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NONLINEAR TIME-HISTORY ANALYSIS OF RC FRAMES RETROFITTED WITH BRB

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Abstract

Reinforced concrete buildings built prior to the enactment of modern seismic provisions are vulnerable to high-magnitude earthquakes. These buildings may present deficiencies related to ductility and/or strength, with insufficient energy dissipation capacity to mitigate the effects associated with strong ground excitations. This research examines the advantages of Buckling-Restrained Braces (BRBs) as a retrofit technique for seismically deficient reinforced concrete frames, the dominant structural system of pre-1970s concrete construction. The efficiency of Superelastic-Shape Memory Alloy (SE-SMA) bars as the core material of the BRB system is also evaluated. Improvements in lateral capacity, and residual displacements of the SMA-BRB retrofitted frames are assessed against previously tested bare concrete frame and a frame retrofitted with BRBs incorporating stainless steel core bar. Finite-element modelling of the frames was conducted with Program VecTor2, and the numerical models were validated through experimental results available in the literature. The finite element results satisfactorily simulated the experimental responses illustrating the effectiveness of buckling restrained braces in reinforced concrete frames. The validated numerical models were further modified to address deficiencies in the original BRB design implemented in the testing program. The modifications included improvements in restraining of the buckling, which for the frame retrofitted with the modified BRB incorporating conventional steel core resulted in higher strength and ductility capacities and greater residual displacements in comparison to the bare frame and previously tested braced frames. The numerical modelling also considered a BRB with an SMA core bar that included improvements in the restraining in buckling. The SMA-BRB provided lateral strength comparable to the modified steel BRB models with enhancements in ductility capacity. Notably, the SMA-BRB system did not accumulate any additional residual displacements beyond those experienced by the bare frame alone, and failure of the BRB systems was controlled by fracturing of the core bar. Furthermore, nonlinear time-history analyses were conducted. Earthquake records that simulate moderate and high seismic regions in Canada were selected from available databases. In the time-history analyses, the BRB frame incorporating an SMA core exhibited improved seismic performance relative to the bare concrete frame, including a decrease in the peak lateral drift, a reduction of cracking in the beam-column joints, and insignificant accumulation of permanent displacements. The reduction in residual displacements illustrates one of the main benefits of SMA bars in comparison to conventional steel core BRBs, leading to a viable and efficient alternative to increase the seismic performance and reduce damage in structural elements located in high seismic zones.

Keywords: Reinforced concrete frames, buckling-restrained braces, superelastic shape-memory alloys

1. Introduction

Significant changes have been implemented in seismic provisions in the last decades due to advancements in seismological knowledge. Existing buildings designed and constructed prior to the enactment of these seismic provisions, 1970 decade, lack sufficient reinforcement detailing to ensure life safety of the occupants. Pre-1970s structures, therefore, may require retrofitting to attend adequate seismic performance levels, mitigate potential structural damage and ensure life safety of residents. Given that non-ductile or limited ductility reinforced concrete moment-resisting frames represent the dominant structural system of pre-1970s structures, these buildings are highly susceptible to failure under seismic events. That is because the frames were designed based on the strong beam-weak column theory that underestimates the required transverse reinforcement resulting in widely spaced and poorly anchored stirrups. Thus, under a seismic event, beam-column joints and the base of the columns are susceptible to shear failure that may lead to reduction in axial load resistance of the column and, subsequently, increasing the collapse potential.

Seismic retrofit is utilized to assure life safety by improving the performance of structural systems to satisfy current seismic provisions. However, improving the seismic performance is not limited to a strength capacity increase. The challenge when retrofitting a structure resides in selecting a technique capable of mitigating the damage inflicted by an earthquake to the serviceability state of a structure. Seismic bracing is a viable technique to increase lateral strength and stiffness of RC frames. Buckling Restrained Braces (BRBs) are a form of steel bracing that restrains the effects of buckling on the brace allowing it to yield under both tension and compression. Conventional steel core BRBs, however, induce a decrease in ductility and increase in residual deformations of the frames due to yielding of the core; often failing due to localized buckling of the core in internal and external reverse gaps or due to concentration of stresses inside the brace. To reduce recovery time and repair costs associated with permanent deformations, this paper aims to evaluate the seismic response of reinforced concrete moment-resisting frames retrofitted with a BRB incorporating a Superelastic Shape-Memory Alloy (SE-SMA) core bar.

SMAs are a class of material capable of recovering deformations due to superelastic (SE) and shape memory properties. The former recovers deformations once the applied stresses are released at ambient temperature. The latter, on the other hand, recovers deformation due to an increase of temperature that allows the polycrystalline structure of the material to transform from its martensite to its austenite phase (DesRoches et al., 2004). As SMAs can recover its initial shape, its application in a BRB can potentially address deficiencies of traditional braces. Traditional braces can improve lateral strength and ductility of frames, but the induced residual deformation can induce brittle failure of the structure and require repair/demolition of the element after a seismic event. Retrofitting a RC frame with a BRB incorporating an SMA core improves the re-centering capability of the structure reducing residual deformations and overall damage, while improving strength, ductility and allowing the structure to dissipate energy. Zhu and Zhang (2007) implemented SMA wires as a secondary energy dissipating component of the brace. The frames exhibited self-centering hysteresis and excellent fatigue properties to the brace. This paper, however, intends to apply SMAs not as secondary components, but as the core of the BRB; characteristics of the SMA would, therefore, control the behavior of the retrofitted frame.

The objective of this research is to evaluate the performance of a pre-1970s seismically deficient reinforced concrete frame designed in accordance with the 1965 edition of National Building Code of Canada. This paper also intends to evaluate the seismic response improvement of the prototype frame once it is retrofitted with a BRB. To minimize permanent deformations and damage on the RC frame generated from conventional steel core BRBs, this paper addresses the application of a self-centering material (SE-SMA) as the bracing core bar. The seismic performance of the frame and deficiencies of common BRB designs are assessed through the finite element method program VecTor2. Modifications implemented in conventional BRBs are presented to improve the restraining of the core bar and are assessed in the finite element model. As a result, the main objective of this research is to retrofit seismically deficient buildings to meet current seismic code requirements based on seismic demands without inducing an increase in permanent deformations nor a decrease in ductility beyond what is experienced by the bare concrete frame.

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2. Prototype structure and retrofit methodology

The prototype frame was scaled from a six-storey reinforced concrete moment-resisting frame structure that represents a typical seismically deficient building built based on the 1965 NBCC. The building was adopted from the Concrete Design Handbook (Cement Association of Canada, 2006). Figure 1 illustrates the two-third scaled frame used in this study. The reinforcement details depict the widely spaced stirrups at the base of the columns and near the beam-column joints. In the columns, the stirrups were spaced at 200mm, while in the beams they were spaced at 150mm. The compressive strength of the concrete was also selected to portray characteristic of pre-1970s construction materials, it was then defined as 31MPa. The prototype frame has been previously tested elsewhere (Al-Sadoon, 2016); the experimental response was used to validate the finite element model.

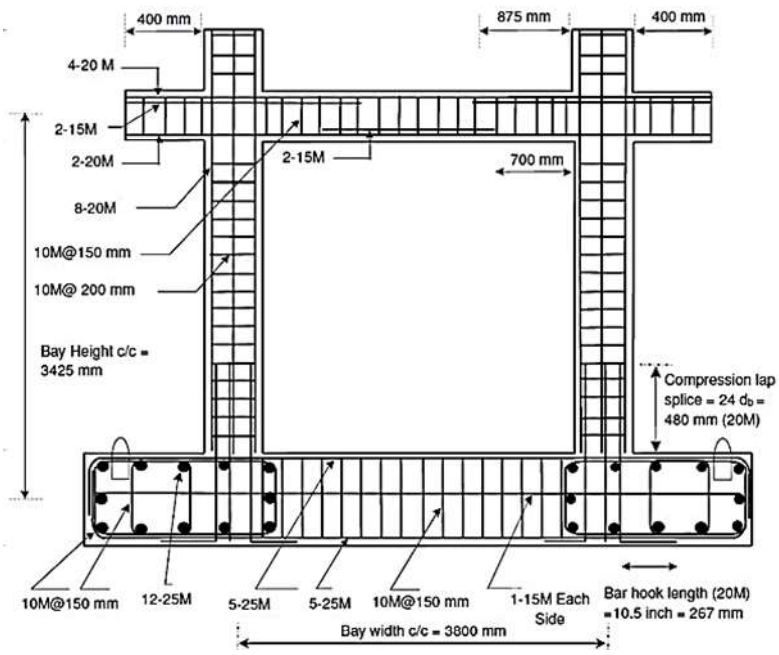

Figure 1: Reinforced concrete frame details (Al-Sadoon, 2016)

The retrofit methodology consists of a diagonal BRB that utilizes the full compressive and tensile capacity of the core bar. Figure 2 illustrates the BRB, which was proposed and designed by Al-Sadoon (2016). This bracing system was designed to eliminate internal and external reverse gaps along the brace by restraining deformations in the transverse direction at the edges of the core bar. The BRB is composed of a circular rigid steel element that prevents the core from buckling over its entire length. The total length of the streel core bar is 2700mm with a diameter of 44.5mm. To promote yielding away from the threaded ends of the core, the bar diameter is reduced to 31.8mm near the mid-length. Four 12.7mm bar spacers are placed along the core bar to accommodate the strain gauges and wiring and to prevent buckling of the core bar. An epoxy-sand mixture is utilized to fill the area between spacers and core bar. The core bar is then encased into a 59mm-diameter steel pipe surrounded by mortar (Sikacrete-087), which, in turn, is encased in a 168mmx8mm HSS. This BRB design has been proven to annulate local buckling at the edges of the core bar, allowing the core to experience larger deformations. In this paper, an AISI Type 304 Stainless Steel bar was also employed as the BRB core in order to validate the FE model of the retrofitted frame. In addition to the conventional steel core bar, a shape memory alloy bar was utilized as to improve the seismic performance of the RC frame.

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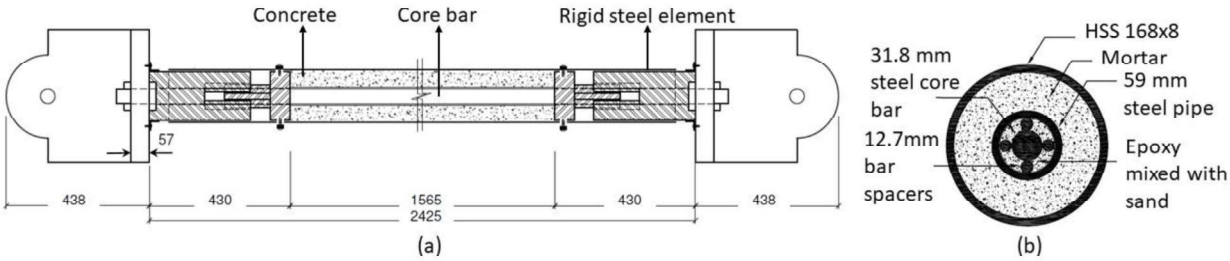

Figure 2: BRB brace: (a) Side view; and (b) cross-section of BRB (Al-Sadoon, 2016)

According to DesRoches et al. (2004), Nickel-Titanium (NiTi) superelastic SMAs are an attractive alternative for seismic retrofitting of reinforced concrete structures due to its excellent strain capacity, hysteretic damping and deformation recovery. In addition, NiTi-SMAs exhibit low temperature sensitivity (Melton 1990), which allows the material to recover deformations at ambient temperatures. Figure 3 illustrates the stress-strain relationship of a NiTi SE-SMA, where the recoverable capacity is approximately 6%. Therefore, when compared to conventional steel cores, the SE-SMA is expected to restrict the residual deformation to what the bare concrete frame would experience. Table 1 lists typical mechanical properties of NiTi SE-SMA large diameter bars at room temperatures (Yin et al, 2015). SMAs present an asymmetrical stress-strain compressive and tensile behaviour. However, it is sufficiently accurate to assume the tensile-based models to represent the entire behaviour of this material, according to Tarzav and Saiidi (2015) after investigating different NiTi commercial samples.

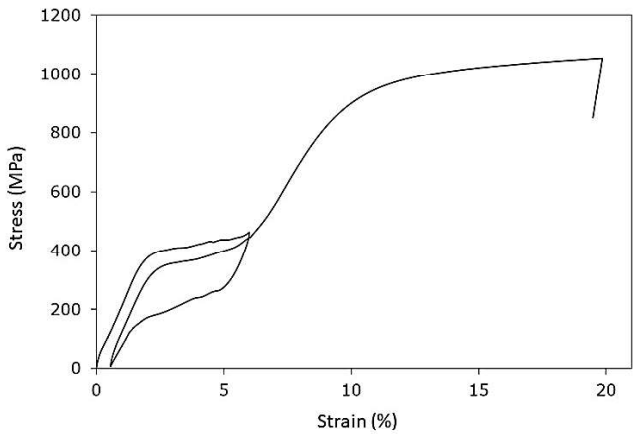

Figure 3: Stress-strain relationship of NiTi superelastic shape memory alloy bars (Cortes-Puentes, 2017)

Table 1: Mechanical properties of large diameter SMA bars at ambient temperatures (Yin et al., 2015)

Property	***Value***
Upper plateau stress	*400-500 MPa*
Elastic modulus	*40-75 GPa*
Ultimate tensile stress	*1100-1400 MPa*
Ultimate elongation	*15-20%*

3. Numerical model

Numerical analyses were conducted using VecTor2, a two-dimensional nonlinear finite element program that is applicable to membrane structures under static and dynamic loading. The program has been selected since it is capable of successfully capturing the behaviour of numerous reinforced concrete structural elements without requiring extended computational time when compared to other commercial programs. Figure 4

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illustrates the FE models developed for analysis. Figure 4 a) illustrates the Bare Concrete Frame (BCF), while Figure 4 b) represents the BRB retrofitted frame. In the models, the transverse reinforcement was modeled as smeared within four-node, eight-degrees-of-freedom plane stress rectangular concrete elements. The longitudinal reinforcement, however, was modeled as discrete, two-node truss bars with uniform cross-sectional area. The discrete reinforcement was perfectly bonded to the concrete elements. For the retrofitted frame, the brace was placed diagonally at an angle of 41 degrees.

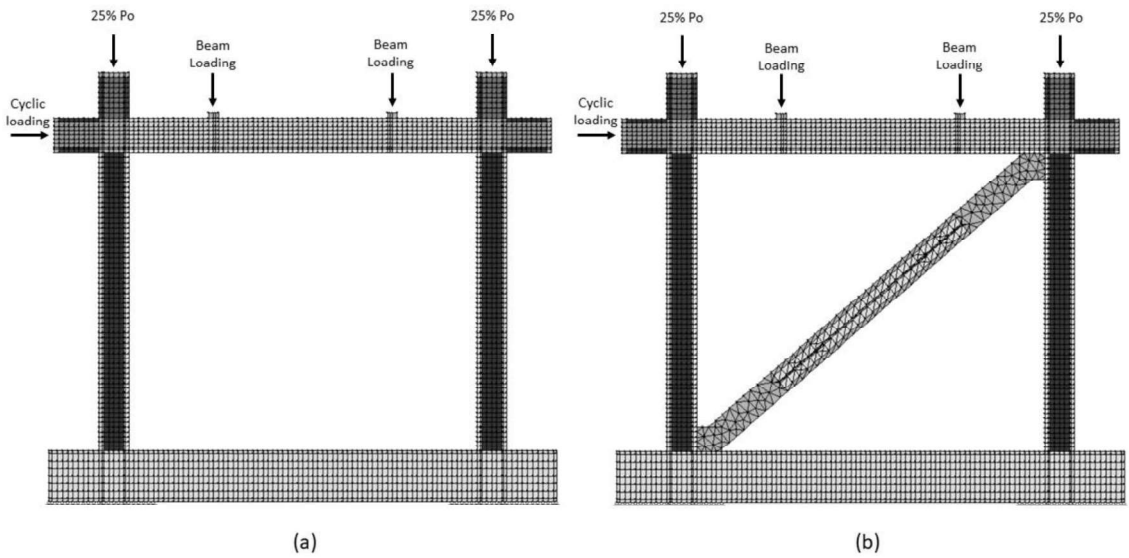

Figure 4: Finite element models: (a) BCF; and (b) BRB-retrofitted RC frame

Figure 4 also illustrates the applied lateral and gravity loading. The frames were analyzed under constant axial load applied on the columns and transverse load applied to the beam. The reverse cyclic load was applied at the center of the beam. The lateral loading pattern was selected to induce forces that simulate high levels of inelastic deformations that may be experienced by RC frames under severe earthquakes. In addition, the loading consisted of three cycles of incrementally increasing lateral displacements reversals as per ACI-374.1-05 (2006) loading protocol. The behaviour of the materials was defined through the default constitutive models in VecTor2. Ductile steel reinforcement was used for the discrete steel bars characterized by a linear-elastic region, yielding plateau, and strain-hardening until rupture (Seckin, 1981) as depicted in Figure 5 a). The SE-SMA BRB core bar followed the constitutive model proposed by Abdulridha et al. (2013) as illustrated in Figure 5 b). The SE-SMA constitutive model can capture the pseudo-yielding characteristic of shape memory alloys and can be employed for the material under tensile or compressive stress states.

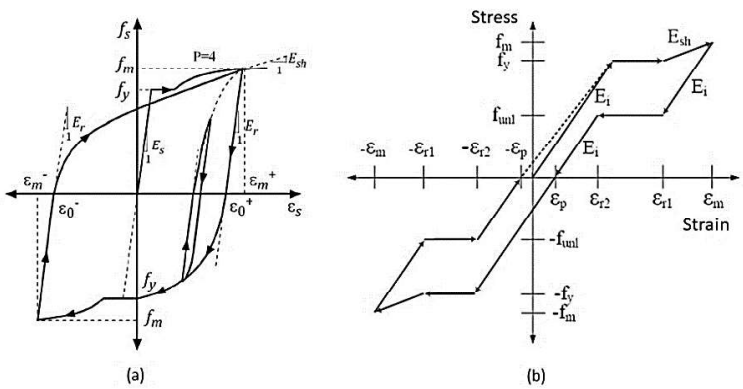

Figure 5: Stress-strain relationship of: (a) steel bars (Seckin, 1981); and (b) SE-SMA bars (Abdulridha et al., 2013)

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To perform time-history analyses, simplifications in the finite element model were necessary to attend the restrictions in the program. In order to reduce the structure's stiffness matrix, the number of degrees of freedom were reduced by increasing the mesh size. Furthermore, the seismic weight was converted into mass and applied along the beam. The total applied mass on the structure was equal to 38160 kg. The ground motion acceleration was considered in the horizontal direction. As the first mode of vibration of the prototype frame accumulates more than 90% of the structural mass, a viscous damping of 1 % was assigned to this mode. The applied earthquake records were selected from the Pacific Earthquake Engineering Research (PEER) database and scaled to match the Uniform Hazard Spectra prescribed by the NBCC 2015 (NRC, 2015) of the city of Vancouver in Canada. The spectral matching technique developed by Atkinson (2009) was utilized in this research since it provides an accurate fit between the target spectrum and historical records over a determined period range. To accurately evaluate residual displacements, the time-history analyses were carried out with additional 20 seconds beyond the earthquake record duration as this allowed the structure to decay from free vibration.

4. Static response and model validation

The numerical lateral load-lateral displacement responses were compared to the experimental results from Al-Sadoon (2016). Figure 6 illustrates the experimental and numerical hysteretic lateral load – lateral displacement response for the bare concrete frame (BCF). The FE model satisfactorily simulated the response of the frame. The maximum lateral load capacity recorded during the experimental testing was 233 kN and 219 kN in the positive and negative directions, respectively; while for the numerical model, it was 238 kN and 219 kN, respectively. The frame was considered to have reached its maximum drift capacity at a displacement of 95 mm (3% drift) given that the lateral strength decreases by 20% from the peak beyond that drift. At the end of the FE analysis, significant cracks were captured at the base of the columns and at the beam-column joints. This was consistent with the observed behaviour during testing. Furthermore, a residual drift of 1.6% was experienced by the frame in the numerical model, which is similar to the permanent drift observed on the tested frame. Therefore, the numerical model was considered representative of the behaviour of the bare concrete frame. For the frame retrofitted with a BRB incorporating a Stainless-Steel (SS) core bar, the validation focused on the strength capacity, ultimate displacement, failure mode, and crack patterns. The numerical response of the retrofitted frame was compared to the experimental results from Al-Sadoon (2017).

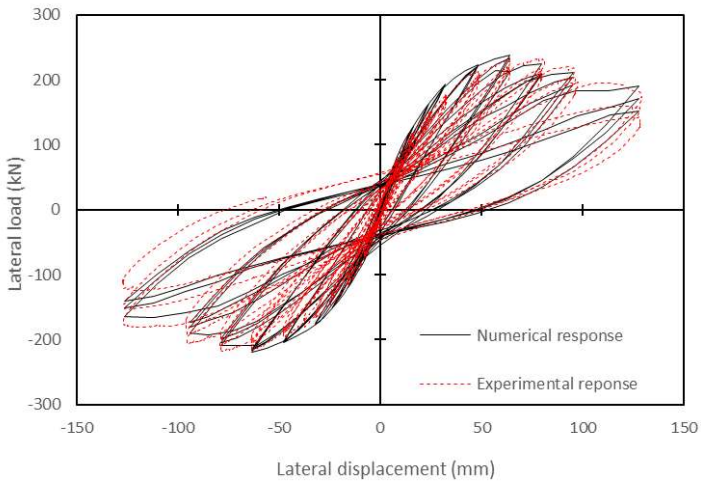

Figure 6: Numerical and experimental hysteretic responses of BCF

Figure 7 illustrates the hysteretic response of the steel core BRB retrofitted frame. For the BRBs with stainless-steel core, the increase in lateral strength capacity was approximately 2.5 times higher relative to the bare concrete frame, which is inline with the current base shear force requirements of the seismic provisions. However, the steel core BRB model resulted in a lower drift capacity in comparison to the original frame

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(BCF). Failure of the retrofitted frames is attributed to the concentration of stresses within the BRB caused by localized buckling. When comparing the numerical response of the retrofitted frame against the experimental one, it was found the numerical result corroborate that the response of the numerical model of the retrofitted system comply with the experimental observations. The strength capacity in the retrofitted frame differed less than 5% between the numerical and experimental response. Furthermore, the numerical and experimental frame failed at the same drift ratio and faced the same cracking patterns at yielding and right before collapse.

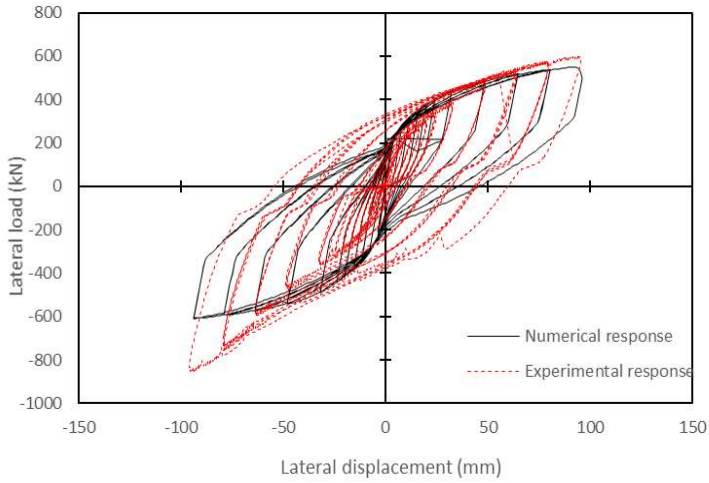

Figure 7: Numerical hysteretic response of BRB retrofitted frame equipped with a Stainless-Steel core

The retrofitted frame presented lower ductility compared to the BCF due to concentration of stresses and local buckling on the brace, these deficiencies of the current BRB design were addressed to increase the restraining of the core and improve the response of the frame, which will be discussed in Section 5. Furthermore, the residual displacement of the retrofitted frames was higher than what was experienced by the bare frame. As a result, the BRB increased lateral strength capacity, increased permanent deformations, and decreased ductility of the frame relative to the bare concrete frame.

5. Modified BRB design

The utilized BRB design for the comparisons previously presented were based on the work of Al-Sadoon (2016). For the SS retrofitted frame, failure was due to concentration of stresses within the core bar. Deficiencies in the original brace design have been assessed and a modified design is suggested herein to improve the response. For the modified BRB design, local buckling effects have been mitigated by omitting the bar spacers and decreasing the steel pipe radius from 59mm to 44.5mm, as illustrated in Figure 2 b). The improved hysteretic response is presented in Figure 8 for the conventional steel core BRB. The failure mode of the retrofitted frame was not controlled by the core bar, as previously identified, but by the structural damage experienced by the frame at the base of the columns. Therefore, the drift level of the retrofitted frame, independent on the core bar material, was increased in addition to the lateral load capacity.

Figure 9 illustrates the behaviour of the BRB retrofitted frame with a SE-SMA core. The lateral capacity of the retrofitted frame increased by a factor of 3.7 relative to the bare frame (BCF). Furthermore, improvements in ductility are evident, and the self-centering capacity of the alloy core bar led to reduced structural damage. In terms of permanent displacements, at a drift of 3%, the control frame accumulated a residual displacement of 28.6 mm, while the frame retrofitted with the modified BRB with S-S and SE-SMA core accumulated 44.5 mm and 4.34 mm, respectively. That indicates that the conventional steel core bar experienced an increase in the permanent displacement of the frame, which can be translated as extensive structural damage and costly repair. Conversely, the frame retrofitted with the modified BRB and an SE-SMA core bar accumulated only 15% of what was accumulated by the control frame.

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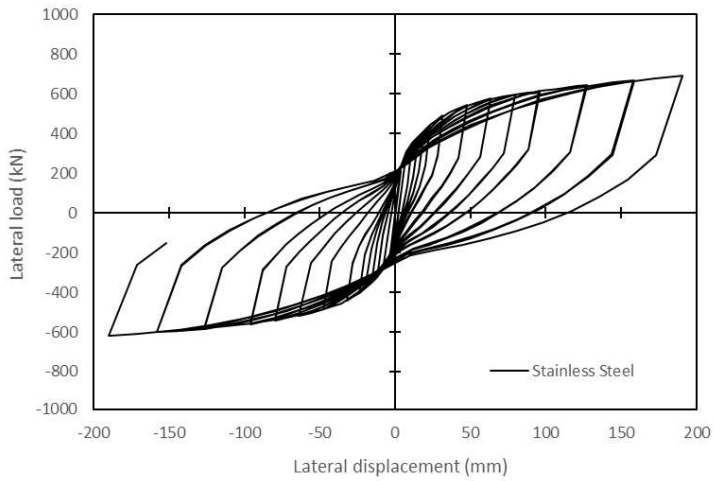

Figure 8: Numerical hysteretic response of the frame retrofitted with an improved BRB design

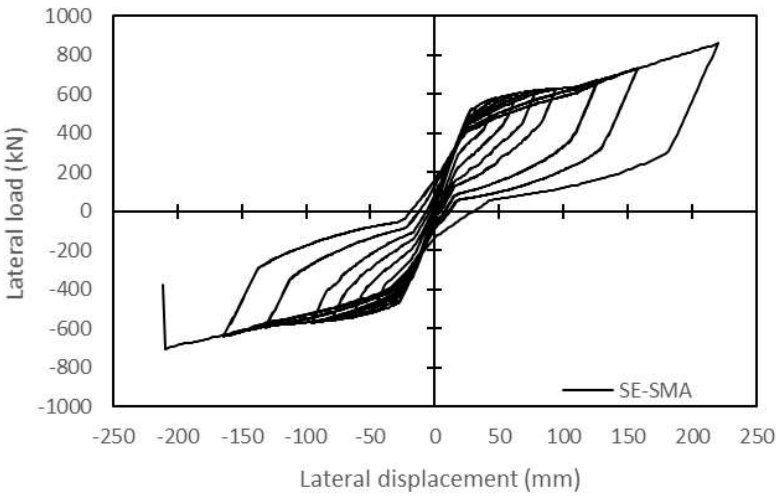

Figure 9: Numerical hysteretic response of SE-SMA BRB retrofitted frame

6. Time-history response

The performance of the frame retrofitted with the modified BRB incorporating an SE-SMA core is assessed against the response of the bare concrete frame under the same earthquake records characteristic of the city of Vancouver. Three M-R scenarios are generated for the city of Vancouver according to Tremblay et al. (2015), the first M-R scenario covers fundamental periods between 0.2 to 0.8s, the second from 0.3 to 1.5s, and the third from 1 to 1.5 seconds. Under the earthquake records, the mean transient displacement was 82.774 mm. This displacement is above the yielding point and the frame is expected to experience inelastic displacements and ratcheting. The frame exceeded the yielding point in all eleven ground motion records, and in three records it exceeded the ultimate displacement experienced by the bare frame in the static analysis. For the first M-R scenario, the maximum transient displacement was 95.52 mm (drift of 3.01%), where the frame experienced ratcheting, and a residual displacement of 6.05 mm was observed at the end of the earthquake record. A transient drift above 2.5% characterizes a structure that falls under the structural stability performance level, where extensive structural damage is observed, but the principal load carrying elements are able to transfer the load to the foundation avoiding building collapse (FEMA, 2012). Therefore, the control frame could ensure life safety, but an extensive repair would be necessary before reoccupation.

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For the ground records that portray the second M-R scenario, one induced transient displacement up to 144.75mm to the control frame, which is equivalent to a drift of 4.56%. This displacement exceeds the maximum displacement the frame can sustain according to the static analysis. The frame experienced ratcheting, and the damage accumulated in one of the directions, which is corroborated by the presence of a residual displacement at the end of the analysis corresponding to 19.55 mm (drift of 0.61%). The earthquake records representative of the third M-R scenario for the city of Vancouver induced a transient displacement of 134.72 mm (drift of 4.24%) to the BCF, and a permanent displacement of 22.4 mm (0.7% drift) was observed once the frame reaches free vibration. The peak drift was higher than the displacement capacity of the frame. Therefore, the frame performed beyond the structural stability level, while it performed within the life safety level when evaluating the permanent drift. By observing the crack patterns of the BCF, both flexural and shear cracks were observed in all eleven earthquake records. Flexural cracks were noted at the beam mid-span and at the base of the columns, while shear cracks were verified adjacent to the beam-column joints given that the joint was poorly transversely reinforced.

The retrofitted frame behaved within the operational performance level, which is characterized by low risk of life-threatening injuries from structural failure and immediate re-occupancy, in all the earthquake records. The mean transient displacement experienced by the retrofitted frame was 0.85 mm corresponding to a drift of 0.026%, while the mean permanent displacement corresponded to 0.003 mm. The stiffness added by the brace diminished both the transient and permanent drifts improving the seismic performance of the frame. Figure 10 illustrates the improvement in seismic performance by retrofitting the frame with the modified BRB subjected to an earthquake record characteristic of the second M-R scenario; it is evident that the transient and residual drifts of the retrofitted frame are negligible. The advantages of utilizing the SE-SMA core in place of steel core is better evaluated under higher seismic loads. Therefore, the frames were also subjected to amplified ground motions to closely evaluate the response of the BRB retrofitted frame incorporating an SE-SMA core.

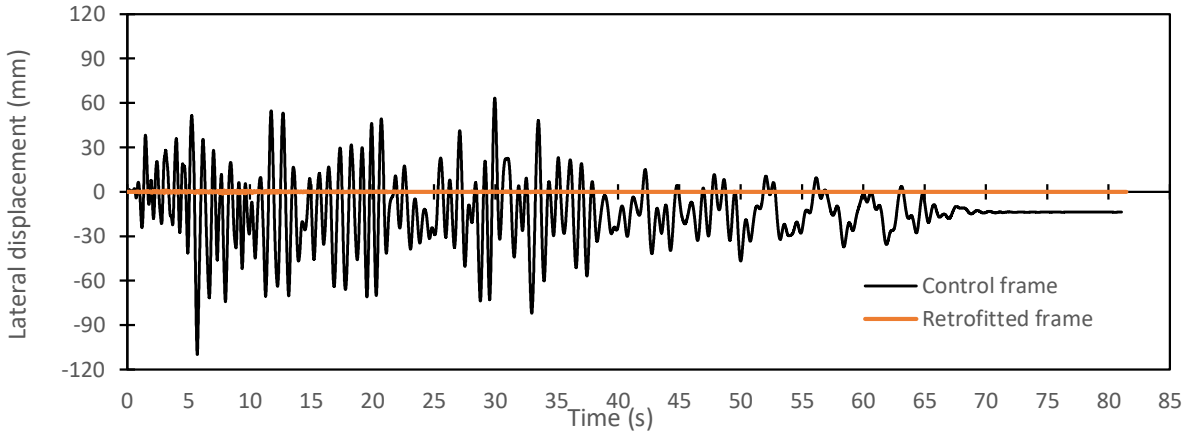

Figure 10: Lateral displacement-time-history of the bare concrete frame and retrofitted frame under a specified earthquake record

Figure 11 illustrates the response of both frames under the earthquake record amplified by 100%, where the red dashed lines define the ultimate displacement for the control frame evaluated from the nonlinear static analysis. The small transient displacements experienced by the retrofitted frame can be attributed to its large stiffness acquired after placing the brace. Ratcheting is evident in the response of the bare concrete frame, and an accumulation of damage is clear 50 seconds after the beginning of the record. The residual displacement recorded for the control frame at the end of the earthquake record corresponded to 26.534 mm (0.84 % drift). As the transient drift is above 2.5%, the frame is in severe risk of collapse. Regarding the BRB retrofitted frame, the maximum transient displacement was approximately 2.5 mm. The amplified record did not induce yielding nor inelastic displacements in the structure. An assessment of the crack patterns suggests concrete spalling in the control frame, which contributed to deterioration of the structure and degradation of its lateral strength. The control frame failed due to structural deterioration after concrete spalling was observed. After

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the peak displacement was experienced, the maximum lateral capacity of the frame was achieved and the longitudinal steel reinforcement at the base of the column fractured; thus, the capacity of the frame decreased more than 20% in the succeeding imposed displacements. For the retrofitted frame, shear cracks were evident in the column and near the beam-column joint, but the frame did not fail during the earthquake record.

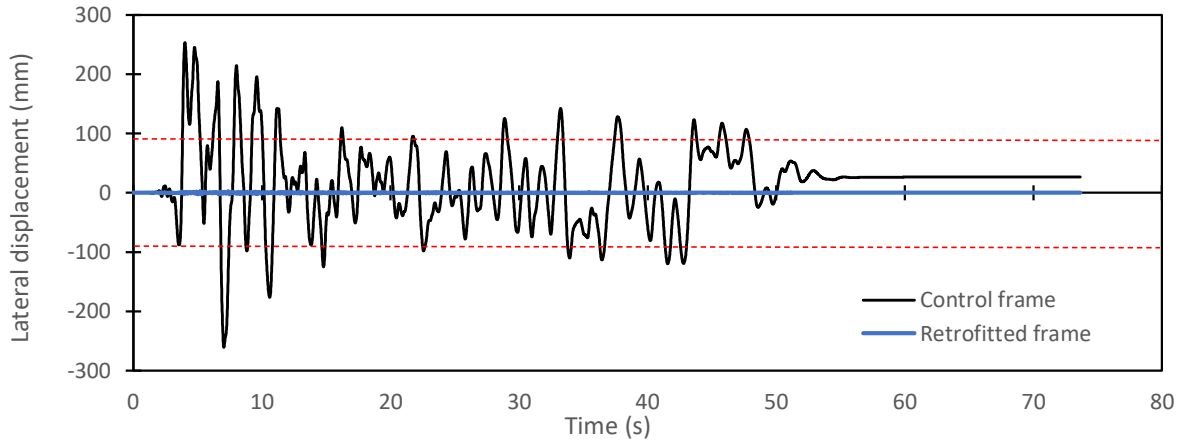

Figure 11: Displacement-time history of the bare and retrofitted frames under a 100% amplified earthquake record

7. Conclusions

In this study a finite element model was successfully developed and validated against experimental data, which permitted: an assessment of deficiencies of a proposed BRB design, improvements in the design, and numerical evaluation of the efficiency of the changes. This study has corroborated that buckling-restrained braces are an alternative to improve the seismic performance of pre-1970s reinforced concrete moment-resisting frames. Two types of core bars for the BRB were investigated, including Stainless Steel and Shape Memory Alloy (SMA). The SMA-BRB retrofitted frame provided the largest increases in lateral strength capacity and ductility. Although the steel core bar improved the lateral strength capacity, it resulted in an increase in the permanent deformation of the concrete frame, which was not experienced when the SE-SMA core bar was utilized.

Deficiencies of the original BRB design were identified and possible solutions to improve the response were presented. A numerical assessment of the improvements illustrated that localized buckling effects could be mitigated. As a result, the concrete frame experienced higher lateral drifts that were independent of the BRB core bar material. However, using common steel core increased the permanent deformation, which is indicative of additional damage that would not permit repair as an option post-earthquake. An alternative to minimize the damage is to use a self-centering material as the core bar, and for this study, Superelastic (SE) SMAs were investigated. The replacement of the core bar with an SE-SMA core bar lead to a retrofitted frame with the capacity to sustain large deformations, increase lateral load capacity, and control permanent deformations.

The time-history analyses indicated the improved seismic performance of the retrofitted frame in comparison to the bare concrete frame. While the control frame experienced transient displacements beyond the structural stability from the FEMA P-58-1 (FEMA, 2012), the retrofitted frame performed within the operational level in which immediate re-occupancy is allowed after an earthquake. Subjected to an amplified ground motion record, the bare concrete frame reached its maximum transient displacement, where longitudinal reinforcing bars at the base of the columns and near the beam-column joint fractured. Ratcheting was observed resulting in a considerable residual displacement, while the retrofitted frame did not yield and the structure was able to withstand the amplified ground motions without accumulating structural damage nor considerably presenting transient drifts, indicating the superiority of the retrofitted frame relative to the bare concrete one.

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