | 795                                     | 3449                               | 527                        | 8622                 | 301        |
|                |              | 2-6         | 686                                     | 3646                               | 357                        | 7873                 | 376        |
|                | Exterior A&D | 1           | 540                                     | 2664                               | 316                        | 6662                 | 224        |
|                |              | 2-5         | 540                                     | 2664                               | 316                        | 6662                 | 269        |
| Exterior (1&8) | Interior B&C | 1           | 636                                     | 2964                               | 464                        | 7410                 | 224        |
|                |              | 2-6         | 540                                     | 2664                               | 316                        | 6661                 | 269        |
|                | Exterior A&D | 1           | 540                                     | 2664                               | 316                        | 6661                 | 192        |
|                |              | 2-6         | 540                                     | 2664                               | 316                        | 6661                 | 224        |



Table 8.9: Axial and flexural resistances and calculated plastic hinge properties for the columns of the prototype building

| Column location                                       | Floor level | P (kN) | $M_y$ (kN·m) | $L_p$ (m) | $\Theta_y$ (rad) | $\Theta_u / \Theta_y$ |
|-------------------------------------------------------|-------------|--------|--------------|-----------|------------------|-----------------------|
| Interior frame grid (2-7), Exterior column grid (A&D) | 1           | 1208   | 452          | 0.39      | 0.0035           | 4.33                  |
|                                                       | 2           | 984    | 434          | 0.34      | 0.0032           | 4.71                  |
|                                                       | 3           | 790    | 414          | 0.34      | 0.0034           | 5.17                  |
|                                                       | 4           | 596    | 391          | 0.34      | 0.0037           | 5.60                  |
|                                                       | 5           | 401    | 366          | 0.34      | 0.0041           | 6.29                  |
|                                                       | 6           | 207    | 336          | 0.34      | 0.0046           | 7.23                  |
| Interior frame grid (2-7), Interior column grid (B&C) | 1           | 2090   | 712          | 0.39      | 0.0031           | 3.69                  |
|                                                       | 2           | 1702   | 688          | 0.34      | 0.0030           | 3.69                  |
|                                                       | 3           | 1366   | 661          | 0.34      | 0.0032           | 3.85                  |
|                                                       | 4           | 1030   | 632          | 0.34      | 0.0034           | 4.10                  |
|                                                       | 5           | 694    | 601          | 0.34      | 0.0035           | 4.38                  |
|                                                       | 6           | 359    | 557          | 0.34      | 0.0037           | 5.32                  |
| Exterior frame grid (1&8), Exterior column grid (A&D) | 1           | 606    | 392          | 0.39      | 0.0035           | 6.79                  |
|                                                       | 2           | 493    | 378          | 0.34      | 0.0037           | 6.22                  |
|                                                       | 3           | 396    | 365          | 0.34      | 0.0040           | 6.38                  |
|                                                       | 4           | 299    | 350          | 0.34      | 0.0044           | 6.66                  |
|                                                       | 5           | 201    | 335          | 0.34      | 0.0047           | 7.14                  |
|                                                       | 6           | 104    | 319          | 0.34      | 0.0050           | 7.85                  |
| Exterior frame grid (1&8), Interior column grid (B&C) | 1           | 1045   | 539          | 0.39      | 0.0037           | 4.24                  |
|                                                       | 2           | 851    | 421          | 0.34      | 0.0034           | 4.95                  |
|                                                       | 3           | 683    | 402          | 0.34      | 0.0035           | 5.39                  |
|                                                       | 4           | 515    | 381          | 0.34      | 0.0038           | 5.92                  |
|                                                       | 5           | 347    | 357          | 0.34      | 0.0042           | 6.40                  |
|                                                       | 6           | 179    | 331          | 0.34      | 0.0044           | 7.93                  |

Table 8.10: Flexural resistances and calculated plastic hinge properties for the beams of the Prototype Building

| Beam location and floor level              | Positive bending moment<br>(Bottom steel) |              |                     |                     | Negative bending moment<br>(Top steel) |              |                     |                     |
|--------------------------------------------|-------------------------------------------|--------------|---------------------|---------------------|----------------------------------------|--------------|---------------------|---------------------|
|                                            | $M_y$<br>(KN·m)                           | $L_p$<br>(m) | $\Theta_y$<br>(rad) | $\Theta_u/\Theta_y$ | $M_y$<br>(KN·m)                        | $L_p$<br>(m) | $\Theta_y$<br>(rad) | $\Theta_u/\Theta_y$ |
| Interior frame grid (2-7), Floor (1)       | 441                                       | 0.47         | 0.0053              | 12.17               | 795                                    | 0.47         | 0.0042              | 7.86                |
| Interior frame grid (2-7), Floor (2-Roof)  | 322                                       | 0.47         | 0.0041              | 17.67               | 700                                    | 0.47         | 0.0031              | 10.24               |
| Exterior frame grid (1&8), Floor (1)       | 321                                       | 0.49         | 0.0069              | 10.36               | 606                                    | 0.49         | 0.0039              | 11.13               |
| Exterior frame grid (1&8), Floor (2- Roof) | 251                                       | 0.49         | 0.0050              | 18.27               | 515                                    | 0.49         | 0.0029              | 17.17               |

Table 8.11: Mechanical properties of BRBs used for the retrofitted buildings in Ottawa and Vancouver

| BRB bay location within external frames | Building type and configuration             | Floor level | Reduced section |                 |                  |                  | Un-reduced sections |                 |                  |                  |
|-----------------------------------------|---------------------------------------------|-------------|-----------------|-----------------|------------------|------------------|---------------------|-----------------|------------------|------------------|
|                                         |                                             |             | Diam. (mm)      | Yield def. (mm) | Yield force (kN) | Stiffness (kN/m) | Diam. (mm)          | Yield def. (mm) | Yield force (kN) | Stiffness (kN/m) |
| Central                                 | Retrofitted Ottawa (I)                      | 5-6         | 25.4            | 5.1             | 226              | 43,873           | 31.8                | 2.6             | 353              | 137,103          |
|                                         |                                             | 3-4         | 31.8            | 5.1             | 353              | 68,551           | 38.3                | 2.6             | 508              | 197,428          |
|                                         |                                             | 2           | 38.3            | 5.1             | 508              | 98,714           | 44.5                | 2.6             | 692              | 268,721          |
|                                         |                                             | 1           | 38.3            | 6.0             | 508              | 85,259           | 44.5                | 3.0             | 692              | 232,094          |
| Central                                 | Retrofitted Vancouver (I)                   | 5-6         | 50.8            | 5.1             | 904              | 175,492          | 57.2                | 2.6             | 1144             | 444,213          |
|                                         |                                             | 3-4         | 57.2            | 5.1             | 1,144            | 222,107          | 63.5                | 2.6             | 1412             | 548,411          |
|                                         |                                             | 2           | 63.5            | 5.1             | 1,412            | 274,206          | 69.9                | 2.6             | 1709             | 663,578          |
|                                         |                                             | 1           | 63.5            | 6.0             | 1,412            | 236,831          | 69.9                | 3.0             | 1709             | 573,131          |
| Side/s                                  | Retrofitted Vancouver (II), (III), and (IV) | 5-6         | 31.8            | 7.5             | 353              | 47,128           | 38.3                | 3.7             | 508              | 135,727          |
|                                         |                                             | 3-4         | 38.3            | 7.5             | 508              | 67,864           | 44.5                | 3.7             | 692              | 184,740          |
|                                         |                                             | 2           | 44.5            | 7.5             | 692              | 92,370           | 50.8                | 3.7             | 904              | 241,293          |
|                                         |                                             | 1           | 44.5            | 8.1             | 692              | 85,330           | 50.8                | 4.1             | 904              | 222,902          |
| Central                                 | Retrofitted Vancouver (III) and (IV)        | 5-6         | 31.8            | 5.1             | 353              | 68,551           | 38.3                | 2.6             | 508              | 197,428          |
|                                         |                                             | 3-4         | 38.3            | 5.1             | 508              | 98,714           | 44.5                | 2.6             | 692              | 268,721          |
|                                         |                                             | 2           | 44.5            | 5.1             | 692              | 134,361          | 50.8                | 2.6             | 904              | 350,983          |
|                                         |                                             | 1           | 44.5            | 5.9             | 692              | 116,047          | 50.8                | 3.0             | 904              | 303,144          |

Table 8.12: Free vibration periods and damping coefficients for the un-retrofitted and retrofitted structural models

| Building type and configuration   | Period (sec) |              | Mass damping coefficient | Stiffness damping coefficient |
|-----------------------------------|--------------|--------------|--------------------------|-------------------------------|
|                                   | Mode shape1  | Mode shape 2 |                          |                               |
| Un-retrofitted building           | 1.52         | 0.48         | 0.316                    | 0.0058                        |
| Retrofitted building – East       | 1.34         | 0.44         | 0.353                    | 0.0053                        |
| Retrofitted building – West (I)   | 1.25         | 0.42         | 0.376                    | 0.005                         |
| Retrofitted building – West (II)  | 1.18         | 0.4          | 0.398                    | 0.0047                        |
| Retrofitted building – West (III) | 1.2          | 0.4          | 0.393                    | 0.0047                        |
| Retrofitted building – West (IV)  | 1.09         | 0.37         | 0.43                     | 0.0044                        |

Table 8.13: Characteristics of the artificial earthquake records used in dynamic analyses

| Prototype building location | Earthquake record    | Peak ground acceleration (cm/sec <sup>2</sup> ) | Time increment $\Delta T$ , (sec) | No. of data | Duration (sec) |
|-----------------------------|----------------------|-------------------------------------------------|-----------------------------------|-------------|----------------|
| East                        | Short Event No. 1    | 418.9                                           | 0.002                             | 1263        | 2.524          |
|                             | Short Event No. 2    | 482.6                                           | 0.002                             | 2396        | 4.79           |
|                             | Short Event No. 3    | 330.2                                           | 0.002                             | 1937        | 3.872          |
|                             | Short Event No. 4    | 329.2                                           | 0.002                             | 1799        | 3.596          |
|                             | Long Event No. 1     | 196.4                                           | 0.002                             | 6604        | 13.206         |
|                             | Long Event No. 2     | 198.6                                           | 0.002                             | 5717        | 11.432         |
|                             | Long Event No. 3     | 232.7                                           | 0.002                             | 5129        | 10.256         |
|                             | Long Event No. 4     | 214.6                                           | 0.002                             | 5588        | 11.174         |
| West                        | Short Event No. 1    | 438.8                                           | 0.005                             | 2729        | 13.64          |
|                             | Short Event No. 2    | 310                                             | 0.005                             | 2756        | 13.775         |
|                             | Short Event No. 3    | 324.1                                           | 0.005                             | 3049        | 15.24          |
|                             | Short Event No. 4    | 355.6                                           | 0.005                             | 2785        | 13.92          |
|                             | Long Event No. 1     | 326.5                                           | 0.005                             | 13039       | 65.19          |
|                             | Long Event No. 2     | 207.3                                           | 0.005                             | 9106        | 45.525         |
|                             | Long Event No. 3     | 390.1                                           | 0.005                             | 11541       | 57.7           |
|                             | Long Event No. 4     | 259.8                                           | 0.005                             | 12683       | 63.41          |
|                             | Cascadia Event No. 1 | 206.3                                           | 0.01                              | 25488       | 254.87         |
|                             | Cascadia Event No. 2 | 174.4                                           | 0.01                              | 25969       | 259.68         |
|                             | Cascadia Event No. 3 | 217.2                                           | 0.01                              | 26150       | 261.49         |
|                             | Cascadia Event No. 4 | 200                                             | 0.01                              | 26079       | 260.78         |

Table 8.14: Maximum interstorey and roof displacements under UHS code compatible and amplified artificial records

| Prototype building type    | Region | Max. interstorey |                 |                            | Max. roof |                 |                            |
|----------------------------|--------|------------------|-----------------|----------------------------|-----------|-----------------|----------------------------|
|                            |        | Floor Level      | Drift ratio (%) | Governed earthquake record | Disp. (m) | Drift ratio (%) | Governed earthquake record |
| Un-retrofitted building    | East   | 2                | 0.47            | L2                         | 0.073     | 0.33            | L2                         |
|                            |        |                  | 0.94            | L2 Scale 2                 | 0.149     | 0.68            | L2 Scale 2                 |
| Un-retrofitted building    | West   | 1                | 2.3             | C1                         | 0.264     | 1.2             | C1                         |
| Retrofitted building (I)   | East   | 1                | 0.43            | L4                         | 0.071     | 0.32            | L4                         |
|                            |        | 2                | 0.85            | L4 Scale 2                 | 0.138     | 0.63            | L4 Scale 2                 |
| Retrofitted building (I)   | West   | 1                | 1.22            | C1                         | 0.179     | 0.81            | C3                         |
| Retrofitted building (II)  |        |                  | 1.16            | C1                         | 0.176     | 0.80            | C3                         |
| Retrofitted building (III) |        |                  | 1.13            | C1                         | 0.173     | 0.79            | C3                         |
| Retrofitted building (IV)  |        |                  | 0.92            | C3                         | 0.143     | 0.65            | C3                         |



Table 8.15: Maximum base shears and fundamental periods of prototype buildings based on the NBCC Equivalent Static Force Approach

| Prototype building type      | Region | NBCC 1965              | NBCC 2010 Empirical |                        | NBCC 2010 Permissible |                        |
|------------------------------|--------|------------------------|---------------------|------------------------|-----------------------|------------------------|
|                              |        | Max. $V_{Static}$ (kN) | Period (sec)        | Max. $V_{Static}$ (kN) | Period (sec)          | Max. $V_{Static}$ (kN) |
| Un-retrofitted building      | East   | 3403                   | 0.76                | 4630                   | 1.14                  | 2652                   |
| Un-retrofitted building      | West   | 3384                   | 0.76                | 9992                   | 1.14                  | 6426                   |
| Retrofitted buildings (I)    | East   | -                      | 0.55                | 4275                   | 1.1                   | 1906                   |
| Retrofitted buildings (I-IV) | West   | -                      | 0.55                | 8873                   | 1.1                   | 4575                   |

Table 8.16: Maximum dynamic base shears of prototype buildings under governing UHS code compatible and amplified artificial records

| Prototype building type    | Region | Max. $V_{\text{Dynamic}}$ (kN) | Governed earthquake record | $V_{\text{Static}}$ (NBCC 1965)/<br>$V_{\text{Dynamic}}$ |
|----------------------------|--------|--------------------------------|----------------------------|----------------------------------------------------------|
| Un-retrofitted building    | East   | 3747                           | L4                         | 0.908                                                    |
|                            |        | 6965                           | L4<br>Scale 2              | 0.489                                                    |
| Un-retrofitted building    | West   | 9113                           | C1                         | 0.373                                                    |
| Retrofitted building (I)   | East   | 4284                           | L4                         | 0.794                                                    |
|                            |        | 7444                           | L4<br>Scale 2              | 0.457                                                    |
| Retrofitted building (I)   | West   | 10350                          | C1                         | 0.327                                                    |
| Retrofitted building (II)  |        | 10280                          | C1                         | 0.329                                                    |
| Retrofitted building (III) |        | 10110                          | C1                         | 0.335                                                    |
| Retrofitted building (IV)  |        | 9130                           | C1                         | 0.371                                                    |

Table 8.17: Number of columns exceeded shear capacities in the prototype buildings

| Prototype building type    | Region | Floor level | Governed earthquake record | Columns         |                 |       |
|----------------------------|--------|-------------|----------------------------|-----------------|-----------------|-------|
|                            |        |             |                            | External frames | Internal frames | Total |
| Un-retrofitted building    | East   | -           | L4                         | 0               | 0               | 0     |
|                            |        | 1           | L4 Scale 2                 | 0               | 6               | 6     |
| Un-retrofitted building    | West   | 1, 2, 3     | C1                         | 14              | 30              | 44    |
| Retrofitted building (I)   | East   | -           | L4 Scale 2                 | 0               | 0               | 0     |
| Retrofitted building (I)   | West   | 1, 2        | C1                         | 6               | 12              | 18    |
| Retrofitted building (II)  |        | 1           | C1                         | 4               | 12              | 16    |
| Retrofitted building (III) |        | 1           | C1                         | 6               | 12              | 18    |
| Retrofitted building (IV)  |        | -           | C1                         | 0               | 0               | 0     |

Table 8.18: Ductility demands of BRBs in core sections for retrofitted prototype buildings

| Prototype building type    | Region | Bay location within exterior frame | Un-reduced BRB sections | Reduced BRB sections |      |      |      |      |      |
|----------------------------|--------|------------------------------------|-------------------------|----------------------|------|------|------|------|------|
|                            |        |                                    | Floor level             | Floor level          |      |      |      |      |      |
|                            |        |                                    | 1                       | 1                    | 2    | 3    | 4    | 5    | 6    |
| Retrofitted building (I)   | East   | Central                            | 1.28                    | 3.55                 | 3.44 | 3.14 | 2.43 | 1.82 | 1.0  |
| Retrofitted building (I)   | West   | Central                            | 2.0                     | 5.0                  | 3.3  | 2.52 | 1.73 | 1.0  | 0.36 |
| Retrofitted building (II)  |        | Left                               | 1.5                     | 4.1                  | 2.8  | 2.4  | 1.82 | 1.3  | 0.66 |
|                            |        | Right                              | 1.48                    | 4                    | 2.74 | 2.34 | 1.76 | 1.22 | 0.59 |
| Retrofitted building (III) |        | Central                            | 1.56                    | 5.15                 | 3.85 | 3.22 | 2.33 | 1.56 | 0.7  |
|                            |        | Left                               | 1.45                    | 3.91                 | 2.72 | 2.35 | 1.79 | 1.27 | 0.65 |
| Retrofitted building (IV)  |        | Left                               | 1.02                    | 2.57                 | 1.85 | 1.61 | 1.82 | 0.89 | 0.47 |
|                            |        | Central                            | 1.47                    | 2.95                 | 2.57 | 2.49 | 1.82 | 1.12 | 0.53 |
|                            |        | Right                              | 1.0                     | 2.48                 | 1.84 | 1.65 | 1.26 | 0.92 | 0.5  |

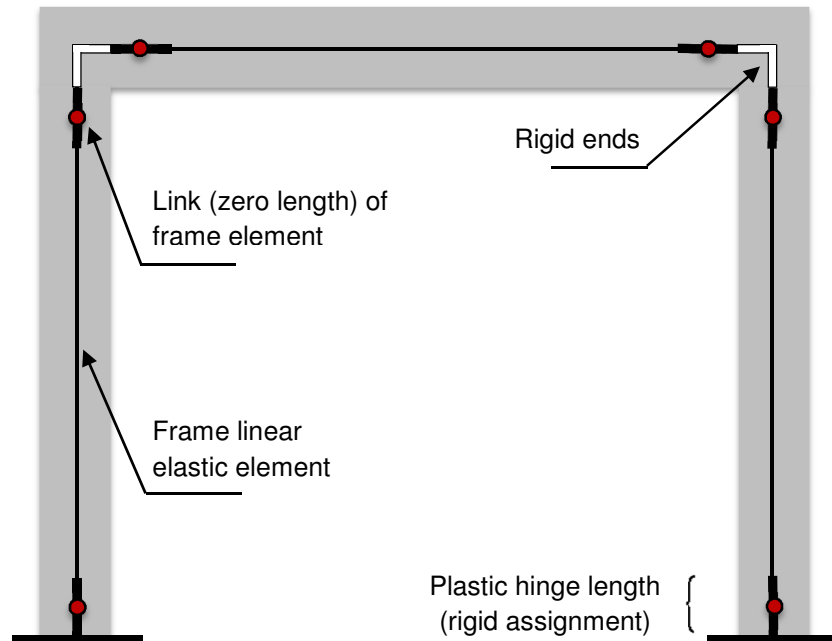


Figure 8.1: Idealized bare control frame model

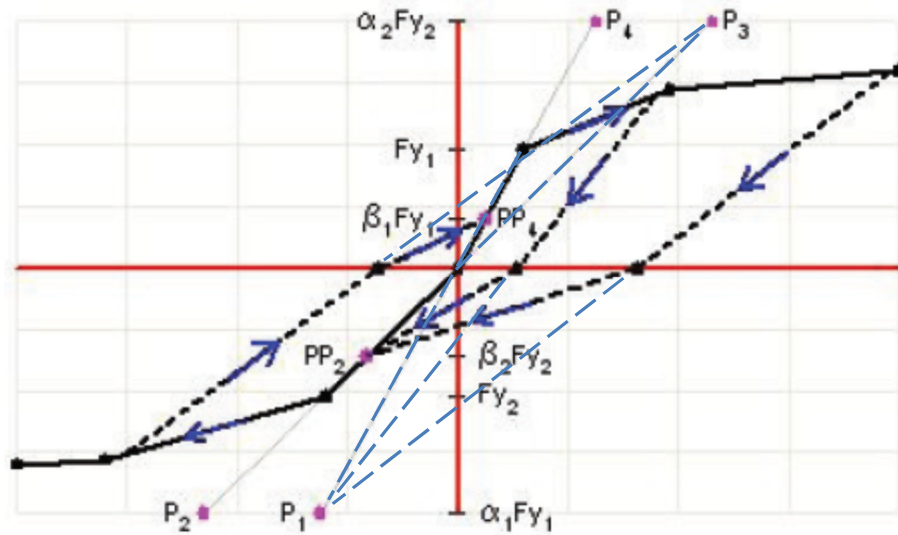


Figure 8.2: Multi-linear Takeda plasticity degrading hysteretic parameters (CSI 2009)

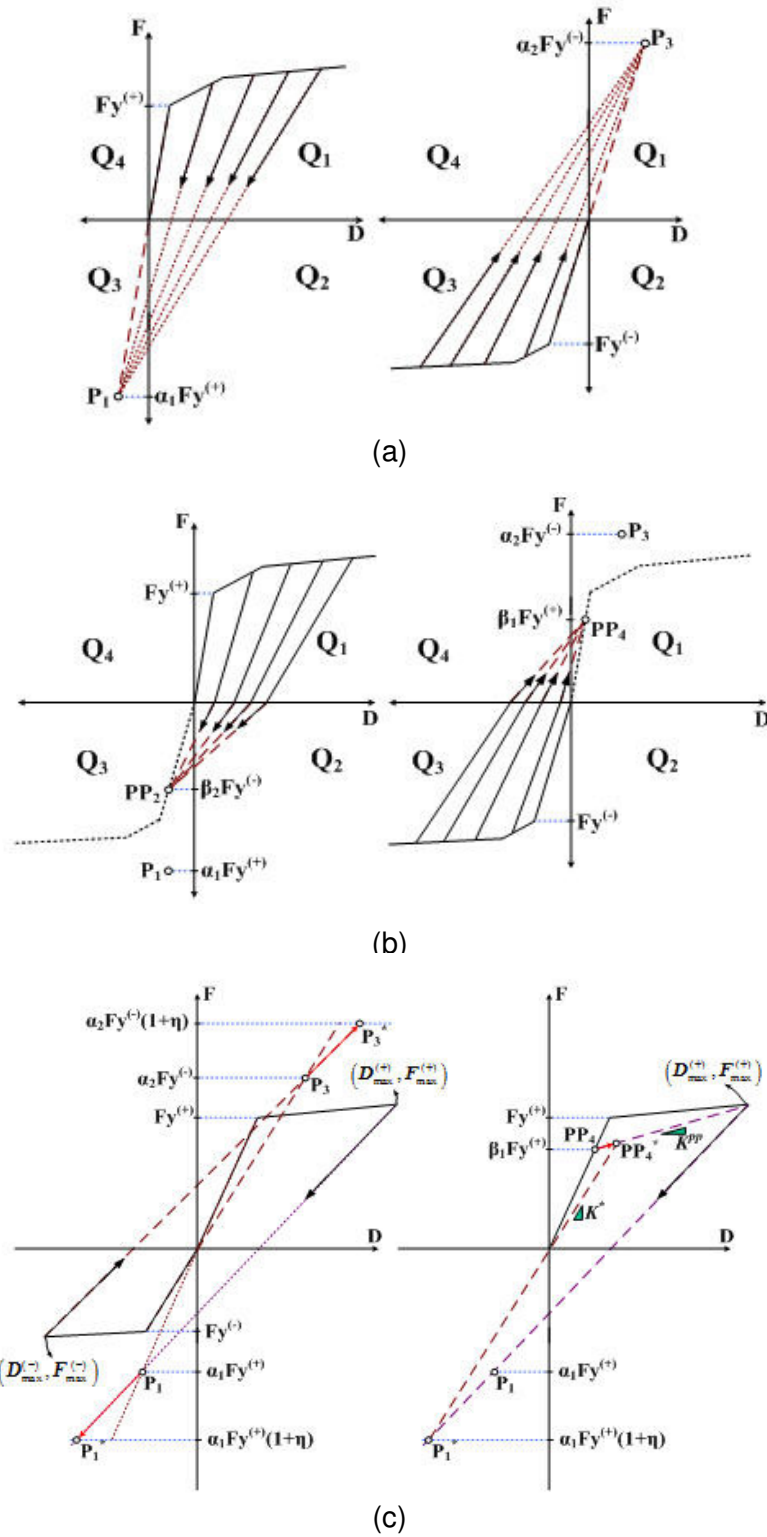


Figure 8.3: Multi-linear Takeda plasticity degrading hysteretic parameters: (a)  $\alpha$ ; (b)  $\beta$ ; and (c)  $\eta$ , (MIDAS Information Technology Co. Ltd 2015)



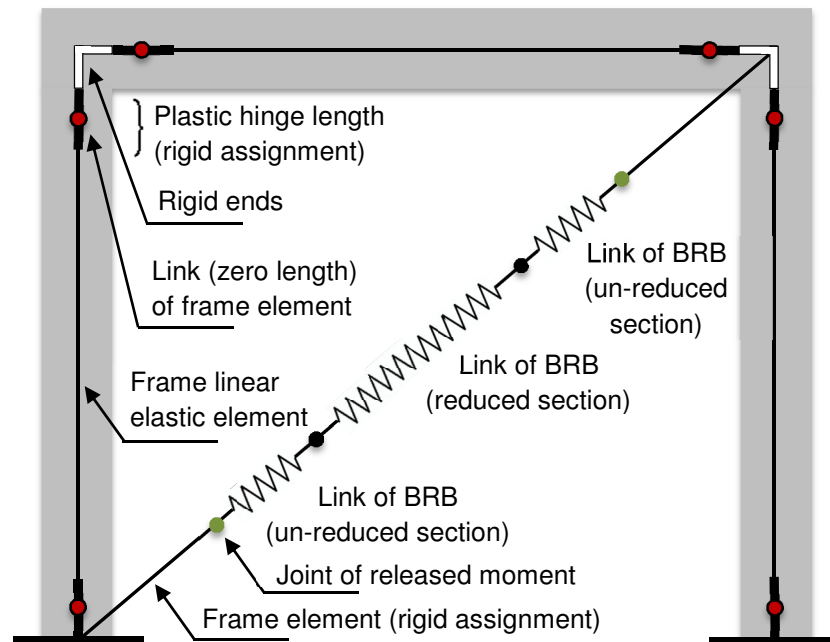


Figure 8.4: Idealized retrofitted frame model

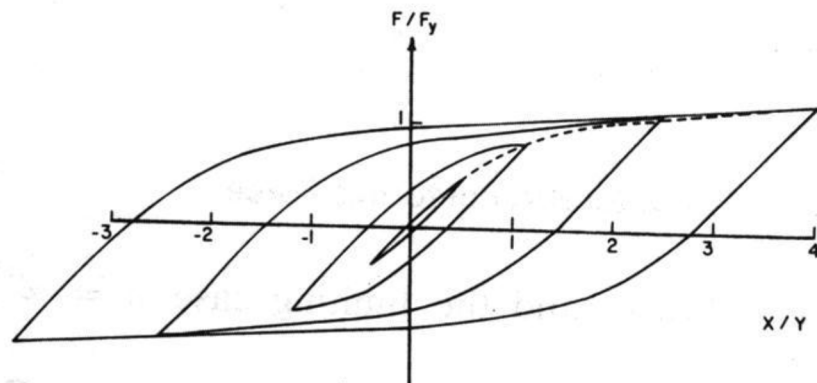


Figure 8.5: Wen plasticity force-displacement relationship model, CSI (Wen, 1976)

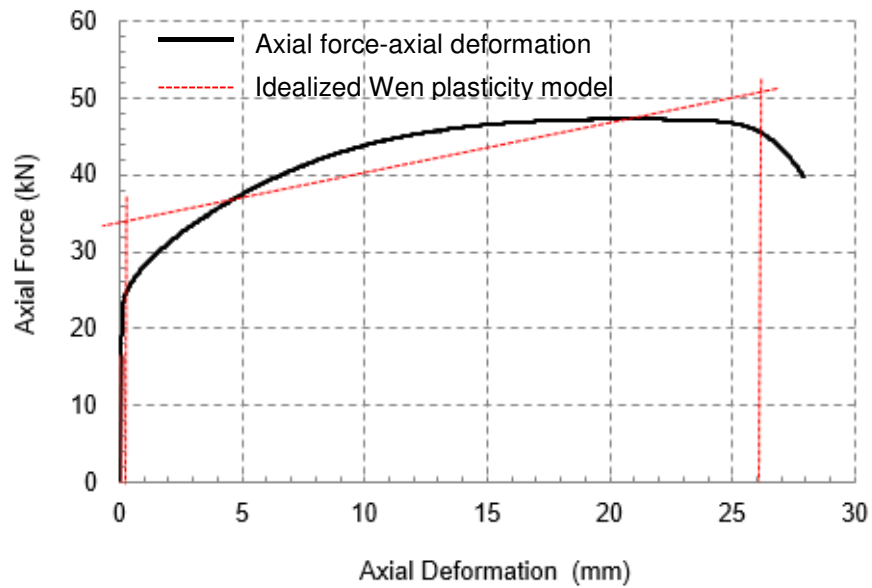


Figure 8.6: Typical axial force-axial deformation relationship of a BRB steel core bar obtained by a coupon test and Wen plasticity model idealization

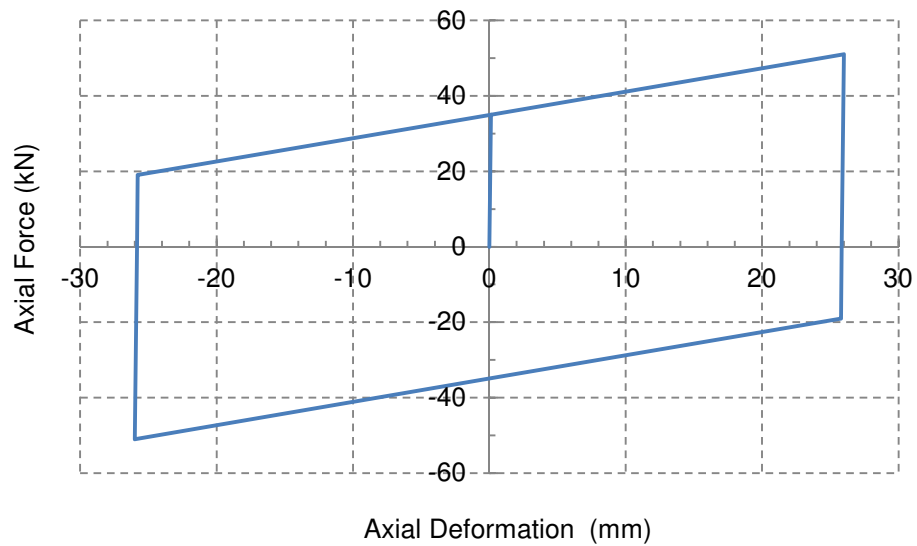
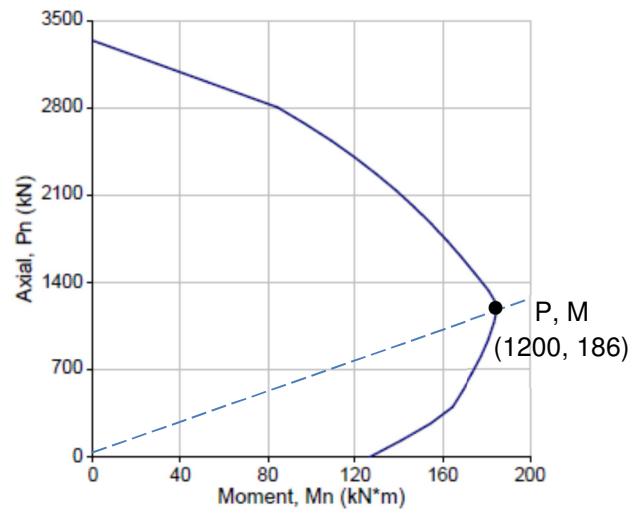
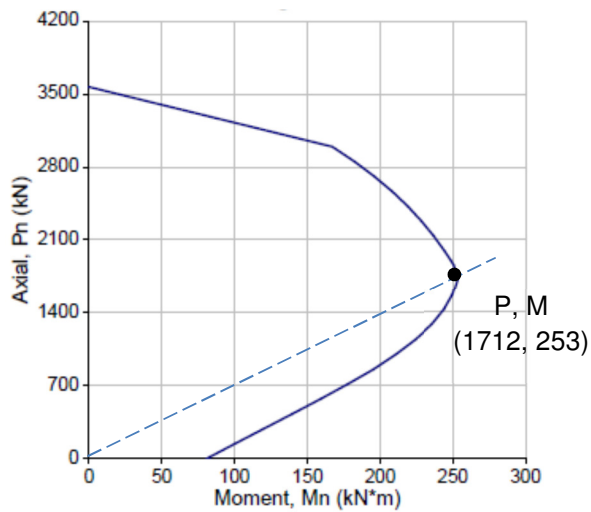


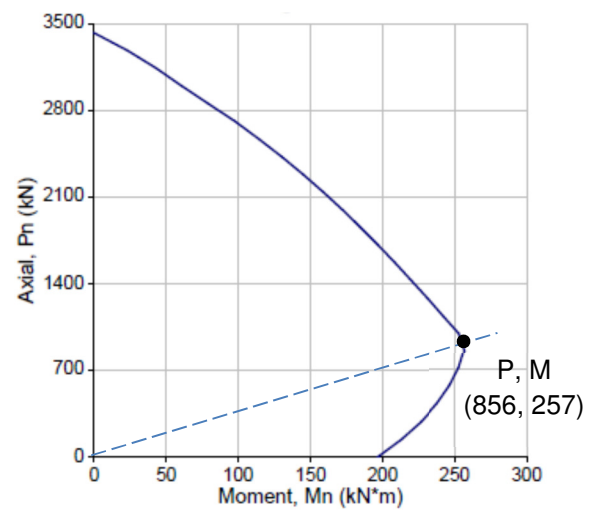
Figure 8.7: Wen plasticity model for hysteretic axial force-axial deformation relationship of a BRB steel core bar



(a)



(b)



(c)

Figure 8.8: Axial force-moment interaction diagrams for frame members (nominal quantities); (a) columns; (b) beams in positive bending; (c) beams in negative bending

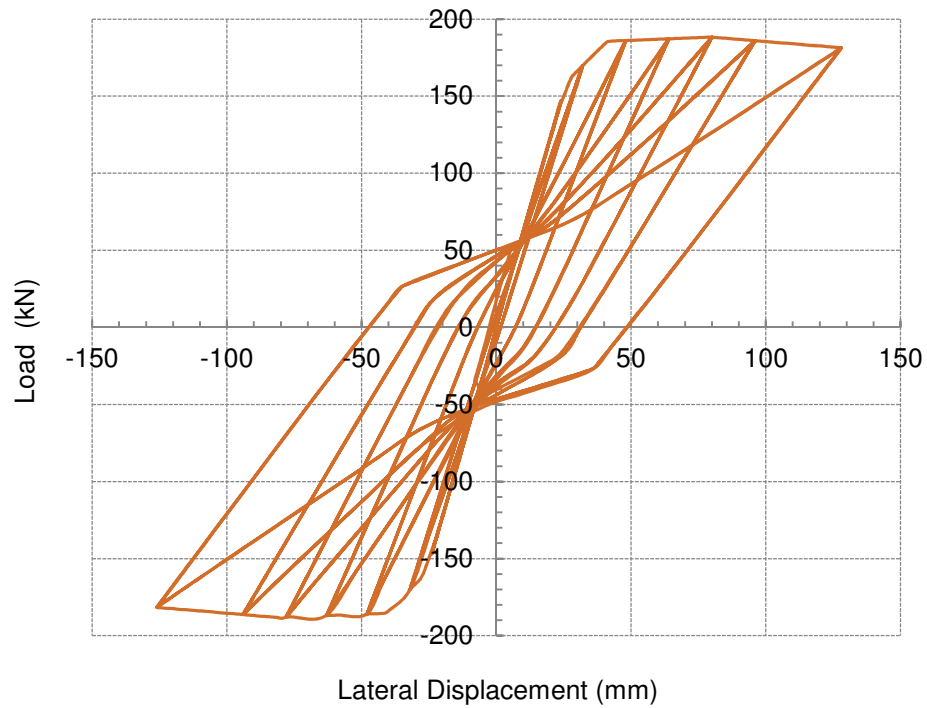


Figure 8.9: Analytical hysteretic lateral load-lateral displacement relationship for Frame BCF (3 cycles at each drift level)

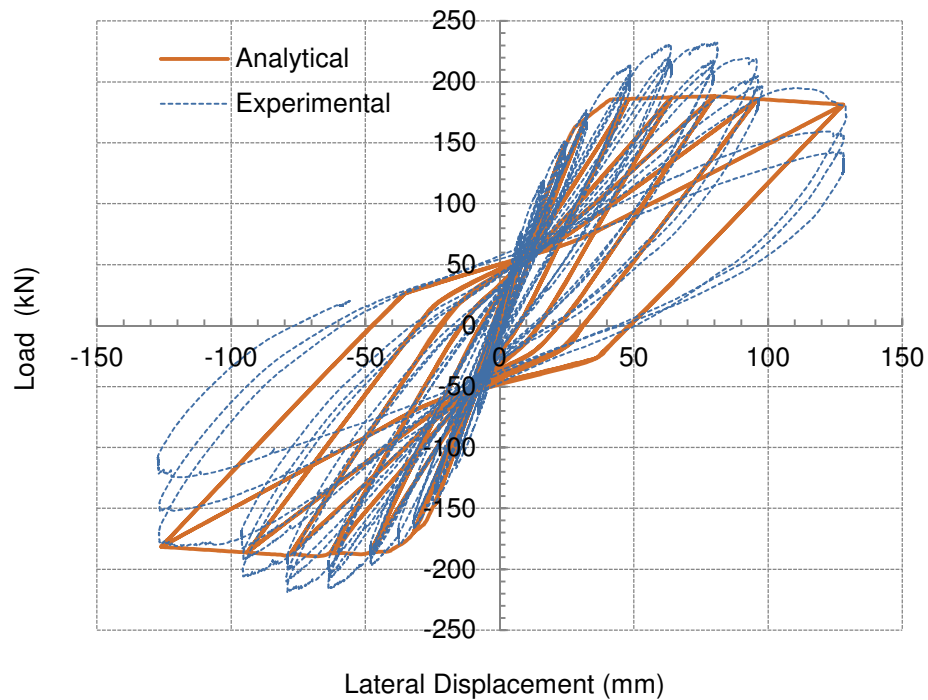


Figure 8.10: Comparison of analytical and experimental hysteretic lateral load-lateral displacement relationships for Frame BCF (with 3 cycles at each drift level)

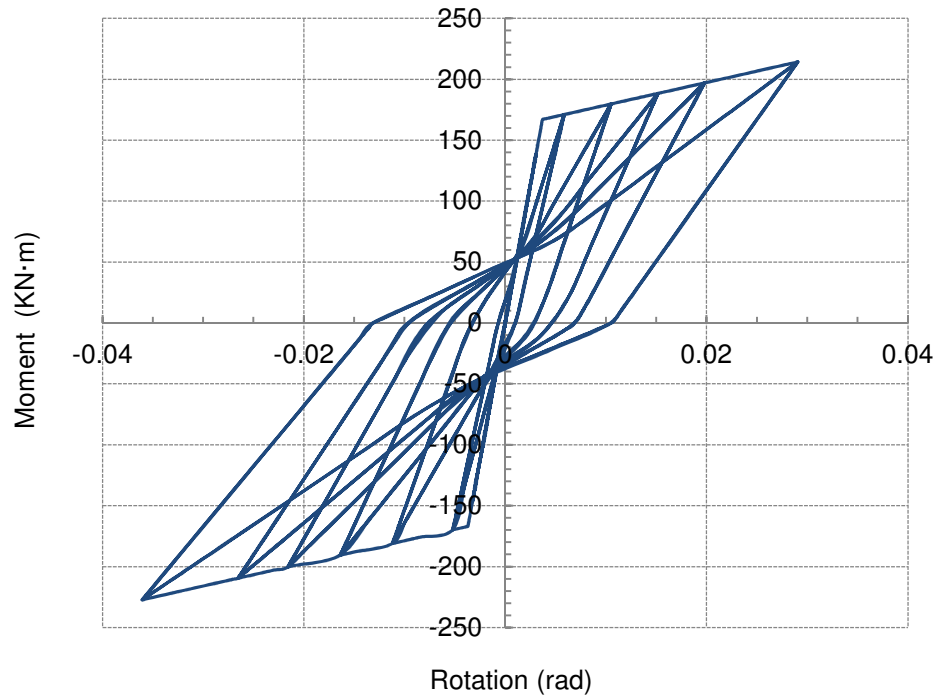


Figure 8.11: Moment-chord rotation response of column hinging region near the base for Frame BCF

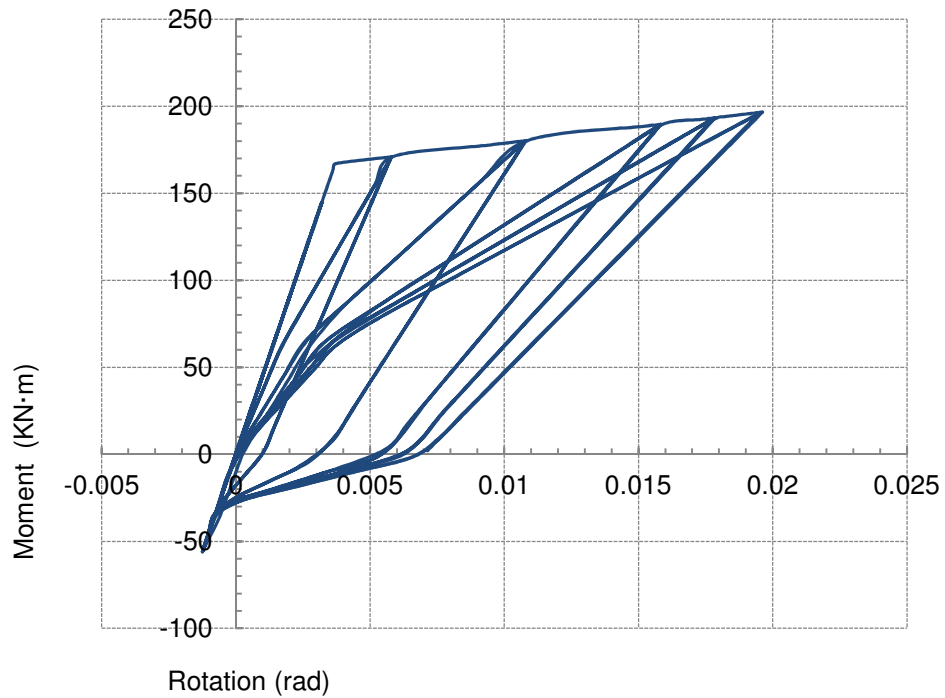


Figure 8.12: Moment-chord rotation response of a first-storey column hinging region near the beam-column joint for Frame BCF

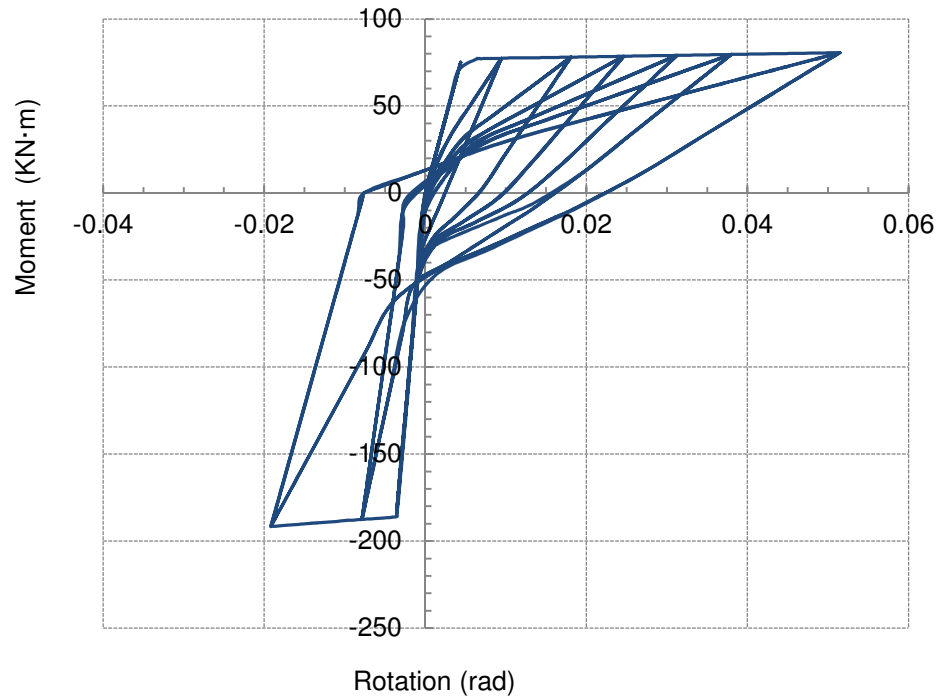


Figure 8.13: Moment-chord rotation response of a beam hinging region near the column face for Frame BCF

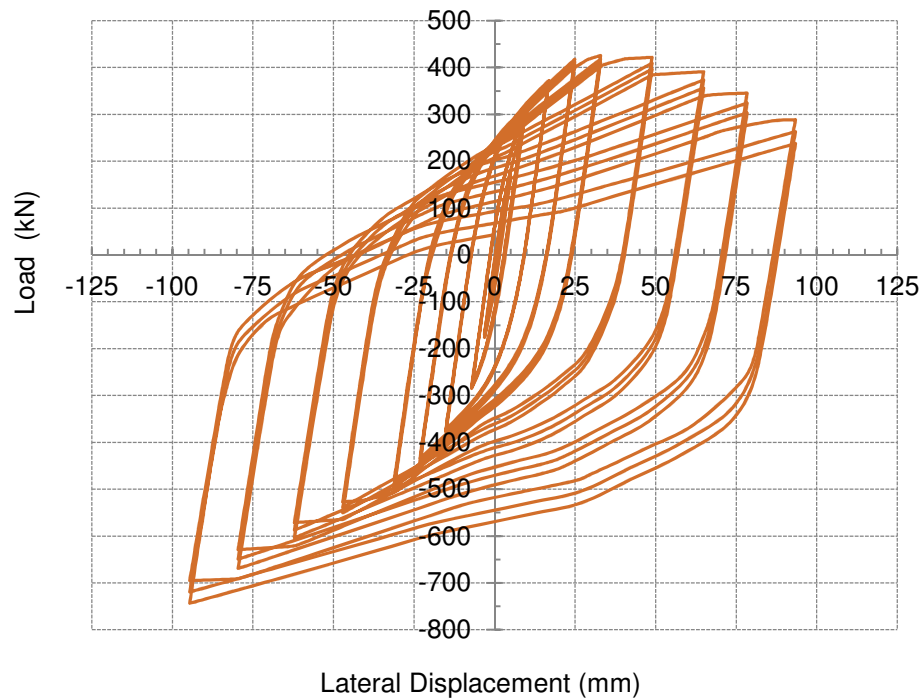


Figure 8.14: Analytical hysteretic lateral load-lateral displacement relationship of repaired retrofitted frame RRF (3 cycles at each drift level)



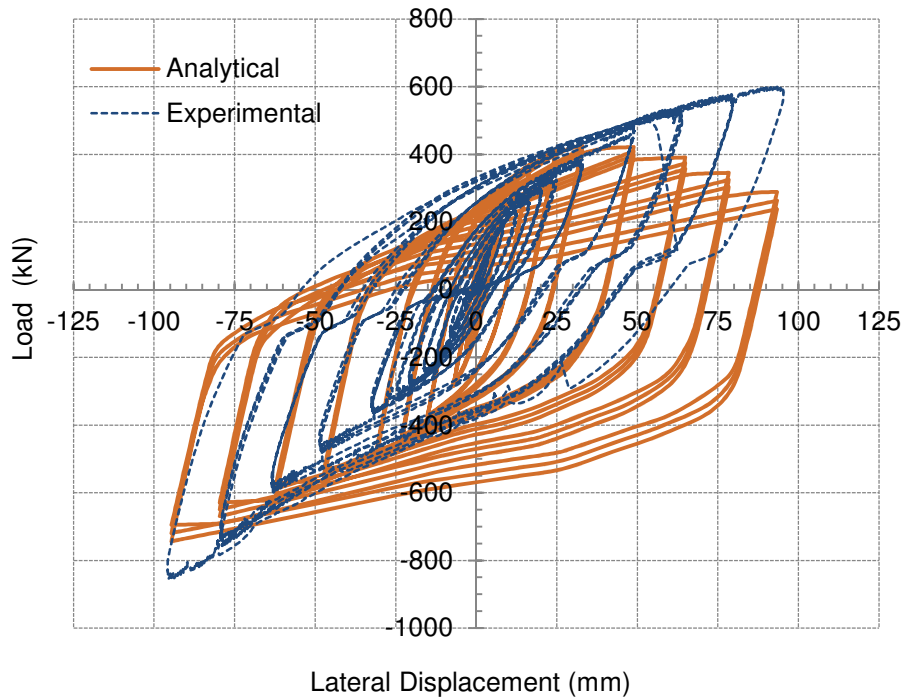


Figure 8.15: Comparison of analytical and experimental hysteretic lateral load-lateral displacement relationships for repaired retrofitted frame RRF (3 cycles at each drift level)

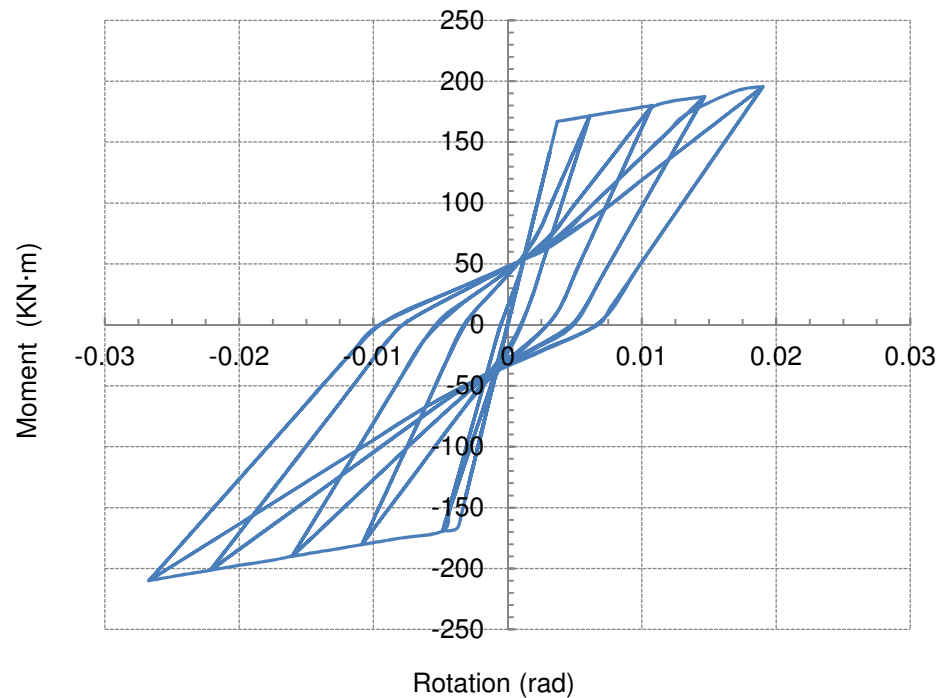


Figure 8.16: Moment-chord rotation response of a column hinging region near the base for Frame RRF

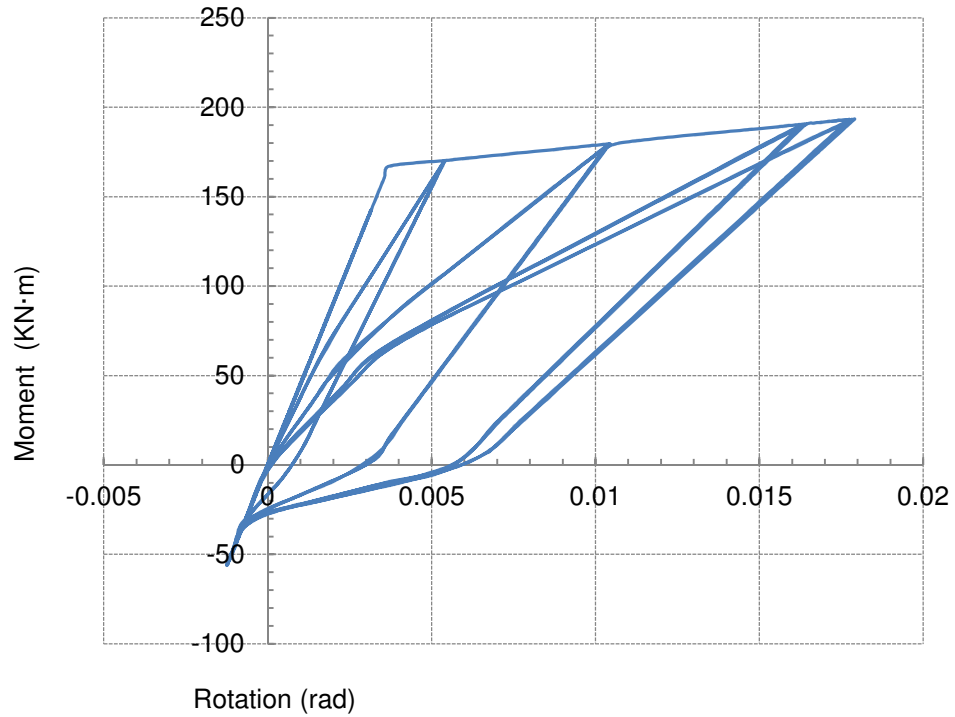


Figure 8.17: Moment-chord rotation response of a column hinging region near the beam-column joint for Frame RRF

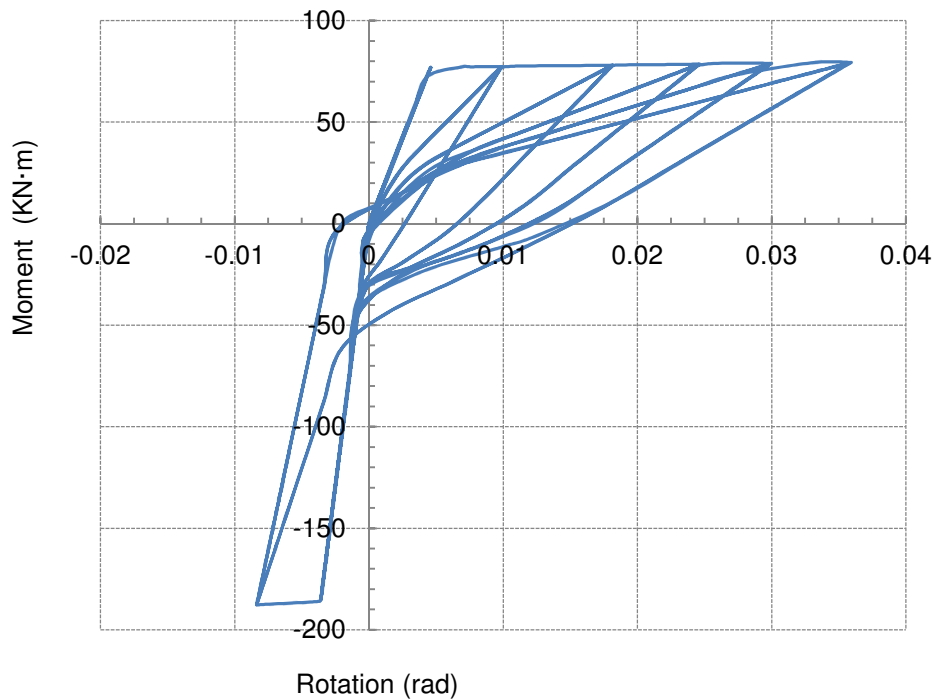


Figure 8.18: Moment-chord rotation response of a beam hinging region near the column face for Frame RR

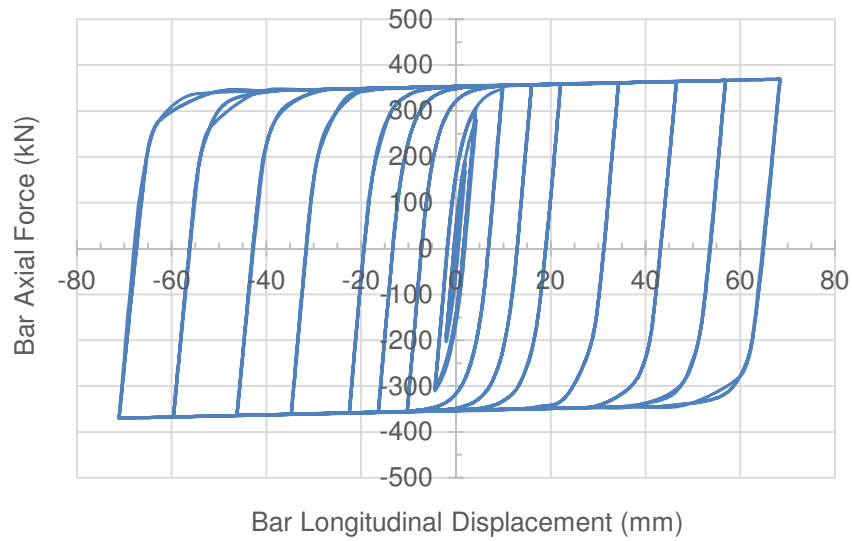


Figure 8.19: Analytical axial force-axial deformation response of the BRB steel core bar within the reduced section segment

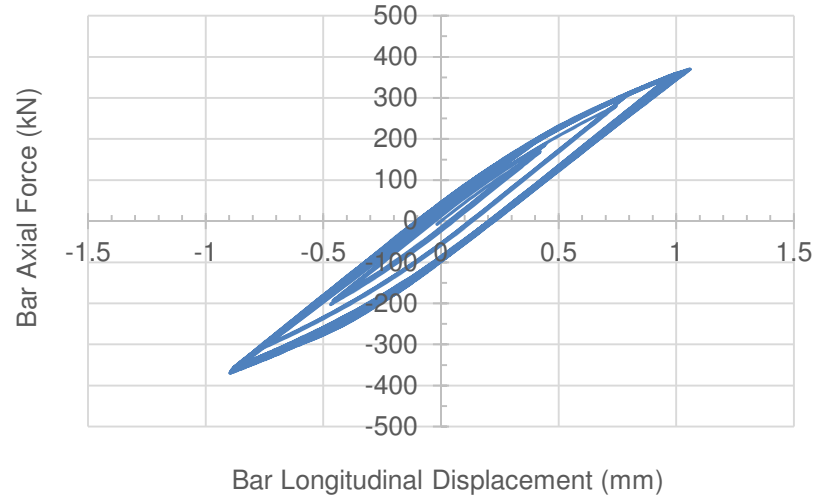


Figure 8.20: Analytical axial force-axial deformation response of the BRB steel core bar at the upper and lower un-reduced section segments

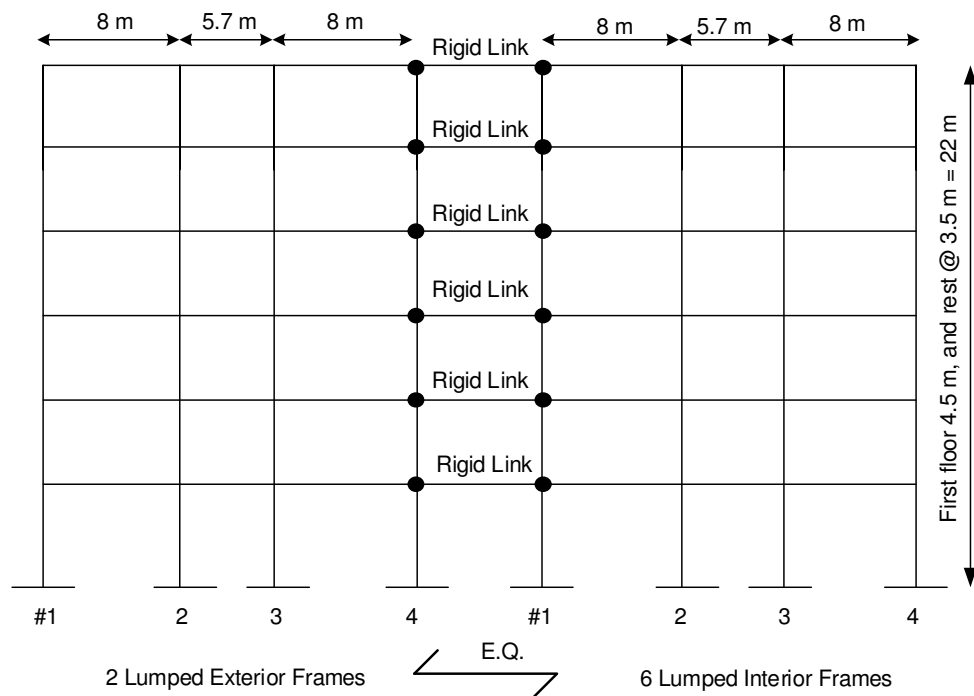


Figure 8.21: Two-dimensional SAP2000 model for un-retrofitted buildings in Ottawa and Vancouver with each lumped frame representing multiple building frames

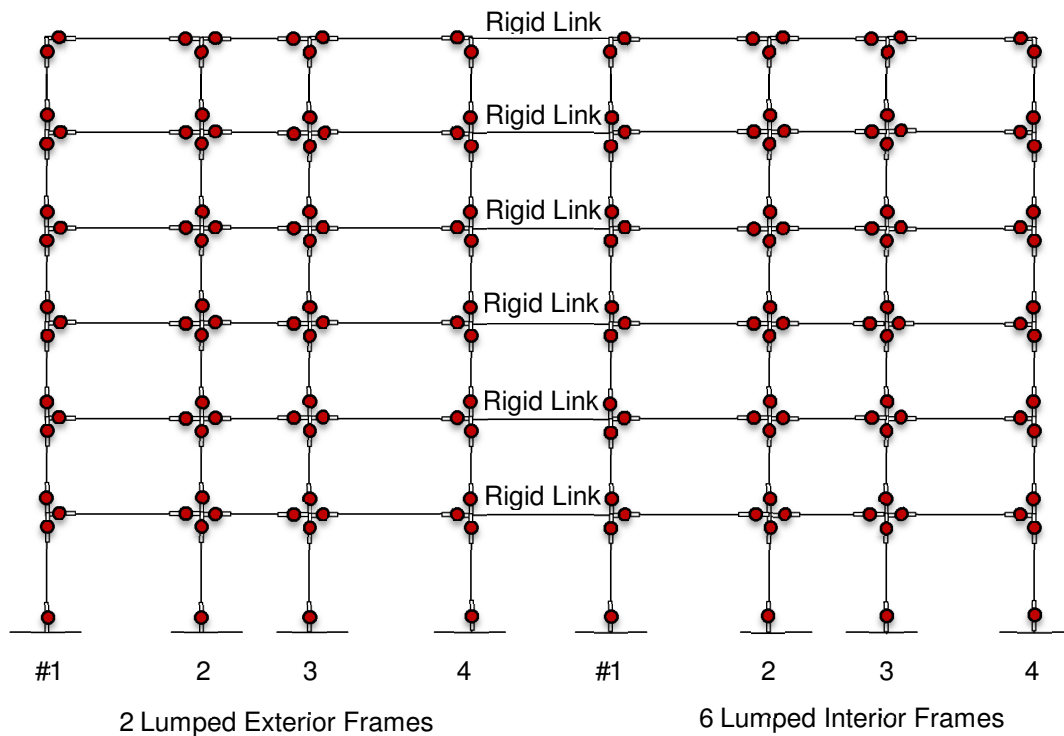


Figure 8.22: Lumped plastic hinges locations of analyzed un-retrofitted two-dimensional lumped frames for buildings located in the Cities of Ottawa and Vancouver



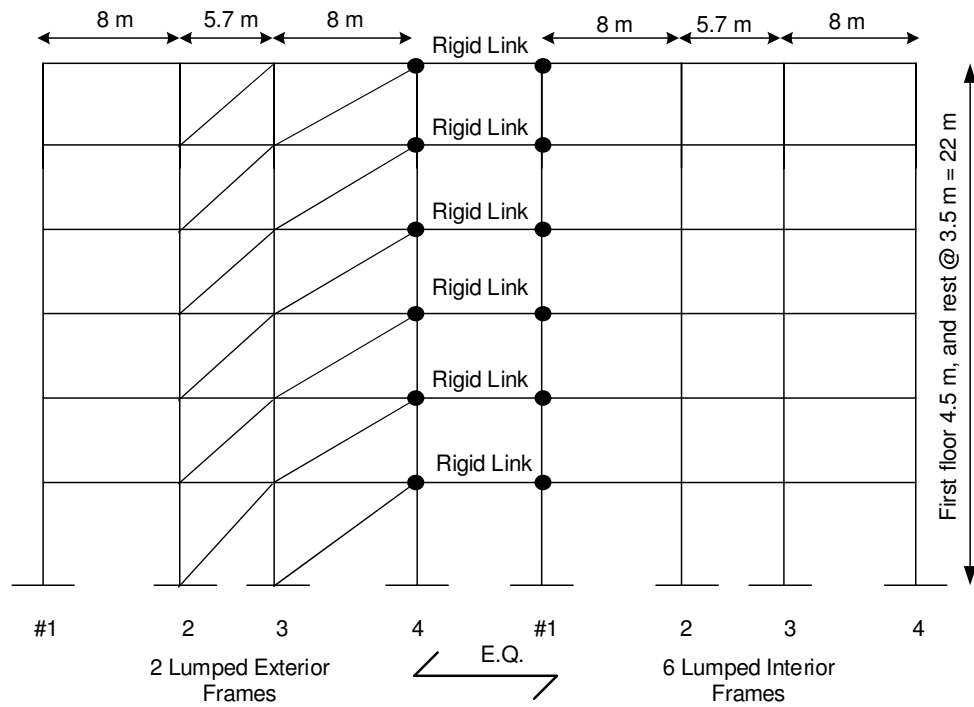


Figure 8.25: Retrofit Configuration III for building in Vancouver

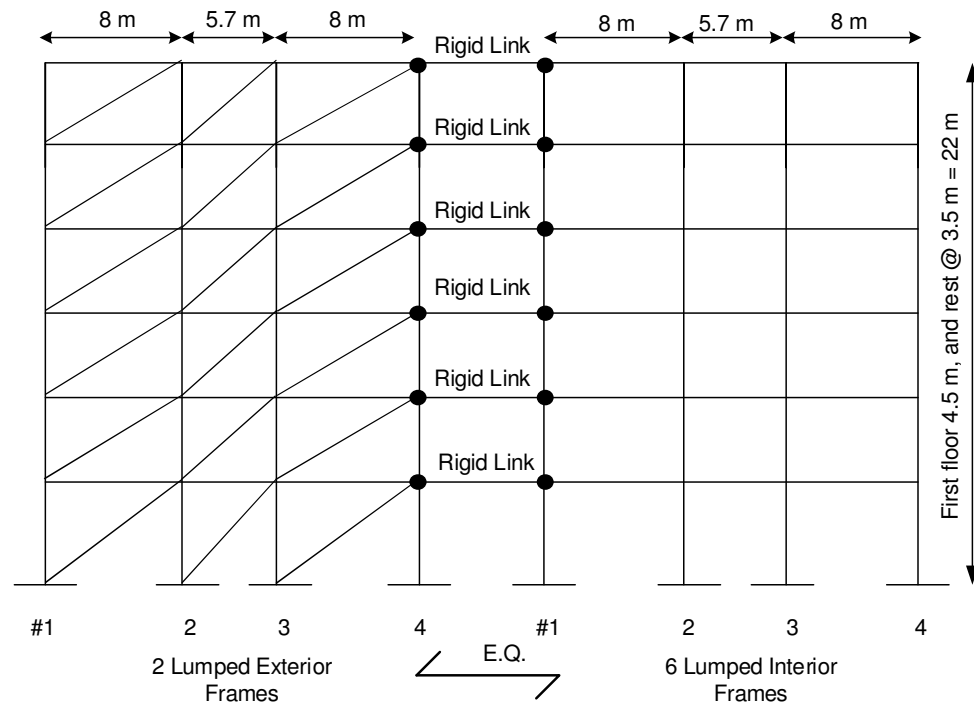
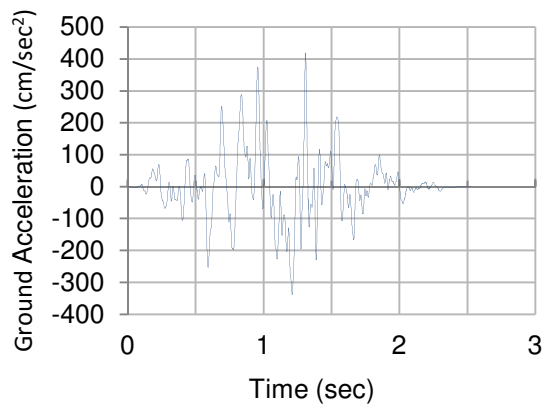
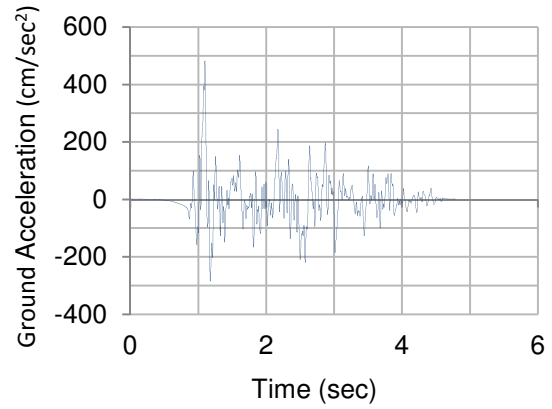


Figure 8.26: Retrofit Configuration IV for building in Vancouver

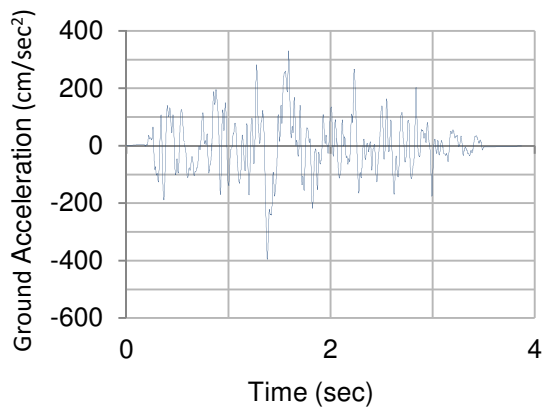




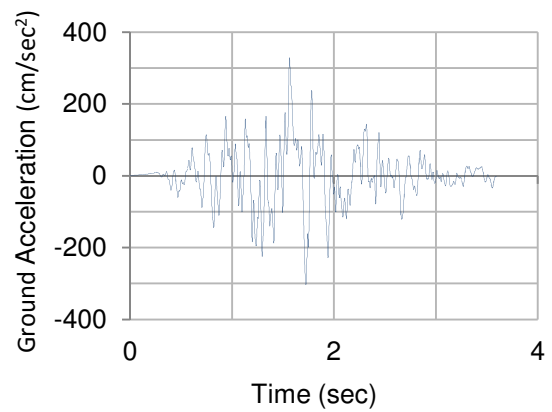
a) Short Event No.1



b) Short Event No.2

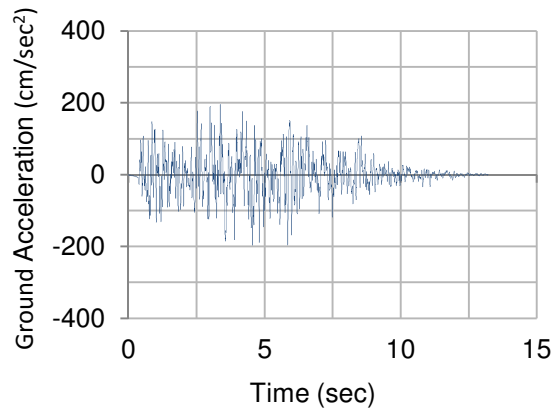


c) Short Event No.3

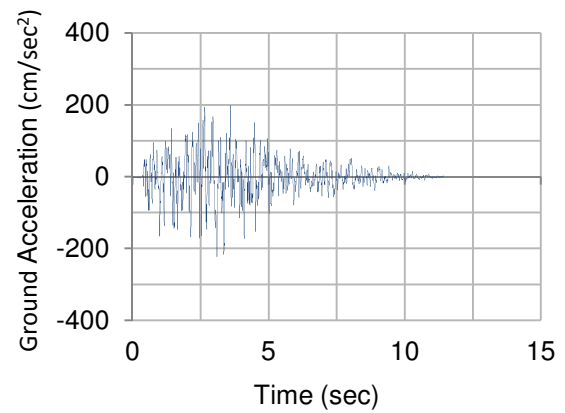


d) Short Event No.4

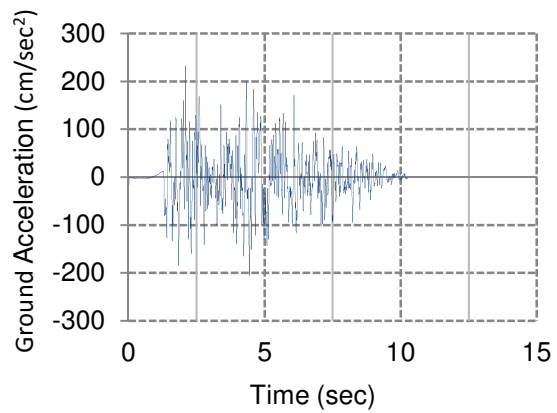
Figure 8.27: Earthquake records for the short events used in dynamic analyses of buildings in Ottawa



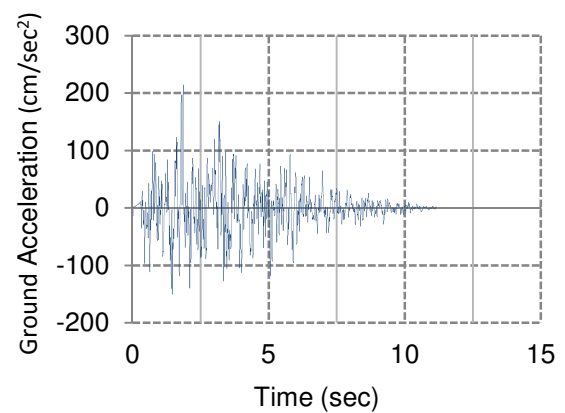
a) Long Event No.1



b) Long Event No.2

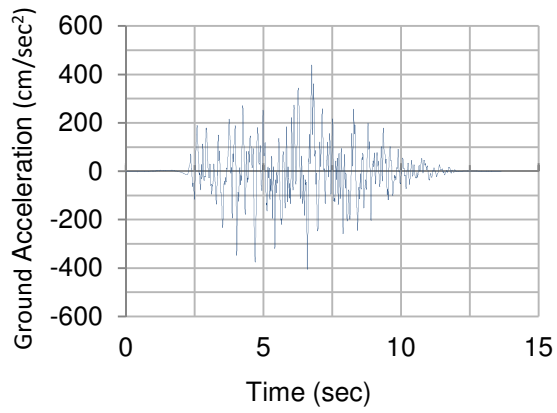


c) Long Event No.3

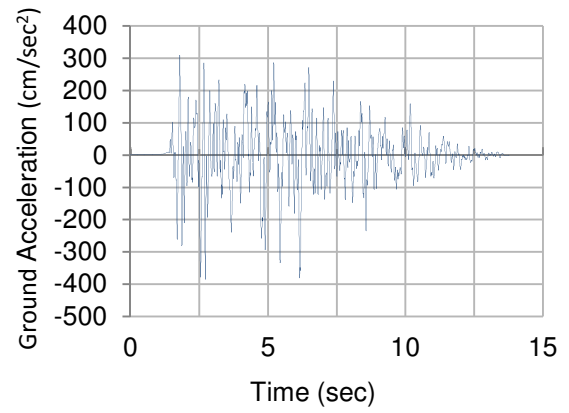


d) Long Event No.4

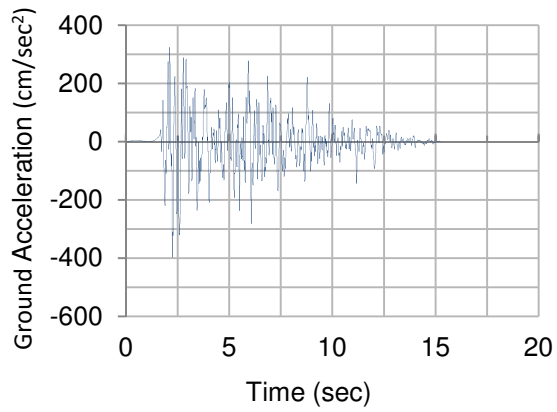
Figure 8.28: Earthquake records of the long events used in dynamic analyses of buildings in Ottawa



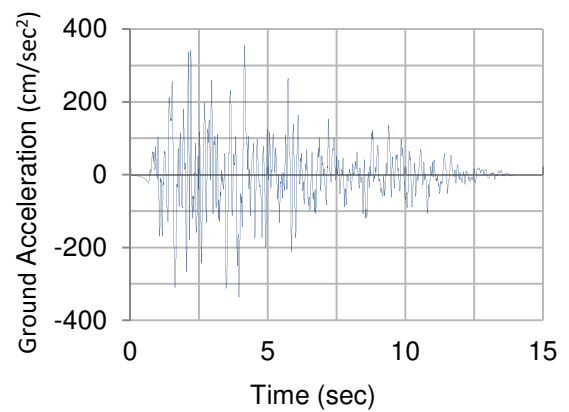
a) Short Event No.1



b) Short Event No.2

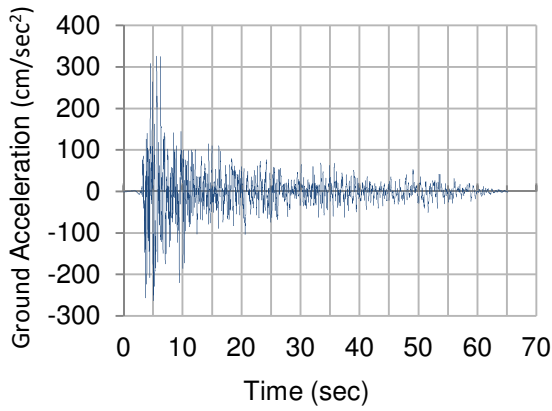


c) Short Event No.3

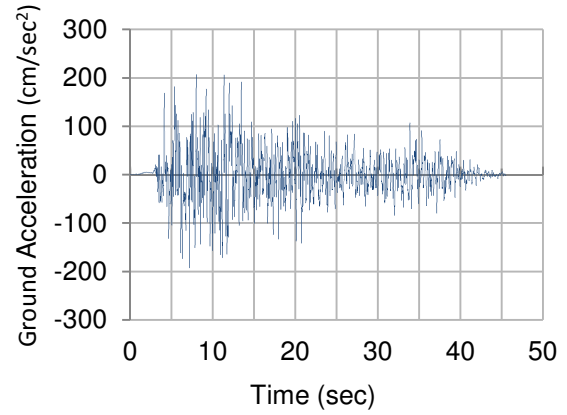


d) Short Event No.4

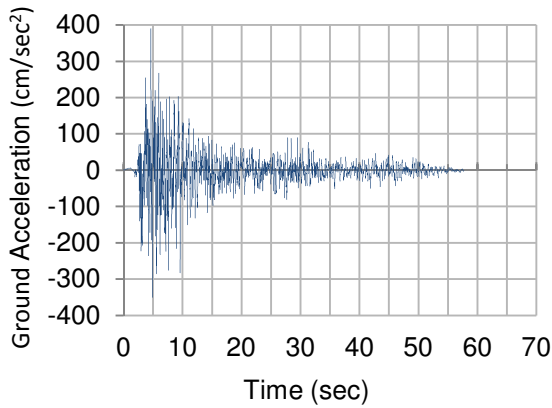
Figure 8.29: Earthquake records for the short events used in dynamic analyses of buildings in Vancouver



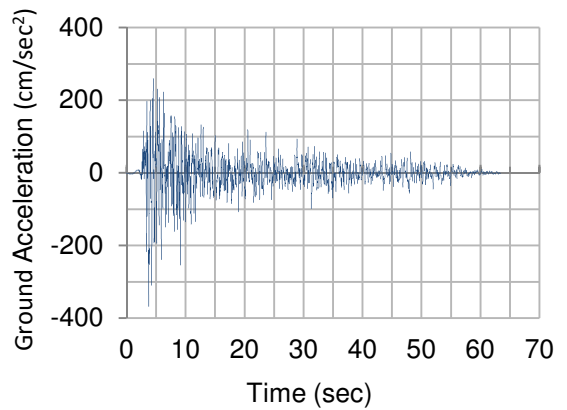
a) Long Event No.1



b) Long Event No.2

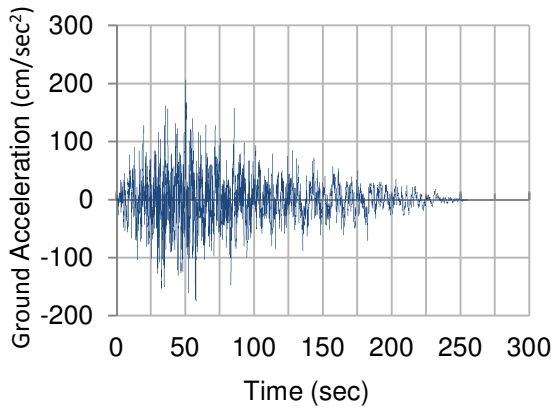


c) Long Event No.3

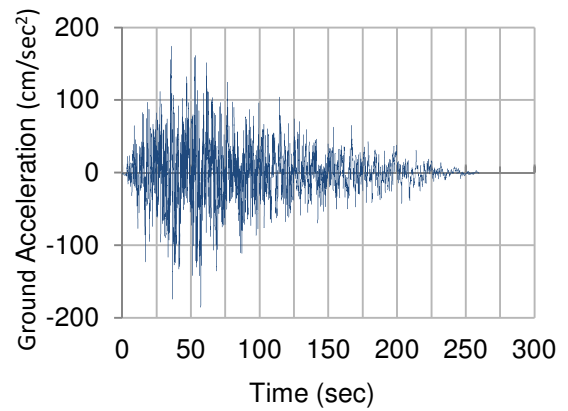


d) Long Event No.4

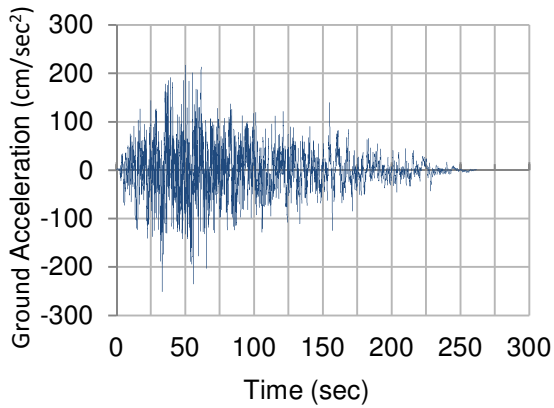
Figure 8.30: Earthquake records for the long events used in dynamic analyses of buildings in Vancouver



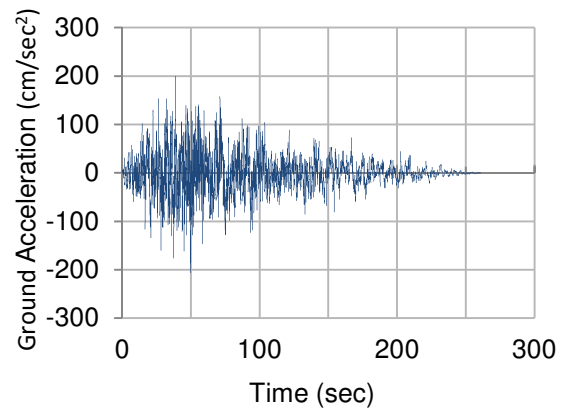
a) Cascadia Event No.1



b) Cascadia Event No.2



c) Cascadia Event No.3



d) Cascadia Event No.4

Figure 8.31: Earthquake records for the Cascadia events used in dynamic analyses of buildings in Vancouver

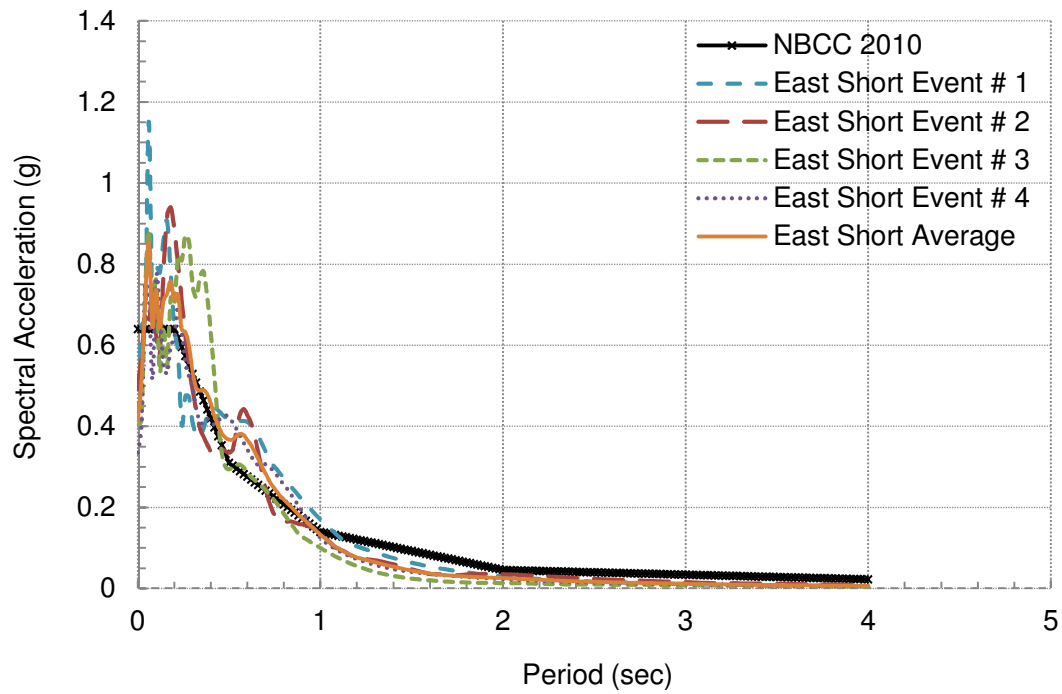


Figure 8.32: Response spectra for short events used in analyses of buildings in Ottawa

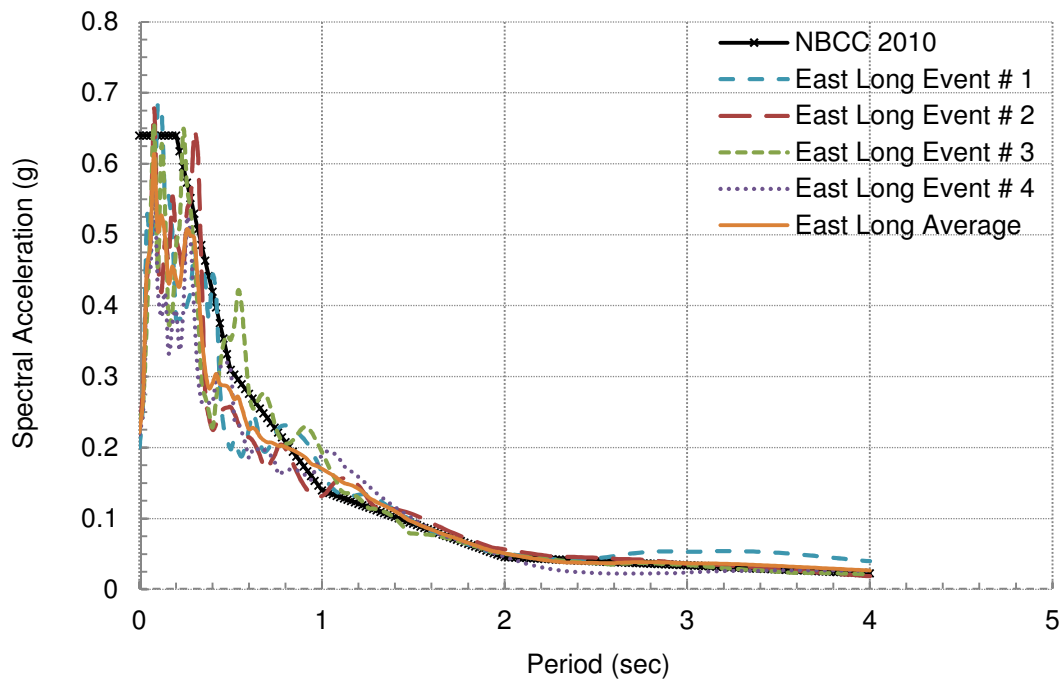


Figure 8.33: Response spectra for long events used in analyses of buildings in Ottawa



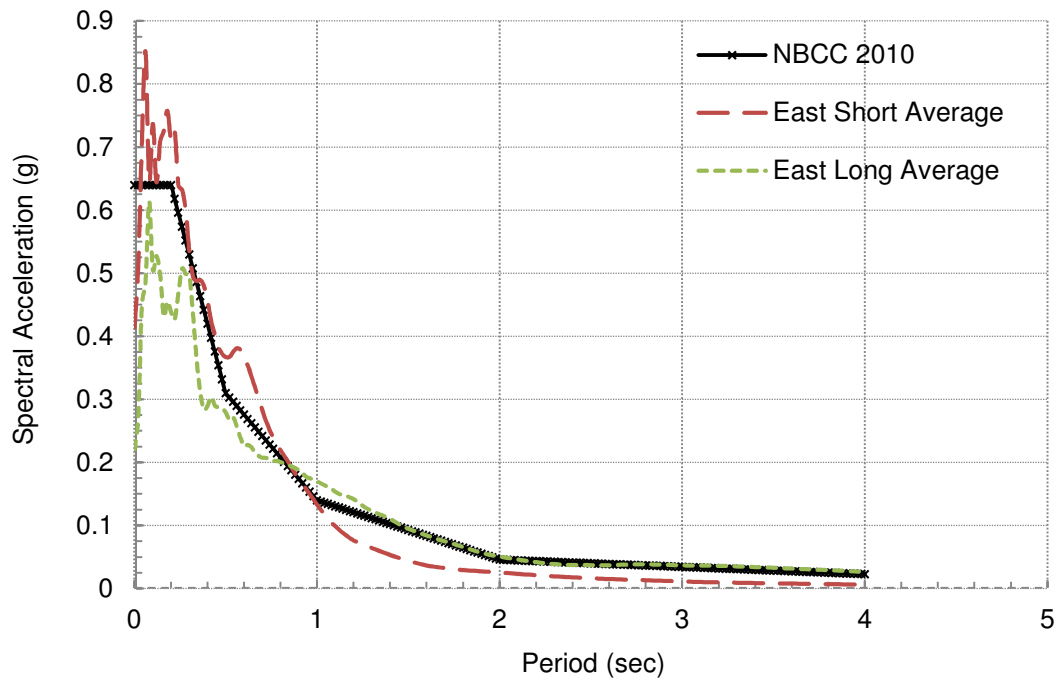


Figure 8.34: Average of response spectra for short and long events used in analyses of buildings in Ottawa

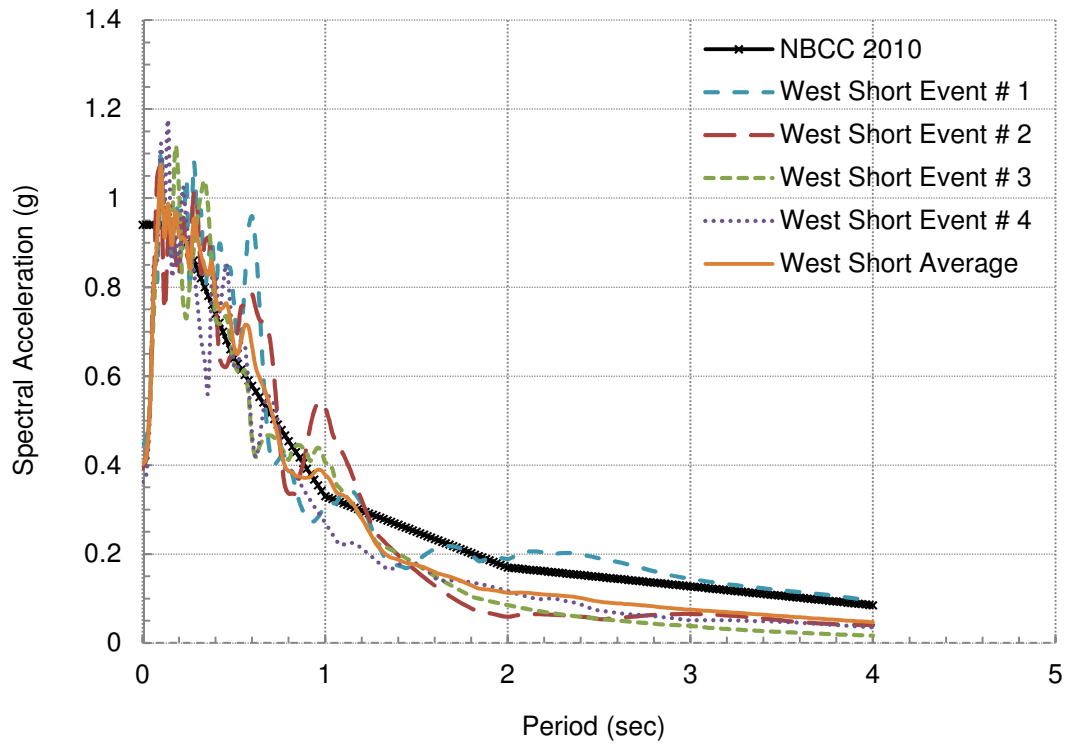


Figure 8.35: Response spectra for short events used in analyses of buildings in Vancouver

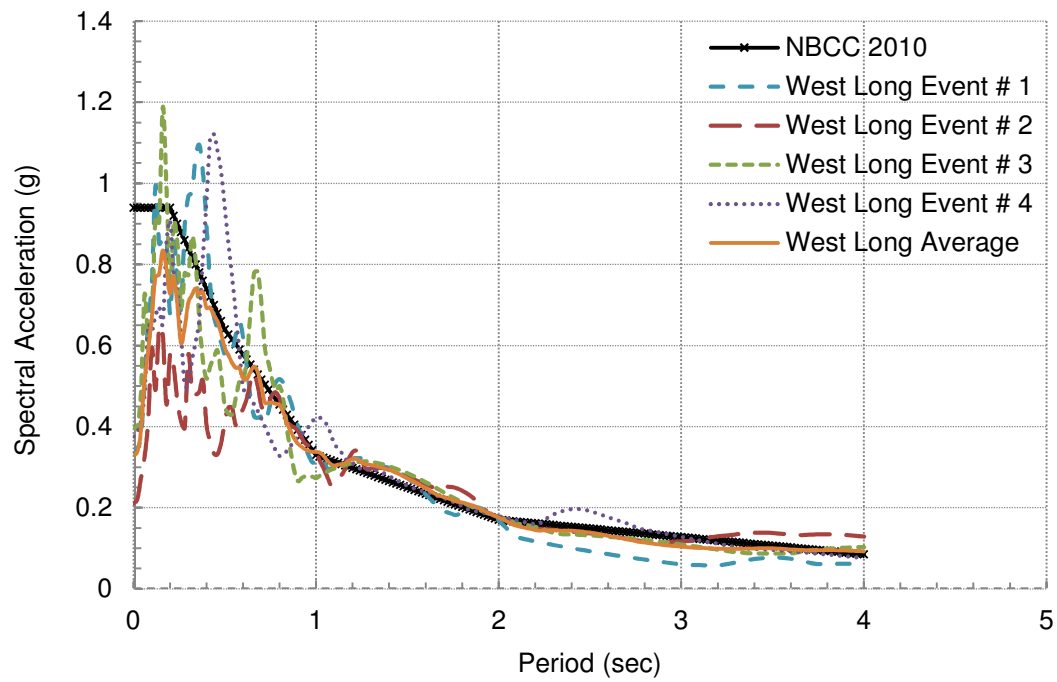


Figure 8.36: Response spectra for long events used in analyses of buildings in Vancouver

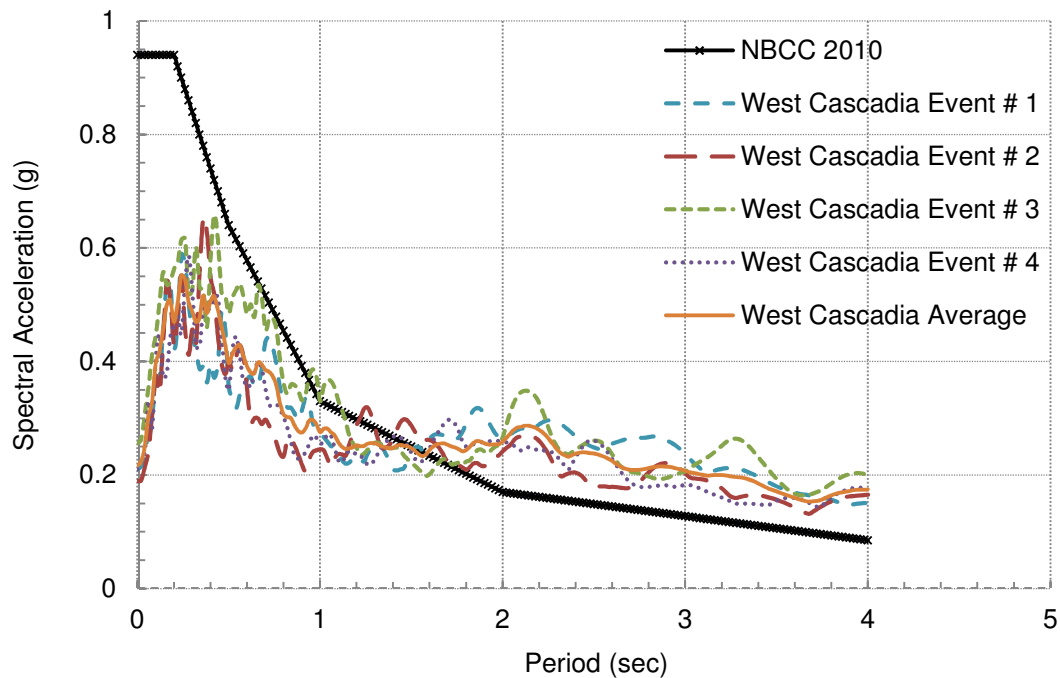


Figure 8.37: Response spectra for Cascadia events used in analyses of buildings in Vancouver

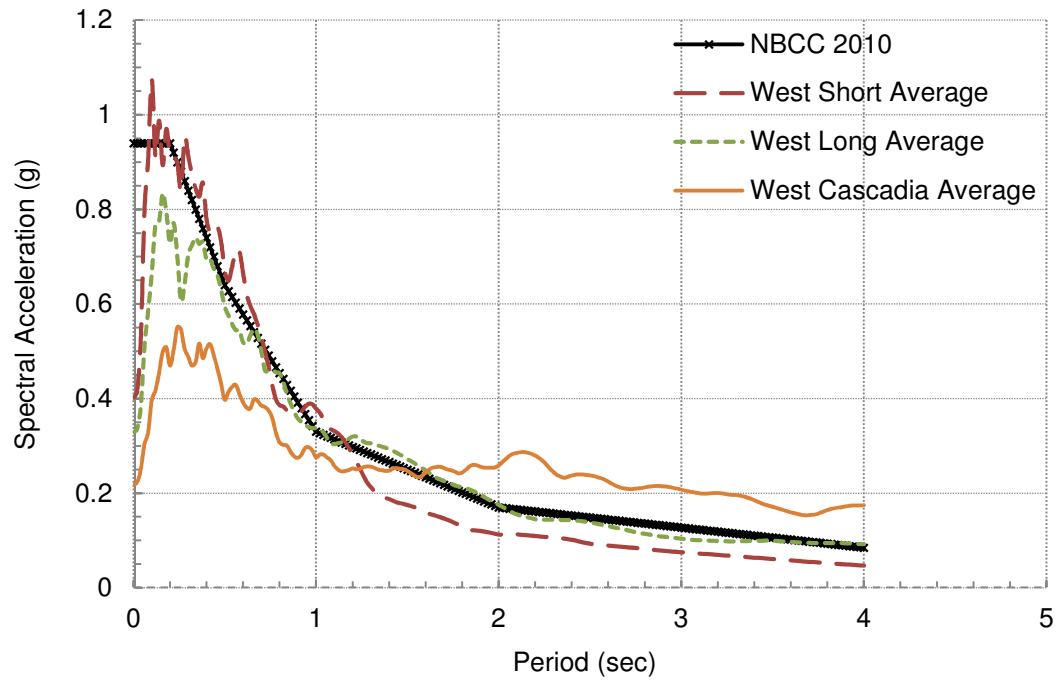


Figure 8.38: Average of response spectra for short, long, and Cascadia events used in analyses of buildings in Vancouver

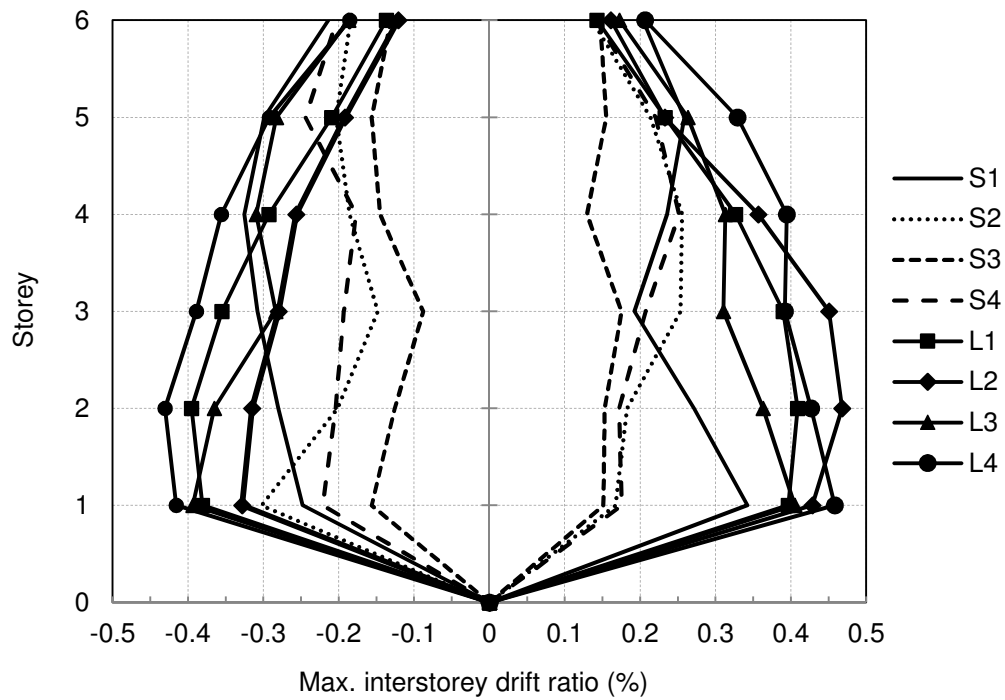


Figure 8.39: Maximum interstorey drift ratios for the un-retrofitted building in Ottawa, analyzed under earthquake records of short and long events

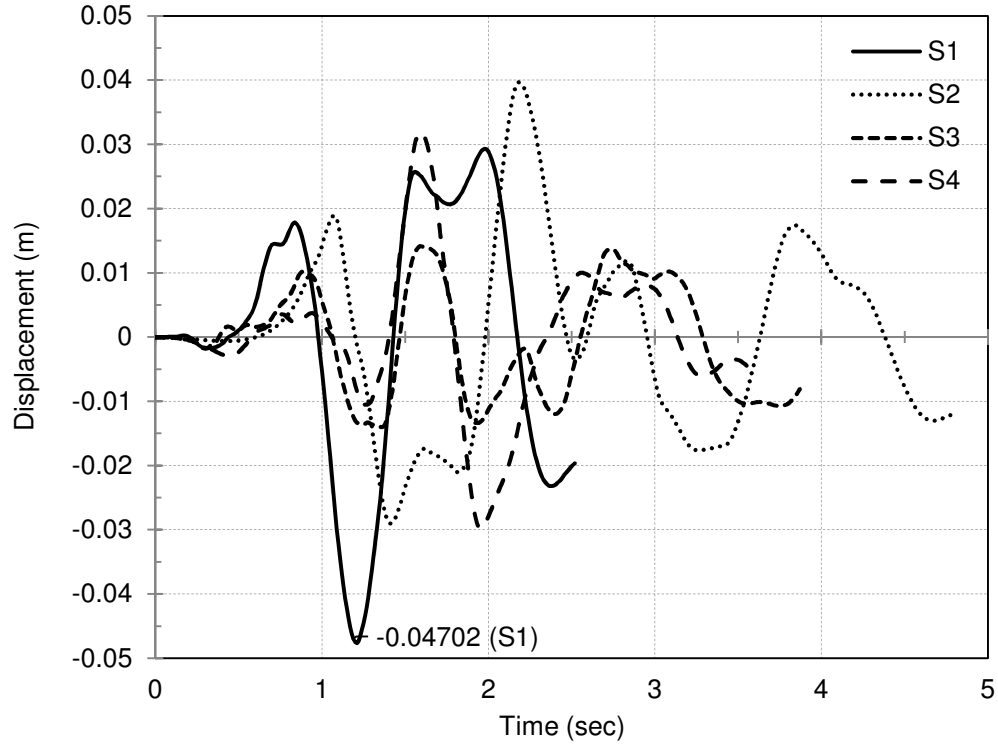


Figure 8.40: Top displacement time histories for the un-retrofitted building in Ottawa, analyzed under earthquake records of short events

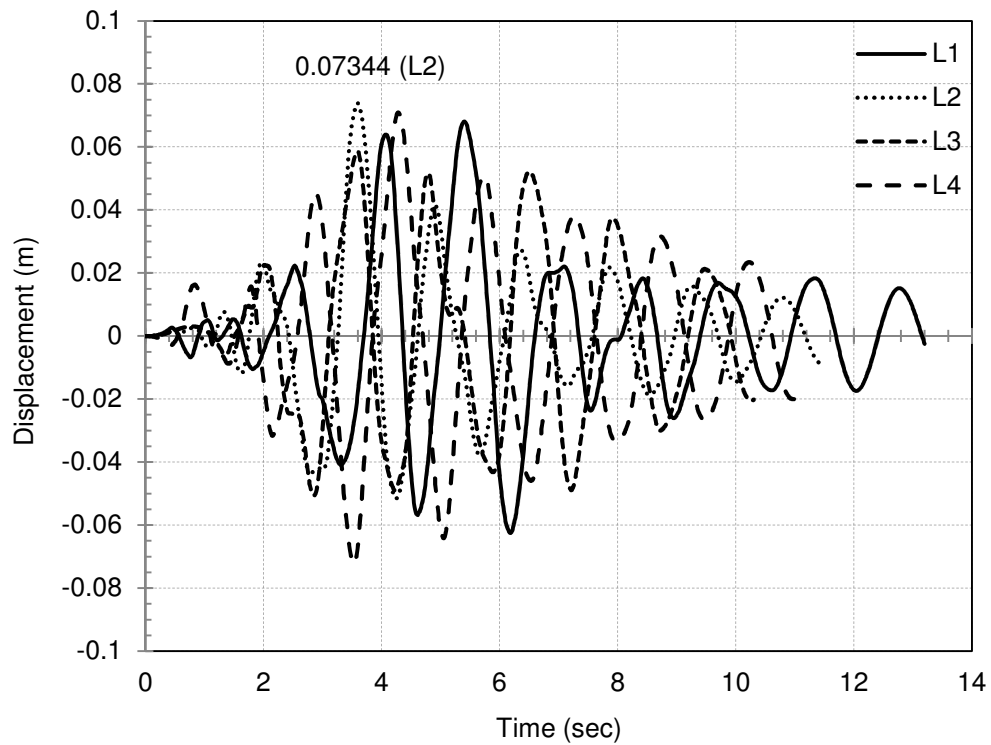


Figure 8.41: Top displacement time histories for the un-retrofitted building in Ottawa, analyzed under earthquake records of long events

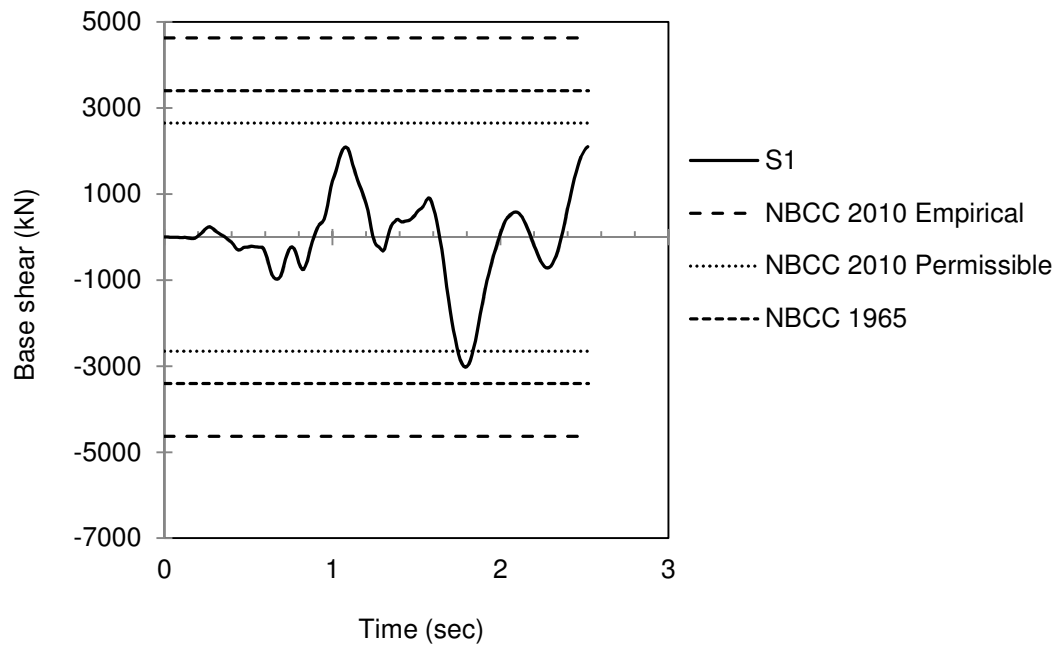


Figure 8.42: Comparison of dynamic base shear demands for Short Event #1 and static base shears calculated based on NBCC for the un-retrofitted building in Ottawa

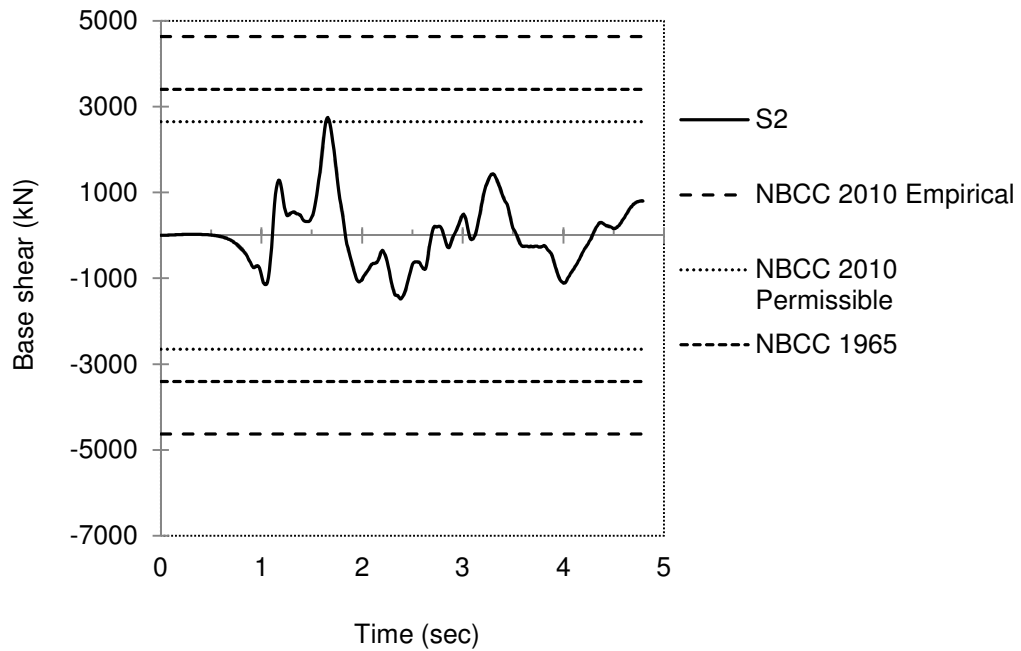


Figure 8.43: Comparison of dynamic base shear demands for Short Event #2 and static base shears calculated based on NBCC for the un-retrofitted building in Ottawa

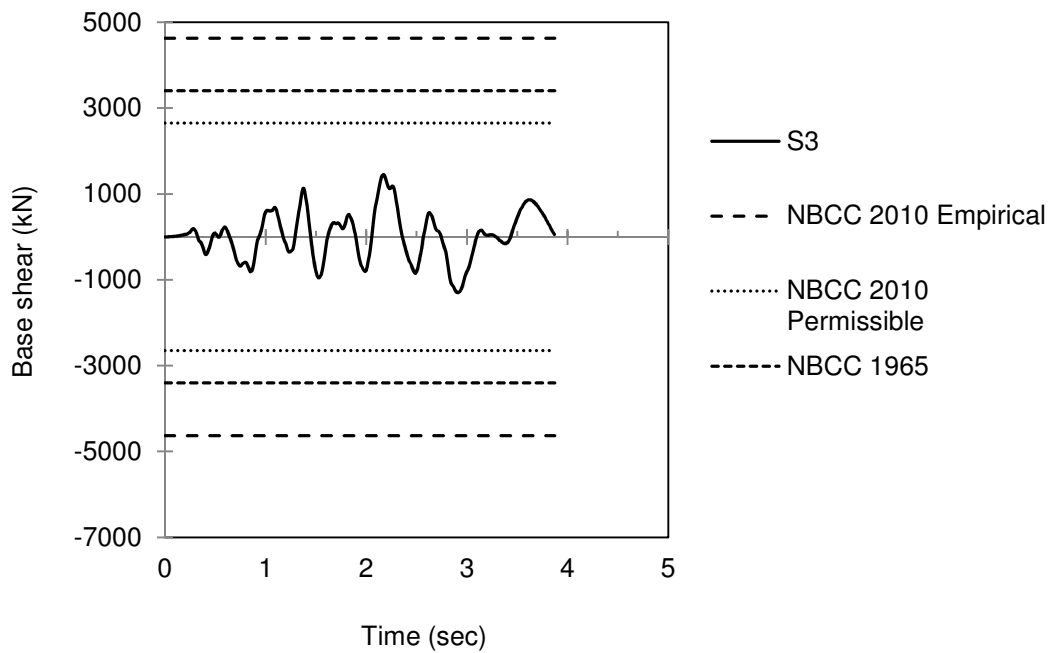


Figure 8.44: Comparison of dynamic base shear demands for Short Event #3 and static base shears calculated based on NBCC for the un-retrofitted building in Ottawa



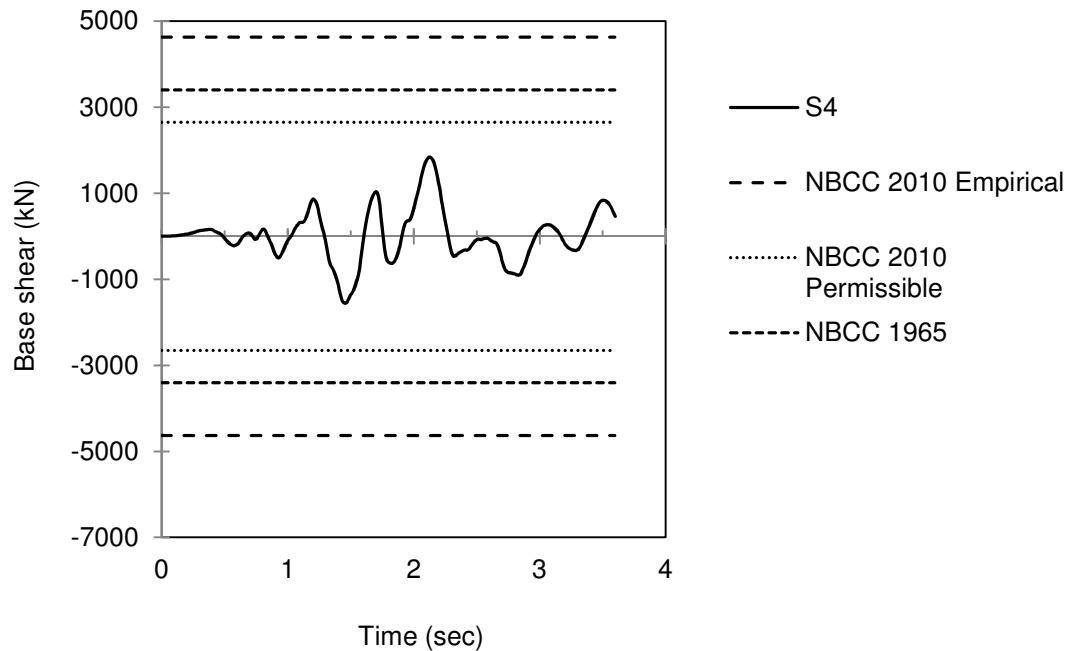


Figure 8.45: Comparison of dynamic base shear demands for Short Event #4 and static base shears calculated based on NBCC for the un-retrofitted building in Ottawa

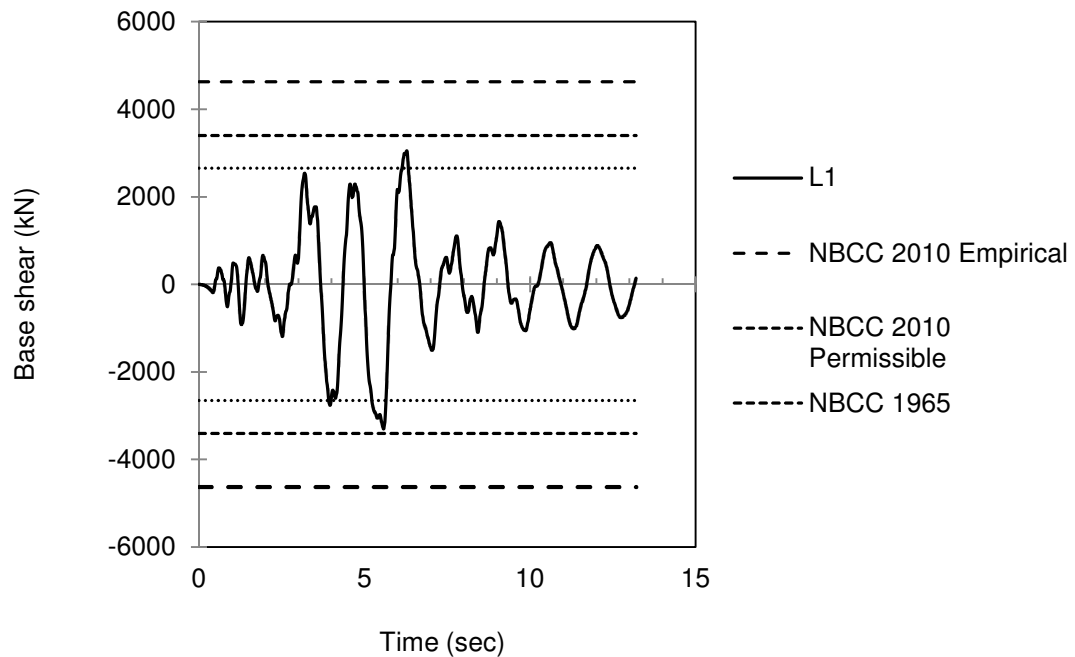


Figure 8.46: Comparison of dynamic base shear demands for Long Event #1 and static base shears calculated based on NBCC for the un-retrofitted building in Ottawa

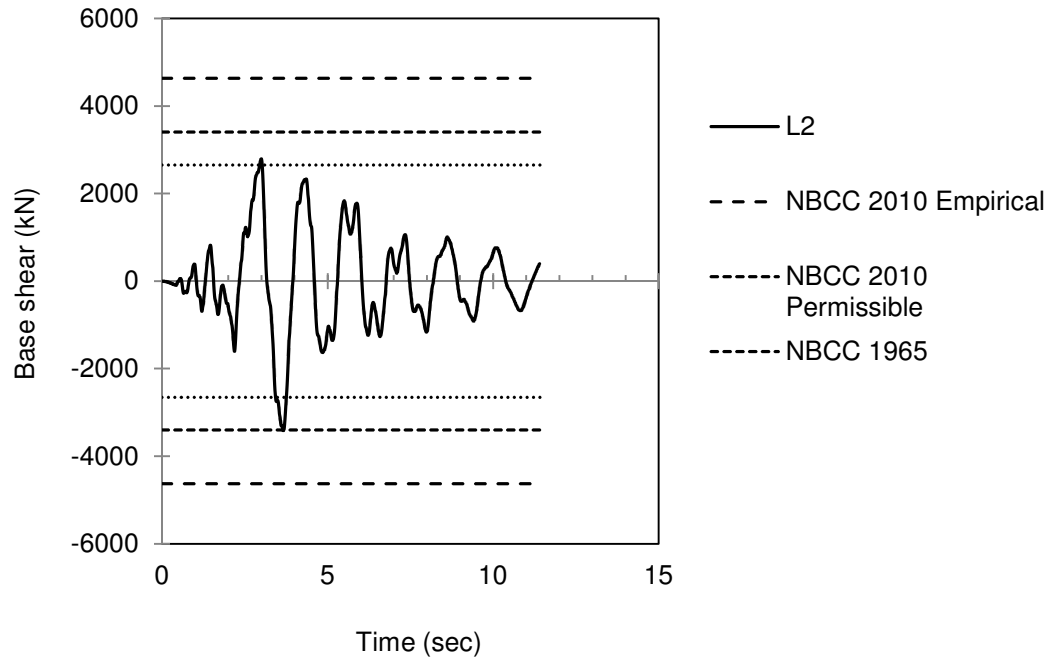


Figure 8.47: Comparison of dynamic base shear demands for Long Event #2 and static base shears calculated based on NBCC for the un-retrofitted building in Ottawa

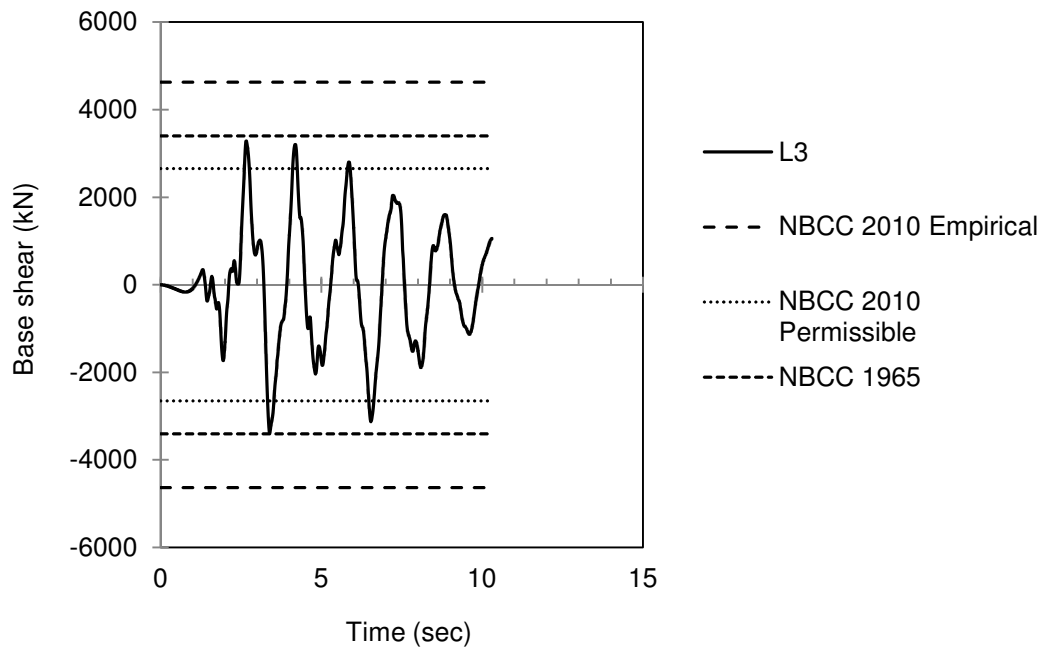


Figure 8.48: Comparison of dynamic base shear demands for Long Event #3 and static base shears calculated based on NBCC requirements for the un-retrofitted building in Ottawa

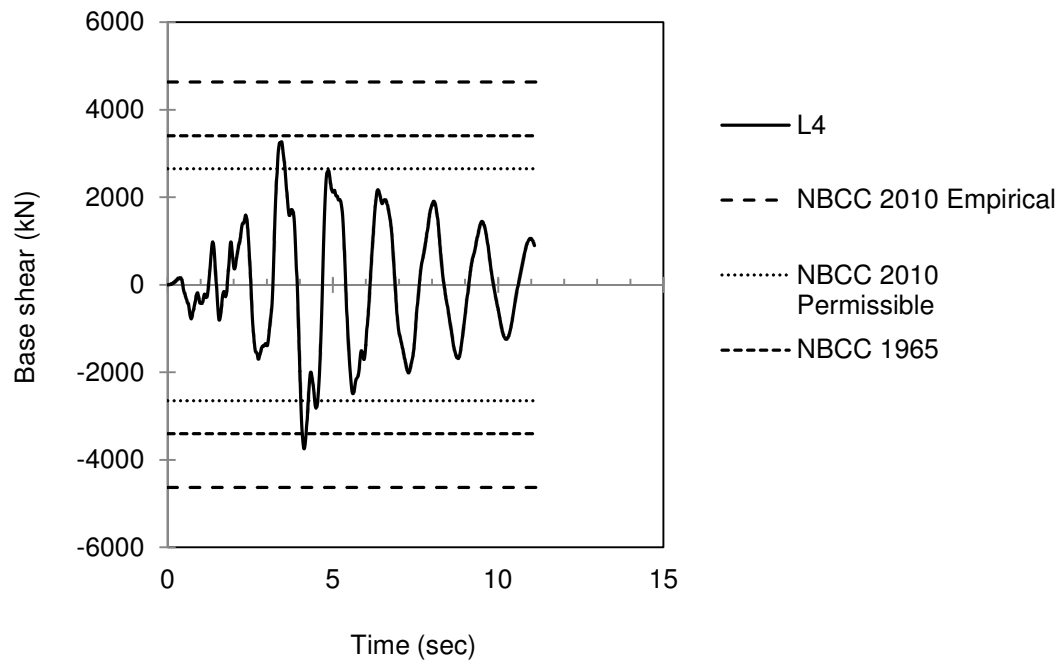
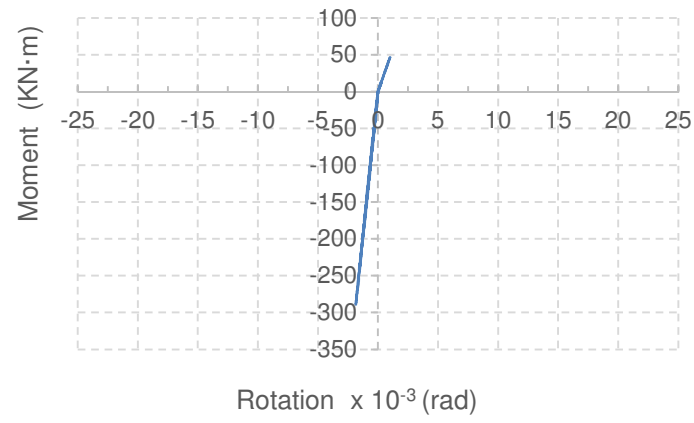


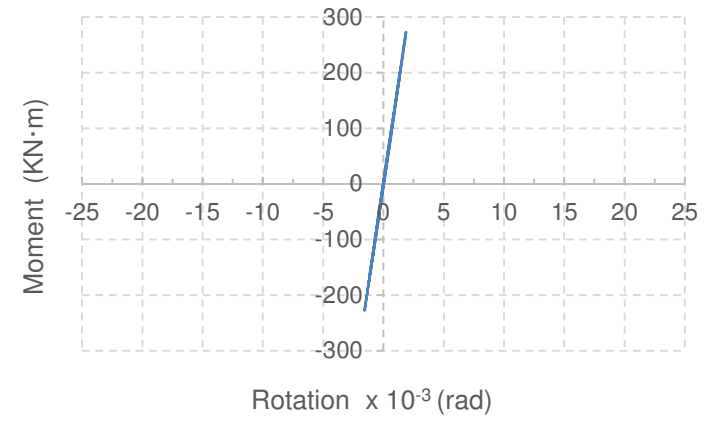
Figure 8.49: Comparison of dynamic base shear demands for Long Event #4 and static base shears calculated based on NBCC requirements for the un-retrofitted building in Ottawa

|                              |                                |                                |                              |                                |                                |                                |                                |
|------------------------------|--------------------------------|--------------------------------|------------------------------|--------------------------------|--------------------------------|--------------------------------|--------------------------------|
| V = 75<br>P = 99<br>M = 56   | V = 77<br>P = 164<br>M = 63    | V = 72<br>P = 165<br>M = 60    | V = 77<br>P = 100<br>M = 57  | V = 128<br>P = 186<br>M = 99   | V = 132<br>P = 331<br>M = 99   | V = 123<br>P = 334<br>M = 92   | V = 131<br>P = 188<br>M = 101  |
| V = 71<br>P = 211<br>M = 71  | V = 94<br>P = 323<br>M = 96    | V = 86<br>P = 325<br>M = 88    | V = 76<br>P = 215<br>M = 75  | V = 107<br>P = 385<br>M = 113  | V = 144<br>P = 648<br>M = 148  | V = 129<br>P = 655<br>M = 133  | V = 113<br>P = 391<br>M = 117  |
| V = 84<br>P = 331<br>M = 89  | V = 119<br>P = 489<br>M = 129  | V = 108<br>P = 494<br>M = 115  | V = 91<br>P = 338<br>M = 97  | V = 130<br>P = 595<br>M = 139  | V = 170<br>P = 973<br>M = 182  | V = 156<br>P = 984<br>M = 165  | V = 139<br>P = 606<br>M = 151  |
| V = 84<br>P = 455<br>M = 89  | V = 114<br>P = 659<br>M = 129  | V = 113<br>P = 666<br>M = 122  | V = 87<br>P = 465<br>M = 100 | V = 126<br>P = 813<br>M = 135  | V = 160<br>P = 1296<br>M = 188 | V = 157<br>P = 1292<br>M = 172 | V = 129<br>P = 829<br>M = 145  |
| V = 96<br>P = 581<br>M = 110 | V = 130<br>P = 831<br>M = 142  | V = 131<br>P = 836<br>M = 150  | V = 96<br>P = 590<br>M = 107 | V = 154<br>P = 1035<br>M = 183 | V = 195<br>P = 1625<br>M = 217 | V = 193<br>P = 1620<br>M = 224 | V = 152<br>P = 1046<br>M = 178 |
| V = 88<br>P = 722<br>M = 193 | V = 120<br>P = 1027<br>M = 250 | V = 106<br>P = 1027<br>M = 235 | V = 98<br>P = 725<br>M = 215 | V = 107<br>P = 1280<br>M = 223 | V = 177<br>P = 2007<br>M = 381 | V = 156<br>P = 1996<br>M = 345 | V = 118<br>P = 1283<br>M = 246 |
| 1                            | 2                              | 3                              | 4                            | 1                              | 2                              | 3                              | 4                              |

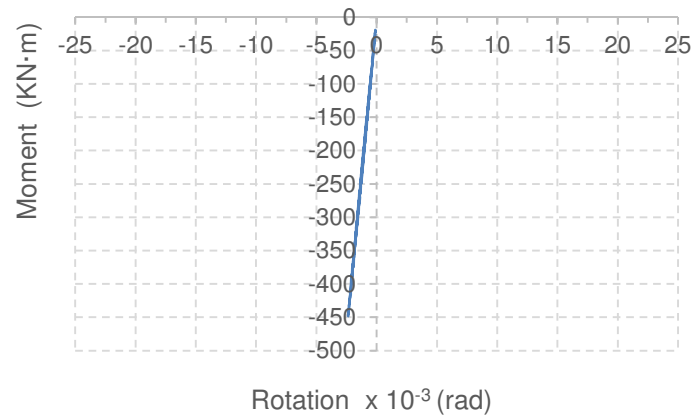
Figure 8.50: Column shear force, axial force, and moment demands for the un-retrofitted building in Ottawa, analyzed under Long Event #4



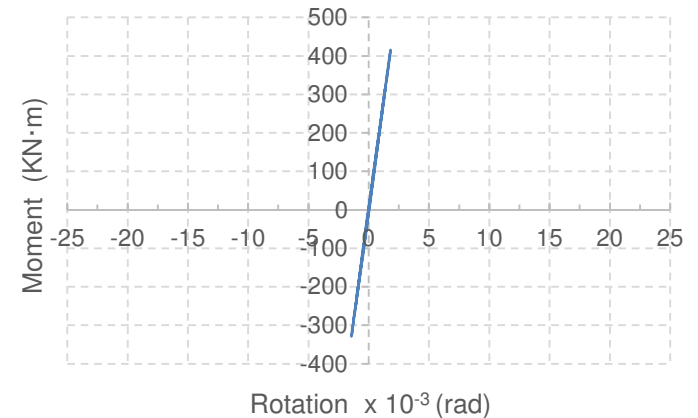
(a)



(b)



(c)



(d)

Figure 8.51 : Moment-rotation hysteretic relationships for selected beams and columns of the un-retrofitted building in Ottawa analyzed under Long Event #4: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames

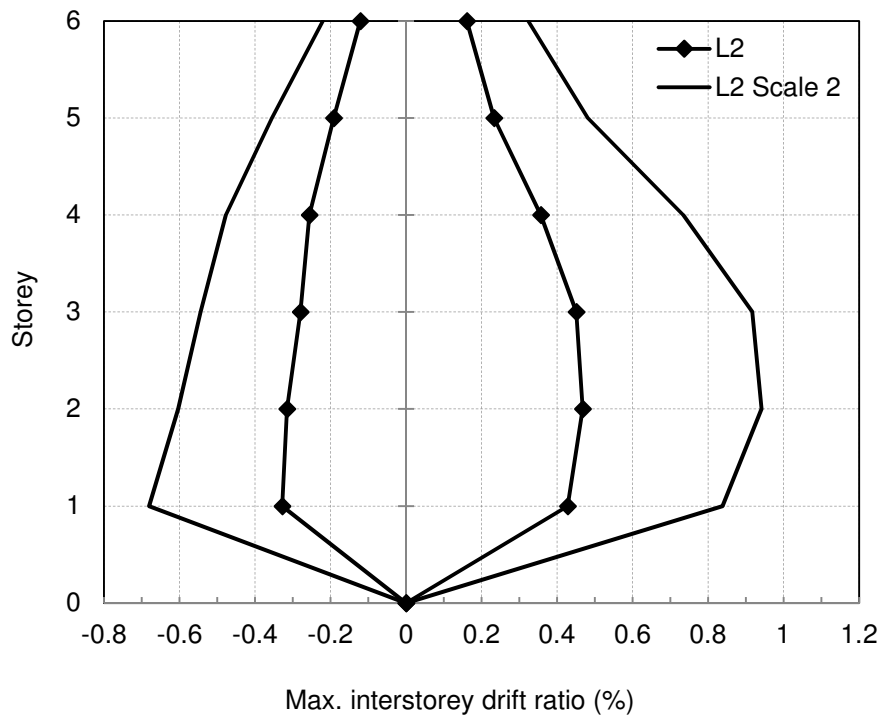


Figure 8.52: Maximum interstorey drift ratios for the un-retrofitted building in Ottawa, analyzed under amplified Long Event #2

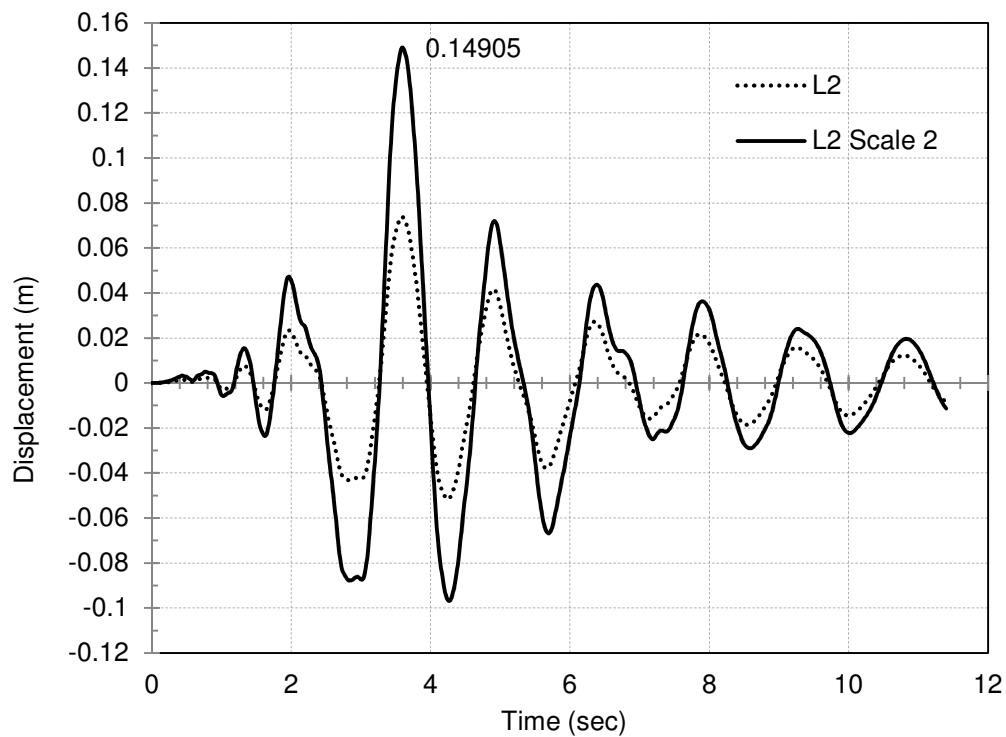


Figure 8.53: Top displacement time history for the un-retrofitted building in Ottawa, analyzed under amplified Long Event #2



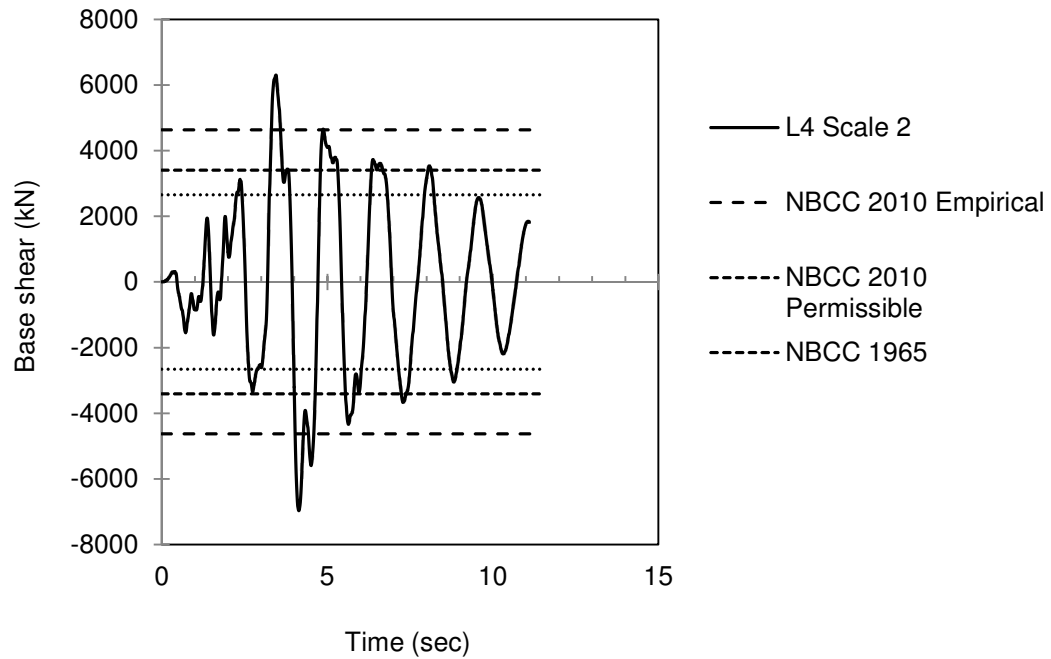


Figure 8.54: Comparison of dynamic base shear demands for amplified Long Event #4 and static base shears calculated based on NBCC for the un-retrofitted building in Ottawa

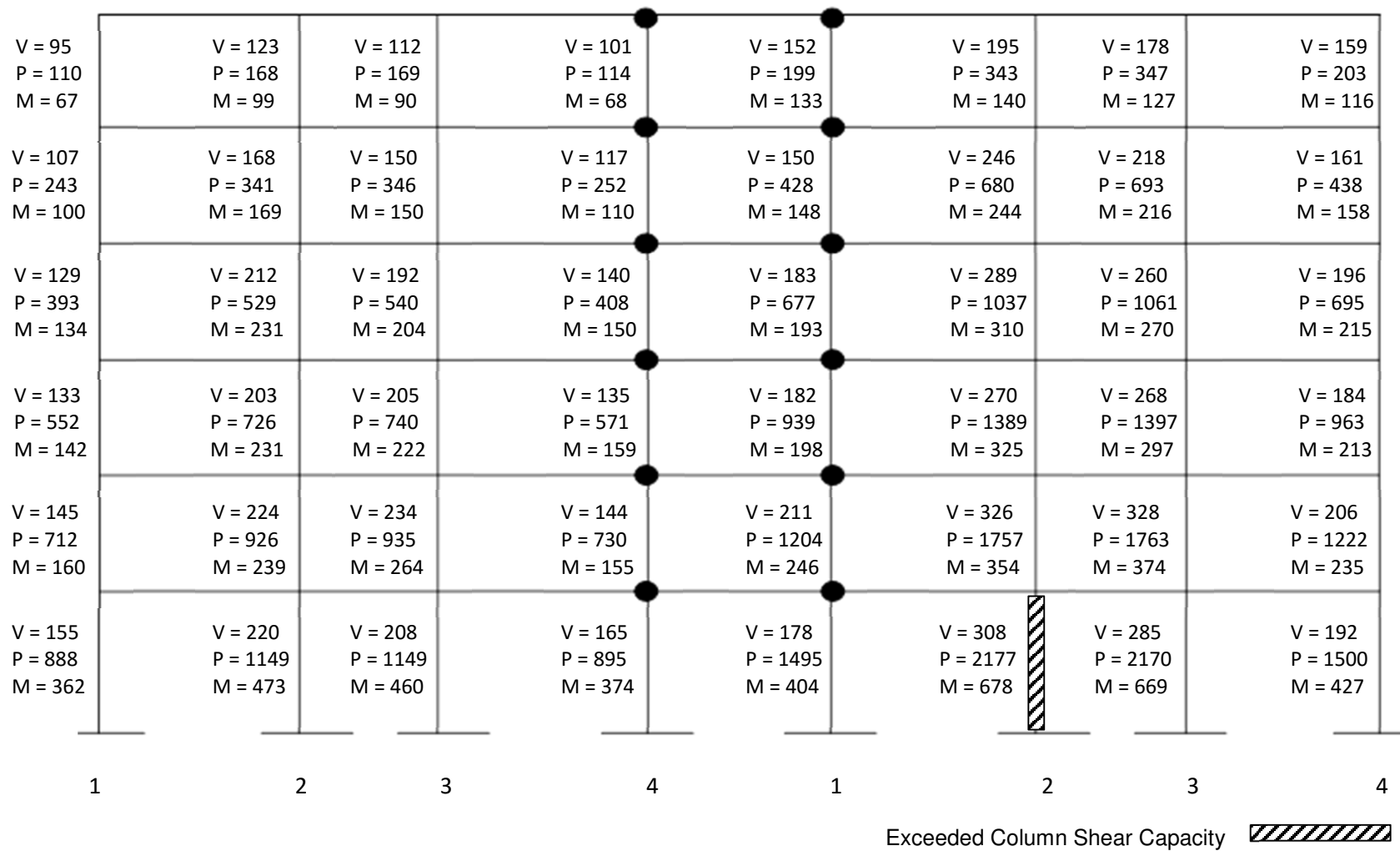
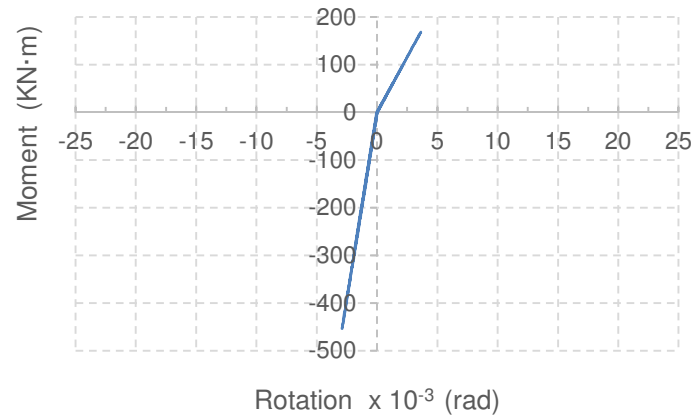
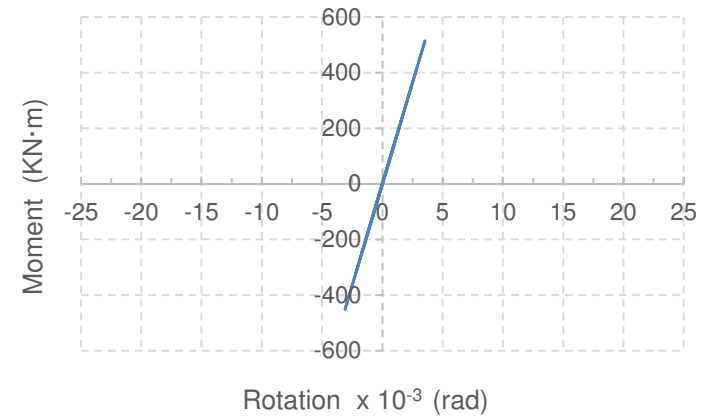


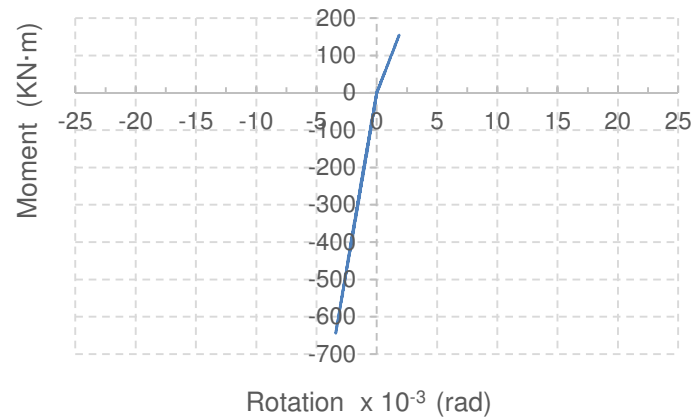
Figure 8.55: Column shear force, axial force, and moment demands for the un-retrofitted building in Ottawa, analyzed under amplified Long Event #4



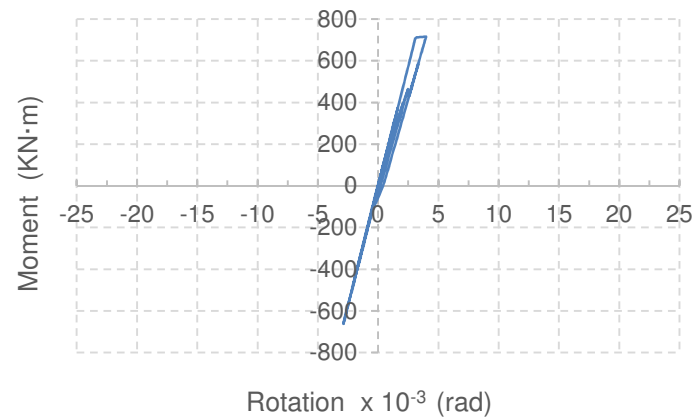
(a)



(b)



(c)



(d)

Figure 8.56 : Moment-rotation hysteretic relationships for selected beams and columns of the un-retrofitted building in Ottawa analyzed under amplified Long Event #4: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames

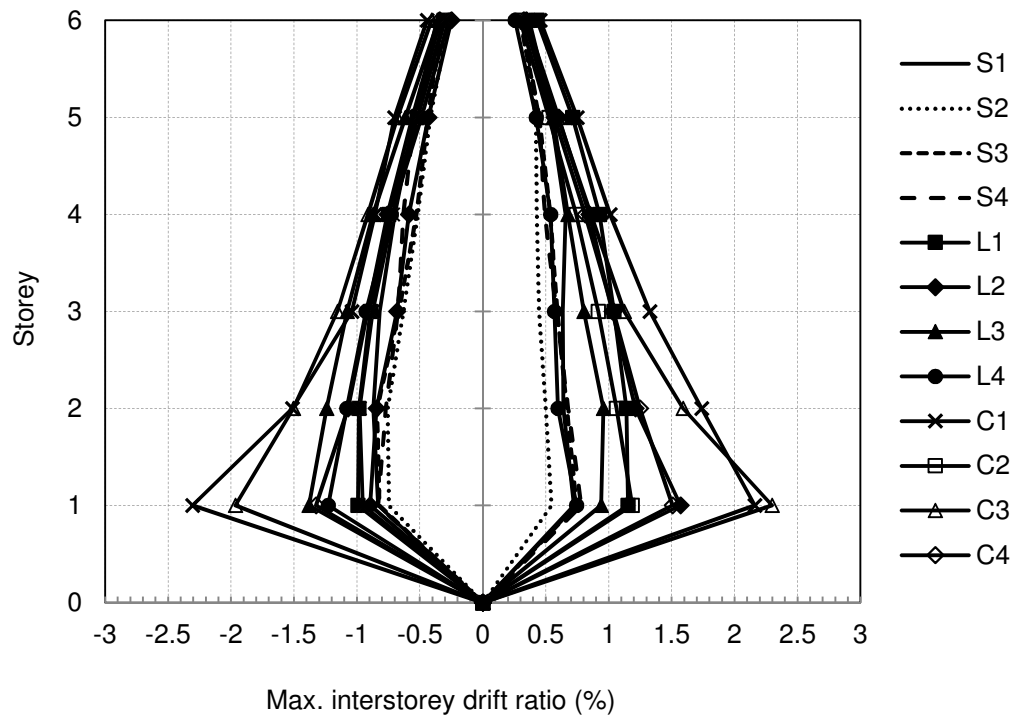


Figure 8.57: Maximum interstorey drift ratios for the un-retrofitted building in Vancouver, analyzed under earthquake records of short, long, and Cascadia events

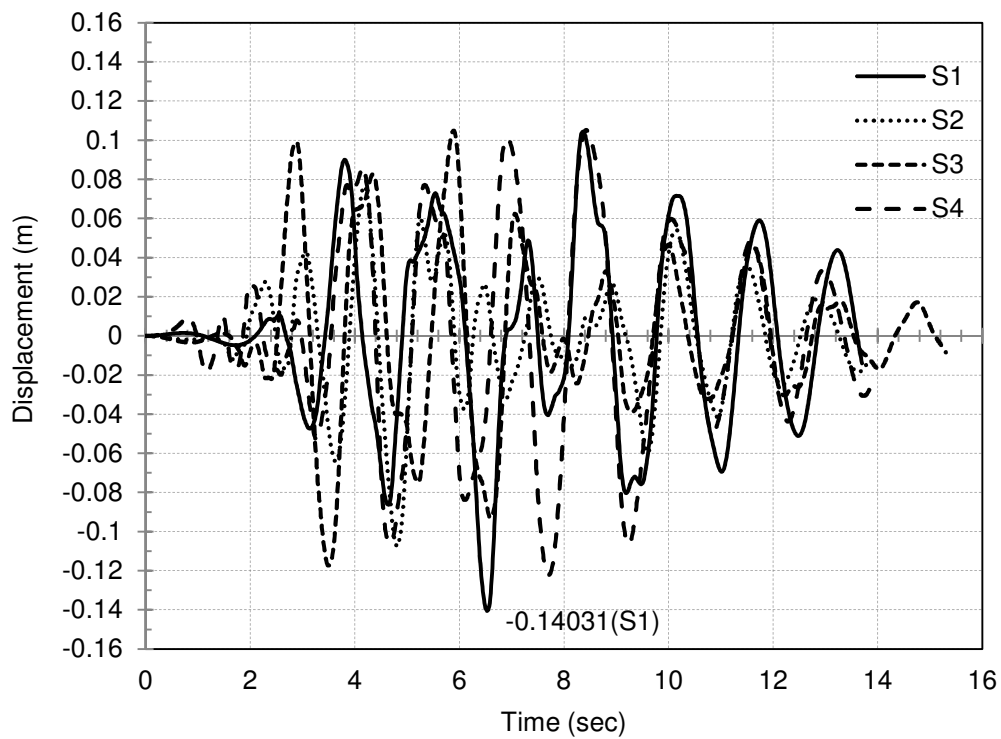


Figure 8.58: Top displacement time histories for the un-retrofitted building in Vancouver, analyzed under earthquake records of short events

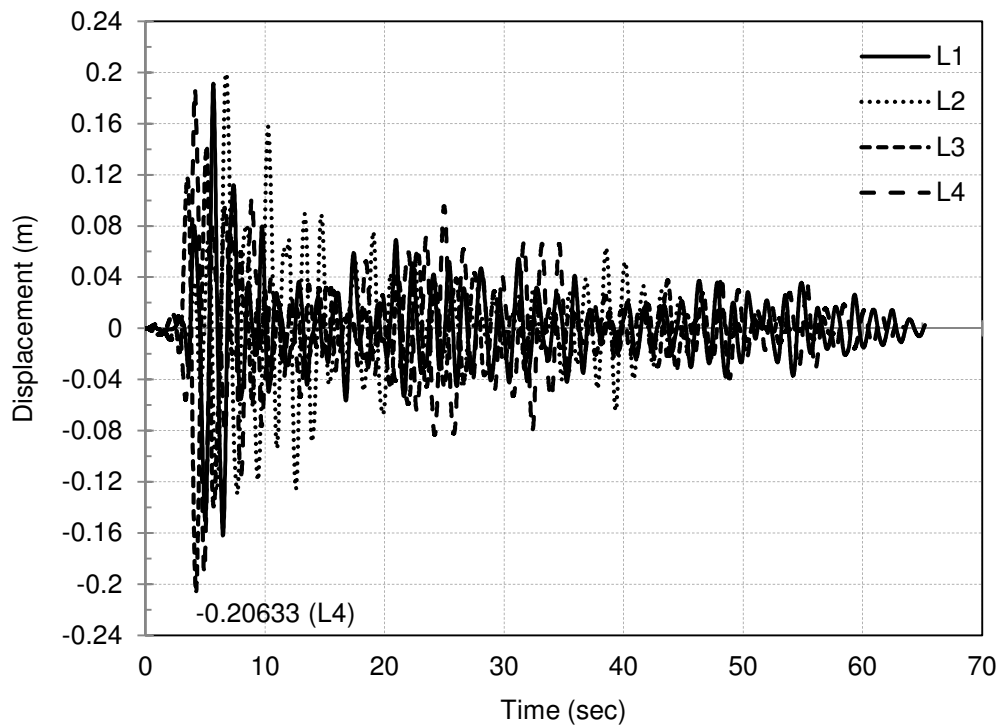


Figure 8.59: Top displacement time histories for the un-retrofitted building in Vancouver, analyzed under earthquake records of long events

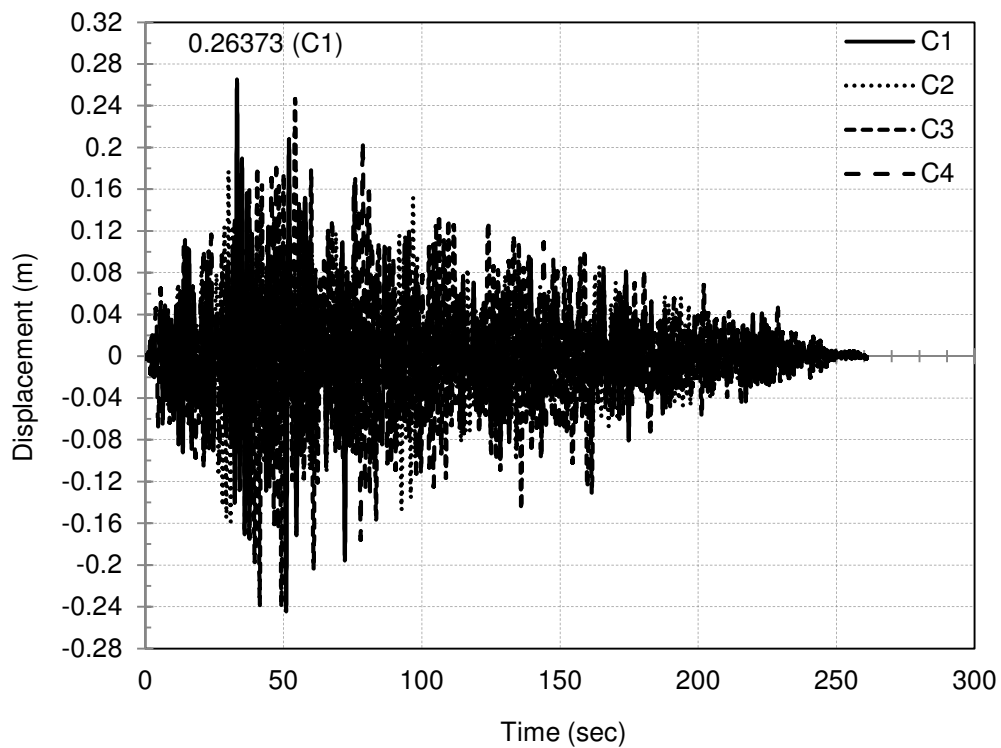


Figure 8.60: Top displacement time histories for the un-retrofitted building in Vancouver, analyzed under earthquake records of Cascadia events

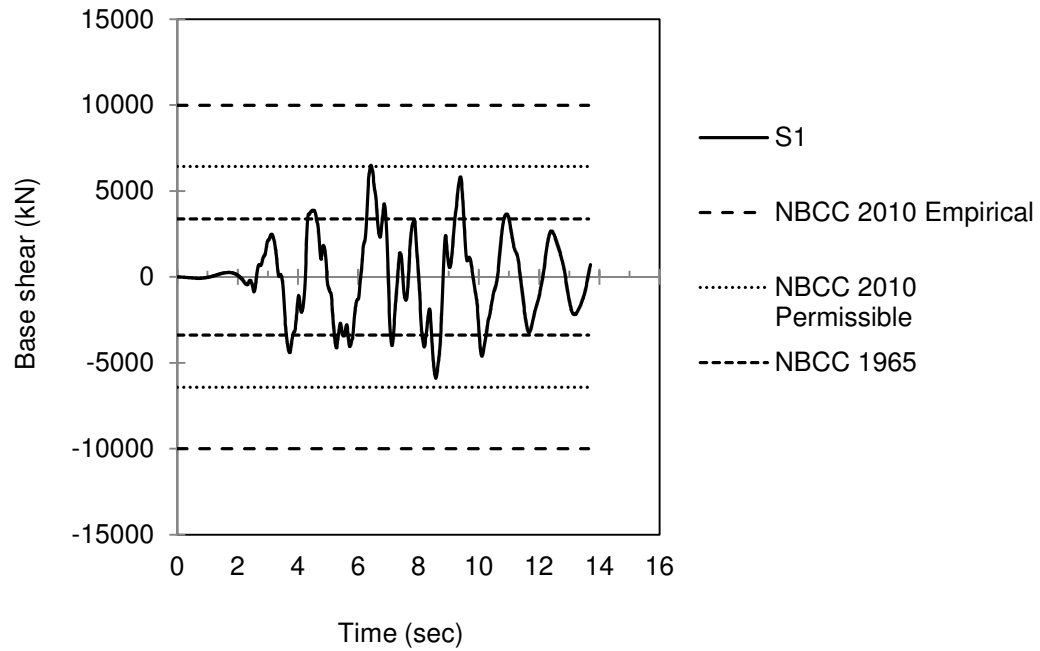


Figure 8.61: Comparison of dynamic base shear demands for Short Event #1 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

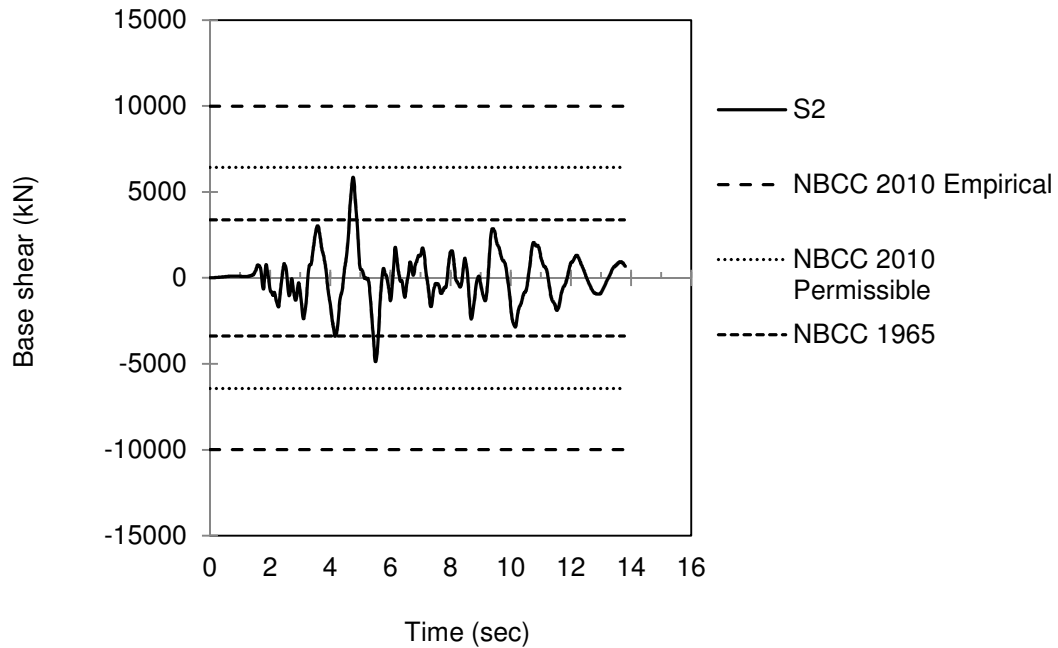


Figure 8.62: Comparison of dynamic base shear demands for Short Event #2 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver



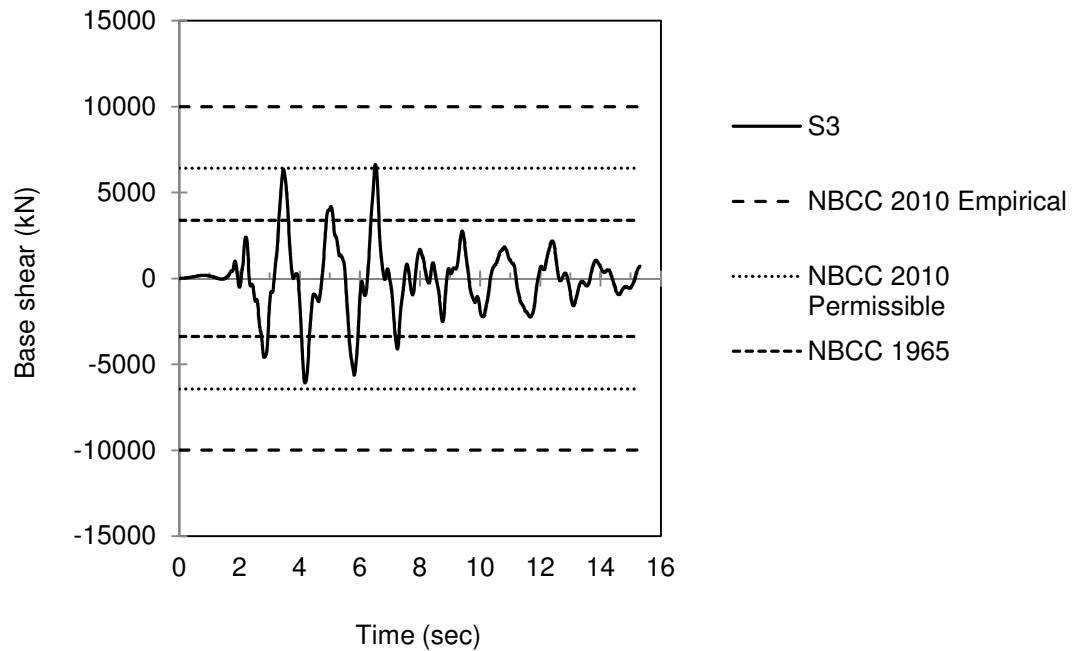


Figure 8.63: Comparison of dynamic base shear demands for Short Event #3 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

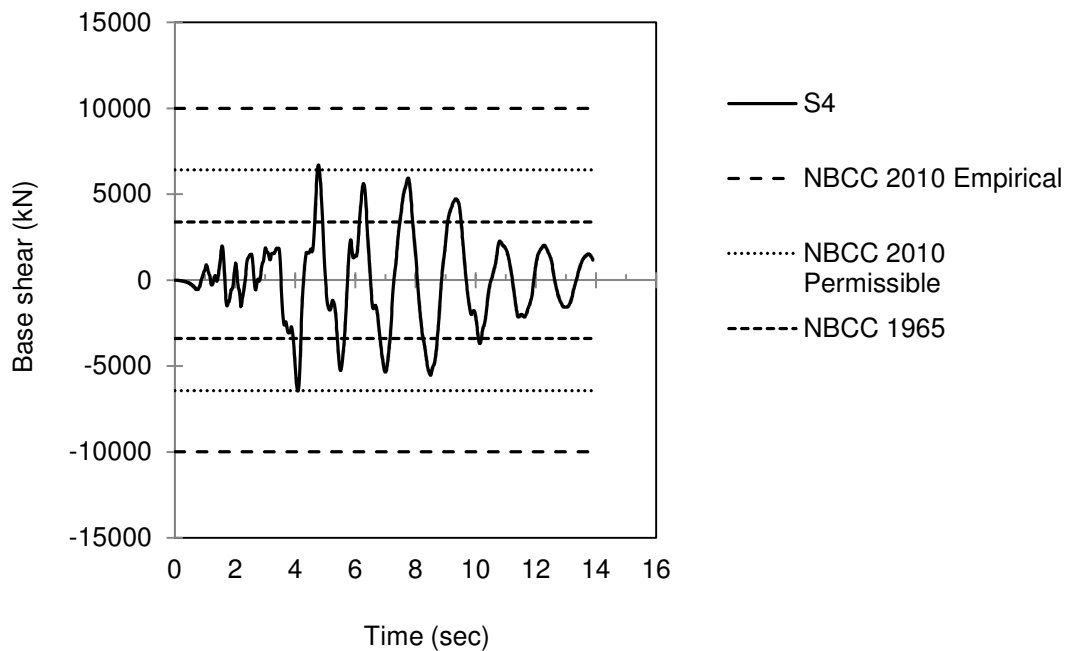


Figure 8.64: Comparison of dynamic base shear demands for Short Event #4 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

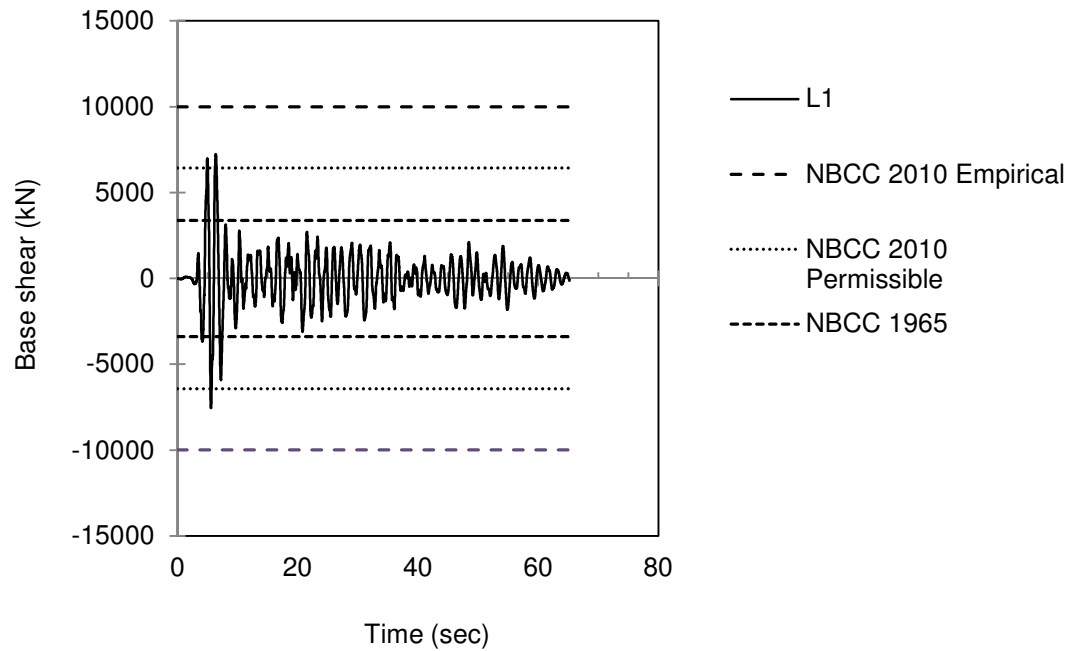


Figure 8.65: Comparison of dynamic base shear demands for Long Event #1 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

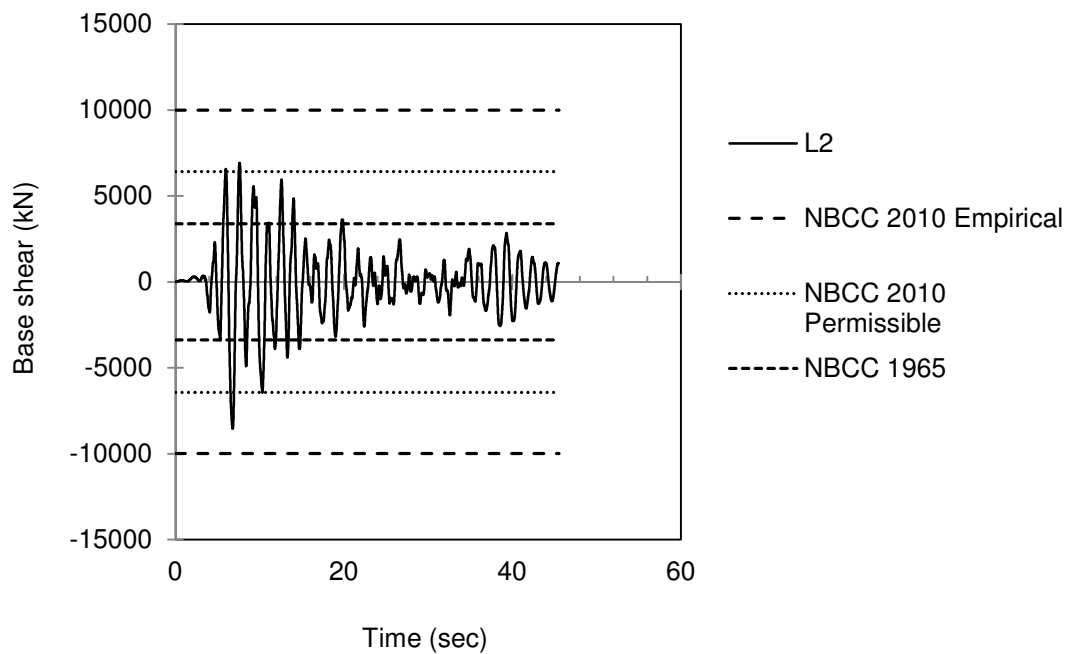


Figure 8.66: Comparison of dynamic base shear demands for Long Event #2 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

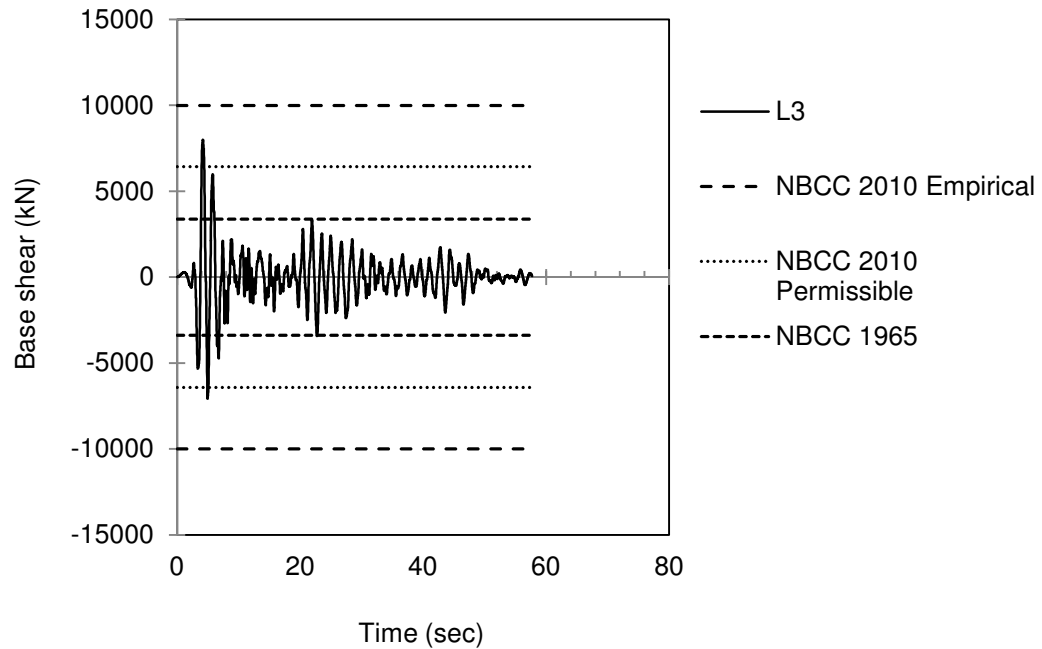


Figure 8.67: Comparison of dynamic base shear demands for Long Event #3 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

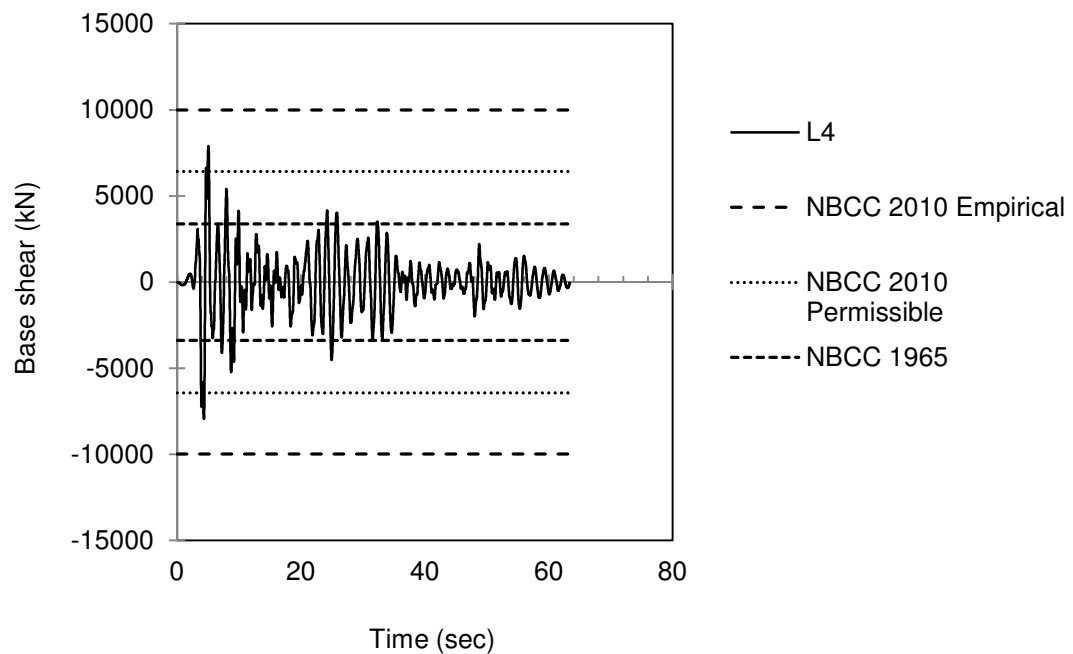


Figure 8.68: Comparison of dynamic base shear demands for Long Event #4 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

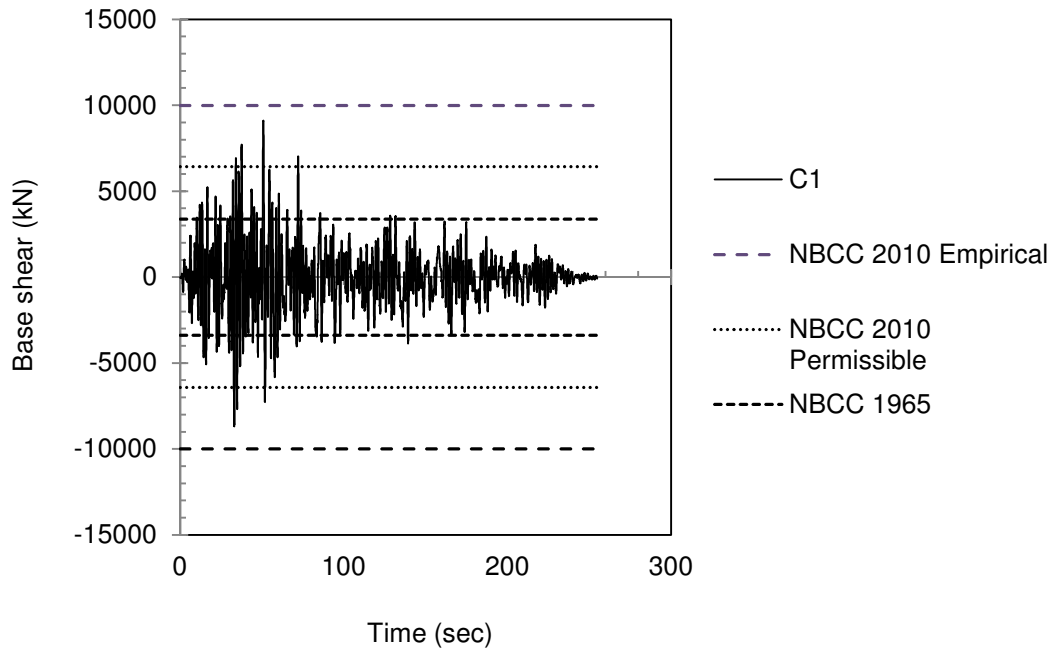


Figure 8.69: Comparison of dynamic base shear demands for Cascadia Event #1 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

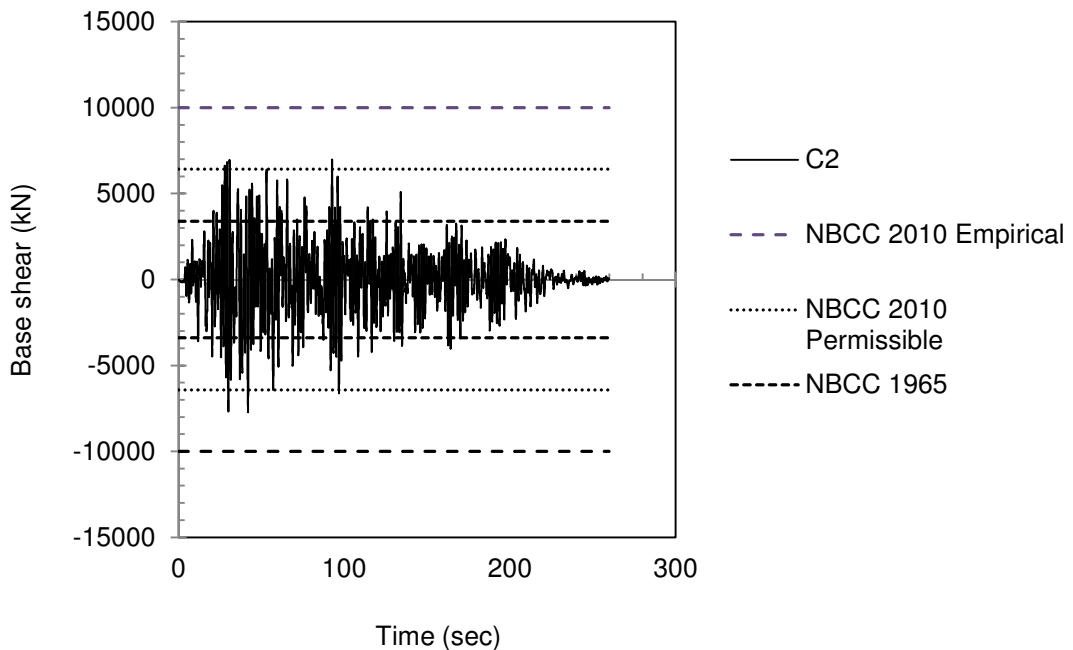


Figure 8.70: Comparison of dynamic base shear demands for Cascadia Event #2 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

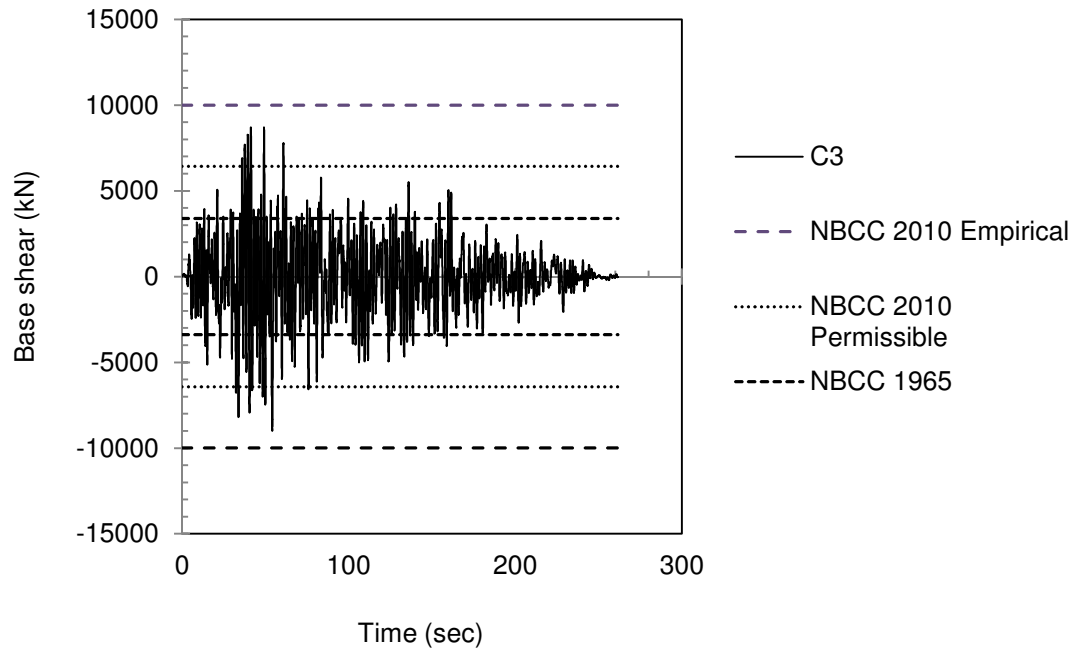


Figure 8.71: Comparison of dynamic base shear demands for Cascadia Event #3 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

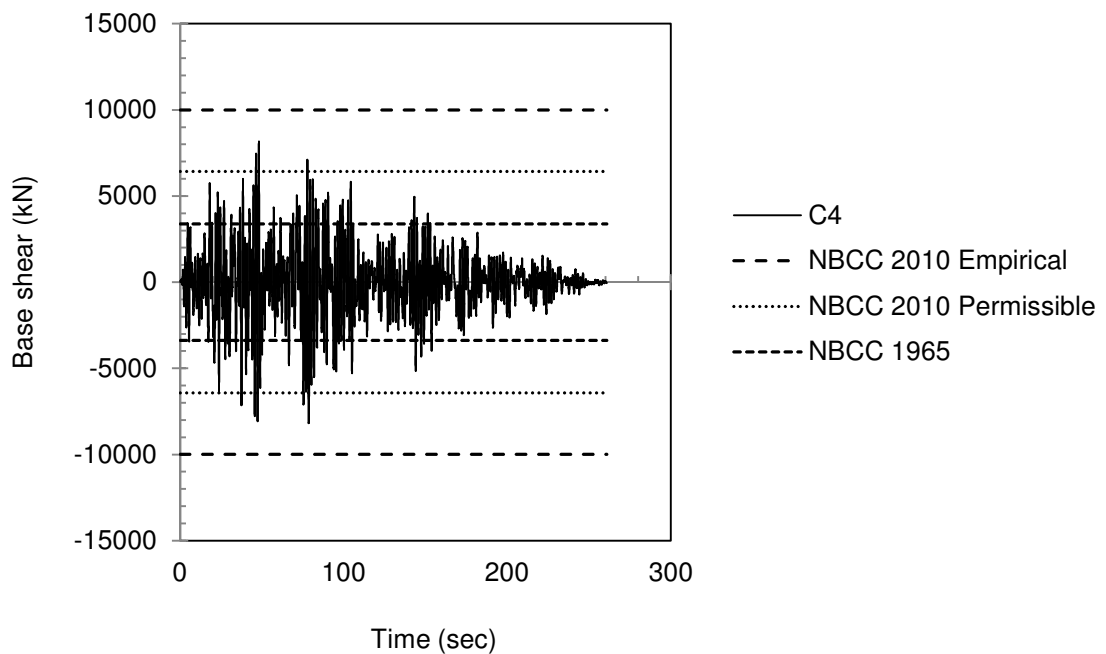


Figure 8.72: Comparison of dynamic base shear demands for Cascadia Event #4 and static base shears calculated based on NBCC for the un-retrofitted building in Vancouver

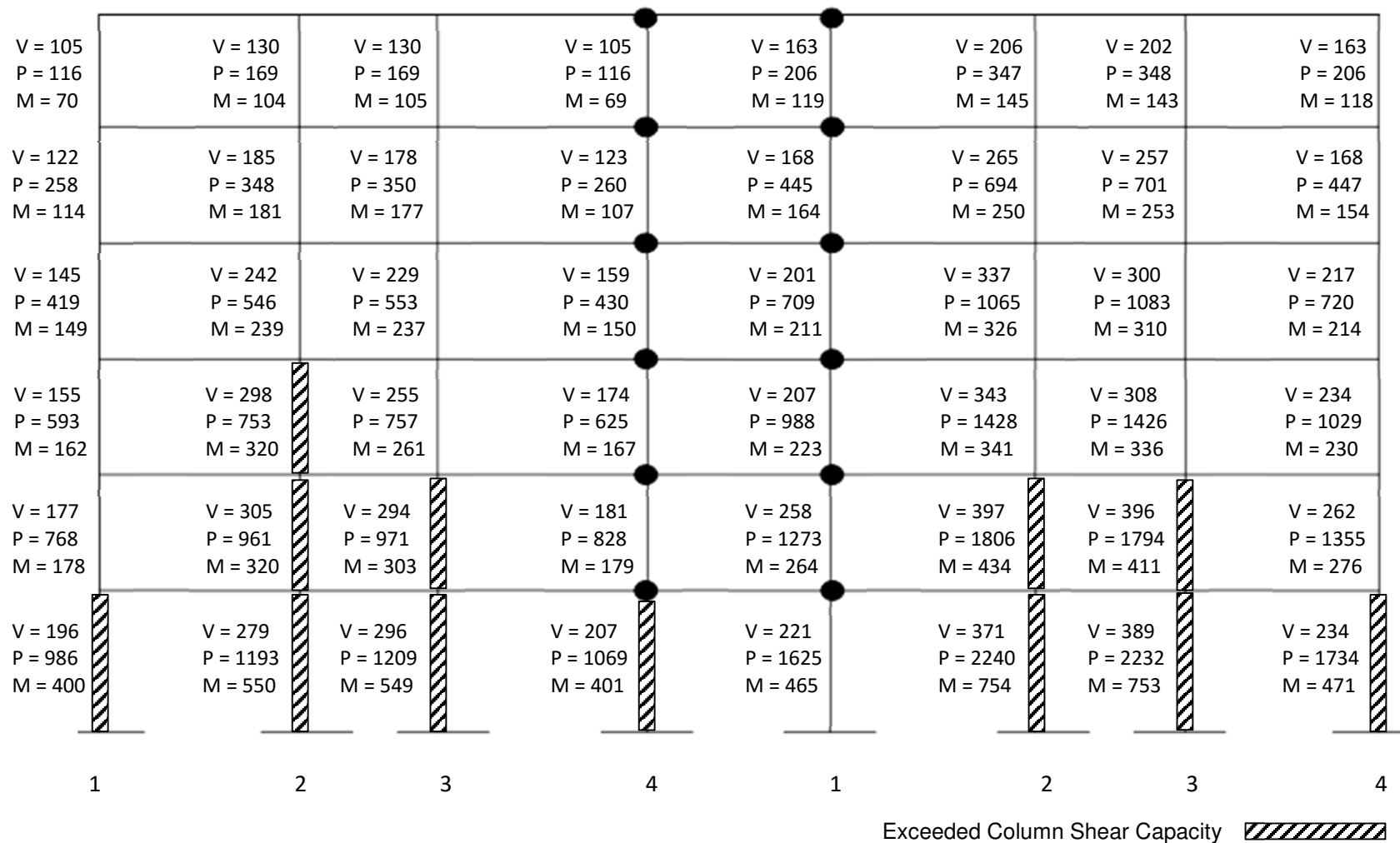
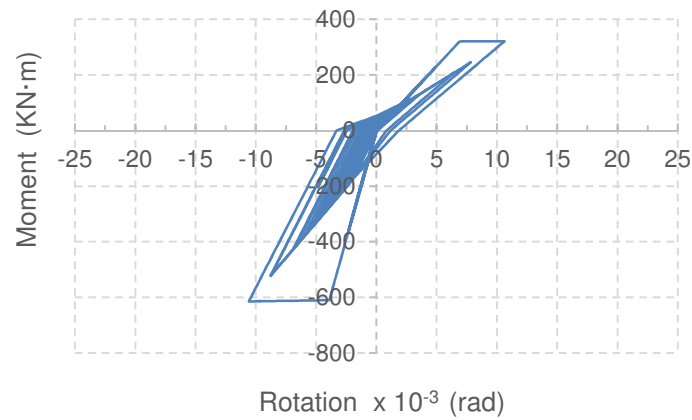
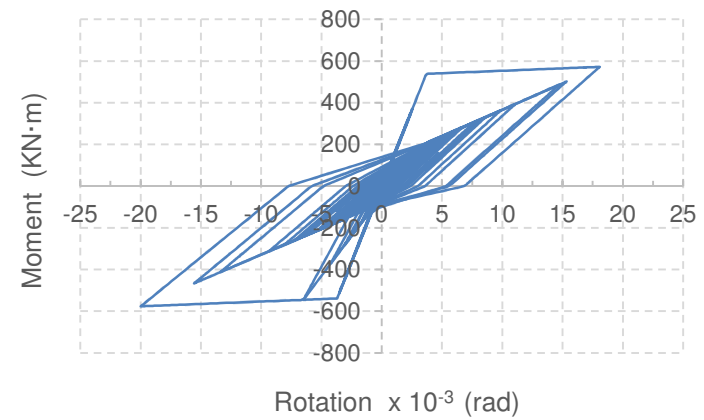


Figure 8.73: Column shear force, axial force, and moment demands for the un-retrofitted building in Vancouver, analyzed under Cascadia Event #1

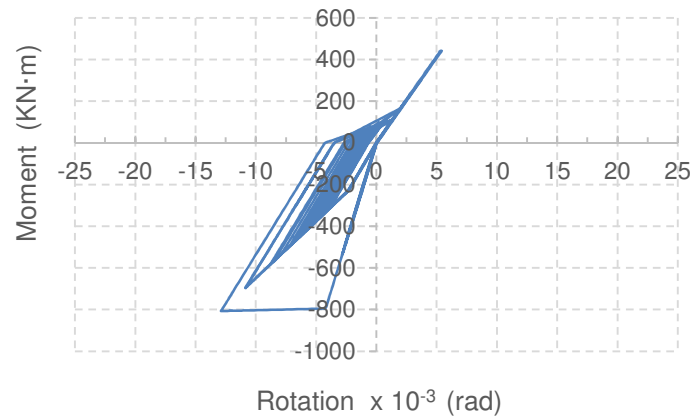




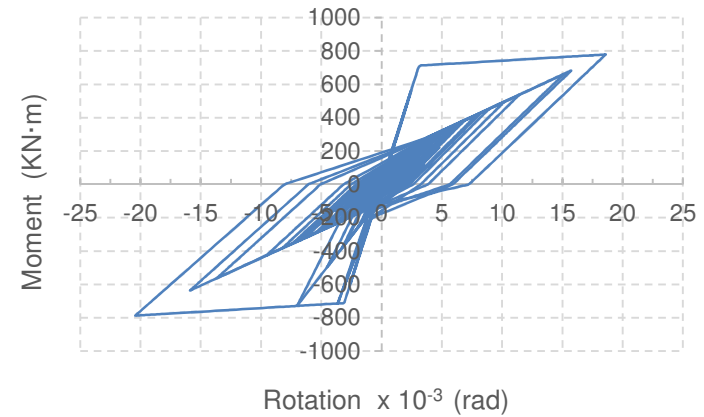
(a)



(b)



(c)



(d)

Figure 8.74: Moment-rotation hysteretic relationships for selected beams and columns of the un-retrofitted building in Vancouver analyzed under Cascadia Event #1: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames

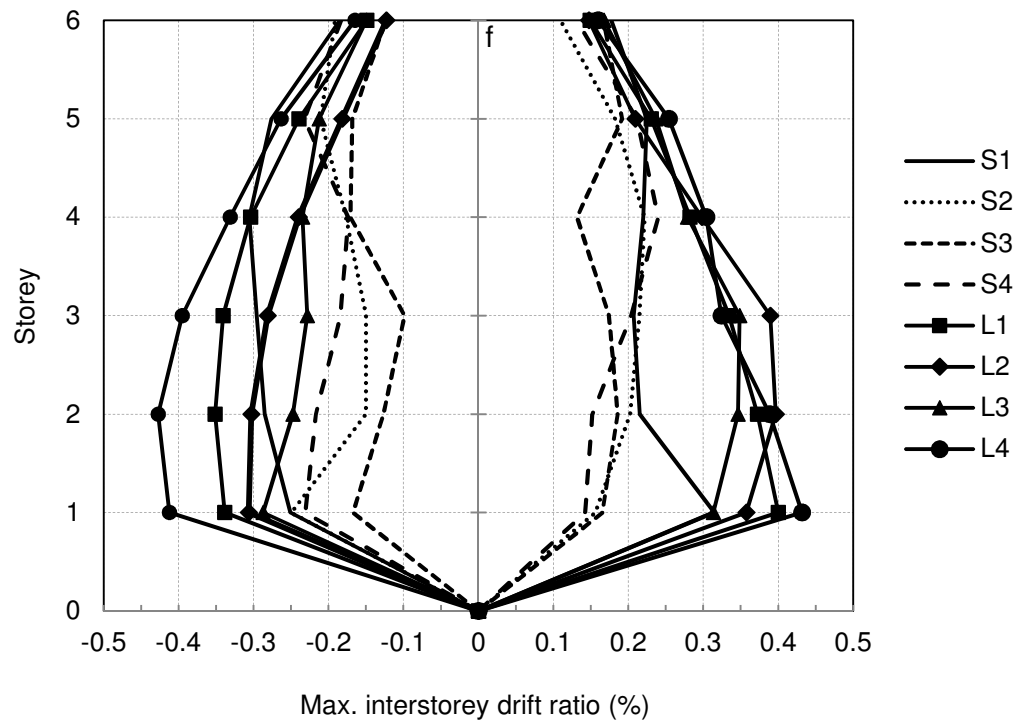


Figure 8.75: Maximum interstorey drift ratios for the retrofitted building in Ottawa, analyzed under earthquake records of short and long events

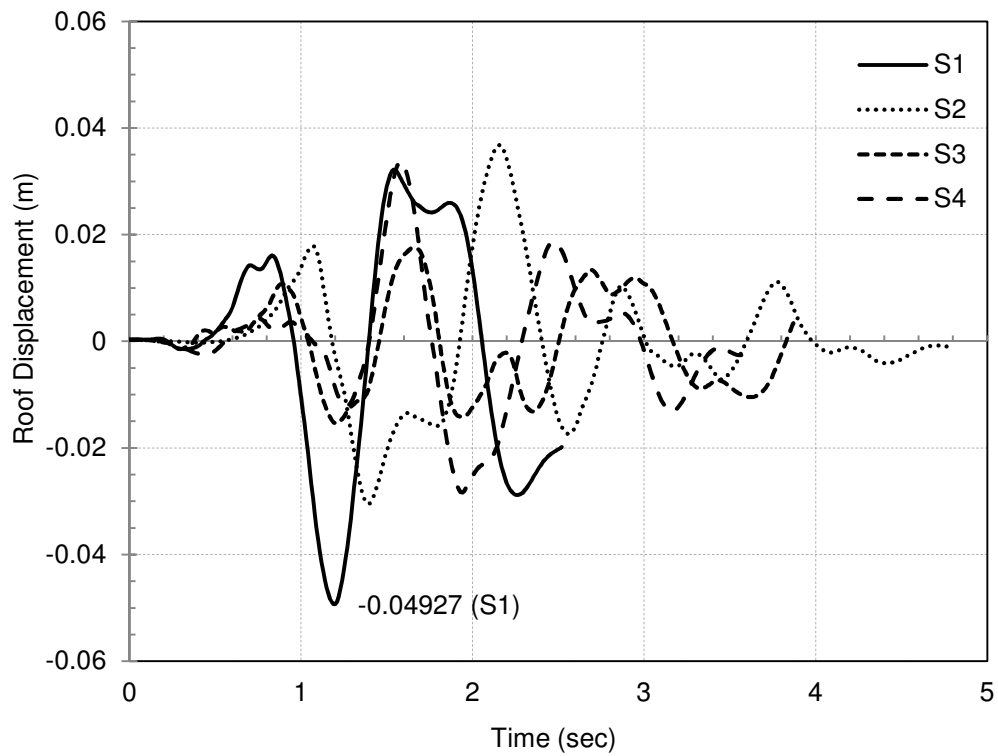


Figure 8.76: Top displacement time histories for the retrofitted building in Ottawa, analyzed under earthquake records of short events

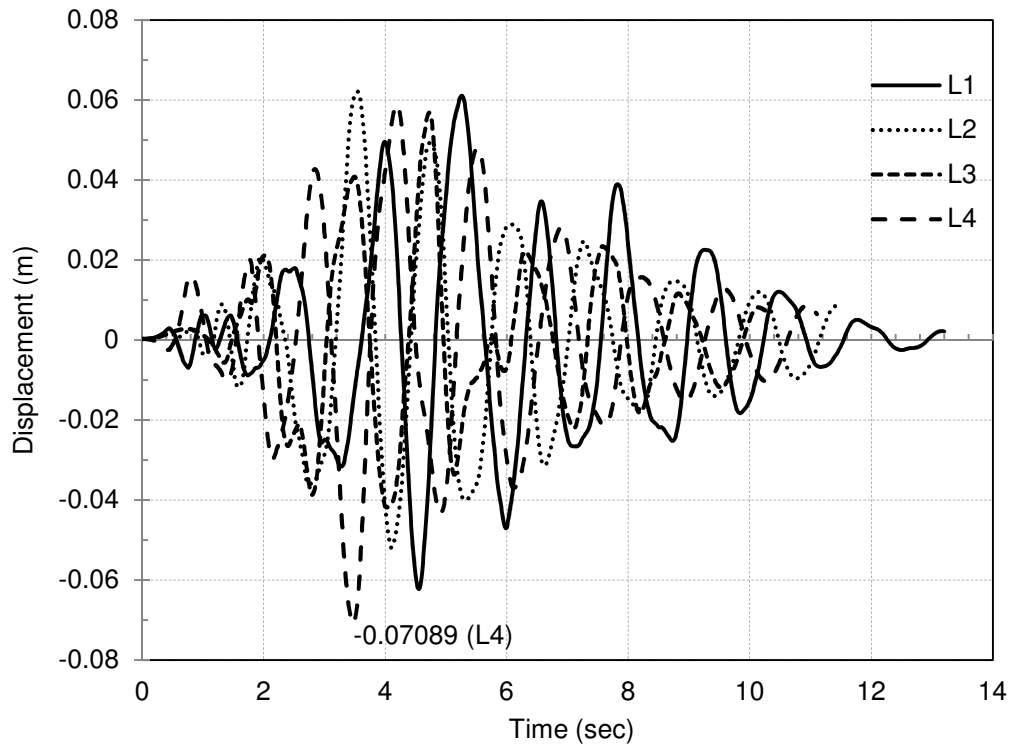


Figure 8.77: Top displacement time histories for the retrofitted building in Ottawa, analyzed under earthquake records of long events

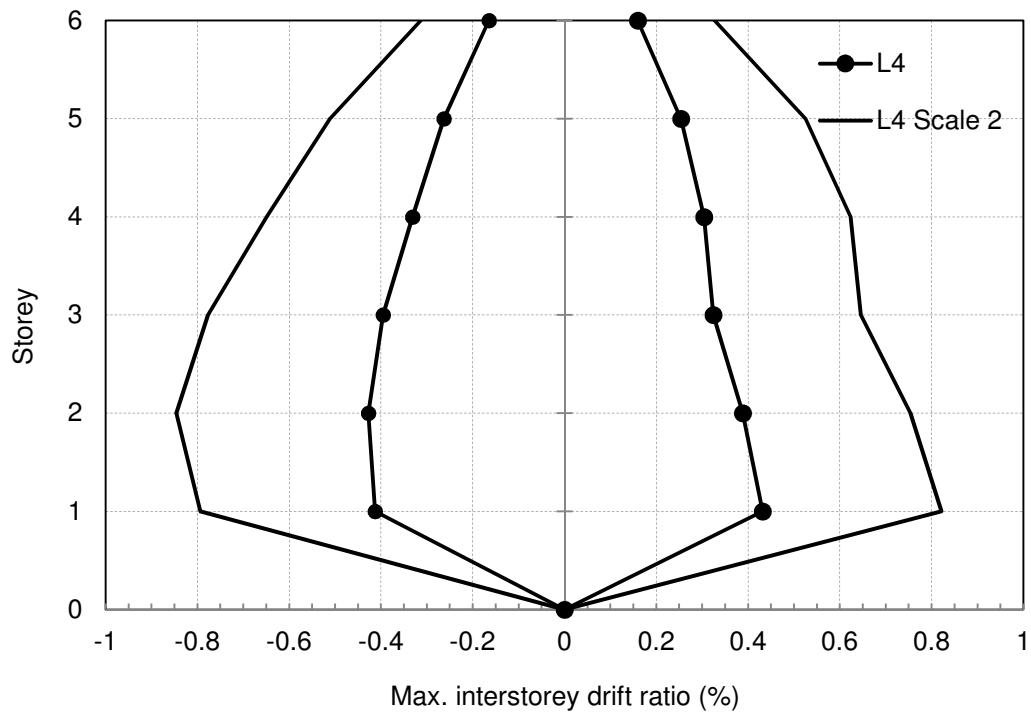


Figure 8.78: Maximum interstorey drift ratios for the retrofitted building in Ottawa, analyzed under UHS code compatible and amplified Long Event #4

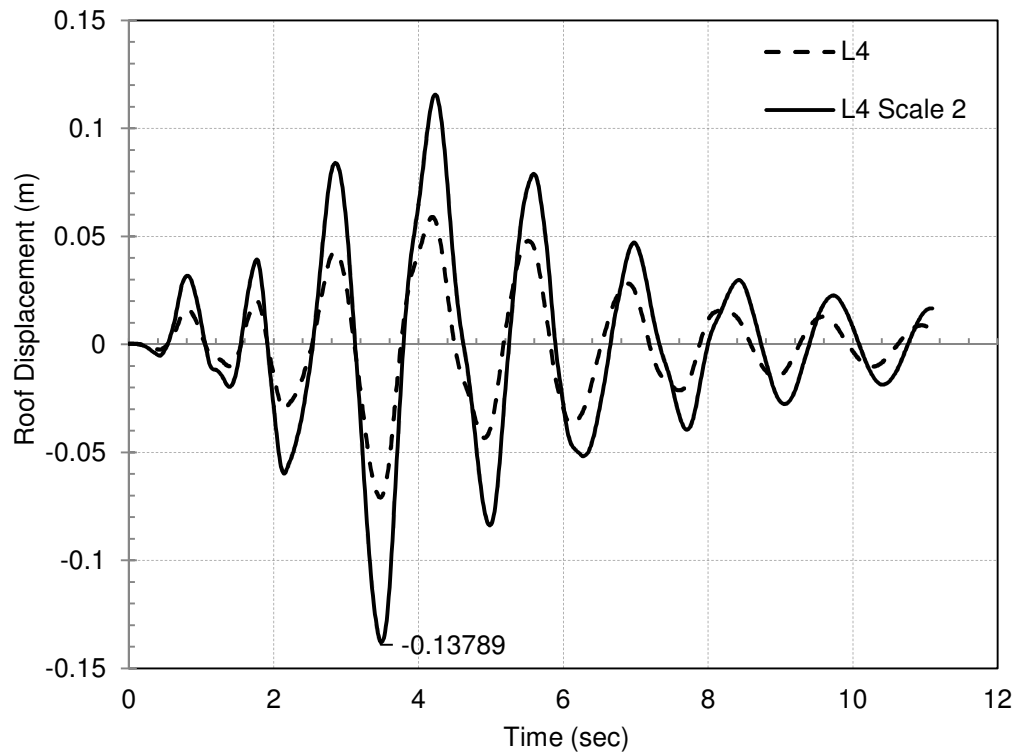


Figure 8.79: Top displacement time history for the retrofitted building in Ottawa, analyzed under UHS code compatible and amplified Long Event #4

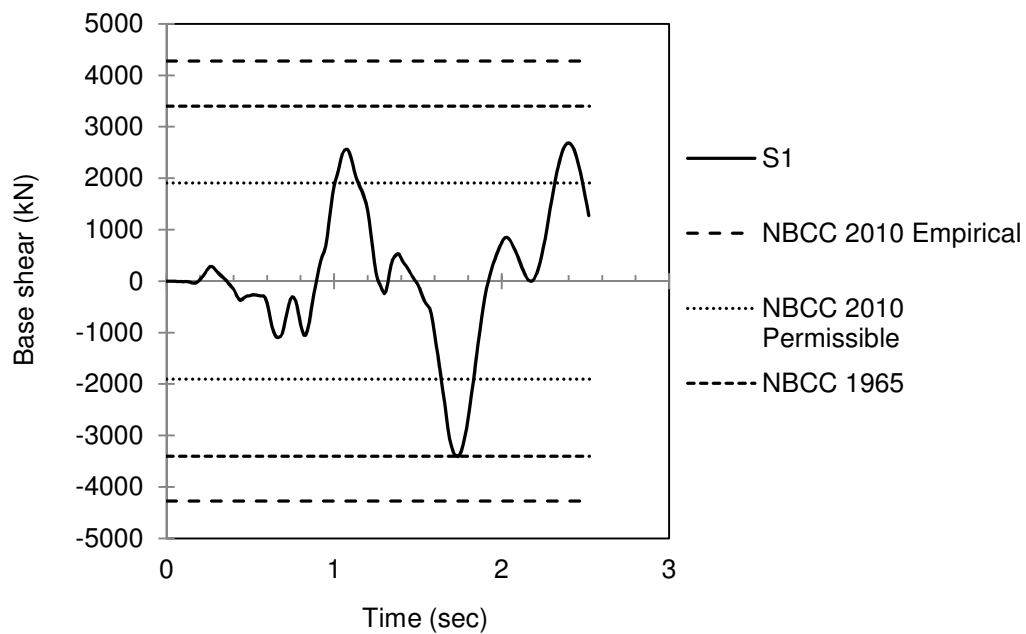


Figure 8.80: Comparison of dynamic base shear demands for Short Event #1 and static base shears based on NBCC for the retrofitted building in Ottawa

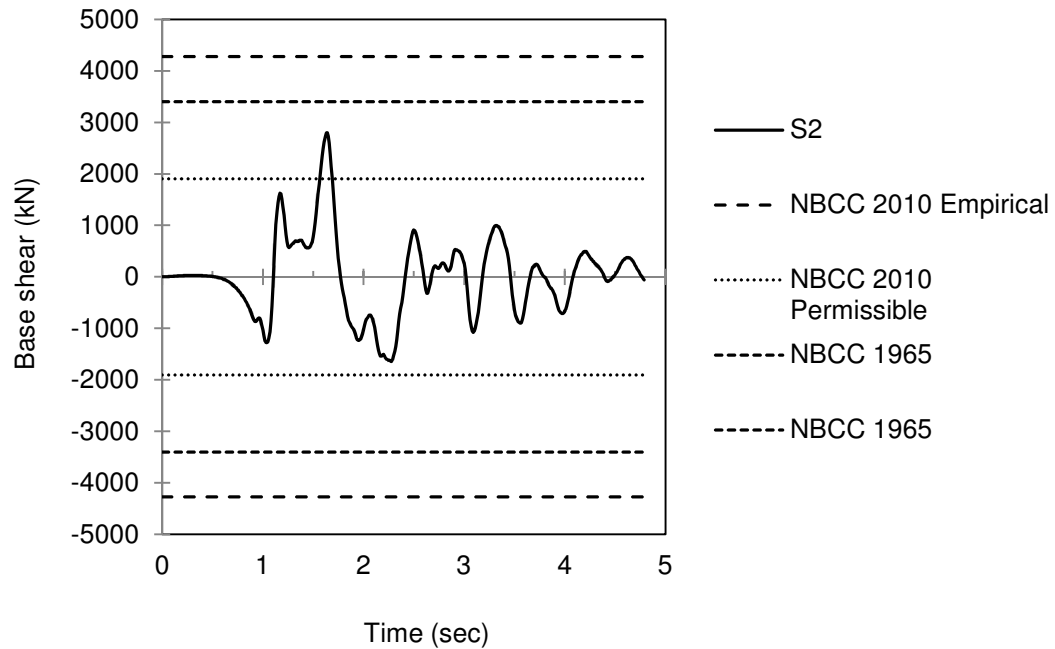


Figure 8.81: Comparison of dynamic base shear demands for Short Event #2 and static base shears based on NBCC for the retrofitted building in Ottawa

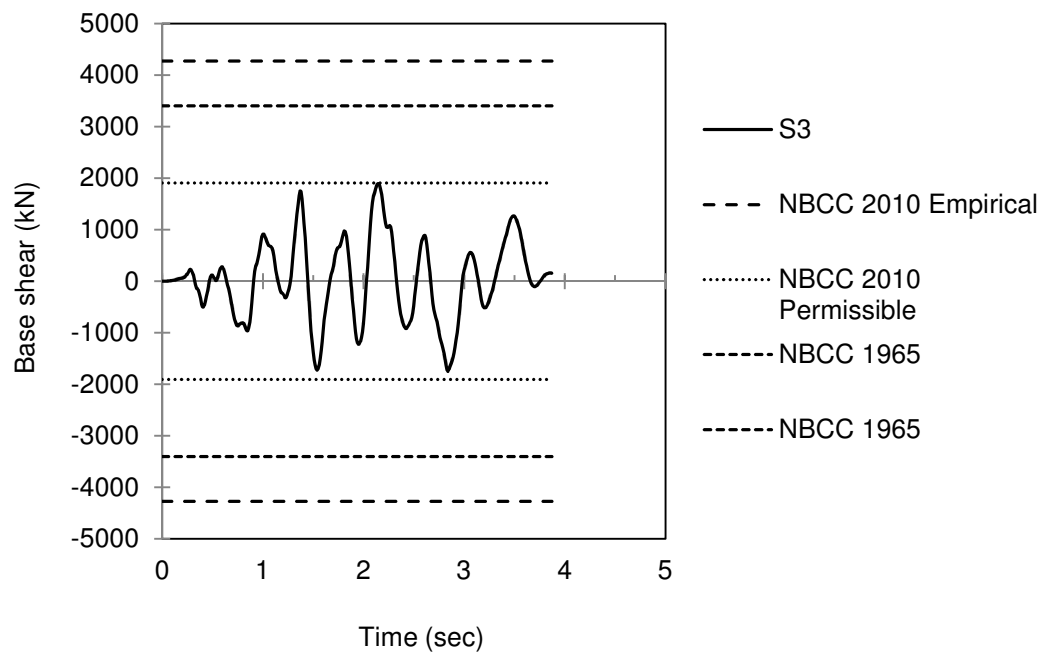


Figure 8.82: Comparison of dynamic base shear demands for Short Event #3 and static base shears based on NBCC for the retrofitted building in Ottawa

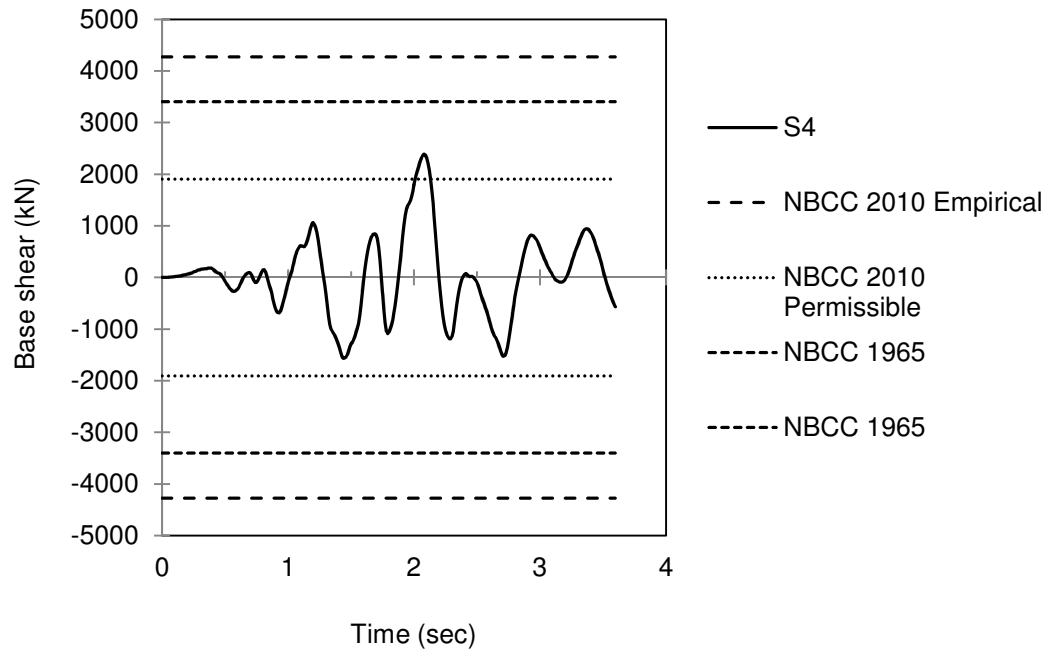


Figure 8.83: Comparison of dynamic base shear demands for Short Event #4 and static base shears based on NBCC for the retrofitted building in Ottawa

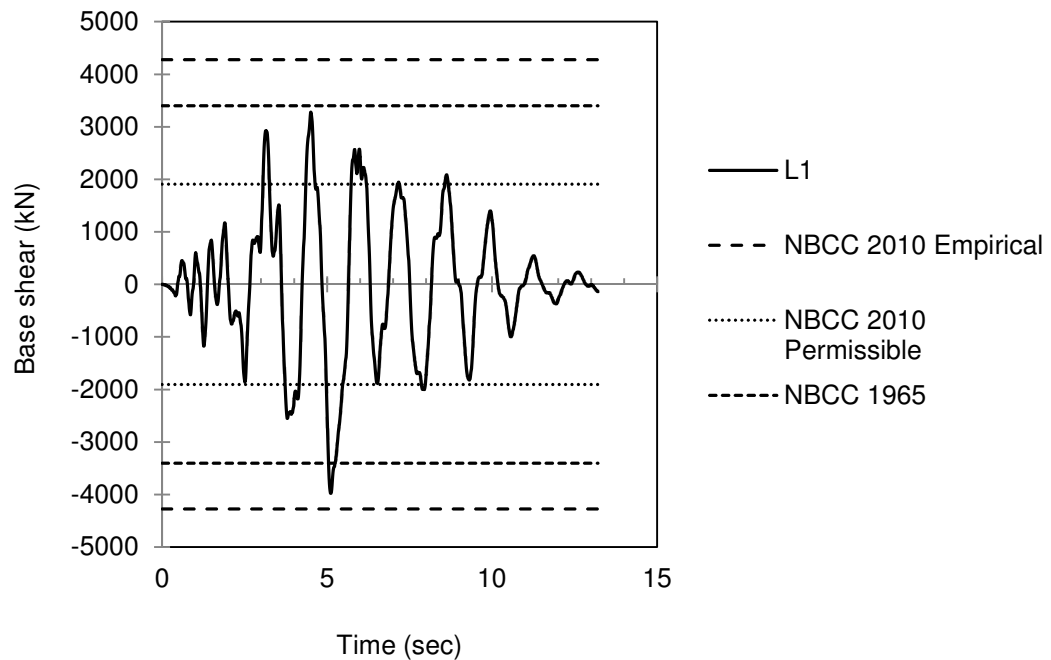


Figure 8.84: Comparison of dynamic base shear demands for Long Event #1 and static base shears based on NBCC for the retrofitted building in Ottawa



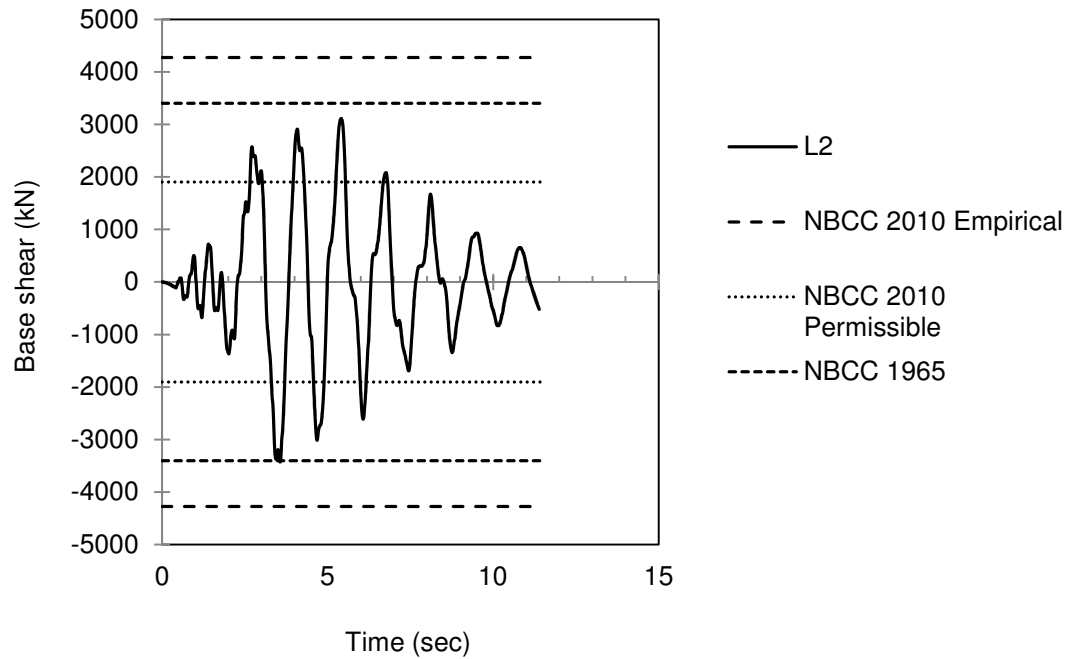


Figure 8.85: Comparison of dynamic base shear demands for Event #2 and static base shears based on NBCC for the un-retrofitted building in Ottawa

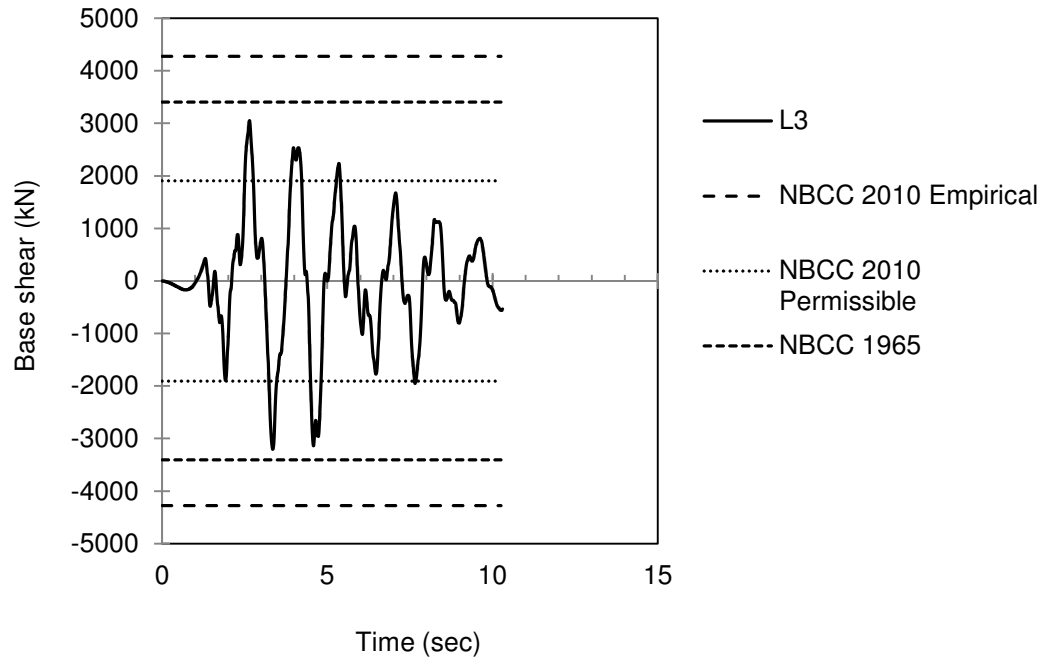


Figure 8.86: Comparison of dynamic base shear demands for Long Event #3 and static base shears based on NBCC for the retrofitted building in Ottawa

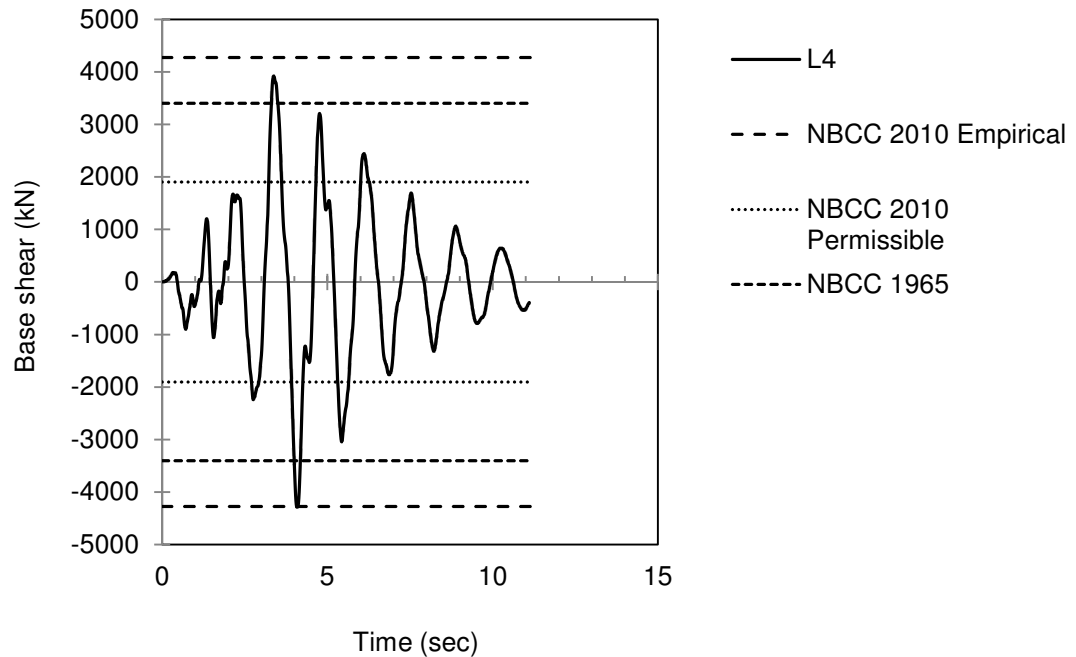


Figure 8.87: Comparison of dynamic base shear demands for Long Event #4 and static base shears based on NBCC for the retrofitted building in Ottawa

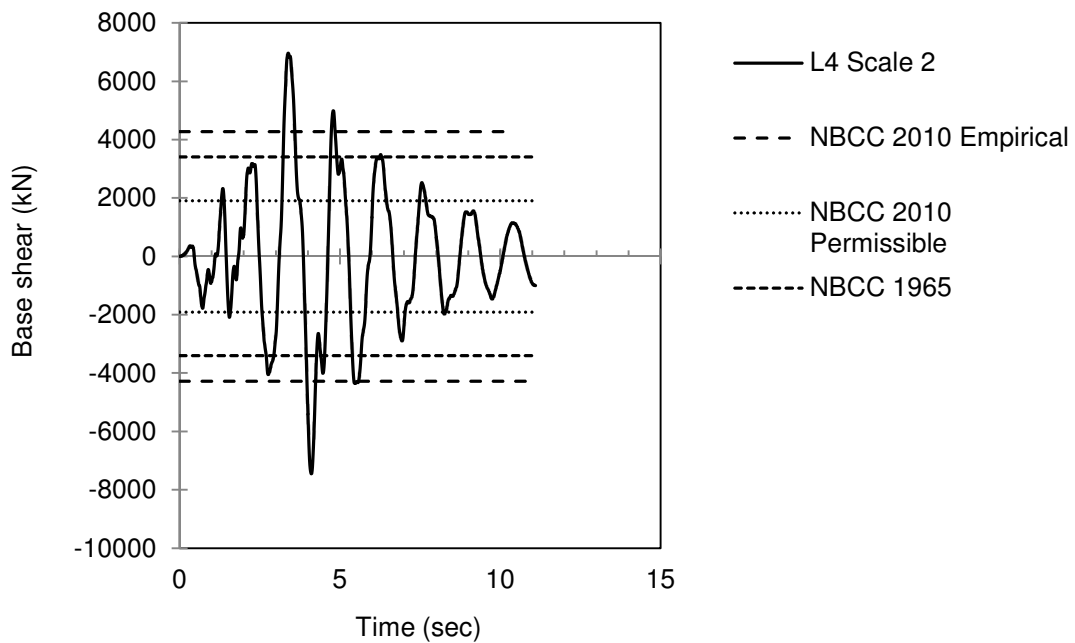


Figure 8.88: Comparison of dynamic base shear demands for amplified Long Event #4 and static base shear based on NBCC for the retrofitted building in Ottawa

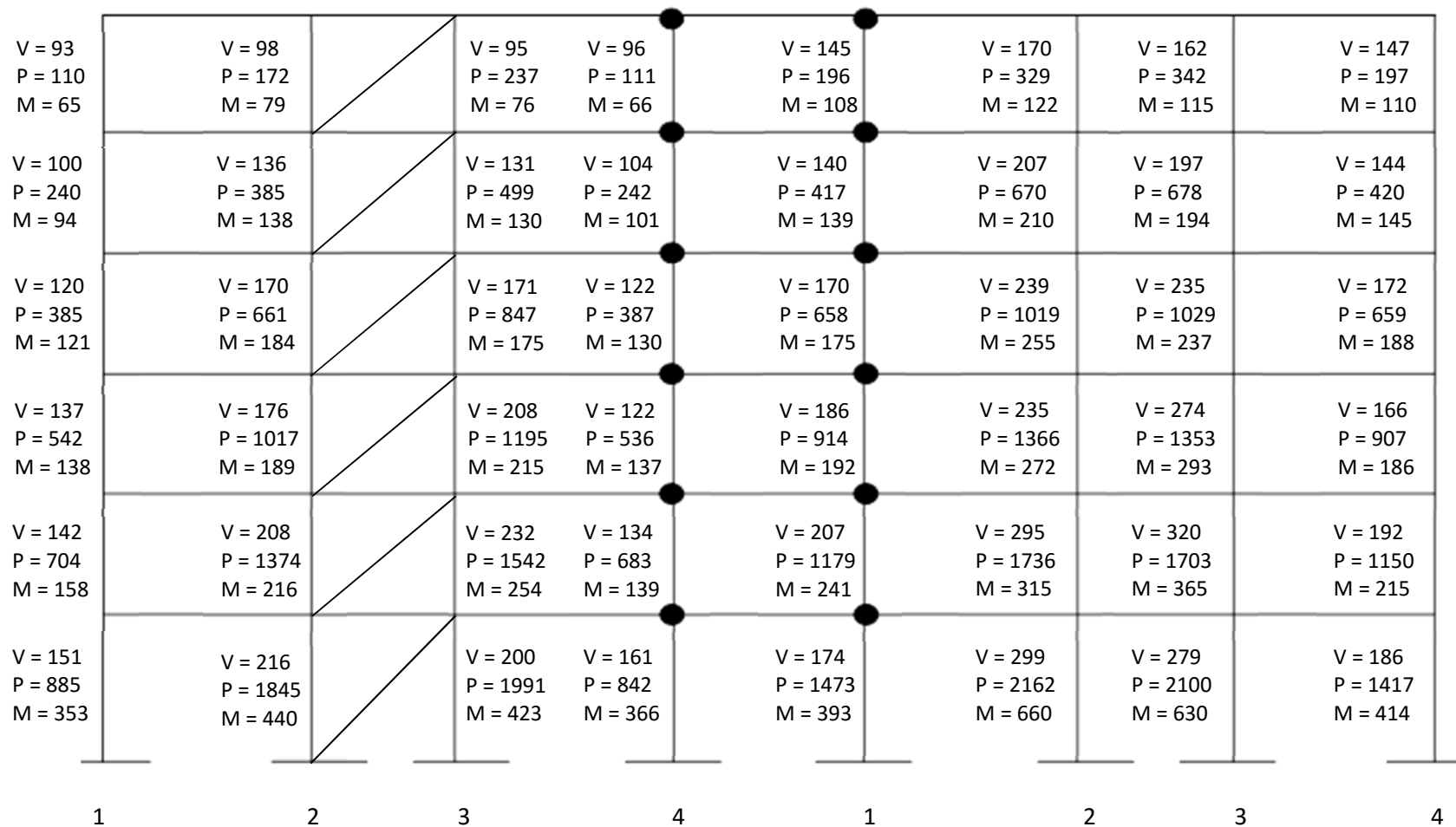
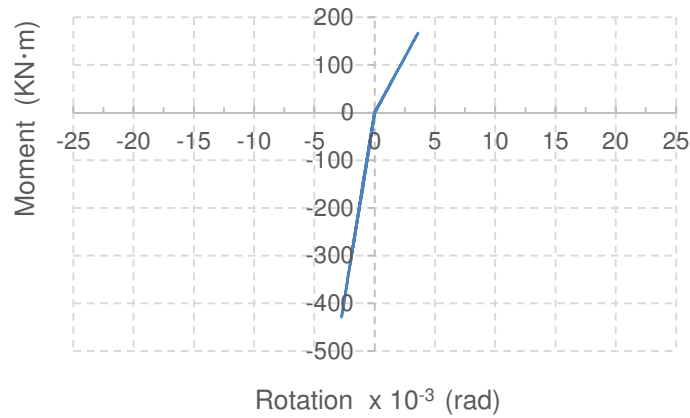
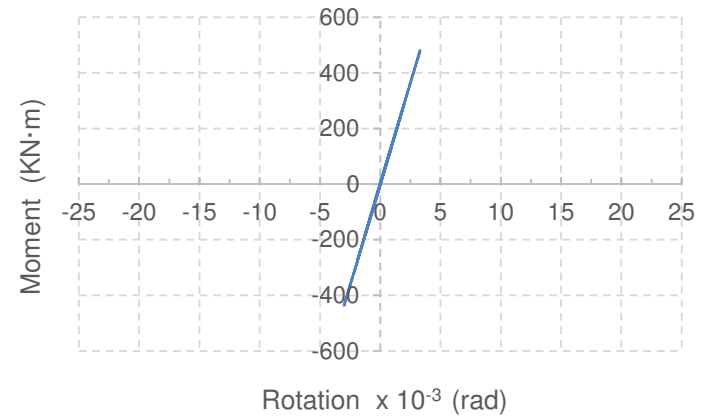


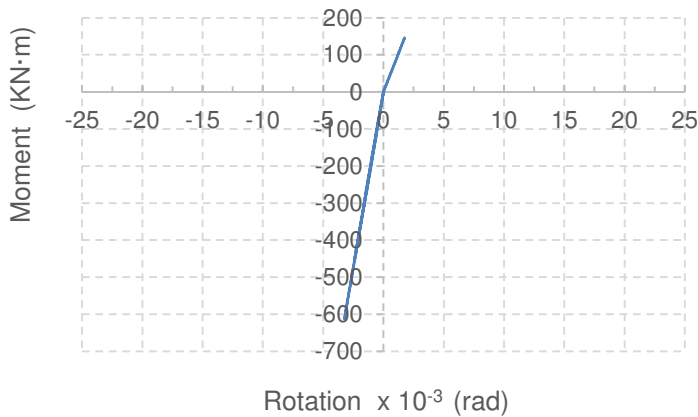
Figure 8.89: Column shear force, axial force, and moment demands for the retrofitted building in Ottawa, analyzed under amplified Long Event #4



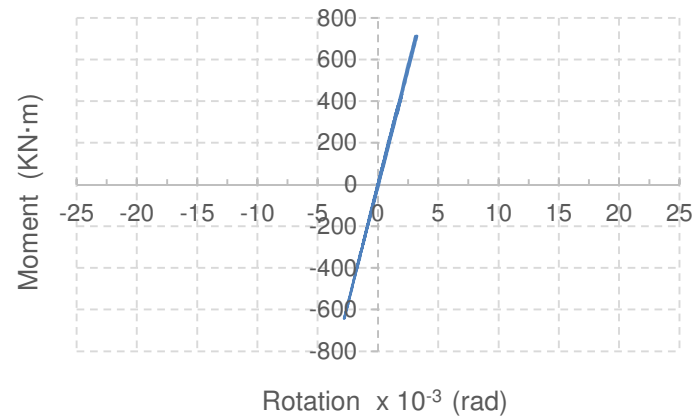
(a)



(b)

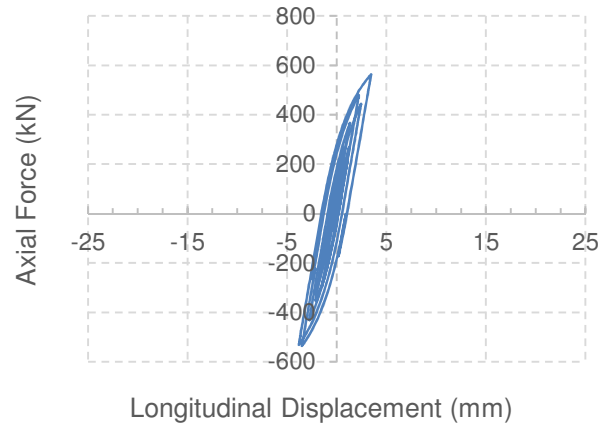


(c)

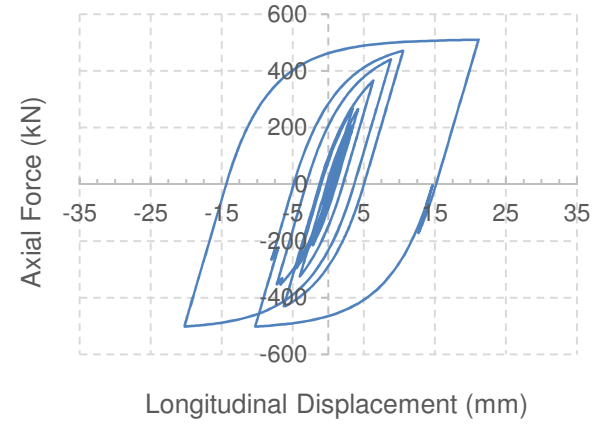


(d)

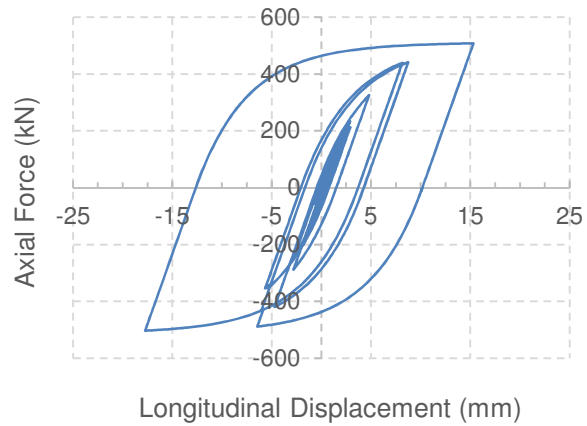
Figure 8.90 : Moment-rotation hysteretic relationship for selected beams and columns of the retrofitted building in Ottawa analyzed under amplified Long Event #2: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames



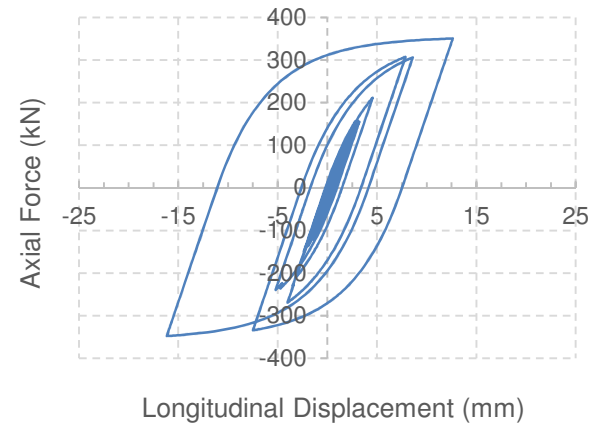
(a)



(b)

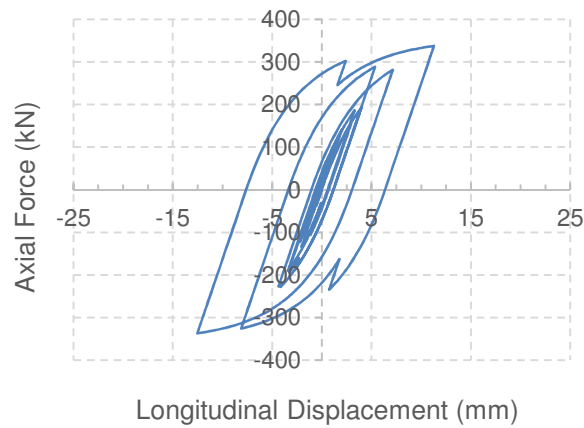


(c)

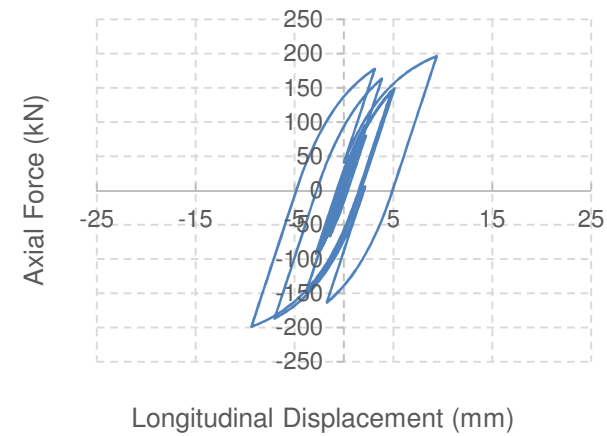


(d)

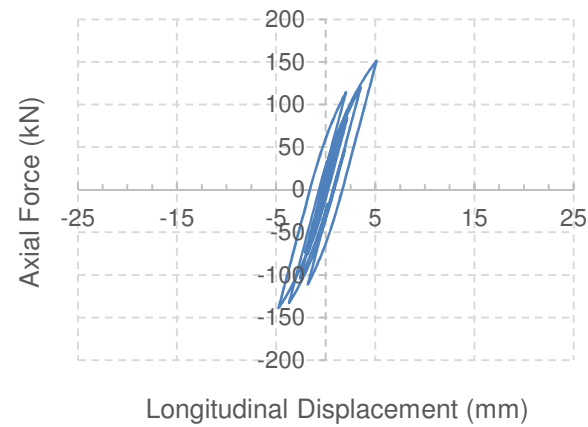
Figure 8.91: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the retrofitted building in Ottawa analyzed under amplified Long Event #2: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)



(f)



(g)

Figure 8.91 (Cont'd): Axial force-axial deformation response for the buckling restrained braces of the retrofitted building in Ottawa analyzed under amplified Long Event #2 earthquake record: e) reduced section at the fourth storey; f) reduced section at the fifth storey; and g) reduced section at the six storey



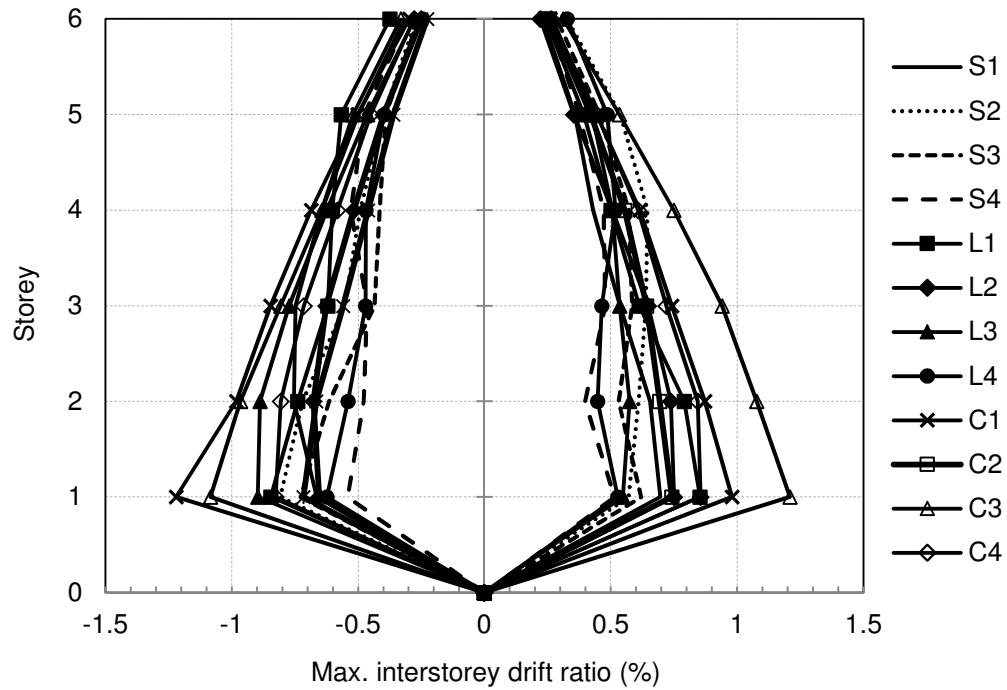


Figure 8.92: Maximum interstorey drift ratios for the retrofitted building in Vancouver (Configuration I), analyzed under earthquake records of short, long, and Cascadia events

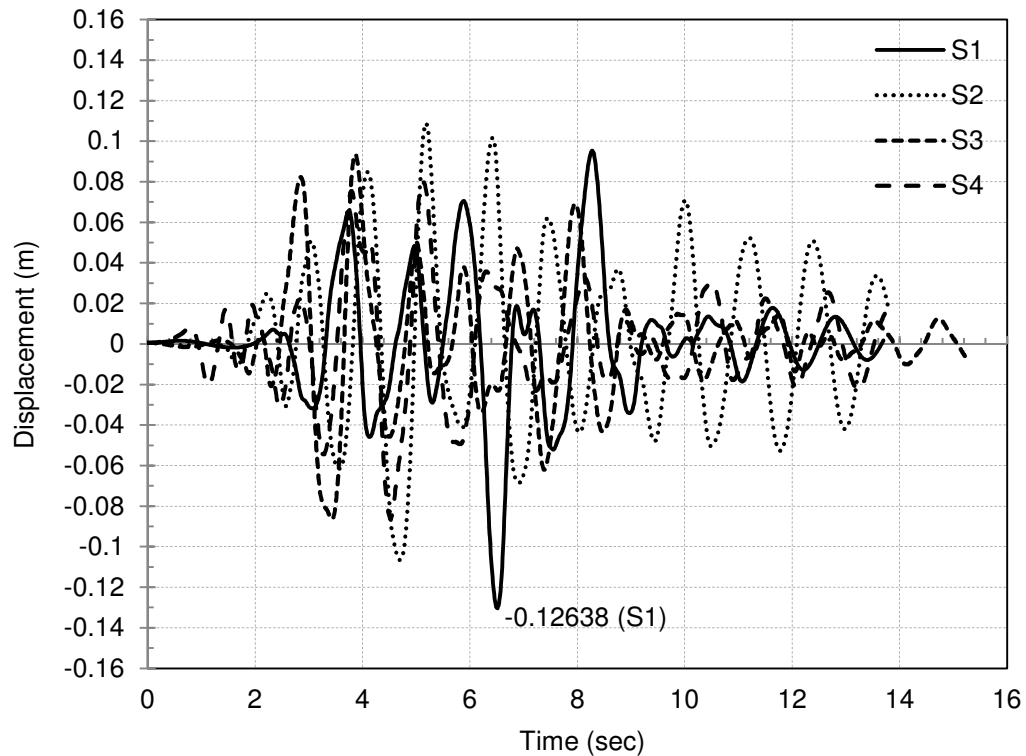


Figure 8.93: Top displacement time histories for the retrofitted building in Vancouver (Configuration I), analyzed under earthquake records of short events

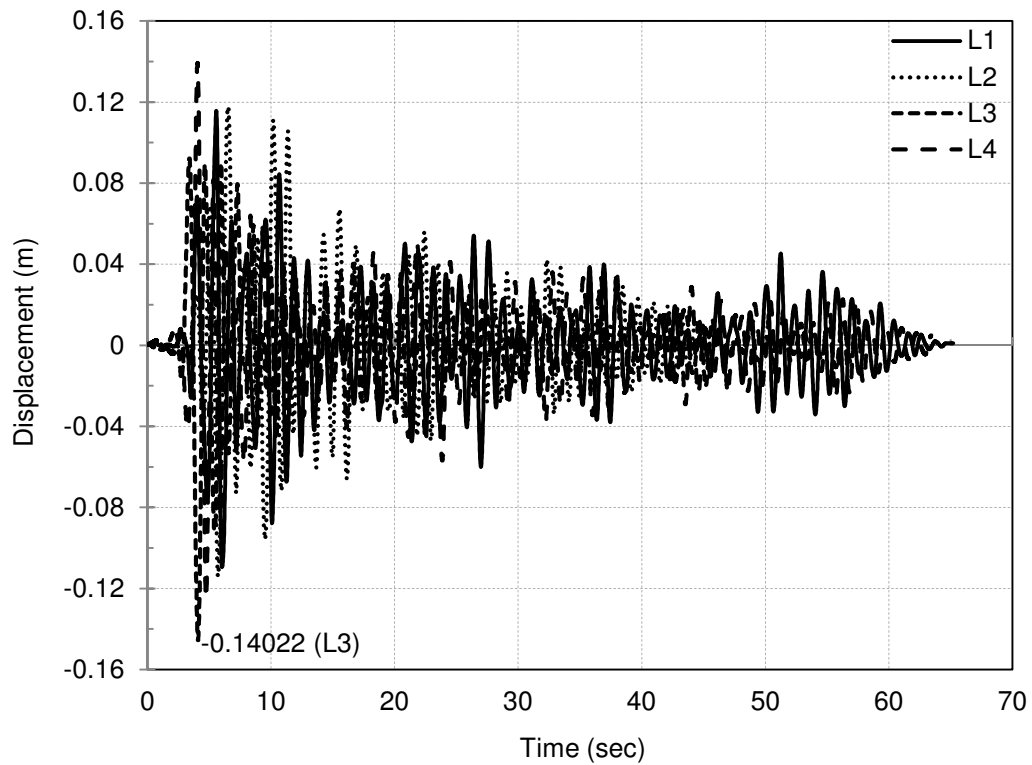


Figure 8.94: Top displacement time histories for the retrofitted building in Vancouver (Configuration I), analyzed under earthquake records of long events

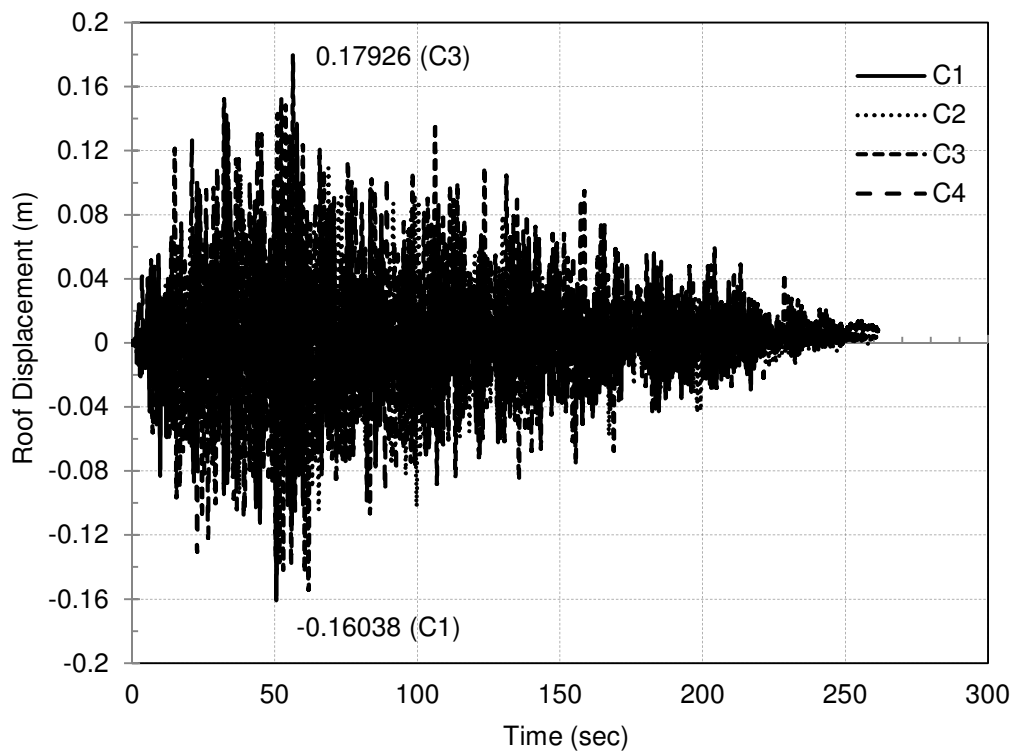


Figure 8.95: Top displacement time histories for the retrofitted building in Vancouver (Configuration I), analyzed under earthquake records of Cascadia events

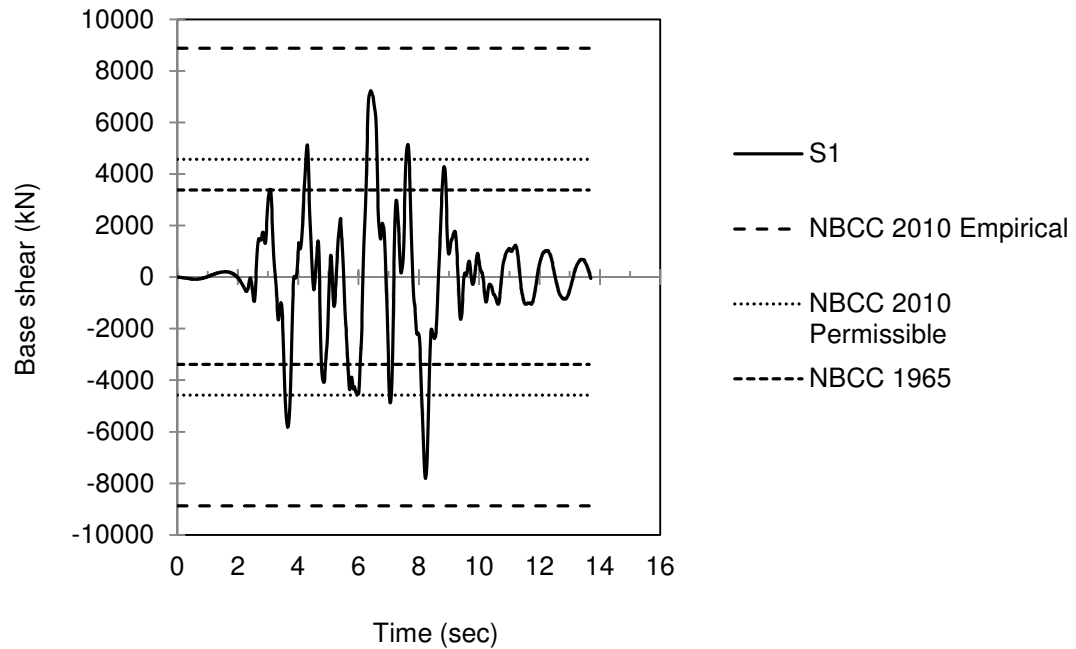


Figure 8.96: Comparison of dynamic base shear demands for Short Event #1 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

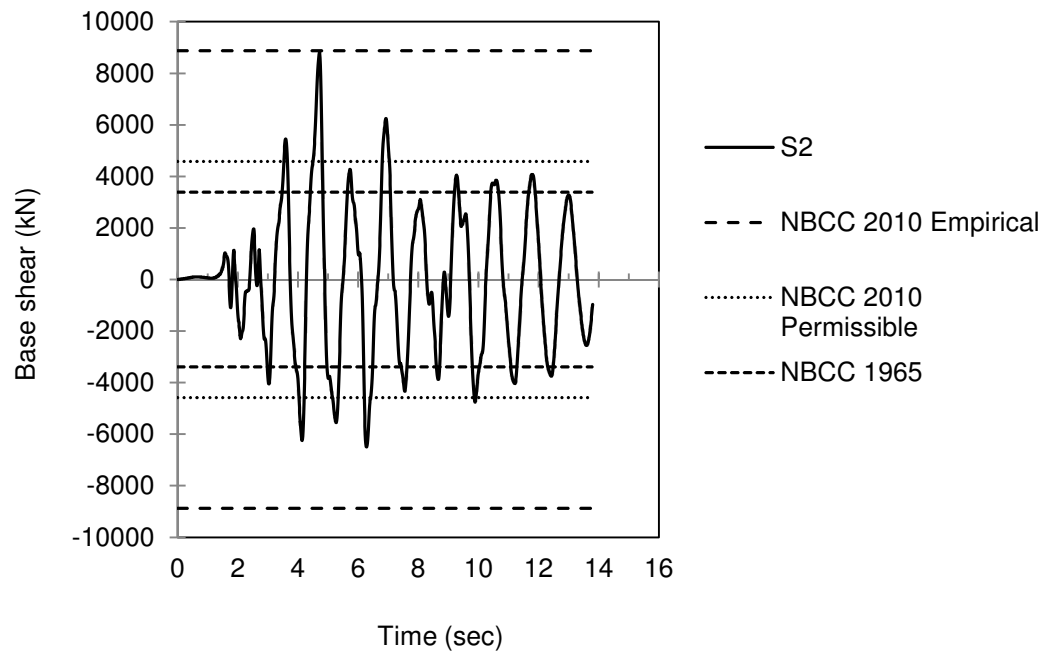


Figure 8.97: Comparison of dynamic base shear demands for Short Event #2 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

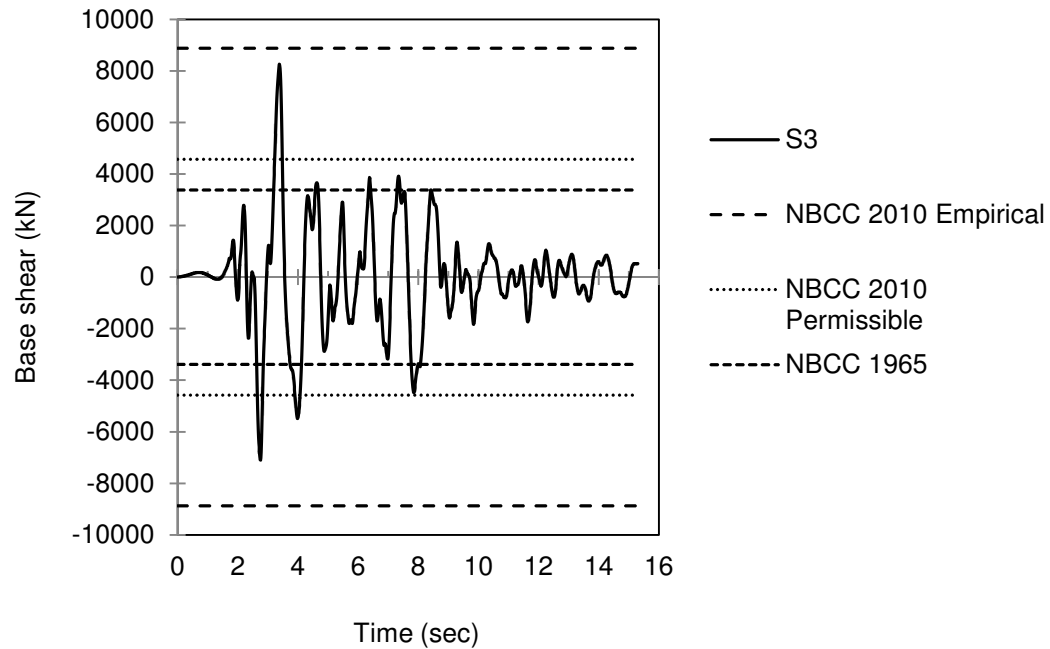


Figure 8.98: Comparison of dynamic base shear demands for Short Event #3 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

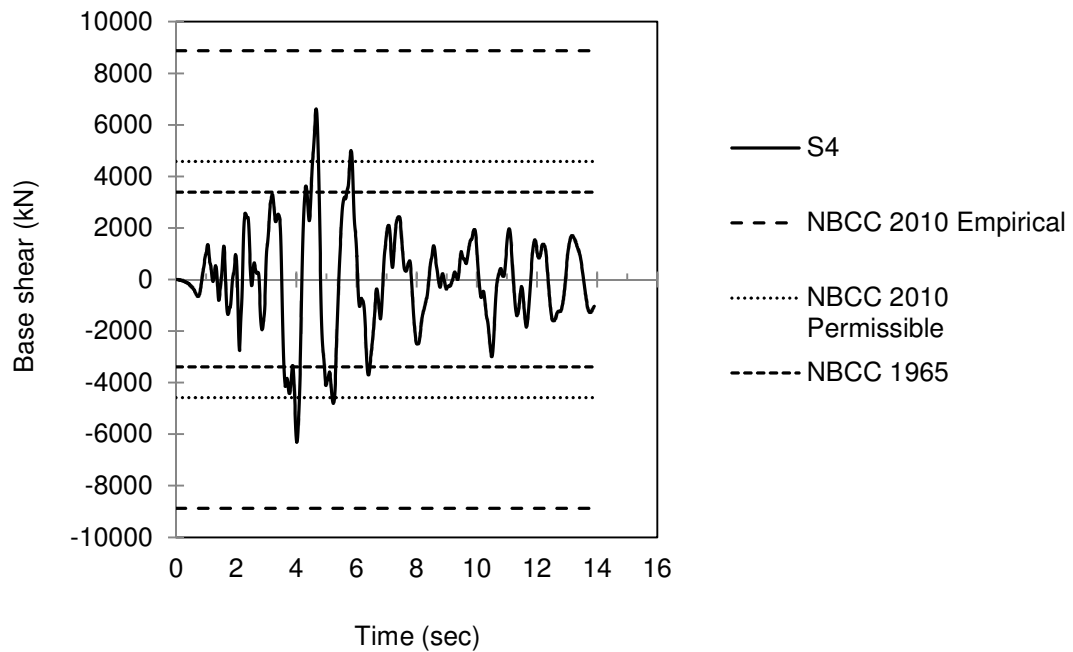


Figure 8.99: Comparison of dynamic base shear demands for Short Event #4 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

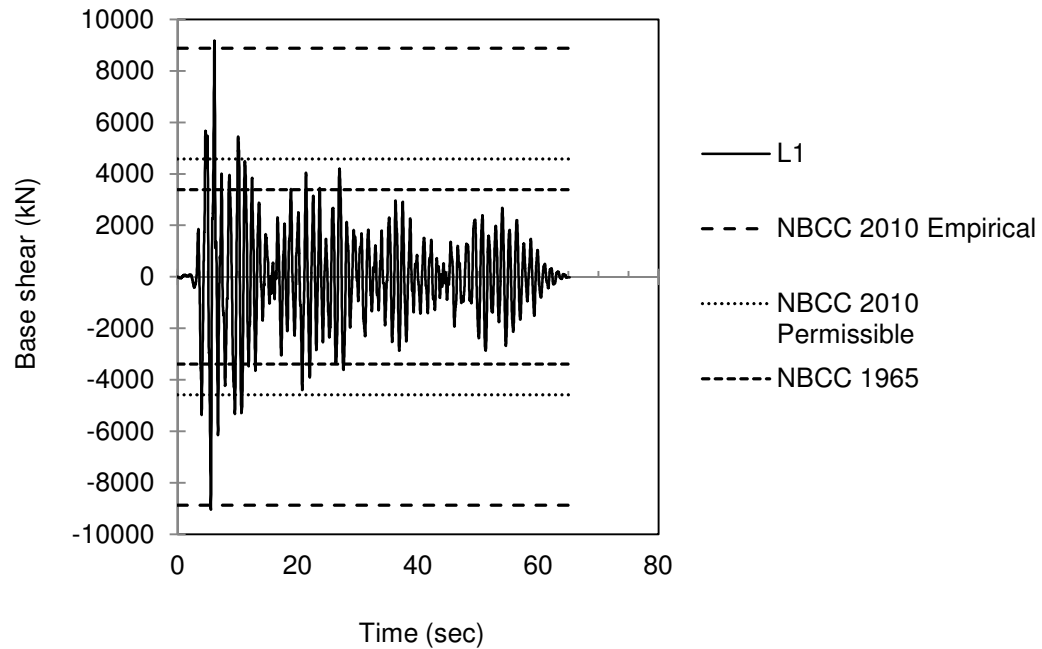


Figure 8.100: Comparison of dynamic base shear demands for Long Event #1 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

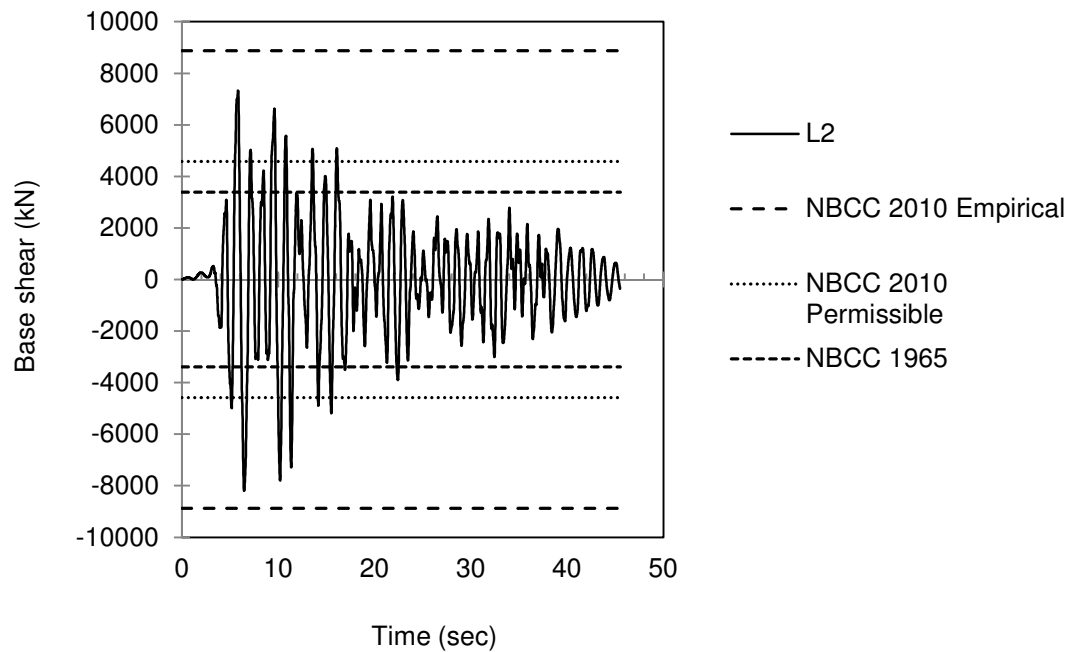


Figure 8.101: Comparison of dynamic base shear demands for Long Event #2 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

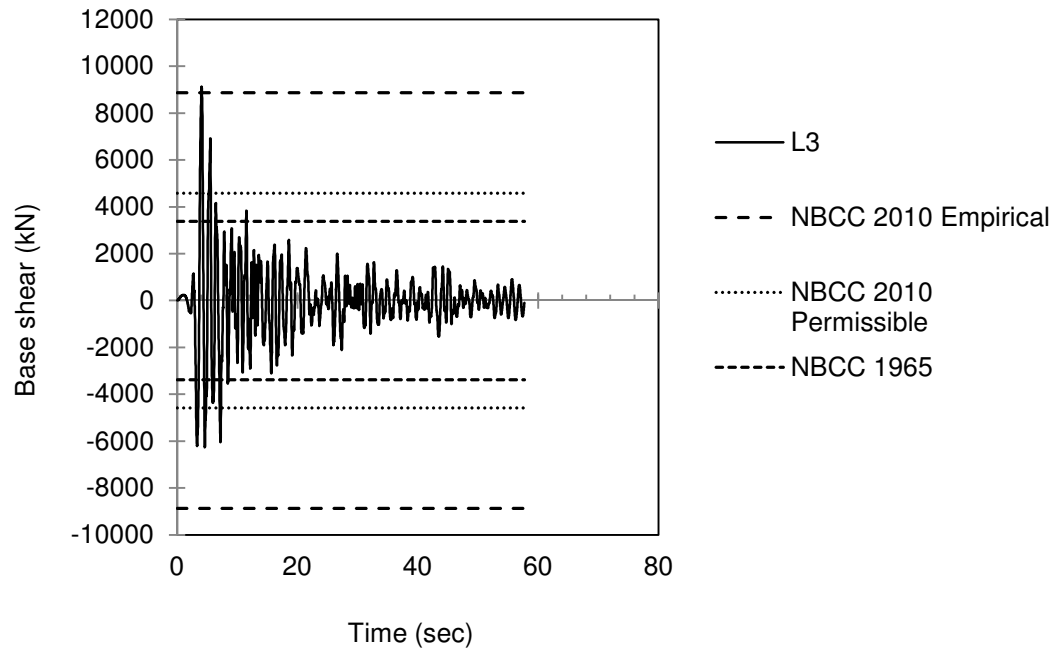


Figure 8.102: Comparison of dynamic base shear demands for Long Event #3 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

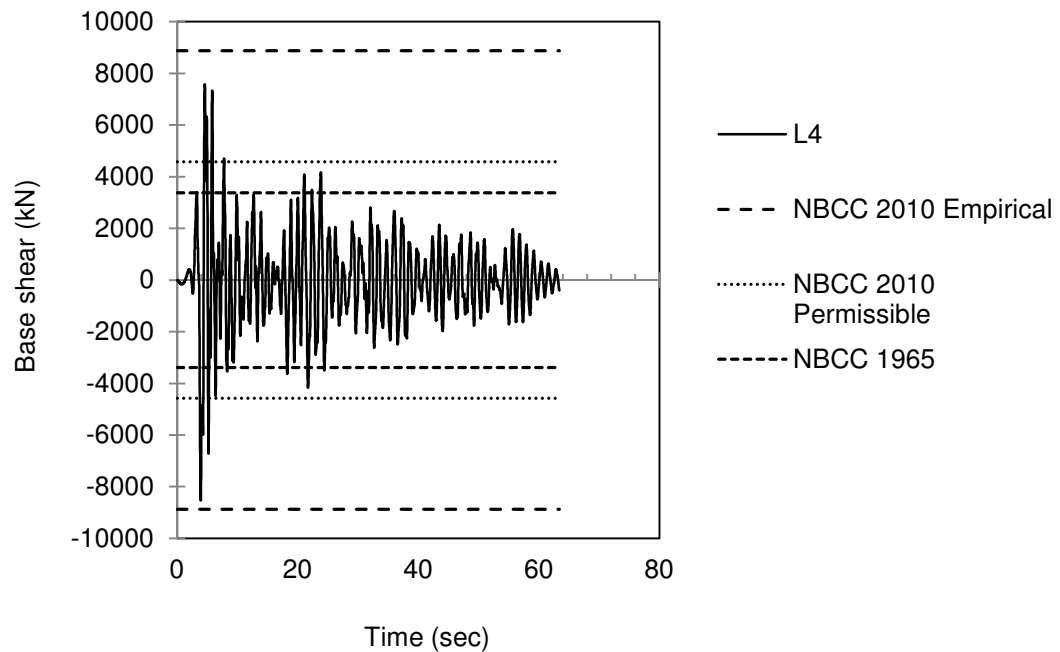


Figure 8.103: Comparison of dynamic base shear demands for Long Event #4 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)



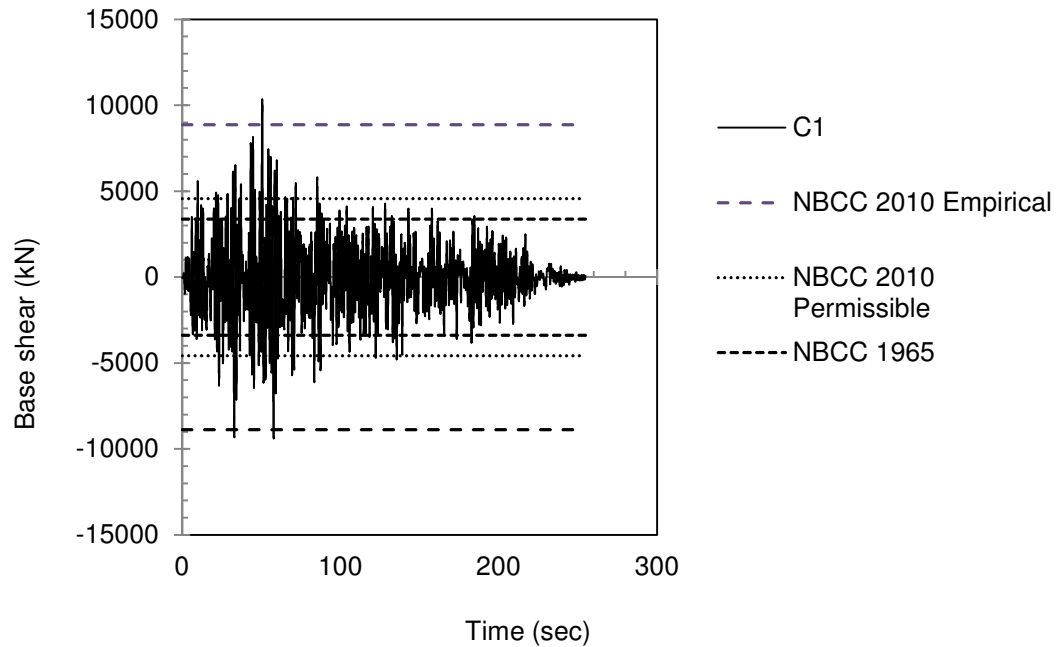


Figure 8.104: Comparison of dynamic base shear demands for Cascadia Event #1 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

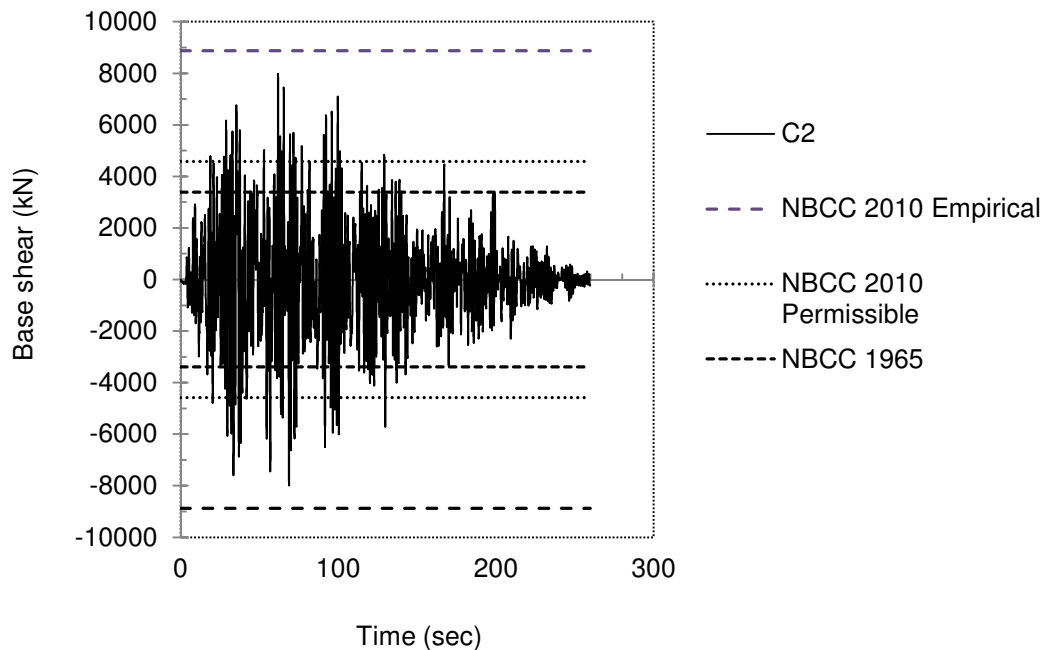


Figure 8.105: Comparison of dynamic base shear demands for Cascadia Event #2 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

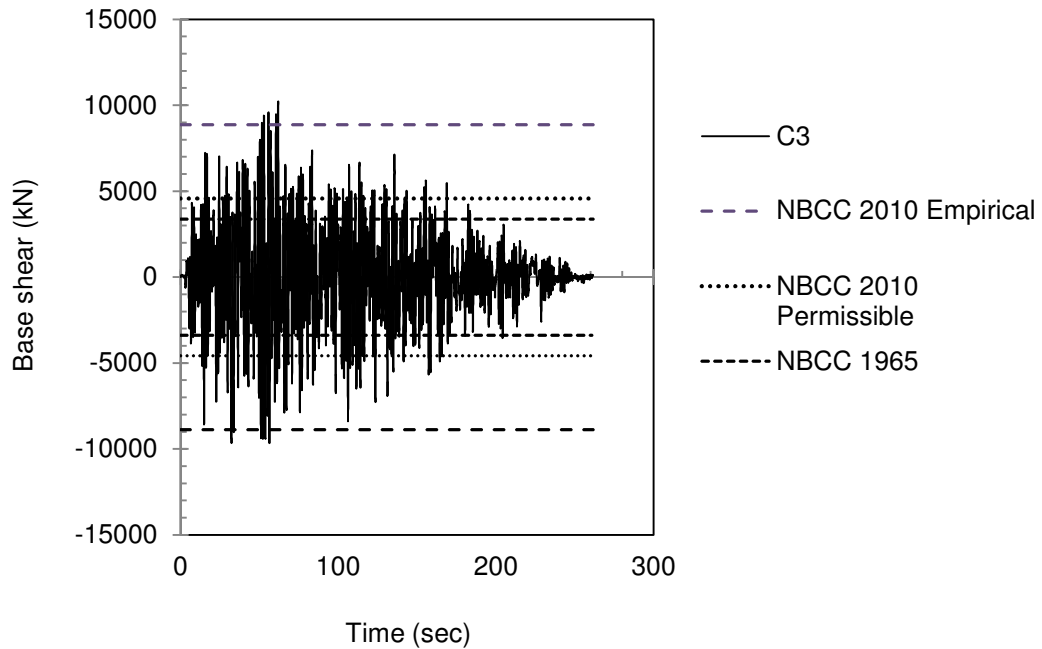


Figure 8.106: Comparison of dynamic base shear demands for Cascadia Event #3 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

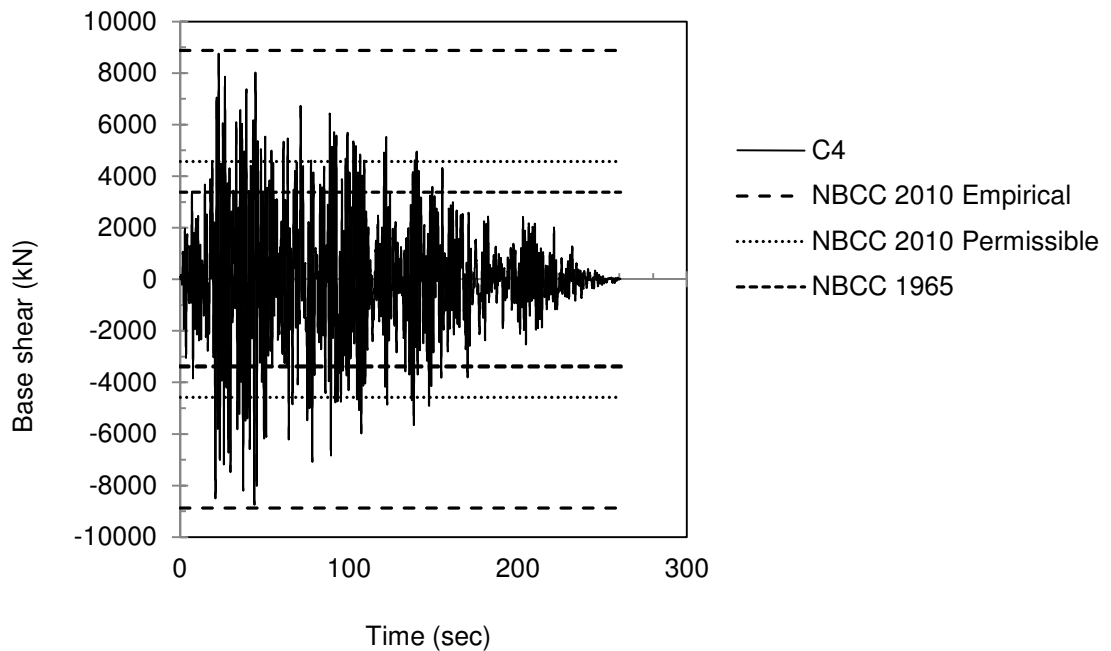


Figure 8.107: Comparison of dynamic base shear demands for Cascadia Event #4 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration I)

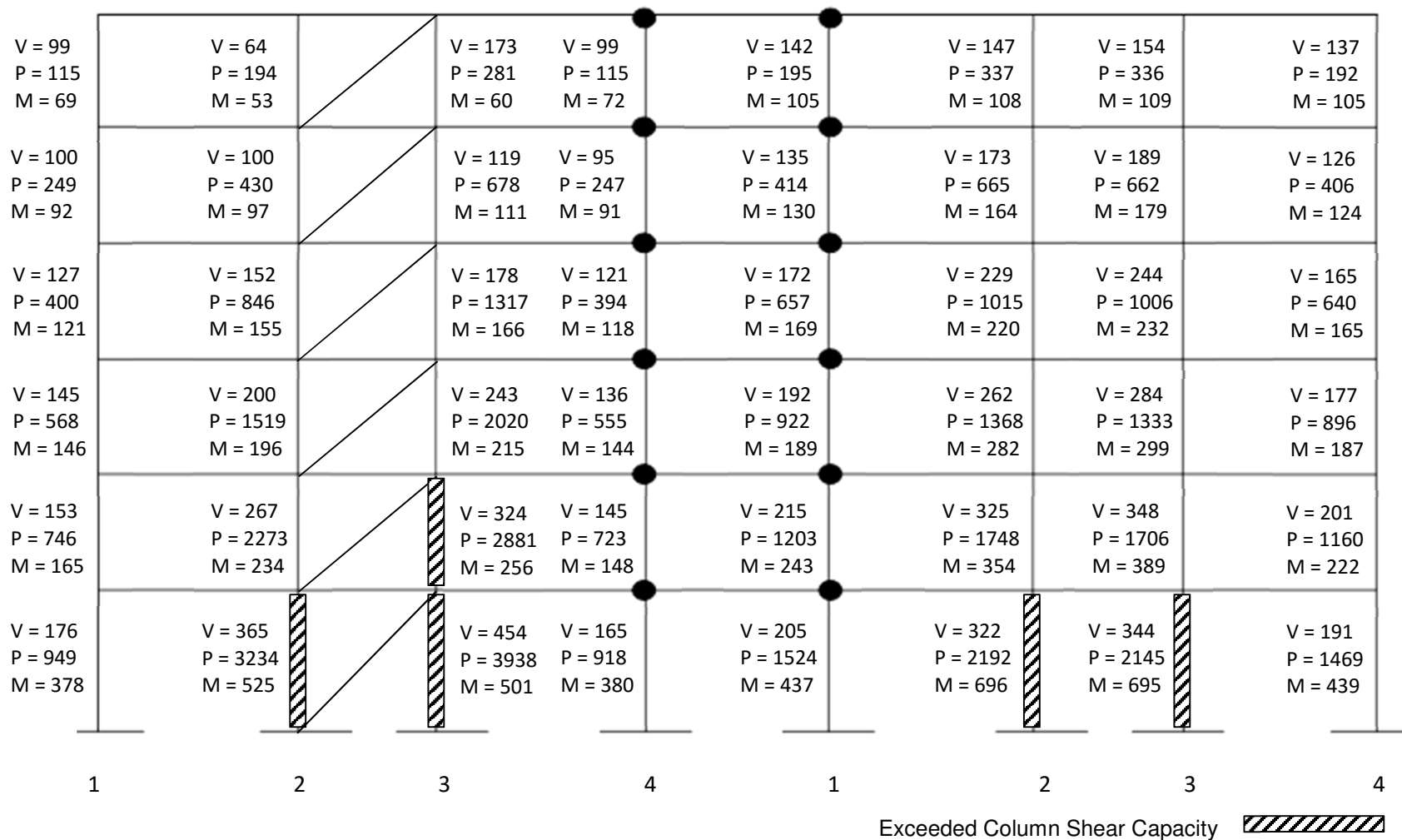
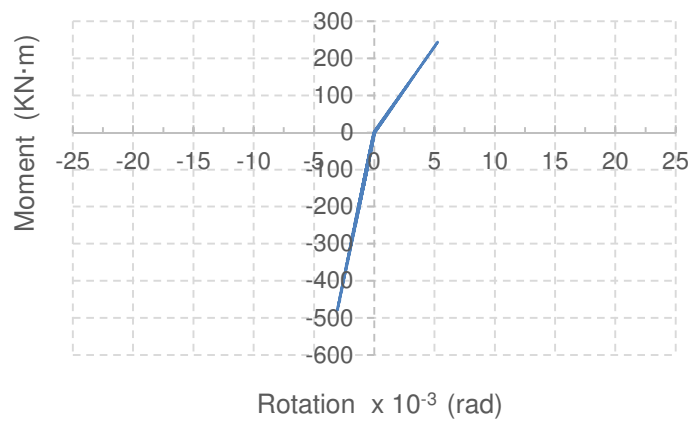
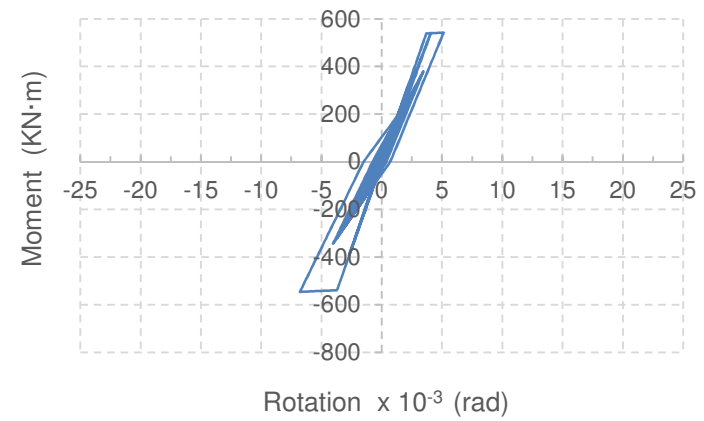


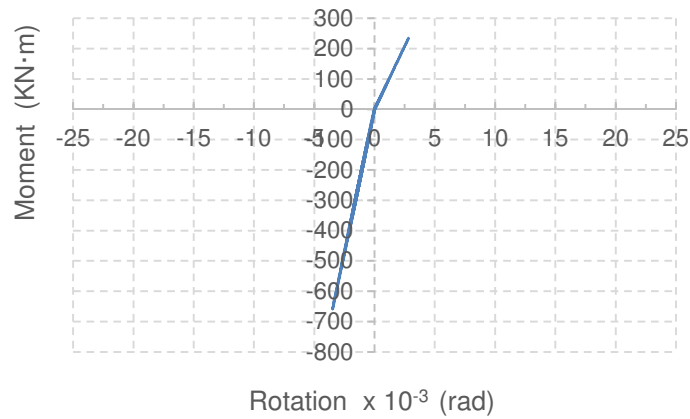
Figure 8.108: Column shear force, axial force, and moment demands for the retrofitted building in Vancouver (Configuration I), analyzed under Cascadia Event #1



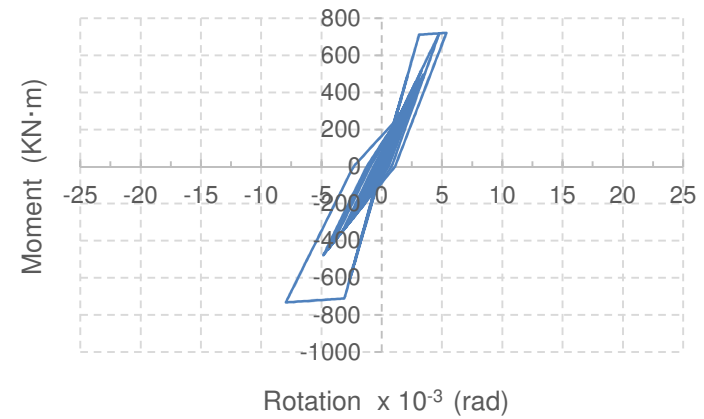
(a)



(b)

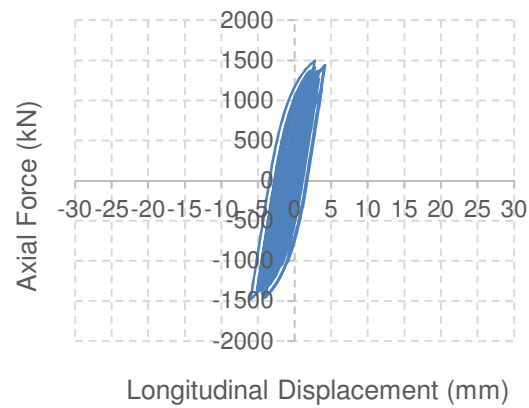


(c)

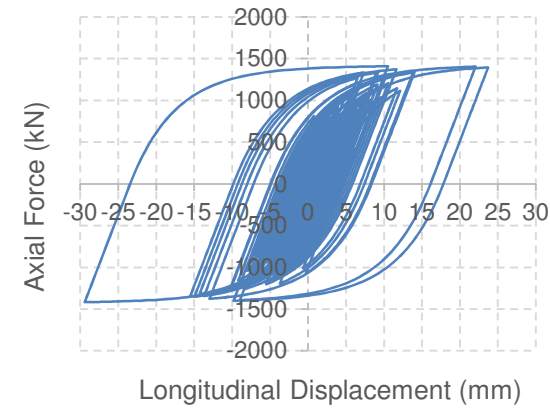


(d)

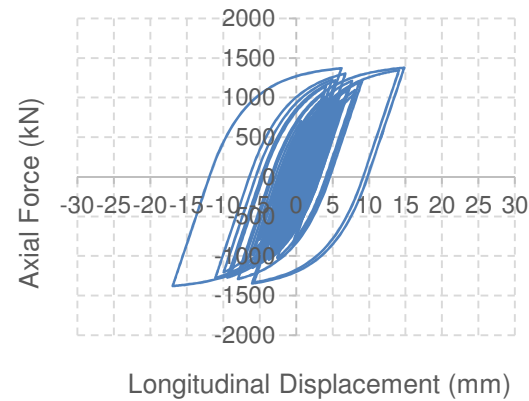
Figure 8.109: Moment-rotation hysteretic relationships for selected beams and columns of the retrofitted building in Vancouver, (Configuration I), analyzed under Cascadia Event #1: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames



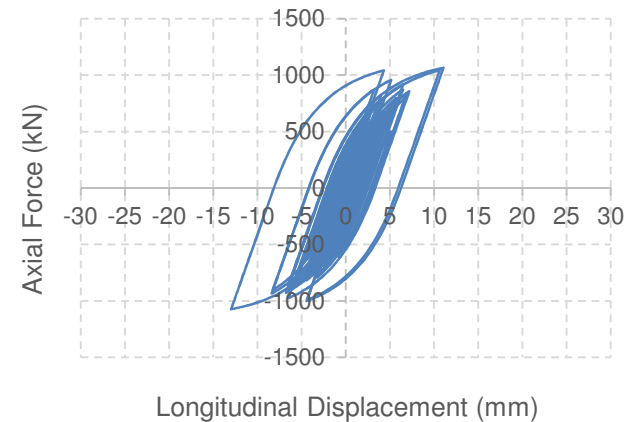
(a)



(b)

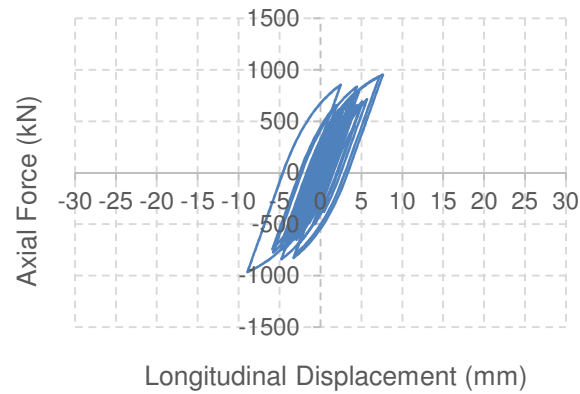


(c)

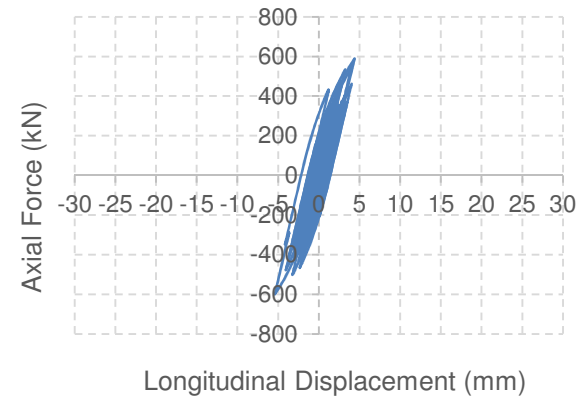


(d)

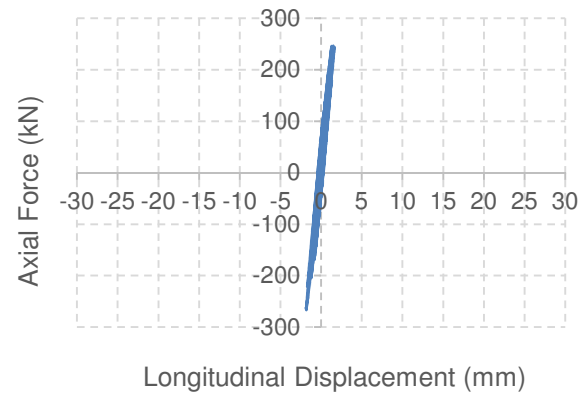
Figure 8.110: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the retrofitted building in Vancouver (Configuration I), analyzed under Cascadia Event #1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)



(f)



(g)

Figure 8.110 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the retrofitted building in Vancouver (Configuration I), analyzed under Cascadia Event #1: e) reduced section at the fourth storey; f) reduced section at the fifth storey; and g) reduced section at the six storey



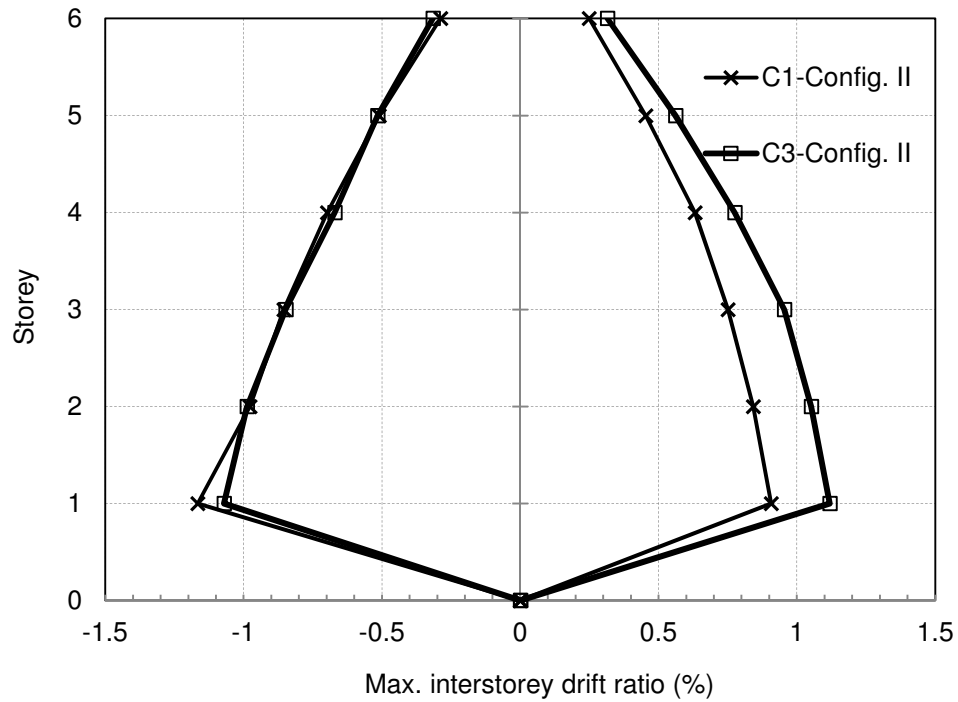


Figure 8.111: Maximum interstorey drift ratios for the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Events #1 and #3

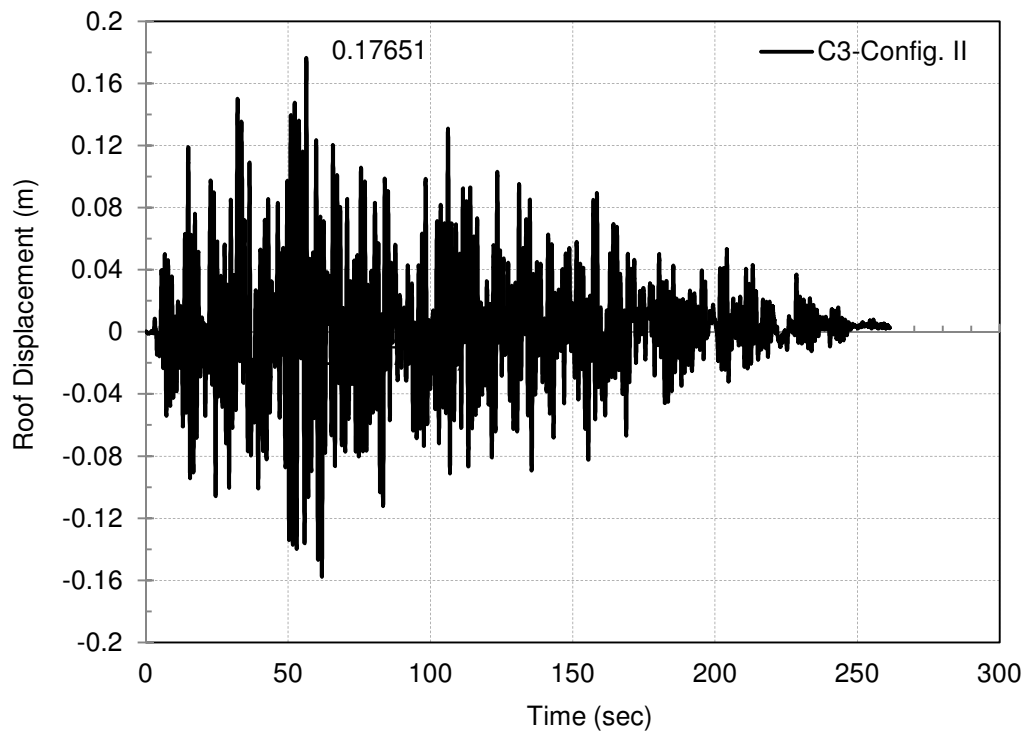


Figure 8.112: Top displacement time history for the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Event #3

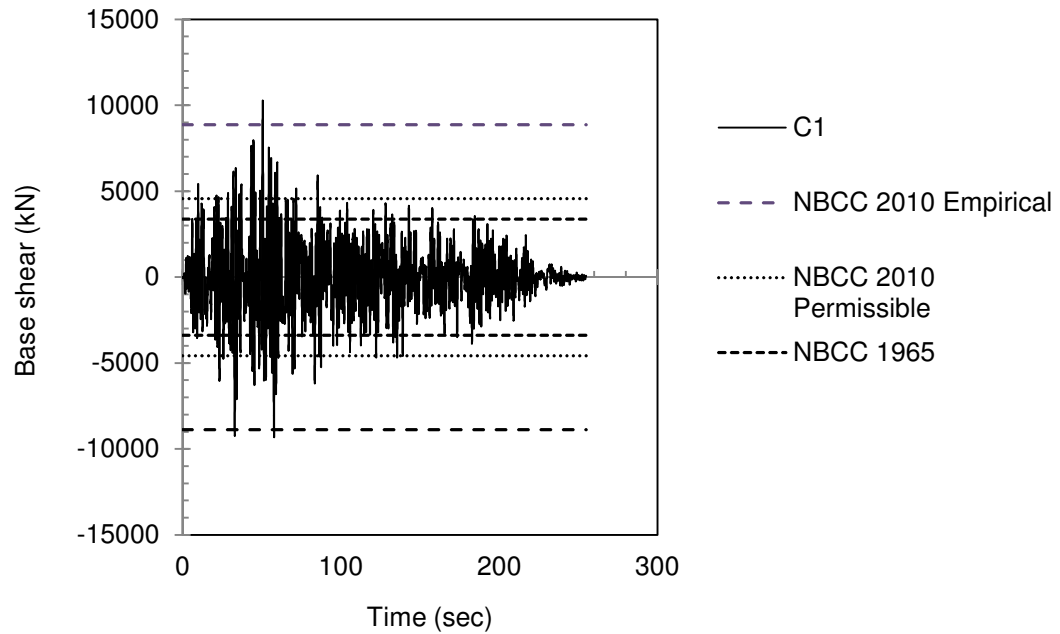


Figure 8.113: Comparison of dynamic base shear demands for Cascadia Event #1 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration II)

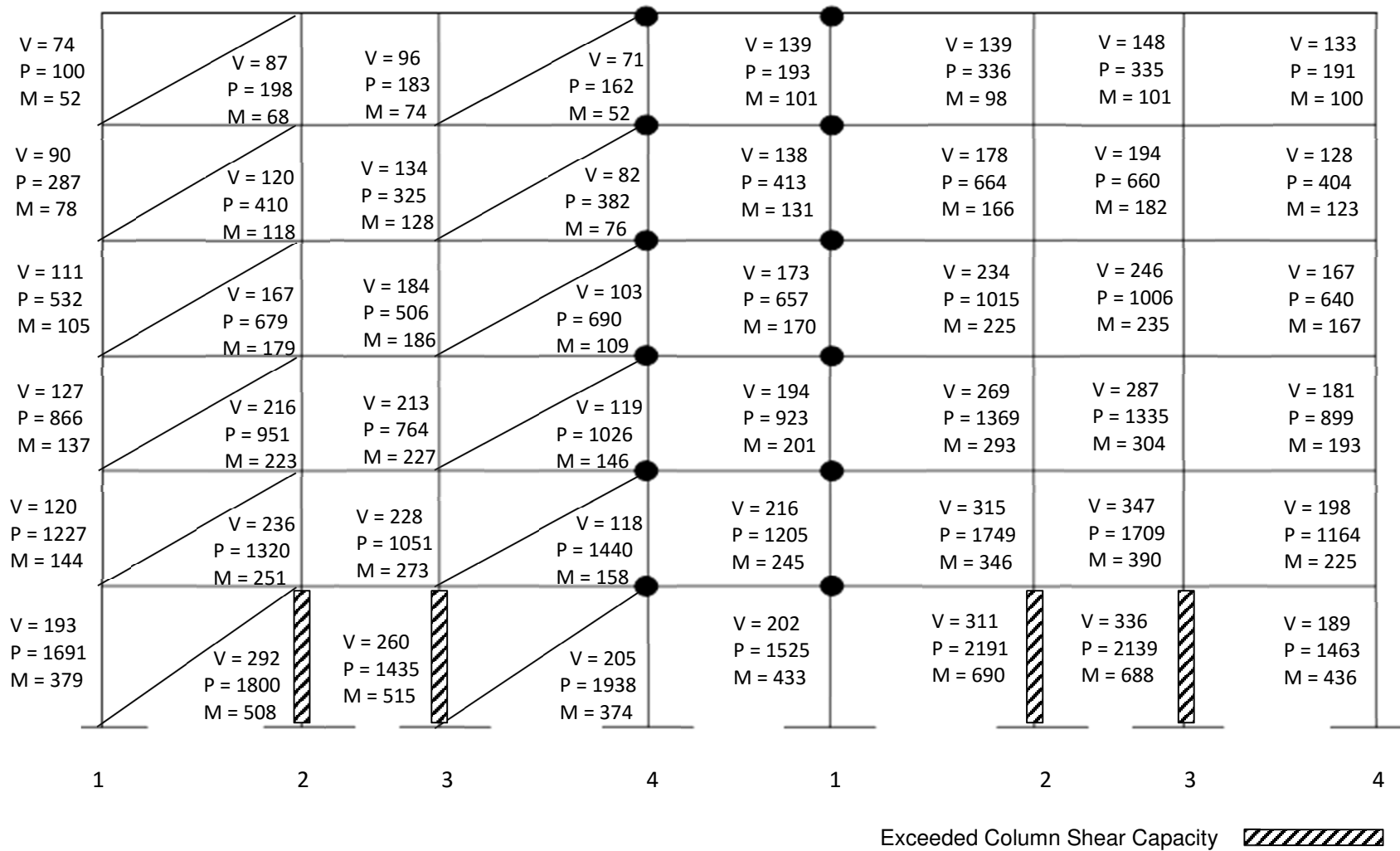
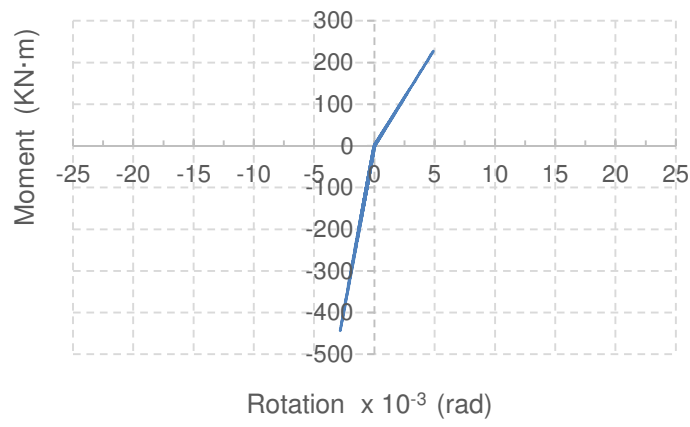
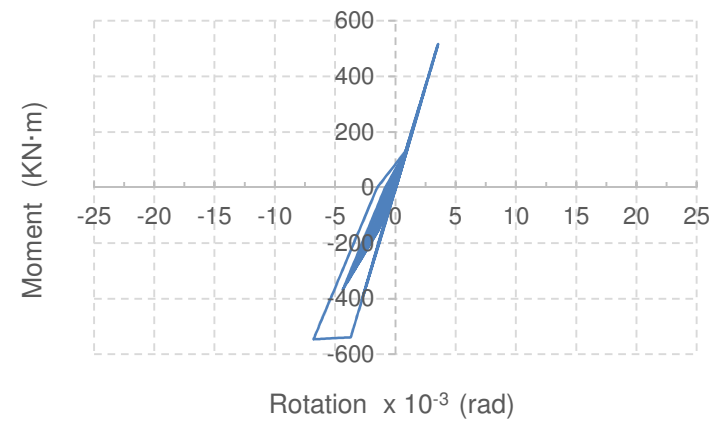


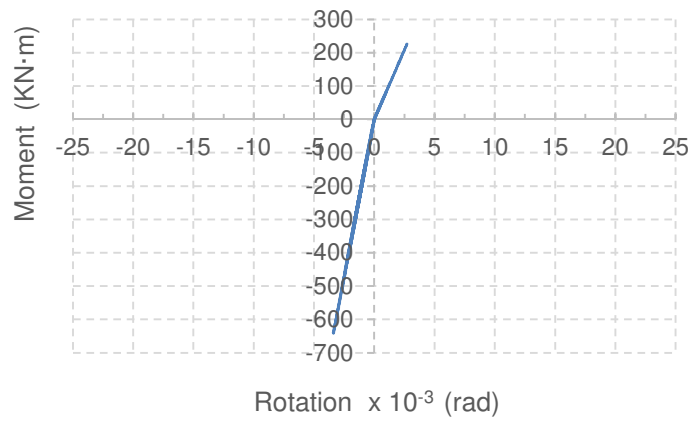
Figure 8.114: Column shear force, axial force, and moment demands for the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Event #1



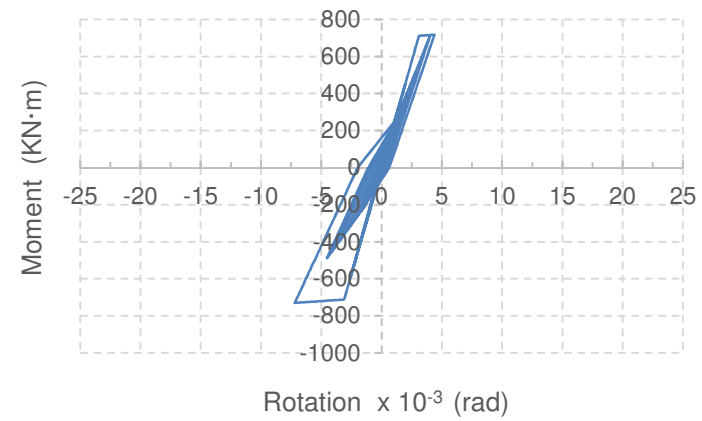
(a)



(b)

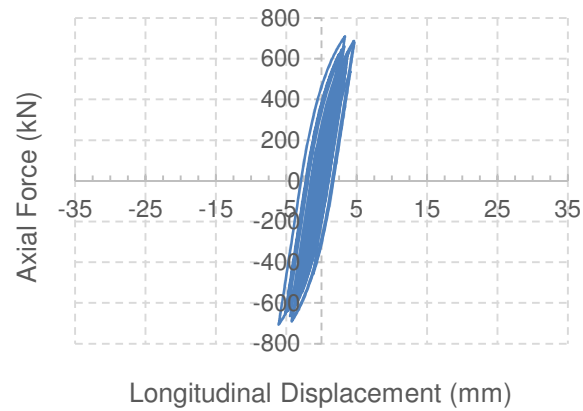


(c)

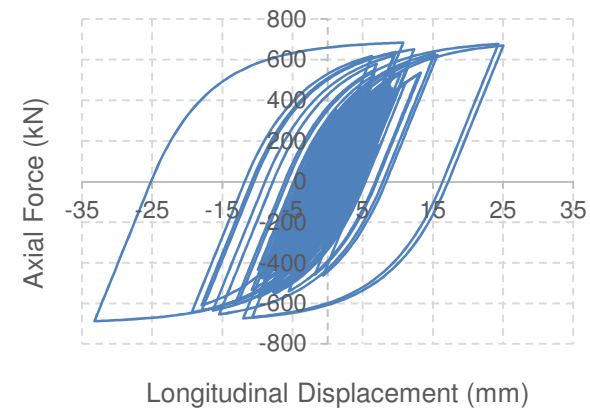


(d)

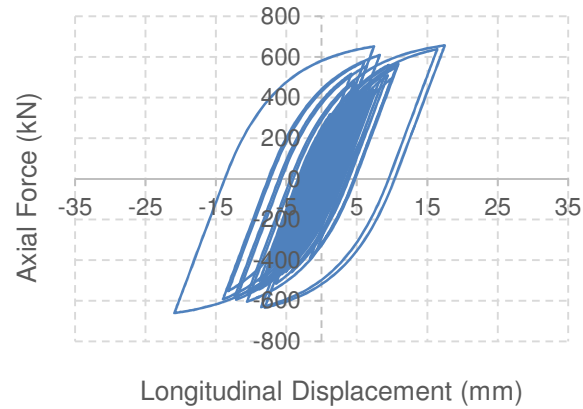
Figure 8.115: Moment-chord rotation hysteretic relationships for selected beams and columns of the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Event 1: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames



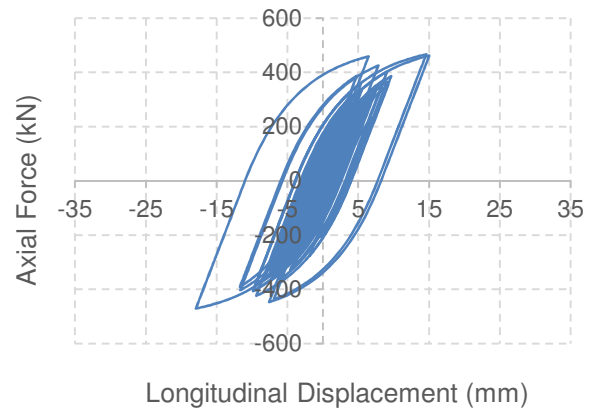
(a)



(b)

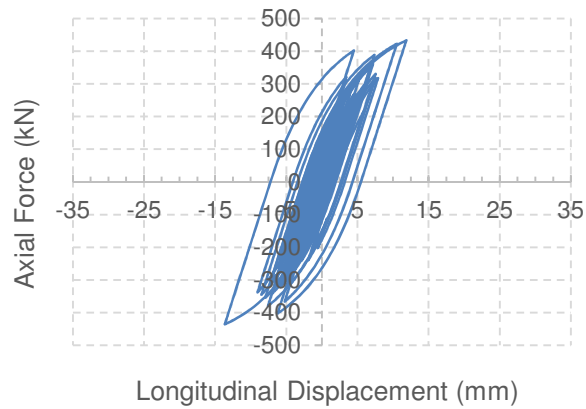


(c)

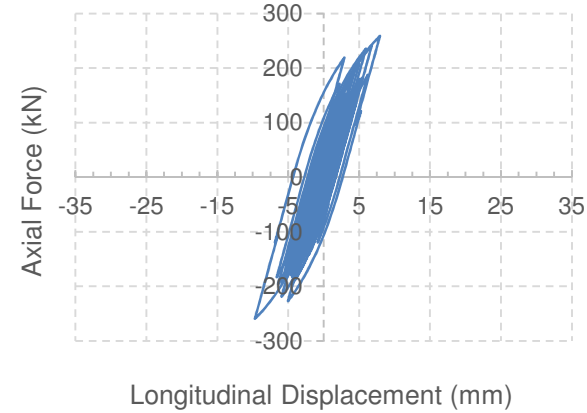


(d)

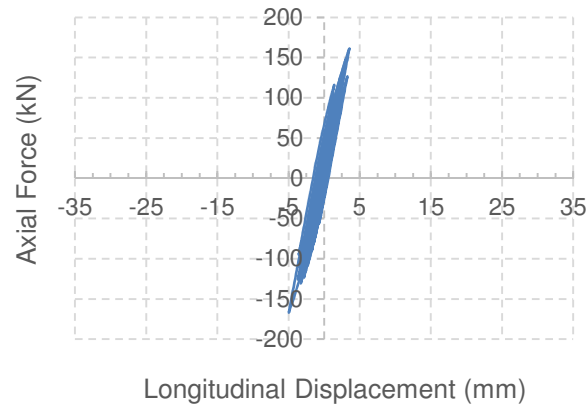
Figure 8.116: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the left bay of the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Event 1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)



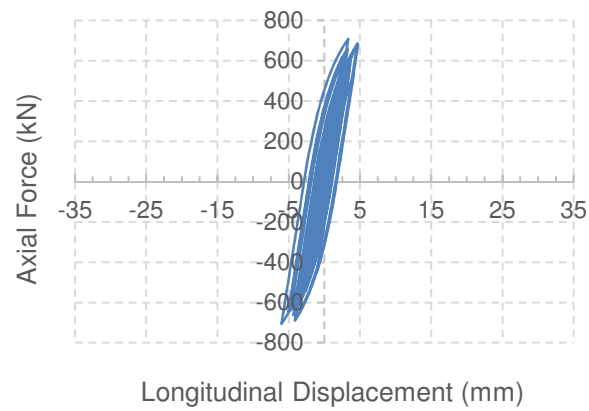
(f)



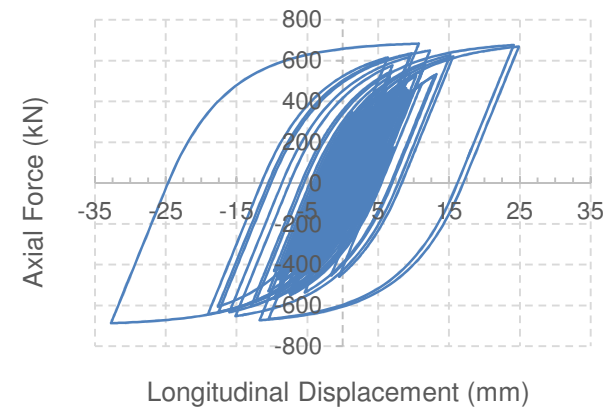
(g)

Figure 8.116 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the left bay of the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Event 1: e) reduced section of fourth storey; f) reduced section of fifth storey; and g) reduced section of six storey

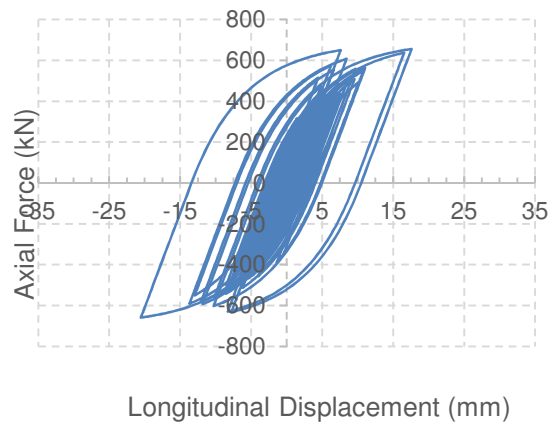




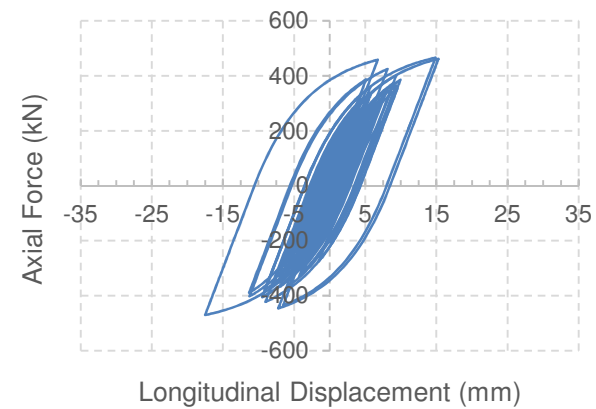
(a)



(b)

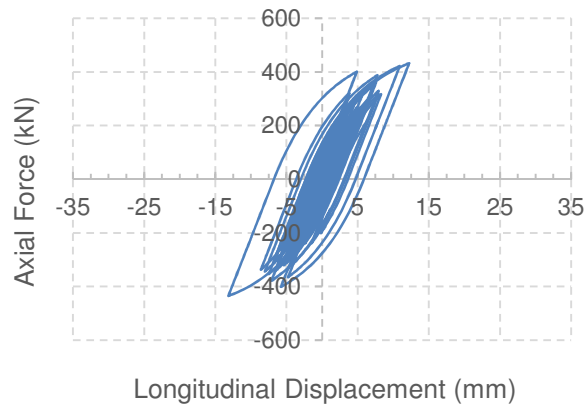


(c)

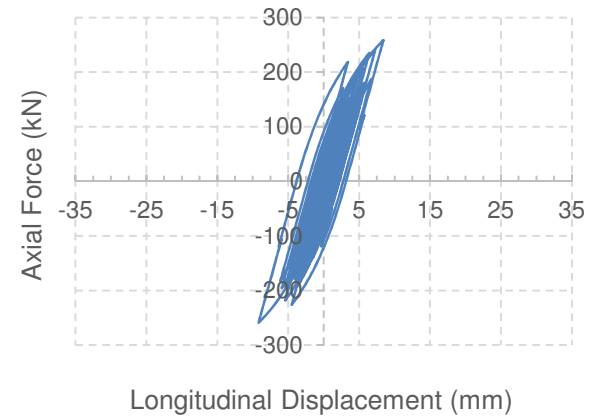


(d)

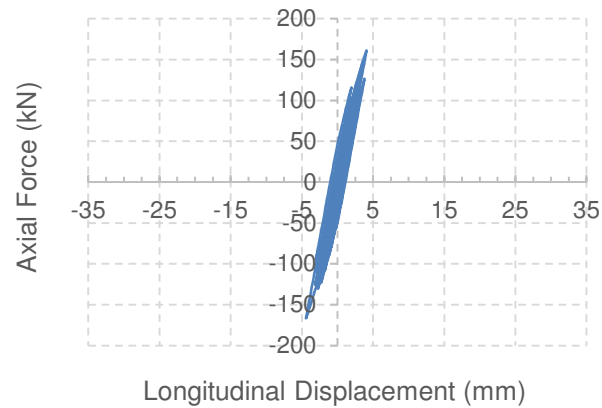
Figure 8.117: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the right bay of the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Event 1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)



(f)



(g)

Figure 8.117 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the right bay of the retrofitted building in Vancouver (Configuration II), analyzed under Cascadia Event 1: e) reduced section of fourth storey; f) reduced section of fifth storey; and g) reduced section of sixth storey

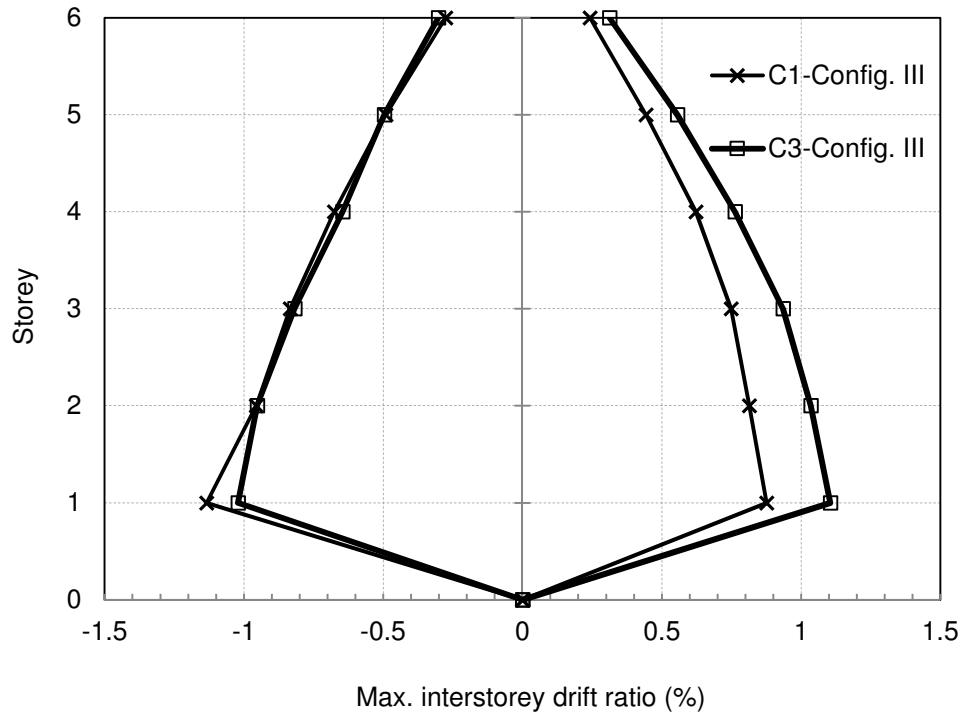


Figure 8.118: Maximum interstorey drift ratios for the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Events #1 and #3

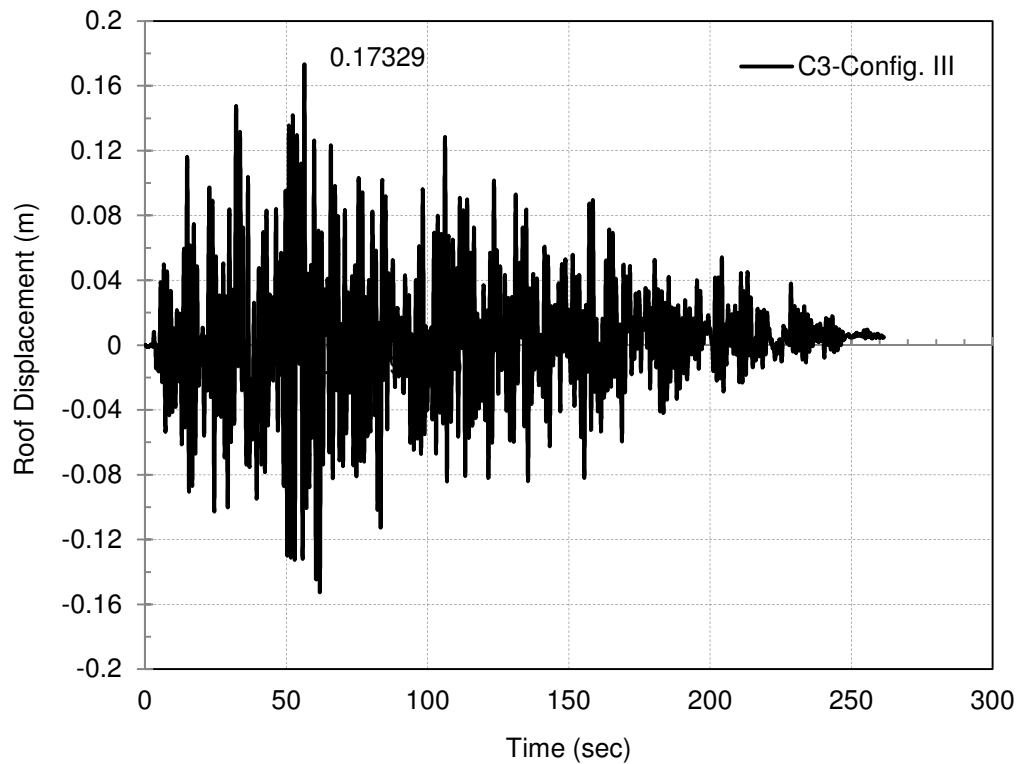


Figure 8.119: Top displacement time history for the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Event #3

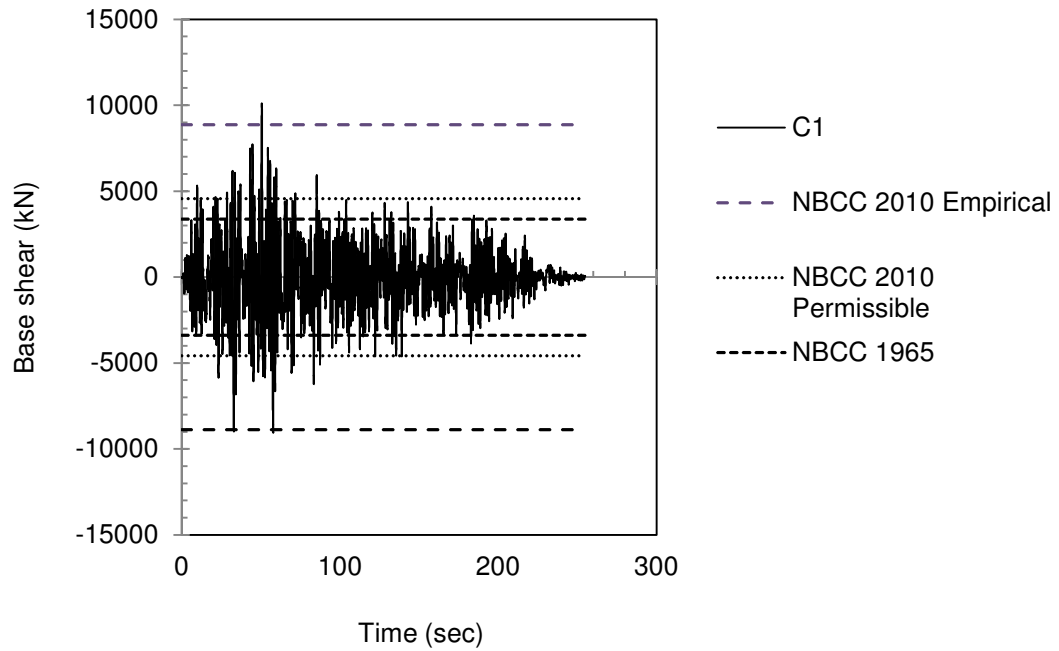


Figure 8.120: Comparison of dynamic base shear demands for Cascadia Event #1 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration III)

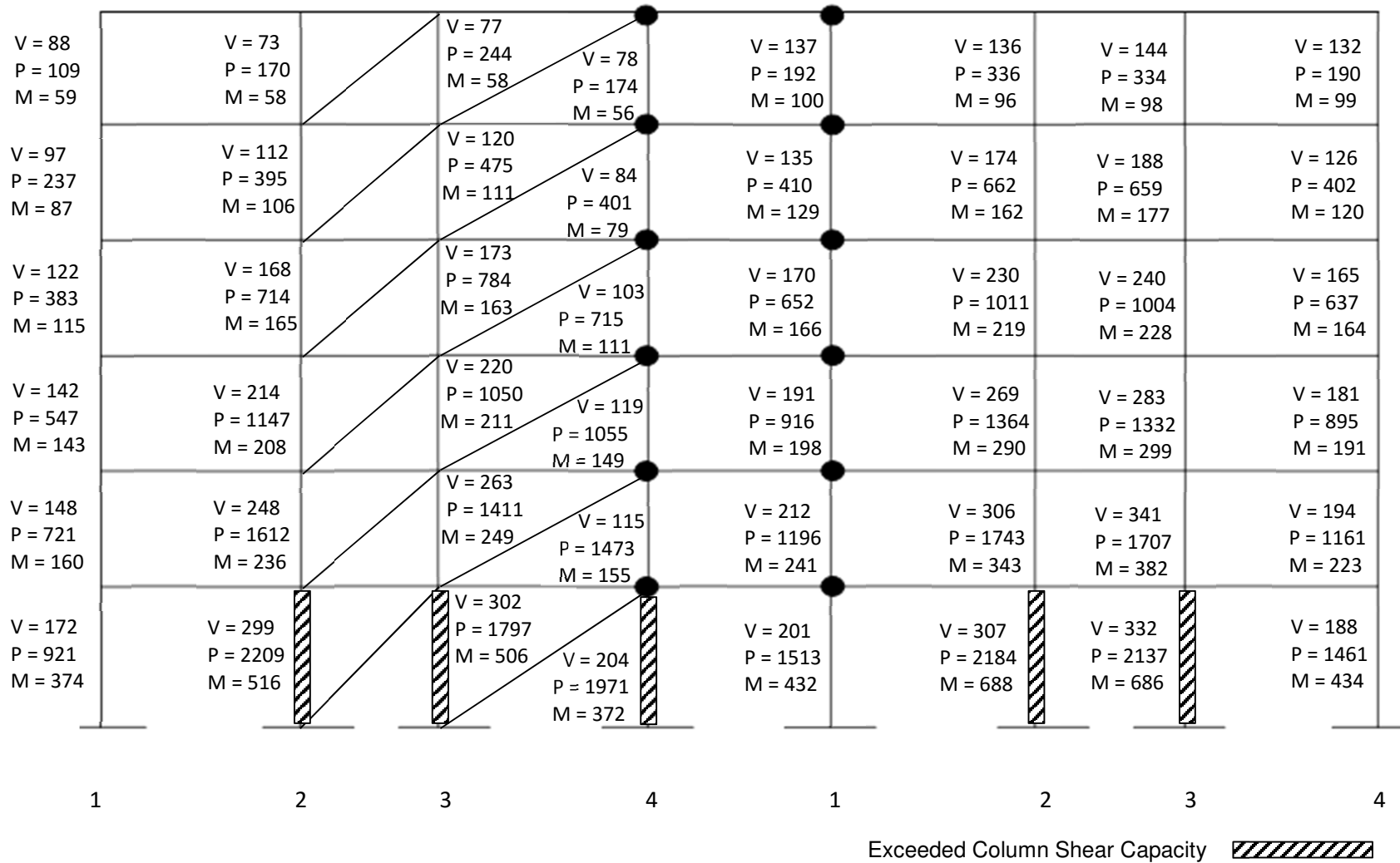
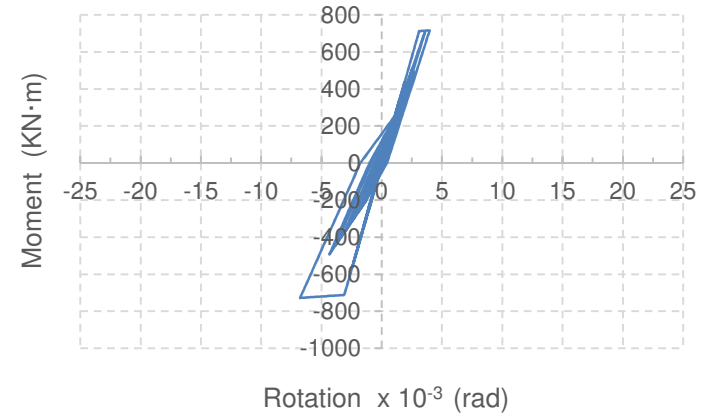
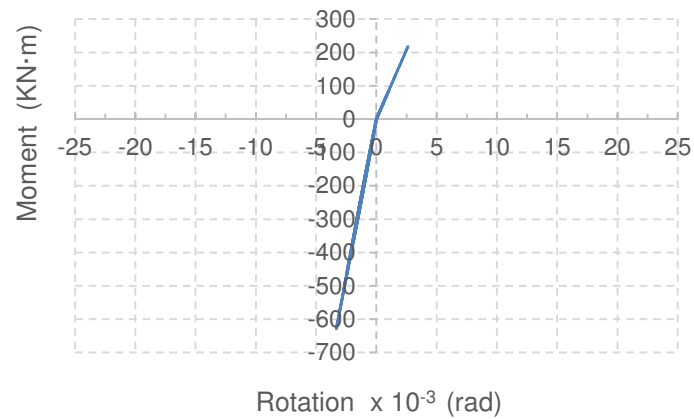
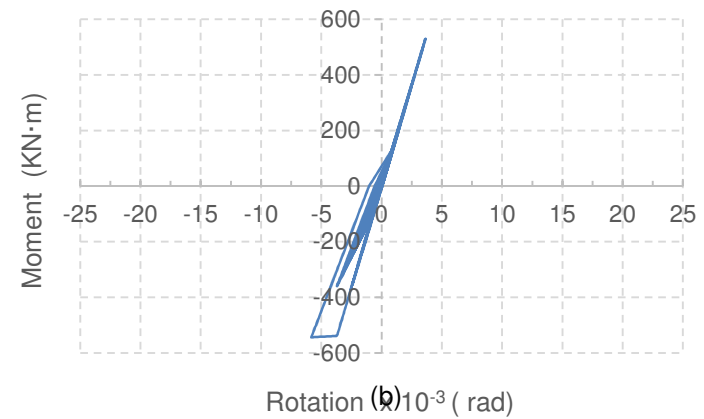
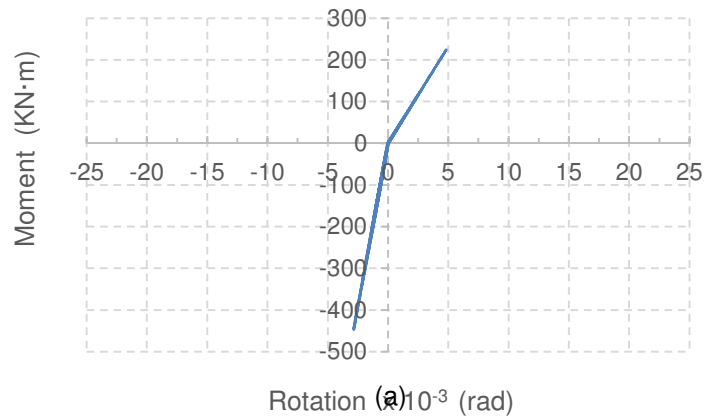


Figure 8.121: Column shear force, axial force, and moment demands for the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Event #1

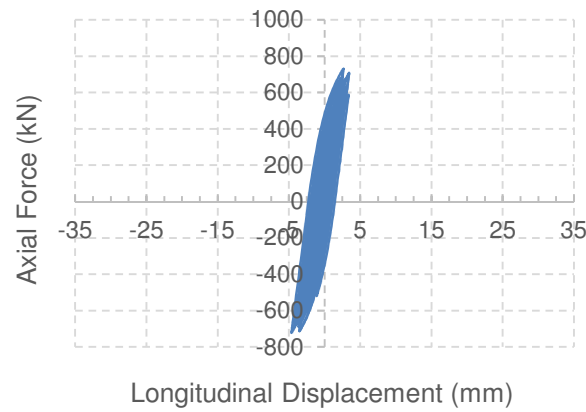


(c)

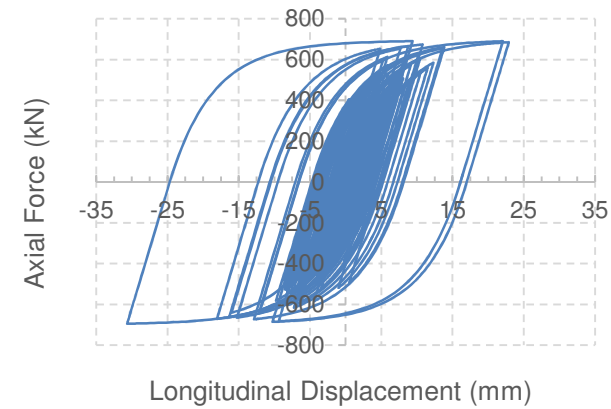
(d)

Figure 8.122 : Moment-chord rotation hysteretic relationships for selected beams and columns of the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Event 1: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames

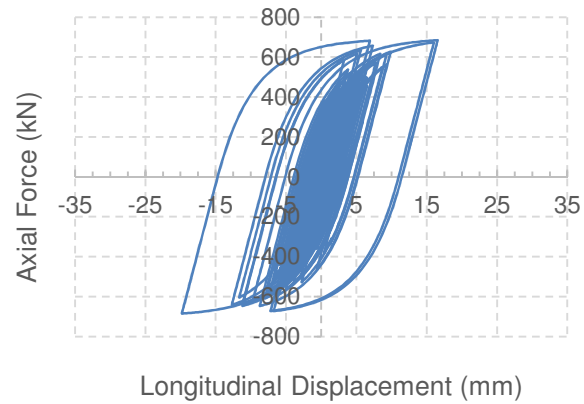




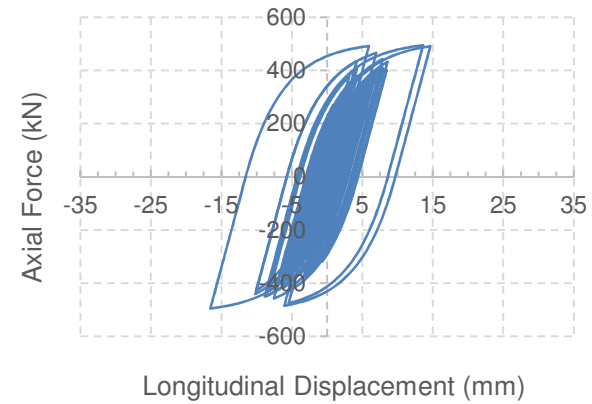
(a)



(b)

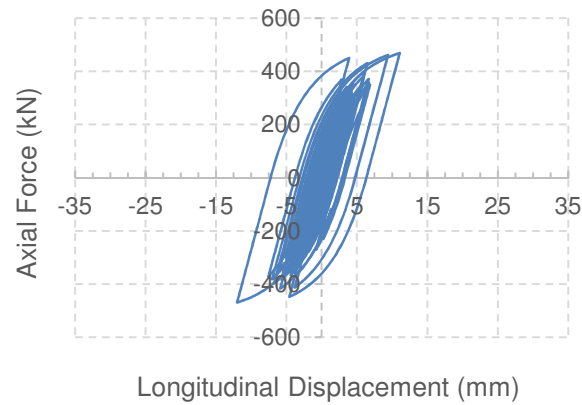


(c)

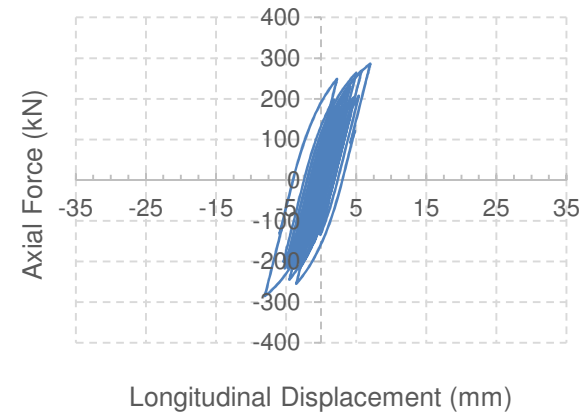


(d)

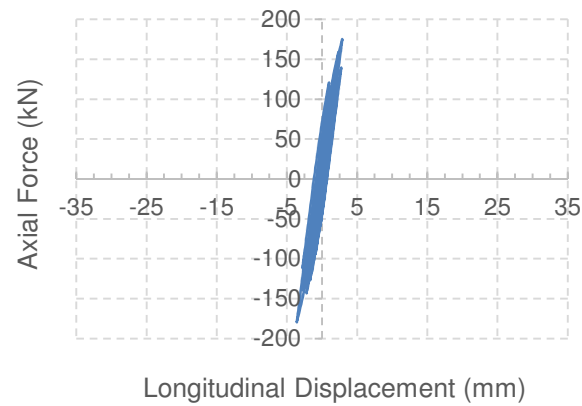
Figure 8.123: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the central bay of the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Event 1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)

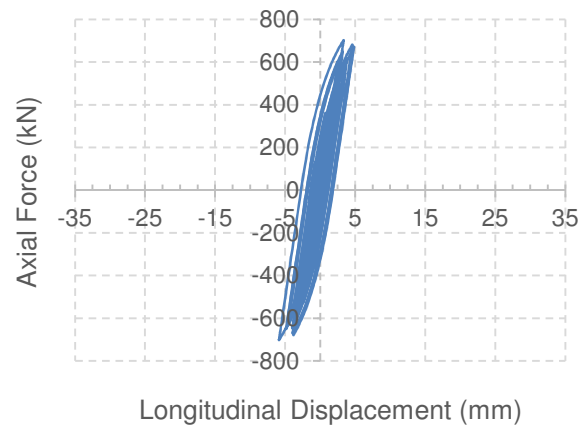


(f)

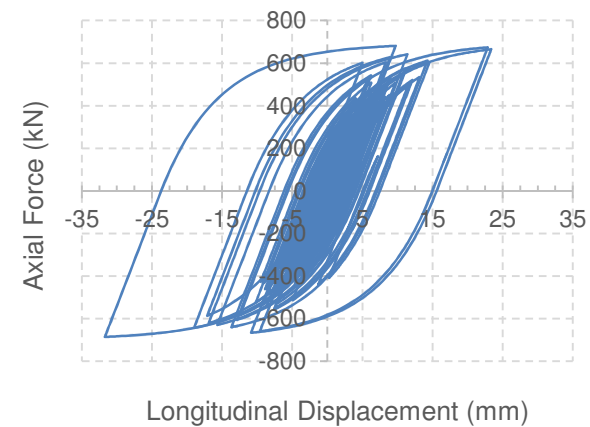


(g)

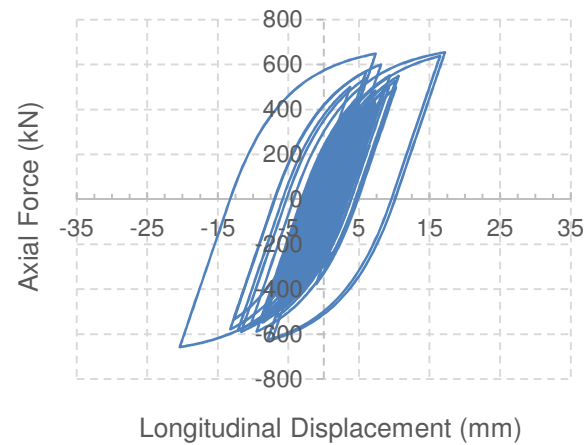
Figure 8.123 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the central bay of the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Event 1: e) reduced section of fourth storey; f) reduced section of fifth storey; and g) reduced section of sixth storey



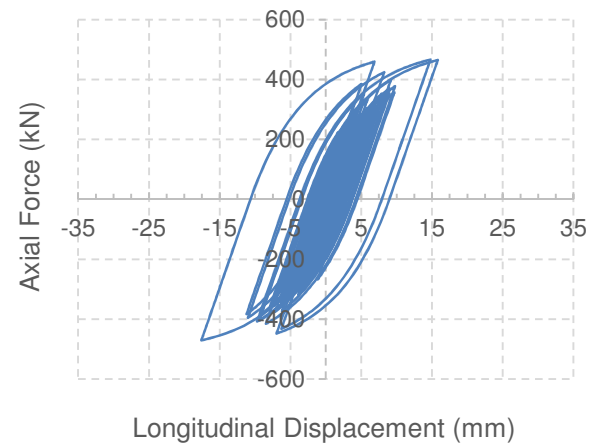
(a)



(b)

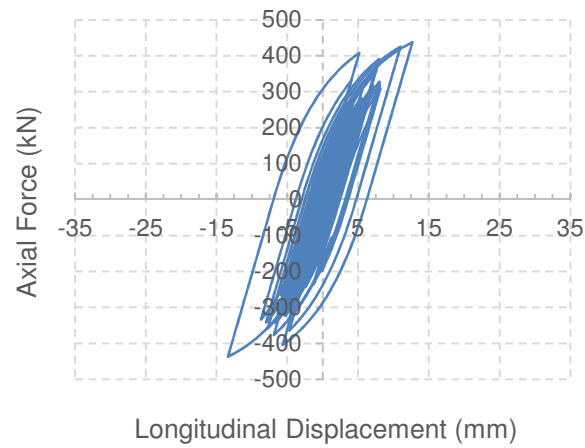


(c)

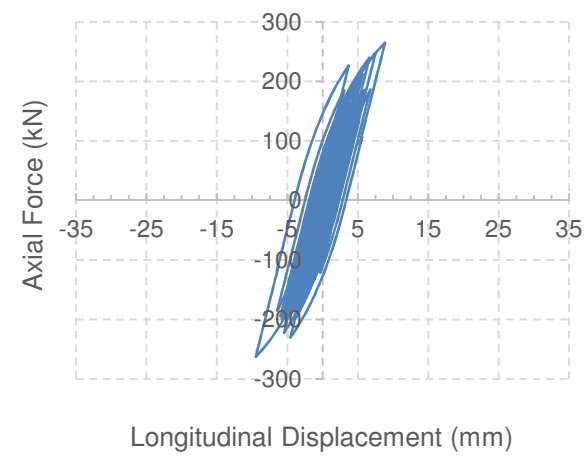


(d)

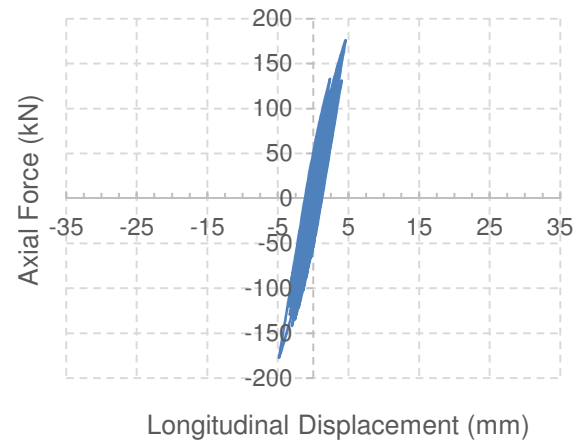
Figure 8.124: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the right bay of the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Event 1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)



(f)



(g)

Figure 8.124 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the right bay of the retrofitted building in Vancouver (Configuration III), analyzed under Cascadia Event 1: e) reduced section of fourth storey; f) reduced section of fifth storey; and g) reduced section of sixth storey

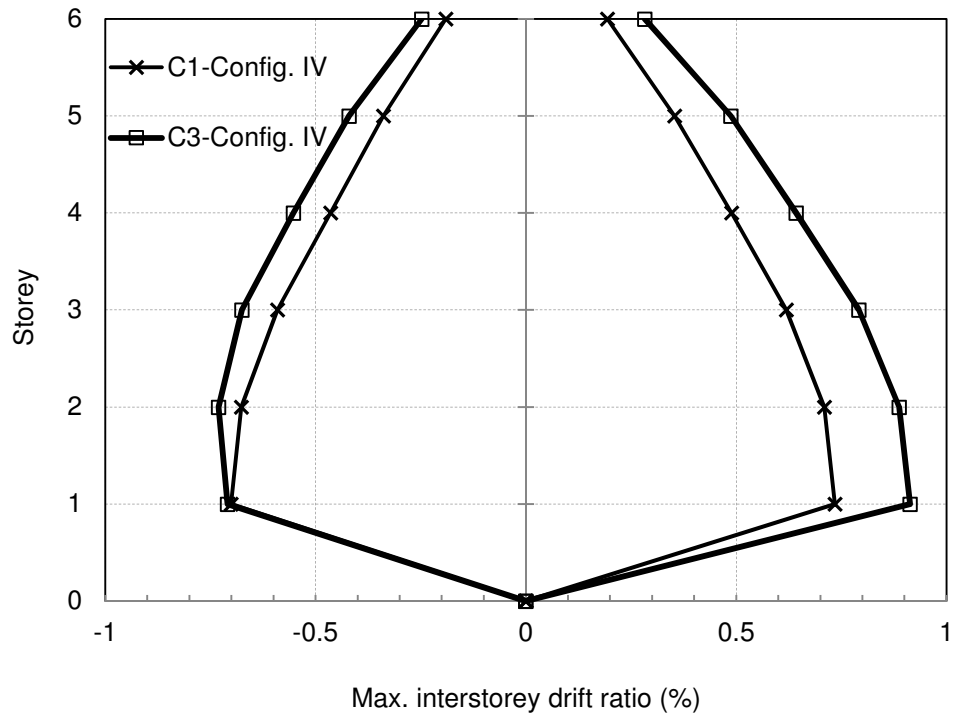


Figure 8.125: Maximum interstorey drift ratios for the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Events #1 and #3

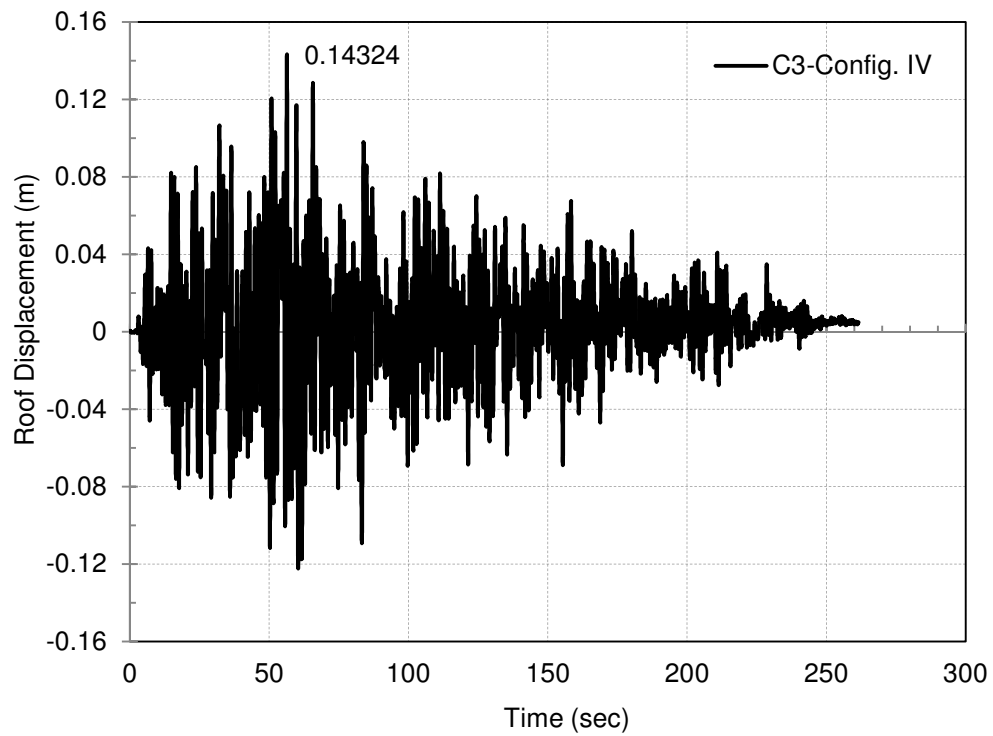


Figure 8.126: Top displacement time history for the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event #3

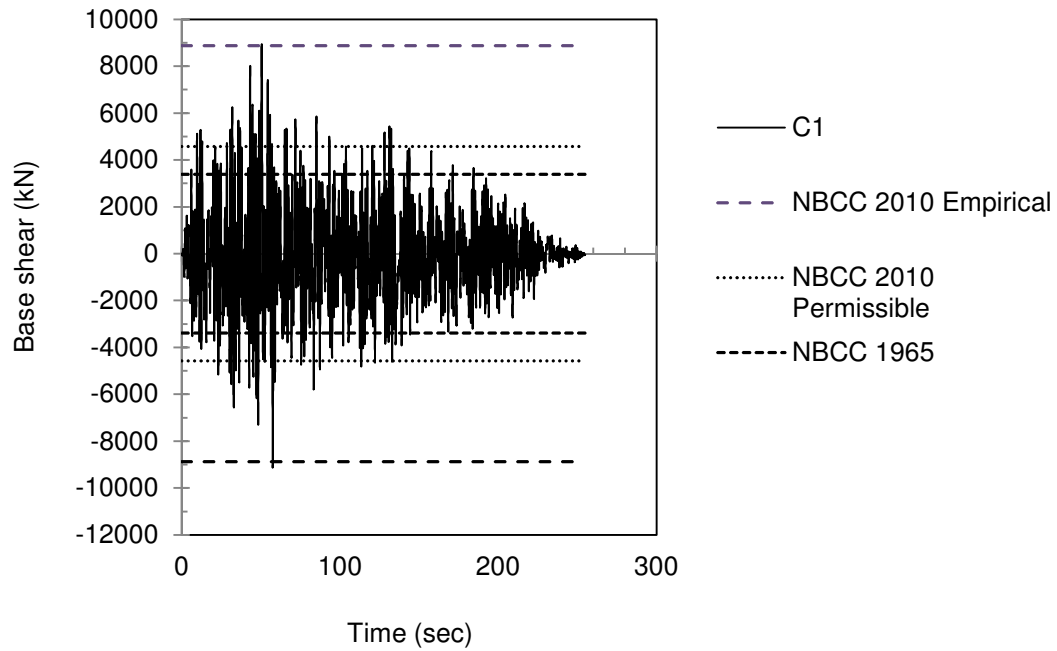


Figure 8:127: Comparison of dynamic base shear demands for Cascadia Event #1 and static base shears based on NBCC for the retrofitted building in Vancouver (Configuration IV)



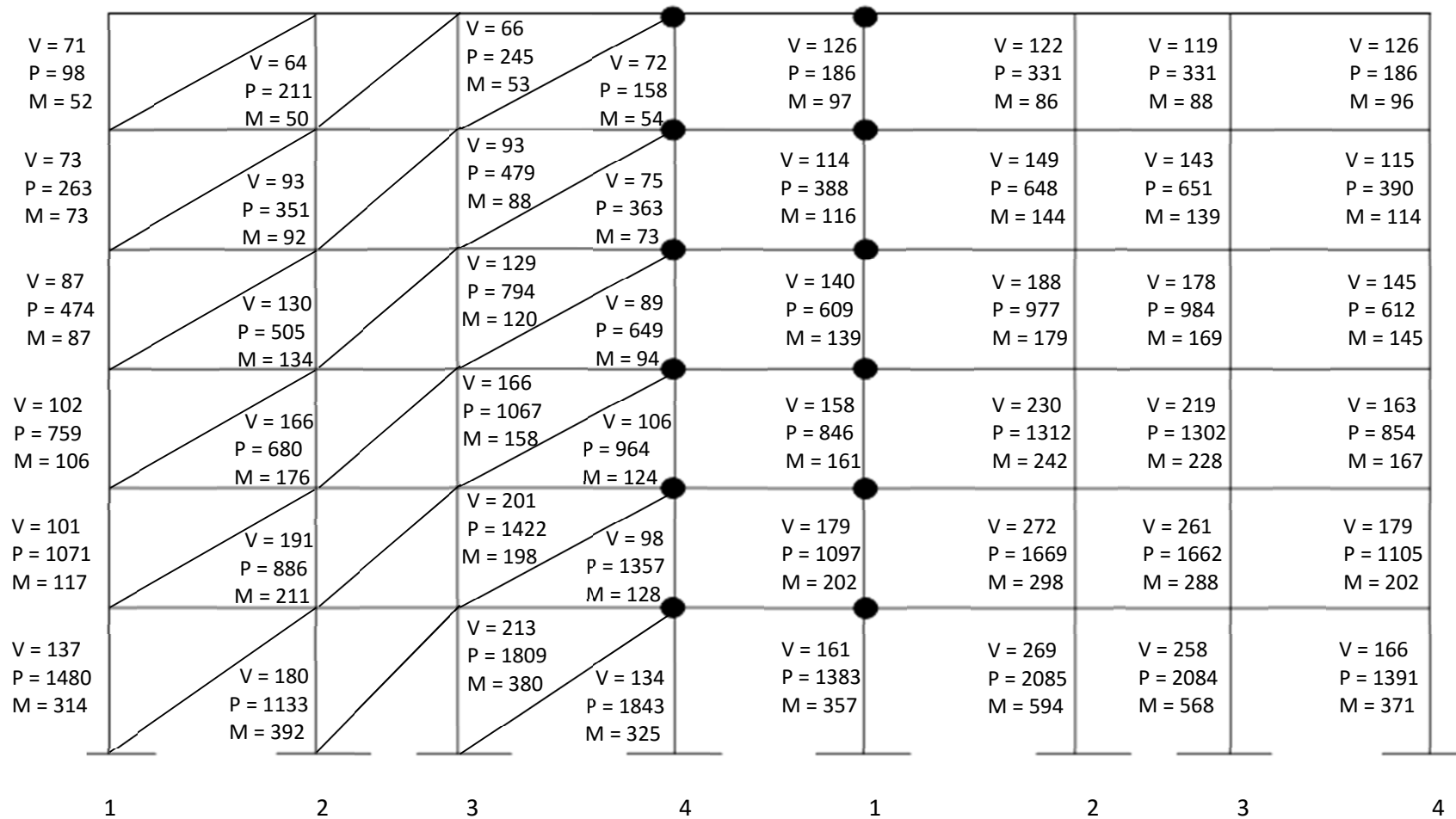
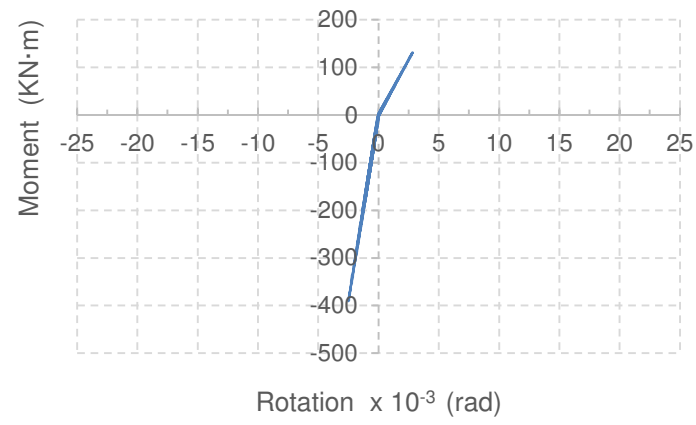
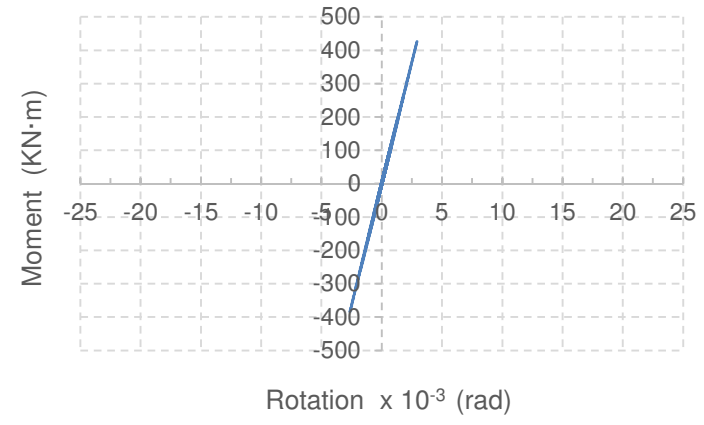


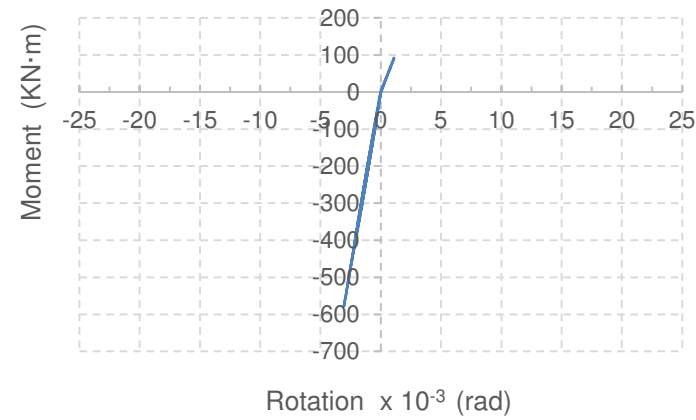
Figure 8.128: Column shear force, axial force, and moment demands for the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event #1



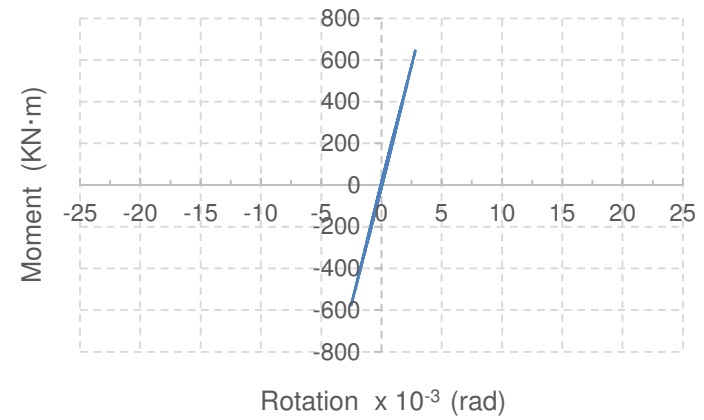
(a)



(b)

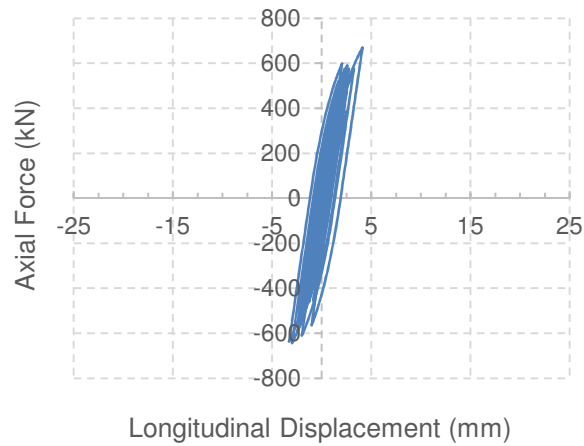


(c)

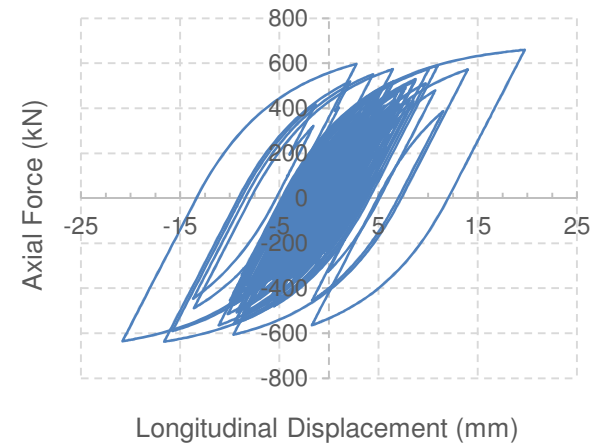


(d)

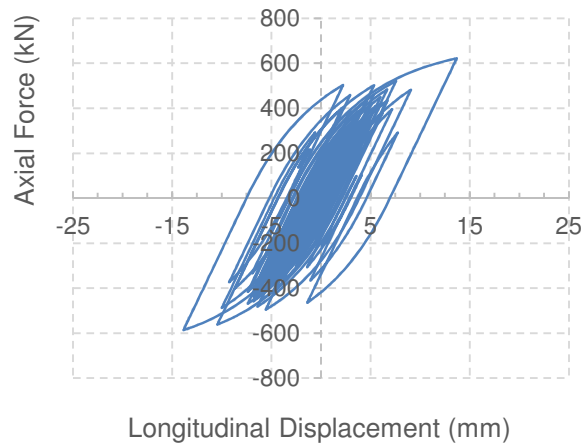
Figure 8.129 : Moment-chord rotation hysteretic relationships for selected beams and columns of the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event 1: a) beam of first storey exterior frames; b) column of first storey exterior frames; c) beam of first storey interior frames; and d) column of first storey interior frames



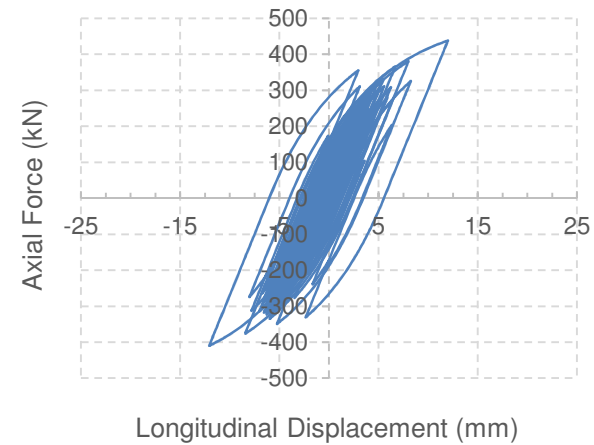
(a)



(b)

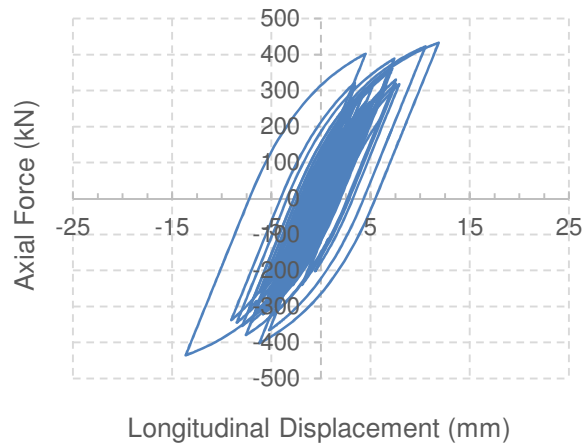


(c)

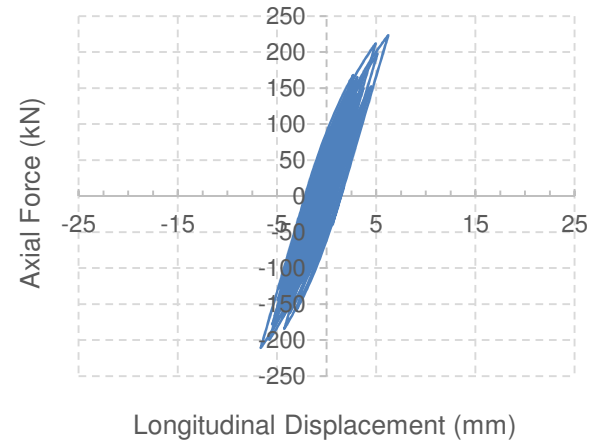


(d)

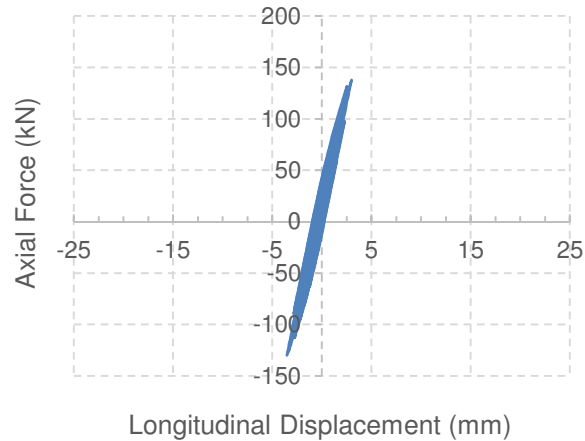
Figure 8.130: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the left bay of the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event 1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)

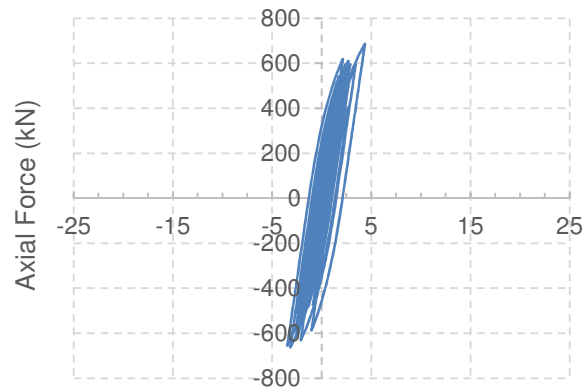


(f)



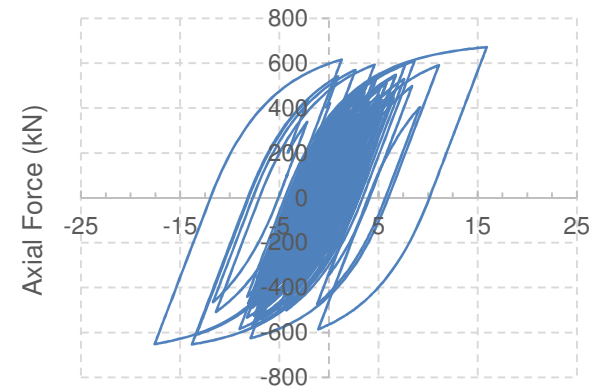
(g)

Figure 8.130 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the left bay of the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event 1: e) reduced section of fourth storey; f) reduced section of fifth storey; and g) reduced section of sixth storey



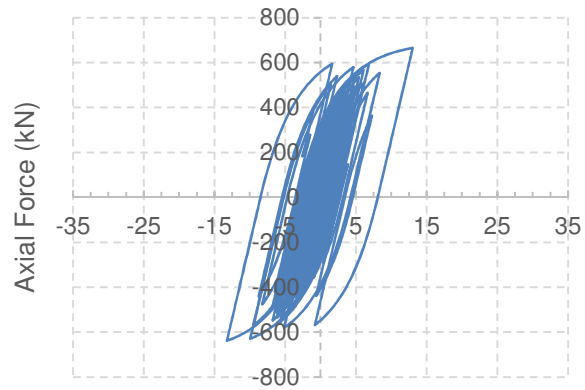
Longitudinal Displacement (mm)

(a)



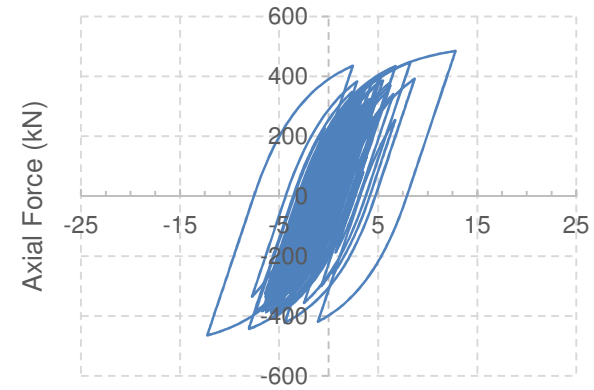
Longitudinal Displacement (mm)

(b)



Longitudinal Displacement (mm)

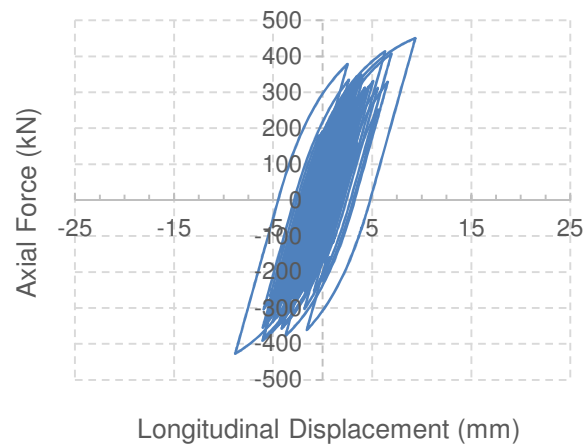
(c)



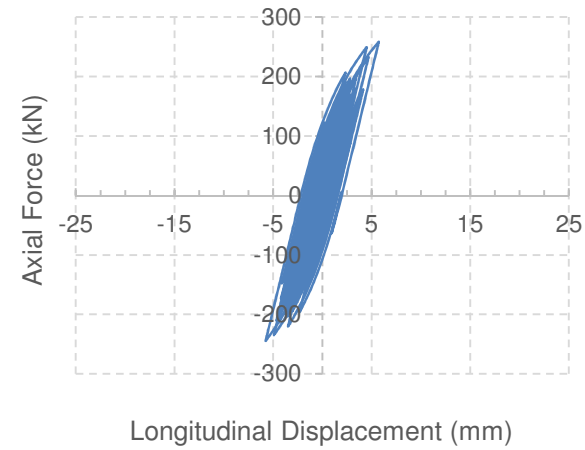
Longitudinal Displacement (mm)

(d)

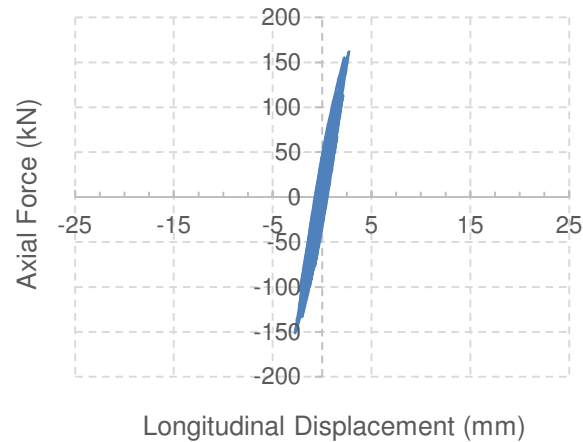
Figure 8.131: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the central bay of the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event 1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)



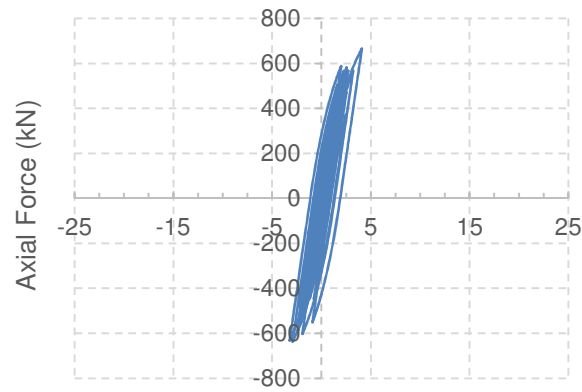
(f)



(g)

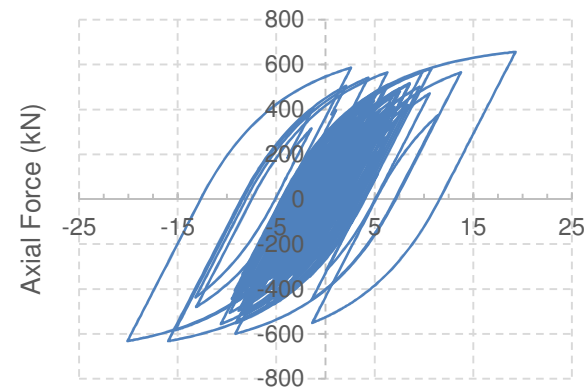
Figure 8.131 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the central bay of the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event 1: e) reduced section of fourth storey; f) reduced section of fifth storey; and g) reduced section of sixth storey





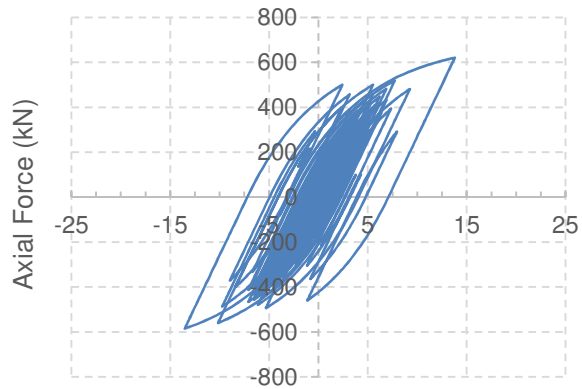
Longitudinal Displacement (mm)

(a)



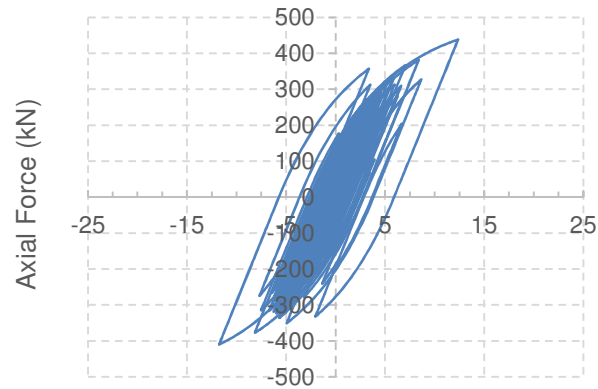
Longitudinal Displacement (mm)

(b)



Longitudinal Displacement (mm)

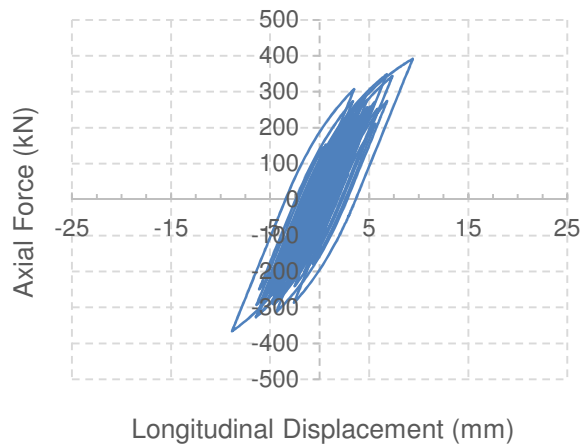
(c)



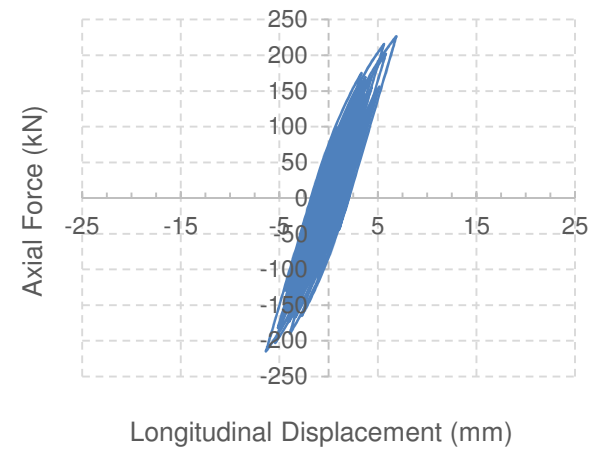
Longitudinal Displacement (mm)

(d)

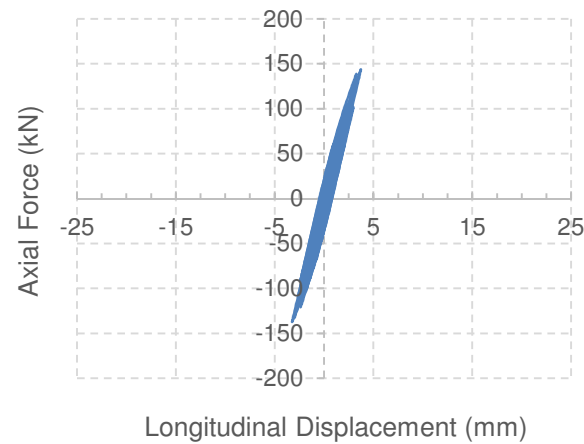
Figure 8.132: Axial force-axial deformation hysteretic relationships for buckling restrained braces of the right bay of the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event 1: a) un-reduced section at the first storey; b) reduced section at the first storey; c) reduced section at the second storey; and d) reduced section at the third storey



(e)



(f)



(g)

Figure 8.132 (Cont'd): Axial force-axial deformation hysteretic relationships for buckling restrained braces of the right bay of the retrofitted building in Vancouver (Configuration IV), analyzed under Cascadia Event 1: e) reduced section of fourth storey; f) reduced section of fifth storey; and g) reduced section of sixth storey

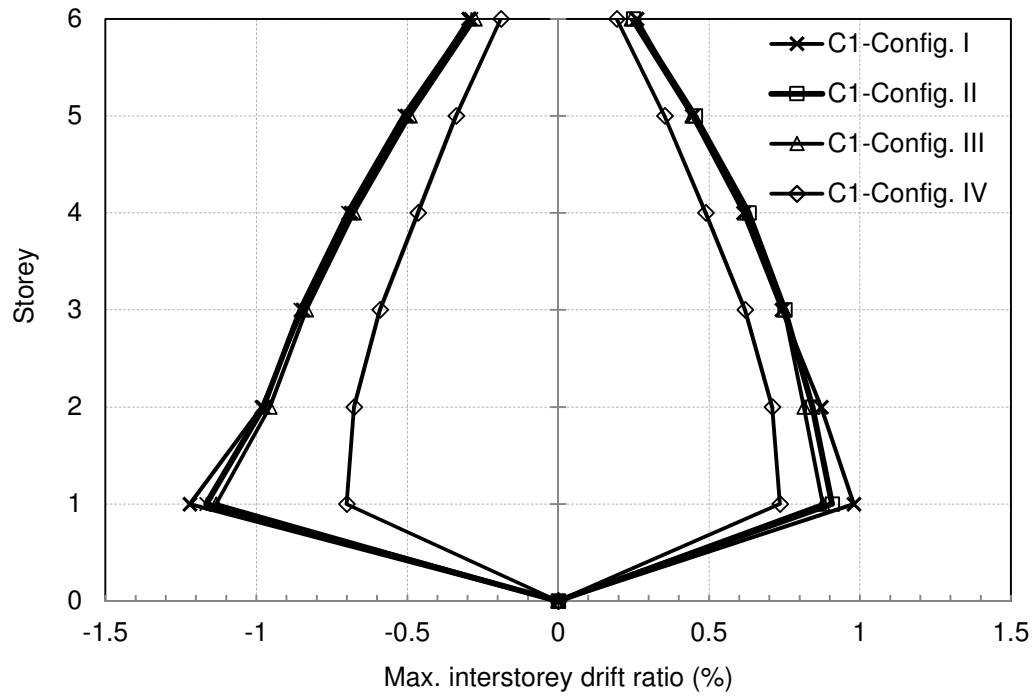


Figure 8.133: Maximum interstorey drift ratios for the retrofitted building in Vancouver (Configurations I- IV), analyzed under Cascadia Event #1

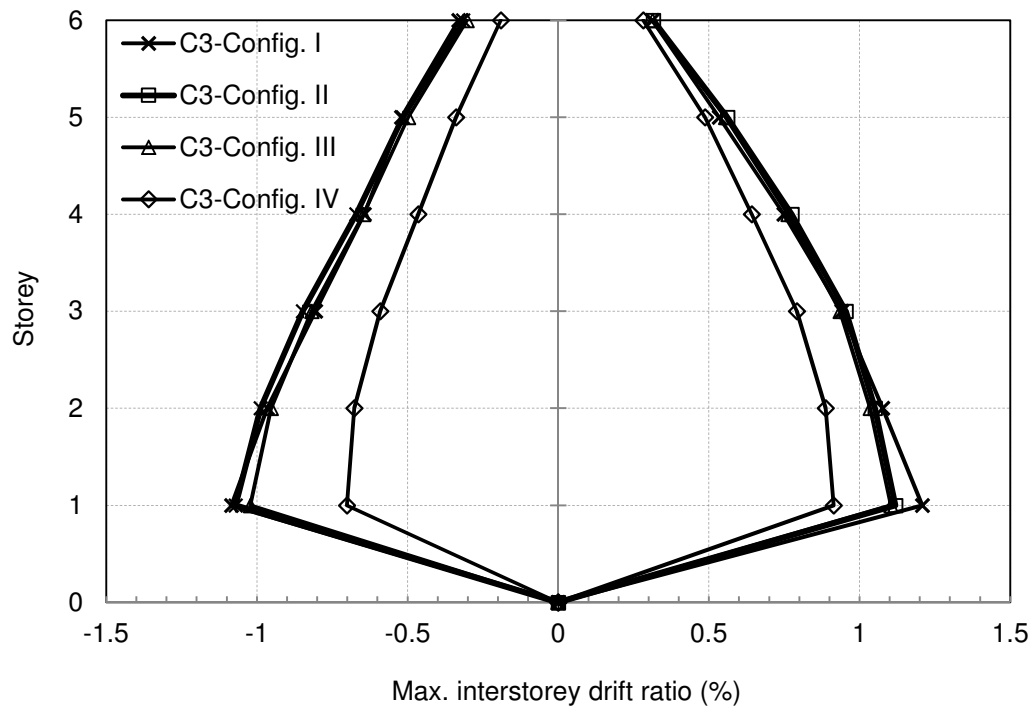


Figure 8.134: Maximum interstorey drift ratios for the retrofitted building in Vancouver (Configurations I- IV), analyzed under Cascadia Event #3

## Chapter 9

### Summary, Proposed Design Strategy, and Conclusions

#### 9.1 Summary

Many existing reinforced concrete frame structures built before the enactment of modern building codes are seismically deficient. These buildings were designed primarily for gravity and wind loads. Building Code requirements for earthquake effects have become more stringent in recent years. However, a significant stock of existing infrastructure remains seismically deficient and poses a seismic risk. A comprehensive review of the literature was first conducted, including both analytical and experimental studies. The history of code development in Canada was thoroughly investigated to assess the seismic deficiencies of existing buildings. The 1965 edition of the National Building Code of Canada (NBCC) was selected as representative of the construction practice for a large number of reinforced concrete engineered buildings spanning over two decades. Developments of seismic detailing provisions in Canadian Standard Association (CSA) standard A23.3 Design of Concrete Structures was also studied since its inception.

The effectiveness of several types of bracing systems was first investigated as a retrofit technique. A ten-storey moment-resisting frame building was designed for Ottawa for this purpose. Two-dimensional and three-dimensional analytical models were generated for SAP2000 and ETABS software, respectively. The Equivalent Static Force Procedure (ESFP) and linear dynamic analyses were employed in the analyses to simulate the earthquake induced forces. The dynamic analyses were based on the modal response spectrum method using Ritz vector analysis. The retrofit methods consisted of lateral bracing by adding reinforced concrete shear walls, diagonal steel bracing, or diagonal cables. The retrofitting components were placed in the central bay of the exterior frames of the building.

An experimental investigation was conducted to develop and investigate the effectiveness of retrofitting seismically deficient reinforced concrete frames using a novel Buckling Restrained Braces (BRBs). Six-storey buildings were designed for Ottawa and Vancouver, following the requirements of the 1965 NBCC. An interior bay of the second floor or an exterior bay of the ground floor of the prototype building was scaled to 2/3rd of the actual dimensions and used as

scaled, single bay and single storey test frames. The test setup was designed to apply incrementally increasing lateral deformation reversals using high-capacity hydraulic actuators. Gravity loads were applied to the columns and the beam through prestressing of vertical strands. The first frame was tested, repaired, and retrofitted to evaluate the effectiveness of the retrofit methodology for buildings subjected to earthquakes in the Ottawa region. The second frame served as a virgin frame building that was retrofitted and tested with the same retrofitting concept based on the higher earthquake demands to represent buildings located in Vancouver. The retrofit involved the application of the proposed BRB that was developed in this research. The BRB consists of a circular steel bar that is free to yield under tensile stresses. The bar is encased in a circular steel tube that is filled with mortar to prevent buckling in compression and permit compression yielding. Novel end units were developed to eliminate any unstrained gaps typical of commercial BRBs. The BRB was connected to the concrete test frames along one of the diagonals using end steel hinge joints.

Additional nonlinear analyses were conducted on the test frames in which incrementally increasing lateral displacement reversals were imposed to simulate the test conditions. Two-dimensional, one-bay, one-storey models were created for the bare control frame (BCF) and the repaired and retrofitted frame (RRF) in which the AISI Type 304 stainless steel bar was used as the yielding element in the buckling restrained brace. The frame members were modeled with linear elastic elements, while inelastic moment-rotation relationships were concentrated at the plastic hinge regions of the ends of the members in the frames. The BRB was modeled by elements assigned with nonlinear force-deformation relationships. A comparison with the test results demonstrated that the models were reliable and could reasonably represent the linear and nonlinear responses of the frame members and the retrofitting elements of the test frames. The models were also deemed appropriate to represent the corresponding frame elements used to model the prototype six-storey building for inelastic time-history analysis.

The second set of analyses consisting of inelastic time-history analysis was conducted to investigate the feasibility of the new technique to retrofit multi-storey reinforced concrete frame buildings located in the eastern and western regions of Canada. The structure was modelled to simulate inelasticity in the frame elements and BRBs. Uniform hazard spectrum compatible earthquake records were selected as the input motions.

## 9.2 Proposed Design Strategy

Buckling restrained braces have recently been adopted in building codes as an acceptable seismic-force-resisting system that can be selected by structural engineers. The basic principles are available in building codes, standards, and guidelines developed in North America, such as: S16-14 (CSA 2014), NBCC (2010), ANSI/AISC 341-10 (AISC 2010), AISC Seismic Design Manual (AISC 2012), ASCE/SEI Standard 7-10 (ASCE 2010), ASCE 41-13 (ASCE 2014), and NEHRP Seismic Design Technical Brief Seismic Design of Steel Buckling Restrained Braced Frames (NIST 2015). These documents provide state-of-the-art guidelines for the analysis, design, and qualification tests of buckling restrained braces that are used as dissipative elements to reduce the seismic force and deformation demands on otherwise seismically deficient buildings. NIST (2015) suggested a design procedure for buckling restrained braced frames based on elastic analysis that employs either the Equivalent Lateral Force Procedure (ELFP) or Modal Response Spectrum Analysis (MRSA) procedure. AISC (2012) demonstrated a design example for a single buckling restrained brace used as the seismic-force-resisting system for a four-storey building; while López and Sabelli (2004) illustrated a procedure to design buckling restraining braces used in a seven-storey building. Both design examples were based on new steel structures using ELFP for elastic analyses. Therefore, it is crucial to complement the research conducted during the last two decades by providing design guidance on buckling restrained braces used to retrofit older buildings. Furthermore, nonlinear dynamic analysis has become a viable method to assess structural performance levels for all types of regular and irregular structures in recent years, which has become the preferred analysis approach since the publication of the 2005 edition of NBCC.

This section presents a step-by-step seismic evaluation and design procedure for retrofitting existing seismically deficient reinforced concrete buildings with the proposed buckling restrained brace (BRB) system, while incorporating dynamic inelastic analysis and performance-based design principles in the procedure. The procedure is primarily intended for buildings with reinforced concrete frames forming the lateral force resisting system (SFRS). However, it may apply to buildings with frame-wall interactive systems if the SFRS is deficient in strength and has sufficient flexibility to benefit from the BRBs. The following steps summarize the design procedure, which is also presented in Figure 9.1 in the form of a flow chart.

1. Establish the appropriate level of seismic base shear capacity as a retrofit trigger value in terms of the percentage of the base shear required for an equivalent new building by the current



building code. This may be done relative to the design base shear required by the most recent edition of the National Building Code of Canada (from now on referred to as “the building code”). The trigger level may be as low as 60% for ordinary buildings, as outlined in Commentary L of the 2010 NBCC, or it may be any value between 60% and 100% for more important buildings, including schools, community centres, post-disaster shelters, hospitals and other post-disaster recovery buildings. Also, establish the required performance level by following the performance limits specified in ASCE 41-13 (ASCE 2014) as outlined in Section 8.9.1 of this thesis. Both the trigger value and the performance level can be decided by the building owner or the authority having jurisdiction on the building based on use and occupancy of the building; consequence of seismic damage and failure, as well as the economic considerations.

2. Conduct preliminary seismic vulnerability assessment of the building by performing an elastic frame analysis under equivalent static seismic loads, in combination with factored gravity loads, as per the procedure outlined in the building code. While two dimensional (2-D) analysis is sufficient at the preliminary stage for most buildings, highly irregular buildings with torsional sensitivity may require three dimensional (3-D) analysis. The preliminary analysis may be conducted by following the following sub steps;
  - i. Build an analytical model for the building using its geometric properties, including member lengths, cross-sectional dimensions and effective widths of slabs, diaphragms, wall struts, and other 2-way members, while accounting for finite widths of members at connections.
  - ii. Compute effective flexural, shear, and axial rigidities for members while allowing for concrete cracking; and assign them to the analytical building model.
  - iii. Compute the fundamental period of the building using the applicable empirical code expression, while accounting for the permissible elongation in period as per the building code, and obtain the corresponding uniform hazard spectral value, also given in the code reflecting the most recent hazard value for the region.
  - iv. Compute equivalent static seismic forces as per the requirements of the building code, and conduct elastic static analysis using the building model described above under the equivalent static seismic forces determined, while incorporating the gravity load effects. The ductility and over-strength related force modification factors ( $R_d$  and  $R_o$ ) has to be selected with due considerations given to the level of dependable ductility and over-

strength in the existing building. Buildings designed prior to the implementation of modern seismic design and detailing requirements for reinforced concrete (buildings prior to 1975) may be considered as conventional buildings for which  $R_d = 1.5$  and  $R_o = 1.3$  may be appropriate. The analysis results provide the design base shear force for an equivalent new building.

3. Compute factored flexural, shear and axial load resistances for the existing frame elements using the on-site conditions. These resistances can be computed by following the requirements of the CSA Standard A23.3, applicable to new buildings, as referenced by the building code. From the computed first storey column (and wall) resistances, determine the total factored base shear resistance (capacity) and compare it with the design base shear of an equivalent new building (demand) established in Step 2.
4. If the capacity-demand comparison of Step 3 indicates higher capacity than the trigger value (established in Step 1 as percentage of demand), then go to Step 6 and verify if the performance level decided in Step 1 is met through dynamic inelastic analysis. Otherwise, compute the initial estimate of BRB locations and sizes as part of the retrofit strategy.
5. BRB locations should be determined such that the functionality of the building is not compromised while the effectiveness of the bracing action is maximized. Typically, BRBs are placed in one or more of the exterior and/or interior frames with least interference. The configuration should result in at least one continuous load path to transfer the seismic forces to the foundation level. The initial estimate of the size of BRBs at the base level (often the first-storey braces) can be computed by assigning the difference between the base shear demand and the base shear capacity to the sum of all the horizontal components of the BRB forces at yield. The BRB size may be reduced along the height of the building following a pattern similar to the reduction in storey shear demands with height.
6. Conduct dynamic inelastic time history analysis to establish strength and ductility demands in structural elements as well as in BRBs (for retrofitted buildings). The following sub steps are recommended for the analysis:
  - i. Create a mathematical model for the structure being investigated. Two-dimensional models are permitted for regular structures, while 3-D models are more appropriate for complex structures with irregularities. Specific requirements for the selection of the

mathematical model are outlined in Section 4.1.8.12 of the NBCC (2010) and Section 16.2.2 of ASCE (2010). Frame members are modeled with elements that are assigned linear elastic properties with reduced stiffness to account for cracked section properties. Plastic hinges containing lumped elastic and plastic deformations are defined at either end of each linear elastic beam and column element in the model. The hinges are assigned moment-chord rotation ( $M-\Theta$ ) hysteretic relationships. Depending on the software employed for the analysis, either the primary moment rotation relationship or the properties of the entire hysteretic model may have to be specified in the form of the slopes of loading, unloading and reloading branches of the model, with or without a strength decay guideline. Similarly, appropriate nonlinear force-deformation characteristics under reversed cyclic loading should be assigned to the BRB model. Appropriate load combinations prescribed in the building code shall be used to establish the loads present during the design ground motion.

- ii. Select ground motions as per the requirement of the building code. Atkinson (2009b) suggests a minimum of five records that are compatible with the current code Uniform Hazard Spectra having a probability of exceedance of 2% in 50 years. Earthquake records of higher probabilities of exceedance may be used if justified. In such cases, the hazard levels must be correlated with target performance levels. Designers are referred to the ASCE 41-13 (ASCE 2014) for such an approach.
7. Check compliance with the performance levels selected in Step 1. This can be done by checking inelastic deformation demands obtained from the dynamic inelastic analysis against the acceptance criteria outlined in ASCE 41-13 (ASCE 2014) for the frame elements (ex: beams and columns) and the BRBs (for the retrofitted building).
8. If the compliance check of Step 7 pertains to the un-retrofitted building, and the building is found to be compliant, then no retrofitting is necessary. If the building is found to be non-compliant, then a BRB system must be designed through a trial and error process, with the locations of BRBs determined to minimize deformations in frame elements at critical locations (this may be lower stories or at locations of irregularities). The process between Steps 6 through 8 should be repeated with improved BRB layouts and trial sizes.
9. If the compliance check pertains to the retrofitted building, and the building is found to be compliant, then the retrofit design is completed. If the building is found to be non-compliant,

then the preliminary BRB layout and sizes established in Step 5 will have to be refined through trial and error. This requires an iterative process by repeating the Steps 6; 7 and 9.

10. Once the BRB design is completed, by fulfilling the performance requirements outlined in Step 1, the effects of retrofit on the existing structural elements, including the foundation must be checked, and if needed further refinements should be made in design. This may include the consideration of additional retrofit strategies to remedy the potentially negative effects of the BRB retrofit system. In particular, attention should be given to the changes in column axial loads (both tension and compression), arising from the bracing forces, as well as foundation capacities.

### **9.3 Conclusions**

The following key conclusions can be drawn based on the results of the experimental and analytical research conducted in the current research project:

1. A new and innovative buckling restrained brace (BRB) system was developed for seismically deficient reinforced concrete frame structures. The new BRB system provides the required strength and ductility enhancements while dissipating seismic-induced energy through significant yielding in both tension and compression without buckling. The large-scale frames retrofitted with the new BRB system and tested in the experimental phase of the current investigation provided an increase in the lateral strength and ductility capacity by factors of 3.9 and 2.6, respectively, relative to the un-retrofitted control frame. Furthermore, the stiffness and energy dissipation of the retrofitted frames increased by factors of 3.9 and 8.2, respectively, demonstrating superior performance of the new bracing system.
2. The new buckling restrained brace, consisting of a circular steel core bar, protected against buckling by means of a steel/mortar composite sleeve and novel end pieces that provide continuous restraint against buckling, performed well. The end pieces consist of two halves at each end, with matching protrusions in the form of “fingers” that slide into the corresponding opening while encircling the steel core bar inside, providing effective control against buckling. This technology allows the manufacturing of reusable BRBs with replaceable inner cores. The brace system has the advantage of permitting easy assembly at the construction site using standard off-the-shelf items (with the exception of the end finger pieces) with large tolerances that can be adjusted to fit imperfect frames at reduced cost.

3. The use of AISI Type 304 stainless steel core bar within the BRB demonstrated superior behaviour compared to the other types of steel core bars considered. Retrofitting with the AISI Type 304 stainless steel bar resulted in larger lateral load, stiffness, and ductility capacities.
4. The SAP2000 computer software, with its link elements and the built-in “Pivot Hysteretic Model” for hinging regions of frame elements, and the hysteretic model suggested by Wen for the BRBs provide very good correlations with experimental results under slowly applied lateral deformation reversals. The accuracy of the analytical results can be improved if the effects of variable axial loads on inelastic response is incorporated.
5. Mid-rise reinforced concrete frame buildings designed and built in Ottawa based on the requirements of NBCC 1965 may not be vulnerable in resisting the 2010 NBCC UHS compatible earthquakes. The nonlinear time history analysis of a 6-storey building, designed based on the 1965 NBCC, remained elastic, did not require any seismic retrofit, and developed a maximum interstorey drift of less than 0.5%. The building remained within the Immediate Occupancy performance level. The design base shear was only 10% smaller than that determined by the analysis.
6. Mid-rise reinforced concrete frame buildings designed and built in Ottawa based on the requirements of NBCC 1965 may be vulnerable to strong earthquakes having twice the intensity considered by the 2010 NBCC for design. The nonlinear time history analysis of a 6-storey building, designed based on the 1965 NBCC, indicated that the building under the amplified ground motion was able to resist only 50% of the required base shear established by the equivalent static lateral load analysis.
7. Mid-rise reinforced concrete frame buildings designed and built in Vancouver based on the requirements of NBCC 1965 are vulnerable to the 2010 NBCC UHS compatible earthquakes. The nonlinear time history analysis of a 6-storey building, designed based on the 1965 NBCC, indicated a maximum interstorey drift of 2.3%, which is within the Collapse Prevention structural performance level. The design base shear force was about 37% of force demands determined by the equivalent static lateral load analysis.
8. Retrofitting mid-rise non-ductile reinforced concrete frame buildings in Vancouver with the newly developed BRB system results in sufficient drift control and improved performance. The

nonlinear time history analysis of a 6-storey building, designed based on the 1965 NBCC, indicated a maximum interstorey drift of 0.92%, bringing the building to the Life Safety structural performance level. All shear forces, axial forces, and bending moments of the members of the building determined by the analysis were within the capacity limits and remained elastic during response. This implies a structural performance within the Immediate Occupancy level.

9. Results from the nonlinear time-history analyses for the retrofitted buildings located in Ottawa and Vancouver demonstrated that the BRBs provided an effective means to enhance strength and ductility capacities. The BRBs provided stable hysteretic behaviour, bringing the building performance to the Enhanced Safety structural performance range and did not exceed the Damage Control structural performance level.
10. In retrofitting frames with BRBs, it is preferable to use a larger number of smaller size BRBs, as opposed to a fewer number of larger size braces to avoid damage on the attached frame elements caused by the concentration of higher bracing forces on fewer existing frame elements. This has to be balanced with practicality, functionality and the cost of retrofitting. The retrofit applications on a 6-storey reinforced concrete frame building indicated analytically that the use of one, two or three BRBs per exterior frame resulted in a progressively improved performance, with the smallest additional burden on the attached frame elements and the foundation in the three-BRB configuration.
11. The comparative analytical investigation conducted on a 10-storey seismically deficient reinforced concrete frame building designed for Ottawa and retrofitting using three different lateral bracing systems consisting of concrete shear walls, diagonal structural steel braces and diagonal prestressing strands indicated effective drift control by all three systems, reducing the roof drift demands to about 1/3 to 2/3 of the un-retrofitted building. The structural steel bracing technique proved to be more effective; while the addition of shear walls resulted in substantially higher lateral stiffness leading to a decrease in building period at the detriment of attracting higher seismic forces.
12. The seismic design force levels in the NBCC generally increased over the years. The seismic base shear design forces for six- and ten-storey concrete moment resisting frame buildings built based on the 1965 edition of the NBCC illustrated that these buildings lack sufficient



capacity. The two buildings were designed for a base shear of 34% and 40%, respectively, relative to the 2010 edition of the NBCC, when designed for Vancouver, and 74% and 95%, respectively, when designed for Ottawa.

13. Seismic design and detailing requirements for reinforced concrete structures, according to the Canadian Standard Association (CSA) Standard A23.3 “Design of Concrete Structures,” evolved over the years. Enhanced ductile detailing requirements and the use of capacity design principles after the 1984 edition resulted in ductile buildings. Concrete buildings designed earlier may potentially have significant seismic vulnerabilities and hence may require seismic retrofitting.

#### **9.4 Original Contributions to the Field of Structural Engineering**

The following original contributions were made to the field of structural engineering in the area of buckling restrained braces:

1. A new buckling restrained brace (BRB) was developed and experimentally verified as an innovative seismic retrofit strategy for seismically deficient frame buildings. The new BRB system is structurally sound and economically attractive, providing significant strength and ductility enhancements to the retrofitted building. The technique is also suited for new construction in seismically active regions, for which lateral strength and ductility are required for improved seismic resistance. The system offers an improvement over existing BRBs, which have potential difficulties in maintaining lateral restraints within the gaps required to accommodate extension and contraction of the inner core against buckling. The new buckling restrained brace technique is the first of its kind that provides a continuous support against buckling as the steel core yields in compression. In addition, the proposed brace is less intrusive and easy-to-replace, minimizing downtime after a seismic event. The novel BRB developed in this research has been patented with the United States Patent and Trademark Office.
2. A design procedure was developed within the framework of performance-based design approaches provided in current codes of practices for seismic retrofit of reinforced concrete frame buildings using the newly developed buckling restrained braces. The procedure provides relevant aspects of seismic evaluation and design for retrofitting existing reinforced concrete buildings using nonlinear time-history analysis.

3. Seismic vulnerability assessment of 6-storey non-ductile reinforced concrete frame buildings were conducted to demonstrate their expected seismic performances in eastern and western Canada when subjected to 2010 NBCC compatible design level earthquakes. The improvements in building performance, resulting from the use of the new BRB retrofit technique was assessed and the feasibility of the new system was presented for use in Canada.

## **9.5 Recommendations for Future Research**

The following recommendations are made for further research, identified as research gaps, to expand the scope of the current research undertaking:

1. Cyclic uniaxial tests of the new buckling restrained brace under incrementally increasing axial deformation reversals. These tests will help refine the parameters of BRB design while shedding further light on the hysteretic response of the BRB. The results will also provide much needed data to define the nonlinear axial load-axial deformation hysteretic model for use in analytical models. Tests of core bars of different materials and steel grades will provide a broader inventory of the inelastic deformation capacities of the buckling restrained braces. Furthermore, fine-tuning of the restraining end elements, outer HSS casing, and steel hinge connections through further testing is recommended.
2. Analytical research involving dynamic inelastic response history analysis of reinforced concrete frame buildings with different year of construction, structural layouts, building heights and geographic locations. This analytical research will provide further verification of the applicability of the new BRB system to seismically deficient frame buildings in Canada. In addition, the analytical research should be extended to cover other seismic force resisting systems, including frame-shear wall interactive systems. Also, a significant expansion of the analytical research is recommended to include buildings with different strength, stiffness, mass and geometric irregularities.
3. Verify the feasibility of the new retrofitting technique for existing and new steel buildings of different heights and seismic regions. This will help to generalize and to broaden the research conclusions for both concrete and steel structures.
4. Further examine the influence of different configurations of the buckling restrained braces within retrofitted buildings. Furthermore, investigate the use of buckling restrained braces as

supplementary energy dissipating elements along with other primary seismic-force-resisting systems. These should be studied with care to avoid any detrimental effects on building performance.

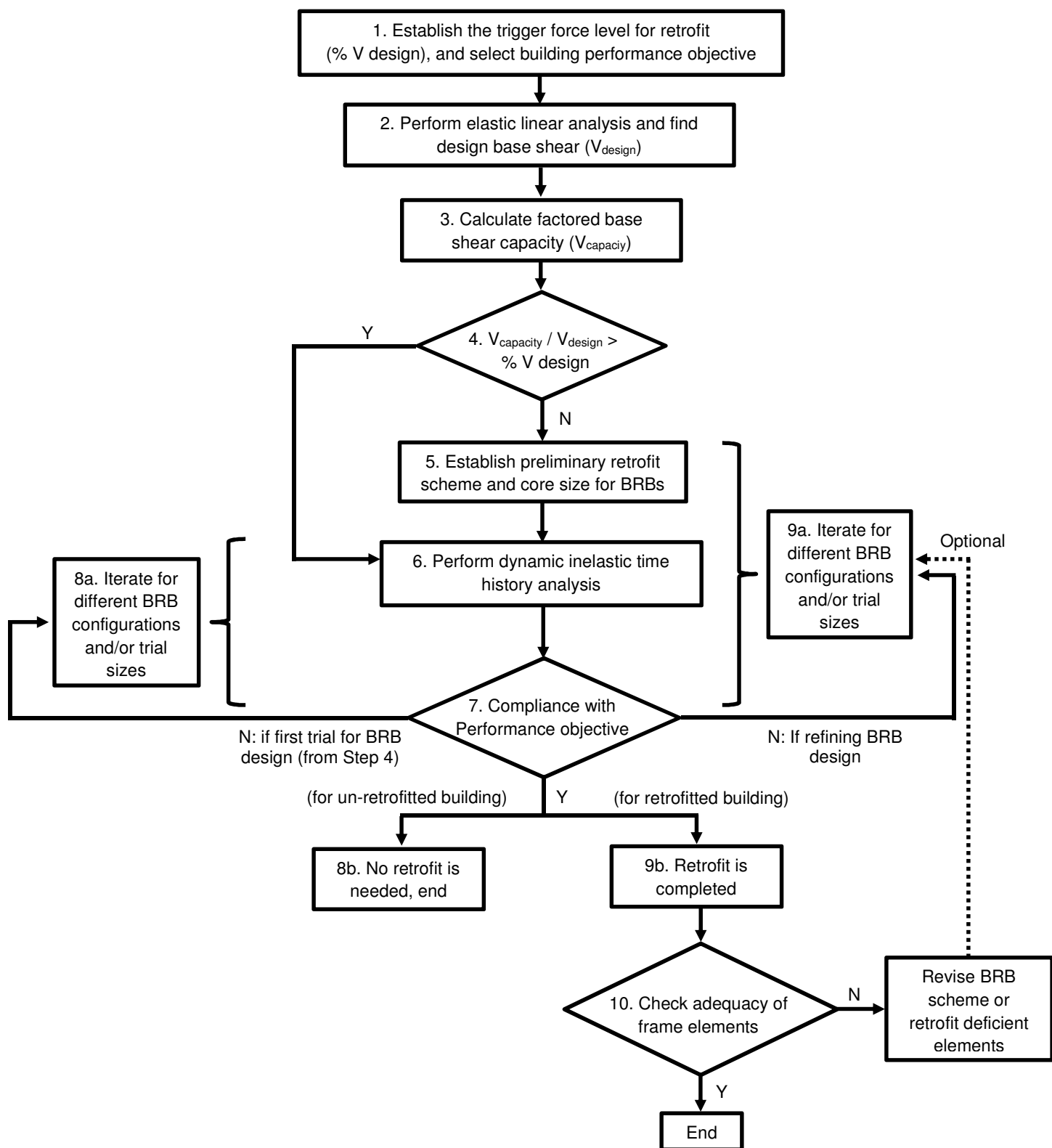


Figure 9.1: Flow chart for seismic evaluation and retrofit design of reinforced concrete buildings using buckling restrained braces (BRBs)

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## Appendix I

### Historical Comparison of Lateral Base Shear Design Force for Non-Ductile Six Storey Concrete Frame Building Based on the NBCC

#### Location and Geometry:

Location: Ottawa & Vancouver, 6 storey office buildings

Slab thickness: 0.11 m

Interiors columns: 0.5 x 0.5 m

Exteriors columns: 0.45 x 0.45 m

Secondary beams: 0.3 x 0.35 m

Main beams: 0.4 x 0.6 m

#### Material Properties:

Concrete self-weight:  $SW = 24 \text{ kN/m}^3$

#### Floor Areas:

Floor area:  $(8+8+5.7) \times 42 = 912 \text{ m}^2$

Corridor area:  $5.7 \times 42 = 240 \text{ m}^2$

#### Dead Loads (DL):

|                                                         |     |    |
|---------------------------------------------------------|-----|----|
| Mechanical corridor : $1.6 \text{ kN/m}^2 \times 240 =$ | 384 | kN |
|---------------------------------------------------------|-----|----|

|                                                         |     |    |
|---------------------------------------------------------|-----|----|
| Mechanical and Roofing: $1 \text{ kN/m}^2 \times 912 =$ | 912 | kN |
|---------------------------------------------------------|-----|----|

|                                             |     |    |
|---------------------------------------------|-----|----|
| Partitions: $1 \text{ kN/m}^2 \times 912 =$ | 912 | kN |
|---------------------------------------------|-----|----|

|                                                       |      |    |
|-------------------------------------------------------|------|----|
| SW slab (0.11 m thick)= $24 \times 0.11 \times 912 =$ | 2408 | kN |
|-------------------------------------------------------|------|----|

#### Beam Lengths:

|                                                 |     |        |
|-------------------------------------------------|-----|--------|
| Secondary: $[6 \times (42.4 - 7 \times 0.4)] =$ | 238 | m/flr. |
|-------------------------------------------------|-----|--------|

|                                                                                                        |     |        |
|--------------------------------------------------------------------------------------------------------|-----|--------|
| Main: $[8 \times (21.7 - 2 \times 0.45 - 2 \times 0.55) + 4 (42.4 - 6 \times 0.55 - 1 \times 0.45)] =$ | 313 | m/flr. |
|--------------------------------------------------------------------------------------------------------|-----|--------|

|                                                                                     |      |      |
|-------------------------------------------------------------------------------------|------|------|
| Secondary beams: $(0.3 \times 0.35 \text{ m}) = 0.3 \times (0.35-0.11) \times 24 =$ | 1.73 | kN/m |
|-------------------------------------------------------------------------------------|------|------|

|                                                                              |     |      |
|------------------------------------------------------------------------------|-----|------|
| Main beams: $(0.4 \times 0.6 \text{ m}) = 0.4 \times (0.6-0.11) \times 24 =$ | 4.7 | kN/m |
|------------------------------------------------------------------------------|-----|------|

|                                                               |      |      |
|---------------------------------------------------------------|------|------|
| Typical interior column SW = $(0.45 \times 0.45) \times 24 =$ | 4.86 | kN/m |
|---------------------------------------------------------------|------|------|

|                                                             |   |      |
|-------------------------------------------------------------|---|------|
| Typical exterior column SW = $(0.5 \times 0.5) \times 24 =$ | 6 | kN/m |
|-------------------------------------------------------------|---|------|

|                                                                  |      |      |
|------------------------------------------------------------------|------|------|
| Typical column SW avg. = $(4.86 \times 20 + 6 \times 12) / 32 =$ | 5.28 | kN/m |
|------------------------------------------------------------------|------|------|

|                                                              |     |    |
|--------------------------------------------------------------|-----|----|
| Secondary beams = $1.73 \text{ kN/m} \times 238 \text{ m} =$ | 412 | kN |
|--------------------------------------------------------------|-----|----|

|                                              |      |    |
|----------------------------------------------|------|----|
| Main beams = $4.7 \text{ kN/m} \times 313 =$ | 1471 | kN |
|----------------------------------------------|------|----|

|        |      |    |
|--------|------|----|
| Total: | 1883 | kN |
|--------|------|----|

Typical column SW:

1<sup>st</sup> floor =  $5.28 \text{ kN/m} \times (4.5 - 0.11) \times 32 \text{ Columns} = 742 \text{ kN}$

Typical floor =  $5.28 \text{ kN/m} \times (3.5 - 0.11) \times 32 \text{ Columns} = 573 \text{ kN}$

1<sup>st</sup> floor avg. =  $(742+573)/2 = 658 \text{ kN}$

| Floor # | Snow (kN)   | Mech. (kN) | Partition (kN) | Roofing & Mech. (kN) | Slab (kN) | Beams (kN) | Columns (kN) | $\Sigma W_i$ (kN) |
|---------|-------------|------------|----------------|----------------------|-----------|------------|--------------|-------------------|
| 6       | City Varies | 384        | 912            | 912                  | 2408      | 1883       | 286.5        | 6786              |
| 5       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688              |
| 4       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688              |
| 3       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688              |
| 2       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688              |
| 1       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 657.5        | 6773              |

$\Sigma \text{DL} = 40310$

Live Loads (LL):

First floor:  $(4.9 \text{ kN/m}^2)$ ; Upper floors  $(2.4 \text{ kN/m}^2)$

No live load reduction

| Floor # | LL (kN/m <sup>2</sup> ) | LL (kN) |
|---------|-------------------------|---------|
| 6       | 0.0                     | 0.0     |
| 5       | 2.4                     | 106.8   |
| 4       | 2.4                     | 106.8   |
| 3       | 2.4                     | 106.8   |
| 2       | 2.4                     | 106.8   |
| 1       | 4.9                     | 218.1   |

$\Sigma \text{LL} = 654.3$

Snow Loads:

- NBCC 1953-1985 inclusive (No snow loading in E.Q. calculations in 1941 NBCC)

$48 \text{ lb/ft}^2 (2.3 \text{ kN/m}^2) \times 912 = 2097.6 \text{ kN}$     Ottawa

$27 \text{ lb/ft}^2 (1.3 \text{ kN/m}^2) \times 912 = 1185.6 \text{ kN}$     Vancouver

- NBCC 1990-1995 inclusive

$$S = (S_c \cdot C_b + S_r) = (2.2 \cdot 0.8 + 0.4) = 2.2 \text{ kN/m}^2 \times 912 = 2006 \text{ kN} \quad \text{Ottawa}$$

$$S = (S_c \cdot C_b + S_r) = (1.6 \cdot 0.8 + 0.2) = 1.5 \text{ kN/m}^2 \times 912 = 1368 \text{ kN} \quad \text{Vancouver}$$

- NBCC 2005-2010 inclusive

$$S = I_s (S_c \cdot C_b + S_r) = 1 (2.4 \cdot 0.8 + 0.4) = 2.4 \text{ kN/m}^2 \times 912 = 2189 \text{ kN} \quad \text{Ottawa}$$

$$S = I_s (S_c \cdot C_b + S_r) = 1 (2.4 \cdot 0.8 + 0.2) = 2.12 \text{ kN/m}^2 \times 912 = 1933 \text{ kN} \quad \text{Vancouver}$$

### **(NBC 1941)**

$$F = C W$$

$$W = DL + 1/2 LL \quad 40633 \text{ kN (Ottawa and Vancouver)}$$

C = Factor ranging between 0.02 (Allowable soil bearing value > 2000 lb/ft<sup>2</sup> = 96kN/m<sup>2</sup>)  
and 0.05 (Allowable soil bearing value ≤ 2000 lb/ft<sup>2</sup>)

$$C = 0.02 \text{ (Ottawa & Vancouver)}$$

Equivalent factor for using working stress design method (limit steel to 50% Yield) = 2

|           | F(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1625  | 0.040 |
| Vancouver | 1625  | 0.040 |

### **(NBCC 1953, 1960)**

$$F = C W$$

$$W = DL + 1/4 \text{ Snow Load} = 40834 \text{ kN (Ottawa)}$$

$$40606 \text{ kN (Vancouver)}$$

N: number of stories = 6

$$C = 0.15 / (N + 4.5) = 0.15 / (6 + 4.5) = 0.014$$

Ottawa & Vancouver: Zone 3; Factor = 4

Equivalent factor for using working stress design method (limit steel to 50% Yield) = 2



|           | F(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 4667  | 0.114 |
| Vancouver | 4641  | 0.114 |

### **(NBCC 1965)**

$$V = K W$$

$$W = DL + 1/4 \text{ Snow Load} = 40834 \text{ kN (Ottawa)}$$

$$40606 \text{ kN (Vancouver)}$$

$$K = RCIFS$$

$$R = 4 \quad \text{Ottawa \& Vancouver: Climatic information for NBCC 1965}$$

$$C \text{ (Type of construction)} = 1.25 \text{ (Varies from 0.75 for MRF or shear walls with ductile response to 1.25 all other buildings)}$$

$$I \text{ (Importance factor)} = 1 \quad \text{All other Buildings not designed for post disaster}$$

$$F \text{ (Soil compressibility)} = 1 \quad (1.5 \text{ for buildings founded on highly compressible soil, } 1 \text{ for sub-soil conditions)}$$

$$N: \text{ number of stories} = 6$$

$$S = 0.25 / (9 + N) = 0.25 / (9 + 6) = 0.017$$

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 3410  | 0.083 |
| Vancouver | 3384  | 0.083 |

### **(NBCC 1970)**

$$V = (1/4) RKCIF W$$

$$W = DL + 1/4 \text{ Snow Load} = 40834 \text{ kN (Ottawa)}$$

$$40606 \text{ kN (Vancouver)}$$

$$R = 2 \quad \text{Ottawa}$$

$$4 \quad \text{Vancouver (Climatic information for NBCC 1970)}$$

$$K = 1 \quad \text{All other non-ductile frames (Table No 4.1.7.A, All Building framing systems)}$$

$$T \text{ (sec)} = 0.1 \quad N = 0.6 \quad \text{(Moment resisting space frame)}$$

$$C = 0.05 / (T)^{1/3} = 0.06 < 0.1 \text{ O.K.}$$

$$I \text{ (Importance factor)} = 1 \quad \text{All other Buildings not designed for post disaster}$$

F (Soil compressibility) = 1 (1.5 for buildings founded on highly compressible soil, low dynamic shear modulus, 1 for all others soils)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1210  | 0.030 |
| Vancouver | 2407  | 0.059 |

**(NBCC 1975, 1977)**

V= ASKIF W

W =DL + 1/4 Snow Load = 40834 kN (Ottawa)  
40606 kN (Vancouver)

|             |   |      |                                                                            |
|-------------|---|------|----------------------------------------------------------------------------|
| Zone, A (g) | 2 | 0.04 | Ottawa                                                                     |
|             | 3 | 0.08 | Vancouver (Climatic information for NBCC's 1975 & 1977 (Part 4, Table J-2) |

K = 1.3 (Table No 4.1.9.A, Buildings with non-ductile system)

$T \text{ (sec)} = 0.1 \quad N = 0.6$  (Moment resisting space frame)

$$S = 0.5 / (T)^{1/3} = 0.5 / (0.6)^{1/3} = 0.59 < 1.0 \text{ O.K.}$$

I (Importance factor) = 1 (All other Buildings not designed for post disaster)

F (Soil compressibility) = 1 (1.5 very soft loose, 1.3 compact coarse grained soil, 1.0 rock and very dense soil)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1259  | 0.031 |
| Vancouver | 2503  | 0.062 |

**(NBCC 1980)**

V= ASKIF W

W =DL + 1/4 Snow Load = 40834 kN (Ottawa)  
40606 kN (Vancouver)

|             |   |      |                                                                        |
|-------------|---|------|------------------------------------------------------------------------|
| Zone, A (g) | 2 | 0.04 | Ottawa                                                                 |
|             | 3 | 0.08 | Vancouver (Climatic information for NBCC 1980 -<br>(Part 4, Table J-2) |

$K = 1.3$  (Table No. 4.1.9.A, Buildings with non-ductile system)  
 $T \text{ (sec)} = 0.1 N = 0.6$  (Moment resisting space frame)  
 $S = 0.5 / (T)^{1/2} = 0.5 / (0.6)^{1/2} = 0.65 < 1.0$  O.K.  
 $I \text{ (Importance factor)} = 1$  (All other Buildings not designed for post disaster)  
 $F \text{ (Soil compressibility)} = 1$  (1.5 very soft loose, 1.3 compact coarse grained soil, 1.0 rock and very dense soil)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1371  | 0.034 |
| Vancouver | 2726  | 0.067 |

### **(NBCC 1985)**

$V = vSKIF W$

$W = DL + 1/4 \text{ Snow Load} = 40834 \text{ kN (Ottawa)}$   
 $40606 \text{ kN (Vancouver)}$

|               |       |       |     |                                    |
|---------------|-------|-------|-----|------------------------------------|
| Seismic Zones | $Z_a$ | $Z_v$ | $v$ | Climatic information for NBCC 1985 |
|               | 4     | 2     | 0.1 | Ottawa                             |
|               | 4     | 4     | 0.2 | Vancouver                          |

$K = 1.3$  (Table No 4.1.9.B, Buildings with non-ductile system)  
 $T \text{ (sec)} = 0.1 N = 0.6$  (Moment resisting space frame)  
 $S = 0.22 / (T)^{1/2} = 0.22 / (0.6)^{1/2} = 0.28 < 1.0$  O.K.  
 $I \text{ (Importance Factor)} = 1$  (All other Buildings not designed for post disaster)  
 $F \text{ (Soil Compressibility)} = 1$  (1.5 very soft loose, 1.3 compact coarse grained soil, 1.0 rock and very dense soil)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1508  | 0.037 |
| Vancouver | 2999  | 0.074 |

### **(NBCC 1990)**

$V = (v \text{ SIF}) WU/R$

$W = DL + 1/4 \text{ Snow Load} = 40812 \text{ kN (Ottawa)}$   
 $40652 \text{ kN (Vancouver)}$

|               |       |       |     |                                    |
|---------------|-------|-------|-----|------------------------------------|
| Seismic Zones | $Z_a$ | $Z_v$ | $v$ | Climatic information for NBCC 1990 |
|               | 4     | 2     | 0.1 | Ottawa                             |
|               | 4     | 4     | 0.2 | Vancouver                          |

$R = 1.5$  (Table No 4.1.9.1 B, Buildings with non-ductile system)  
 $U = 0.6$   
 $T \text{ (sec)} = 0.1 \text{ N} = 0.6$  (Moment resisting space frame)  
 $S = 1.5 / (T)^{1/2} = 1.5 / (0.6)^{1/2} = 1.94$   
 $I \text{ (Importance Factor)} = 1$  (All other Buildings not designed for post disaster)  
 $F \text{ (Soil Compressibility)} = 1$  (2.0 very soft, 1.5 very loose, 1.3 compact coarse grained soil, 1.0 rock and very dense soil)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 3161  | 0.077 |
| Vancouver | 6298  | 0.155 |

### **(NBCC 1995)**

$V = (v \text{ SIF}) WU/R$

$W = \text{DL} + 1/4 \text{ Snow Load} = 40812 \text{ kN (Ottawa)}$   
 $40652 \text{ kN (Vancouver)}$

|               |       |       |     |                                    |
|---------------|-------|-------|-----|------------------------------------|
| Seismic Zones | $Z_a$ | $Z_v$ | $v$ | Climatic information for NBCC 1995 |
|               | 4     | 2     | 0.1 | Ottawa                             |
|               | 4     | 4     | 0.2 | Vancouver                          |

$R = 1.5$  (Table No 4.1.9.1 B, Buildings with non-ductile system)  
 $U = 0.6$   
 Building height (m) = 22  
 $T = 0.075 (h_n)^{0.75} = 0.76$  (Moment resisting space frame)  
 $S = 1.5 / (T)^{1/2} = 1.5 / (0.76)^{1/2} = 1.72$   
 $I \text{ (Importance Factor)} = 1$  (All other Buildings not designed for post disaster)  
 $F \text{ (Soil Compressibility)} = 1$  (2.0 very soft, 1.5 very loose, 1.3 compact coarse grained soil, 1.0 rock and very dense soil)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 2805  | 0.069 |
| Vancouver | 5589  | 0.137 |

**(NBCC 2005)**

$$V_{\min} = S(2.0) M_v I_E W / (R_d R_o) < V = S(T_a) M_v I_E W / (R_d R_o) < V_{\max} = (2/3) [S(0.2) I_E W / (R_d R_o)]$$

$$W = DL + 1/4 \text{ Snow Load} = 40857 \text{ kN (Ottawa)}$$

$$40793 \text{ kN (Vancouver)}$$

$$\text{Building height (m)} = 22$$

$$T = 0.075 (h_n)^{0.75} = 0.76 \quad (\text{Moment resisting space frame})$$

| <u>T (sec)</u>         | <u>S(T), Vancouver</u> | <u>S(T), Ottawa</u> |
|------------------------|------------------------|---------------------|
| 0.2                    | 0.95                   | 0.66                |
| 0.5                    | 0.65                   | 0.32                |
| 1                      | 0.34                   | 0.13                |
| 2                      | 0.17                   | 0.044               |
| 4                      | 0.085                  | 0.022               |
| $S_a(0.2)/S_a(2)$      | 5.59                   | 15.00               |
| $M_v$ (Table 4.1.8.11) | 1                      | 1                   |

$$I_E \text{ (Importance factor)} = 1$$

$$S(0.76) = 0.220 \quad \text{Ottawa}$$

$$0.488 \quad \text{Vancouver}$$

$$R_d = 1.5 \quad \text{Conventional Construction}$$

$$R_o = 1.3$$

|                                              | <u>Ottawa</u>             | <u>Vancouver</u>          |
|----------------------------------------------|---------------------------|---------------------------|
| $V = S(T_a) M_v I_E W / (R_d R_o)$           | 4562                      | 10073                     |
| $V_{\min} = S(2.0) M_v I_E W / (R_d R_o)$    | 910                       | 3511                      |
| $V_{\max} = (2/3)[S(0.2) I_E W / (R_d R_o)]$ | 9103                      | 13082                     |
|                                              | $V_{\min} < V < V_{\max}$ | $V_{\min} < V < V_{\max}$ |

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 4620  | 0.113 |
| Vancouver | 10201 | 0.250 |

**(NBCC 2010)**

$$V_{\min} = S(2.0) M_v I_E W / (R_d R_o) < V = S(T_a) M_v I_E W / (R_d R_o) < V_{\max} = (2/3)[S(0.2) I_E W / (R_d R_o)]$$

$$W = \text{DL} + 1/4 \text{ Snow Load} = 40857 \text{ kN (Ottawa)}$$

$$40793 \text{ kN (Vancouver)}$$

$$\text{Building height (m)} = 22$$

$$T = 0.075 (h_n)^{0.75} = 0.76 \quad (\text{Moment resisting space frame})$$

| <u>T (sec)</u>         | <u>S(T), Vancouver</u> | <u>S(T), Ottawa</u> |
|------------------------|------------------------|---------------------|
| 0.2                    | 0.94                   | 0.64                |
| 0.5                    | 0.64                   | 0.31                |
| 1                      | 0.34                   | 0.14                |
| 2                      | 0.17                   | 0.046               |
| 4                      | 0.085                  | 0.023               |
| $S_a(0.2)/S_a(2)$      | 5.53                   | 13.91               |
| $M_v$ (Table 4.1.8.11) | 1                      | 1                   |

$$I_E \text{ (Importance factor)} = 1$$

$$S(0.76) = 0.221 \quad \text{Ottawa}$$

$$0.478 \quad \text{Vancouver}$$

$$R_d = 1.5 \quad \text{Conventional Construction}$$

$$R_o = 1.3$$

|                                              | <u>Ottawa</u> | <u>Vancouver</u> |
|----------------------------------------------|---------------|------------------|
| $V = S(T_a) M_v I_E W / (R_d R_o)$           | 4630          | 9992             |
| $V_{\min} = S(2.0) M_v I_E W / (R_d R_o)$    | 964           | 3556             |
| $V_{\max} = (2/3)[S(0.2) I_E W / (R_d R_o)]$ | 8940          | 13110            |

$$V_{\min} < V < V_{\max} \quad V_{\min} < V < V_{\max}$$



|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 4630  | 0.113 |
| Vancouver | 9992  | 0.245 |

## Appendix II

### Historical Comparison of Lateral Base Shear Design Force for Non-Ductile Ten Storey Concrete Frame Building Based on the NBCC

#### Location and Geometry:

Location: Ottawa & Vancouver, 10 Storey office buildings

Slab thickness: 0.2 m

Interior columns: 0.55 x 0.55 m

Exterior Columns: 0.5 x 0.5 m

All Beams: 0.35 x 0.50 m

#### Material Properties:

Concrete self-weight:  $SW = 24 \text{ kN/m}^3$

#### Floor Areas:

Typical floor area =  $564.25 \text{ m}^2$

Corridor area =  $183 \text{ m}^2$

#### Dead Loads (DL):

2.0 kN/m<sup>2</sup> mechanical roof loading over the middle bay in the E-W direction

Typical floor dead load of 1.0 kN/m<sup>2</sup>

Typical floor partitions: 0.5 kN/m<sup>2</sup>

Typical roof dead load of 1.0 kN/m<sup>2</sup>

|                                                                                      |        |         |      |
|--------------------------------------------------------------------------------------|--------|---------|------|
| Mechanical roof corridor : $2 \text{ kN/m}^2 \times 183 =$                           | 366    | kN      |      |
| Typical floor dead load: $1 \text{ kN/m}^2 \times 564.25 =$                          | 564.3  | kN      |      |
| Partitions: $0.5 \text{ kN/m}^2 \times 564.25 =$                                     | 282.2  | kN      |      |
| SW slab (0.2 m thick) = $24 \times 0.2 \times 564.25 =$                              | 2708.4 | kN      |      |
|                                                                                      |        |         |      |
| Beam lengths = $4 \times [(6 \times 5) + 0.5] + 6 [(6 \times 3) + 0.5 - 4 (0.35)] =$ | 225    | m/flr.  |      |
| Typical beam SW = $(0.35 \times 0.3 \times 24) =$                                    | 2.52   | kN/m    |      |
| Beams wt /floor = $2.52 \text{ kN/m} \times 225 =$                                   | 567    | kN/flr. | 5.28 |
| Typical interior column SW = $(0.55 \times 0.55) \times 24 =$                        | 7.26   | kN/m    |      |
| Typical exterior column SW = $(0.5 \times 0.5) \times 24 =$                          | 6.0    | kN/m    |      |
| Columns height floor to floor = $3.5 - 0.2 =$                                        | 3.3    | m       |      |
| Interior columns SW = $8 \times 7.26 \text{ kN/m} \times 3.3 =$                      | 192    | kN      |      |
| Exterior columns SW = $16 \times 6 \text{ kN/m} \times 3.3 =$                        | 317    | kN      |      |
| Total column weights =                                                               | 509    | kN/flr. |      |

| Floor # | Snow (kN)   | Mech. (kN) | Partitions (kN) | Typ. flr. dead load (kN) | Slab (kN) | Beams (kN) | Columns (kN) | $\Sigma W_i$ (kN) |
|---------|-------------|------------|-----------------|--------------------------|-----------|------------|--------------|-------------------|
| Roof    | City Varies | 366        | 282.2           | 564.3                    | 2708.4    | 567        | 254.5        | 4460.8            |
| 9       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 8       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 7       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 6       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 5       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 4       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 3       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 2       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |
| 1       | 0           | 0          | 282.2           | 564.3                    | 2708.4    | 567        | 509          | 4631.5            |

$\Sigma DL = 46144.3$

#### Live Loads (LL):

First floor: (4.9 kN/m<sup>2</sup>), Upper floors (2.4 kN/m<sup>2</sup>)

No live load reductions

| Floor # | LL (kN/m <sup>2</sup> ) | LL (kN/flr.) |
|---------|-------------------------|--------------|
| Roof    | 0.0                     | 0.0          |
| 9       | 2.4                     | 66.1         |
| 8       | 2.4                     | 66.1         |
| 7       | 2.4                     | 66.1         |
| 6       | 2.4                     | 66.1         |
| 5       | 2.4                     | 66.1         |
| 4       | 2.4                     | 66.1         |
| 3       | 2.4                     | 66.1         |
| 2       | 2.4                     | 66.1         |
| 1       | 4.9                     | 134.9        |

$\Sigma LL = 663.7$

#### Snow Loads:

- NBCC 1953-1985 inclusive (No snow loading in E.Q. calculations in 1941 NBCC)

48 lb/ft<sup>2</sup> (2.3 kN/m<sup>2</sup>) x 540 = 1297.9 kN      Ottawa

27 lb/ft<sup>2</sup> (1.3 kN/m<sup>2</sup>) x 540 = 733.6 kN      Vancouver

- NBCC 1990-1995 inclusive

$$S = (S_c \cdot C_b + S_r) = (2.2 \cdot 0.8 + 0.4) = 2.2 \text{ kN/m}^2 \times 540 = 1241 \text{ kN} \quad \text{Ottawa}$$

$$S = (S_c \cdot C_b + S_r) = (1.6 \cdot 0.8 + 0.2) = 1.5 \text{ kN/m}^2 \times 540 = 846.5 \text{ kN} \quad \text{Vancouver}$$

- NBCC 2005-2010 inclusive

$$S = I_s (S_c \cdot C_b + S_r) = 1 (2.4 \cdot 0.8 + 0.4) = 2.4 \text{ kN/m}^2 \times 540 = 1354 \text{ kN} \quad \text{Ottawa}$$

$$S = I_s (S_c \cdot C_b + S_r) = 1 (2.4 \cdot 0.8 + 0.2) = 2.12 \text{ kN/m}^2 \times 540 = 1196 \text{ kN} \quad \text{Vancouver}$$

### **(NBC 1941)**

$$F = C W$$

$$W = DL + 1/2 LL \quad 46476 \text{ kN (Ottawa and Vancouver)}$$

C = Factor ranging between 0.02 (Allowable soil bearing value > 2000 lb/ft<sup>2</sup> = 96 kN/m<sup>2</sup>)  
and 0.05 (Allowable soil bearing value ≤ 2000 lb/ft<sup>2</sup>)

$$C = 0.02 \text{ (Ottawa & Vancouver)}$$

Equivalent factor for using working stress design method (limit steel to 50% Yield) = 2

|           | F(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1859  | 0.040 |
| Vancouver | 1859  | 0.040 |

### **(NBCC 1953, 1960)**

$$F = C W$$

$$W = DL + 1/4 \text{ Snow Load} = 46469 \text{ kN (Ottawa)}$$

$$46328 \text{ kN (Vancouver)}$$

N: number of storeys = 10

$$C = 0.15 / (N + 4.5) = 0.15 / (10 + 4.5) = 0.01$$

Ottawa & Vancouver: Zone 3, Factor = 4

Equivalent factor for using working stress design method (limit steel to 50% Yield) = 2

|           | F(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 3846  | 0.083 |
| Vancouver | 3834  | 0.083 |

### **(NBCC 1965)**

$$V = K W$$

$$W = DL + 1/4 \text{ Snow Load} = 46469 \text{ kN (Ottawa)}$$

$$46328 \text{ kN (Vancouver)}$$

$$K = R C I F S$$

$$R = 4 \quad \text{Ottawa \& Vancouver: Climatic information for NBCC 1965}$$

$$C \text{ (Type of construction)} = 1.25 \quad (\text{Varies from 0.75 for MRF or shear walls with ductile response to 1.25 all other buildings})$$

$$I \text{ (Importance factor)} = 1 \quad \text{All other Buildings not designed for post disaster}$$

$$F \text{ (Soil compressibility)} = 1 \quad (1.5 \text{ for buildings founded on highly compressible soil, } 1 \text{ for sub-soil conditions})$$

$$N: \text{ number of storeys} = 10$$

$$S = 0.25 / (9 + N) = 0.25 / (9 + 10) = 0.013$$

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 3057  | 0.066 |
| Vancouver | 3048  | 0.066 |

### **(NBCC 1970)**

$$V = (1/4) R K C I F W$$

$$W = DL + 1/4 \text{ Snow Load} = 46469 \text{ kN (Ottawa)}$$

$$46328 \text{ kN (Vancouver)}$$

$$R = \begin{matrix} 2 & \text{Ottawa} \\ 4 & \text{Vancouver (Climatic information for NBCC 1970)} \end{matrix}$$

$$K = 1 \quad \text{All other non-ductile frames (Table No 4.1.7.A, All building framing systems)}$$

$$T \text{ (sec)} = 0.1 N = 1 \quad (\text{Moment resisting space frame})$$

$$C = 0.05 / (T)^{1/3} = 0.05 < 0.1 \text{ O.K.}$$

I (Importance factor) = 1 All other Buildings not designed for post disaster

F (Soil compressibility) = 1 (1.5 for buildings founded on highly compressible soil,  
low dynamic shear modulus, 1 for all others soils)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1162  | 0.025 |
| Vancouver | 2316  | 0.050 |

### **(NBCC 1975, 1977)**

V= ASKIF W

W =DL + 1/4 Snow Load = 46469 kN (Ottawa)  
46328 kN (Vancouver)

Zone, A (g) 2 0.04 Ottawa  
3 0.08 Vancouver (Climatic information for NBCC's 1975 &  
1977 (Part 4, Table J-2)

K = 1.3 (Table No 4.1.9.A, Buildings with non-ductile system)

T (sec) = 0.1 N = 1 (Moment resisting space frame)

$$S = 0.5 / (T)^{1/3} = 0.5 / (1)^{1/3} = 0.5 < 1.0 \text{ O.K.}$$

I (Importance factor) = 1 (All other Buildings not designed for post disaster)

F (Soil compressibility) = 1 (1.5 very soft loose, 1.3 compact coarse grained soil, 1.0  
rock and very dense soil)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 1208  | 0.026 |
| Vancouver | 2409  | 0.052 |

### **(NBCC 1980)**

V= ASKIF W

W =DL + 1/4 Snow Load = 46469 kN (Ottawa)  
46328 kN (Vancouver)





**(NBCC 1990)**

$$V = (\nu \text{ SIF}) WU/R$$

$$W = \text{DL} + 1/4 \text{ Snow Load} = 46455 \text{ kN (Ottawa)}$$

$$46356 \text{ kN (Vancouver)}$$

| Seismic Zones | $Z_a$ | $Z_v$ | $\nu$ | Climatic information for NBCC 1990 |
|---------------|-------|-------|-------|------------------------------------|
|               | 4     | 2     | 0.1   | Ottawa                             |
|               | 4     | 4     | 0.2   | Vancouver                          |

$$R = 1.5 \quad (\text{Table No 4.1.9.1 B, Buildings with non-ductile system})$$

$$U = 0.6$$

$$T \text{ (sec)} = 0.1 \quad N = 1 \quad (\text{Moment resisting space frame})$$

$$S = 1.5 / (T)^{1/2} = 1.5 / (1)^{1/2} = 1.5$$

$$I \text{ (Importance factor)} = 1 \quad (\text{All other Buildings not designed for post disaster})$$

$$F \text{ (Soil compressibility)} = 1 \quad (2.0 \text{ very soft, } 1.5 \text{ very loose, } 1.3 \text{ compact coarse grained soil, } 1.0 \text{ rock and very dense soil})$$

|           | V(kN) | V/W  |
|-----------|-------|------|
| Ottawa    | 2787  | 0.06 |
| Vancouver | 5563  | 0.12 |

**(NBCC 1995)**

$$V = (\nu \text{ SIF}) WU/R$$

$$W = \text{DL} + 1/4 \text{ Snow Load} = 46455 \text{ kN (Ottawa)}$$

$$46356 \text{ kN (Vancouver)}$$

| Seismic Zones | $Z_a$ | $Z_v$ | $\nu$ | Climatic information for NBCC 1995 |
|---------------|-------|-------|-------|------------------------------------|
|               | 4     | 2     | 0.1   | Ottawa                             |
|               | 4     | 4     | 0.2   | Vancouver                          |

$$R = 1.5 \quad (\text{Table No 4.1.9.1 B, Buildings with non-ductile system})$$

$$U = 0.6$$

$$\text{Building height (m)} = 35$$

$$T = 0.075 (h_n)^{0.75} = 1.08 \quad (\text{Moment resisting space frame})$$

$$S = 1.5 / (T)^{1/2} = 1.5 / (1.08)^{1/2} = 1.44$$

$$I \text{ (Importance Factor)} = 1 \quad (\text{All other Buildings not designed for post disaster})$$

F (Soil Compressibility) = 1 (2.0 very soft, 1.5 very loose, 1.3 compact coarse grained soil, 1.0 rock and very dense soil)

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 2683  | 0.058 |
| Vancouver | 5563  | 0.120 |

**(NBCC 2005)**

$$V_{\min} = S(2.0) M_v I_E W / (R_d R_o) < V = S(T_a) M_v I_E W / (R_d R_o) < V_{\max} = (2/3) [S(0.2) I_E W / (R_d R_o)]$$

$$W = \text{DL} + 1/4 \text{ Snow Load} = 46483 \text{ kN (Ottawa)}$$

$$46443 \text{ kN (Vancouver)}$$

$$\text{Building height (m)} = 35$$

$$T = 0.075 (h_n)^{0.75} = 1.08 \quad (\text{Moment resisting space frame})$$

| <u>T (sec)</u>         | <u>S(T), Vancouver</u> | <u>S(T), Ottawa</u> |
|------------------------|------------------------|---------------------|
| 0.2                    | 0.95                   | 0.66                |
| 0.5                    | 0.65                   | 0.32                |
| 1                      | 0.34                   | 0.13                |
| 2                      | 0.17                   | 0.044               |
| 4                      | 0.085                  | 0.022               |
| $S_a(0.2)/S_a(2)$      | 5.59                   | 15.00               |
| $M_v$ (Table 4.1.8.11) | 1                      | 1.02                |

$$I_E \text{ (Importance factor)} = 1$$

$$S(1.08) = 0.123 \quad \text{Ottawa}$$

$$0.327 \quad \text{Vancouver}$$

$$R_d = 1.5 \quad \text{Conventional Construction}$$

$$R_o = 1.3$$

|                                             | <u>Ottawa</u>           | <u>Vancouver</u>        |
|---------------------------------------------|-------------------------|-------------------------|
| $V = S(T_a) M_v I_E W / (R_d R_o)$          | 2983                    | 7777                    |
| $V_{min} = S(2.0) M_v I_E W / (R_d R_o)$    | 1065                    | 4049                    |
| $V_{max} = (2/3)[S(0.2) I_E W / (R_d R_o)]$ | 10488                   | 15084                   |
|                                             | $V_{min} < V < V_{max}$ | $V_{min} < V < V_{max}$ |

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 2983  | 0.064 |
| Vancouver | 7777  | 0.167 |

**(NBCC 2010)**

$$V_{min} = S(2.0) M_v I_E W / (R_d R_o) < V = S(T_a) M_v I_E W / (R_d R_o) < V_{max} = (2/3)[S(0.2) I_E W / (R_d R_o)]$$

$$W = DL + 1/4 \text{ Snow Load} = 46483 \text{ kN (Ottawa)}$$

$$46443 \text{ kN (Vancouver)}$$

$$\text{Building height (m)} = 35$$

$$T = 0.075 (h_n)^{0.75} = 1.08 \quad (\text{Moment resisting space frame})$$

| <u>T (sec)</u>         | <u>S(T), Vancouver</u> | <u>S(T), Ottawa</u> |
|------------------------|------------------------|---------------------|
| 0.2                    | 0.94                   | 0.64                |
| 0.5                    | 0.64                   | 0.31                |
| 1                      | 0.34                   | 0.14                |
| 2                      | 0.17                   | 0.046               |
| 4                      | 0.085                  | 0.023               |
| $S_a(0.2)/S_a(2)$      | 5.53                   | 13.91               |
| $M_v$ (Table 4.1.8.11) | 1                      | 1.02                |

$$I_E \text{ (Importance factor)} = 1$$

$$S(1.08) = 0.133 \quad \text{Ottawa}$$

$$0.317 \quad \text{Vancouver}$$

$$R_d = 1.5 \quad \text{Conventional Construction}$$

$$R_o = 1.3$$

|                                             | <u>Ottawa</u>           | <u>Vancouver</u>        |
|---------------------------------------------|-------------------------|-------------------------|
| $V = S(T_a) M_v I_E W / (R_d R_o)$          | 3223                    | 7558                    |
| $V_{min} = S(2.0) M_v I_E W / (R_d R_o)$    | 1118                    | 4049                    |
| $V_{max} = (2/3)[S(0.2) I_E W / (R_d R_o)]$ | 10171                   | 14925                   |
|                                             | $V_{min} < V < V_{max}$ | $V_{min} < V < V_{max}$ |

|           | V(kN) | V/W   |
|-----------|-------|-------|
| Ottawa    | 3223  | 0.069 |
| Vancouver | 7558  | 0.163 |

## Experimental Frame Design

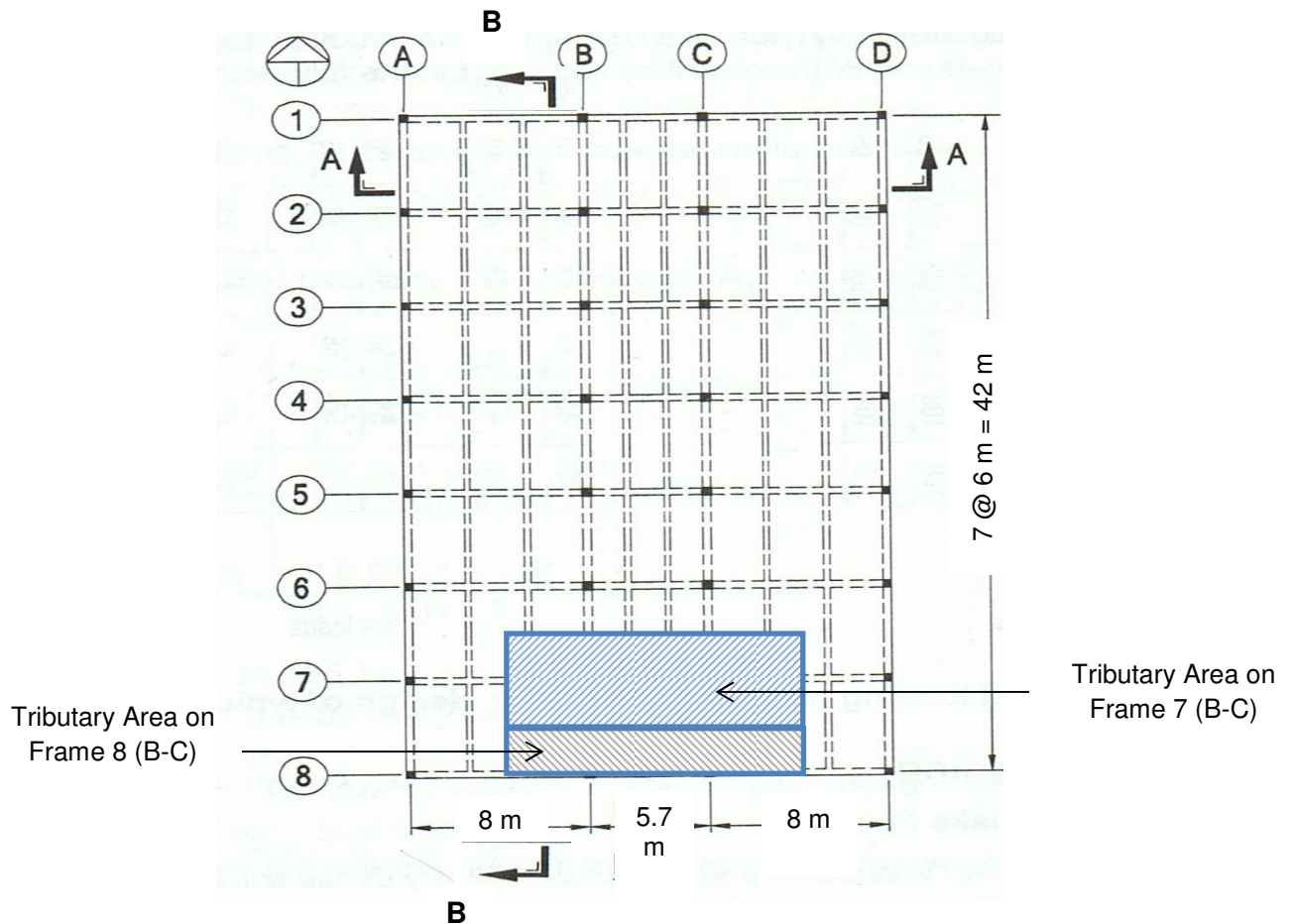
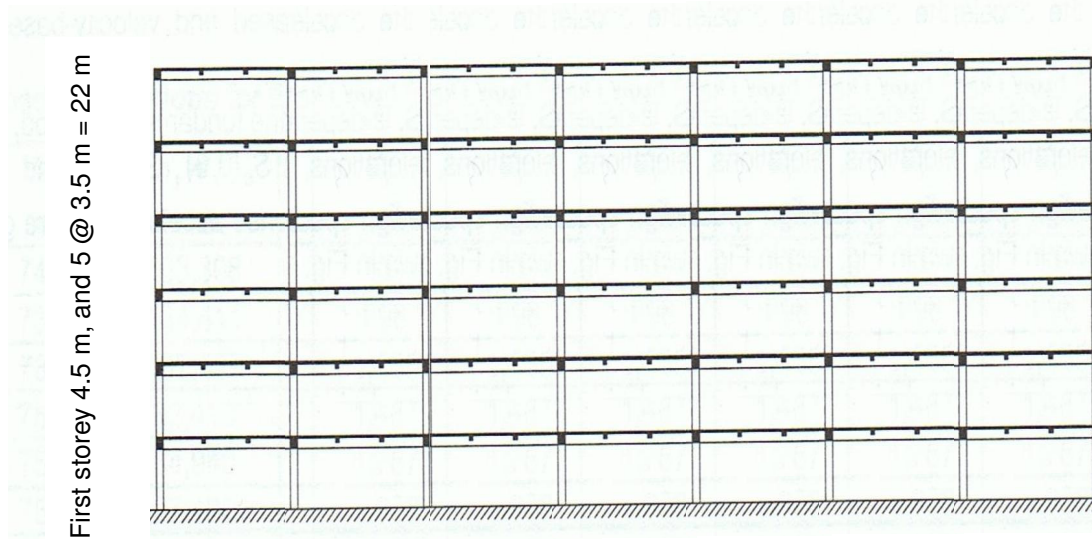
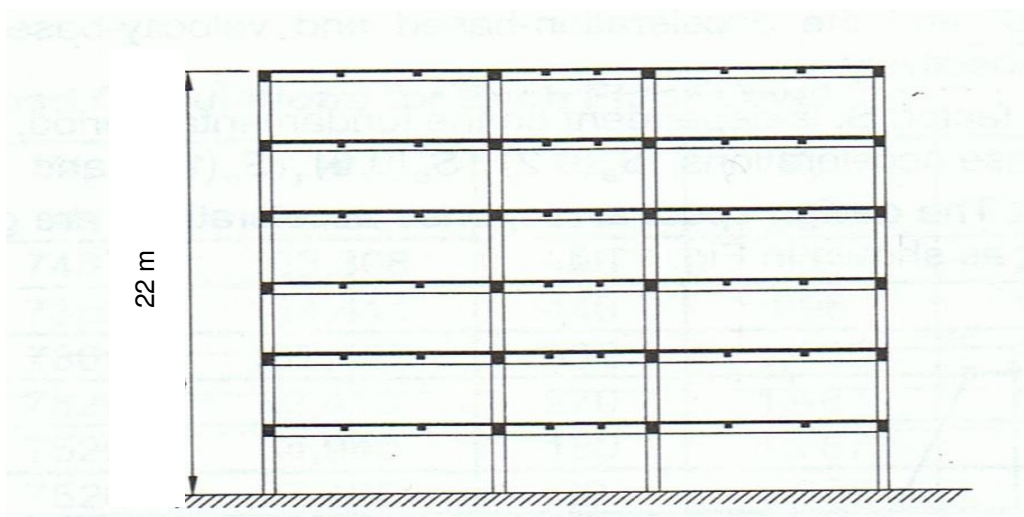


Figure AIII.1: Proposed prototype building: plan and elevation views (modified from Cement Association of Canada 2006)



Section B-B



Section A-A

Figure AIII.1 (Cont'd): Proposed prototype building: plan and elevation views (modified from Cement Association of Canada 2006)

#### Location and Geometry:

Location: Ottawa & Vancouver, 6 Storey Office Buildings (Figure AIII.1)

Slab thickness: 0.11 m

Interiors columns: 0.5 x 0.5 m

Exteriors columns: 0.45 x 0.45 m

Secondary beams: 0.3 x 0.35 m

Main beams: 0.4 x 0.6 m

#### Lab Frame Dimensions:

Frame dimension represent 2/3<sup>rd</sup> scale of central bay (5.7 m length)

Frame height (1<sup>st</sup> storey) = 4.5 m x 2/3 = 3 m

Frame length = 5.7 m x 2/3 = 3.8 m

Column dimension (square) = 2/3 x (0.45 x 0.45; Exterior columns) = 0.3 x 0.3 m

Beam dimension = 2/3 x (0.4 x 0.6; Main beams)  $\approx$  0.3 x 0.35 m. (Note: Exact beam width and depth scaling is 0.267 x 0.4 m. The beam width was enlarged for ease of construction and to match the column width. The sectional depth was reduced by 0.05 m after scaling; however, the reinforcement for the lab frame beam was selected to satisfy the scaled flexural capacity).

#### Material Properties:

Normal density concrete:  $f'_c$  = 30 MPa

Concrete self-weight: SW = 24 kN/m<sup>3</sup>

Steel yield strength:  $f_y$  = 400 MPa

#### Loading (1965 NBCC):

Load combinations: Sec. 4.5.4B.5

$U = 1.5 D + 1.8 L$

$U = 1.35 (D + L + W \text{ or } E)$

$U = 0.9 D + 1.35 W$

Where:

D (Dead Loads); herein (DL)

L (Live Loads); herein (LL)

W (Wind Loads); herein (WL)

E (Earthquake Loads); herein (EL)

#### Dead Loads (DL):

Mechanical: 1.6 kN/m<sup>2</sup> over 5.7 m central corridor

Partitions: 1 kN/m<sup>2</sup>

Roofing and mechanical: (0.5+0.5) kN/m<sup>2</sup> = 1 kN/m<sup>2</sup>

#### Snow Loads: (Part of Dead Loads)

Sec. 4.1.3.7 – 4.1.3.9 and Supplement No. 1

Ottawa: 2.3 kN/m<sup>2</sup>



Vancouver:  $1.3 \text{ kN/m}^2$

Live Loads (LL):

Section 4.1.3 and Table 4.1.3 A

First floor:  $(4.9 \text{ kN/m}^2)$ , Upper floors  $(2.4 \text{ kN/m}^2)$

No live load reduction assumed

Wind Loads (WL):

Sec. 4.1.3.11.2 and Supplement No. 1

Ottawa:  $0.708 \text{ kN/m}^2$

Vancouver:  $1.04 \text{ kN/m}^2$

Earthquake Loads (EL):

Sec. 4.1.3.15 and Supplement No. 1

A) Secondary Beam Analysis (Central Bay):

One rib tributary area =  $6 \times (5.7/3) = 11.4 \text{ m}^2$

Dead Loads (DL):

Mechanical and roofing:  $1 \text{ kN/m}^2 \times 11.4 = 11.4 \text{ kN}$

Partitions:  $1 \text{ kN/m}^2 \times 11.4 = 11.4 \text{ kN}$

SW Slab (0.11 m thick):  $24 \times 0.11 \times 11.4 = 30 \text{ kN}$

Beams:  $0.3 \times (0.35-0.11) \times 24 \times (6) = 11 \text{ kN}$

TOTAL wt (rib) DL =  $64 \text{ kN}$

Live Loads (LL):

Total wt (rib)

$4.9 \text{ kN/m}^2 \times 11.4 = 56 \text{ kN}$  (First floor)

$2.4 \text{ kN/m}^2 \times 11.4 = 28 \text{ kN}$  (Upper floors)

B) Columns Analysis

Dead Loads (DL):

Areas:

Tributary area (interior 2 columns) =  $(8+5.7) \times 6 = 83 \text{ m}^2$

Tributary area corridor =  $5.7 \times 6 = 34.2 \text{ m}^2$

|                                                                 |     |    |
|-----------------------------------------------------------------|-----|----|
| Snow loading: $2.3 \text{ kN/m}^2 \times 83$ (Ottawa governs) = | 191 | kN |
| Mechanical corridor: $1.6 \text{ kN/m}^2 \times 34.2$           | 55  | kN |
| Mechanical and roofing: $1 \text{ kN/m}^2 \times 83 =$          | 83  | kN |
| Partitions: $1 \text{ kN/m}^2 \times 83 =$                      | 83  | kN |

SW slab (0.11 m thick)=  $24 \times 0.11 \times 83 =$  219 kN

Beam lengths:

Secondary:  $4 \times (6 - 0.4) =$  22.4 m/flr.

Main:  $[(8+5.7) - (2 \times 0.5)] + 2 (6 - 0.5) =$  23.7 m/flr.

Secondary beams:  $(0.3 \times 0.35 \text{ m}) = 0.3 \times (0.35-0.11) \times 24 =$  1.73 kN/m

Main beams:  $(0.4 \times 0.6 \text{ m}) = 0.4 \times (0.6-0.11) \times 24 =$  4.7 kN/m

Typical interior column SW =  $(0.45 \times 0.45) \times 24 =$  4.86 kN/m

Typical exterior column SW =  $(0.5 \times 0.5) \times 24 =$  6 kN/m

Typical column SW =  $(4.86 \times 20 + 6 \times 12) / 32 =$  5.28 kN/m

Secondary beams:  $1.73 \text{ kN/m} \times 22.4 \text{ m} =$  39 kN

Main beams:  $4.7 \text{ kN/m} \times 23.7 =$  112 kN

Total: 151 kN

Typical Column SW:

1<sup>st</sup> floor:  $5.28 \text{ kN/m} \times (4.5 - 0.11) \times 2 \text{ Columns} =$  46 kN

Typical floors =  $4.86 \text{ kN/m} \times (3.5 - 0.11) \times 2 \text{ Columns} =$  36 kN

1<sup>st</sup> floor avg. =  $(46+36)/2 =$  41 kN

| DL      | 2 Columns (7:B-C) |                 |                |                           |           |            |              |                                | 1 Column (7:B or 7:C)      |                 |                               |
|---------|-------------------|-----------------|----------------|---------------------------|-----------|------------|--------------|--------------------------------|----------------------------|-----------------|-------------------------------|
| Floor # | Snow (kN)         | Mechanical (kN) | Partition (kN) | Roofing & mechanical (kN) | Slab (kN) | Beams (kN) | Columns (kN) | $\Sigma W_i$ (kN) on 2 Columns | $\Sigma W_i$ (kN) 1 Column | Cumulative load | Cumulative ult. load (1.5 DL) |
| 6       | 191               | 55              | 0              | 83                        | 219       | 151        | 18           | 707                            | 359                        | 359             | 538                           |
| 5       | 0                 | 0               | 83             | 83                        | 219       | 151        | 36           | 560                            | 286                        | 645             | 967                           |
| 4       | 0                 | 0               | 83             | 83                        | 219       | 151        | 36           | 560                            | 286                        | 931             | 1396                          |
| 3       | 0                 | 0               | 83             | 83                        | 219       | 151        | 36           | 560                            | 286                        | 1217            | 1825                          |
| 2       | 0                 | 0               | 83             | 83                        | 219       | 151        | 36           | 560                            | 286                        | 1503            | 2254                          |
| 1       | 0                 | 0               | 83             | 83                        | 219       | 151        | 41           | 565                            | 289                        | 1791            | 2687                          |
|         |                   |                 |                |                           |           |            | DL           | 3582                           | 1791                       |                 | 4478                          |

- Live Loads (LL):

4.8 kN/m<sup>2</sup> x 83 = 398 kN (First floor)

2.4 kN/m<sup>2</sup> x 83 = 199 kN (Upper floors)

| LL      | 2 Columns (7:B-C)           |                    | 1 Column (7:B or 7:C)       |                    |                                     |
|---------|-----------------------------|--------------------|-----------------------------|--------------------|-------------------------------------|
| Floor # | $\sum W_i$ (kN)<br>2 Column | Cumulative<br>load | $\sum W_i$ (kN)<br>1 Column | Cumulative<br>load | Cumulative<br>ult. load<br>(1.8 LL) |
| 6       | 0                           | 0                  | 0                           | 0                  | 0                                   |
| 5       | 199                         | 199                | 100                         | 100                | 179                                 |
| 4       | 199                         | 398                | 100                         | 199                | 358                                 |
| 3       | 199                         | 597                | 100                         | 299                | 537                                 |
| 2       | 199                         | 796                | 100                         | 398                | 716                                 |
| 1       | 398                         | 1194               | 199                         | 597                | 1075                                |

Cumulative Loads on 1 Column: Interior Frame Inside-Column 7:B or 7:C

| Floor # | DL (kN)<br>Service | LL (kN)<br>Service | DL+LL<br>Service | DL+LL<br>(Service) x<br>2/3 scale<br>model | DL+LL<br>(1.5D+1.8L) | DL+LL<br>(1.5D+1.8L)<br>x 2/3 scale<br>model |
|---------|--------------------|--------------------|------------------|--------------------------------------------|----------------------|----------------------------------------------|
| 6       | 359                | 0                  | 359              | 159                                        | 538                  | 239                                          |
| 5       | 645                | 100                | 744              | 331                                        | 1146                 | 509                                          |
| 4       | 931                | 199                | 1130             | 502                                        | 1754                 | 780                                          |
| 3       | 1217               | 299                | 1515             | 673                                        | 2362                 | 1050                                         |
| 2       | 1503               | 398                | 1901             | 845                                        | 2970                 | 1320                                         |
| 1       | 1791               | 597                | 2388             | 1061                                       | 3761                 | 1672                                         |

Cumulative Loads on 1 Column (Exterior Frame Inside = 1/2 Interior Frame Loading):  
Column 8:B or 8:C

| Floor # | DL (kN)<br>Service | LL (kN)<br>Service | DL+LL<br>Service | DL+LL<br>(Service)<br>x 2/3<br>Scale<br>Model | DL+LL<br>(1.5D+1.8L) | DL+LL<br>(1.5D+1.8L)<br>x 2/3 scale<br>model |
|---------|--------------------|--------------------|------------------|-----------------------------------------------|----------------------|----------------------------------------------|
| 6       | 179                | 0                  | 179              | 80                                            | 269                  | 120                                          |
| 5       | 322                | 50                 | 372              | 165                                           | 573                  | 255                                          |
| 4       | 465                | 100                | 565              | 251                                           | 877                  | 390                                          |
| 3       | 608                | 149                | 758              | 337                                           | 1181                 | 525                                          |
| 2       | 751                | 199                | 950              | 422                                           | 1485                 | 660                                          |
| 1       | 896                | 299                | 1194             | 531                                           | 1881                 | 836                                          |

- Wind Loads (WL):

Tributary areas are calculated for interior frames

1<sup>st</sup> floor tributary area (Frame Axis 2-2 to 7-7):  $(3.5+4.5/2) \text{ m} \times 6 = 24 \text{ m}^2$

Typical floor tributary area (Frame Axis 2-2 to 7-7):  $3.5 \text{ m} \times 6 = 21 \text{ m}^2$

Roof tributary area (Frame Axis 2-2 to 7-7):  $3.5/2 \text{ m} \times 6 = 10.5 \text{ m}^2$

| Floor # | Height above ground (m) | Height above ground (ft) | Height factor | Wind pressure (lb/ft <sup>2</sup> ) | Wind pressure (kN/m <sup>2</sup> ) | Tributary area (m <sup>2</sup> ) | Wind force story shear (kN) | Cumulative wind story shear (kN) | Moments to ground floor (kN·m) | Cumulative moments to ground floor (kN·m) | Cumulative story shear (kN) per single column | Cumulative story shear (kN) x 2/3 scale model |
|---------|-------------------------|--------------------------|---------------|-------------------------------------|------------------------------------|----------------------------------|-----------------------------|----------------------------------|--------------------------------|-------------------------------------------|-----------------------------------------------|-----------------------------------------------|
| 6       | 22                      | 72                       | 1             | 18                                  | 1                                  | 11                               | 9                           | 9                                | 203                            | 203                                       | 2.3                                           | 1.0                                           |
| 5       | 18.5                    | 61                       | 1             | 17                                  | 1                                  | 21                               | 17                          | 26                               | 313                            | 516                                       | 6.5                                           | 2.9                                           |
| 4       | 15.                     | 49                       | 1             | 17                                  | 1                                  | 21                               | 17                          | 43                               | 254                            | 769                                       | 10.8                                          | 4.8                                           |
| 3       | 11.5                    | 38                       | 1             | 15                                  | 1                                  | 21                               | 15                          | 58                               | 177                            | 946                                       | 14.6                                          | 6.5                                           |
| 2       | 8                       | 26                       | 1             | 15                                  | 1                                  | 21                               | 15                          | 74                               | 123                            | 1069                                      | 18.4                                          | 8.2                                           |
| 1       | 4.5                     | 15                       | 1             | 15                                  | 1                                  | 24                               | 18                          | 91                               | 79                             | 1148                                      | 22.8                                          | 10.2                                          |

- Dead Loads (DL):

Floor area:  $(8+8+5.7) \times 42 = 912 \text{ m}^2$

Corridor area:  $5.7 \times 42 = 240 \text{ m}^2$

Mechanical corridor:  $1.6 \text{ kN/m}^2 \times 240 = 384 \text{ kN}$   
 Mechanical and roofing:  $1 \text{ kN/m}^2 \times 912 = 912 \text{ kN}$   
 Partitions:  $1 \text{ kN/m}^2 \times 912 = 912 \text{ kN}$   
 SW Slab (0.11 m thick)=  $24 \times 0.11 \times 912 = 2408 \text{ kN}$

Beam Lengths:

Secondary:  $[6 \times (42.4 - 7 \times 0.4)] = 238 \text{ m/flr.}$

Main:  $[8 \times (21.7 - 2 \times 0.45 - 2 \times 0.55) + 4 (42.4 - 6 \times 0.55 - 1 \times 0.45)] = 313 \text{ m/flr.}$

Secondary beams  $(0.3 \times 0.35 \text{ m}) = 0.3 \times (0.35-0.11) \times 24 = 1.73 \text{ kN/m}$

Main beams  $(0.4 \times 0.6 \text{ m}) = 0.4 \times (0.6-0.11) \times 24 = 4.7 \text{ kN/m}$

Typical interior column SW =  $(0.45 \times 0.45) \times 24 = 4.86 \text{ kN/m}$

Typical exterior column SW =  $(0.5 \times 0.5) \times 24 = 6 \text{ kN/m}$

Typical column SW avg. =  $(4.86 \times 20 + 6 \times 12) / 32 = 5.28 \text{ kN/m}$

Secondary beams =  $1.73 \text{ kN/m} \times 238 \text{ m} = 412 \text{ kN}$

Main beams =  $4.7 \text{ kN/m} \times 313 = 1471 \text{ kN}$

Total =  $1883 \text{ kN}$

Typical Column SW

1<sup>st</sup> floor =  $5.28 \text{ kN/m} \times (4.5- 0.11) \times 32 \text{ Columns} = 742 \text{ kN}$

Typical floor =  $5.28 \text{ kN/m} \times (3.5- 0.11) \times 32 \text{ Columns} = 573 \text{ kN}$

1<sup>st</sup> floor avg. =  $(742+573)/2 = 658 \text{ kN}$

- Earthquake Loads (EL)::

DL

| Floor # | Snow (kN)   | Mech. (kN) | Partition (kN) | Roofing & mech. (kN) | Slab (kN) | Beams (kN) | Columns (kN) | $\Sigma W_i$ (kN) |
|---------|-------------|------------|----------------|----------------------|-----------|------------|--------------|-------------------|
| 6       | City Varies | 384        | 912            | 912                  | 2408      | 1883       | 286.5        | 6785.5            |
| 5       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688.0            |
| 4       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688.0            |
| 3       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688.0            |
| 2       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 573.0        | 6688.0            |
| 1       | 0           | 0          | 912            | 912                  | 2408      | 1883       | 657.5        | 6772.5            |

$\Sigma DL = 40310$

Base Shear: (Equivalent Static Force Procedure)

$$V = RCIFS W$$

Snow Loads

$$2.3 \text{ kN/m}^2 \times 912 = 2097.6 \text{ kN Ottawa}$$

$$1.3 \text{ kN/m}^2 \times 912 = 1185.6 \text{ kN Vancouver}$$

$$W = DL + 1/4 \text{ Snow Load} = \begin{matrix} 40835 & \text{kN Ottawa} \\ 40606 & \text{kN Vancouver} \end{matrix}$$

$$R = 4 \quad \text{Ottawa \& Vancouver: Climatic information for 1965 NBCC}$$

$$C = (\text{Type of Construction}) = 1.25 \quad (\text{Varies from 0.75 for MRF or shear walls (ductile behaviour) to 1.25 for all other buildings})$$

$$I = \text{Importance Factor} = 1 \quad \text{All other buildings not designed for post disaster}$$

$$F = (\text{Soil Compressibility}) = 1 \quad (1.5 \text{ for buildings founded on highly compressible Soil, 1 for sub-soil conditions})$$

$$N: \# \text{ of stories} = 6$$

$$S = 0.25/(9+N) = 0.25/(9+6) = 0.017$$

|           | <u>V(kN)</u> | <u>V/W</u>      |
|-----------|--------------|-----------------|
| Ottawa    | 3410         | 0.083 (Governs) |
| Vancouver | 3384         | 0.083           |

$$F_x = V \frac{W_x h_x}{\sum_{i=1}^n W_i h_i}$$

| Floor # | Height above ground (m) | $\sum W_i$ | $h_x W_x$ (kN·m) | $F_x$ (kN) | $V$ (kN) | $V$ of a frame of 2 Col. (kN) = $V$ Floor/16 | $V$ of a frame of 1 Col. (kN) x 2/3 scale model |
|---------|-------------------------|------------|------------------|------------|----------|----------------------------------------------|-------------------------------------------------|
| 6       | 22                      | 7310       | 160820           | 1005       | 0        | 0                                            | 0                                               |
| 5       | 18.5                    | 6688       | 123728           | 773        | 1005     | 63                                           | 14                                              |
| 4       | 15                      | 6688       | 100320           | 627        | 1778     | 111                                          | 25                                              |
| 3       | 11.5                    | 6688       | 76912            | 481        | 2404     | 150                                          | 33                                              |
| 2       | 8                       | 6688       | 53504            | 334        | 2885     | 180                                          | 40                                              |
| 1       | 4.5                     | 6773       | 30476            | 190        | 3219     | 201                                          | 45                                              |
| 0       | 0                       | -          | -                | -          | 3410     | 213                                          | 47                                              |

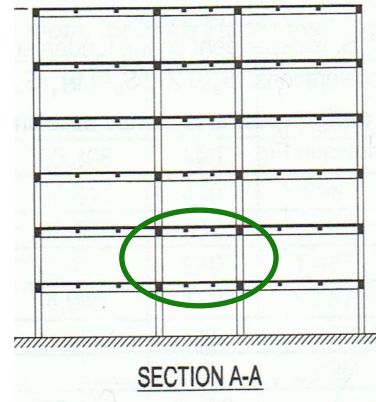
$$\Sigma = \begin{matrix} 40835 & 545760 & 3410 \end{matrix}$$

Check column location of experimental lab frame capacity within the prototype building:

A) 2<sup>nd</sup> Storey Interior Frame 7 (B to C) Columns, (Figure AIII.1)

Summary of Loadings:

| Loading type<br>(Service) | $(2/3)^2 \times 1$ Column<br>load (kN) |
|---------------------------|----------------------------------------|
| DL                        | 668                                    |
| LL                        | 177                                    |
| WL                        | 8.2                                    |
| EL                        | 40                                     |

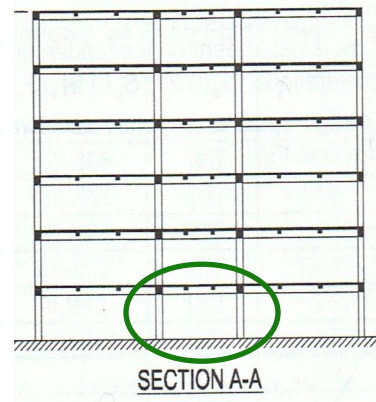


$DL + LL = 668 + 177 = 845 \text{ kN} \approx 800 \text{ kN}$  (Applied load on the lab frame column)  $\approx$  O.K.  
(Considering no live load reduction applied in the analysis)

B) 1<sup>st</sup> Storey Exterior Frame 8 (B to C) Columns, (Figure AIII.1)

Summary of Loadings:

| Loading type<br>(Service) | $(2/3)^2 \times 1$ Column<br>load (kN) |
|---------------------------|----------------------------------------|
| DL                        | 398                                    |
| LL                        | 133                                    |
| WL                        | 10.1                                   |
| EL                        | 45                                     |



$DL + LL = 398 + 133 = 531 \text{ kN} < 800 \text{ kN}$  (Applied load on the lab frame column) O.K.



### Beams Grids 7 (B-C):

| Loading type (Per Rib)     | 2 Rib point loads each of (kN) | 2 Rib point loads each x 2/3 Scale Model (kN) | U=1.5D+1.8L (kN) |
|----------------------------|--------------------------------|-----------------------------------------------|------------------|
| DL                         | 64                             | 28                                            | -                |
| LL (1 <sup>st</sup> floor) | 56                             | 25                                            | 87.3             |
| LL (Upper floors)          | 28                             | 12                                            | 64.9             |

DL + LL = 28 + 25 (1<sup>st</sup> floor) = 53 kN < 60 kN (Two point loads applied to the testing lab frame) O.K.

Simulating the ultimate load combinations of 1965 NBCC and running a SAP2000 model analysis, the governing forces on the two frames (Cases A and B) above are as follows:

| 1965 NBCC | COLUMN        |                 |            | BEAM               |                   |            |
|-----------|---------------|-----------------|------------|--------------------|-------------------|------------|
|           | Moment (KN·m) | Axial Load (kN) | Shear (kN) | - Ve Moment (KN·m) | +Ve Moment (KN·m) | Shear (kN) |
| Value     | 111           | 1340            | 64         | -111               | 89                | 120        |

### Column Design:

$P_o = \Phi [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$ ,  $\Phi = 0.75$ , 1965 NBCC, 4.5.4b.5

Select As: 8 #20 = 2400 mm<sup>2</sup>,  $\rho = 2.67\%$

$P_o = [0.85 \times (30) (300 \times 300 - 2400) + 2400 \times 400] = 3193800 = 3194 \text{ kN}$

Max allowable axial load: % of  $P_o = \Phi P_o = 75\% \times 3194 = 2396 \text{ kN} > 1340 \text{ kN}$  (O.K.)

2<sup>nd</sup> Storey Interior DL+LL (Service) x 2/3 Scale Model = 845/3194 = 26%

1<sup>st</sup> Storey Exterior DL+LL (Service) x 2/3 Scale Model = 531/3194 = 16.6 %

Apply a column load of 25% of  $P_o$  to each frame column =  $0.25 \times 3194 \approx 800 \text{ kN}$

Max. Axial Capacity of 7-Wire Prestressing strands:

$(f_y: 1650 \text{ MPa}) \times (\text{Area of } \#15: 140 \text{ mm}^2) = 231 \text{ kN}$

Use 6 strands (3 @ each side) =  $800/6 = 135 \text{ kN} < 230 \text{ kN}$  (O.K.)

Spacing Due to Shear:

$A_v/s = V_u / \Phi f_y d = 64 \times 1000 / (0.9 \times 400 \times 270) = 0.66 \text{ mm}^2/\text{m}$ ,  $\Phi_s = 0.9$

Provide No.10 ( $A_v = 200 \text{ mm}^2$ )

$S = 200/0.66 = 303 \text{ mm}$ ; Use No.10 @ 200 mm

V Capacity =  $(200/200) \times [(0.9 \times 400 \times 270)/1000] = 97 \text{ kN} > 64 \text{ kN}$  (O.K.)

Columns are checked for moment capacity at axial load of 800 kN using spColumn Structural Software (Structurepoint 2010) as shown in Figure AIII.2.  $M_{\text{capacity}} = 178 \text{ KN}\cdot\text{m} > 111 \text{ KN}\cdot\text{m}$  (O.K.)

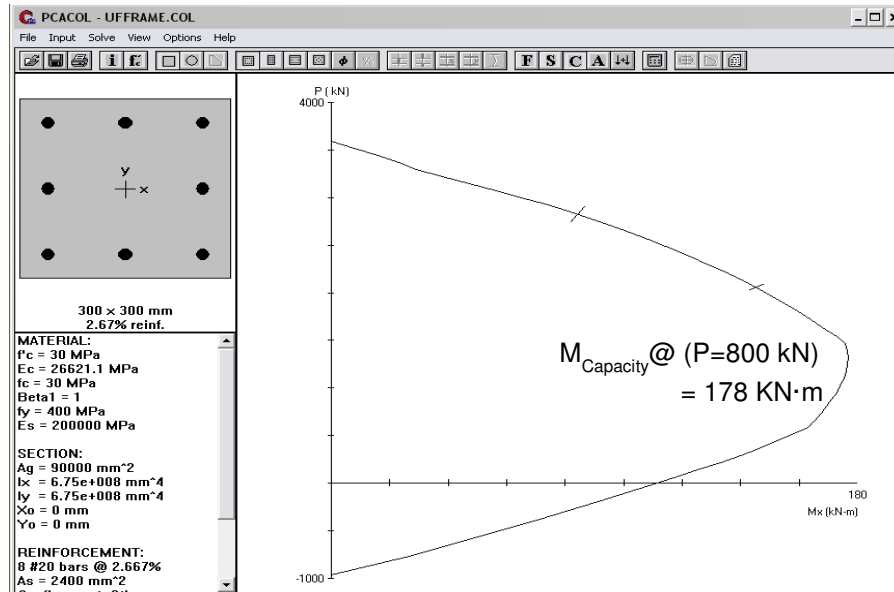


Figure AIII.2: Column sectional analysis of the experimental lab frame

#### Beam Design:

Beam - Ve Mu Moment : 111 KN·m

Beam + Ve Mu Moment : 89 KN·m

Beam  $V_u$  Shear = 120 kN

$$\rho_{\min} = 0.2 (f'_c)^{1/2} / f_y = 0.273\%$$

$$\rho_{\text{balanced}} = (0.85 \beta_1 f'_c / f_y) ((600 / (600 + f_y))) = 3.25\%$$

$$\rho_{\max} = 0.75; \rho_{\text{balanced}} = 0.75 \times 3.25 = 2.43\%$$

$$\Phi_c = 0.9 \text{ (Steel)}, \Phi_c = 0.85 \text{ (Concrete)}$$

$M^{-ve}$ :

Choose 4-20 M + 2-15 M ( $A_s = 1600 \text{ mm}^2$ )

$$a = f_y A_s / (0.85 f'_c b) = 400 \times 1600 / (0.85 \times 30 \times 300) = 84 \text{ mm}$$

$$M_u = \Phi f_y A_s (d - a/2) = 0.9 \times 1600 \times 400 \times (320 - 84/2) \times 10^{-6} = 160 \text{ KN}\cdot\text{m} > 111 \text{ KN}\cdot\text{m}$$

$$a/d = 84/320 = 0.26 < 0.5; \text{ OK } A_s \text{ yielding}$$

$M^{+ve}$ :

Choose 2-20 M + 2-15 M ( $A_s = 1000 \text{ mm}^2$ )

$$a = f_y A_s / 0.85 f'_c b = 400 \times 1000 / (0.85 \times 30 \times 300) = 53 \text{ mm}$$

$$M_u = \Phi f_y A_s (d - a/2) = 0.9 \times 1000 \times 400 \times (320 - 53/2) \times 10^{-6} = 106 \text{ KN}\cdot\text{m} > 89 \text{ KN}\cdot\text{m}$$

$V_u$ :

$$A_v/s = V_u / \Phi f_y d = 120 \times 1000 / (0.9 \times 400 \times 320) = 1.04 \text{ mm}^2/\text{m}$$

Provide No.10 ( $A_v = 200 \text{ mm}^2$ )

$$S = 200 / 1.04 = 192 \text{ mm: Use No.10 @ 150 mm}$$

$$V \text{ Capacity} = (200/150) * [(0.9 \times 400 \times 320) / 1000] = 154 \text{ kN} > 120 \text{ kN}$$

## Appendix IV: Material Specification Data of BRB Steel Core Bars

In this appendix, material specification data of the steel core bars used in the buckling restrained braces are provided (AZoNetwork 2014). The core steel bars include: AISI 12L14 carbon (UNS G12144), AISI Type 304 stainless steel (UNS S30400), and AISI 4140 chrome-molybdenum high tensile steel bar.

### IV.1 AISI 12L14 Carbon Steel (AZoNetwork 2014)

#### Introduction

Carbon is the primary alloying element present in the carbon steels. They contain 0.4% silicon and 1.2% manganese. Small quantities of molybdenum, chromium, nickel, aluminium, and copper are also present in these steels. AISI 12L14 carbon steel is known as the fastest machining bar product.

The following datasheet will discuss in detail about AISI 12L14 carbon steel.

#### Chemical Composition

The following table shows the chemical composition of AISI 12L14 carbon steel.

| Element        | Content (%)   |
|----------------|---------------|
| Iron, Fe       | 97.91 - 98.7  |
| Manganese, Mn  | 0.85 - 1.15   |
| Sulfur, S      | 0.260 - 0.35  |
| Lead, Pb       | 0.15 - 0.35   |
| Carbon, C      | 0.15          |
| Phosphorous, P | 0.040 - 0.090 |

#### Physical Properties

| Properties | Metric                 | Imperial                 |
|------------|------------------------|--------------------------|
| Density    | 7.87 g/cm <sup>3</sup> | 0.284 lb/in <sup>3</sup> |

## Mechanical Properties

The mechanical properties of the cold drawn AISI 12L14 carbon steel are outlined in the following table.

| Properties                                                     | Metric      | Imperial        |
|----------------------------------------------------------------|-------------|-----------------|
| Tensile strength                                               | 540 MPa     | 78300 psi       |
| Yield strength                                                 | 415 MPa     | 60200 psi       |
| Bulk modulus (typical for steel)                               | 140 GPa     | 20300 ksi       |
| Shear modulus (typical for steel)                              | 80.0 GPa    | 11600 ksi       |
| Elastic modulus                                                | 190-210 GPa | 27557-30458 ksi |
| Poisson's ratio                                                | 0.27-0.30   | 0.27-0.30       |
| Elongation at break                                            | 10%         | 10%             |
| Reduction of area                                              | 35%         | 35%             |
| Hardness, Brinell                                              | 163         | 163             |
| Hardness, Knoop (converted from Brinell hardness)              | 184         | 184             |
| Hardness, Rockwell B (converted from Brinell hardness)         | 84          | 84              |
| Hardness, Vickers (converted from Brinell hardness)            | 170         | 170             |
| Machinability (based on 100 machinability for AISI 1212 steel) | 160         | 160             |

## Thermal Properties

The thermal properties of AISI 12L14 carbon steel are given in the following table.

| Properties                                                  | Metric      | Imperial             |
|-------------------------------------------------------------|-------------|----------------------|
| Thermal expansion co-efficient (@ 20°C/68°F, typical steel) | 11.5 µm/m°C | 6.39 µin/in°F        |
| Thermal conductivity (typical steel)                        | 51.9 W/mK   | 360 BTU in/hr.ft².°F |

## Other Designations

Other designations that are equivalent to AISI 12L14 carbon steel include:

- ASTM A576 (12L14)
- SAE J1397 (12L14)
- SAE J403 (12L14)
- SAE J412 (12L14)
- ASTM A108
- ASTM A29
- SAE J414

## Fabrication and Heat Treatment

### Machinability

The machinability rating of AISI 12L14 carbon steel is 190. It can be machined by dispersing lead particles all over the steel.

Date Added: Aug 29, 2012 | Updated: Jul 11, 2013

## IV.2 AISI Type 304 Stainless Steel (AZoNetwork 2014)

### Chemical Formula

Fe, <0.08% C, 17.5-20% Cr, 8-11% Ni, <2% Mn, <1% Si, <0.045% P, <0.03% S

### Key Properties

These properties are specified for flat rolled product (plate, sheet and coil) in ASTM A240/A240M. Similar but not necessarily identical properties are specified for other products such as pipe and bar in their respective specifications.

### Composition

Typical compositional ranges for grade 304 stainless steels are given in table 1.

Table 1. Composition ranges for 304 grade stainless steel

| Grade |      | C     | Mn  | Si   | P      | S     | Cr   | Mo | Ni   | N    |
|-------|------|-------|-----|------|--------|-------|------|----|------|------|
| 304   | min. | -     | -   | -    | -      | -     | 18.0 | -  | 8.0  | -    |
|       | max. | 0.08  | 2.0 | 0.75 | 0.045  | 0.030 | 20.0 | -  | 10.5 | 0.10 |
| 304L  | min. | -     | -   | -    | -      | -     | 18.0 | -  | 8.0  | -    |
|       | max. | 0.030 | 2.0 | 0.75 | 0.045  | 0.030 | 20.0 | -  | 12.0 | 0.10 |
| 304H  | min. | 0.04  | -   | -    | -0.045 | -     | 18.0 | -  | 8.0  | -    |
|       | max. | 0.10  | 2.0 | 0.75 | -      | 0.030 | 20.0 | -  | 10.5 | -    |

### Mechanical Properties

Typical mechanical properties for grade 304 stainless steels are given in table 2.

Table 2. Mechanical properties of 304 grade stainless steel

| Grade                                                                 | Tensile Strength<br>(MPa) min | Yield Strength 0.2% Proof<br>(MPa) min | Elongation (% in<br>50mm) min | Hardness                 |                     |
|-----------------------------------------------------------------------|-------------------------------|----------------------------------------|-------------------------------|--------------------------|---------------------|
|                                                                       |                               |                                        |                               | Rockwell B (HR<br>B) max | Brinell (HB)<br>max |
| 304                                                                   | 515                           | 205                                    | 40                            | 92                       | 201                 |
| 304L                                                                  | 485                           | 170                                    | 40                            | 92                       | 201                 |
| 304H                                                                  | 515                           | 205                                    | 40                            | 92                       | 201                 |
| 304H also has a requirement for a grain size of ASTM No 7 or coarser. |                               |                                        |                               |                          |                     |

## **IV.3 AISI 4140 Chrome-Molybdenum High Tensile Steel (AZoNetwork 2014)**

### **Introduction**

AISI 4140 is a chromium-molybdenum alloy steel. The chromium content provides good hardness penetration, and the molybdenum content ensures uniform hardness and high strength. AISI 4140 chrome-molybdenum steel can be oil hardened to a relatively high level of hardness. The desirable properties of the AISI 4140 include superior toughness, good ductility and good wear resistance in the quenched and tempered condition.

The AISI 4140 cold finished annealed chromium-molybdenum alloy steel can be heated using various methods to yield a wide range of properties, hence it is often used as stock for forging as it has self-scaling properties. AISI 4140 is capable of resisting creep in temperatures up to 538°C (1000°F) and maintaining its properties even after long exposure at comparatively high working temperatures.

The AISI 4140 cold rolled rounds are available in the 41L40 variant that contains 0.15-0.35 lead. The lead content improves machinability, but has significant effect on other desirable properties.

### **Chemical Composition**

AISI 4140 is versatile because of its simple chemistry and has the following composition:

- 0.40 % carbon and 0.85 % manganese which offers toughness and can be heat treated and hardened 0.1 % chromium adds to overall toughness but is not enough to be made into stainless steel
- 0.25 % molybdenum and small amounts of sulfur, silicon and phosphorous

| Element        | Content                         |
|----------------|---------------------------------|
| Carbon, C      | 0.380 - 0.430 %                 |
| Chromium, Cr   | 0.80 - 1.10 %                   |
| Iron, Fe       | 96.785 - 97.77 % (As remainder) |
| Manganese, Mn  | 0.75 - 1.0 %                    |
| Molybdenum, Mo | 0.15 - 0.25 %                   |
| Phosphorous, P | ≤ 0.035 %                       |
| Silicon, Si    | 0.15 - 0.30 %                   |
| Sulfur, S      | ≤ 0.040 %                       |

### **Physical Properties**

#### **Density**

The density of AISI 4140 is 7.7 to 8.03 kg/m<sup>3</sup>



## Mechanical Properties

Mechanical properties for AISI 4140 in the annealed state:

| Mechanical Properties                                                                                              | Metric   | Imperial  |
|--------------------------------------------------------------------------------------------------------------------|----------|-----------|
| Hardness, Brinell                                                                                                  | 197      | 197       |
| Hardness, Knoop (Converted from Brinell hardness)                                                                  | 219      | 219       |
| Hardness, Rockwell B (Converted from Brinell hardness)                                                             | 92       | 92        |
| Hardness, Rockwell C (Converted from Brinell hardness. Value below normal HRC range, for comparison purposes only) | 13.0     | 13.0      |
| Hardness, Vickers (Converted from Brinell hardness)                                                                | 207      | 207       |
| Tensile Strength, Ultimate                                                                                         | 655 MPa  | 95000 psi |
| Tensile Strength, Yield                                                                                            | 415 MPa  | 60200 psi |
| Elongation at Break (in 50 mm)                                                                                     | 25.7 %   | 25.7 %    |
| Reduction of Area                                                                                                  | 56.9 %   | 56.9 %    |
| Modulus of Elasticity (Typical for steel)                                                                          | 205 GPa  | 29700 ksi |
| Bulk Modulus                                                                                                       | 140 GPa  | 20300 ksi |
| Poissons Ratio (Calculated)                                                                                        | 0.290    | 0.290     |
| Machinability (Based on AISI 1212 as 100% machinability)                                                           | 65 %     | 65 %      |
| Shear Modulus                                                                                                      | 80.0 GPa | 11600 ksi |

## Appendix V: Analysis of Experimental Test Frame

A comprehensive analysis is carried out on the bare control frame (BCF) and retrofitted frames (RRF/RF). Included in this appendix are properties of the frame sections, column bars splices and development lengths, imposed force demands, calculation of frame member shear capacities according to the General Method of the CSA A23.4-04, and calculations of elastic deformations of BRB steel core bars.

### V.1 Cracked Section Properties of Frame Members

Beam:  $I_e = 0.4 I_g$

Column:  $I_e = \alpha_c I_g = 0.68 I_g$

$\alpha_c = 0.5 + 0.6 [P_s / (f'_c A_g)] = 0.678 \leq 1.0$

### V.2 Splices and Development Lengths at Columns Bases

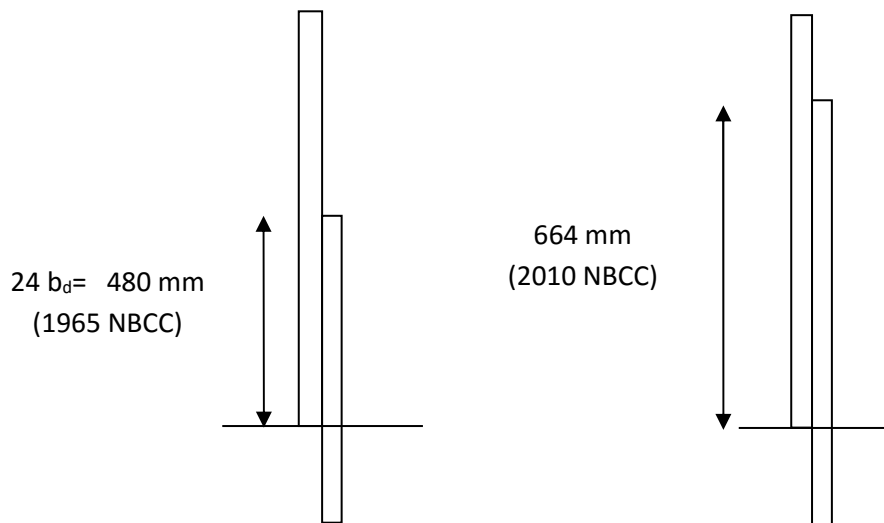


Figure V.1: Comparison of specified compression splice at column bases according to 1965 and 2010 editions of the NBCC

- NBCC 1965: based on ACI-1963:
  - Splice length: (Section 805 – C)
    - Compression length:  $24 d_b = 24(20) = \underline{480 \text{ mm (Governs)}}$

- Tension length:  $36 d_b = 720 \text{ mm}$  (Not used due to governing compression forces)
- NBCC 2010: based on CSA A23.3 - 04:  
 Compression Lap Length: (Clause 12.16.1)
  - Lap length =  $(0.133 f_y - 24) d_b = [0.133(430) - 24](20) = 664 \text{ mm}$  (Governs)
- Development Length: (Clause 12.3.2)
  - Compression length:  

$$l_{db \text{ basic}} = 0.24 d_b f_y / (f_c)^{1/2} = 0.24(20)(430) / (30)^{1/2} = 377 \text{ mm} \geq 0.044 f_y d_b = 378 \text{ mm}$$
  - Tension length:  

$$l_d = 0.45 k_1 k_2 k_3 k_4 f_y d_b / f_c^{1/2} = 0.45(1)(1)(1)(0.8)(430)(20) / (30)^{1/2} = 565 \text{ mm}$$
 (Not used due to governing compression forces)

Steel ( $f_y$ ) to be used at column bases for column sectional capacity assessment:

$$f_y = [(\text{Comp. Splice length, 1965 NBCC}) / (\text{Comp. Lap Length, 2010 NBCC})] \times 430 = (480/664) \times 430 = 311 \text{ MPa}$$

### V.3 Frames Demand Force

The axial and shear forces, and bending moments were determined for the bare control and retrofitted frames due to gravity load and a unit lateral load. Analysis results are provided in the following sections.

### V.3.1 Bare Control Frame

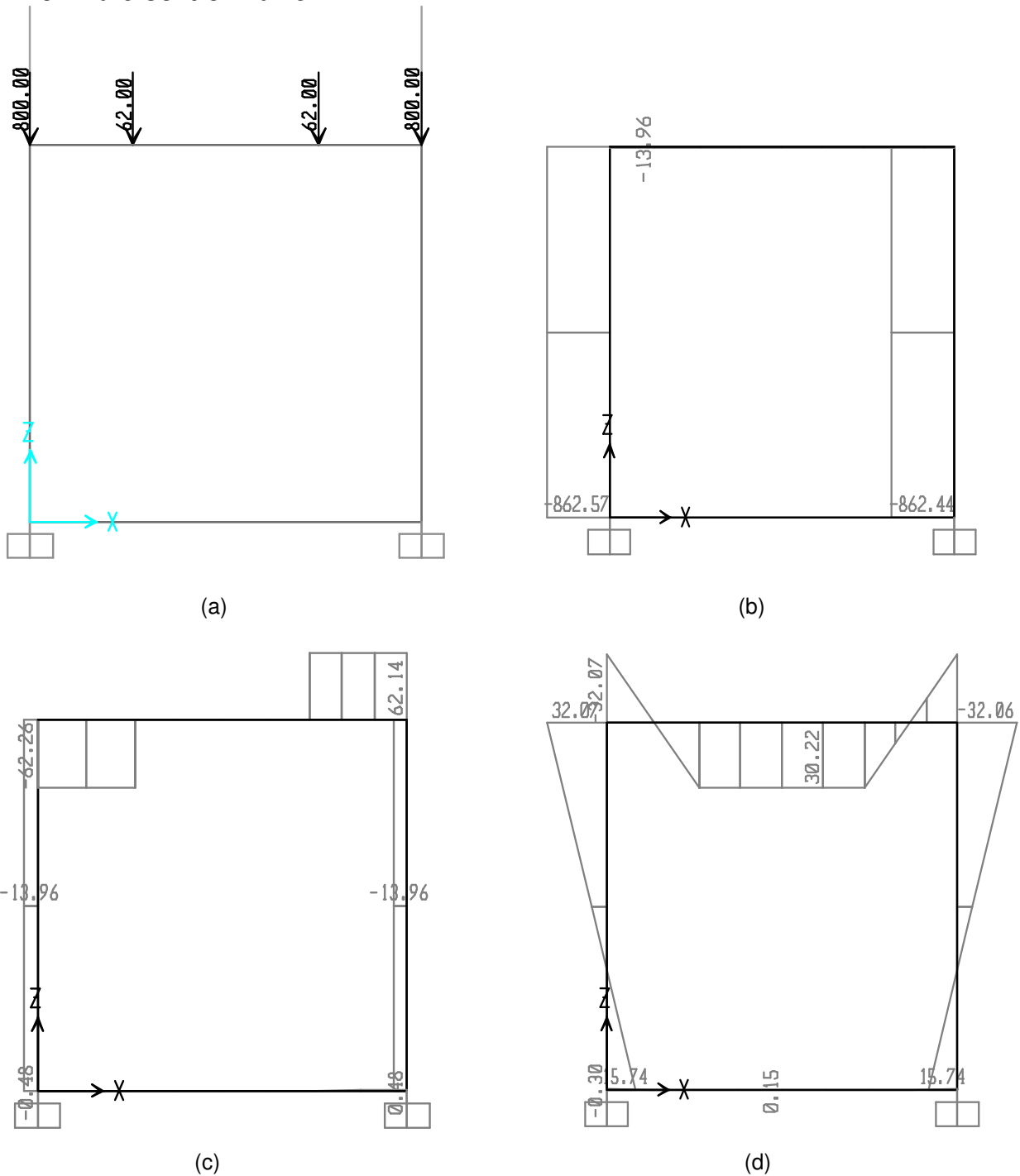


Figure V.2: Gravity loading on the bare control frame and the member forces:  
a) loading (kN); b) member axial forces (kN); c) member shear forces (kN); and  
d) member bending moments (KN·m)

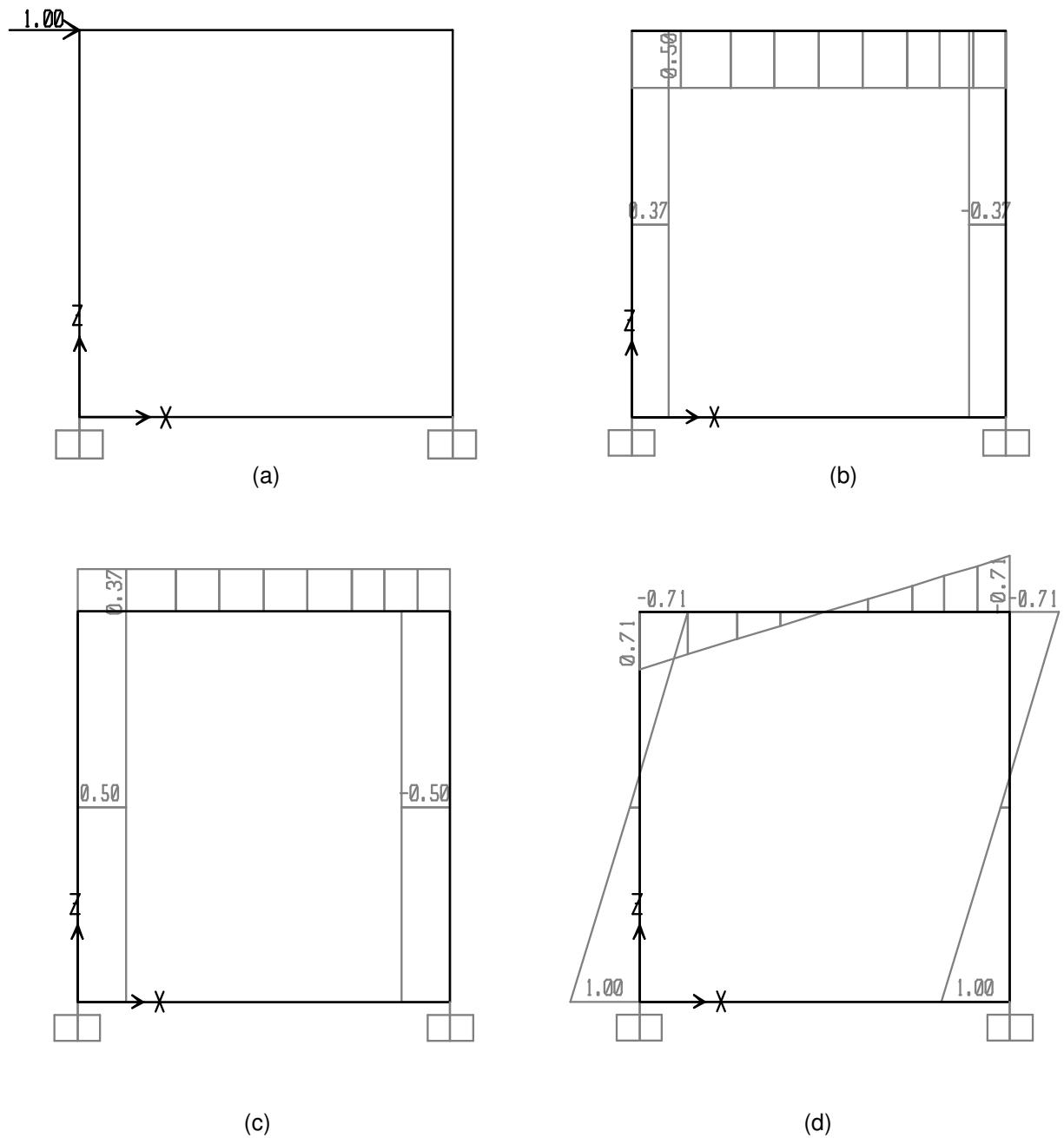
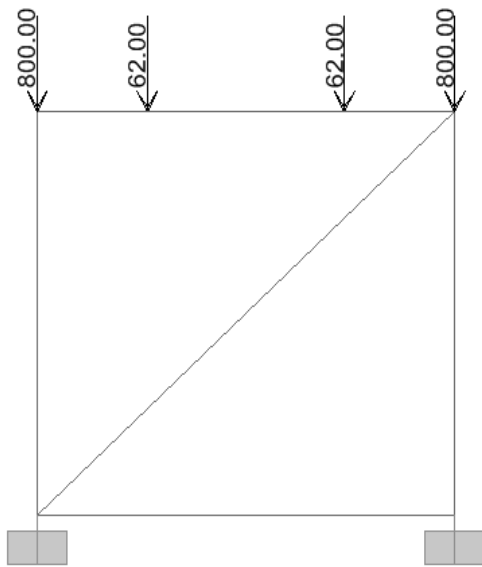
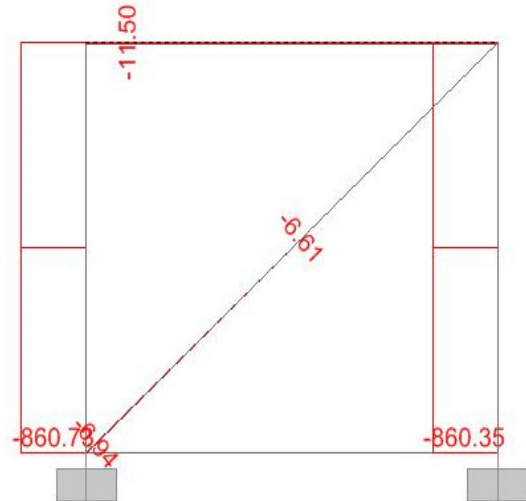


Figure V.3: Unit lateral loading on the bare control frame and the member forces:  
a) loading (kN); b) member axial forces (kN); c) member shear forces (kN); and  
d) members bending moments (kN·m)

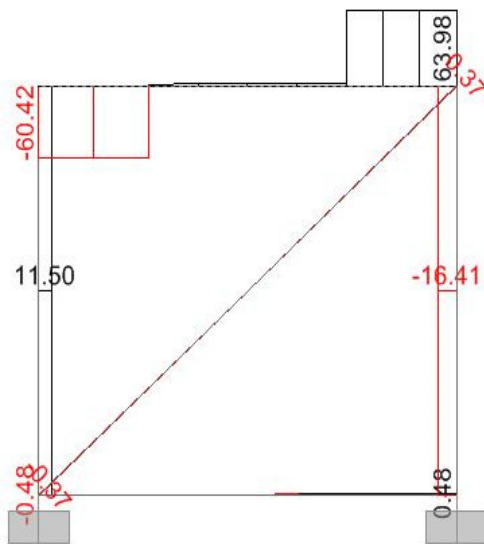
### V.3.2 Retrofitted Frames



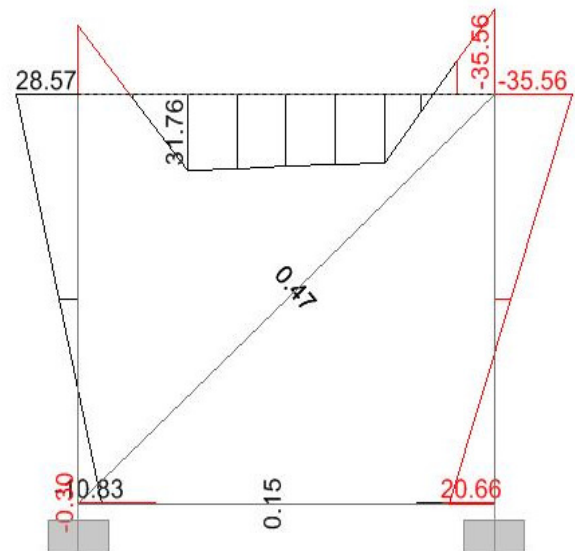
(a)



(b)

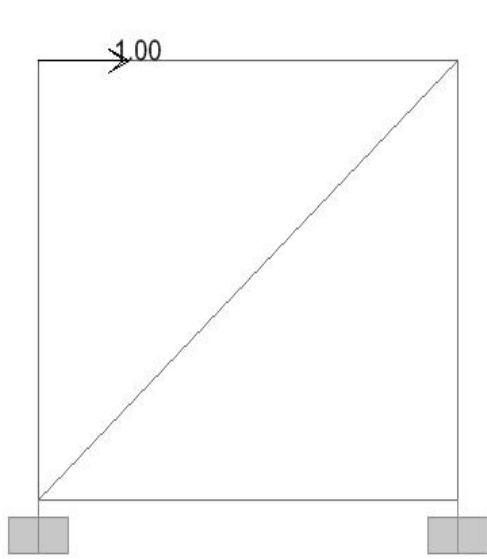


(c)

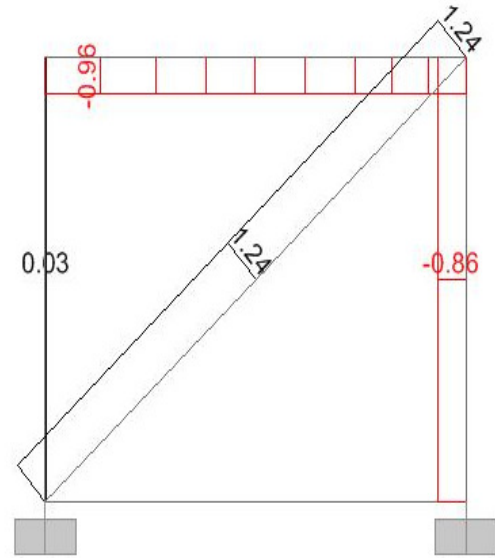


(d)

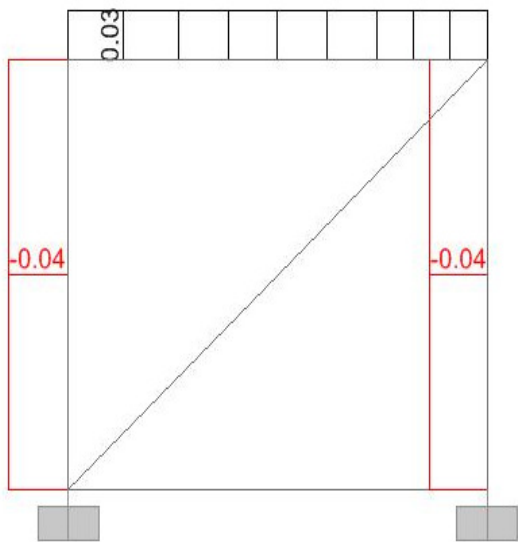
Figure V.4: Gravity loading on the retrofitted frames and the member forces:  
a) loading (kN); b) member axial forces (kN); c) member shear forces (kN); and  
d) member bending moments (kN·m)



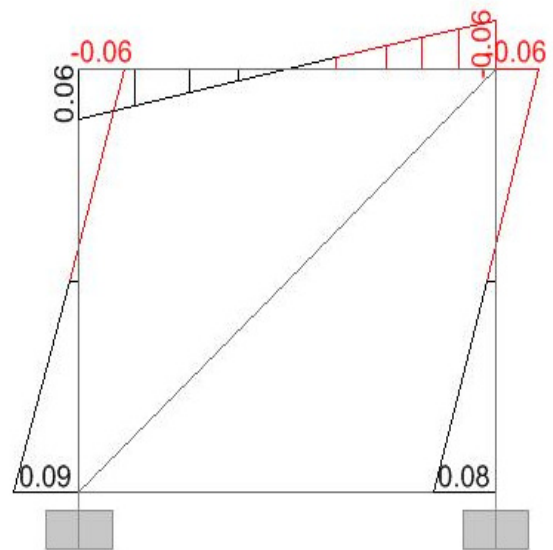
(a)



(b)



(c)



(d)

Figure V.5: Unit lateral loading on the near column of the retrofitted frames and member forces: a) loading (kN); b) member axial forces (kN); c) member shear forces (kN); and d) members bending moments (KN·m)



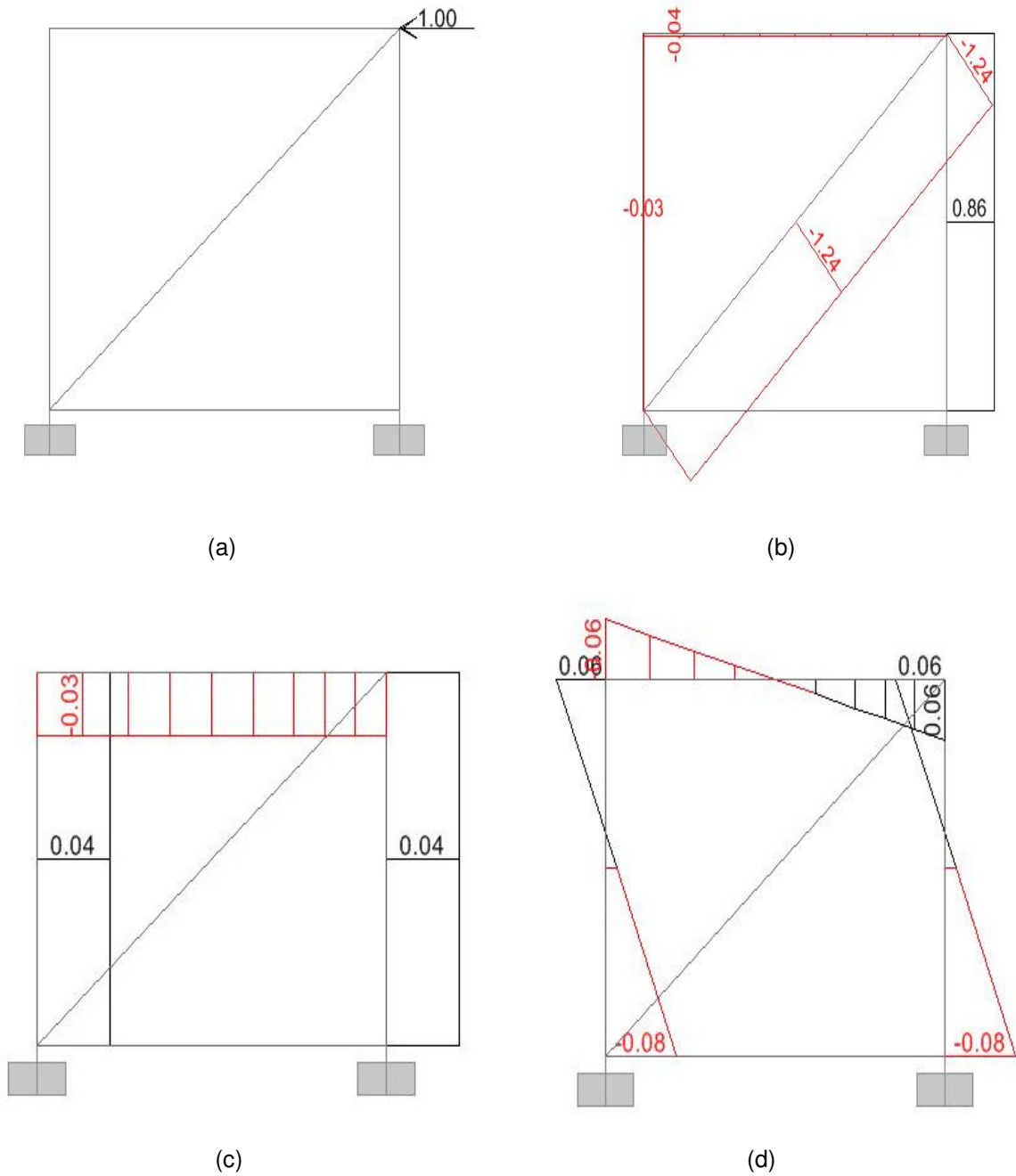


Figure V.6: Unit lateral loading on the far column of the retrofitted frames and member forces: a) loading (kN); b) member axial forces (kN); c) member shear forces (kN); and d) members bending moments (KN·m)

#### **V.4 Frames Shear Capacities: CSA A23.4-04 General Method**

The General Method provides more accurate estimate of the sectional shear capacities and therefore it was used to assess the sectional shear capacities of the columns and beams of the bare control and retrofitted frames. Calculations are presented in the following sections.

### V.4.1 Bare Control Frame Columns

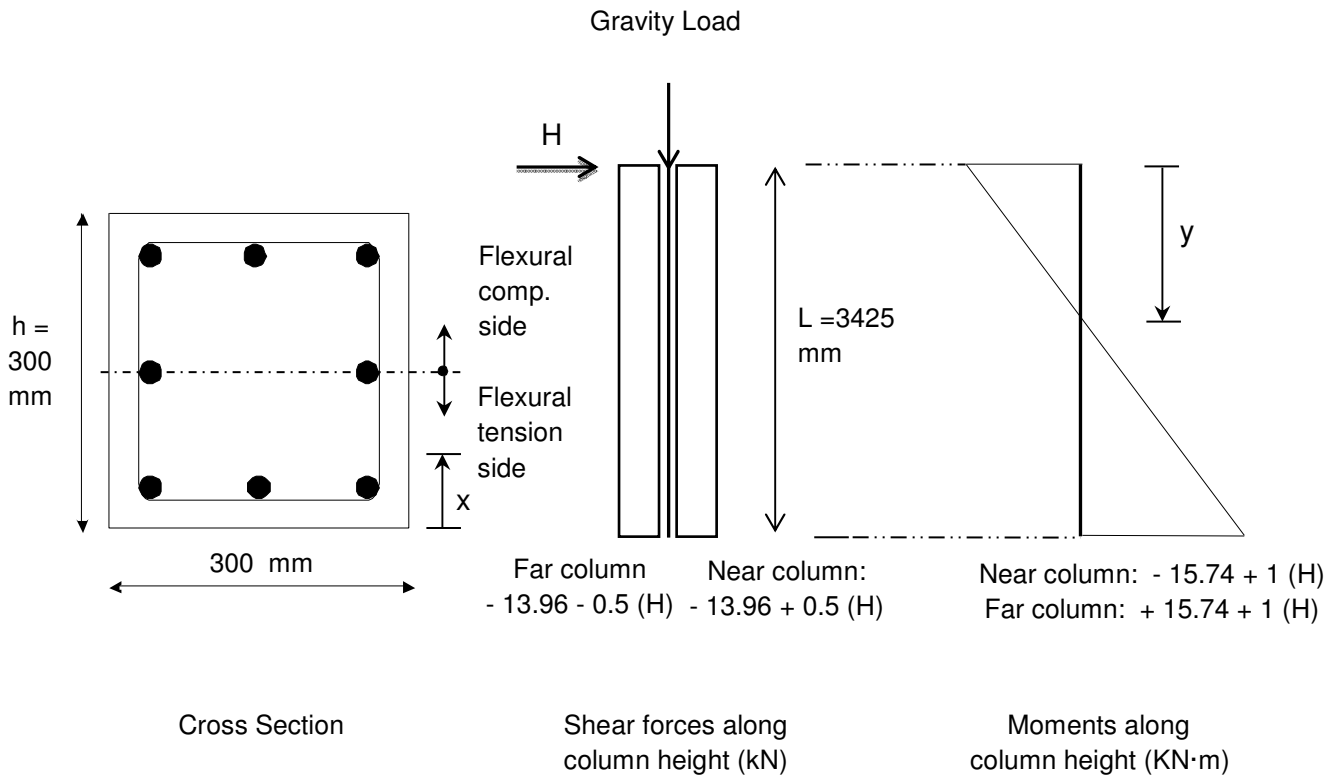


Figure V.7: Application of General Method for the bare control frame columns

#### **Data:**

|                                       |          |                 |
|---------------------------------------|----------|-----------------|
| Stirrups ( $f_y$ )                    | = 481    | MPa             |
| Concrete ( $f'_c$ )                   | = 30     | MPa             |
| Size effect parameter ( $S_{ze}$ )    | = 300    | mm              |
| Section thickness (h)                 | = 300    | mm              |
| Section width ( $b_w$ )               | = 300    | mm              |
| Total stirrup area ( $A_v$ )          | = 200    | mm <sup>2</sup> |
| Stirrup spacing (S)                   | = 200    | mm              |
| Steel Modulus of Elasticity ( $E_s$ ) | = 200000 | MPa             |
| Member height (L)                     | = 3425   | mm              |

Table V.1: Shear capacity of bare control frame columns according to CSA A23.4-04 General Method

| Term                             | Details                                                                | Near Column     |                   | Far Column      |                   |
|----------------------------------|------------------------------------------------------------------------|-----------------|-------------------|-----------------|-------------------|
|                                  |                                                                        | Base            | Beam-column joint | Base            | Beam-column joint |
| $A_s$ tension (mm <sup>2</sup> ) | 4-20M (4 x 300)                                                        | 1200            | 1200              | 1200            | 1200              |
| x (mm)                           | $[(900 \times (25+11.3+19.5 \times 0.5) + (300 \times 150))] / 1200$   | 72.0            | 72.0              | 72.0            | 72.0              |
| d (mm)                           | h - x                                                                  | 228.0           | 228.0             | 228.0           | 228.0             |
| $d_v$ (mm),<br>Greater of        | 0.9 d                                                                  | 205.2           | 205.2             | 205.2           | 205.2             |
|                                  | 0.72 h                                                                 | 216             | 216               | 216             | 216               |
| $\epsilon_x$ (assumed)           | At mid cross section height                                            | 0.002099        | 0.001471          | 0.002101        | 0.001638          |
| $\beta$                          | $\beta = 0.4 \times 1300 / [(1+1500 \epsilon_x) \cdot (1000+S_{ze})]$  | 0.096437828     | 0.12472327        | 0.096368126     | 0.115687182       |
| $\theta^{(0)}$                   | $\theta^{(0)} = 29+7000 \epsilon_x$                                    | 43.6895         | 39.2998           | 43.7035         | 40.4688           |
| $V_c$ (kN)                       | $V_c = \lambda \beta (f'_c)^{1/2} b_w d_v$<br>$\leq 0.25 f'_c b_w d_v$ | 34.2            | 44.3              | 34.2            | 41.1              |
| $V_s$ (kN)                       | $V_s = (A_v f_y d_v \cot \theta) / s$                                  | 108.8           | 126.9             | 108.7           | 121.8             |
| $V_n$ (kN)                       | $V_n = V_c + V_s$                                                      | 143.0           | 171.2             | 142.9           | 162.8             |
| V (kN)                           | From frame analysis                                                    | - 13.96 + 0.5 H | - 13.96 + 0.5 H   | - 13.96 - 0.5 H | - 13.96 - 0.5 H   |

Table V.1 (Cont'd): Shear capacity of bare frame columns according to CSA A23.4-04 General Method

| Term                                                            | Details                                                                                                           | Near Column        |                    | Far Column         |                    |
|-----------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------|--------------------|--------------------|--------------------|--------------------|
|                                                                 |                                                                                                                   | Base               | Beam-column joint  | Base               | Beam-column joint  |
| Applied lateral load, H (kN)                                    | Equalizing $V = V_n$                                                                                              | $2V_n + 27.92$     | $2V_n + 27.92$     | $-2V_n - 27.92$    | $-2V_n - 27.92$    |
| H (kN)                                                          | Absolute value                                                                                                    | 313.9              | 370.3              | 313.7              | 353.6              |
| M (Moment) where the shear capacity is calculated, KN·m         | From frame analysis                                                                                               | $-15.74 + 1 H$     | $+32.07 - 0.71 H$  | $+15.74 + 1 H$     | $-32.07 - 0.71 H$  |
| =                                                               | =                                                                                                                 | 298.2              | -230.9             | 329.5              | -283.1             |
| M (Moment) at the other extreme end, KN·m                       | From frame analysis                                                                                               | $+32.07 - 0.71 H$  | $-15.74 + 1 H$     | $-32.07 - 0.71 H$  | $+15.74 + 1 H$     |
| =                                                               | =                                                                                                                 | -190.8             | 354.6              | -254.8             | 369.3              |
| Distance from beam-column joint height, y, where $M_{col.} = 0$ | $(L * M_{b.j.}) / (M_{base} + M_{b.j.})$                                                                          | 1336               | 1351               | 1494               | 1486               |
| M at $d_v$ from the support, KN·m                               | M at $d_v$ base = $M(L - y - d_v) / (L - y)$<br>M at $d_v$ beam joint = $M(y - d_v) / y$                          | 267.3              | -193.9             | 292.6              | -242.0             |
| N (Axial Load), kN                                              | From frame analysis                                                                                               | $-862.57 + 0.37 H$ | $-862.57 + 0.37 H$ | $-862.57 - 0.37 H$ | $-862.57 - 0.37 H$ |
| =                                                               | =                                                                                                                 | -746.4             | -725.5             | -978.7             | -993.4             |
| $\epsilon_x$ (calculated) at mid cross section height           | $\epsilon_x = [(M \text{ at } d_v, \text{ absolute value}) / d_v] + 0.5 N + V_n / (2.A_s \sigma_{ten} \cdot E_s)$ | 0.002099           | 0.001471           | 0.002101           | 0.001638           |

#### V.4.2 Bare Control Frame Beam

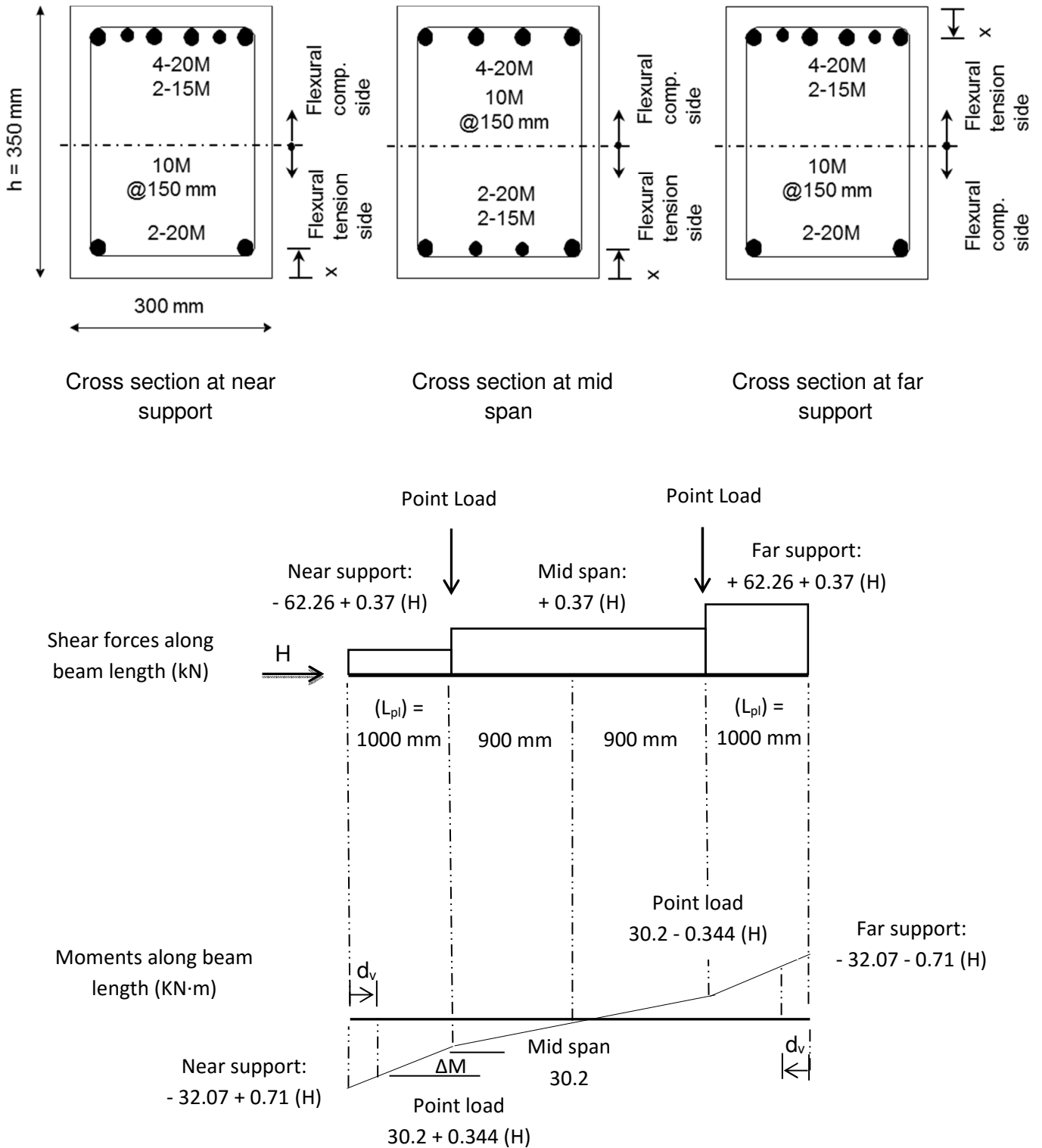


Figure V.8: Application of General Method for bare frame beam

**Data:**

|                                                |          |                 |
|------------------------------------------------|----------|-----------------|
| Stirrup ( $f_y$ )                              | = 481    | MPa             |
| Concrete ( $f'_c$ )                            | = 30     | MPa             |
| Size effect parameter ( $S_{ze}$ )             | = 300    | mm              |
| Section thickness ( $h$ )                      | = 300    | mm              |
| Section width ( $b_w$ )                        | = 300    | mm              |
| Total stirrup area ( $A_v$ )                   | = 200    | mm <sup>2</sup> |
| Stirrup spacing ( $S$ )                        | = 200    | mm              |
| Steel Modulus of Elasticity ( $E_s$ )          | = 200000 | MPa             |
| Member height ( $L$ )                          | = 3425   | Mm              |
| Length from support to point load ( $L_{pl}$ ) | = 1000   | mm              |



Table V.2: Shear capacity of bare frame beam according to CSA A23.4-04 General Method

| Term                             | Details                                                                                                                                                                                                          | Near Support | Mid Span    | Far Support |
|----------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------|-------------|-------------|
| $A_s$ tension (mm <sup>2</sup> ) | at near support: 2-20M,<br>at mid span: 2-20M + 2-15M,<br>at far support: 4-20M + 2-15M                                                                                                                          | 600          | 1000        | 1600        |
| x (mm)                           | at near support: $[600 \times (25 + 11.3 + 19.5 \times 0.5)] / 600$<br>at mid span: $[1000 \times (25 + 11.3 + ((19.5 + 16)/4))] / 1000$<br>at far support: $[1600 \times (25 + 11.3 + ((19.5 + 16)/4))] / 1600$ | 46.1         | 45.2        | 45.2        |
| d (mm)                           | h - x                                                                                                                                                                                                            | 304.0        | 304.8       | 304.8       |
| $d_v$ (mm),<br>Greater of        | 0.9 d                                                                                                                                                                                                            | 273.6        | 274.3       | 274.3       |
|                                  | 0.72 h                                                                                                                                                                                                           | 252          | 252         | 252         |
| $\epsilon_x$ (assumed)           | At mid cross section height                                                                                                                                                                                      | 0.004430     | 0.000558    | 0.001919    |
| $\beta$                          | $\beta = 0.4 \times 1300 / [(1 + 1500 \epsilon_x) \cdot (1000 + S_{ze})]$                                                                                                                                        | 0.052316749  | 0.217746326 | 0.103132654 |
| $\theta^{(0)}$                   | $\theta^{(0)} = 29 + 7000 \epsilon_x$                                                                                                                                                                            | 60.01343     | 32.906      | 42.433      |
| $V_c$ (kN)                       | $V_c = \lambda \beta (f'_c)^{1/2} b_w d_v$<br>$\leq 0.25 f'_c b_w d_v$                                                                                                                                           | 23.5         | 98.2        | 46.5        |
| $V_s$ (kN)                       | $V_s = (A_v f_y d_v \cot \theta) / s$                                                                                                                                                                            | 101.2        | 271.9       | 192.5       |
| $V_n$ (kN)                       | $V_n = V_c + V_s$                                                                                                                                                                                                | 124.8        | 370.1       | 239.0       |

Table V.2 (Cont'd): Shear capacity of bare frame beam according to CSA A23.4-04 General Method

| Term                                                    | Details                                                                                                       | Near Support           | Mid Span        | Far Support            |
|---------------------------------------------------------|---------------------------------------------------------------------------------------------------------------|------------------------|-----------------|------------------------|
| V (kN)                                                  | From frame analysis                                                                                           | - 62.26 + 0.37 H       | 0 + 0.37 H      | 62.26 + 0.37 H         |
| Applied lateral load, H (kN)                            | Equalizing $V = V_n$                                                                                          | $(V_n + 62.26) / 0.37$ | $V_n / 0.37$    | $(V_n - 62.26) / 0.37$ |
| H (kN)                                                  | Absolute value                                                                                                | 505.4                  | 1000.2          | 477.5                  |
| M (Moment) where the shear capacity is calculated, KN·m | From frame analysis                                                                                           | - 32.07 + 0.71 H       | 30.2 + 0 H      | - 32.07 – 0.71 H       |
| =                                                       | =                                                                                                             | 326.8                  | 30.2            | -371.1                 |
| M <sub>pl.</sub> at adjacent point load, KN·m           | From frame analysis                                                                                           | 30.2 + 0.344 H         | 30.2+ 0 H       | 30.2 - 0.344 H         |
| =                                                       | =                                                                                                             | 204.1                  | 30.2            | -134.1                 |
| $\Delta M$ (KN·m)                                       | $\Delta M = [(L_{pl.} - d_v) (M - M_{pl.})] / (L_{pl.})$                                                      | 89                     | 0               | -172                   |
| M at $d_v$ from supports, KN·m<br>M at mid span, KN·m   | M at $d_v = M_{pl.} + \Delta M$                                                                               | 293.2                  | 30.2            | -306.1                 |
| N (Axial Load), kN                                      | From frame analysis                                                                                           | - 13.96 - 0.5 H        | - 13.96 - 0.5 H | - 13.96 - 0.5 H        |
| =                                                       | =                                                                                                             | -266.7                 | -514.0          | -252.7                 |
| $\epsilon_x$ (calculated) at mid cross section height   | $\epsilon_x = [(M \text{ at } d_v, \text{ absolute value}) / d_v] + 0.5N + V_n] / (2.A_s \sigma_{ten} . E_s)$ | 0.004430               | 0.000558        | 0.001919               |

### V.4.3 Retrofitted Frame Columns

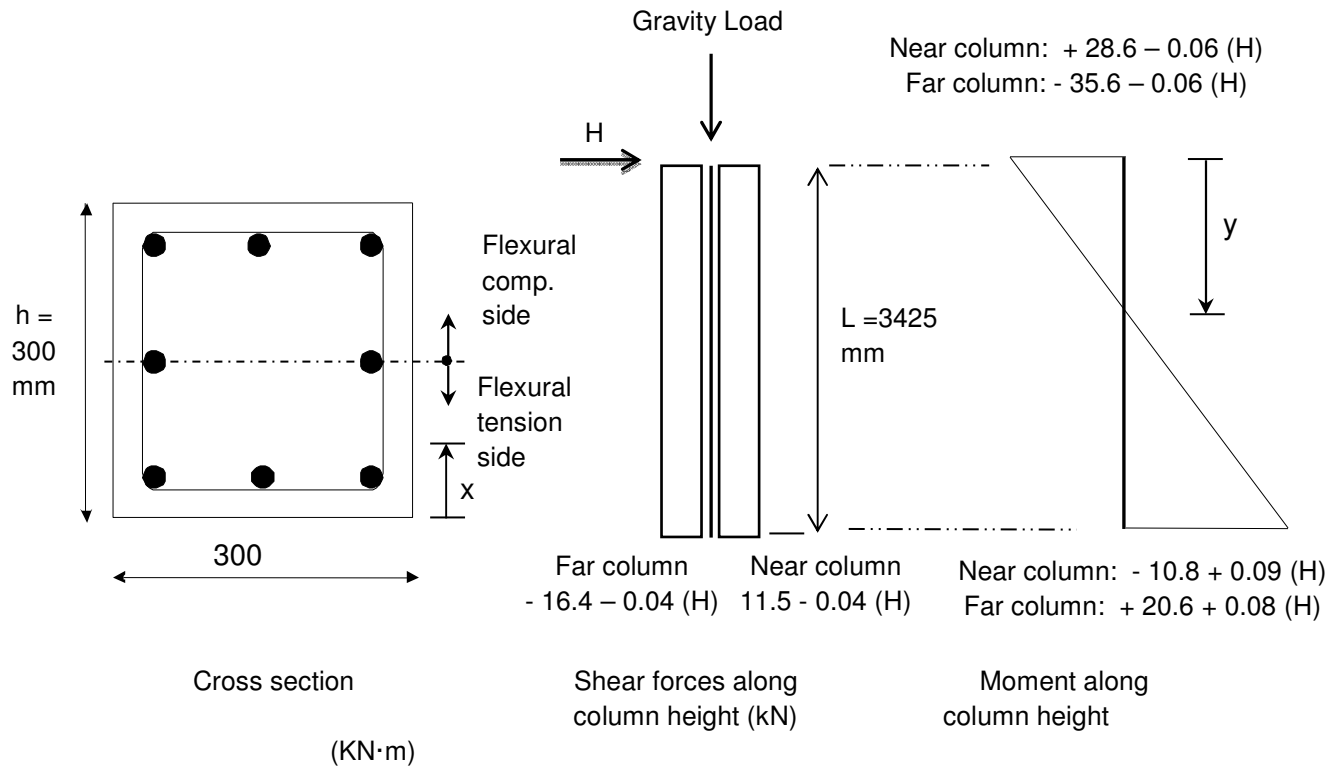


Figure V.9: Application of General Method for columns of the retrofitted frames

#### **Data:**

|                                       |          |                 |
|---------------------------------------|----------|-----------------|
| Stirrup ( $f_y$ )                     | = 481    | MPa             |
| Concrete ( $f'_c$ )                   | = 31     | MPa             |
| Size effect parameter ( $S_{ze}$ )    | = 300    | mm              |
| Section thickness (h)                 | = 300    | mm              |
| Section width ( $b_w$ )               | = 300    | mm              |
| Total stirrup area ( $A_v$ )          | = 200    | mm <sup>2</sup> |
| Stirrup spacing (S)                   | = 200    | mm              |
| Steel Modulus of Elasticity ( $E_s$ ) | = 200000 | MPa             |
| Member height (L)                     | = 3425   | mm              |

Table V.3: Shear capacity of retrofitted frame columns according to CSA A23.4-04 General Method

| Term                             | Details                                                               | Near Column     |                   | Far Column        |                   |
|----------------------------------|-----------------------------------------------------------------------|-----------------|-------------------|-------------------|-------------------|
|                                  |                                                                       | Base            | Beam-column joint | Base              | Beam-column joint |
| $A_s$ tension (mm <sup>2</sup> ) | 4-20M (4 x 300)                                                       | 1200            | 1200              | 1200              | 1200              |
| $x$ (mm)                         | $[(900 \times (25+11.3+19.5 \times 0.5) + (300 \times 150))] / 1200$  | 72.0            | 72.0              | 72.0              | 72.0              |
| $d$ (mm)                         | $h - x$                                                               | 228.0           | 228.0             | 228.0             | 228.0             |
| $d_v$ (mm),<br>Greater of        | $0.9 d$                                                               | 205.2           | 205.2             | 205.2             | 205.2             |
|                                  | $0.72 h$                                                              | 216             | 216               | 216               | 216               |
| $\epsilon_x$ (assumed)           | At mid cross section height                                           | 0.002041        | 0.001371          | -0.0020           | -0.0020           |
| $\beta$                          | $\beta = 0.4 \times 1300 / [(1+1500 \epsilon_x) \cdot (1000+S_{ze})]$ | 0.098493056     | 0.130839746       | -0.2              | -0.2              |
| $\theta^{(0)}$                   | $\theta^{(0)} = 29 + 7000 \epsilon_x$                                 | 43.2856         | 38.60015          | 15                | 15                |
| $V_c$ (kN)                       | $V_c = \lambda B (f'_c)^{1/2} b_w d_v$<br>$\leq 0.25 f'_c b_w d_v$    | 35.5            | 47.2              | -72.2             | -72.2             |
| $V_s$ (kN)                       | $V_s = (A_v f_y d_v \cot \theta) / s$                                 | 110.3           | 130.1             | 387.7             | 387.7             |
| $V_n$ (kN)                       | $V_n = V_c + V_s$                                                     | 145.8           | 177.4             | 315.6             | 315.6             |
| $V$ (kN)                         | From frame analysis                                                   | 11.5 - 0.04 (H) | 11.5 - 0.04 (H)   | - 16.4 - 0.04 (H) | - 16.4 - 0.04 (H) |

Table V.3 (Cont'd): Shear capacity of retrofitted frame columns according to CSA A23.4-04 General Method

| Term                                                            | Details                                                                                                             | Near Column         |                     | Far Column          |                     |
|-----------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------|---------------------|---------------------|---------------------|---------------------|
|                                                                 |                                                                                                                     | Base                | Beam-column joint   | Base                | Beam-column joint   |
| Applied lateral load, H (kN)                                    | Equalizing $V = V_n$                                                                                                | 25 ( $V_n - 11.5$ ) | 25 ( $V_n - 11.5$ ) | 25 ( $V_n + 16.4$ ) | 26 ( $V_n + 16.4$ ) |
| H (kN)                                                          | Absolute value                                                                                                      | 3359                | 4146                | 8300                | 8300                |
| M (Moment) where the shear capacity is calculated, KN·m         | From frame analysis                                                                                                 | - 10.8 + 0.09 (H)   | 28.6 - 0.06 (H)     | 20.6 + 0.08 (H)     | - 35.6 - 0.06 (H)   |
| =                                                               | =                                                                                                                   | 291.5               | -220.2              | 643.4               | -533.6              |
| M (Moment) at the other extreme end, (KN·m)                     | From frame analysis                                                                                                 | 28.6 - 0.06 (H) =   | - 10.8 + 0.09 (H)   | - 35.6 - 0.06 (H)   | 20.6 + 0.08 (H)     |
| =                                                               | =                                                                                                                   | -172.9              | 362.4               | -533.6              | 684.6               |
| Distance from Beam-column joint height, y, where $M_{col.} = 0$ | $(L * M_{b.j.}) / (M_{base} + M_{b.j.})$                                                                            | 1275                | 1295                | 1553                | 1500                |
| M at $d_v$ from the support, KN·m                               | M at $d_v$ base = $M (L - y - d_v) / (L - y)$<br>M at $d_v$ beam Junction = $M (y - d_v) / y$                       | 262.2               | -183.4              | 569.1               | -456.8              |
| N (Axial Load), kN                                              | From frame analysis                                                                                                 | - 861 + 0.03 (H)    | - 861 + 0.03 (H)    | - 861 - 0.86 (H)    | - 861 - 0.86 (H)    |
| =                                                               | =                                                                                                                   | -760.2              | -736.6              | -7998.7             | -7998.7             |
| $\epsilon_x$ (calculated) at mid cross section height           | $*\epsilon_x = [(M \text{ at } d_v, \text{ absolute value}) / d_v] + 0.5 N + V_n] / (2.A_s \sigma_{ten} \cdot E_s)$ | 0.002041            | 0.001371            | -0.000324           | -0.000485           |

\*  $\epsilon_x = [(M \text{ at } d_v, \text{ absolute value}) / d_v] + 0.5 N + V_n] / (2.(A_s \sigma_{ten} \cdot E_s + E_c A_{ct}))$  used if the value of  $\epsilon_x$  is negative and should be  $\geq - 0.002$  (CSA A23.4 CL 11.36.4)

#### V.4.4 Retrofitted Frame Beam

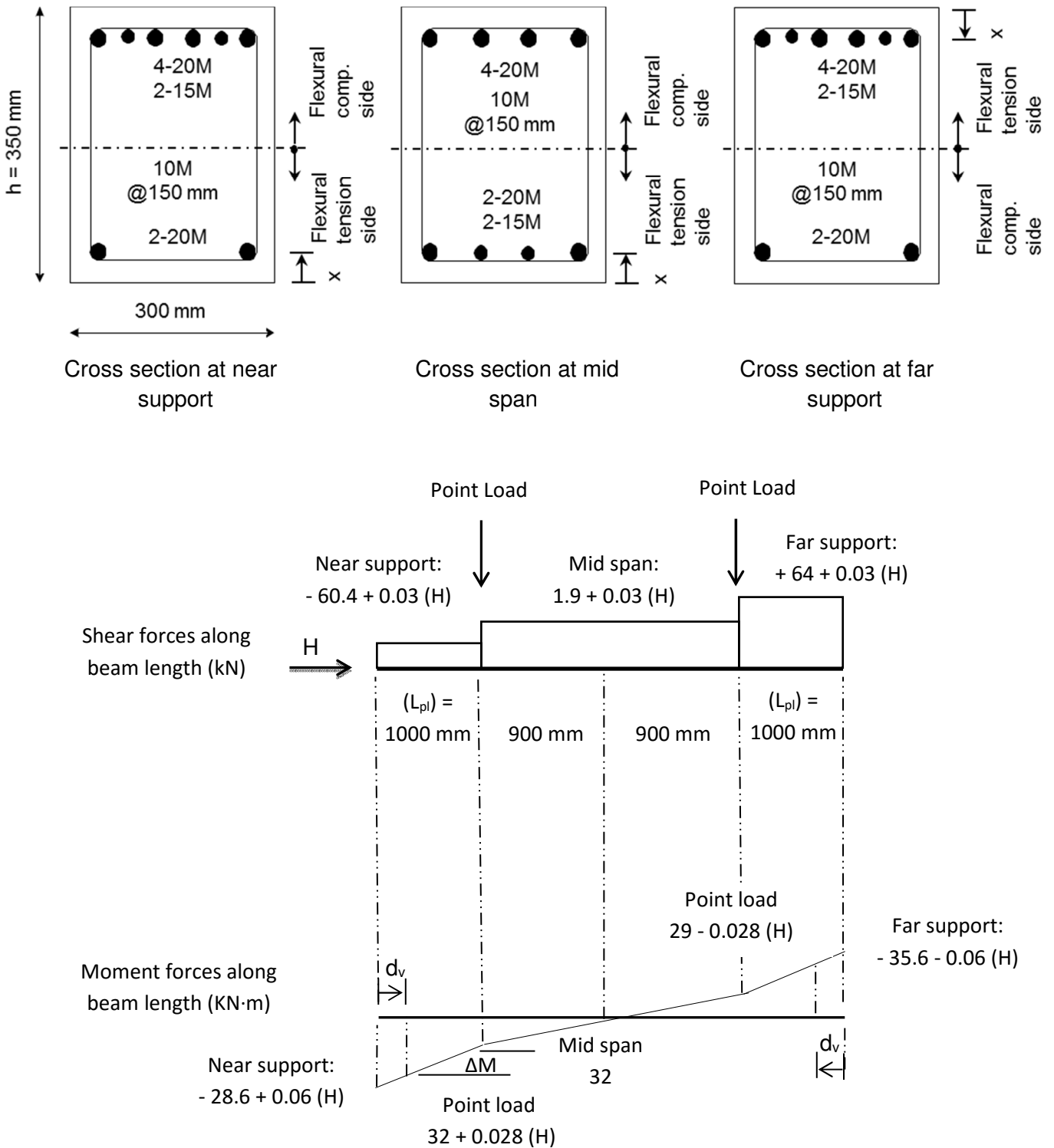


Figure V.10: Application of General Method for retrofitted frame beam

**Data:**

|                                                |          |                 |
|------------------------------------------------|----------|-----------------|
| Stirrup ( $f_y$ )                              | = 481    | MPa             |
| Concrete ( $f'_c$ )                            | = 31     | MPa             |
| Size effect parameter ( $S_{ze}$ )             | = 300    | mm              |
| Section thickness ( $h$ )                      | = 300    | mm              |
| Section width ( $b_w$ )                        | = 300    | mm              |
| Total stirrup area ( $A_v$ )                   | = 200    | mm <sup>2</sup> |
| Stirrup spacing ( $S$ )                        | = 200    | mm              |
| Steel Modulus of Elasticity ( $E_s$ )          | = 200000 | MPa             |
| Member height ( $L$ )                          | = 3425   | Mm              |
| Length from support to point load ( $L_{pl}$ ) | = 1000   | mm              |



Table V.4: Shear capacity of retrofitted frame beam according to CSA A23.4-04 General Method

| Term                             | Details                                                                                                                                                                                                          | Near Support | Mid Span | Far Support |
|----------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------|----------|-------------|
| $A_s$ tension (mm <sup>2</sup> ) | at near support: 2-20M,<br>at mid span: 2-20M + 2-15M,<br>at far support: 4-20M + 2-15M                                                                                                                          | 600          | 1000     | 1600        |
| x (mm)                           | at near support: $[600 \times (25 + 11.3 + 19.5 \times 0.5)] / 600$<br>at mid span: $[1000 \times (25 + 11.3 + ((19.5 + 16)/4))] / 1000$<br>at far support: $[1600 \times (25 + 11.3 + ((19.5 + 16)/4))] / 1600$ | 46.1         | 45.2     | 45.2        |
| d (mm)                           | h - x                                                                                                                                                                                                            | 304.0        | 304.8    | 304.8       |
| $d_v$ (mm),<br>Greater of        | 0.9 d                                                                                                                                                                                                            | 273.6        | 274.3    | 274.3       |
|                                  | 0.72 h                                                                                                                                                                                                           | 252          | 252      | 252         |
| $\epsilon_x$ (assumed)           | At mid cross section height                                                                                                                                                                                      | -0.0020      | -0.0020  | -0.0020     |
| $\beta$                          | $\beta = 0.4 \times 1300 / [(1 + 1500 \epsilon_x) (1000 + S_{ze})]$                                                                                                                                              | -0.2         | -0.2     | -0.2        |
| $\theta^{(0)}$                   | $\theta^{(0)} = 29 + 7000 \epsilon_x$                                                                                                                                                                            | 15           | 15       | 15          |
| $V_c$ (kN)                       | $V_c = \lambda B (f'_c)^{1/2} b_w d_v$<br>$\leq 0.25 f'_c b_w d_v$                                                                                                                                               | -91.4        | -91.6    | -91.6       |
| $V_s$ (kN)                       | $V_s = (A_v f_y d_v \cot \theta) / s$                                                                                                                                                                            | 654.8        | 656.6    | 656.6       |
| $V_n$ (kN)                       | $V_n = V_c + V_s$                                                                                                                                                                                                | 563.4        | 565.0    | 565.0       |

Table V.4 (Cont'd): Shear capacity of retrofitted frame beam according to CSA A23.4-04 General Method

| Term                                                    | Details                                                                                                          | Near Support          | Mid Span             | Far Support           |
|---------------------------------------------------------|------------------------------------------------------------------------------------------------------------------|-----------------------|----------------------|-----------------------|
| V (kN)                                                  | From frame analysis                                                                                              | - 60.4 + 0.03 (H)     | 1.9 + 0.03 (H)       | 64 + 0.03 (H)         |
| Applied lateral load, H (kN)                            | Equalizing $V = V_n$                                                                                             | $(V_n + 60.4) / 0.03$ | $(V_n - 1.9) / 0.03$ | $(V_n - 60.4) / 0.03$ |
| H (kN)                                                  | Absolute value                                                                                                   | 20792.2               | 18769.6              | 16819.6               |
| M (Moment) where the shear capacity is calculated, KN·m | From frame analysis                                                                                              | - 28.6 + 0.06 (H)     | 31.8 + 0 (H)         | - 35.6 - 0.06 (H)     |
| =                                                       | =                                                                                                                | 1218.9                | 31.8                 | -973.6                |
| $M_{pl.}$ at the adjacent point load, KN·m              | From frame analysis                                                                                              | 32 + 0.028 (H)        | 31.8                 | 29 - 0.028 (H)        |
| =                                                       | =                                                                                                                | 614.2                 | 31.8                 | -441.9                |
| $\Delta M$ (KN·m)                                       | $\Delta M = [(L_{pl.} - d_v) (M - M_{pl.})] / (L_{pl.})$                                                         | 439                   | 0                    | -386                  |
| M at $d_v$ from supports, KN·m<br>M at mid span, KN·m   | M at $d_v = M_{pl.} + \Delta M$                                                                                  | 1053.5                | 31.8                 | -827.7                |
| N (Axial Load), kN                                      | From frame analysis                                                                                              | - 11.5 - 0.96 (H)     | - 11.5 - 0.96 (H)    | - 11.5 - 0.96 (H)     |
| =                                                       | =                                                                                                                | -19972.0              | -18030.3             | -16158.3              |
| $\epsilon_x$ (calculated) at mid cross section height   | $\epsilon_x = [(M \text{ at } d_v, \text{ absolute value}) / d_v] + 0.5 N + V_n] / (2 \cdot A_{sten} \cdot E_s)$ | -0.001750             | -0.002637            | -0.001245             |

\*  $\epsilon_x = [(M \text{ at } d_v, \text{ absolute value}) / d_v] + 0.5 N + V_n] / 2 \cdot (A_{sten} \cdot E_s + E_c A_{ct})$  used if the value of  $\epsilon_x$  is negative and should be  $\geq - 0.002$  (CSA A23.4 CL 11.36.4)

## V.5 Elastic deformations of BRB steel core bars

- Material Properties:

Bar AISI 12L14 carbon steel

$f_y$  (MPa) = 480 MPa

$E$  (MPa) = 200500

AISI Type 304 stainless (non-constant cross section)

$f_y$  (MPa) = 303

$E$  (MPa) = 199200

AISI 4140 chrome-molybdenum (non-constant cross section)

$f_y$  (MPa) = 390

$E$  (MPa) = 190500

- Geometric Properties:

Bar AISI 12L14 carbon steel brace bar (Figure 6.35)

$L_t$  (mm) = 2700 = total bar length

AISI 304 stainless and AISI 4140 chrome-molybdenum (Figure 6.35)

$L_r$  (mm) = 1350 = length of reduced area

$L_t$  (mm) = 2436 = total bar length

Diam. of un-reduced length (mm) = 44.5 (Diameter reduction due to threading is ignored)

Area of un-reduced length (mm<sup>2</sup>) = 1555.28

Diam. of reduced length (mm) = 31.8

Area of reduced length (mm<sup>2</sup>) = 794.23

- Elastic Deformations:

- Bar AISI 12L14 carbon steel (constant cross section):

$$\Delta L_e \text{ (mm)} = (f_y L_t / E) = 5.8$$

- AISI 304 stainless and AISI 4140 chrome-molybdenum brace bars (non-constant cross sections):

$$\Delta L_e \text{ (mm)} = (f_y L_r / E) + [(f_y L_t / E) (\sigma_{unr} / \sigma_r) - (f_y L_t / E) (\sigma_{unr} / \sigma_r) (L_r / L_t)]$$

$$= (f_y L_t / E) [\lambda + \eta (1 - \lambda)], \text{ Where } \lambda = L_r / L_t, \text{ and } \eta = \sigma_{unr} / \sigma_r$$

$$= (\text{Elastic deformation of the reduced bar length}) + [\text{elastic deformation of the un-reduced bar length}]$$

$$\Delta L_e \text{ (mm)} = 2.05 + 0.84 = 2.9 \text{ (AISI 304 stainless brace bar)}$$

$$\Delta L_e \text{ (mm)} = 2.76 + 1.14 = 3.9 \text{ (AISI 4140 chrome-molybdenum brace bar)}$$

## Appendix VI: Instrumentation Data

For this research project, tests were performed on limited ductility frames including: bare control frame (BCF) and repaired and non-damaged frames (RRF/RF) to assess the benefit of retrofitting. These frames, including the BRB retrofitting system, were well instrumented with Displacement Cable Transducers (DCT), Linear Variable Displacement Transducers (LVDT), and strain gauges. The strain gauges were placed on the internal reinforcement and the steel core bars of the BRB; while the LVDTs were used to record vertical displacements of the columns at heights of 25 mm and 300 mm from the foundation level. The data recorded from the instrumentation for each test is provided herein.

### VI.A Bare Control Frame (BCF) Test Data

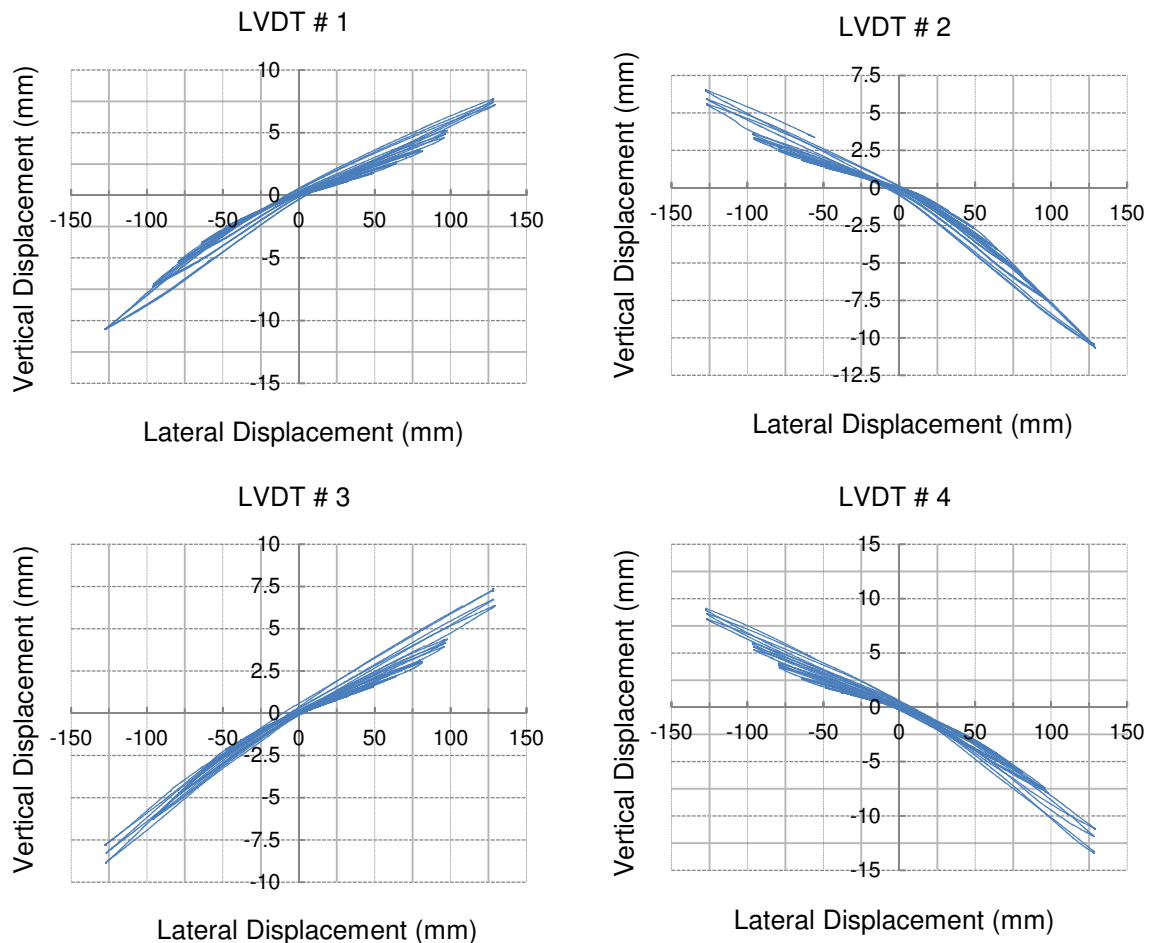


Figure VI.1: Frame lateral displacement vs. vertical displacement recorded by LVDTs 1-4 (300 mm height from foundation of columns)

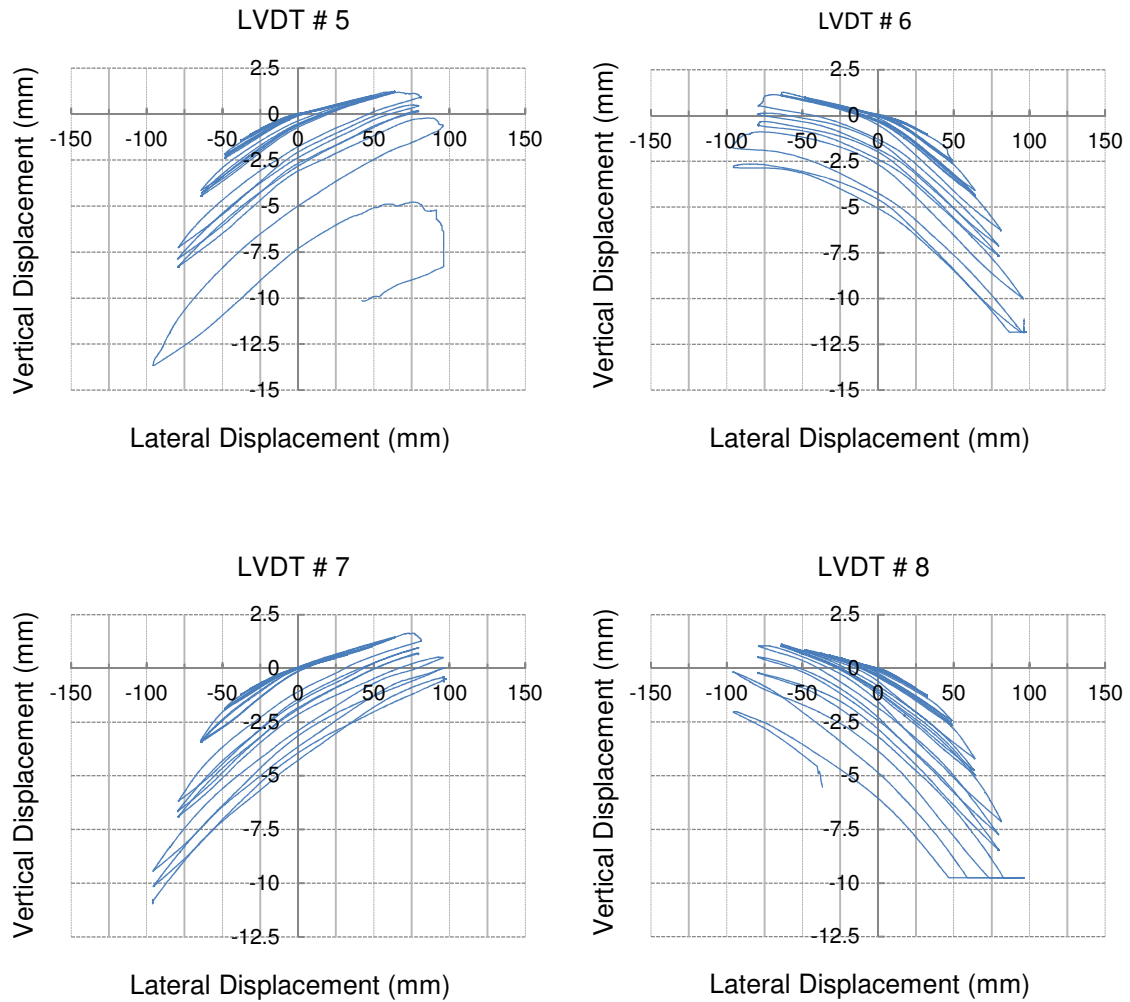


Figure VI.2: Frame lateral displacement vs. vertical displacement recorded by LVDTs 5-8 (25 mm height from foundation of columns)

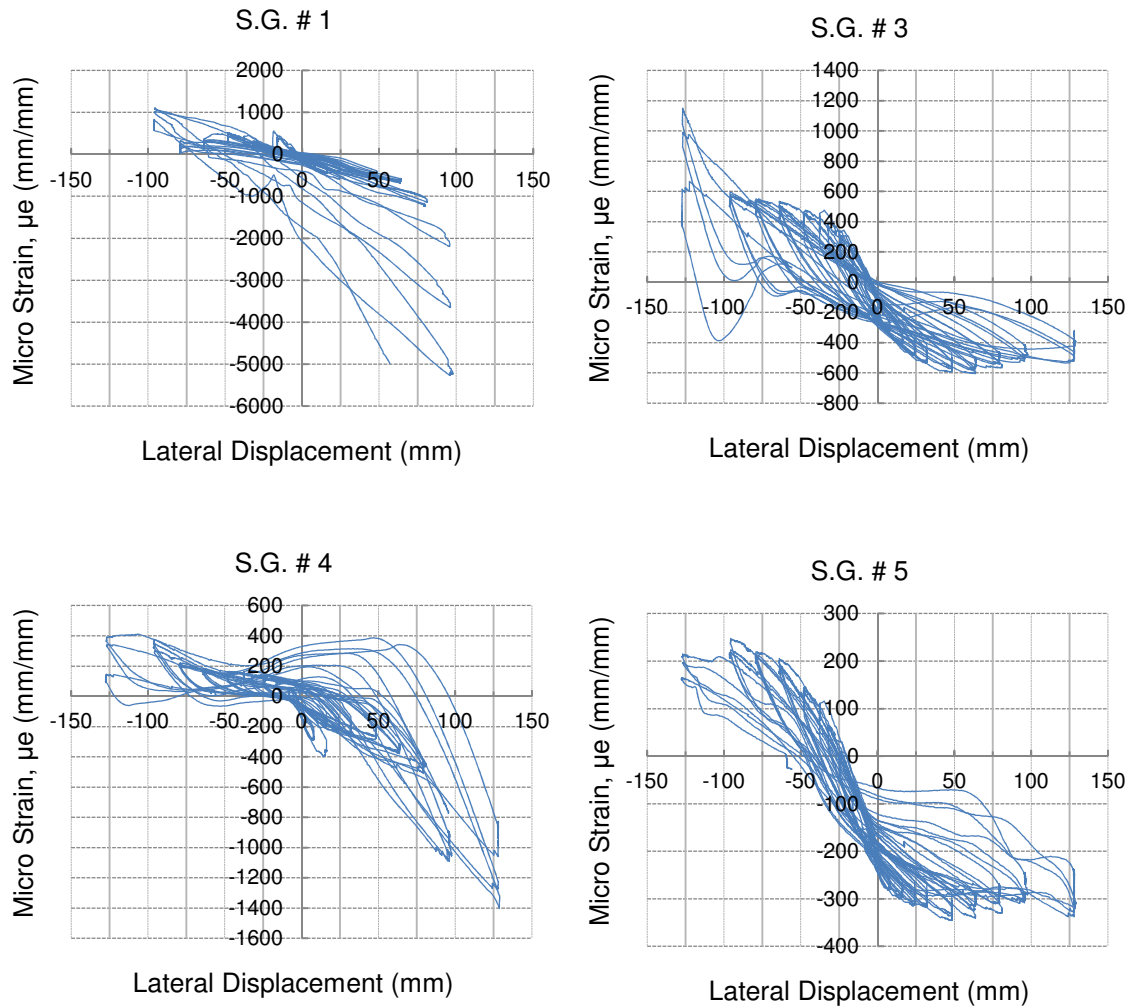


Figure VI.3: Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at far column base (Outer face)

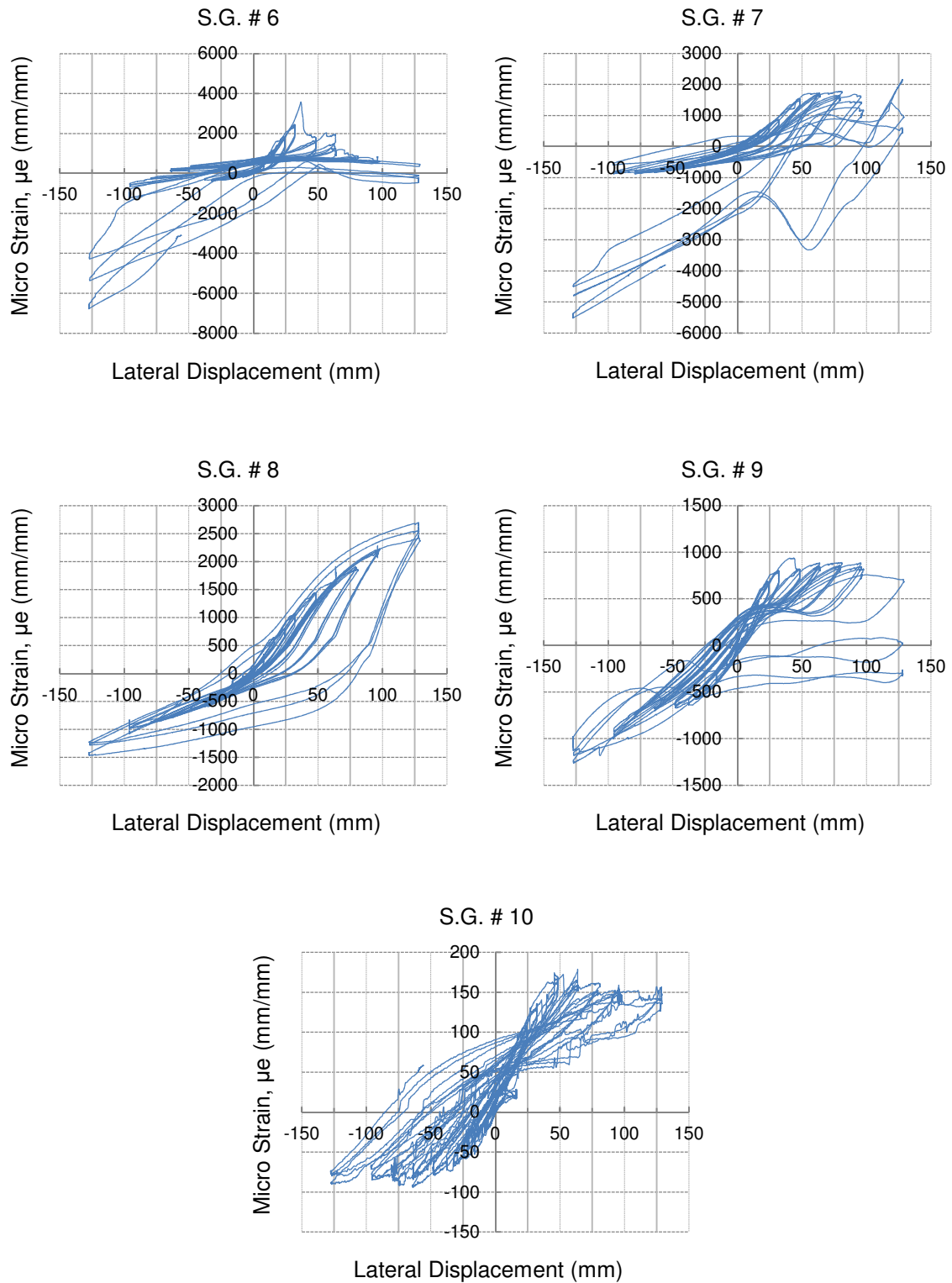


Figure VI.4: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars at far column base (Inner face)



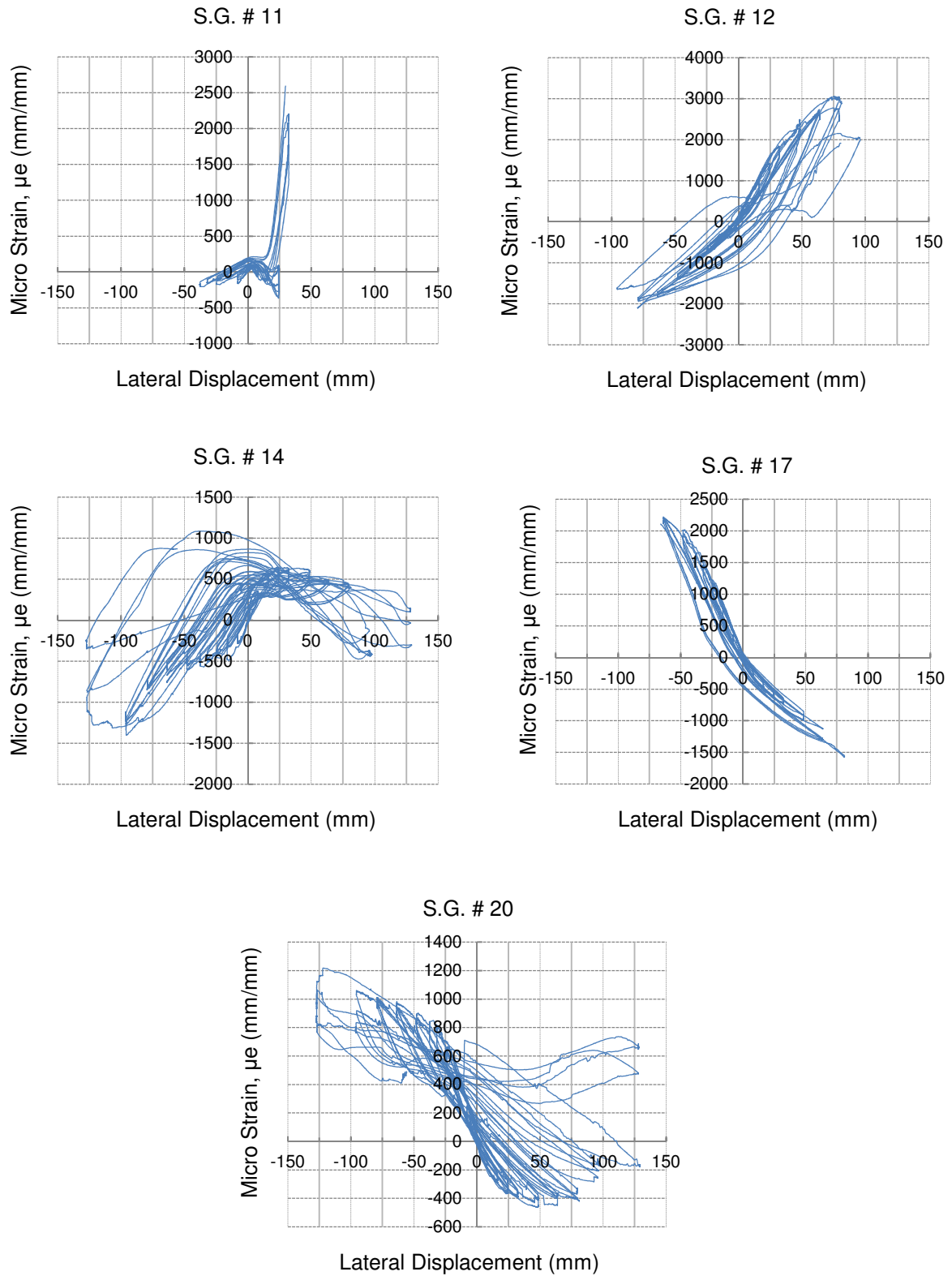


Figure VI.5: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars at near column base (Outer and inner faces)

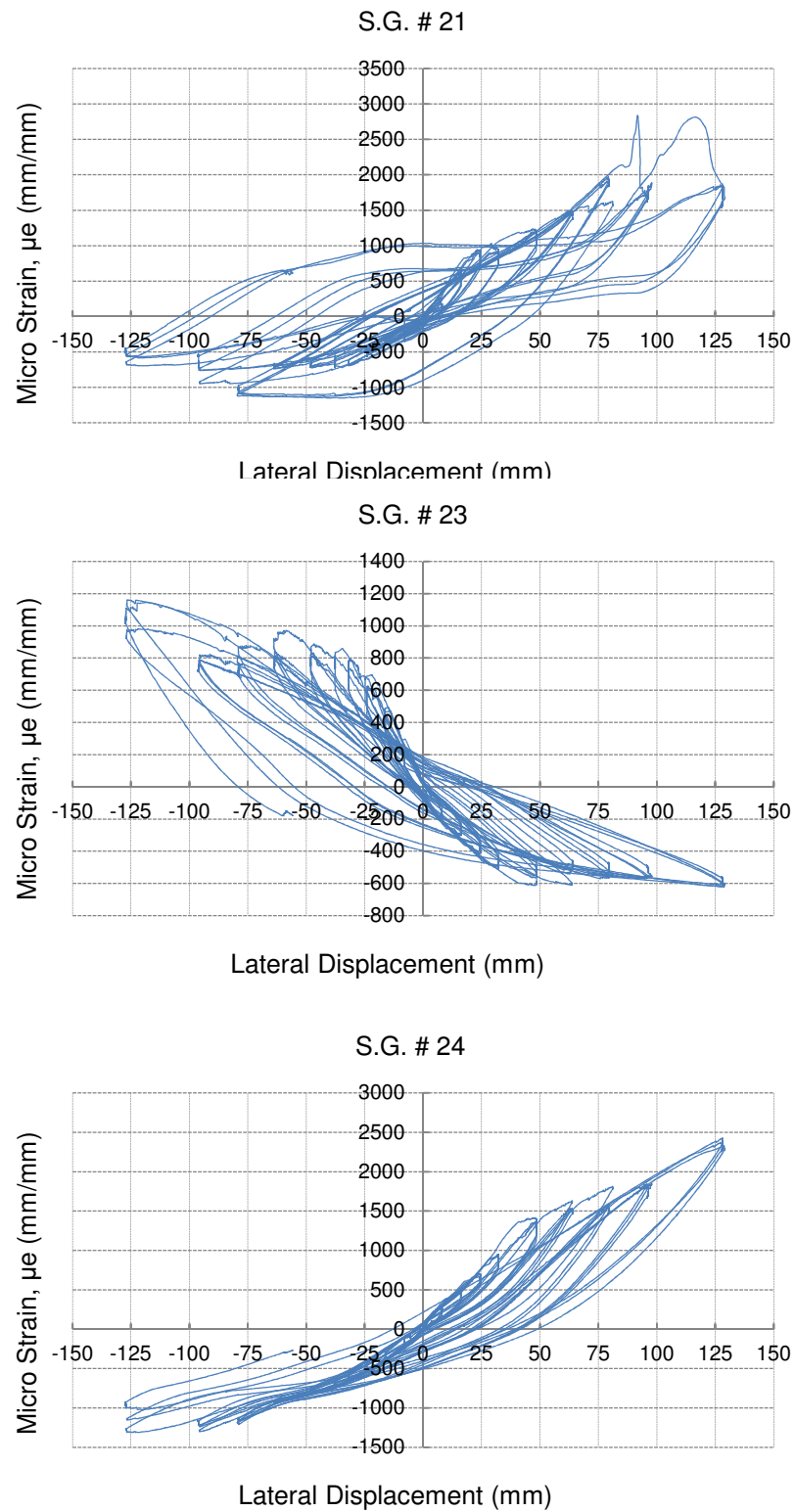


Figure VI.6: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars of columns near beam-column joints

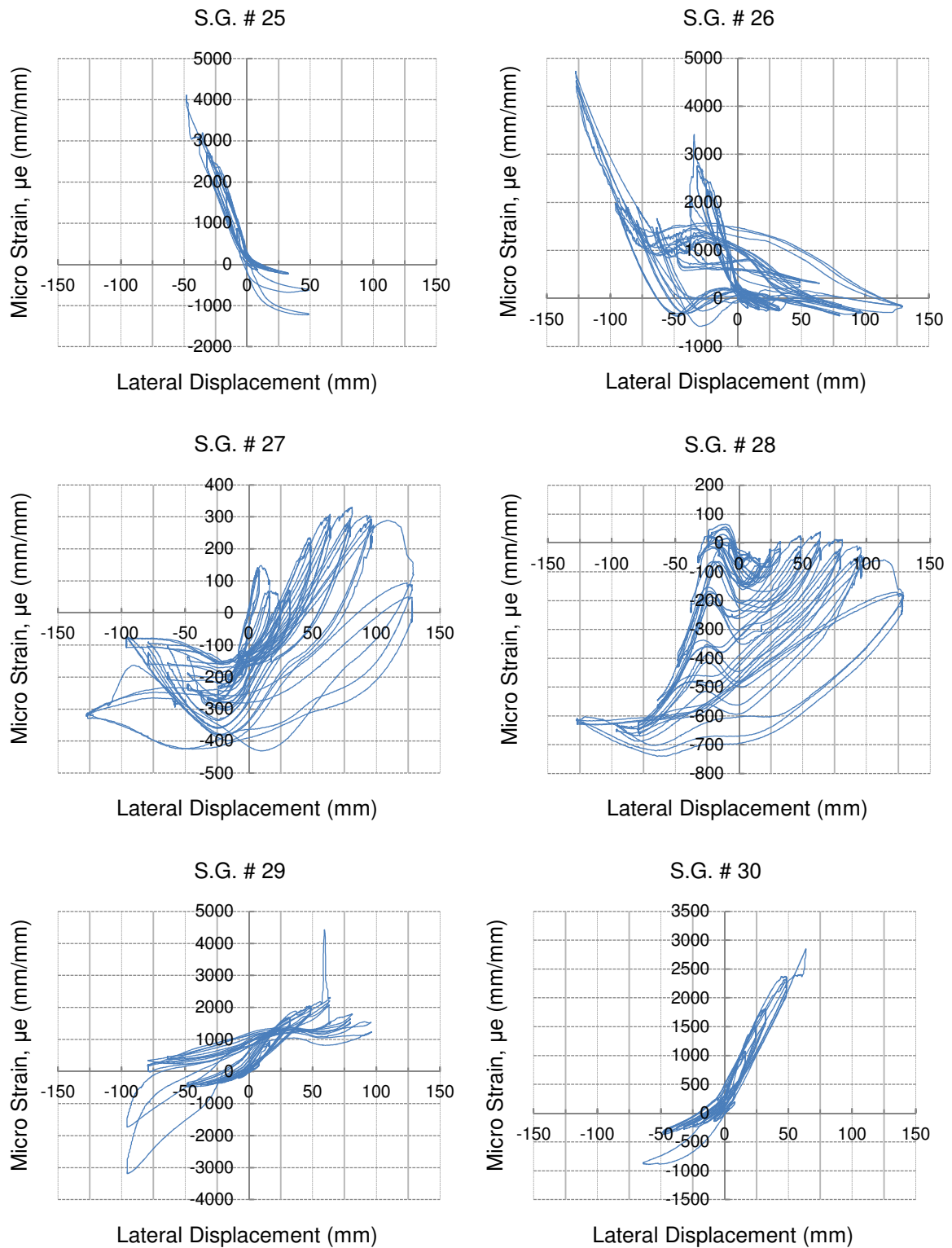


Figure VI.7: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars of beam

## VI.B Repaired Retrofitted Frame Test Data

Two tests were performed to retrofit the repaired bare control frame using two types of BRB steel core bars: AISI 12L14 carbon and AISI Type 304 stainless steel. The data recorded from these tests are illustrated in the following sections.

### VI.B.1 AISI 12L14 Carbon Steel BRB Core Bar Test Data (Frame RRF)

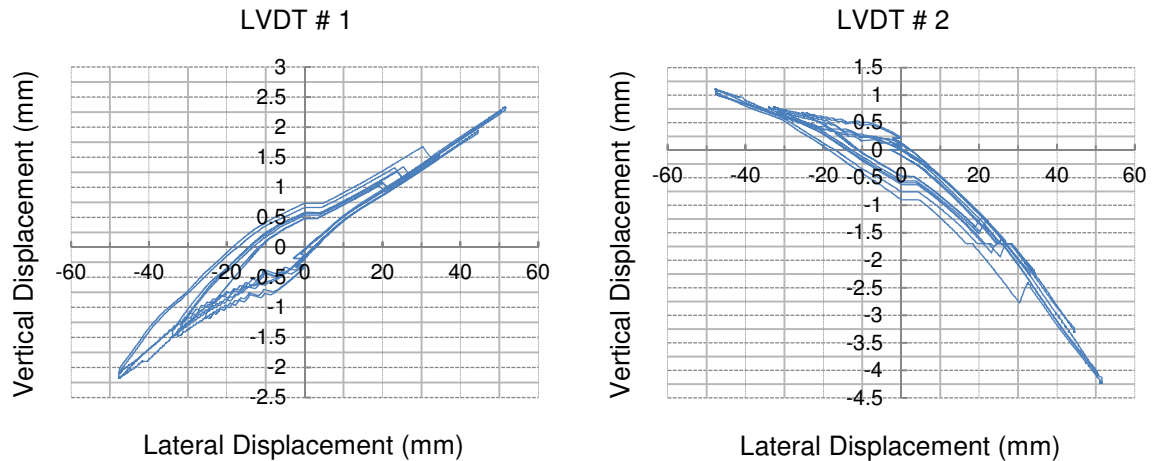


Figure VI.8: Frame lateral displacement vs. vertical displacement recorded by LVDTs 1-2 (300 mm height from foundation of columns)

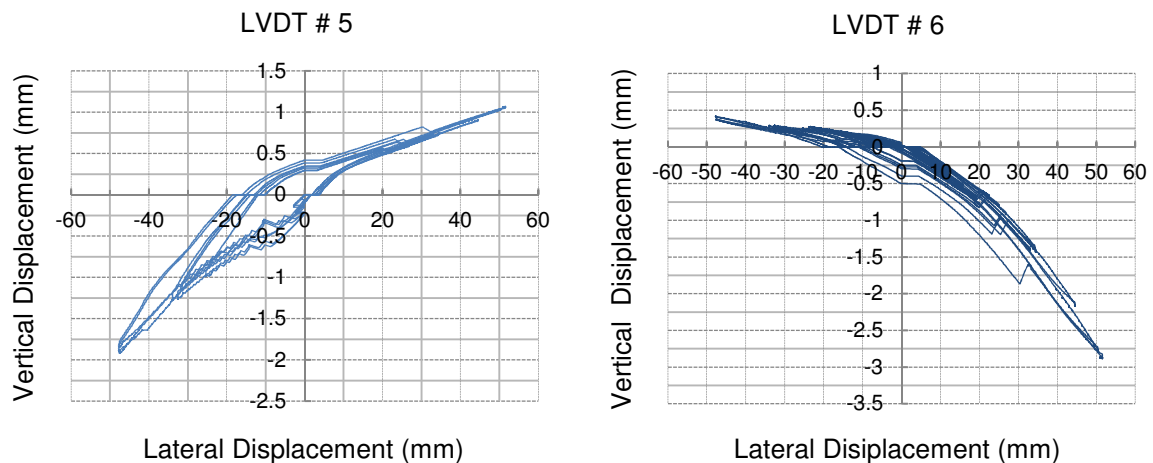


Figure VI.9: Frame lateral displacement vs. vertical displacement recorded by LVDTs 5-6 (25 mm height from foundation of columns)

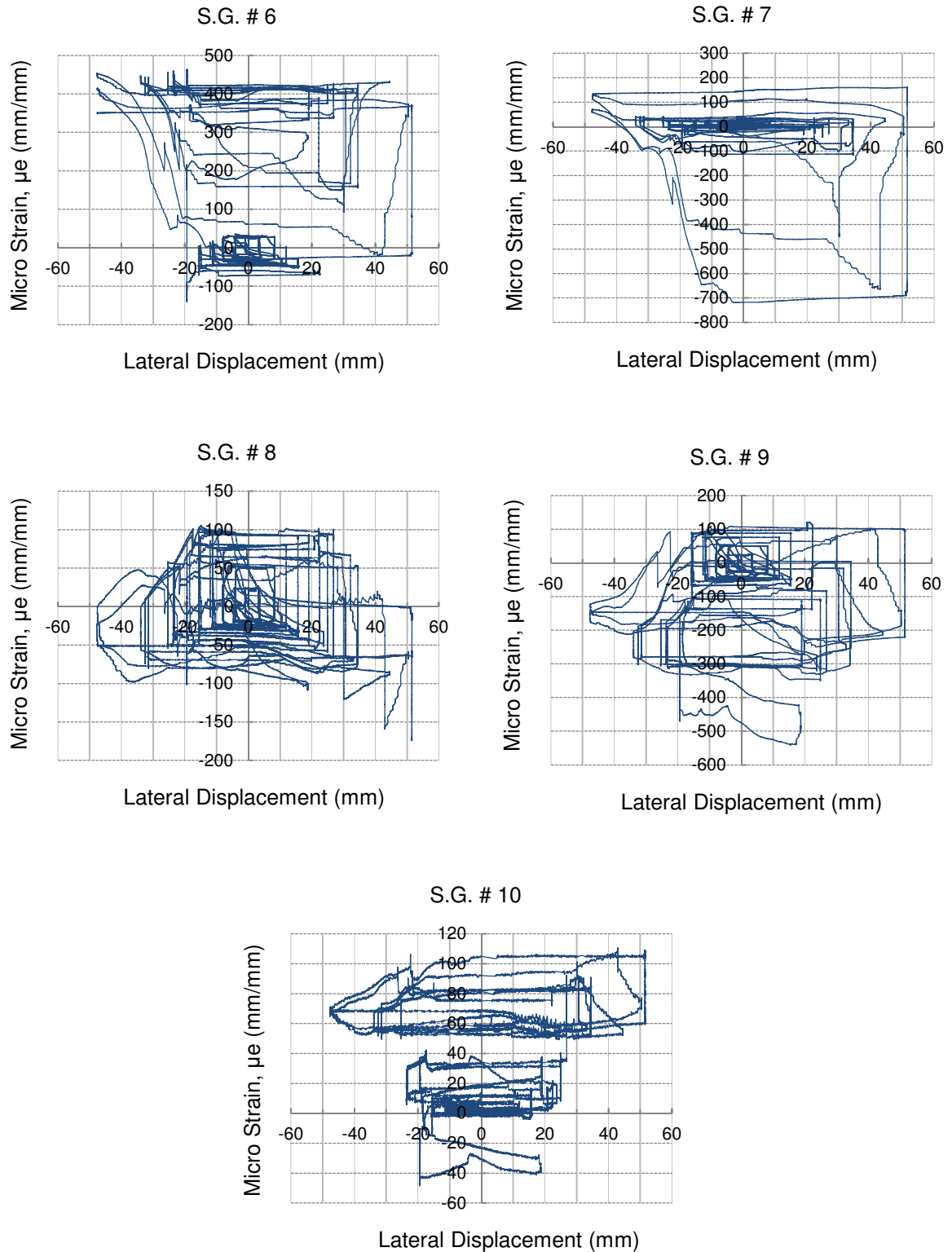


Figure VI.10: Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at far column base

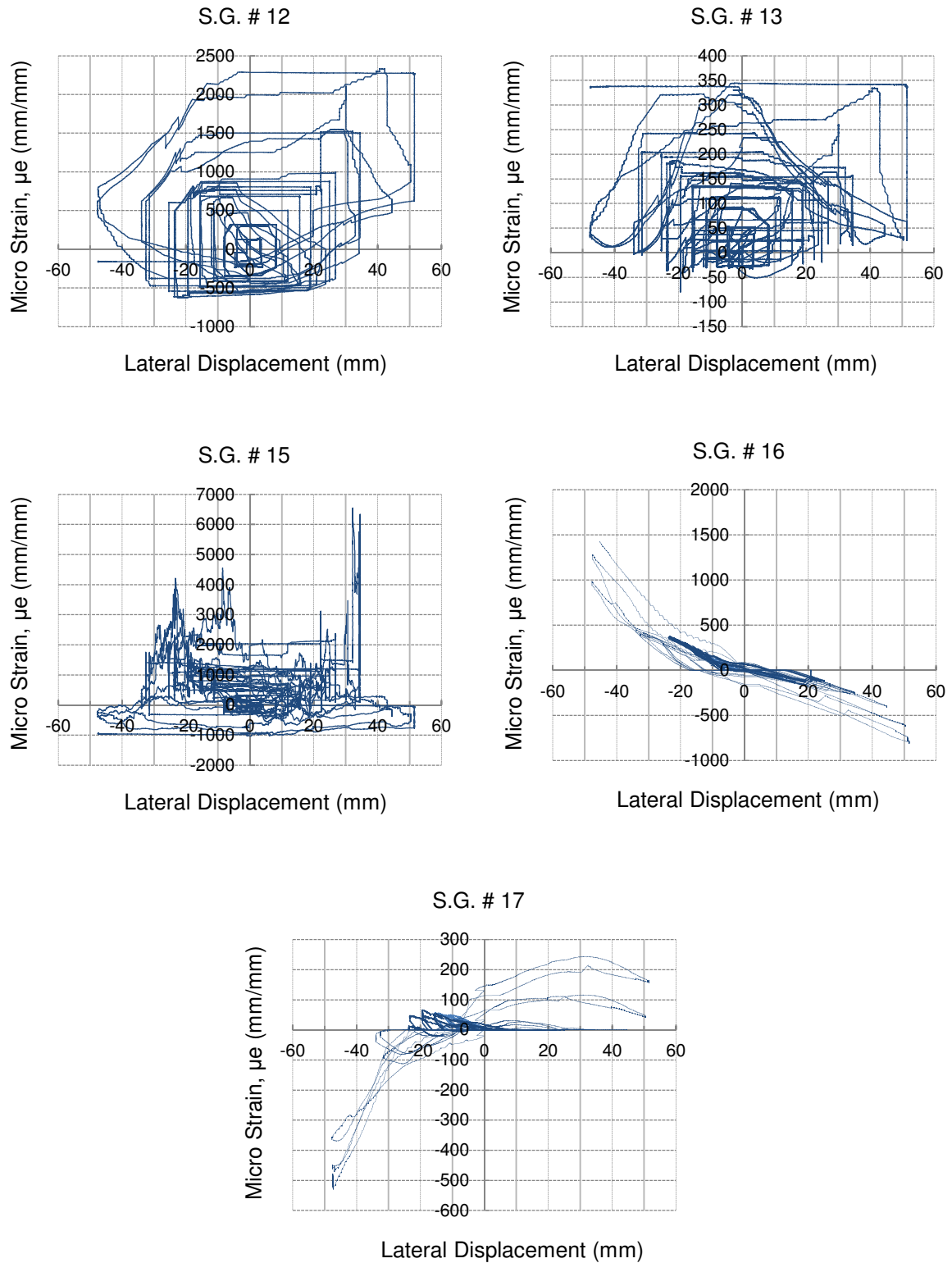


Figure VI.11: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars at near column base

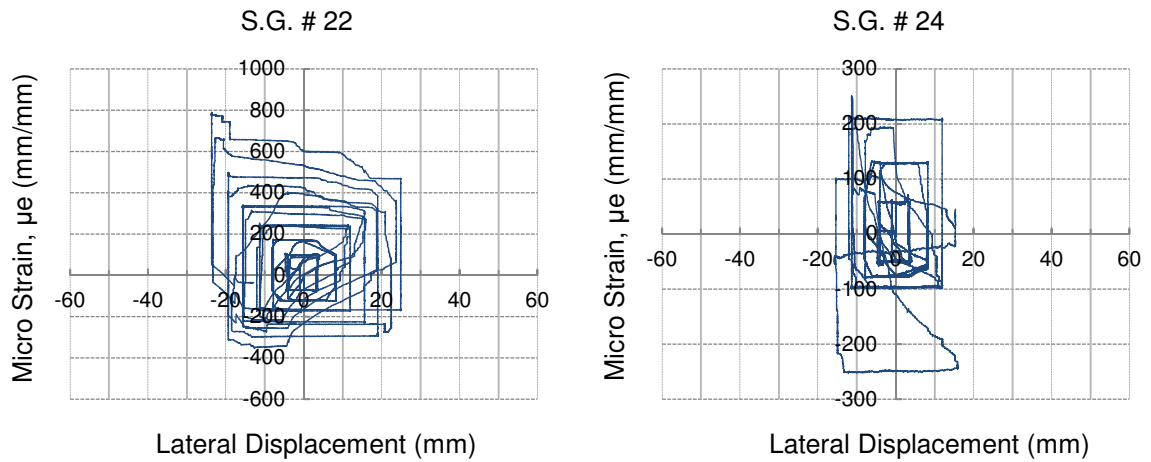


Figure VI.12: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars of columns near beam-column joints

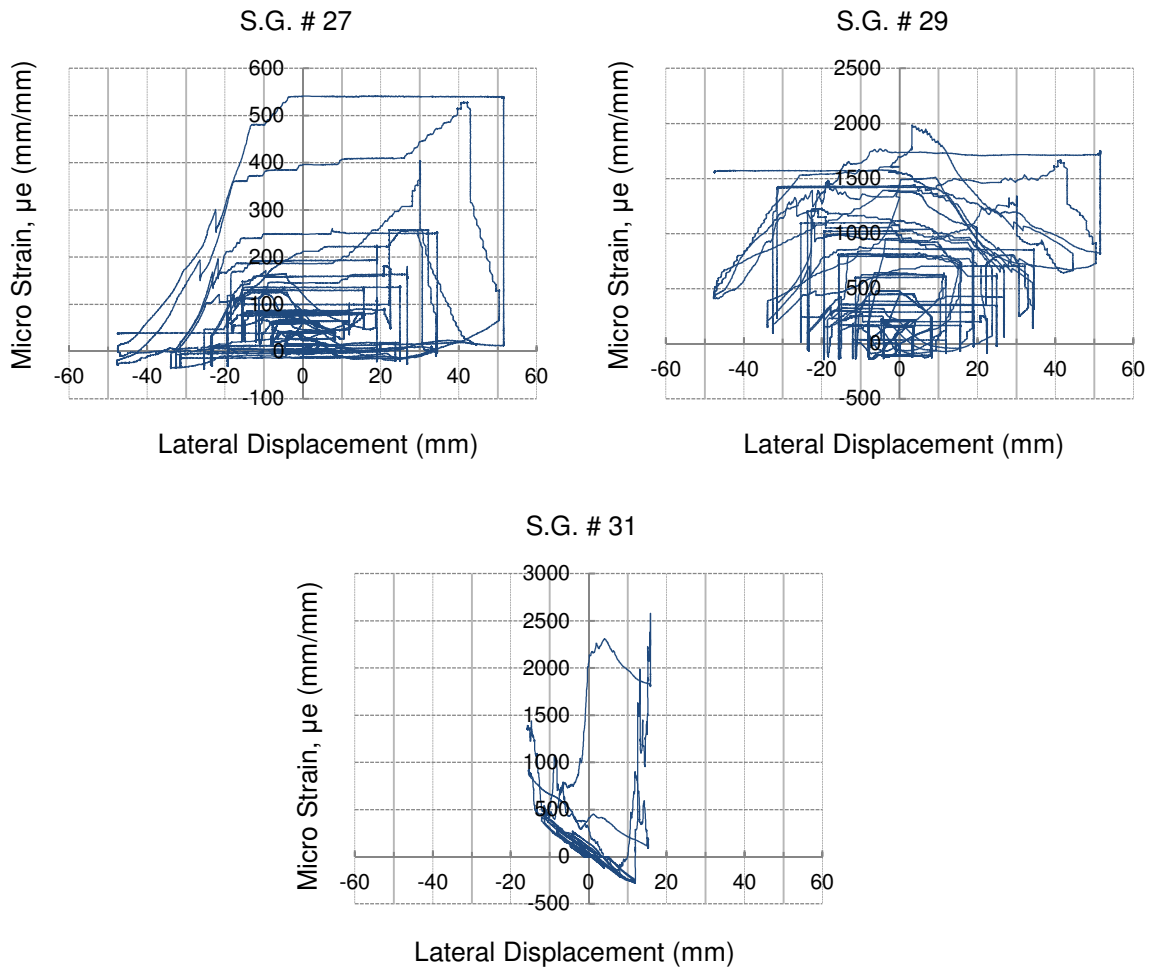


Figure VI.13: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars of beam



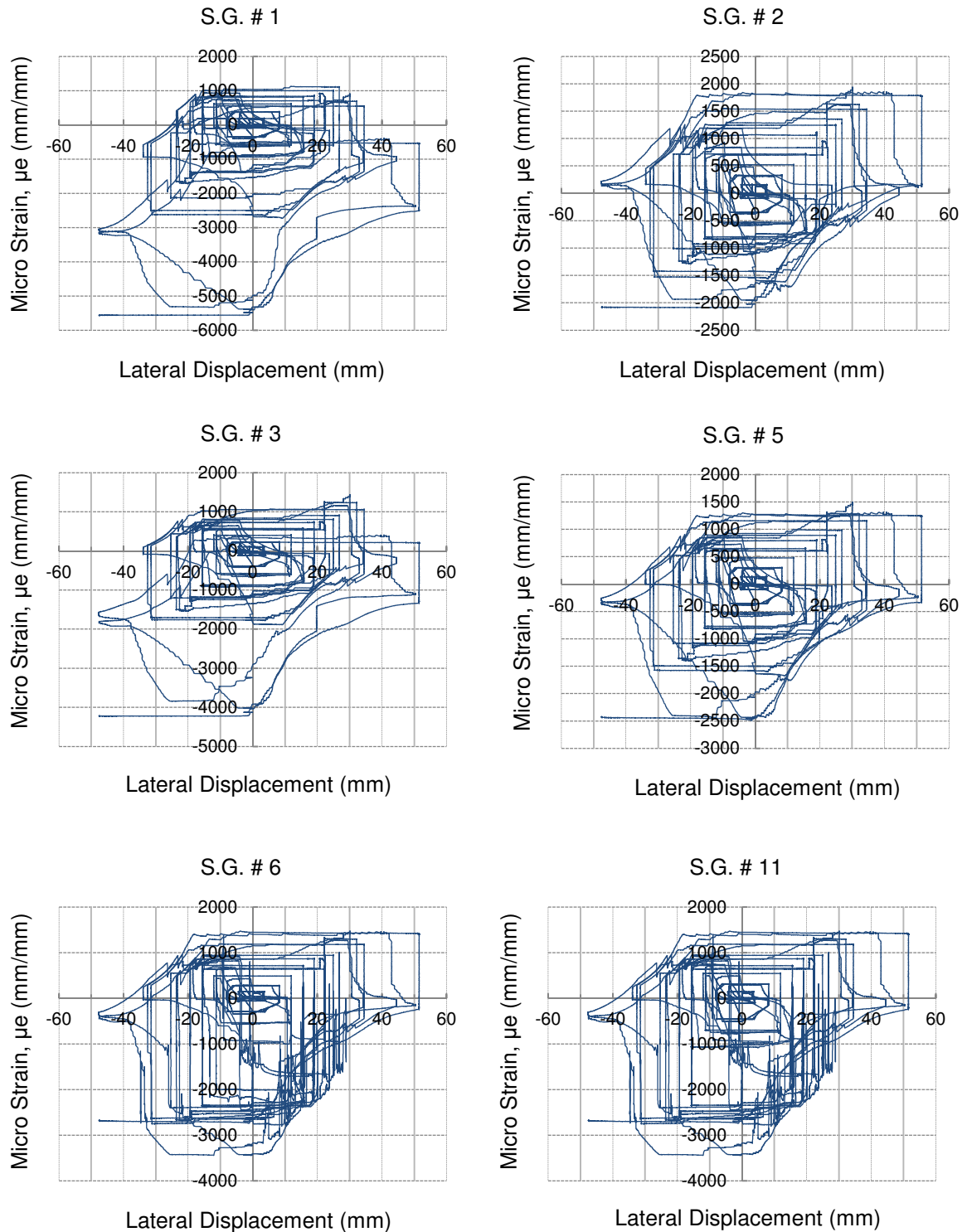


Figure VI.14: Frame lateral displacement vs. strain recorded by strain gauges on the BRB steel core bar

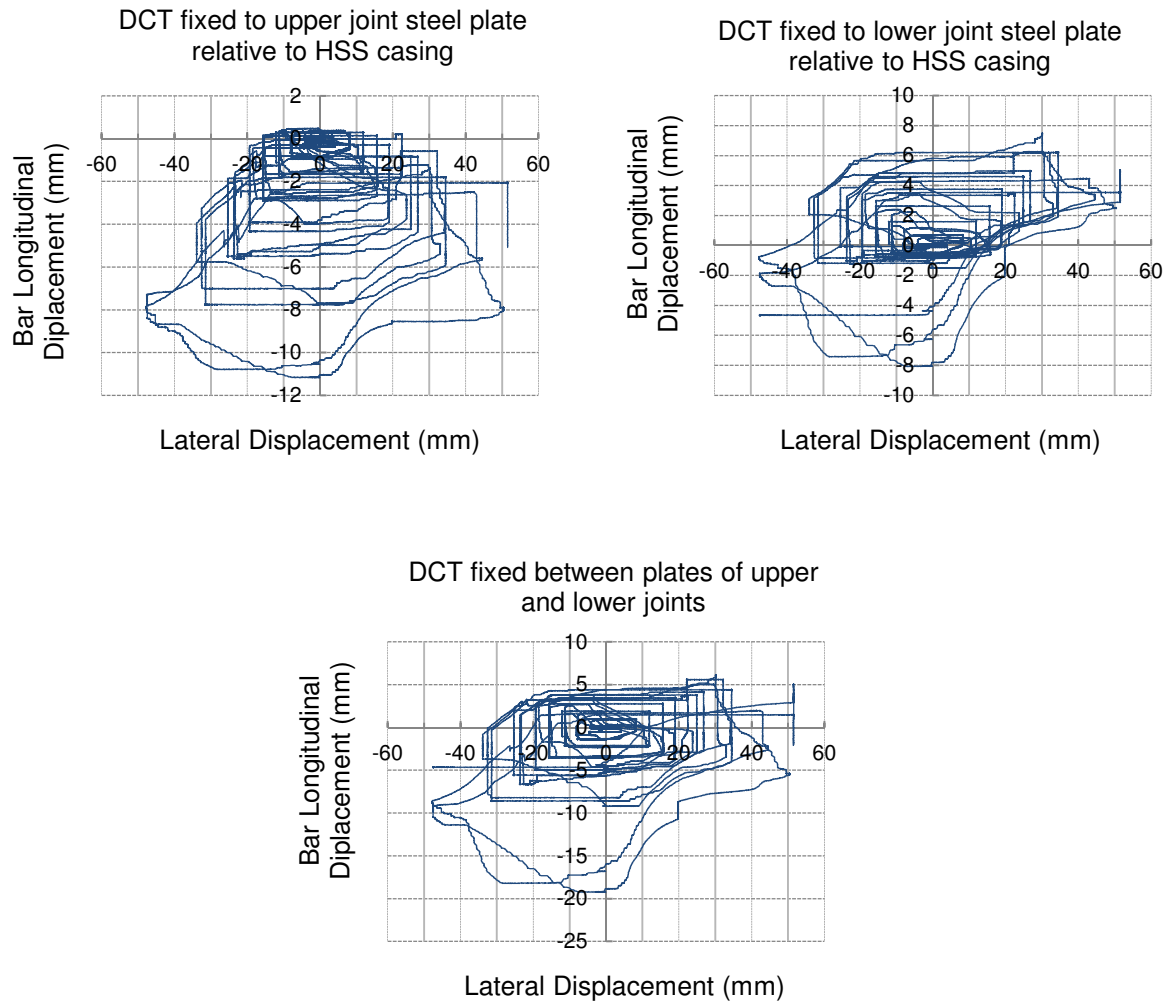


Figure VI.15: Frame lateral displacement vs. elongation of steel core bar recorded by displacement cable transducers

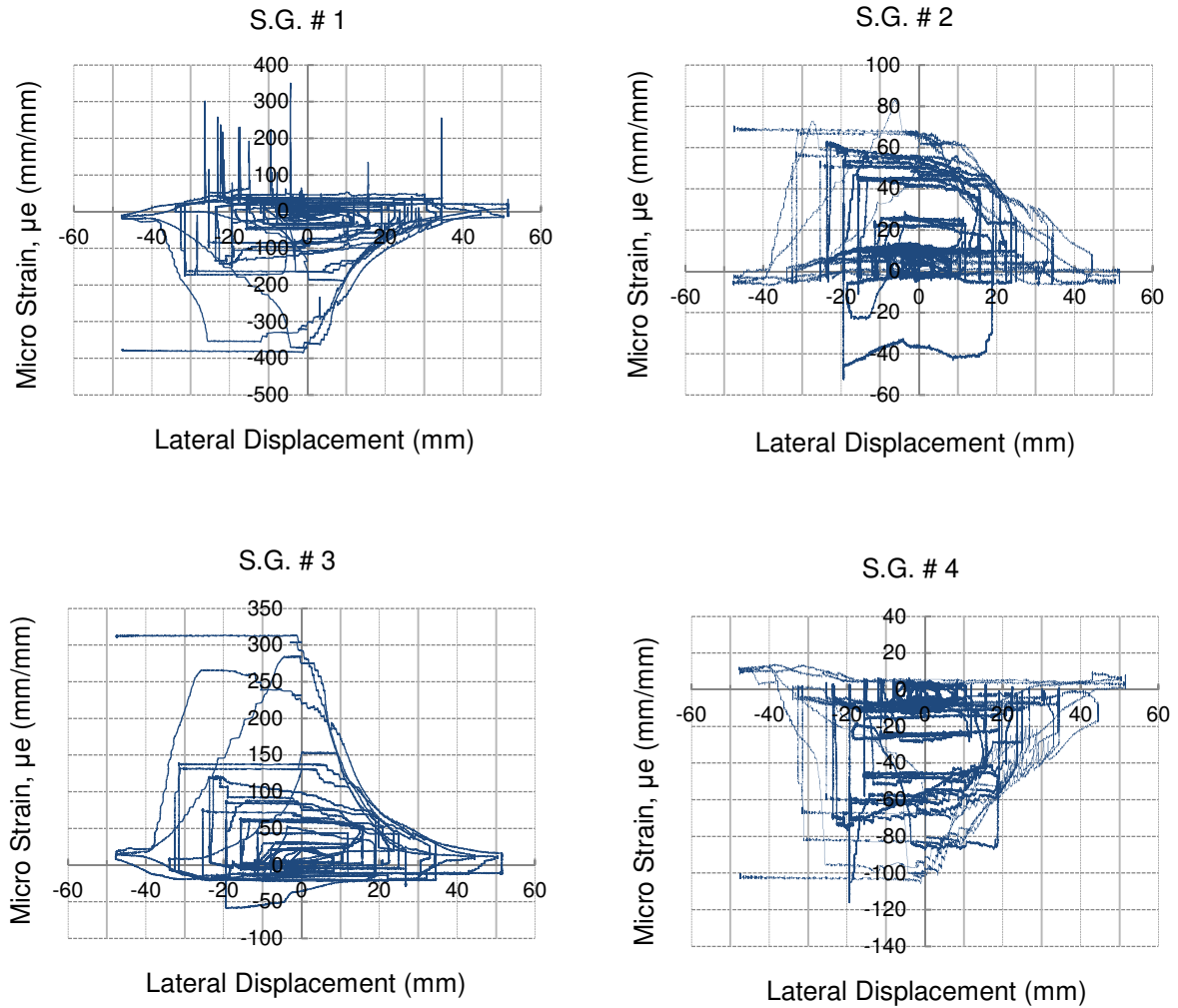


Figure VI.16: Frame lateral displacement vs. strain recorded by longitudinal strain gauges on the BRB HSS casing

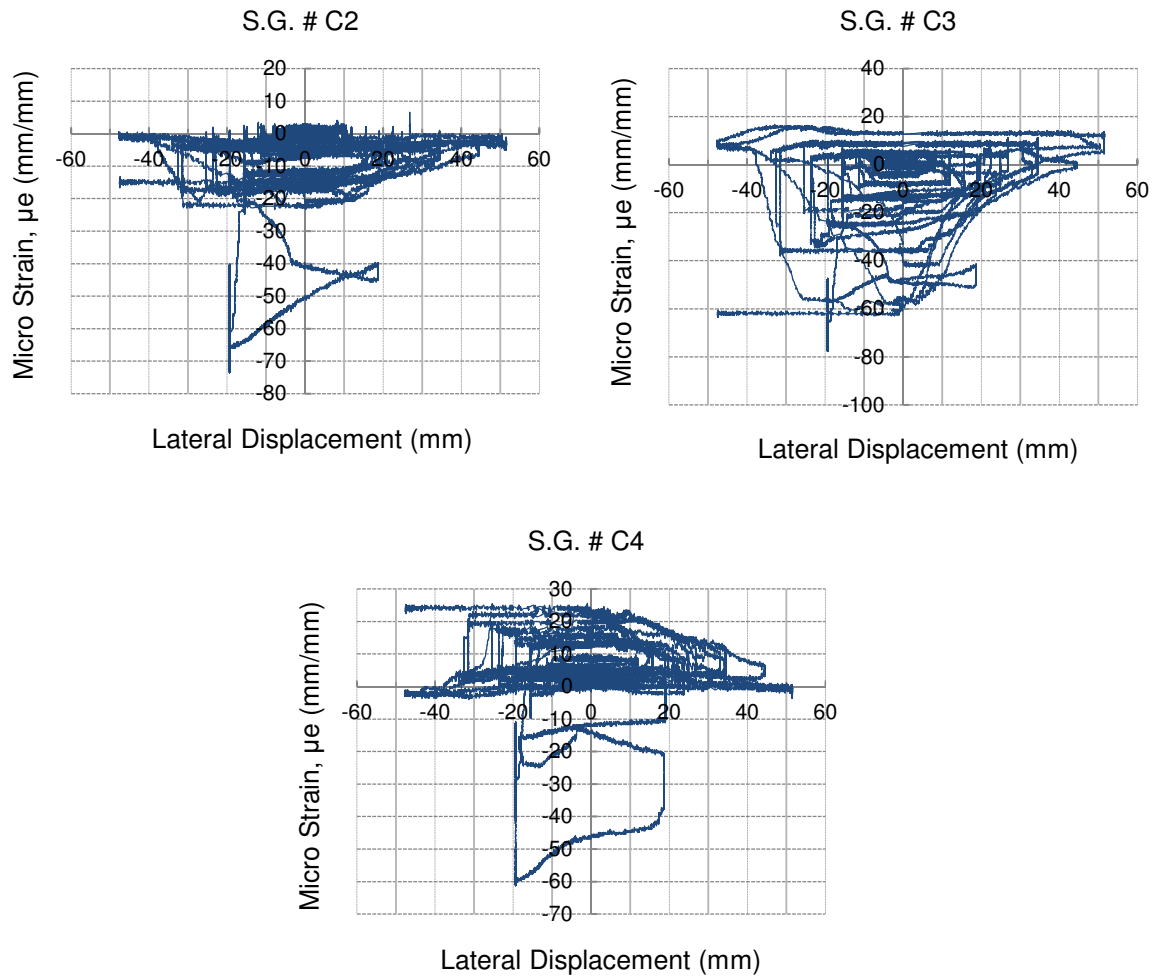


Figure VI.17: Frame lateral displacement vs. strain recorded by circumferential strain gauges on the BRB HSS casing

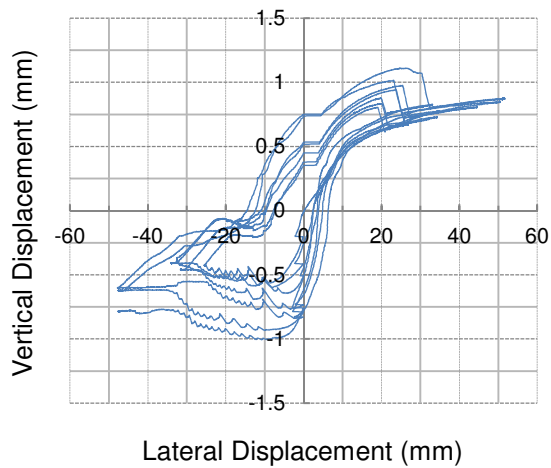


Figure VI.18: Frame lateral displacement vs. uplift displacement of the foundation

### VI.B.2 AISI Type 304 Stainless Steel BRB Core Bar Test Data (Frame RRF)

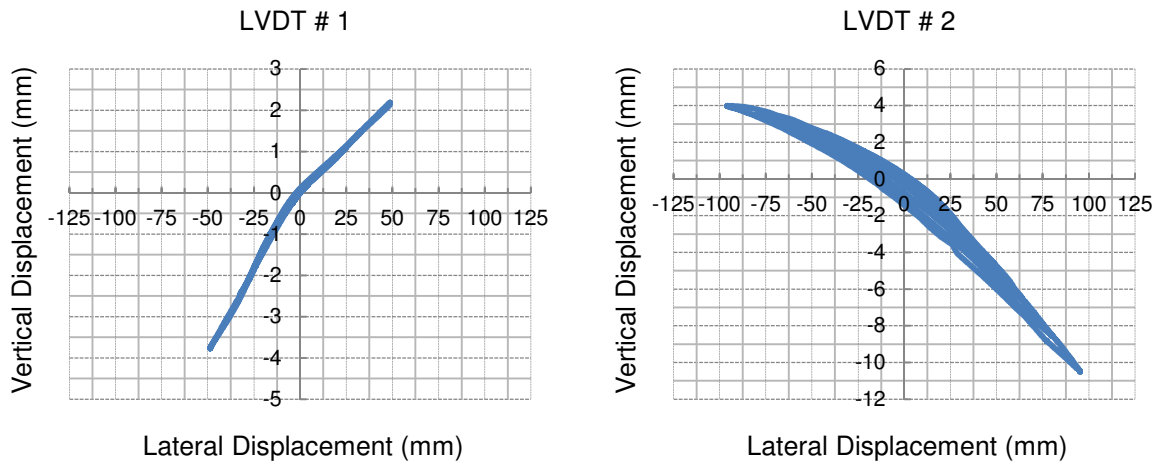


Figure VI.19: Frame lateral displacement vs. vertical displacement recorded by LVDTs 1-2 (300 mm height from foundation of columns)

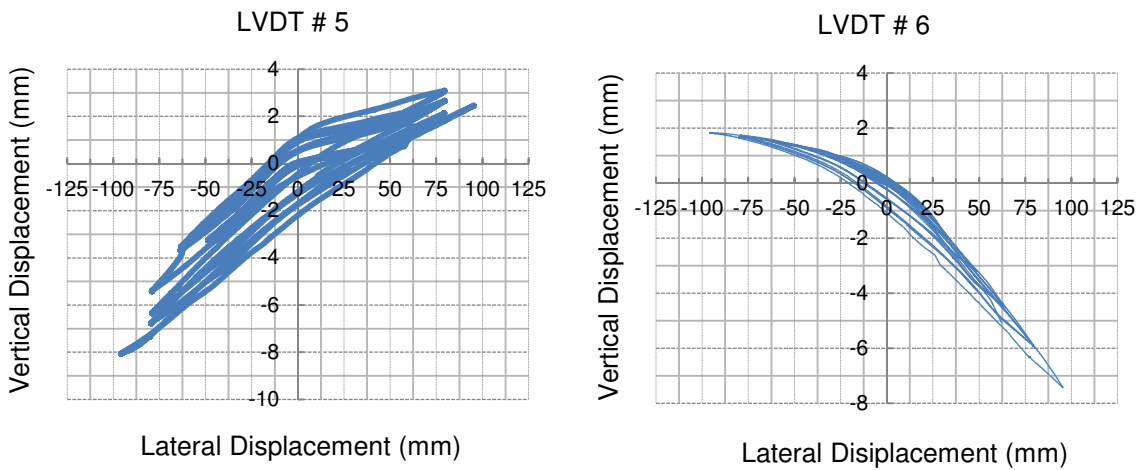


Figure VI.20: Frame lateral displacement vs. vertical displacement recorded by LVDTs 5-6 (25 mm height from foundation of columns)

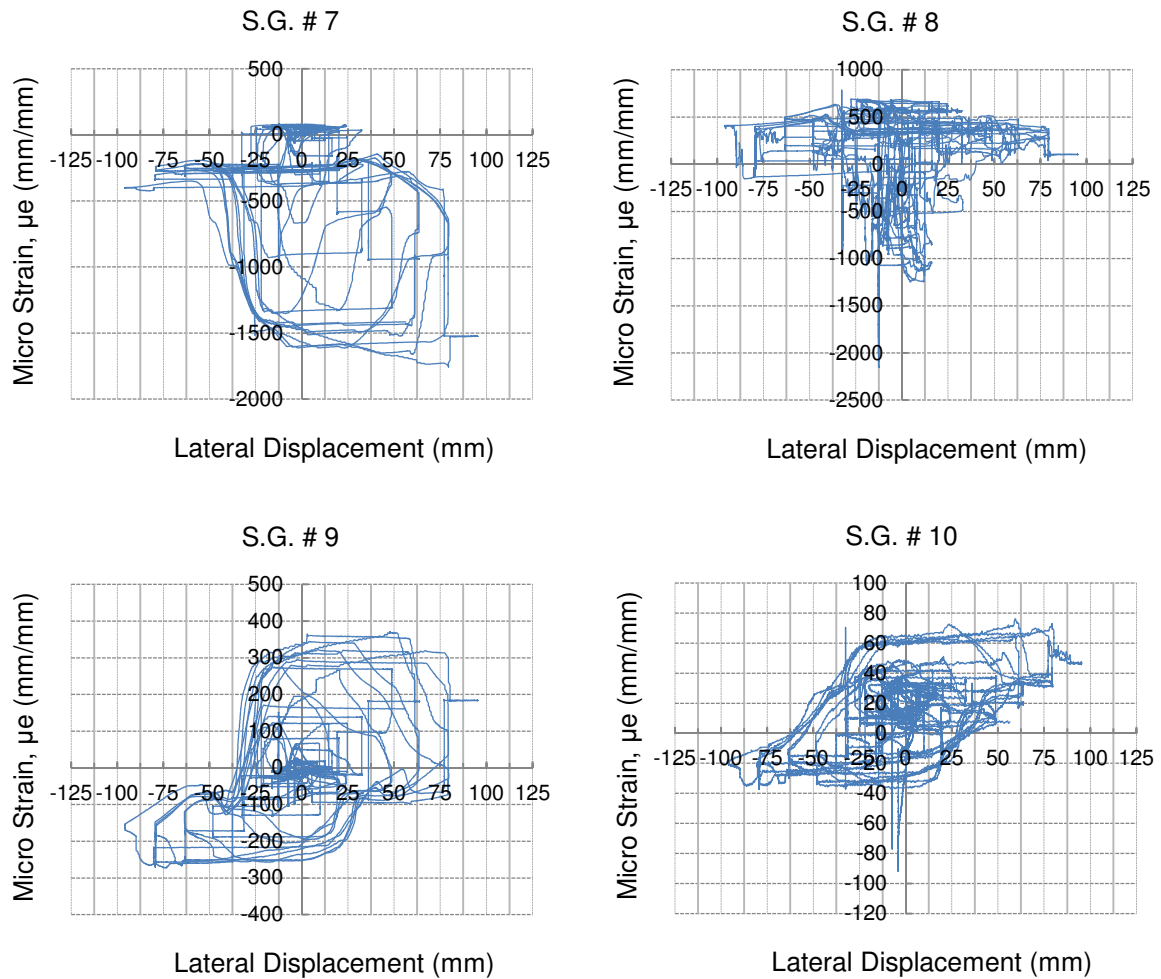


Figure VI.21: Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at far column base

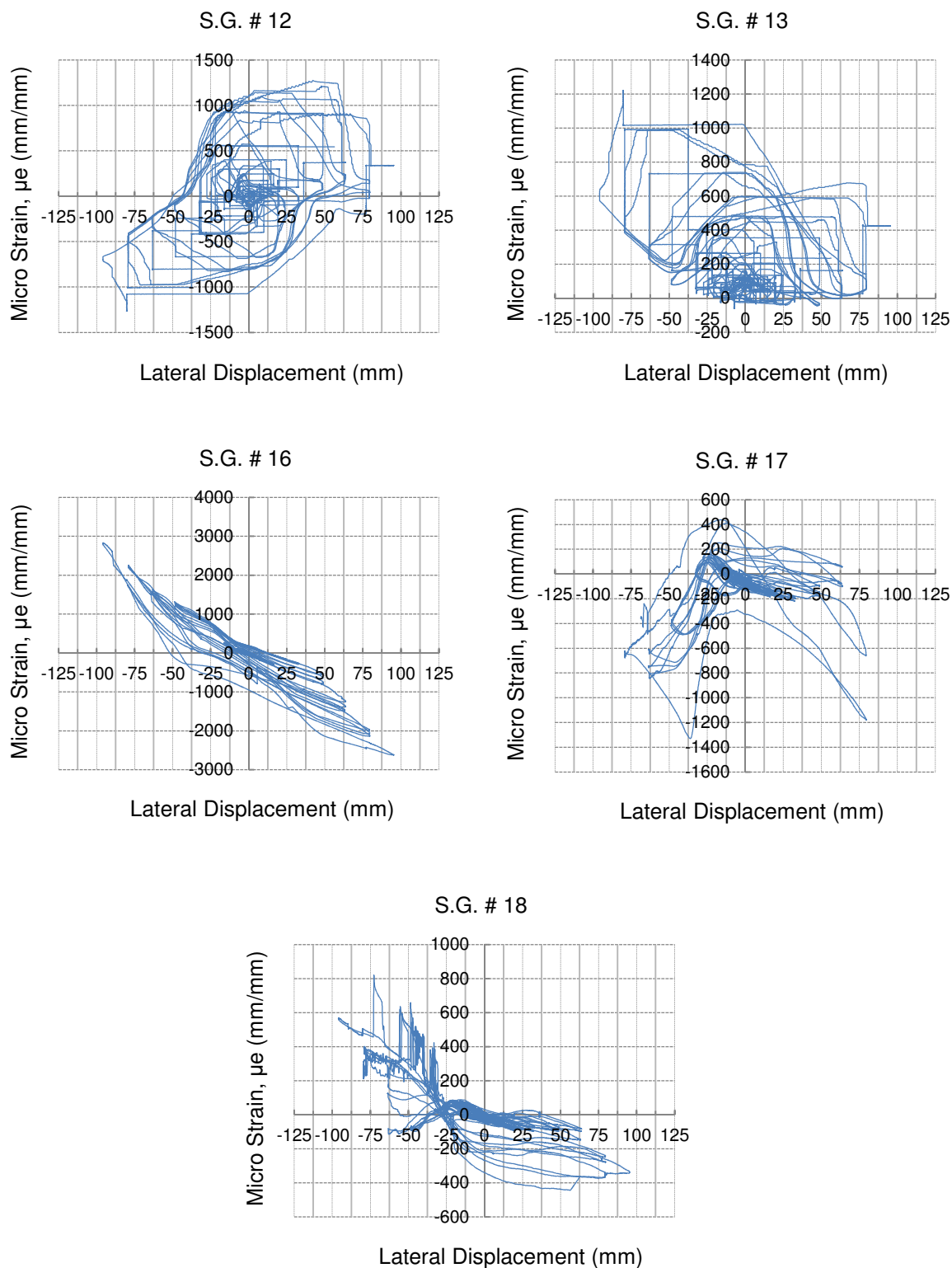


Figure VI.22: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars at near column base



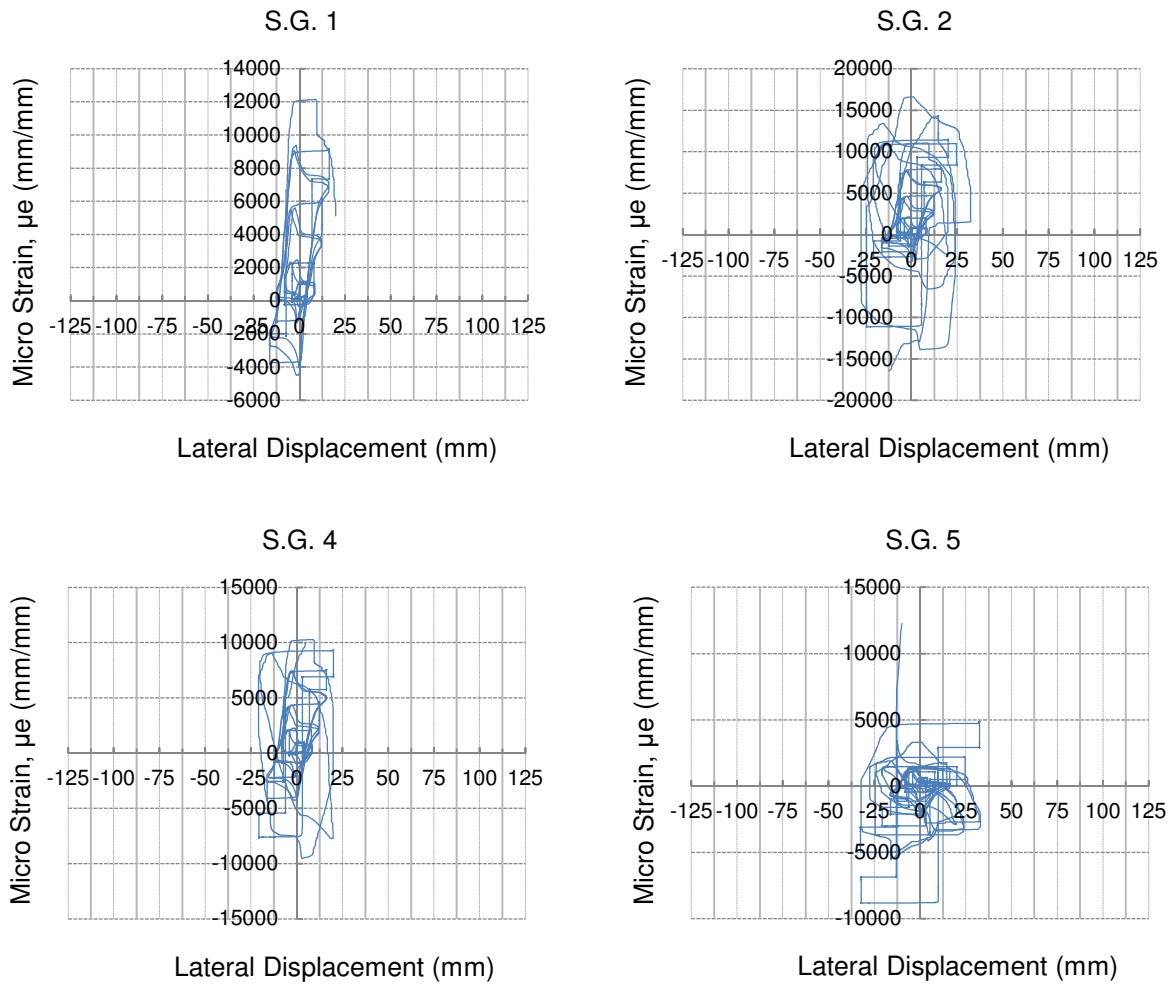


Figure VI.23: Frame lateral displacement vs. strain recorded by strain gauges on the BRB steel core bar

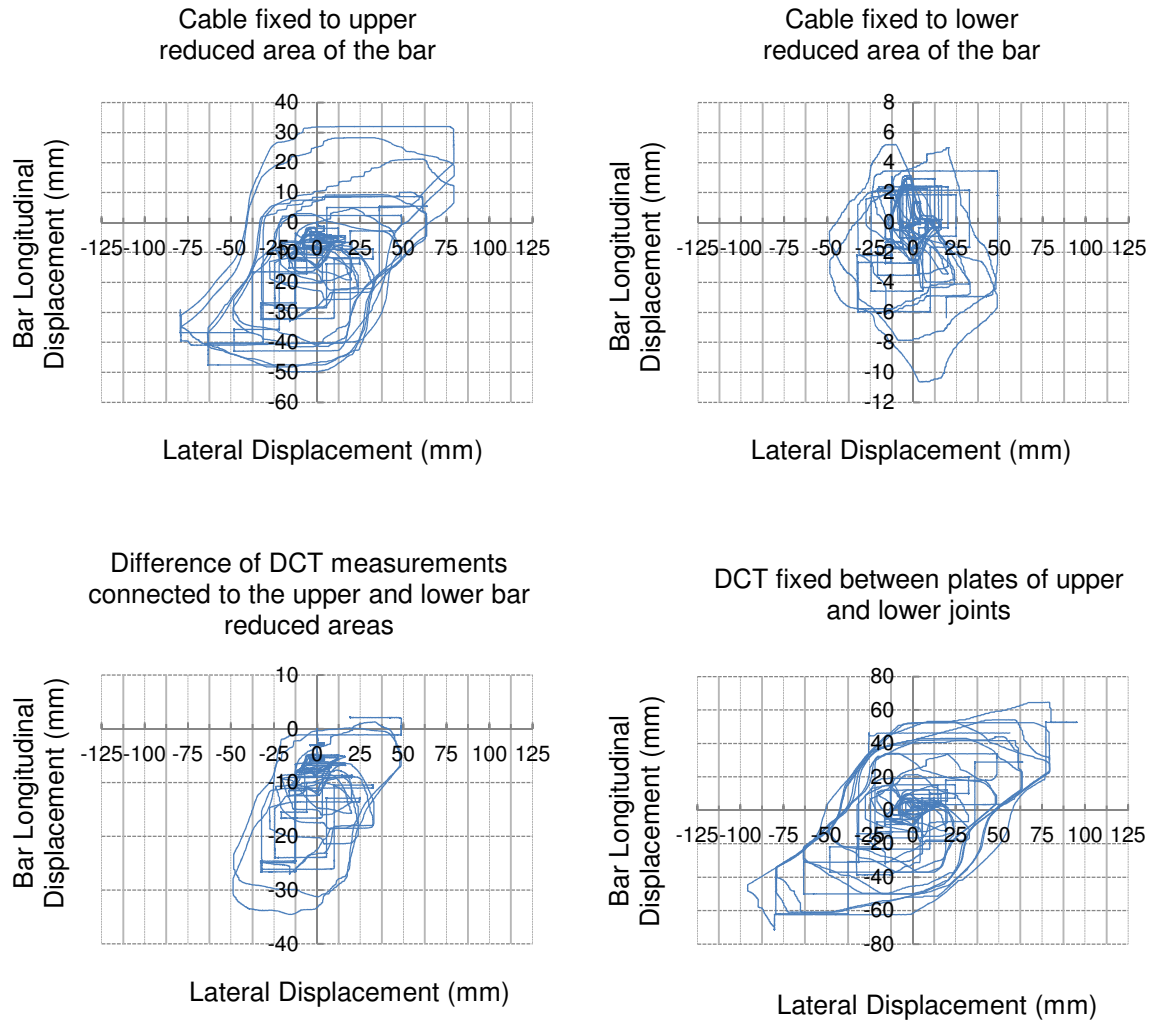


Figure VI.24: Frame lateral displacement vs. elongation of steel core bar recorded by displacement cable transducers

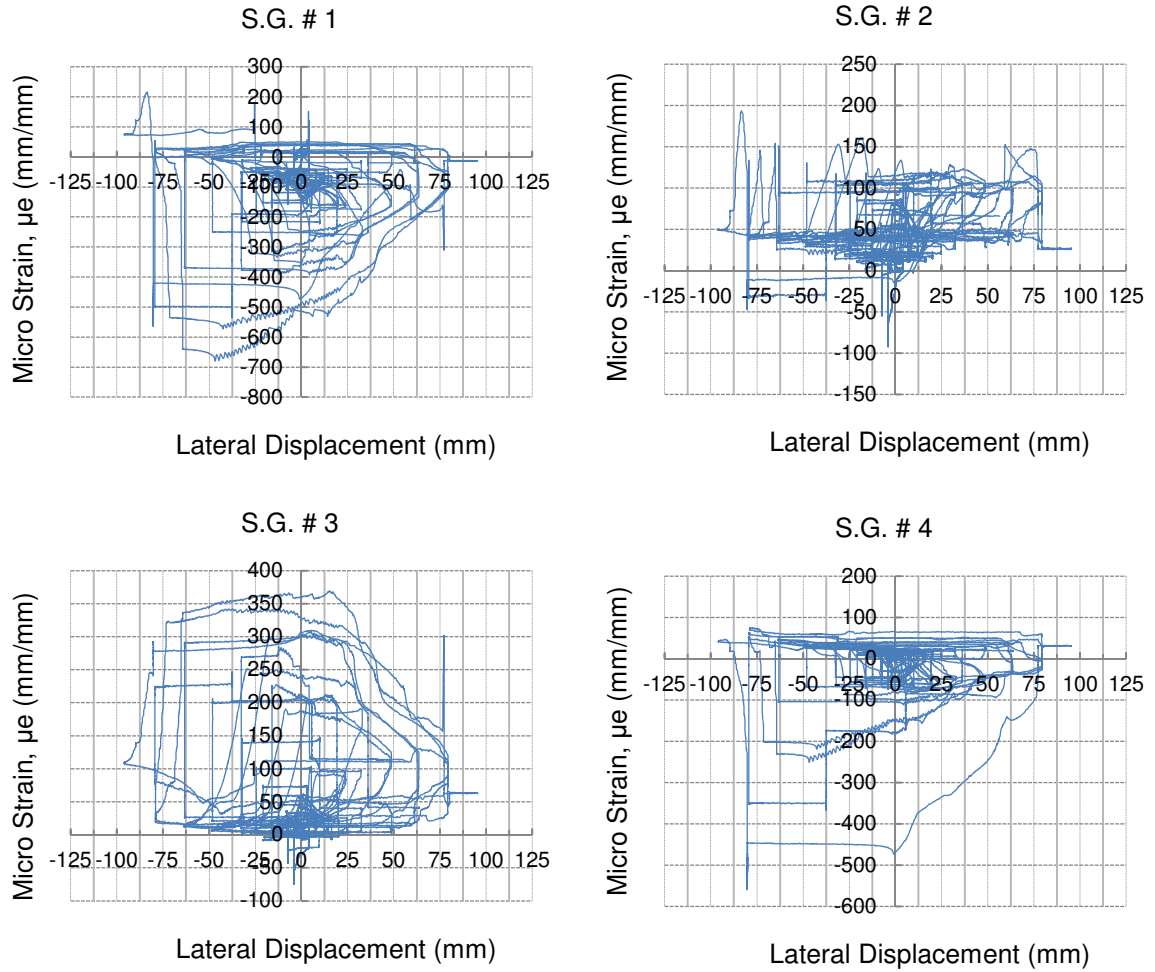


Figure VI.25: Frame lateral displacement vs. strain recorded by longitudinal strain gauges on the BRB HSS casing

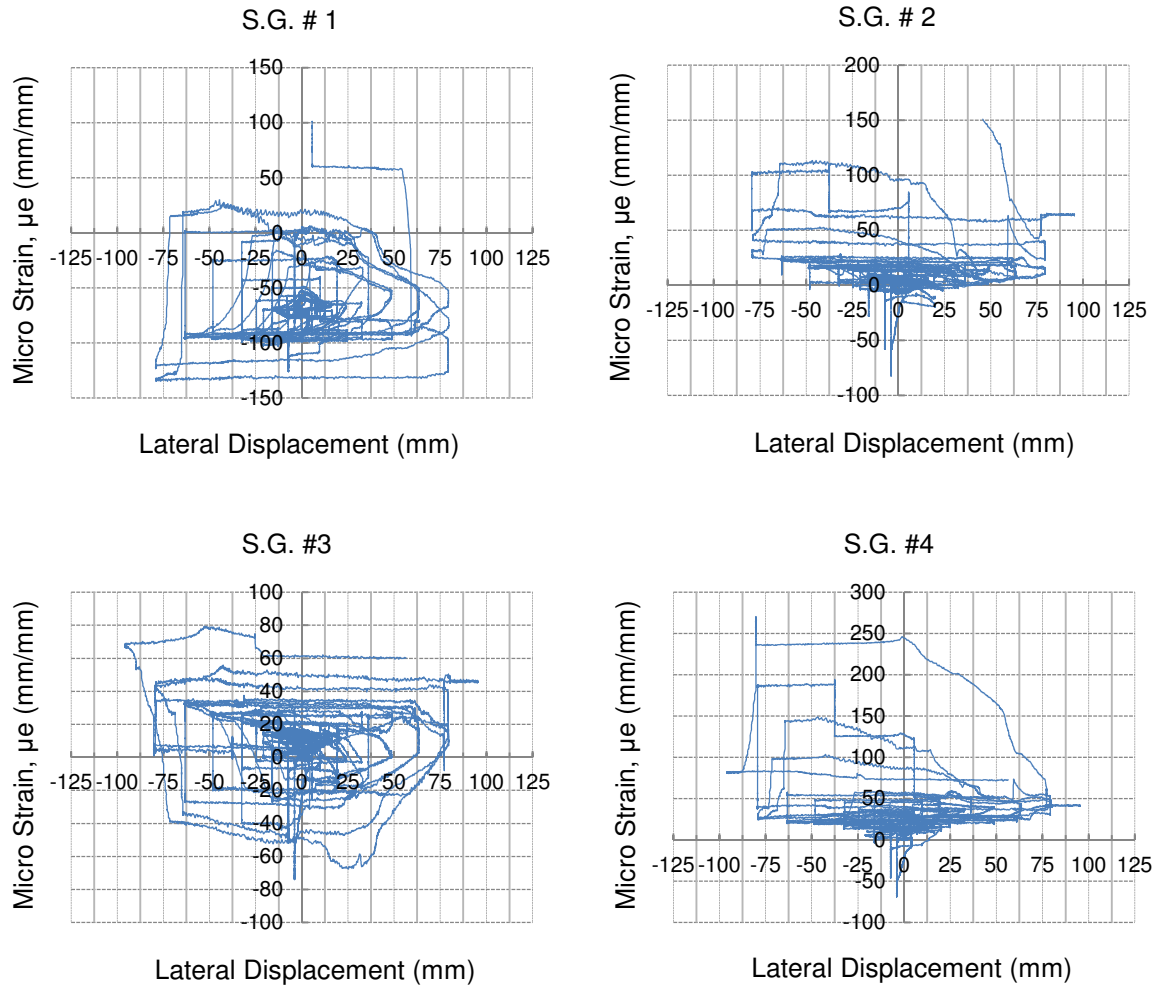


Figure VI.26: Frame lateral displacement vs. strain recorded by circumferential strain gauges on the BRB HSS casing

## VI.C AISI 4140 High Tensile Steel BRB Core Bar Test Data (Frame RF)

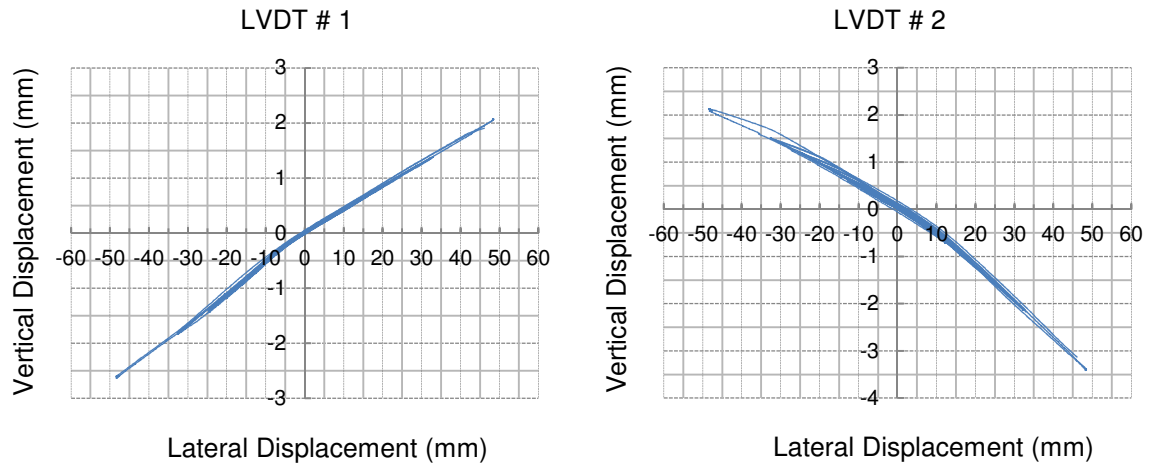


Figure VI.27: Frame lateral displacement vs. vertical displacement recorded by LVDTs 1-2 (300 mm height from foundation of columns)

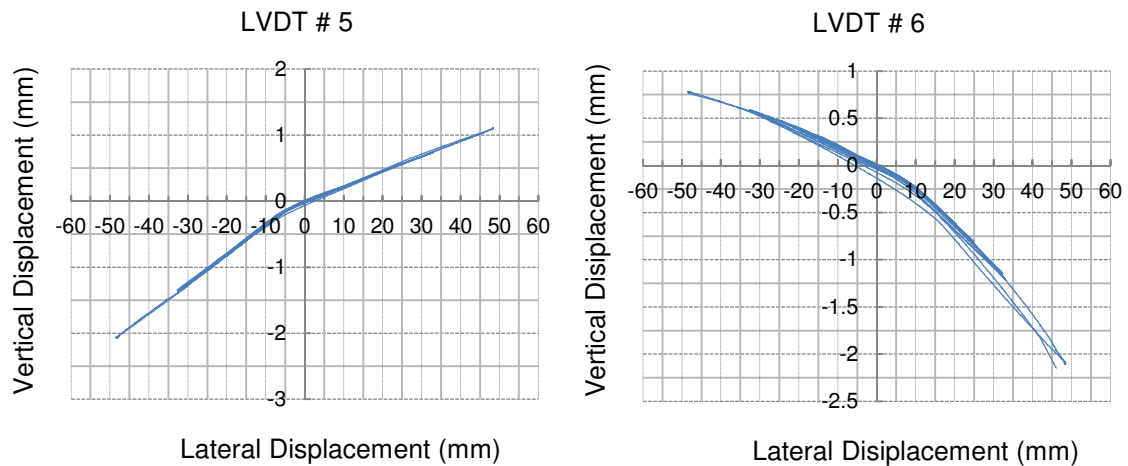


Figure VI.28: Frame lateral displacement vs. vertical displacement recorded by LVDTs 5-6 (25 mm height from foundation of columns)

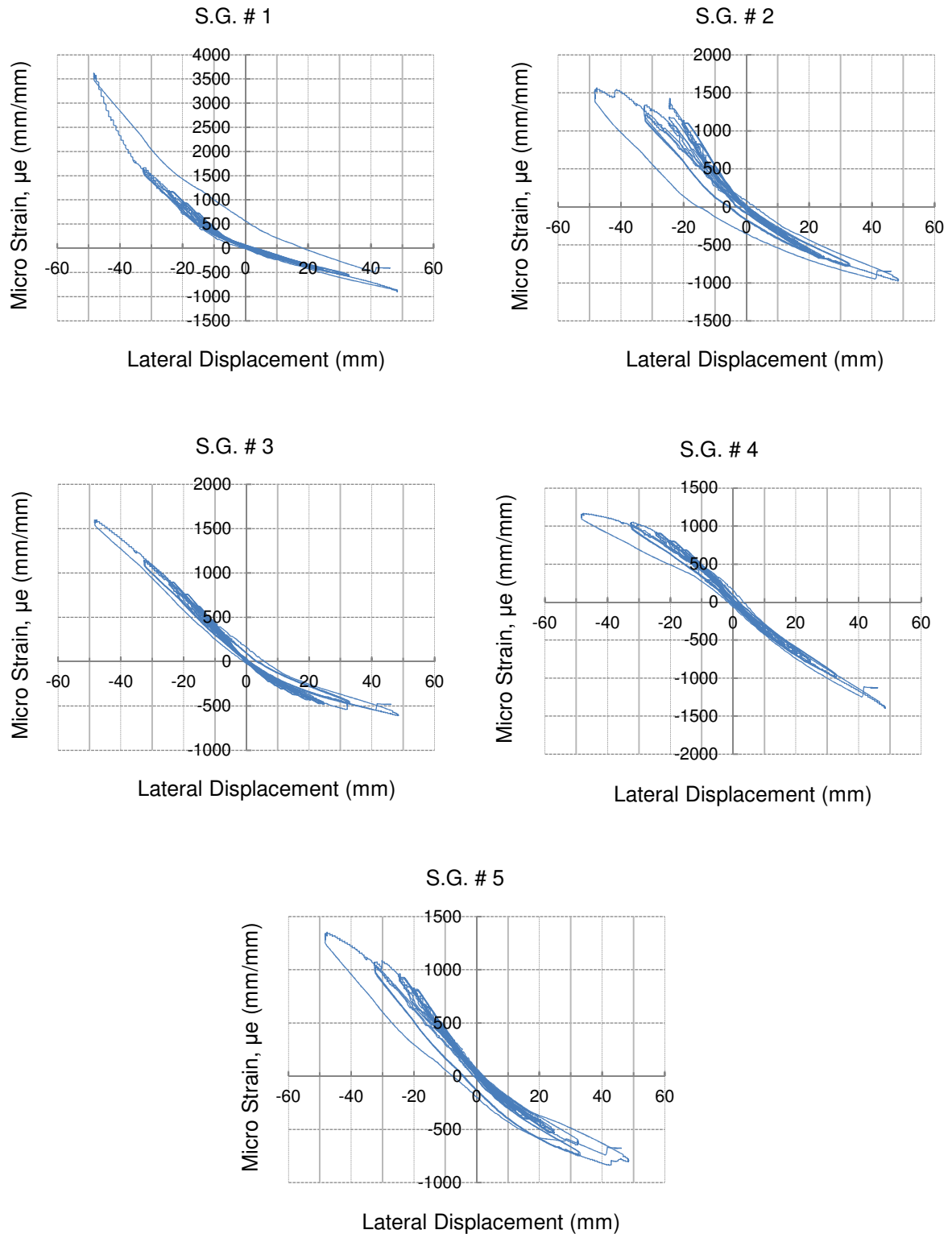


Figure VI.29: Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at far column base

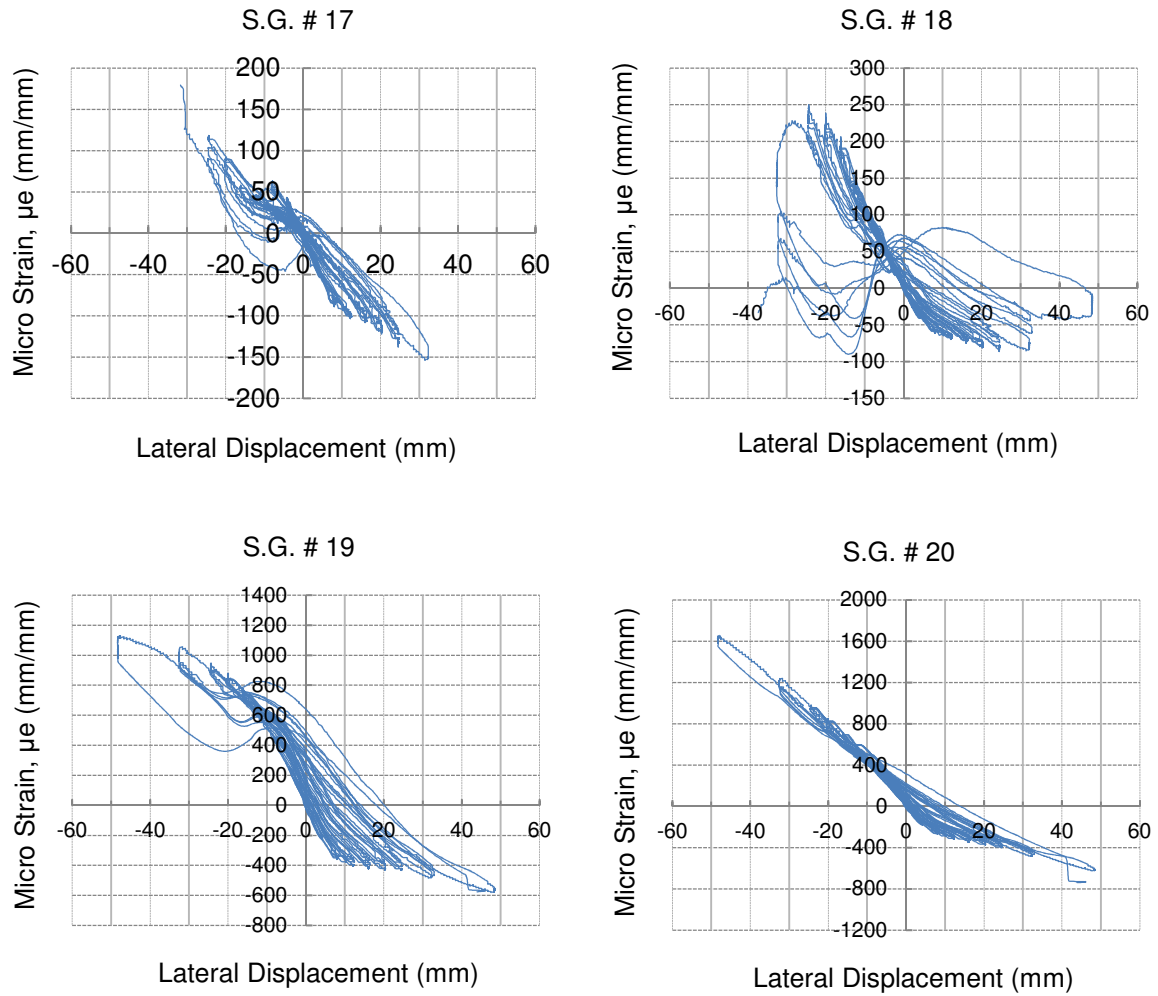


Figure VI.30: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars at near column base



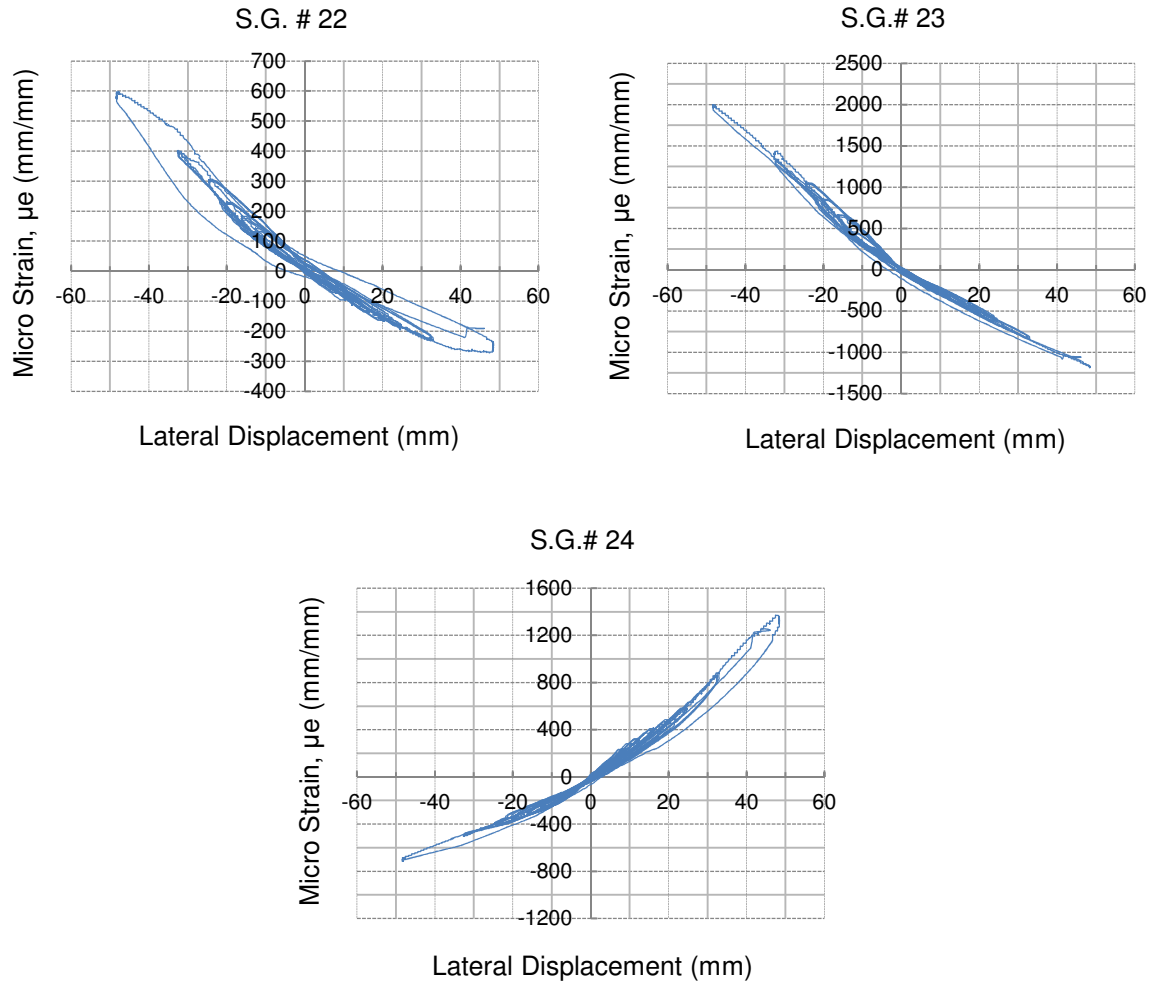


Figure VI.31: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars of columns near beam-column joints

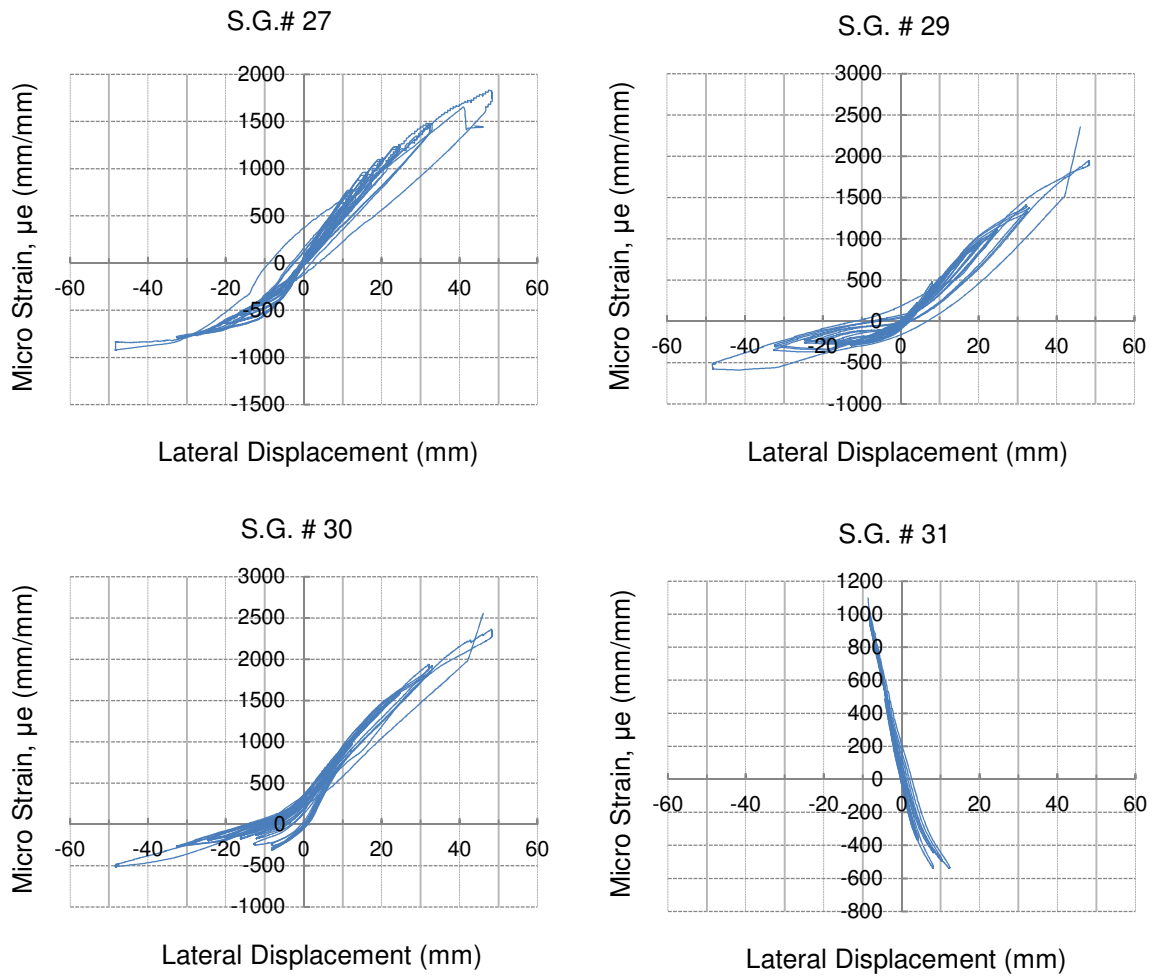


Figure VI.32: Frame lateral displacement vs. strain recorded by strain gauges on the reinforcing bars of beam

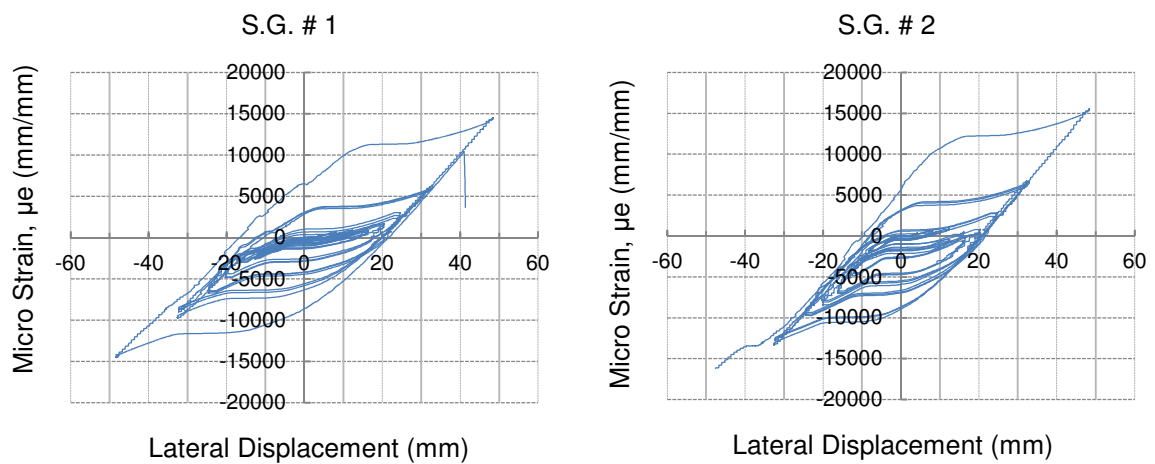


Figure VI.33: Frame lateral displacement vs. strain recorded by strain gauges on the BRB steel core bar

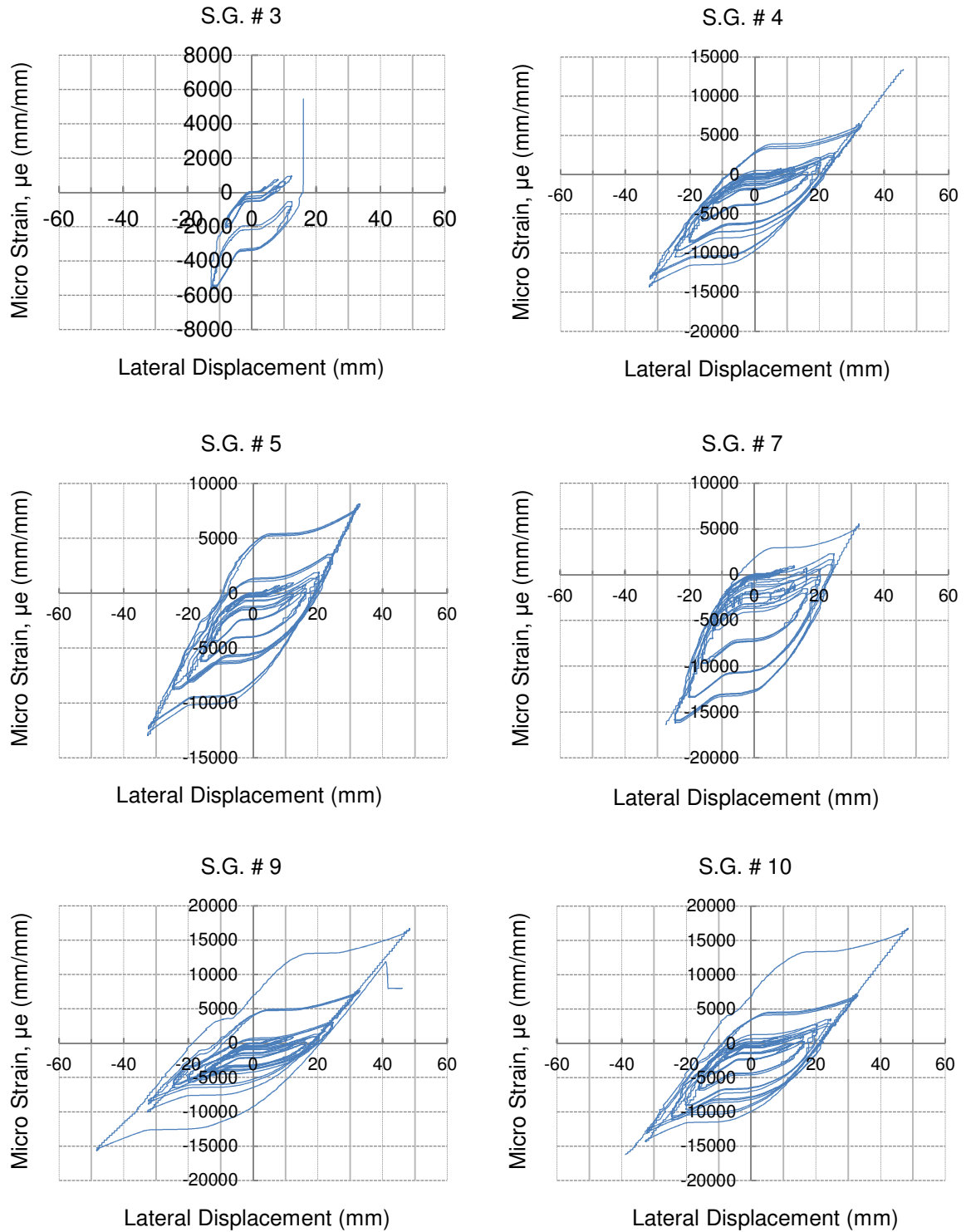


Figure VI.33 (Cont'd): Frame lateral displacement vs. strain recorded by strain gauges on the BRB steel core bar

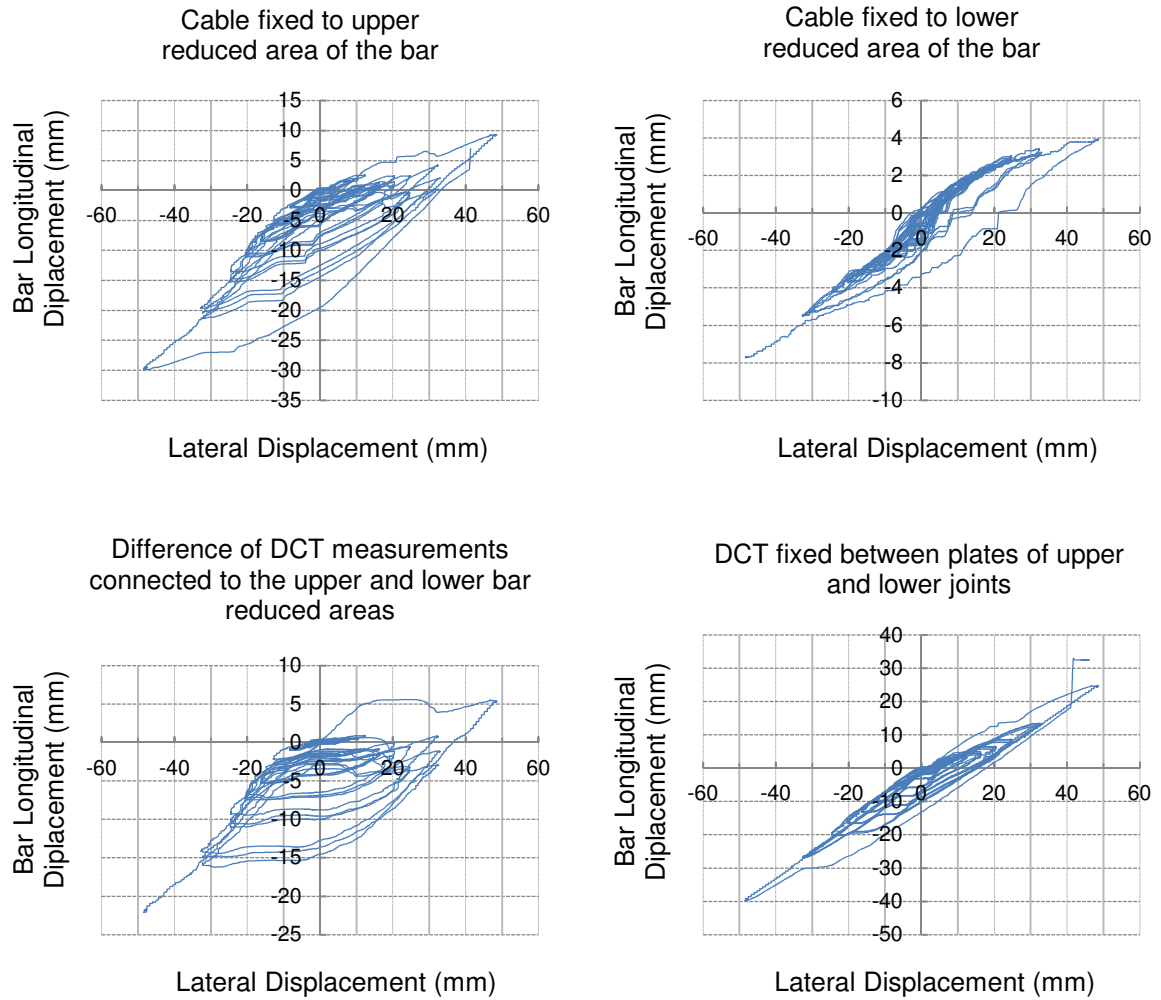


Figure VI.34: Frame lateral displacement vs. elongation of steel core bar recorded by displacement cable transducers

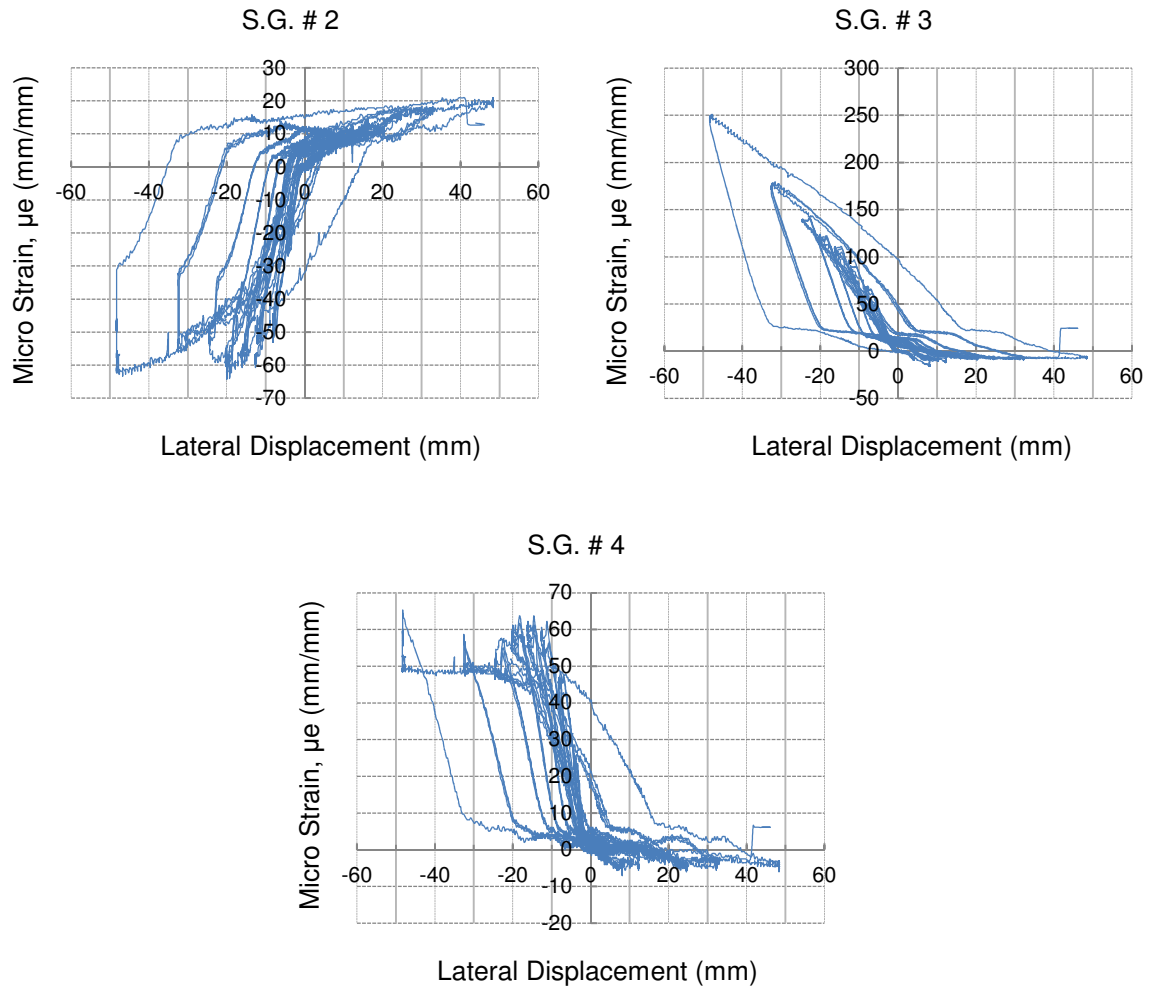


Figure VI.35: Frame lateral displacement vs. strain recorded by longitudinal strain gauges on the BRB HSS casing

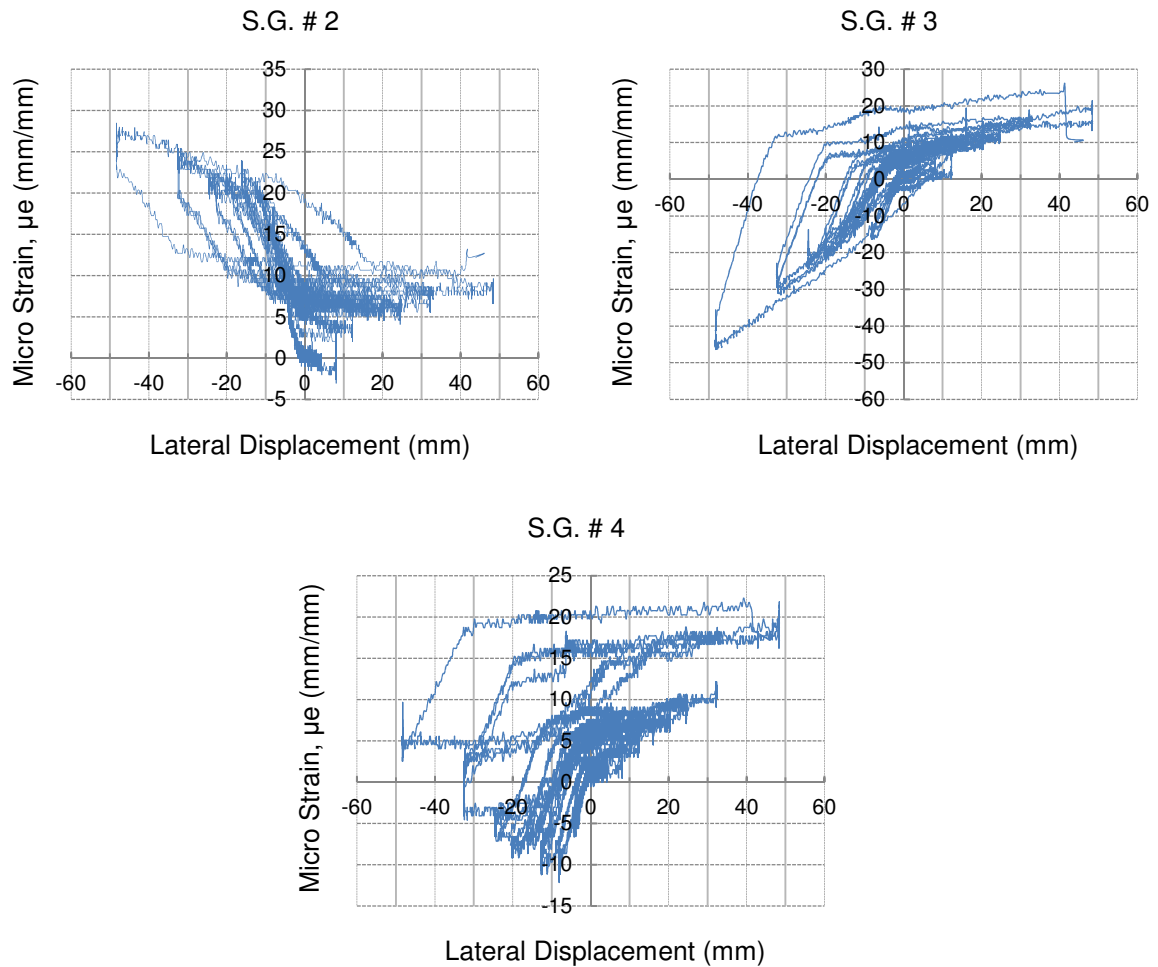


Figure VI.36: Frame lateral displacement vs. strain recorded by circumferential strain gauges on the BRB HSS casing

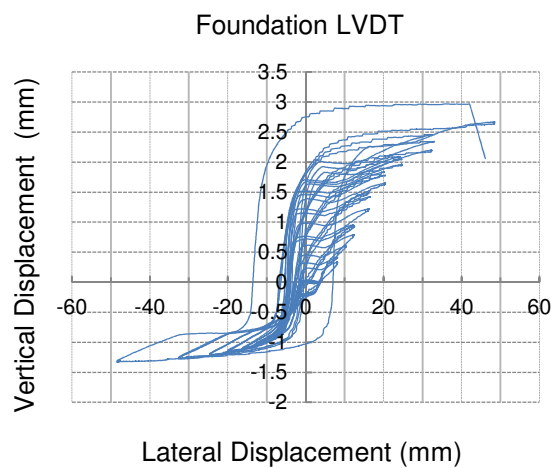


Figure VI.37: Frame lateral displacement vs. uplift displacement of the foundation