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REPAIR AND RETROFIT OF NON-DUCTILE REINFORCED CONCRETE FRAMES

F. Caron[1] and D. Palermo[2]

ABSTRACT

Reinforced concrete frames built before the 1970s were designed prior to the enactment of seismic codes, and thus, do not satisfy requirements of the capacity design philosophy. These frames possess a number of structural deficiencies, including inadequate shear capacity, poorly reinforced joints, deficient lap splice lengths, and insufficient buckling and confinement reinforcement. These frames are characterised by inadequate ductility, which is one of the key performance indicators in seismic design. Methods to retrofit a reinforced concrete frame include the addition of new structural elements such as infill walls, while others attempt to strengthen the structural elements: joints, beams, and columns. This can be achieved by using fibre reinforced polymers (FRPs) or steel jackets. The focus has been to increase the stiffness of the structure, thus increasing the lateral load carrying capacity of the frames, while reducing the inelastic ductility demands. Traditional retrofitting techniques are invasive and require significant detailing, particularly where the tension capacity of the retrofitting material is the main contribution to the lateral resistance. The objective of this research is to assess diagonal X-bracing as a compression strut to retrofit non-ductile reinforced concrete frames in an attempt to increase stiffness and lateral load capacity. The advantage of this retrofit methodology is the elimination of costly and detailed connections between the X-bracing and the concrete frame typically found in X-bracing that rely on tension capacity. Two, single storey, one-bay non-ductile reinforced concrete frames previously damaged were first repaired to their original condition, which included removing damaged concrete, and replacing buckled reinforcing bars in the columns. One of the frames will be tested to assess the seismic performance of the repair methodology. The companion frame will be retrofitted with hollow structural steel diagonal X-bracing. The strategy focuses on the compression capacity of the X-brace to increase stiffness and lateral load carrying capacity, while reducing inelastic drift demands. The repaired and retrofitted frames were assessed using the nonlinear finite element method. The retrofit demonstrated an enhancement in stiffness and strength relative to the repaired frame, illustrating the potential of compression struts with simple connections as an alternative retrofit methodology for non-ductile concrete frames.

[1]Graduate Student, Dept. of Civil Engineering, University of Ottawa, Ottawa, ON
[2] Assistant Professor, Dept. of Civil Engineering, University of Ottawa, Ottawa, ON

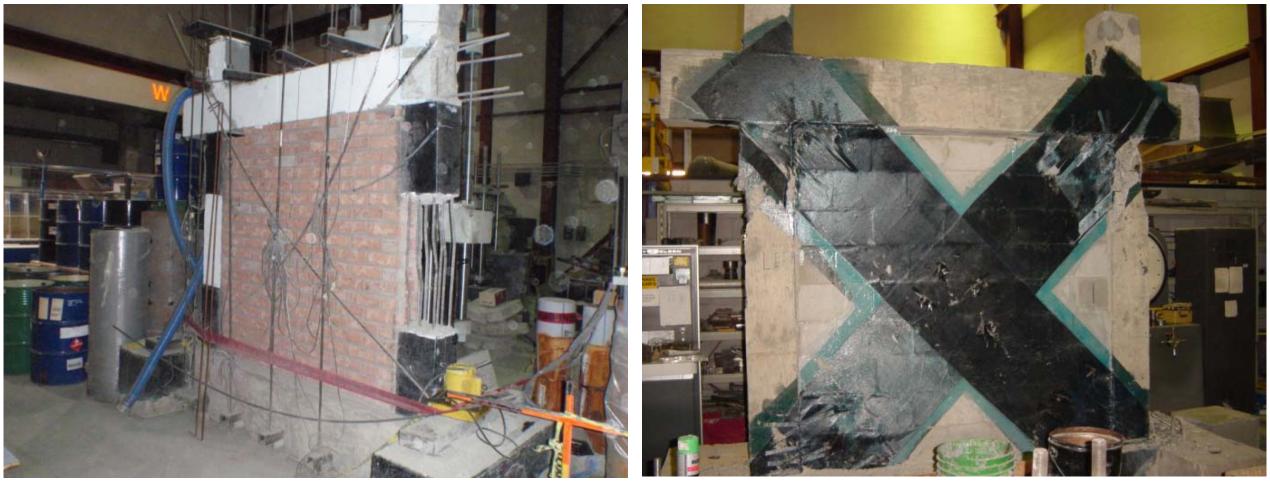
Introduction

Earthquake design has progressed to the stage where new or retrofitted concrete structures can tolerate the design earthquake event without collapse, but experience damage in pre-selected locations. Complete replacement of structures that do not conform to modern seismic design codes is not economically feasible or viable. Therefore, retrofit methods have developed and others are emerging to reinforce non-ductile frames and reduce damage that could be caused by future earthquakes. Significant effort has been geared toward retrofit of reinforced concrete structures with fibre reinforced polymers (FRPs). Serrato (2002) experimentally investigated non-ductile reinforced concrete frames with masonry infill walls. FRP sheets were applied diagonally over the infill wall in two layers. The results demonstrated an increase in strength and stiffness but not ductility. Shalouf (2005) studied non-ductile reinforced concrete frames in-filled with masonry and retrofitted with diagonal prestressed cables or diagonal FRP strips. Each methodology was capable of increasing the strength and stiffness but not ductility. Duong et al. (2007) investigated the seismic behaviour of a single-bay, two-storey reinforced concrete frame with shear-critical beams. After testing, the severely shear damaged beams were repaired with CFRP and then retested. The failure mechanism changed from brittle to ductile after being repaired. Less research has focused on adding steel bracing to reinforced concrete frames. Maheri and Sahebi (1997) conducted an experimental study using steel bracing to reinforce concrete frames. The bracing systems included diagonal tension bracing, diagonal compression bracing, and a combination of compression and tension bracing. The test results indicated a significant increase in the lateral load carrying capacity of the frame when either a tension or compression brace was connected to the frame. Substantially higher load carrying capacity was recorded when the frame was retrofitted with an X-brace capable of resisting tension and compression loading. The authors also noted the importance of proper connection details to ensure the braces utilize their full capacity.

Research has generally focused on the tension capacity of retrofit materials including FRP sheets, prestressing cables, and diagonal steel braces to increase the strength and stiffness of reinforced concrete frames. The connection details pose a significant challenge. Furthermore, the connections can be invasive, often requiring punching through the floor slab for proper anchorage. An alternative approach is to use diagonal compression braces, which require minimal connection details and can be implemented and removed with little effort. Therefore, compression braces can also act as structural fuses and be easily replaced after an earthquake. The objective for this research is to evaluate the response of non-ductile reinforced concrete frames retrofitted with diagonal X-bracing compression struts. The main focus is to develop a quick, easy and inexpensive retrofit method that can be used before or after the structure is damaged. The advantage of this retrofit methodology is the elimination of costly and detailed connections between the X-bracing and the frame. In addition, the compression strut can easily be removed after an earthquake and replaced with a new strut, thus, acting as a structural fuse.

Experimental Program

The repaired and retrofitted frames of this study were previously tested by Shalouf (2005). Specimen BR-3 contained an infill masonry brick wall, and was retrofitted with FRP wrapping of the columns at the top and bottom to prevent failure at the column ends. In addition, diagonal prestressing cables were used. Specimen BL-3 included an infill masonry block wall with diagonal FRP sheets adhered to the block wall and joints. The original frames were part of an investigation focused on retrofitting non-ductile reinforced concrete frames with diagonal elements. The objective was to assess the increase in lateral load carrying capacity and stiffness provided by the retrofit methods. Fig. 1 illustrates the state of the frames at the end of the original tests.



(a) (b)

Figure 1. Condition of frames prior repair and retrofitting: a) BR-3; b) BL-3.

Specimen BR-3 was repaired and renamed BR-3R for this research study. Specimen BL-3 was also repaired and will be retrofitted with hollow structural steel X-bracing compression struts. The frame was renamed BL-3R. Frames BR-3 and BL-3 were originally designed according to ACI 318-1963, representing the design of reinforced concrete frames prior to the enactment of seismic design. Details of the frames are shown in Fig. 2.

The frames were built on a stiff I-shape foundation. The foundation was 500 mm deep, 1320 mm wide at each end and 480 mm wide between the columns, and 3270 mm long. The columns were square with dimension of 250 mm. The beam was 250 mm wide and 350 mm deep. The clear spans of the columns and beam were 1825 mm. The ends of the beam and columns were extended by 400 mm to anchor the reinforcement. The cover was 25 mm in the beam and columns. The transverse shear reinforcement for both the beam and columns consisted of 6.35 mm diameter closed stirrups spaced at 125 mm. There were no stirrups in the joints. At the interior face of the column, the longitudinal reinforcement in the beam included 3-15M bars at the top and 2-15M bars at the bottom. The middle 15M bar at the top of the beam was cut off at a distance of 750 mm from the face of the column. The column contained of 8-15M bars. The lap splice for the longitudinal reinforcement at the base of the column was 390 mm.

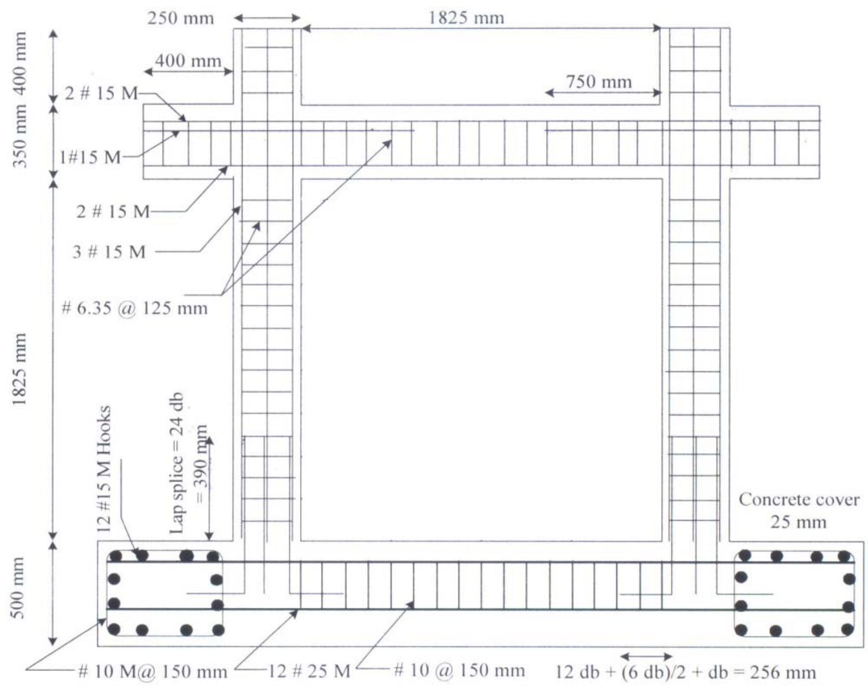


Figure 2. Details of frames (Shalouf 2005).

The material properties reported by Shalouf (2005) included concrete compressive strengths of 33.2 MPa and 32.1 MPa, respectively for BR-3 and BL-3. The yield strengths were approximately 495 MPa and 500 MPa for the 15M and 6.35 mm diameter smooth bar, respectively.

Repair of Reinforced Concrete Frames

For Frame BL-3, the diagonal FRP sheets and masonry block wall were first removed. The damage was concentrated at the top of the columns adjacent the joints. The concrete in this area was removed followed by the stirrups. The longitudinal reinforcement in the left column was significantly buckled. The buckled portions of the reinforcing bars were cut and removed, and replaced with new pieces of reinforcement, which were welded to the ends of the bars that were cut. The longitudinal reinforcement in the right column experienced some degree of buckling; however, removal of these bars was not necessary. New pieces of reinforcement were welded against the buckled bars. The stirrups were replaced prior to forming the joint and pouring new concrete. The final step included injecting mortar between the column and the joint to fill voids remaining after removal of the formwork.

For Frame BR-3, damage was concentrated in the right column at mid height. A similar repair procedure was followed. The prestressing cables were removed followed by the masonry brick wall. The buckled longitudinal reinforcement in the column was cut and removed. New reinforcement was welded to the ends of the cut reinforcement followed by the addition of new stirrups. Formwork was built around the column and new concrete was poured. In addition, during the test setup, while the gravity loads were being applied on the beam, significant shear cracking developed. This necessitated full replacement of the concrete and stirrups in the beam. All repair materials for both BR-3R and BL-3R had similar mechanical properties to the original materials. Fig. 3 is a photo of BR-3R after repair and during the test setup.

Figure 3. Repaired specimen BR-3R.

Retrofit of Reinforced Concrete Frames

Frame BL-3 was first repaired as previously discussed. Currently, the frame is further being retrofitted. The retrofitting includes square hollow structural steel X-bracing designed to respond as compression struts. The objective is to devise a strategy that can easily be implemented, while increasing the strength and stiffness of a non-ductile reinforced concrete frame. Such structures do not possess the necessary ductility to resist major earthquakes, thus, it is necessary to add stiffness to the system. A ductility retrofit technique may require upgrading to the beams and columns. One possible method would be to use FRP to wrap the beams to increase the shear resistance, wrap the columns to increase shear and ductility, and add longitudinal FRP sheets to increase flexural resistance where necessary. Such a procedure would decommission a structure for a significant amount of time, and furthermore, not be economical. The use of HSS compression struts can be implemented with relative ease, since limited connection details are required. In addition, numerical analyses can be used to determine the most effective position of the diagonal bracing in a structure to control drift. The compression strut offers advantages over tension braces, such as the elimination of detailed tension connections to the surrounding concrete frame. Furthermore, a compression strut can act as a structural fuse and easily be replaced after an earthquake. Conversely, a diagonal tension element would require significant labour to disconnect the member from the frame. Fig. 4 provides a drawing of Frame BL-3R retrofitted with diagonal compression struts. The enhancements in strength and stiffness provided by compression strut diagonals in non-ductile reinforced concrete frames are discussed in the following section with the support of nonlinear finite element analysis.

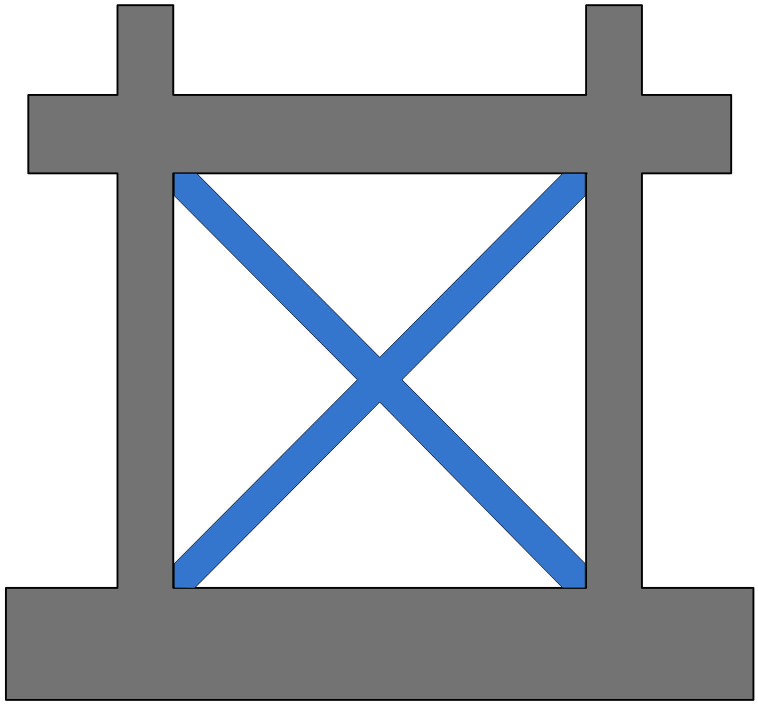
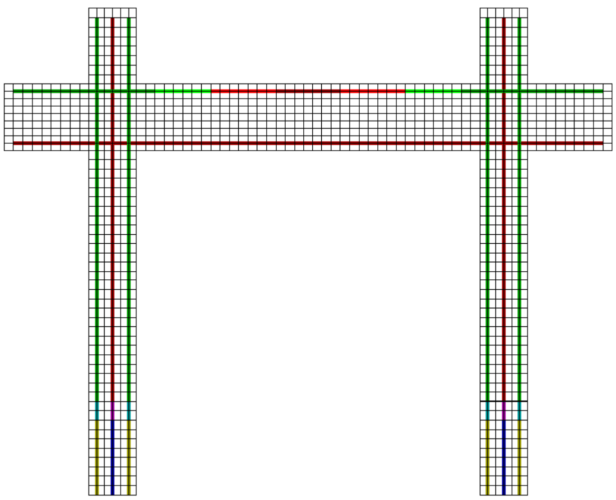


Figure 4. Retrofitted specimen BL-3R.

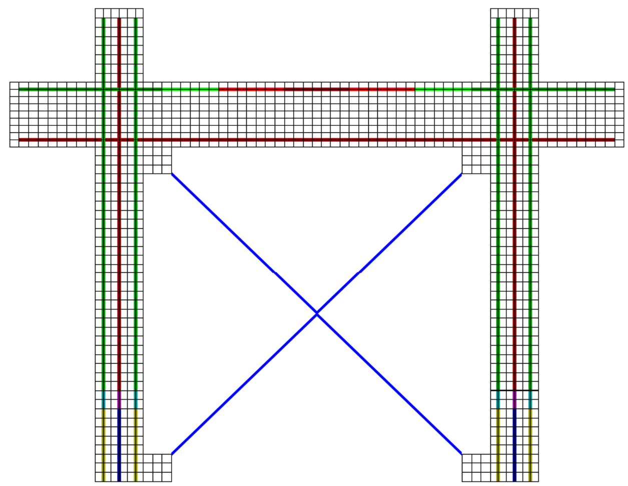
Finite Element Analysis of Repaired and Retrofitted Frames

Numerical analyses were conducted using Program VecTor2 (Wong and Vecchio 2002), a nonlinear two-dimensional finite element program applicable for membrane structures. VecTor2 uses a smeared, rotating-crack formulation based on the Modified Compression Field Theory (1986) and the Disturbed Stress Field Model (2000). The program algorithm is based on a secant stiffness formulation using a total-load iterative procedure. The program can provide the following structural performance indicators: strength, ductility, post-peak behaviour, failure mode, deflections and cracking. The concrete is modeled with low-powered elements: 4-node rectangular or quadrilateral elements, and 3-node triangular elements. The reinforcement can be modelled discretely with a 2-node truss bar element or smeared within the concrete elements. In addition, bond slip is modelled with either a 2-node link element or a 4-node contact element.

Figs. 5 a) and b) show the finite element models of Frame BR-3R (repaired) and Frame BL-3R (repaired and retrofitted), respectively, developed for Program VecTor2. A total of 1149 rectangular elements were used to model the concrete frame. All longitudinal reinforcement in the columns and beam were discretely modeled with truss bar elements. The transverse shear reinforcement in the columns and beam were smeared within the rectangular concrete elements. Development lengths in the beam and splice lengths in the columns were modeled by reducing the strength capacity of the reinforcement in those areas. For the retrofitted frame, two additional truss elements were used to represent the X-bracing. The truss bars were specified as compression only reinforcement. In addition, stiff rectangular elements were included at the ends of the diagonal truss elements to prevent localized crushing of the concrete in the frame structure. Each column was loaded with 400 kN of gravity loading, spread across the top of the columns. The lateral loading was imposed at the end of the beam in a displacement controlled mode. Three repetitions were applied at each displacement level. The displacement was incremented by 2 mm until 10 mm of lateral displacement; thereafter, the displacements were incremented by 5 mm until failure. Note that the frames were modelled as fully repaired with no residual damage. The objective of the analyses is to demonstrate the response of non-ductile reinforced concrete frames and the enhancements in strength and stiffness offered by diagonal compression braces. The residual damage was similar in both frames, thus, a similar relative increase in strength and stiffness would be expected if the residual damage was accounted for in the model.



(a)



(b)

Figure 5. Numerical model of frames: a) repaired; b) retrofitted.

The concrete and reinforcement properties assumed in the analyses were those reported for the original frames: BR-3 and BL-3. The buckling capacity of the HSS diagonal compression braces were considered in the numerical model. The capacity of the braces was reduced from the full yield capacity as governed by buckling. These values were determined according to CAN/CSA-S16.1-04 Limit States Design of Steel Structures (CISC 2004). The default material models suggested by VecTor2 were selected for all analyses. Fig. 6 a) provides the predicted hysteretic response of the bare (B3-3R) and retrofitted (BL-3R) frames. Frame BL-3R was retrofitted with an HSS 102 x 102 x 8 mm X-brace. For comparison purposes, Fig. 6 b) illustrates the predicted monotonic behaviour of the bare frame and the frame retrofitted with various HSS brace sizes. The frame retrofitted with HSS 102 x 102 x 8* provides the response if buckling is prevented and the compression X-brace yields. Table 1 lists the properties of the HSS sections including the cross sectional area, and the nominal yield capacity and the approximate buckling capacity for Grade 350 steel. Note that the buckling capacity was based on a conservative diagonal length of 2750 mm to account for possible reductions in brace strength due to the rotation of the joint during lateral loading and misalignment of the brace during erection.

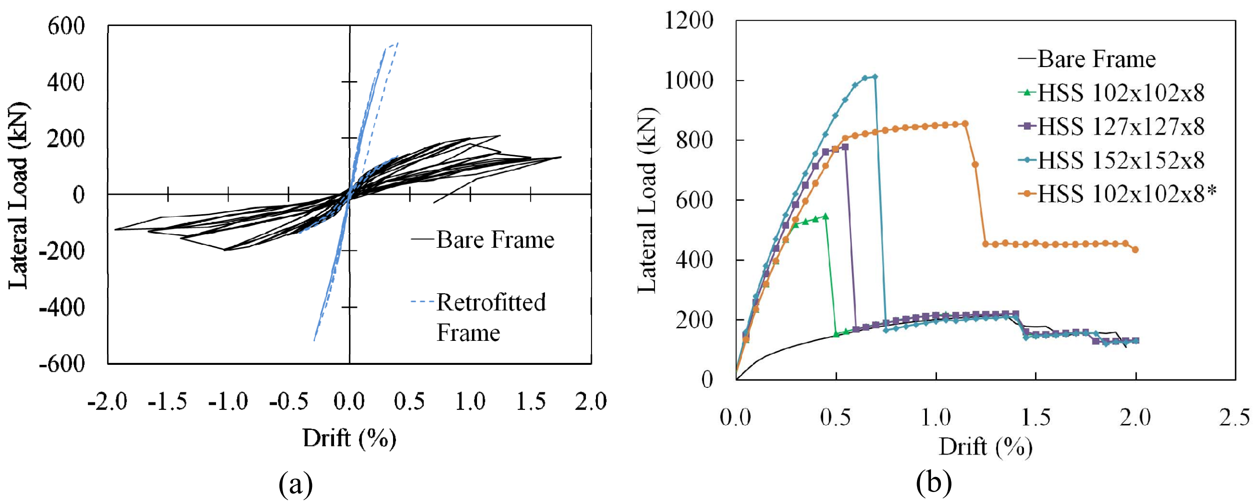


Figure 6. Numerical results: a) hysteretic response; b) monotonic response.

Table 1. Properties of X-braces.

HSS Section (mm)	Area (mm^2)	Yield Capacity (kN)	Buckling Capacity (kN)
102 x 102 x 8	2820	987	603
127 x 127 x 8	3620	1267	943
152 x 152 x 8	4430	1550	1289

The hysteretic response of the bare frame demonstrated limited ductility. Significant strength degradation was predicted at approximate drifts of 1.25 % and 1.0 %, in the positive and negative directions respectively. This was caused by the insufficient lap splices at the base of the columns. Furthermore, the pinched hysteretic response is also characteristic of non-ductile behaviour. The maximum lateral strength capacity was approximately 210 kN at approximately 1.25 % drift. The retrofitted frame illustrated a significant increase in strength and stiffness. The maximum lateral load capacity was recorded at approximately 0.4 % drift in the positive direction, corresponding to 537 kN and representing an increase of 2.57 relative to the bare frame. The X-brace buckled at the peak load and thereafter the response was identical to the response of the bare frame. The frame ultimately failed due to inadequate splice lengths at the base of the columns. Fig. 6 b) provides a comparison of lateral strength and drift capacities of various sizes of HSS sections. The predicted responses indicate an increase in strength and drift capacities as the size of the X-brace increased. Based on the increase in seismic forces prescribed by current seismic codes relative to those suggested at the time the frame was constructed, an appropriate size for an X-brace can be obtained.

Table 2 provides the capacities for each retrofit. An additional analysis was conducted with HSS 102 x 102 x 8 where the X-brace was permitted to yield (HSS 102 x 102 x 8*). It is apparent that a yielding member provides an increase in strength and drift capacities if buckling is prevented. Failure is also governed by the inadequate splice lengths at the base of the columns.

Table 2. Predicted capacities of retrofitted frame.

Retrofit	Strength Capacity (kN)	Drift Capacity (%)
Bare Frame	216	1.35
102 x 102 x 8	547	0.45
127 x 127 x 8	770	0.5
152 x 152 x 8	1012	0.7
102 x 102 x 8 (Yielding)	885	1.15

The finite element analyses indicated that compression X-braces provide an increase in strength and stiffness for non-ductile reinforced concrete frames. The buckling capacity for the brace governed the peak lateral strength capacity of the system for the sizes of braces analyzed. However, the deficient lap splice length will govern the peak lateral load when the buckling capacity becomes large. If the X-braces are designed to yield, enhancements in strength and ductility are possible. The main advantage of this system is the elimination of costly and invasive connection details required for tension retrofitting members including braces.

Conclusions

This paper has presented an alternative seismic retrofit strategy for non-ductile reinforced concrete frames built prior to the enactment of seismic design codes. The methodology uses hollow structural steel sections as diagonal compression struts. The struts can easily be installed or removed after an earthquake, thus acting as a structural fuse. The main advantage of this system is the elimination of costly and labour intensive detailing connections commonly required for tension members. Nonlinear finite element analyses were presented to demonstrate the enhanced strength and stiffness provided by compression braces. In addition, an experimental program is currently ongoing, which will test the new strategy and assess the seismic performance of non-ductile reinforced concrete frames retrofitted with compression only X-bracing.

Acknowledgements

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