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STRENGTHENING OF DEFICIENT REINFORCED CONCRETE BUILDINGS USING A NEW TYPE OF BUCKLING RESTRAINED BRACE

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Abstract

This paper highlights research to assess and upgrade seismically deficient reinforced concrete frames using Buckling Restrained Braces (BRBs). A new BRB was developed by the authors at the University of Ottawa, Canada, and has been verified experimentally by testing two 2/3rd scale, single-storey reinforced concrete frames that were designed as representatives of frame buildings located in Vancouver, Canada. The first frame served as a control frame, while the second served as a companion-retrofitted frame. The retrofitted frame demonstrated excellent performance, with substantial increases in the lateral load capacity, coupled with drift control and energy dissipation. The research program has been extended to verify numerically the feasibility of the new technique in multi-storey reinforced concrete frame buildings located in western Canada. Inelastic response time history analyses were conducted using models that permit inelasticity in frame elements and the BRBs. Code-compatible earthquake records were selected for the analyses. The results indicate that multiple uses of BRBs improve building performance, reduce seismic shear demands in brittle frame columns while limiting lateral drift to 1%. The numerical results were assessed against performance-based design objectives prescribed in current codes. The paper presents the results of both the experimental and numerical investigations.

Keywords: buckling restrained brace; concrete frame, seismic retrofit; dynamic time-history analysis, inelastic modelling

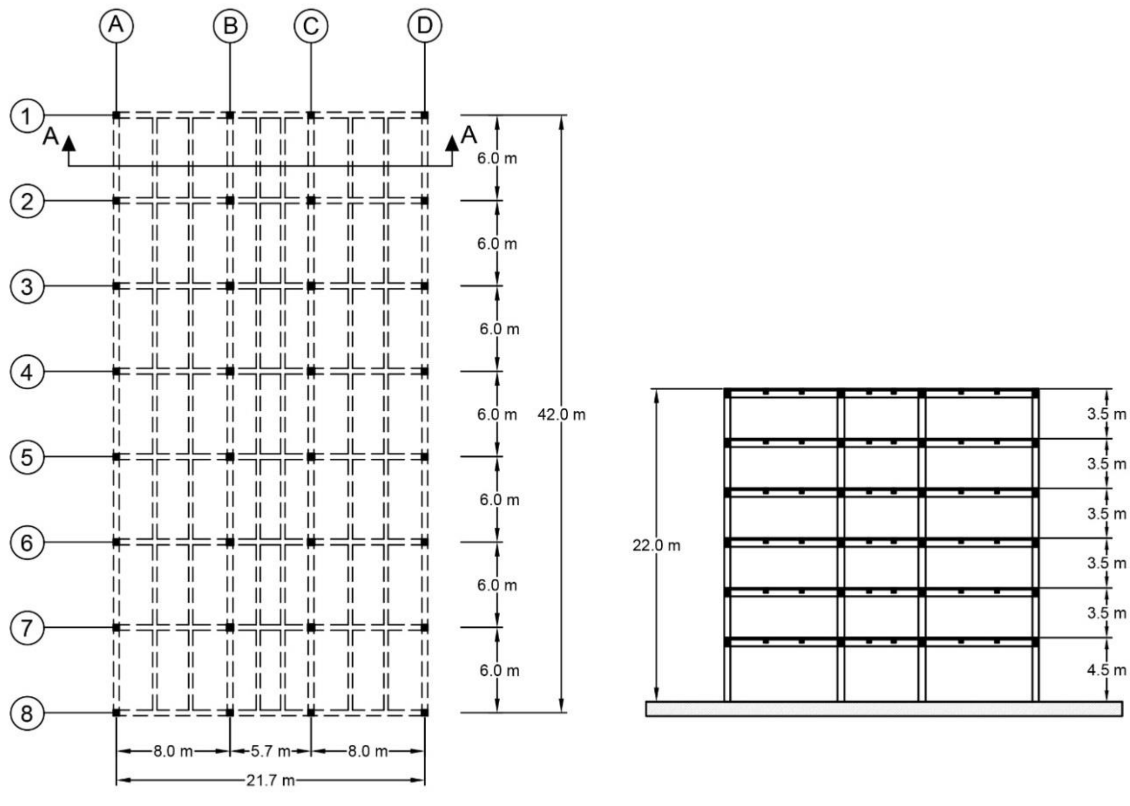
1. Introduction

The global building inventory consists of non-ductile reinforced concrete frames, designed and built prior to the enactment of modern seismic codes. These buildings are considered deficient with insufficient detailing of frame members, and lack of strength and ductility for responding adequetly to strong earthquakes, such as those prescribed in recent codes. In Canada, seismic design requirements in high seismic regions have increased by as much as 100 % since the early 1970's [1]. Therefore, ductile design and detailing requirements prescribed in newer codes to reduce seismic vulnerabilities were not implemented in the majority of existing older buildings. Vulnerability assessment is an integral component of seismic risk mitigation for existing buildings with a view to implement seismic retrofitting when appropriate. Buckling Restrained Braced (BRB) frames are a relatively new and special class of concentrically braced frames. They were initially developed in Japan [2] and extensively investigated over the last four decades [3]. They were used as hysteretic dampers for seismic strengthening frame buildings after the 1995 Kobe Earthquake. It has been suggested that the seismic performance, efficiency, and potential use in practice are superior to conventional bracing systems [4]. According to NIST [5], research on BRBs in Canada first started by Tremblay et. al [6]. The system was further deveopled by Tremblay et. al [7], Wu et al. [8], and Judd et al. [9]. However, limited experimental research has been performed to assess the behaviour of RC moment-resisting frames strengthened with buckling restrained braces. Furthermore insufficient research data is available on the challenges of connecting these braces to beam-column joints and assessing their adverse effects on columns due to the addional axial loads and secondary moments imposed [10].

The objective of this paper is to assess the effectiveness of a retrofit technique based on a new buckling restrained brace technology for seismically deficient reinforced concrete frame buildings, developed by the authors through experimental research. The seismic structural performance was first assessed experimentally under simulated seismic loading through testing two single storey, single bay reinforced concrete frames that represent 2/3 scale of a six-storey prototype frame building. One frame was tested as designed and the was tested after retrofitting with the new BRB system. Upon confirmation of successful performance of the new system in the laboratory, the performance of the prototype building , retrofitted with the new BRF system, was assessed through nonlinear dynamic time history analyses. The focus of the analyses was placed on the verification of the new retrofit technique in multi-storey reinforced concrete frame buildings in western Canada and assess its behaviour relative to the performance-based design objectives prescribed in recent codes.

2. Prototype Building

A 6-storey reinforced concrete Moment Resisting Frame (MRF) in Vancouver, Canada was used as the prototype building in this study. Figure 1 shows the plan and elevation views of the building. The building represented a typical seismically deficient older medium-rise building constructed between 1960s and early 1970s. The building is a modified version of the building used as a design example in the Canadian Concrete Design Handbook [11]. The typical interstorey height is 3.5 m with a first storey height of 4.5 m. The floor plan of 21.7 m x 42 m consists of 3 bays in the short direction and 7 bays in the long direction. The slab thickness is 110 mm and the interior and exterior columns are 500 mm x 500 mm and 450 mm x 450 mm, respectively. The girders are 400 mm wide and 600 mm deep and the beams are 300 mm wide and 350 mm deep. The building is intended for office occupancy.



(a) (b)

Fig. 1 - The prototype building: a) plan view; and b) elevation view (Section A-A)

The seismic design base shear for the prototype building was computed using the 1965 and 2010 editions of NBCC [12, 13] to assess the difference in design force levels between the assumed year of construction and the recent code requirement. The original design resulted in a non-ductile building. The corresponding building design for a non-ductile conventional building in the 2010 NBCC is performed using a ductility force modification factor (R_d) of 1.5 and an over-strength force modification factor (R_o) of 1.3. The comparison of the non-ductile original design and that of the 2010 NBCC for non-ductile conventional construction revealed that the strength capacity of the building had to be increased by a factor of 2.95 to satisfy the strength requirement of the 2010 NBCC. This assumes that the retrofit strategy would be based on strength increase only, without any consideration of ductility enhancement. The prototype building selected was used to demonstrate the application of the new retrofit system that was developed as part of the current research project, providing both strength and ductility enhancements. The details of the new BRB system are discussed in the following section.

3. BRB Design

The proposed retrofit scheme consists of a single diagonal BRB that utilizes full compression and tension force capacities during reversed cycling loading. The brace is connected to beam column joints through a steel assembly that consists of two HSS sections, one on either side of the joint, connected together by means of dywidag bars. The BRB consists of a ductile inner steel core, made from AISI 4140 chrome-molybdenum high-tensile steel, and a circular sleeve that consists of two circular steel sections with different diameters. The gap between the two sections is filled with cement mortar to provide additional lateral restraint against buckling of inner steel core when subjected to compressive stresses. The length of the steel core bar was 2700 mm, including the 220 mm threaded end sections to accommodate the connection to the end steel plate. The bar diameter was 1 ¾" (44.5 mm), while the diameter of the threaded sections was reduced to 1 ½" (38.3 mm). The diameter of the bar was further reduced to 1 ¼" (31.8 mm) for a length of 1350 mm along the mid-length segment to promote yielding in this region. The inner core bar was connected to novel end units that allow extension and contraction during tension-compression cycles under cyclic loading while providing lateral restraint to the bar against buckling. The core bar was inserted into a steel pipe having 59 mm inner diameter, which was encased by self-consolidating cement (Sikacrete-08). The bar-pipe assembly was then placed in an HSS casing (168 mm in diameter and 8 mm in thickness). Fig. 2 illustrates the dimensional details of the bare steel core bar and the components of the BRB.

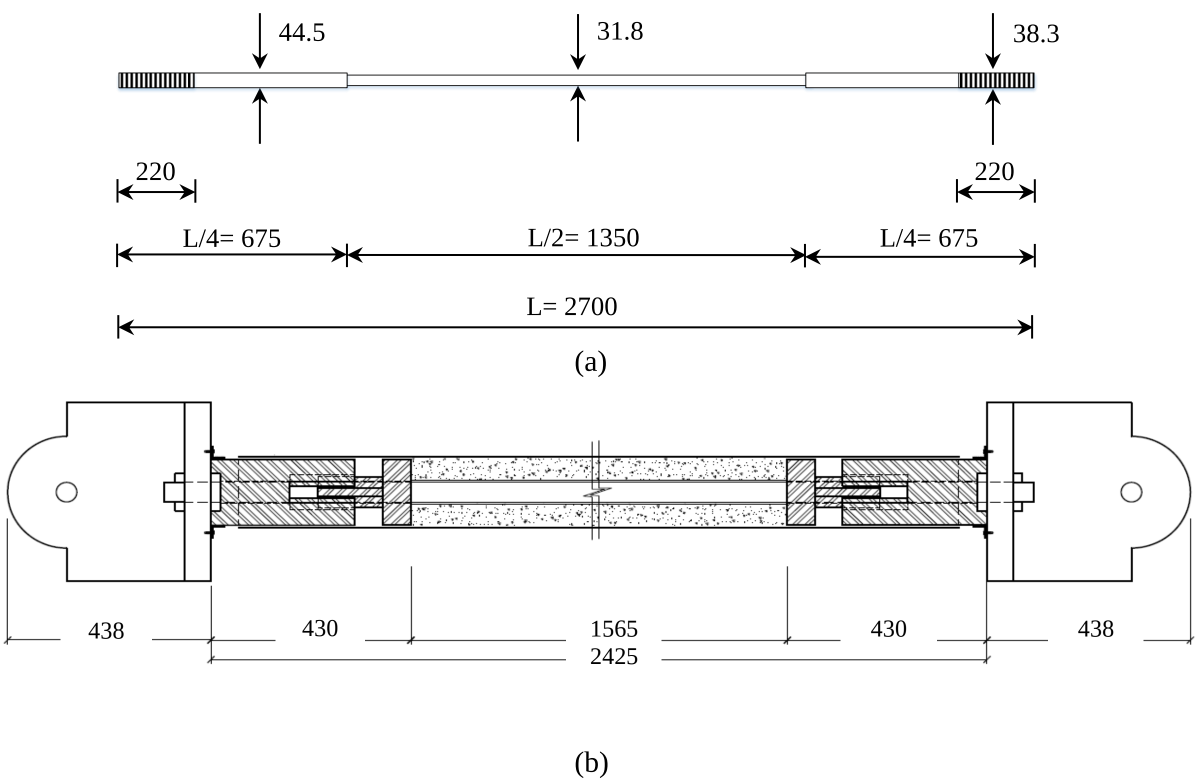


Fig. 2 - Side elevation of the BRB: a) steel core bar; and b) BRB components (All dimensions in mm)

4. Experimental Investigation

4.1 Test Specimen and Setup

The new BRB system was developed through experimental research. The experimental program consisted of testing two identical, 2/3rd scale, single storey-single bay reinforced concrete frames. The first frame served as the control frame while the second was retrofitted using the proposed BRB system. The frames were designed according to the combination of dead, live, wind, and earthquake loads, based on the 1965 edition of the NBCC. The test frames were built to represent an interior frame of the second storey, or an exterior frame of the ground floor of the 5.7 m long centre bay (Grid lines 7B-7C or 8B-8C) of the prototype building. The frame centre-to-centre 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 column reinforcement details consisted of 8-20M (19.5 mm) longitudinal bars, equaly distribured along the perimeter of the section, with 10M (11.3 mm) column ties with 90^0 hooks spaced at 200 mm along the column height. The beams had 4-20M top longitudinal bars continuous along the length of the beam and 2-20M bars continuous at the bottom with additional 2-15 M longitudinal bars placed at the top in the support regions and at the bottom in the mid-span region. 10M stirrups were used at 150 mm spacing along the full beam length. The frames were constructed on a rigid I-shaped foundation with 500 mm depth. The foundation at column location was 1500 mm wide, while the width of the foundation between the two columns was 500 mm. Concrete compressive strengths on the day of testing were 27.6 MPa and 28.2 MPa for the control and retrofitted frames, respectively. Average yield strength measured from tension coupon tests for 10M, 15M, and 20M reinforcing bars were 481 MPa, 450 MPa, and 430 MPa, respectively.

Four high-strength bolts were used to secure the frames to the laboratory strong floor to provide fixity at the foundation level. The gravity loading on the columns and the beam were applied through seven-wire prestressing strands (size 15). These loads were established and scaled from the prototype frame building. The frames were subjected to in-plane reverse cyclic lateral displacements using a hydraulic actuator that was set parallel to the frame at the beam level. The displacements were incrementally increased to simulate seismic loading. The frames were instrumented with Displacement Cable Transducers (DCTs), Linear Variable Displacement Transducers (LVDTs), and electrical resistance strain gauges that were placed on the internal reinforcing steel. Fig. 3 illustrates the setup of the frames before testing.

(a) (b)

Fig. 3 - Setup prior to testing: a) control frame; and b) retrofitted frame

4.2 Test Results

The results presented herein include observations and data recorded during testing of the seismically deficient reinforced concrete bare control frame and the retrofitted frame.

Backbone curve of the hysteretic force-deformation relationship for the control frame and full hysteretic relationship for the retrofitted frame are shown in Fig. 4. Based on the backbone curve, the control frame experienced initiation of yielding at a drift of 1.3% (41 mm). This yield displacement for the control frame 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 capacity. This is based on the definition of yield for reinforced concrete members established by Park [14]. At 2.5% lateral drift, the frame attained its maximum lateral resistance of 233 kN and 219 kN during the first cycle of loading in the push and pull directions, respectively. The frame reached its maximum drift capacity of 3% in both push and pull directions. This drift capacity corresponded to displacement at 20% strength decay beyond the peak resistance. At this drift, the frame experienced significant spalling of concrete cover, which resulted in exposing the longitudinal reinforcing bars in the columns and the beam near the joints. The corresponding displacement ductility capacity was 2.3.

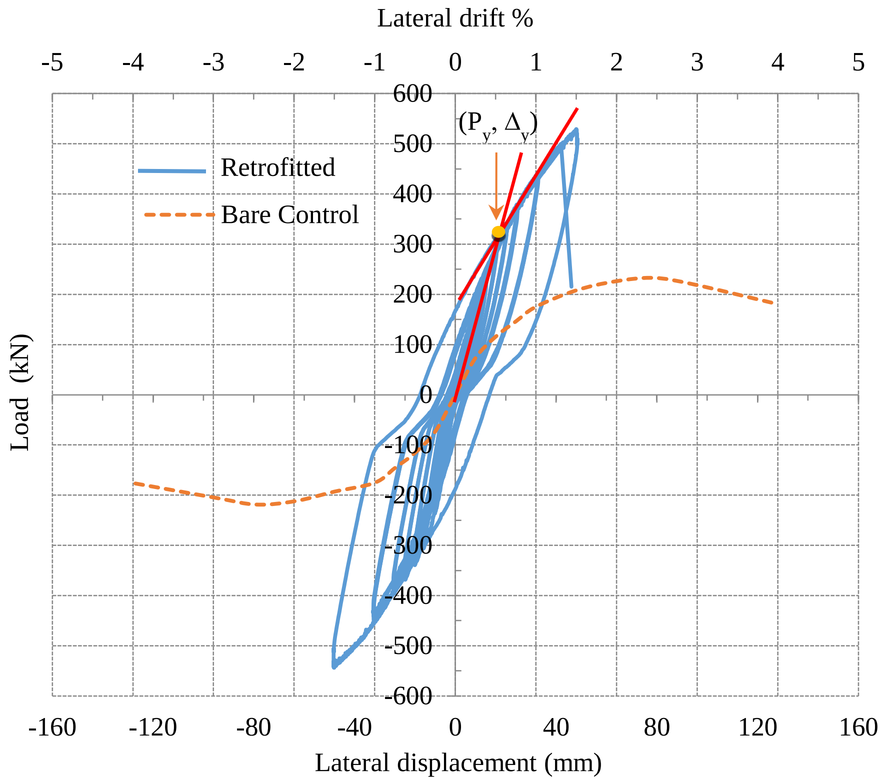
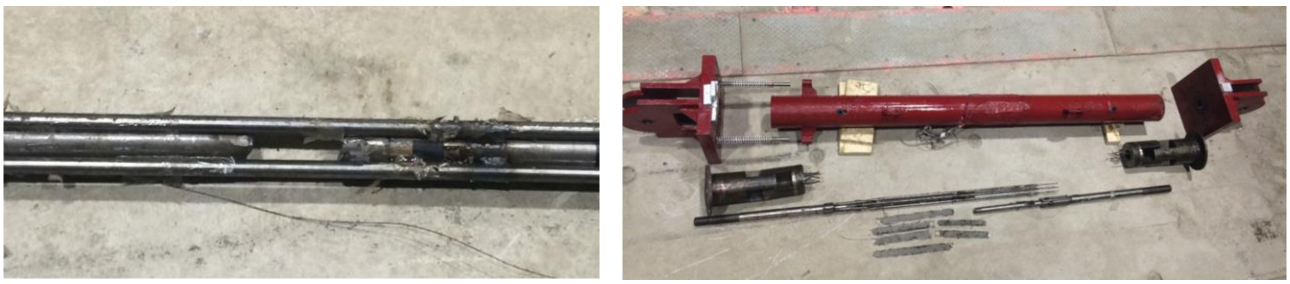


Fig. 4 - Envelope and hysteretic lateral load-lateral displacement responses for the control and retrofitted frames

The hysteretic lateral load-lateral displacement response of the retrofitted frame indicated that yielding of the BRB steel core and yielding of the concrete frame, based on the data obtained from strain gauges, controlled the softening response of the global behaviour. The hysteretic response illustrates rigid elastic behaviour until the yielding of the BRB core steel at a lateral drift of 0.5% (16 mm), after which further softening was observed at a drift of 1.2% (38 mm). This further softening corresponded to the yielding of column longitudinal reinforcement near the base. Nevertheless, the frame remained intact due to to the preserved capacity of the frame members in the post-yield range and remained capable of sustaining the imposed gravity and lateral loads. The yielding of the composite structure (the concrete frame and the steel bar in BRB) were established graphically by intersecting two lines on the envelope of the hysteretic response: one parallel to the initial stiffness, and the other parallel to the ascending curve of the hysteretic response. The frame attained its maximum drift level during the first push cycle to 48 mm (1.5% drift) which corresponded to a lateral strength capacity of 529 kN. During the second push cycle towards the maximum drift, the BRB steel core bar fractured in the middle segment of the bar where the bar area had been reduced. The lateral load capacities of the retrofitted frame in two directions were 510 kN at a lateral drift capacity of 1.33% (42 mm), and 543 kN at a lateral drift capacity of 1.5% (48 mm) during the push and pull cycles, respectively. This implies an increase in force capacity of 2.2 and 2.5 relative to the control frame. The retrofitted frame experienced similar lateral force capacities in both directions of loading, providing evidence that the brace system is capable of preventing buckling and promoting yielding in tension and compression. The ductility ratios were calculated based on the maximum attained frame displacement capacity prior to the BRB core bar fracture, to the frame yield displacement in the push and pull directions, respectively. The retrofitted frame attained ductility ratios of 2.6 and 3.0 in the push and pull directions, respectively. This marked an increase in ductility of 1.13 and 1.29, respectively, relative to the control frame. Fig. 5 illustrates the BRB core bar after rupturing and the components after dismantling the BRB.

The cumulative energy absorbed was calculated for both the control and retrofitted frames from the area of closed loops of the first push cycles of the hysteretic lateral load-lateral displacement response. The retrofitted frame was capable of dissipating 26027 kN.mm of energy at a lateral displacement of 48 mm (1.5% drift ratio). This corresponded to an increase factor of 6.6 relative to the control frame (3937 kN.mm) at the same lateral displacement.



(a) (b)

Fig. 5 - BRB core bar at failure: a) rupture of BRB core bar; and b) BRB components after dismantling

5. Implementation in Practice

5.1 Design of the Prototype Building

The six-storey moment-resisting frame prototype building, described in Fig. 1, was subjected to nonlinear time-history response analysis. The building contained two exterior and six interior frames and was designed for the level of seismicity expected in western Canada, specifically for the City of Vancouver. SAP2000 [15] was used to conduct linear elastic static analysis for design purposes. The building model consisted of individual two-dimensional lumped frames for external and internal frames, connected by rigid diaphragms. 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. Design moments, axial forces and shear forces were computed for frame members, and section sizes and reinforcement details were determined for beams and columns. Tables 1 and 2 provide a summary of the section sizes and reinforcement details of the beams and columns, respectively.

Table 1: Beam section sizes and reinforcement arrangements

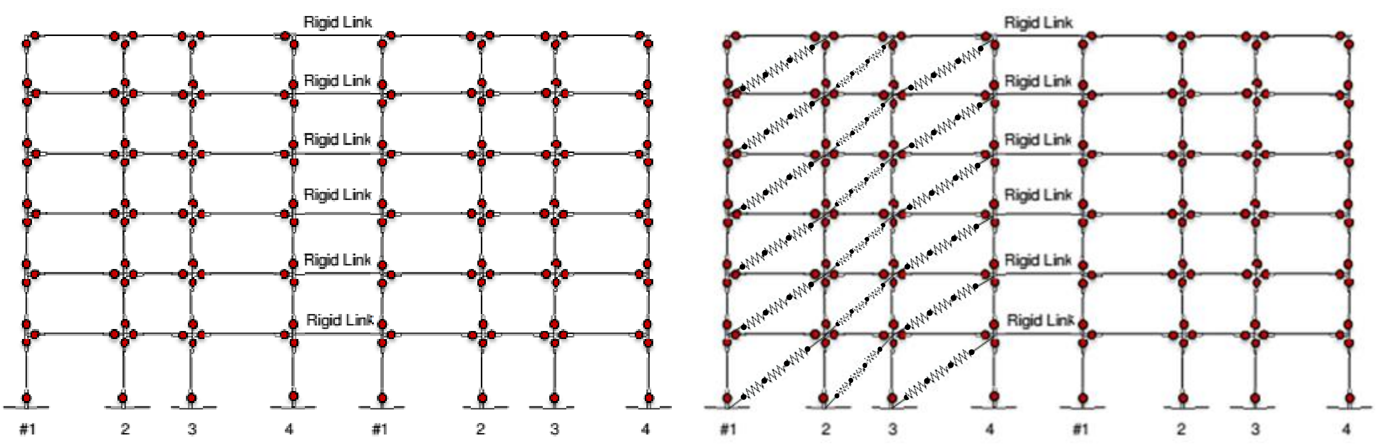
Frame (Grid)	Beam (Grid)	Floor Level	Beam section (mm) and reinforcement			
			Section (mm)	Top steel	Bot. steel	Stirrups (Bar No. @ mm)
Interior (2-7)	A-D	1	400 x 600	8-25M	7-20M	10M @ 75
	A-D	2-6	400 x 600	7-25M	5-20M	10M @ 100
Exterior (1&8)	A-D	1	400 x 600	6-25M	5-20M	10M @ 150
	A-D	2-6	400 x 600	5-25M	4-20M	10M @ 175

Table 2: Column section sizes and reinforcement arrangements

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	500 x 500	12-25M	10M @ 125	450 x 450	8-25M	10M @ 150
	2-6	500 x 500	8-25M	10M @ 100	450 x 450	8-25M	10M @ 125
Exterior (1&8)	1	450 x 450	12-25M	10M @ 150	450 x 450	8-25M	10M @ 175
	2-6	450 x 450	8-25M	10M @ 125	450 x 450	8-25M	10M @ 150

5.2 Linear and Nonlinear Mathematical Modeling

A two-dimensional lumped frame control model, shown in Fig. 6(a) was developed in SAP2000 for seismic analysis of the prototype building illustrated in Fig. 1. The interior frames were lumped together and connected to the exterior frames, also lumped together, to simulate the entire structural stiffness and strength. Rigid links with infinite axial rigidity and negligible flexural rigidity were used to connect the lumped frames to simulate rigid floor diaphragms, ensuring equal displacements at each floor level without transferring moments. The gravity loads were calculated based on the loads prescribed in NBCC 2010, which were then converted into masses specified at each floor, and assigned to the floor joints. The frame elements were modeled using linear elastic elements with reduced flexural rigidities of $0.4EI_g$ and $0.67EI_g$ for beams and columns, respectively, to account for the cracking effects as per the recommendations in CSA A23.3 [16]. A similar approach of having effective flexural rigidity values is allowed in ASCE 41-13 [17]. The columns were fully fixed at the base of the building, and beam-column joints were considered infinitely rigid.



2 Lumped Exterior Frames

6 Lumped Interior Frames

2 Lumped Exterior Frames

6 Lumped Interior Frames

(a)

(b)

Fig. 6 - Locations of lumped plastic hinges: a) control building; and b) retrofitted building

Material nonlinearity was modeled within the control building using "Links" that were concentrated at the center of the plastic hinges within the ends of each frame element (displacements for the control retrofitted. 6(a)). These hinging regions incorporated all the lumped linear and nonlinear properties that were determined by performing sectional analyses and defined by moment-rotation properties for the flexural frame members.

The nonlinear properties of the retrofitted building are similar to those of the control building with the exception of adding diagonal BRBs at each floor level of the exterior frames, as illustrated schematically in Fig. 6(b). Each BRB steel core bar, illustrated schematically in Fig. 2(a), was modeled with three link elements connected in parallel, and were assigned nonlinear force-deformation properties in the uniaxial direction. They represented the reduced yielding segment (middle ½ length) and two un-reduced elastic segments (each of ¼ end length). The mechanical properties of the BRBs used for the retrofitted building are summarized in Table 3. 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 test frame with an assigned high flexural rigidity (factor of unity for fully rigid end assignment). Moment releases were also specified at joints connecting these rigid frame elements with BRB core link elements to simulate hinge connections and eliminate any secondary moments at BRB bar ends.

Table 3: Mechanical properties of BRBs used in the retrofitted building

BRB bay location within external frames	Floor #	Reduced section				Un-reduced sections			
		Diam. (mm)	Yield def. (mm)	Yield force (kN)	Stiffness (x 10^3 kN/m)	Diam. (mm)	Yield def. (mm)	Yield force (kN)	Stiffness (x 10^3 kN/m)
Sides	5-6	31.8	7.5	353	47.1	38.1	3.7	508	135.7
	3-4	38.1	7.5	508	67.9	44.5	3.7	692	184.7
	2	44.5	7.5	692	92.4	50.8	3.7	904	241.3
	1	44.5	8.1	692	85.3	50.8	4.1	904	222.9
Central	5-6	31.8	5.1	353	68.6	38.1	2.6	508	197.4
	3-4	38.1	5.1	508	98.7	44.5	2.6	692	268.7
	2	44.5	5.	692	134.4	50.8	2.6	904	351.0
	1	44.5	5.9	692	116.0	50.8	2.9	904	303.1

5.3 Selection of Earthquake Ground Records

Uniform Hazard Spectrum (UHS)-compatible ground motion records, generated synthetically by Atkinson [18], were used in dynamic analyses. Twelve synthetic earthquake records were selected from 225 artificial earthquake records prescribed for the City of Vancouver, as they were the best matches for the UHS. The records consisted of four short duration records, four long duration records, and four Cascadia records. The average values of the four records in each set were calculated and then compared with the NBCC UHS for the City of Vancouver as shown in Fig. 7.

6. Results of Dynamic Analyses

The primary objective of dynamic analysis was to assess the deficiencies of older reinforced concrete buildings that were designed prior to the enactment of modern seismic code provisions incorporated in recent building codes. In addition, the objective was to evaluate the effectiveness of the newly developed BRB seismic retrofit methodology for strength and ductility enhancements. Assessment of the performance of beam-column joints was not within the scope of this paper. However, there was no significant softening observed in the joints during the experimental program, within the drift range considered in the analysis as acceptable lateral drift level to ensure frame safety. Response time-history analyses of the control and retrofitted buildings were performed under the earthquake records whose response spectra are shown in Fig. 7. Their behaviours were assessed relative to the performance-based design objectives stated in ACI 374.2R-13 [19], ASCE 41-06 [20], and ASCE 41-13 [17].

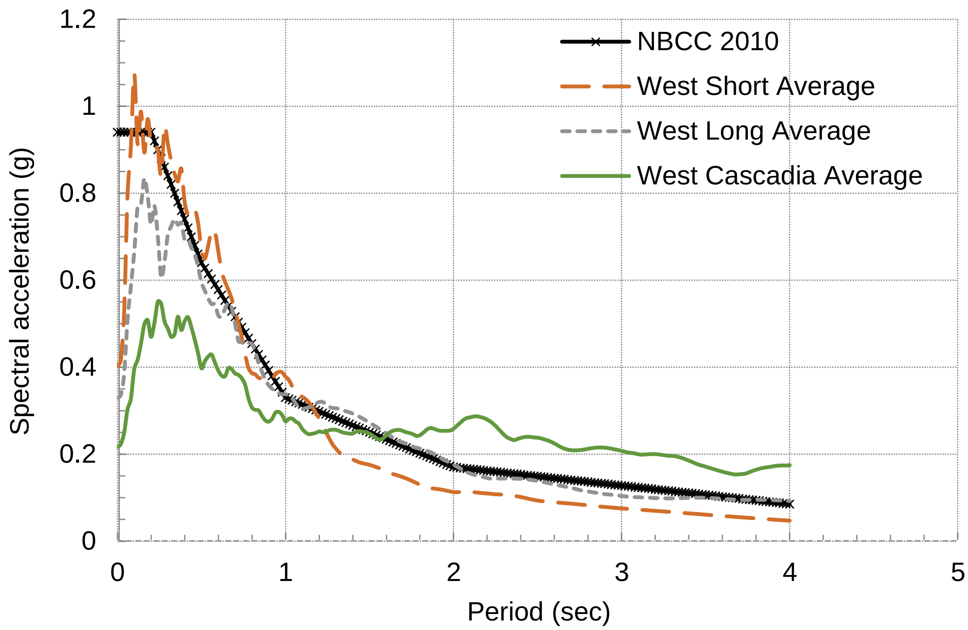


Fig. 7 - Average of response spectra for short, long, and Cascadia events used in analyses of the buildings in Vancouver

The lateral response quantities used included both interstorey and overall roof drift ratios. Dynamic base shear demands were assessed to investigate the effectiveness of BRB retrofitting. The base shear was also calculated using the Equivalent Static Force Procedure (ESFP) of NBCC 1965. Shear force, bending moment, and axial force response histories of columns were investigated; the moment-chord rotation hysteretic relationships of critical columns and beams were examined. The axial force-axial deformation response for each BRB used in the retrofitted building was investigated to assess their performance.

6.1 Drift Response

The un-retrofitted building experienced a maximum interstorey drift of 2.3%, recorded at the first floor under the Cascadia Event #1 earthquake record. Therefore the building is considered to be within the Collapse Prevention (S-5) structural performance level. The maximum top lateral displacement was 264 mm (1.2% overall drift for the building), recorded under the same Cascadia Event # 1. Through retrofitting, the interstorey drifts were reduced to a maximum of 0.92%, recorded at the first storey level. Accordingly, the structural performance level of the building at the maximum drift response was at the Life Safety (S-3) level, and the maximum top lateral displacement was reduced to 143 mm corresponding to an overall drift of 0.65%.

6.2 Maximum Base Shears

The base shear-time histories of the control building was analyzed for the 12 earthquake records considered, and then compared with those obtained from the ESFP of NBCC 1965. The maximum 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 37% of the maximum dynamic base shear. When retrofitted, the building was analyzed under the same critical earthquake record, the Cascadia Event #1. The dynamic base shear was increased by 0.2 % (9130 kN).

6.3 Column Demands vs. Capacities

The control building column shear forces, bending moments, and axial forces recorded from the response history analysis for the critical earthquake record (i.e., Cascadia Event #1) were investigated. They were then compared with nominal sectional capacities. Shear capacities of the columns in exterior frames were exceeded in 14 columns in the first three stories of the building. In addition, the shear capacities of 30 columns in interior frames were exceeded in the first and second stories. Therefore 44 columns in total were shear deficient. Deficient columns are identified in Fig. 8, along with computed shear force, bending moment, and axial force demands for all the frame members. In contrast, the retrofitted building subjected to the same shear-critical earthquake record developed force demands that were below the member capacities. Therefore, none of the building columns was shear deficient in the retrofitted building.

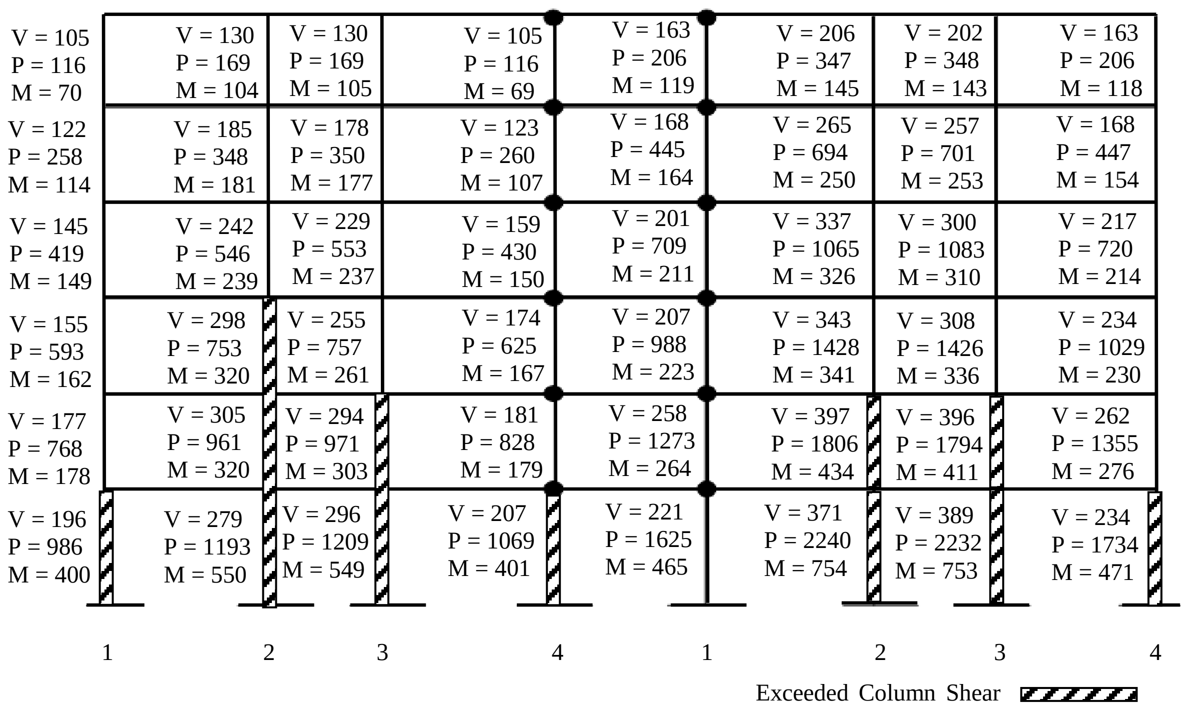


Fig. 8 - Column shear force, axial force, and bending moment demands for the control building subjected to Cascadia Event #1

6.4 Moment-Chord Rotation Hysteretic Responses

The moment-cord rotation responses for selected critical elements of the un-retrofitted (control) and retrofitted buildings, under shear-critical Cascadia Event # 1, were examined to assess the hysteretic characteristics of these 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 frames (Fig. 9). 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. The beams of the exterior and interior frames both developed 97% of their moment capacities and maximum rotational ductilities of 2.7 and 3.1, respectively. The columns of the exterior and interior frames experienced moments that slightly exceeded their capacities (102% and 105%, respectively), and rotations of 127% and 178% of their ultimate rotation capacities, respectively. The columns developed maximum ductilities of 5.4 and 6.6 for the exterior and interior frames, respectively. After retrofitting, 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. Fig. 9 illustrates the inelasticity developed in critical columns of the exterior frames of the control and retrofitted buildings.

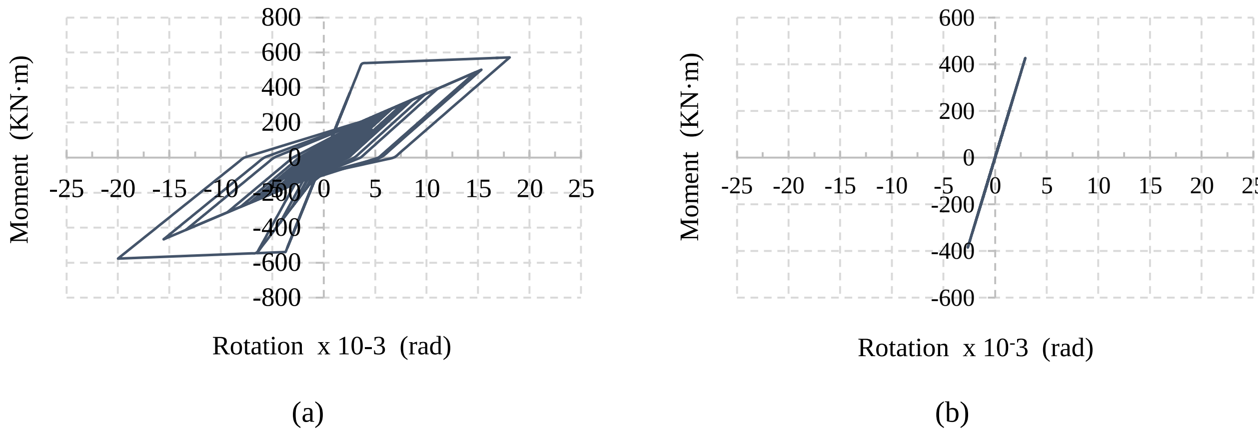


Fig. 9: Moment-cord rotation hysteretic responses of columns at the first storey of the exterior frames due to Cascadia Event #1: a) control building; and b) retrofitted building

6.5 Ductility of the BRBs

The ductility provided by BRBs was examined at all floor levels, under the governing earthquake motion (Cascadia Event #1). The axial force-axial deformation response was investigated by examining the behaviour of link elements in the structural model, representing the reduced section of the BRB steel core bars (Fig. 2(a) and Fig. 6(b)). The un-reduced sections of BRB steel core bars were investigated only at the first-floor level; they were expected to behave elastically elsewhere. The left and right retrofitted bays demonstrated approximately similar response due to symmetry, with lower ductility demands than that for the central bay. The maximum BRB ductility demands for the central bay were 2.95, 2.57, 2.49, 1.82, 1.12, and 0.53 in the reduced sections for the first to sixth stories, respectively. The BRBs of the first storey developed a maximum strain of 0.66%, while the strains developed in the BRBs in upper stories progressively decreased to 0.12% at the sixth storey. None of the BRBs exceeded the maximum strain of 2.5% prescribed by the ASCE 41-13 (ASCE (2014). Furthermore, all BRB responses were within the Enhanced Safety structural performance range, never exceeding the Damage Control (S-2) performance level. Fig. 10 depicts hysteretic responses of reduced sections of the buckling restrained brace at the first storey level. In un-reduced sections, the BRB of the first storey developed a maximum ductility demand of 1.47 and a maximum strain of 0.33%.

7. Conclusions

Experimental investigation was conducted to develop a new Buckling Restrained Brace (BRB) as a seismic retrofit strategy. The effectiveness of the BRB system for reinforced concrete frame buildings was verified experimentally and analytically. A six-storey reinforced concrete frame building was designed for Vancouver by following the requirements of the 1965 NBCC as a prototype building built prior to the enactment of modern building codes. A single bay of the prototype building was scaled to 2/3rd of the actual dimensions, and used as test frames for a laboratory investigation. Two companion frames of the same geometry and structural characteristics were built for testing under slowly applied lateral deformation reversals. The first frame served as the control frame while the second frame was retrofitted using the new BRB system. The BRB resulted in increases in the lateral load capacity by a factor of 2.2 in the push direction and 2.5 in pull direction. The retrofitted frame attained a ductility of 2.6 and 3.0, an increase of 1.13 and 1.29 relative to the control frame in the push and pull directions, respectively. The retrofitted frame remained intact for continued resistance to gravity loads, while providing enhanced lateral load resistance to earthquake loads.

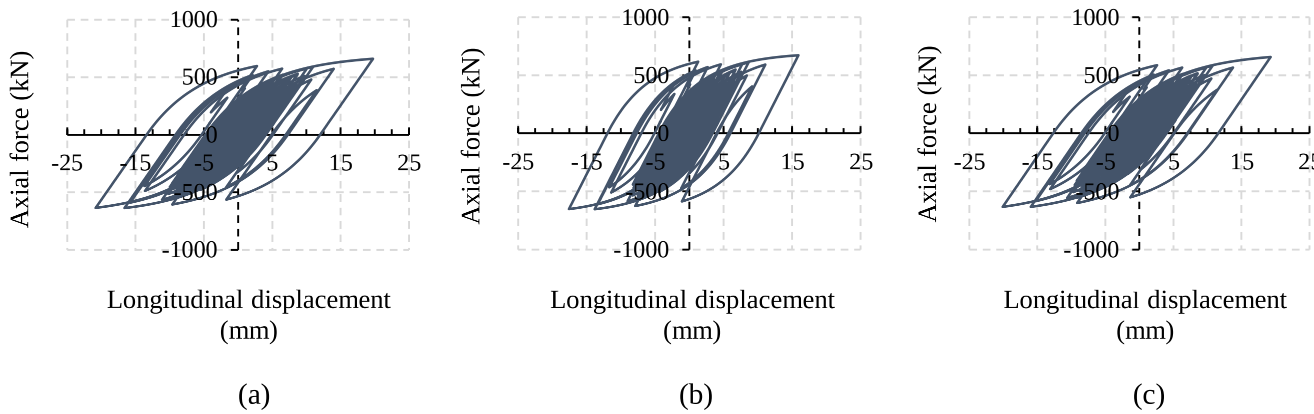


Fig. 10 - Axial force-axial deformation hysteretic responses for the BRB reduced sections at the first storey of the retrofitted building: a) left bay; b) central bay; and c) right bay

The application of the retrofit system to multistorey buildings was investigated by performing inelastic time-history analysis of the prototype building. The structure was modelled to simulate inelasticity in frame elements and BRBs. Computer software SAP2000 was used with UHS compatible earthquake records consistent with the 2010 edition of the NBCC for Vancouver. The un-retrofitted building developed a maximum interstorey drift of 2.3%, which was within the Collapse Prevention structural performance level. The dynamic base shear demand of the building under the critical earthquake record was approximately 2.7 times the original design base shear as per NBCC-1965. Shear capacity was exceeded in 44 columns in the first three stories. The beams developed maximum rotational ductility ratios of 2.7 and 3.1 in exterior and interior frames, respectively. The columns developed maximum ductility ratios of 5.4 and 6.6 for exterior and interior frames, respectively. The un-retrofitted building was deemed to be vulnerable to the 2010 NBCC.

Retrofitting the building with the new BRB system, placed in a single bay of both exterior frames, resulted in a reduced maximum interstorey drift ratio of 0.92%. In addition, all shear force, axial force, and bending moment demands remained below member capacities. The frame members remained elastic during response. This implies that the building fulfilled the Immediate Occupancy performance objective. The BRBs exhibited stable hysteretic behaviour at all floor levels. The BRB core developed a maximum ductility demand of 2.95 at the first floor level, corresponding to a maximum strain of 0.66%. Therefore, all the BRBs were within the Enhanced Safety structural performance range.

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