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RETROFIT OF SEISMICALLY DEFICIENT REINFORCED CONCRETE FRAME WITH BUCKLING RESTRAINED BRACE

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ABSTRACT: Reinforced concrete frame buildings designed and built prior to the enactment of modern seismic codes of the post-1970 era are seismically vulnerable, particularly when they are subjected to strong ground motions. An experimental investigation was conducted to investigate the effectiveness of retrofitting seismically deficient reinforced concrete frame using Buckling Restrained Brace (BRB). Two, 2/3rd scale, single bay and single storey reinforced concrete test frames were designed as representative of a building frame located in Vancouver, Canada, following the requirements of the 1965 National Building Code of Canada (NBCC). The first served as a control frame while the second served as a companion retrofitted frame. The control frame was assessed to be seismically deficient, having seismic base shear capacity approximately equal to 34% of that required by the 2010 edition of the NBCC. The retrofit involved the application of a proposed BRB that was conceived and developed by the authors. The BRB consists of a circular steel bar that is free to yield in tension and is encased in a circular steel tube to prevent buckling in compression. In addition, specially designed ends permit compression yielding without buckling. The brace was connected to the deficient frame along one of the diagonals by means of end steel hinge joints. The retrofitted frame tested in the current research project provided an increase in the lateral strength of approximately 2.5 times relative to the control reference frame. Furthermore, the stiffness and the energy dissipation capacity of the frame were improved through bracing with the BRB.

1. Introduction

There is a considerable concern regarding the stock of reinforced concrete structures around the world, built prior to the enactment of modern building codes in seismically active regions. These structures, for the most part, were designed and built to sustain gravity loads only as prescribed in earlier building codes. In addition, these structures do not conform to modern ductility-prescribed code provisions, which are necessary to survive design-level earthquakes. Moment resisting frames are among the stock of structures that are vulnerable. This particular structural type has inadequate capacity to withstand current-seismic demands and, therefore, potentially poses risk to society. Significant effort has been focused on experimental research to develop strategies to retrofit deficient existing structures. Among retrofitting strategies that have been investigated includes diagonal bracing to improve the strength and stiffness of frames. Buckling Restrained Braces (BRBs) is a class of brace initially developed in Japan by Watanabe (1988) and further examined in the United States through large scale testing of single braces (Clark et al. 1999). Therefore, BRBs are relatively new and the experimental studies that assess the seismic performance of deficient reinforced concrete moment resisting frames retrofitted with these braces are scarce (Dinu et al., 2011 and Maheri and Sahebi, 1997)).

The main objective of this experimental research is to upgrade seismically deficient reinforced concrete frame structures to meet the design requirement associated with seismic demands prescribed in recent building codes. A novel buckling restrained brace (BRB) was conceived and developed, and used as a means to retrofit and improve the performance of deficient reinforced concrete frame structures. This experimental study involved simulated seismic testing of two identically designed large-scale (2/3rd scale) test frames. The first served as a control frame while the second frame served as a retrofitted frame.

2. Prototype Building

The prototype building investigated in this study is a 6-storey reinforced concrete moment resisting frame (MRF) structure located in Vancouver, representing a typical seismically deficient older medium-rise building constructed during the 1960s and early 1970s as shown in Figure 1. The building was modified from that described in the Concrete Design Handbook (Cement Association of Canada, 2006). The modifications included changes to the plan dimensions in the E-W direction and the inter-storey heights. The inter-storey heights are 3.5 m with a first storey of 4.5 m. The floor plan of 21.7 m x 42 m consists of 3 bays in the N-S direction in which the two end bays are 8 m in width and the centre bay is 5.7 m; while there are 7 bays of 6 m width each in the E-W direction. 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 intended for office use occupancy.

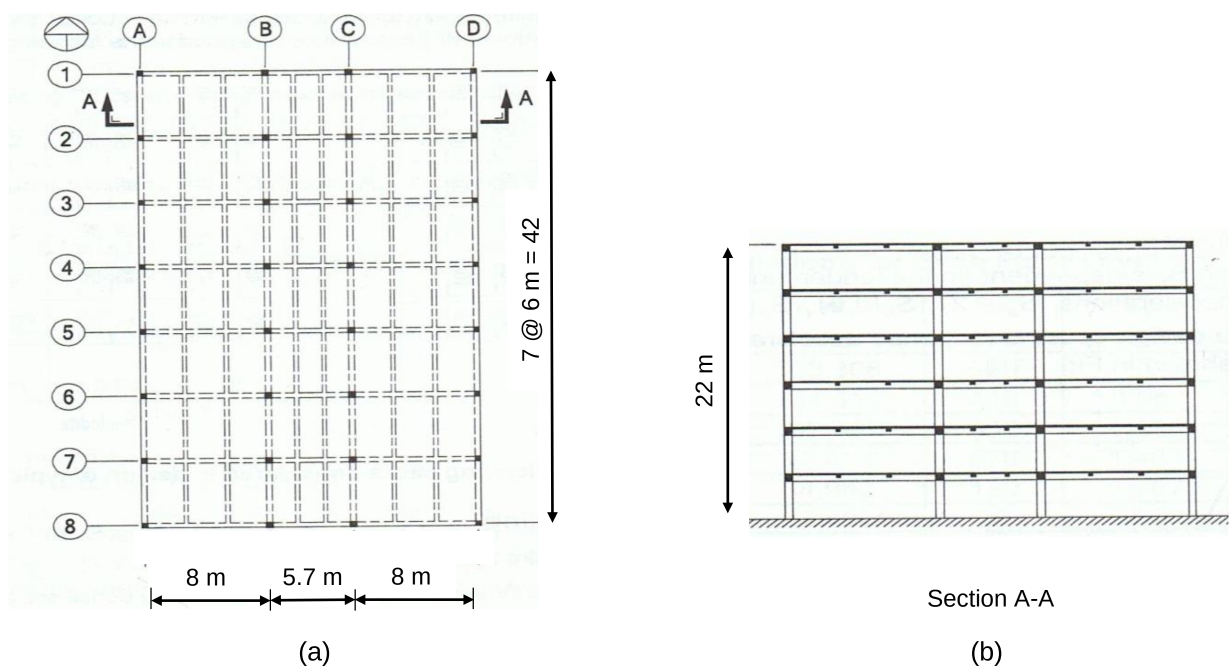


Figure 1: Proposed prototype building: a) plan view; and b) elevation view (modified from Cement Association of Canada, 2006)

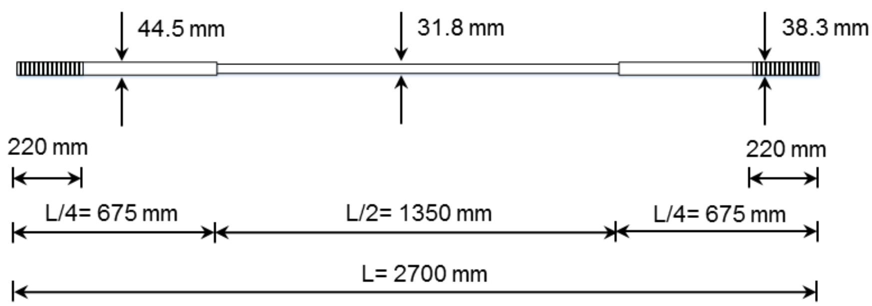
The seismic design base shear for the above building was computed using the requirements of the 1965 (National Building Code of Canada, 1965) and 2010 (National Building Code of Canada, 2010) editions of NBCC. The base shear force was calculated based on the assumption of non-ductile construction type for which coefficient C was taken to be equal to 1.25, and building of normal importance, not designed for post disaster occupancy, hence having an importance factor of 1.0. The foundation factor F was assumed equal to 1 for non-highly compressible soil conditions. The study revealed that the strength capacity of buildings constructed in Vancouver based on NBCC 1965 should be upgraded by a factor of 2.95 to satisfy the strength requirements of recent codes. This is based on a ductility force-modification factor

(R_d) of 1.5 and an over strength force-modification factor (R_o) of 1.3 as prescribed in NBCC 2010 for conventional MRF concrete buildings.

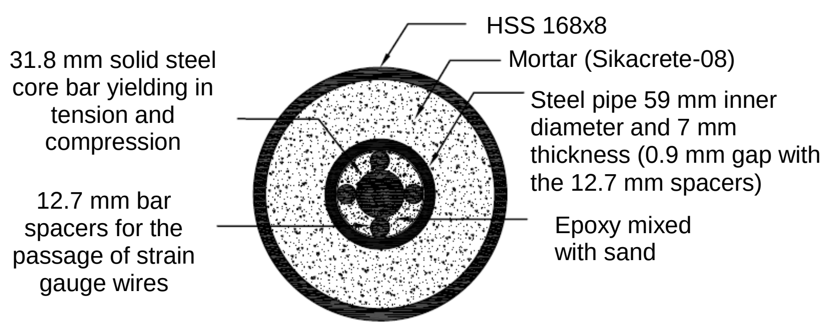
3. Proposed Retrofit Scheme

The proposed frame retrofit methodology consists of a single diagonal BRB that utilizes the full compression and tension force 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 similar lateral strength, stiffness and energy dissipation capacities during reverse cycling loading.

The BRB components consist of a circular steel core and a restraining system that prevents the steel core from buckling over its entire length. AISI 4140 chrome-molybdenum high-tensile steel bar was used for the core. The length of the steel core bar was 2700 mm, including threaded sections of 220 mm to accommodate the connection to the joint steel plates at both ends of the brace. [Note that the bar type selected is typically available as an off-the-shelf product.] The original diameter was 1 ¾" (44.5 mm), while the diameter of the threaded sections was 38.3 mm. The bar area was reduced around mid-length to promote yielding of the bar at this location and away from the threaded end sections. The bar diameter of the reduced area section was 31.8 mm. Four, 6.35 mm-diameter spacers were welded to the steel bar along the bar length to accommodate the strain gauge lead wires placed on the core bar. A thin coat of epoxy-sand mixture was used to fill the area between the spacers. The core bar was inserted into a steel pipe with 59 mm inner diameter, which was encased by mortar (Sikacrete-08) and HSS casing (168 x 8 mm). Figure 2 illustrates details of the steel core bar, the core cross-section, and the encasing components.



(a)



(b)

Figure 2: Details of the BRB steel core bar: a) side view; and b) cross section

Testing of the steel core bar in direct tension provided yield strength of 390 MPa corresponding to a strain of 0.38%. The ultimate tensile strength was 740 MPa corresponding to a strain of 10.8%, and the modulus of elasticity was 190.5 GPa.

4. Experimental Program

4.1. Test Specimens

Two, 2/3rd scale test frames were designed and built to represent an interior frame of the second storey or an exterior frame of the ground floor level of the 5.7 m-wide centre bay (Grid lines 7B-7C) of the six-storey prototype building shown in Figure 1. The frames were designed according to the 1965 edition of the NBCC load combinations for dead, live, wind, and earthquake. Concrete compressive strength of the control and the retrofitted frames were 27.6 MPa and 28.2 MPa, respectively; while the average yield strength measured from tension coupon tests for the 10M, 15M, and 20M reinforcing bars were 481 MPa, 450 MPa, and 430 MPa, respectively.

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 frames were built on a rigid I-shaped foundation of 500 mm depth. The foundations at the locations of the columns were 1500 mm wide, while the width of the foundation between the two columns was 500 mm. The reinforcement details for the two frames are shown in Figure 3.

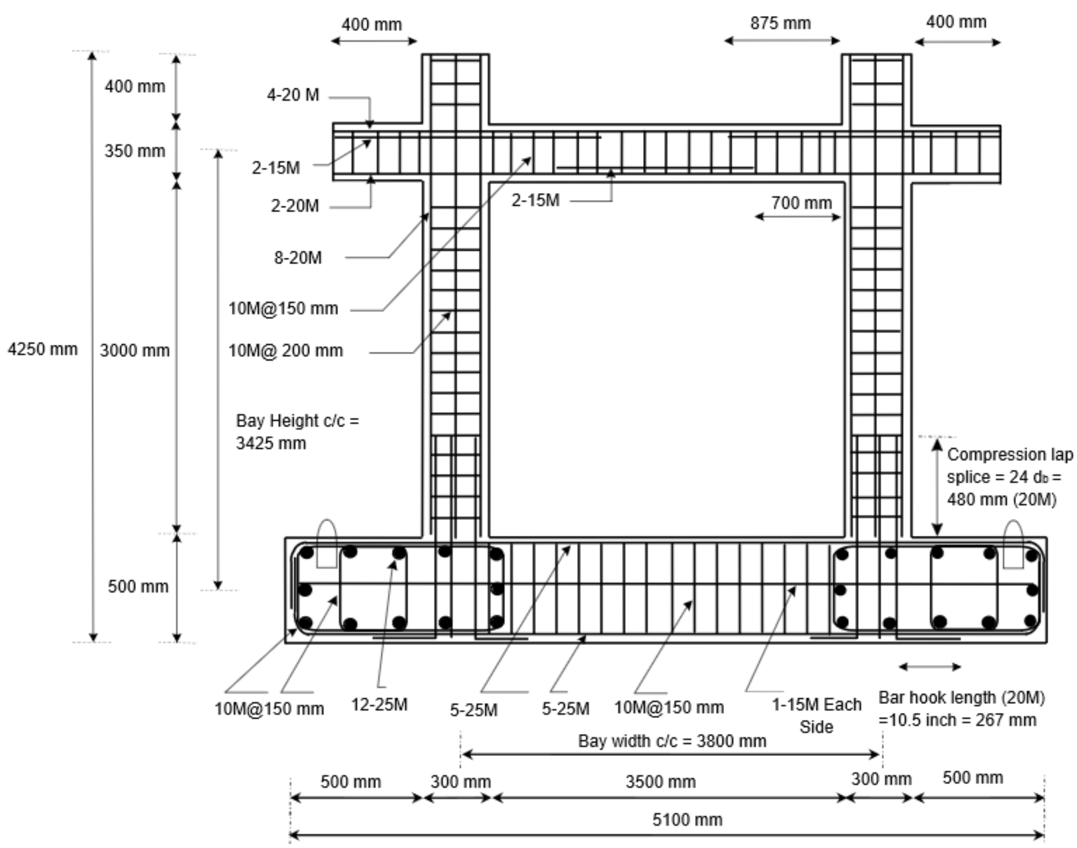


Figure 3: The reinforcement details for the test frames

4.2. Test Procedure and Setup

The frames were secured to the laboratory strong floor by means of four high-strength bolts to provide fixity at the foundation level, as shown in Figure 4. Seven-wire prestressing strands (size 15), were used to impose the gravity loading on the columns and the beam. These loads were established and scaled from the prototype frame building. The test frames were subjected to in-plane reverse cyclic loading by a hydraulic actuator that imposed prescribed incrementally increasing lateral displacements to simulate seismic loading. 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 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 of the frame. All instrumentation was connected to a data acquisition system.

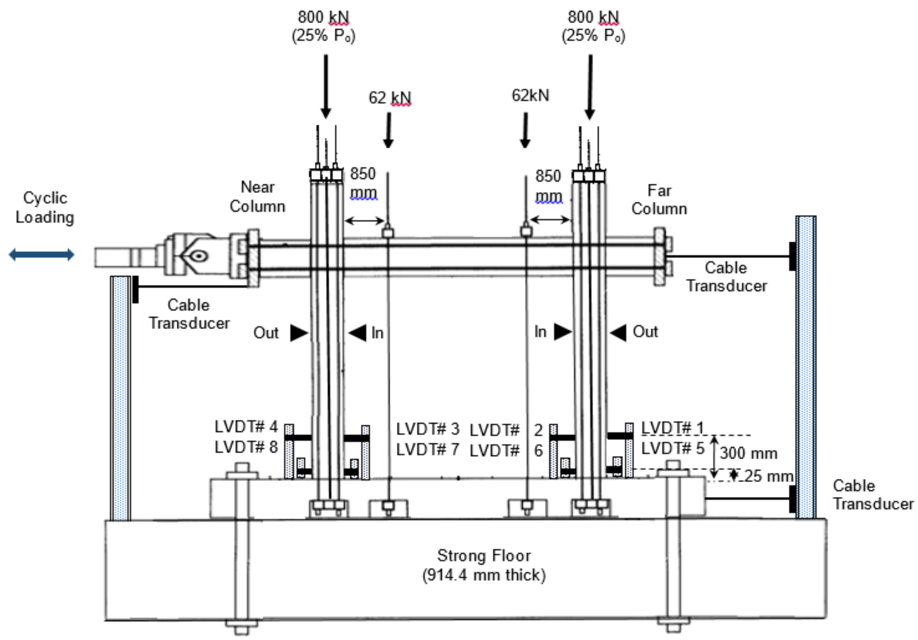


Figure 4: Frame loading setup and instrumentation

The majority of the instrumentation used for testing the retrofitted frame was similar to that used for the control frame, with the exception of additional instrumentation that was added to measure the displacements of the BRB system. These instrumentation consisted of two DCT's that were connected to the hinge steel plates to measure the longitudinal displacements of the brace along the steel core bar near the upper and lower hinge joints. The brace was placed diagonally at an angle of 41º degrees with respect to the foundation level. 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 mid height of the loading beam. Figure 5 illustrates the setup of the frames prior to testing.

5. Test Results

5.1. Hysteretic lateral load-lateral displacement response

Figure 6 illustrates the backbone of the hysteretic response and the full hysteretic response of the lateral load-lateral displacement relationships of the control frame and the retrofitted frame, respectively. The hysteretic backbone curve of the control frame indicated that 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% (79 mm) drift during the first cycle of loading in the push and pull directions, respectively. The frame was considered to have reached its maximum drift capacity of 3% (95 mm) in both push and pull modes, based on a drop in the lateral load capacity exceeding 20% of the maximum lateral resistance beyond this drift level. At this drift ratio, the frame had deteriorated and the longitudinal reinforcing bars in the columns and the beam near the joints were exposed after significant spalling of concrete cover. The corresponding displacement ductility capacity was 2.3.

(a) (b)

Figure 5: Loading setup prior to testing: a) control frame; and b) retrofitted frame

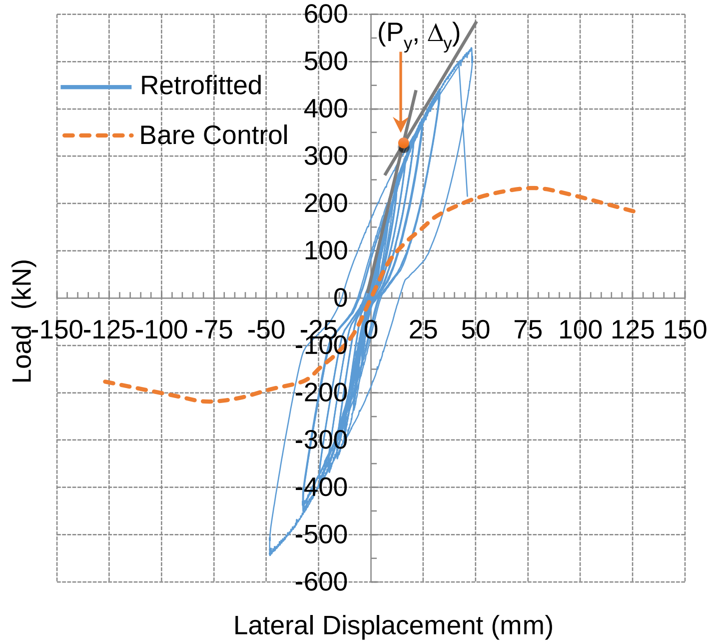


Figure 6: Envelope and hysteretic lateral load-lateral displacement responses for control and retrofitted frames

The experimental 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 data from strain gauges, dictated the softening response of the global behaviour. The hysteretic response illustrates rigid elastic behaviour until yielding of the BRB steel core at a drift ratio of 0.5% (16 mm), and further softening was observed at a drift ratio of 1.2% (38 mm). During the latter deformation level, the yielding of the column longitudinal reinforcement occurred near the base of the far-end column (Figure 4). 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 drift capacities of the retrofitted frame were 1.33% (42 mm) and 1.5% (48 mm) corresponding to lateral load capacities of 510 kN and 543 kN 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. At drift ratio of 1.5%, frame flexural cracks widened and extended along both columns, up to the mid height of the far-end column. No further cracking in the beam was observed during the push and pull cycles. The similar lateral force capacities of the retrofitted frame in both directions of loading provides evidence that the brace system is capable of preventing bucking and promoting yielding in tension and compression. The retrofitted frame attained ductility ratios of 2.7 and 3.0 in the push and pull directions, respectively. This was an increase in ductility of 1.13 and 1.29 relative to the control frame.

5.2. Stiffness

The secant stiffness-lateral displacement envelope response for the control and retrofitted frames are illustrated in Figure 7. The stiffness was calculated by dividing lateral strength by the corresponding lateral displacement at each drift level, and for the first push cycles only, for clarity. The initial stiffness was calculated at a lateral displacement of 4 mm, which was the displacement imposed on the retrofitted frame during the first load cycle. The control frame experienced an initial stiffness of 11.7 kN/mm; while the retrofitted frame experienced an initial stiffness of 20.8 kN/mm, an increase of 1.8 relative to the control frame. The control frame experienced a secant stiffness of 4.9 kN/mm at the yield displacement of 41 mm; while the retrofitted frame experienced a secant stiffness of 18.8 kN/mm at the yield displacement of 16 mm. The retrofitted frame provided increased secant stiffness at yield, relative to that of the control frame by a factor of 3.8. At the yield displacement of the control frame, the retrofitted frame experienced a secant stiffness of 12.2 kN/mm, an increase of 2.5 relative to the control frame.

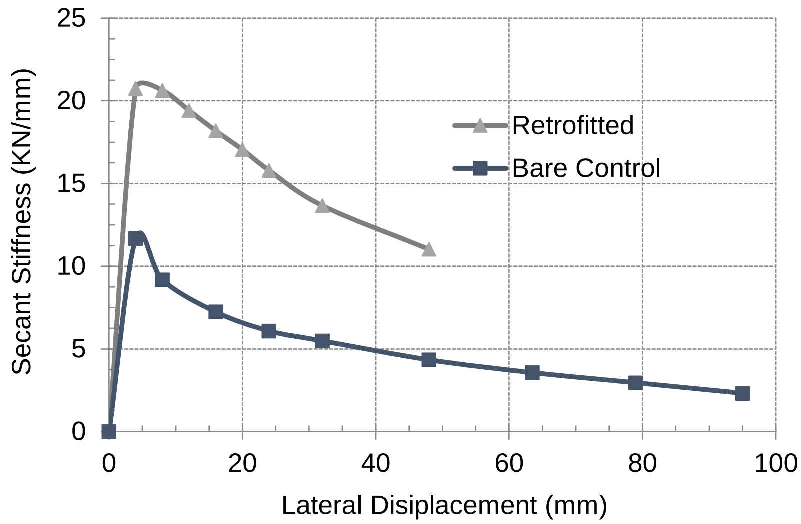


Figure 7: Secant stiffness-lateral displacement envelope responses for control and retrofitted frames

5.3. Energy Dissipation

Energy dissipation is a measure of ductility during plastic response. Figure 8 provides the strain energy-lateral displacement envelope response experienced by the control and retrofitted frames. The strain energy was calculated from the area of the first hysteresis loop at each deformation level. The retrofitted frame experienced an increase in energy dissipation relative to the control frame. The retrofitted frame dissipated 26027 kN.mm of energy at a lateral displacement of 48 mm (1.5% drift ratio), which corresponded to an increase of 6.6 relative to the control frame (3937 kN.mm) at the same lateral displacement.

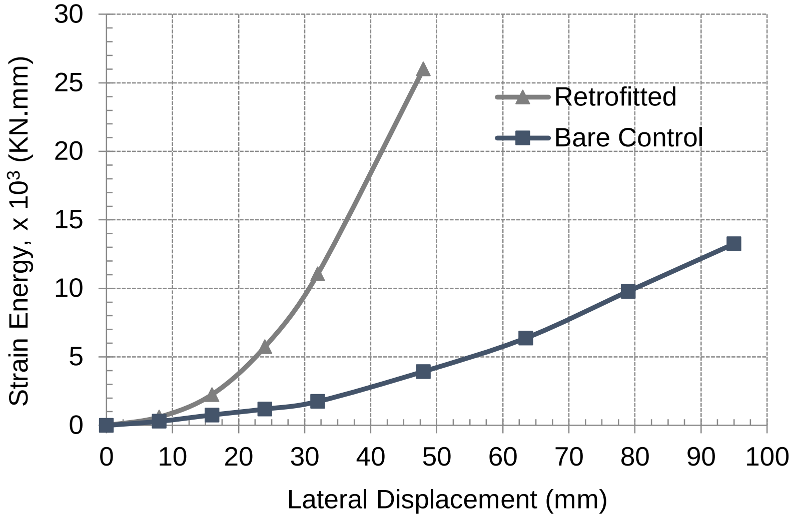


Figure 8: Energy dissipation-lateral displacement responses for control and retrofitted frames

5.4. Moment-Rotation Response

The moment-rotation response is a salient characteristic of reinforced concrete members during earthquakes. The flexural rotations within the plastic hinge region of the far-end column of the control and retrofitted frames were calculated from the data recorded by LVDT # 1 and 2 (Figure 4) and further illustrated in Figure 9. The base moments of the retrofitted frame were assumed similar to the base moments calculated for the control frame. The latter reasonably represents the lateral strength component of the concrete frame (excluding the BRB steel core contribution) of the retrofitted frame. Flexural rotations of the retrofitted frame were calculated up to a lateral displacements of 48 mm (1.5% drift ratio); the displacement at which the peak lateral displacement was experienced. Both frames exhibited similar responses. However, due to slack in the BRB system during testing of the retrofitted frame in the range of lateral displacements of 10-20 mm, which was recorded by strain gauges located on the BRB steel bar, the concrete frame contributed a larger share of the lateral strength component. As a result, slightly larger flexural rotations were experienced for the same base moments relative to the control frame.

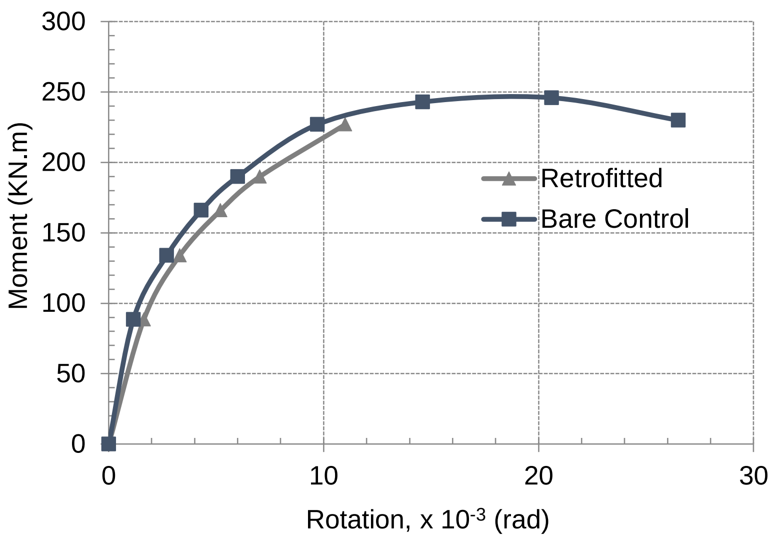


Figure 9: Moment-rotation responses for control and retrofitted frames

6. Conclusions

An experimental investigation was conducted on two identical 2/3rd scale frames that were representative of a 6-story reinforced concrete frame building located in Vancouver, Canada. The frames were designed and built based on the 1965 edition of the NBCC. The first frame served as a control frame while the second frame was retrofitted. A novel buckling restrained brace was conceived and developed as part of this study and was used as a means of retrofitting. Relative to the control frame, the lateral strength capacity in the retrofitted frame increased by factors of 2.2 and 2.5 in the push and pull modes of loading, respectively.

Furthermore, the ductility of the retrofitted frame increased by factors of 1.13 and 1.29 in the push and pull modes, respectively, in comparison to the control frame. The stiffness capacity was improved compared to the control frame. The retrofitted frame provided an increase in initial stiffness by a factor of 1.8 at a drift ratio of 0.125% and an increase in secant stiffness by a factor of 2.5 at the yield displacement of the control frame (drift ratio of 1.3%). The energy dissipation was also increased by a factor of 6.6 relative to the control frame at a drift ratio of 1.5%.The response of the retrofitted frame, which was very similar in the push and pull modes of loading, highlights that the proposed bracing system provided similar resistance in tension and compression. This demonstrates that the bracing system was effective at preventing buckling.

7. Acknowledgements

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8. References

Clark, P., Aiken, I., Kasai, K., Ko, E., and Kimura, I. (1999). Design procedures for buildings incorporating hysteretic damping devices. Proceedings of the 68th Annual Convention, pp. 355-371, Structural Engineers Association of California, Sacramento.

Cement Association of Canada, 2006. Concrete Design Handbook, 3rd edition, Ottawa, Ontario, Canada.

F. Dinu, S. Bordea, and D. Dubina (2011). Strengthening of non-seismic reinforced concrete frames of buckling restrained steel braces, behaviour of steel structures in seismic areas. 1085 -1090.

Maheri, M. R. and Sahebi, A. (1997). Use of steel bracing in reinforced concrete frames. Science, 19 (12), 1018-1024.

National Building Code of Canada. 1965. Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, Ontario, Canada.

National Building Code of Canada. 2010. Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, Ontario, Canada.

Watanabe A., Hitomi Y., Saeki E., Wada A., Fujimoto M. (1988). Properties of brace encased in buckling-restraining concrete and steel tube. Proc. of Ninth World Conf. on Earthquake Eng. Vol. IV. 1988. p. 719-24.