ANALYTICAL STUDY ON UPGRADING THE SEISMIC PERFORMANCE OF NOMINALLY DUCTILE RC FRAME STRUCTURES USING DIFFERENT REHABILITATION TECHNIQUES

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ABSTRACT :

Retrofitting or strengthening of existing reinforced concrete (RC) frame structures is one of the important obsessions for many researchers. Most of these structures were designed according to pre1970 strength-based codes; these codes did not consider the ductility of the structure as a main parameter to dissipate the earthquake energy. And hence these buildings are susceptible to abrupt a non-ductile strength deterioration once their ultimate capacity is reached. Such a brittle failure will lead to a reduction in the energy dissipation capacity of the building and hence the seismic resistance of the structure. Therefore, upgrading the seismic performance of such structures becomes essential to minimize the disastrous consequences of earthquakes. Several rehabilitation techniques can be used to enhance the seismic resistance of structures. This paper investigates analytically the effectiveness of four different rehabilitation techniques that can be used for enhancing the seismic performance of existing nominally ductile RC frame structures. The studied rehabilitation techniques include (1) introducing a RC structural wall, (2) using steel X-bracings (3) using diagonal fibre reinforced polymer strips (FRP bracings), and (4) wrapping or partially wrapping the frame members (columns and beams) using FRP composites. 5- and 15-storey existing frame models were studied representing low- and high-rise frames. The cases of bare frame and masonry-infilled frame were also considered. The existing frames were rehabilitated and subjected to three types of ground motion records with different frequency contents. The seismic performance of the existing and rehabilitated frames was evaluated in terms of the maximum applied peak ground acceleration resisted by the frames, maximum inter-storey drift ratio, maximum storey shear-to-weight ratio and energy dissipation capacity.

KEYWORDS: Reinforced concrete, rehabilitation, frames, seismic performance, nonlinear dynamic analysis.

INTRODUCTION

In previous earthquake events, it has been reported that several existing reinforced concrete (RC) frame structures designed according to pre1970 codes experienced severe damage, or even collapsed (Pampanin et al. 2006). This is mainly due to the fact that pre1970 codes adopted a strength-based concept which did not enforce the ductility measures and energy dissipation capacity of the structure. The lack of appropriate reinforcement detailing of the frame columns, beams, and joints led to low shear capacity of the beams and columns, and hence nonductile strength deterioration occurred when that shear capacity was reached. Despite the fact that many nominally ductile existing structures did survive previous low-to-moderate ground motion events, the level of damage attained in these structures deems them vulnerable to collapse in future earthquake events. Therefore, rehabilitation of such structures is essential and cannot be neglected.

Performance-based (PB) seismic engineering is the modern approach to earthquake resistant design. Figure 1 shows the typical seismic performance of existing structures versus structures designed according to performance-based seismic engineering. Seismic performance (*performance level*) is described by designating the maximum allowable damage state (*damage parameter*) for an identified seismic hazard (*hazard level*). Performance levels describe the state of a structure after being subjected to a certain hazard level as: Fully operational, Operational, Life safe, Near collapse, or Collapse (FEMA 1997; SEAOC 1995). Overall lateral deflection, ductility demand, and inter-storey drift are the most commonly used damage parameters. The five qualitative performance levels are related to corresponding five quantitative maximum inter-storey drift limits (as a damage parameter) to be: <0.2, <0.5, <1.5, <2.5, and >2.5%, respectively. These limits are functions of the lateral force resisting system and the type of non-structural elements in the building. Therefore, the permissible drift limits should be evaluated using caution and judgment. The hazard level can be represented by the probability of exceedence of 50, 10 and 2% in 50 years for low, medium and high intensities of ground motions, respectively. From the schematic it can be seen that upgrading the seismic performance of existing nominally ductile structures can be achieved by increasing the capacity of the structure with or without reducing its drift. Increasing the capacity while reducing the lateral interstorey drifts can be achieved by increasing the stiffness of the building, e.g., by using RC walls or steel bracings. Increasing the capacity without reducing the drifts can be achieved by increasing the ductility capacity of the structural elements of the building without altering their stiffness, e.g., by using FRP wrapping.

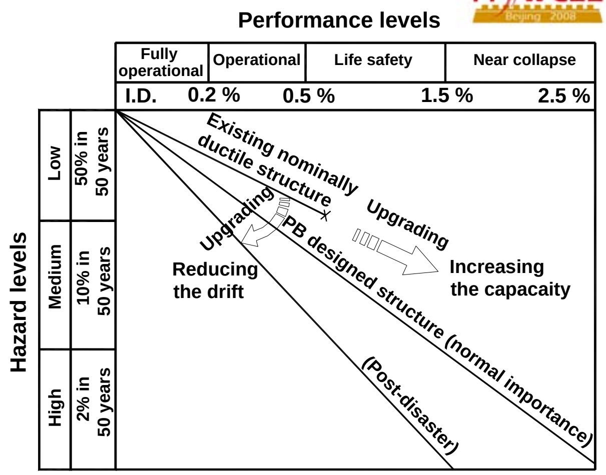


Figure 1 Seismic performance of existing structures and possible ways of upgrading.

The objective of this paper is to conduct a comparative study on the effectiveness of different rehabilitation techniques used in upgrading the seismic performance of existing nominally ductile RC frame structures. The study investigates the behaviour of two RC frame structures with different heights –representing existing buildings designed according to pre1970 strength based codes– when rehabilitated using different rehabilitation schemes and subjected to three types of scaled ground motions records. The studied rehabilitation techniques include (1) introducing a RC structural wall, (2) using steel X-bracings (3) using diagonal fibre reinforced polymer strips (FRP bracings), and (4) wrapping or partially wrapping the frame members (columns and beams) using FRP composites. Five and 15 storey existing frame models were studied representing low- and high-rise frames. The cases of bare frame and masonry-infilled frame were also considered. The existing frames were rehabilitated and subjected to three types of ground motion records with different frequency contents. The seismic performance of existing and rehabilitated frames was evaluated in terms of the maximum applied peak ground acceleration (PGA) resisted by the frames, maximum inter-storey drift (I.D.) ratio, maximum storey shear-to-weight ratio and energy dissipation capacity.

1. PROPERTIES OF THE SELECTED GROUND MOTIONS

Tso et al. (1992) examined the significance of the acceleration/velocity (A/V) ratio of the ground motion record as a parameter to indicate the dynamic characteristics of earthquakes. Three sets of strong ground motion records were analyzed with low, medium and high A/V ratios. It was found that the A/V ratio can be used as a simple indicator of the frequency content of the ground motion. In this study, a set of nine far-field earthquake records was selected for analysis (Tso et al. 1992; PEER 2006). The ground motion records represent earthquakes with low-, medium-, and high-frequency contents. The properties of selected ground motions are shown in Table 1.

Table 1 Properties of the selected ground motions (Adapted from PEER 2006).

No.	Earthquake	Site	Date	A (g)	V (m/s)	A/V (g. s/m)	A/V Level	Duration (s)	Soil condition
1	Lower California	El Centro	Dec. 30, 1934	0.160	0.209	0.766	Low	90.36	Stiff soil
2	San Fernando, Cal.	2500 Wilshire Blvd., LA	Feb. 9, 1971	0.101	0.193	0.518		25.32	Stiff soil
3	Long beach, Cal.	LA Subway Terminal	Mar. 10, 1933	0.097	0.237	0.409		99.00	Rock
4	San Fernando, Cal.	234 Figueroa St., LA	Feb. 9, 1971	0.200	0.167	1.198	Med.	47.10	Stiff soil
5	Kern County, Cal.	Taft Lincoln School Tunnel	July 21, 1952	0.179	0.177	1.011		54.42	Rock
6	Imperial Valley	El Centro	May 18, 1940	0.348	0.334	1.042		53.76	Stiff soil
7	Lytle Creek, Cal.	6074 Park Dr., Wrightwood	Sep. 12, 1970	0.198	0.096	2.063	High	16.74	Rock
8	Parkfield, Cal.	Cholame, Shandon	June 27, 1966	0.434	0.255	1.702		44.04	Rock
9	San Francisco	Golden Gate Park	Mar. 22, 1957	0.105	0.046	2.283		39.88	Rock

2. PROPERTIES OF THE SELECTED BUILDINGS

Two buildings designed according to the pre1970 strength-based code (ACI 1968) were selected for this study. The buildings had five and 15 stories to represent low- and high-rise buildings, respectively. The two buildings had the same floor plan, which consisted of three symmetrical bays in both directions, where the bay width was 6 m. The floors were designed to carry their own weight and a live load of 2 kPa; the floor height was 3.25 m and the total heights of the two buildings were 16.25 and 48.75 m, respectively. The elevations of the two buildings, the concrete dimensions for the beams and columns, as well as the steel reinforcement are shown in Fig. 2. The dimensions of the column section and the steel reinforcement ratios were varied along the height according to the change of axial load acting on each group of columns while the beam dimensions were assumed to be the same for the entire building.

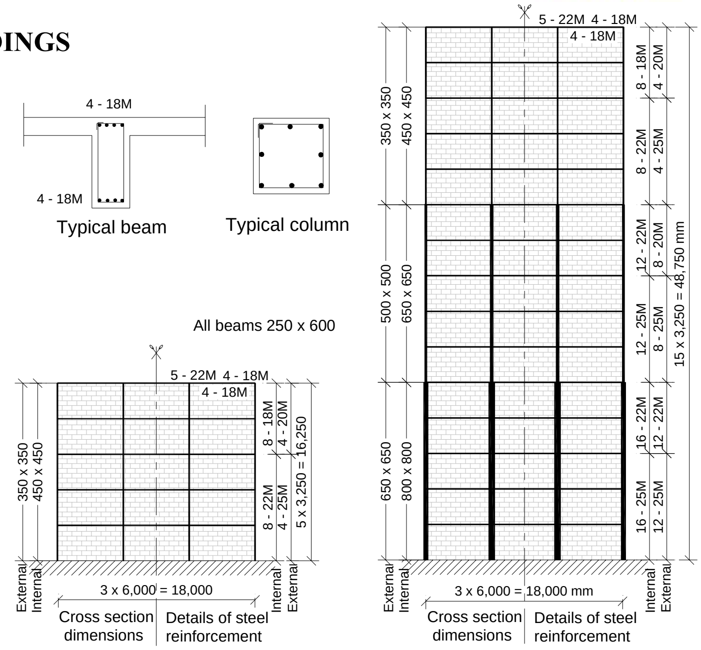


Figure 2 Elevations of the five and 15 storey studied frames.

For existing buildings, the compressive strength of concrete was assumed to be $f_c^{'}$ = 25 MPa and the yield strength of steel was set to be f_y = 400 MPa. The modulus of elasticity for concrete was taken to be 20 GPa and that of steel was taken to be 200 GPa. The concrete density was assumed to be 24 kN/m^3 and concrete Poisson's ratio υ = 0.15.

3. EXISTING STRUCTURES AND DIFFERENT REHABILITATION SCHEMES

Five and 15 storey existing frames representing low- and high-rise building, respectively, were studied. For both frames, bare and masonry-infilled frames were considered in the analysis. In order to investigate the effect of infill stiffness on the seismic response of the frames, two different masonry infill stiffnesses were considered. The infill stiffnesses represent soft and stiff infills. The three types of existing frames (bare, with soft, and stiff masonry infill) were rehabilitated using four techniques; the first is by demolishing the masonry panel in the middle bay and introducing a RC wall. The RC wall used is 6 m long, and has a thickness of 200 mm for the 15-storey frame, and a thickness of 100 mm for the 5-storey frame that represents a RC wall every other frame. The wall dimensions and reinforcement ratio were assumed to remain constant along the height. The second technique is by using steel X-bracings in the middle bay along the frame height. The 5- and 15-storey frames were strengthened using HSS 219 x 6.4 and HSS 273 x 11, with cross-sectional area of 4240 and 9160 mm^2, respectively. The third technique is by using FRP bracings along the frame height. The fourth technique is by wrapping the columns laterally with FRP, while the beams were rehabilitated using FRP U-wraps near their ends. A total of fourteen cases were studied for each frame; the cases of existing, rehabilitated with RC wall, rehabilitated with steel bracings, rehabilitated with FRP bracings, and rehabilitated columns and beams with FRP-confinement were studied for the bare frame, soft infill, and stiff infill frame models. Fig. 3 shows the rehabilitation schemes by introducing RC wall, using steel bracings, and by using FRP bracings.

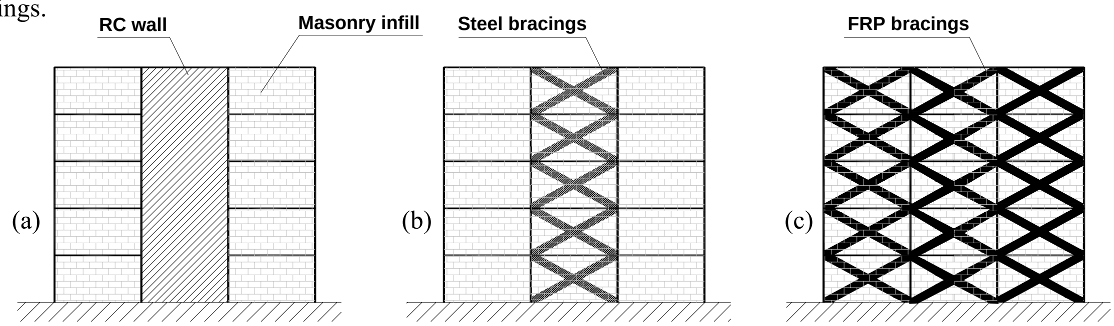


Figure 3 The rehabilitation schemes by (a) introducing RC wall; (b) using steel bracings; (c) using FRP bracings.

4. NONLINEAR MODELS USED IN THE TIME HISTORY ANALYSES

Non-linear dynamic analyses were conducted for the five and 15-storey frames with different rehabilitation schemes, a computer software for three dimensional nonlinear and dynamic structural analysis CANNY (Li 2006) was selected for the analyses. The mass of each floor was lumped at the column joints according to the tributary areas. P-delta effects were considered in the analyses. Fig. 4 shows the idealization of different members of the studied frames assembled in one figure.

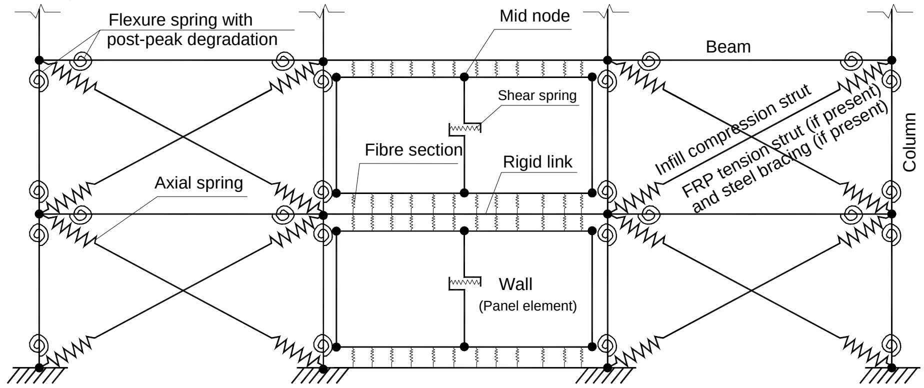


Figure 4 Idealization of different members of the studied frames (generic).

4.1. Modeling of beams and columns

The beams and columns were modeled as linear elastic element with two inelastic single-component flexure rotation springs located at the ends of the member. Deterioration Model CP4 (Li 2006) was used to model the nonlinear flexure rotation spring, which allows representation of the combined flexural and shear backbone curve with a parameter that controls the displacement ductility capacity after which the post-peak degradation occurs as shown in Figure 5. The force-displacement ductility relationship of the columns and beams of the existing frames was assumed to have limited displacement ductility, μ_Δ=2, that is followed by a quick reduction in the lateral load resistance.

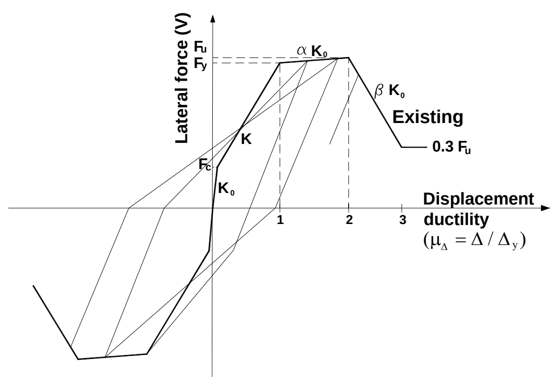
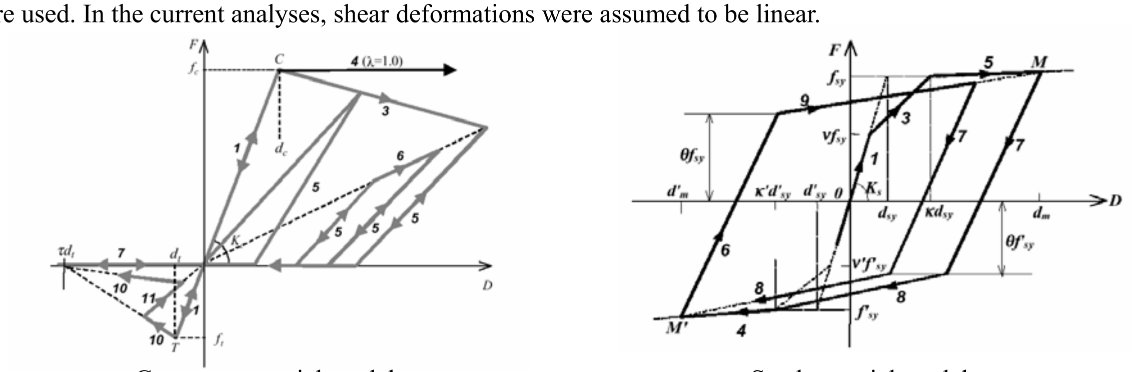


Figure 5 Hysteretic behaviour of the model CP4 (Li 2006).

4.2. Modeling of RC wall

The RC wall was modeled using CANNY panel element. The panel element has four nodes at the corners in addition to a node at the mid points of the top and bottom boundaries. The adjacent panels have compatible deformations at their common three nodes that are connecting them. Multi-Axial spring model was used to represent the flexural and axial tension/compression interaction of the panel elements. Multi-linear curves were used to represent the force-deformation relationship for steel and concrete springs. Figure 6 shows the concrete and steel spring models that were used. In the current analyses, shear deformations were assumed to be linear.



Concrete material model

Steel material model

Figure 6 Hysteretic models of the concrete and steel fibres for the wall element, respectively.

4.3. Modeling of steel bracings

The steel braces were modeled as axial tension/compression struts. CANNY Buckling Model (STB) was used to represent the hysteretic behaviour of the bracing member. The model STB is able to represent the reduction in the maximum compressive strength with increasing the number of load cycles as was observed by Jain et al. (1980) in their experimental tests. The main input data for the model is the maximum tensile force resisted by the member and the effective slenderness ratio which control the value of maximum compressive strength. The hysteretic behaviour of the model (STB) is shown in Figure 7.

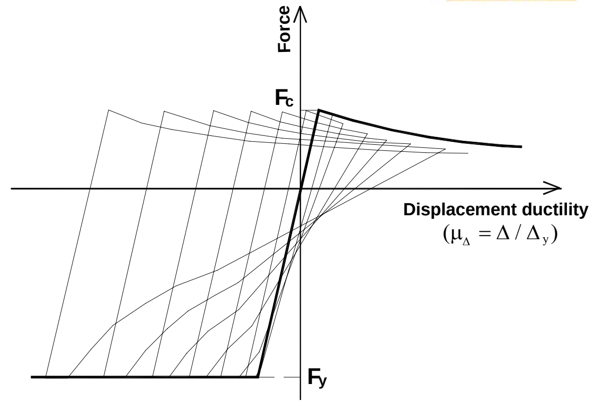


Figure 7 Hysteretic behaviour of CANNY Buckling Model STB (Li 2006).

4.4. Modeling of masonry infill

Özcebe et al. (2003) proposed an analytical representation to predict the behaviour of masonry-infilled frames when rehabilitated with FRP bracings. The analytical model was correlated to experimental tests carried out at Middle East Technical University (METU) on a number of two-storey masonry-infilled frames rehabilitated with different patterns of FRP, and subjected to cyclic displacement excitations at the storey levels. A similar uniaxial model for the masonry infill and FRP bracings was used in the current study. The unrehabilitated infill panels were modeled as compression struts. The properties of the compression strut were chosen based on Özcebe et al. (2003), and they were scaled to match the panel dimensions of the studied frames. The tensile resistance of the masonry infill was neglected in the analyses. In this study, two different masonry infill types with different stiffness were considered, representing soft and stiff infill. Figure 8 (a) shows the axial stress-axial strain relationship for the compression strut of the infill.

4.5. Modeling of FRP bracings

The FRP bracings were modeled as uniaxial tension strut with maximum axial strain of 0.003 and maximum axial stress of 190 MPa, these values takes into account the characteristics of CFRP, infill, and the anchor dowels. Figure 8 (b) shows the axial stress-axial strain relationship for the tension strut of FRP bracings.

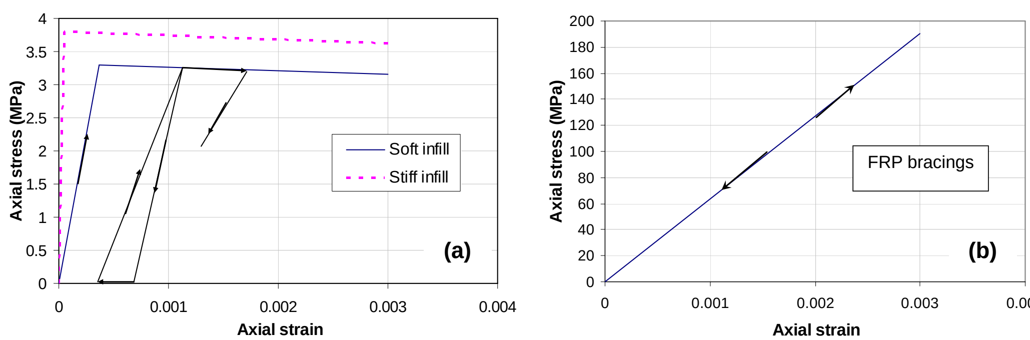


Figure 8 Strut model for (a) masonry infill; (b) FRP bracings

4.6. Modeling of rehabilitated columns and beams with FRP confinement

In this technique, the columns were rehabilitated by wrapping them laterally with FRP composites, while the beams were rehabilitated using FRP U-wrap near their ends. The rehabilitated columns and beams were modeled with the same model as the existing members. The FRP content used in the rehabilitation of the structural elements was assumed to increase their energy dissipation capacities and displacement ductility, $\boldsymbol{\mu_\Delta}$, to be equal 6.0 (compared to 2 in case of existing members). The force-displacement ductility backbone curves for the existing and rehabilitated frames using FRP confinements are compared in Figure 9. From the figure, it can be seen, that strength degradation was considered in the element's lateral force-displacement ductility backbone curve which corresponds to the onset of shear failure.

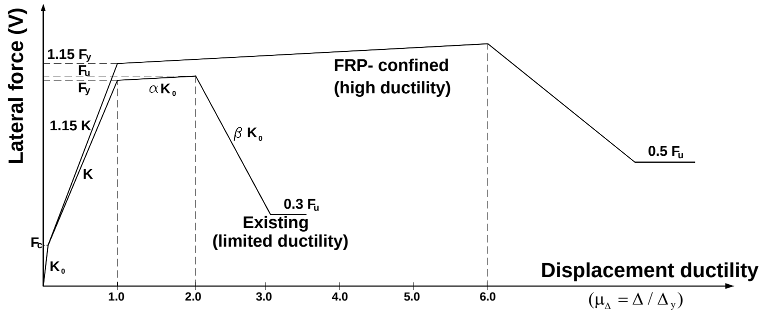


Figure 9 Force-displacement ductility relationships of the existing and rehabilitated frames using FRP confinement.

5. ANALYSIS RESULTS

5.1 Maximum applied PGA

Nine earthquake records were applied on the studied frames. The ground motion records represent a sample of earthquakes with low, medium and high frequency contents. In the analyses, the maximum earthquake intensities that can be resisted by the five and 15 storey frames were evaluated for different rehabilitation schemes and earthquake records. Figure 10 shows the maximum PGA resisted by different rehabilitation patterns and different infill stiffnesses for five and 15 storey frames. The figure shows the average value for the PGA capacity for earthquakes with low, medium and high frequency contents (A/V ratio). This is useful to evaluate the effect of earthquake frequency content on the seismic behaviour of the studied frames. The figure also shows the average PGA capacity value for all earthquakes. From the figure, it can be seen that the use of FRP bracings increases the maximum PGA resisted by the studied frame, while rehabilitating the frames using RC wall or steel bracing has resulted in a higher PGA capacity compared to FRP bracings. The figure also shows that rehabilitating the columns and beams using FRP-confinement is an efficient technique in increasing the PGA capacity of the frames, especially for medium- and high-rise buildings.

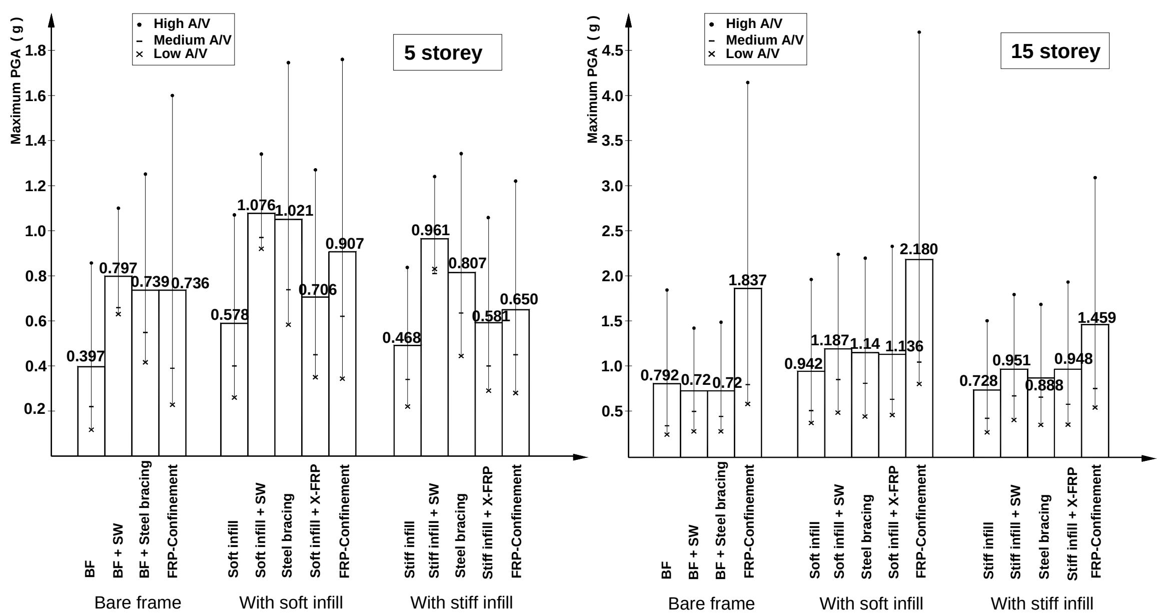


Figure 10 Maximum PGA resisted by low- and high-rise frames.

5.2 Maximum inter-storey drift

Figure 11 shows the maximum inter-storey drift (I.D.) ratio capacity for the five and 15 storey frames for different rehabilitation schemes and different infill stiffnesses. From the figure, it is noticed that FRP bracings has negligible effect on the maximum I.D. ratio capacity, which matches the experimental test results carried out in METU (Özcebe *et al.* 2003). On the other hand, rehabilitating the frames using a RC wall or steel bracing resulted in a reduced maximum I.D. capacity. This can be attributed to the fact that adding the RC wall or steel bracings increased the stiffness of the structure. It should be noted that for the three above mentioned rehabilitation schemes, the failure occurred in the non-ductile columns and beams of the existing structure. Also it can be seen that the maximum I.D.

ratio capacity is high for the rehabilitated frames using FRP confinement for low- and high-rise frames, which leads to a more ductile structure with a higher I.D. ratio capacity. The figure also shows the performance levels that represent the damage degree of structure in terms of inter-storey drifts as recommended by SEAOC (1995) and FEMA (1997).

Figure 12 shows the PGA–I.D.$_{max}$ capacity curves of the existing five and 15 storey frames with soft infill when rehabilitated using different rehabilitation patterns. From the figure, it can be seen that rehabilitating the frame with FRP bracings resulted in a decrease in the maximum I.D. ratio for a certain PGA level when compared to the existing frame. While for the same PGA level, rehabilitating the frames using a RC wall or steel bracing decreased the value of maximum I.D. compared to the use of FRP bracings. It is worth noting that the rehabilitation scheme using steel X-bracing resulted in a significant increase of axial force on the existing columns at the lower stories, which indicates the importance of strengthening the existing columns (e.g. by jacketing them) to increase their axial capacity in order to be able to tolerate the axial force accompanied by the lateral loads due to the introduction of steel braces, also retrofitting of foundation maybe needed. The figure also shows that rehabilitating the frames using FRP confinement did not decrease the value of maximum I.D. ratio at the same PGA level but it increased the maximum PGA and maximum I.D. ratio capacities for the structure significantly.

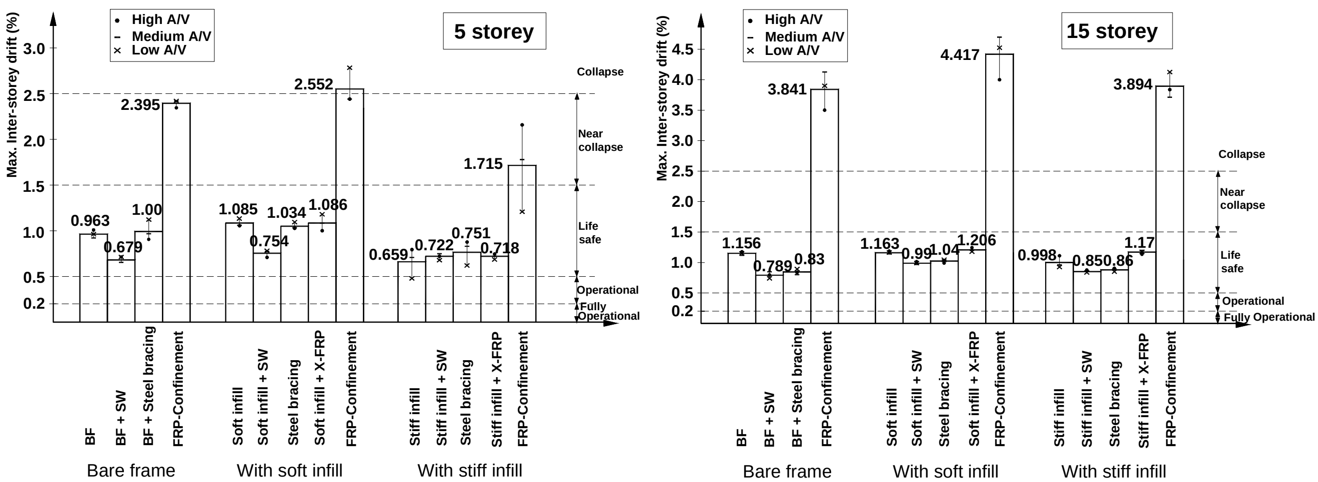


Figure 11 Maximum inter-storey drift ratio capacity for low- and high-rise frames.

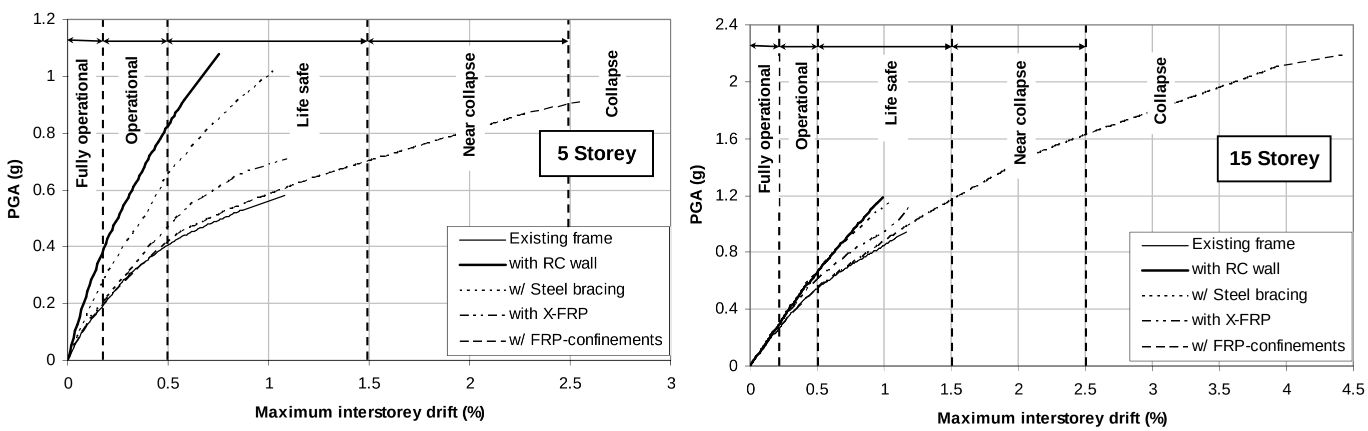


Figure 12 Capacity curves of the five and 15 storey frames with soft infill.

5.3 Maximum storey shear

Figure 13 shows the maximum storey shear-to-the total structure weight ratio obtained for the five and 15 storey frames for different rehabilitation schemes and two infill stiffnesses. It can be seen that rehabilitating the frames using a RC wall or steel bracings attracted higher forces due to the increase of stiffness, which resulted in a reduction in the natural period of the structure. The figure shows also that the presence of masonry infill led to a stiffer structure and hence increasing the demand, thus highlighting the importance of inclusion of infill models in the analysis of masonry-infilled structures. It can be also seen that the rehabilitation using FRP-confinements did not increase the storey shear demand significantly. This can be attributed to the fact that FRP-confinements do not contribute to the structure stiffness, but they increase the ductility capacity of the structural elements of the building without altering their stiffness.

5.4 Energy dissipation capacity

Energy dissipation capacity is an important indicator of the structure's ability to withstand severe ground motions. It can be determined from the area enclosed by the hysteretic loops of the load deformation relationship. Figure 14 shows the maximum energy dissipated by the five and 15 storey frames for different rehabilitation patterns and two infill stiffnesses. It can be seen that for low-rise building, the rehabilitation using a RC wall or steel bracings dissipated higher energy than the case of FRP bracings or FRP confinement, which indicates that the introduction of RC wall will be more efficient in resisting the lateral loads than the use of FRP bracing or FRP-confinements, yet this solution will be impractical if the building was occupied while rehabilitation. In that case the steel bracings, FRP bracings or FRP confinement could be alternative solutions. The figure also indicates that for the studied high-rise frames, wrapping the columns and beams using FRP-confinements will dissipate higher energy than introducing a RC wall, steel bracings or FRP bracings. It can be seen also that accounting for the presence of the masonry infill in the analytical model has increased the energy dissipated by the frames.

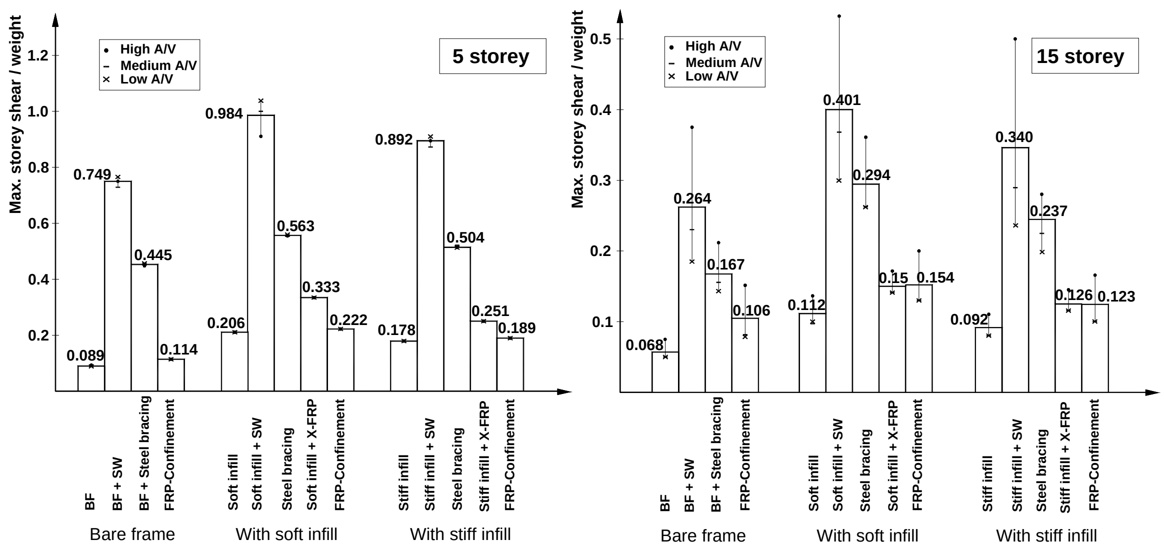


Figure 13 Maximum storey shear-to- weight ratio for low- and high-rise frames.

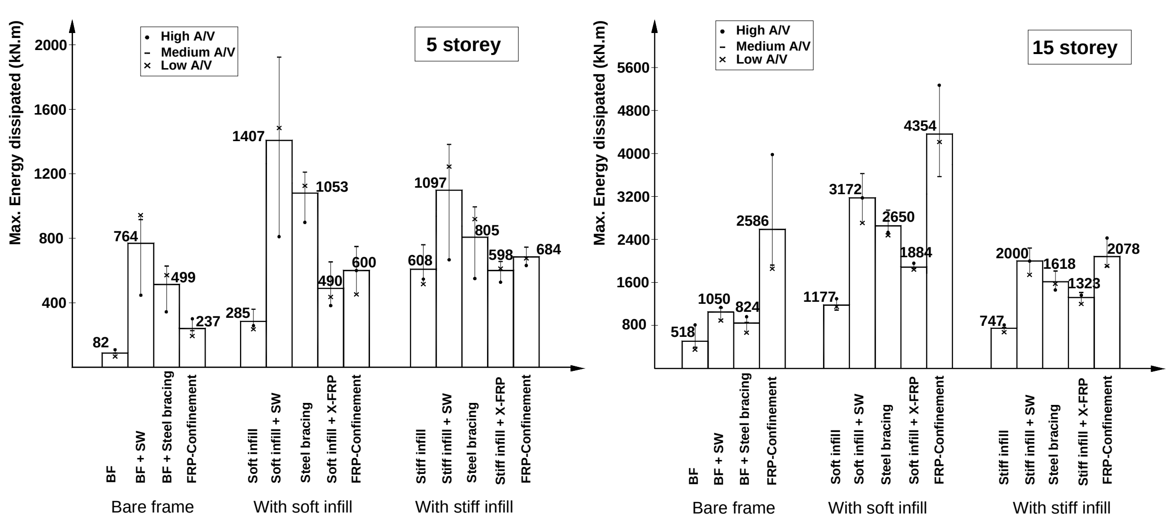


Figure 14 Maximum energy dissipated by low- and high-rise frames.

5.5 Comparison between the behaviour of FRP confinement rehabilitation scheme and other schemes

The nonlinear dynamic analyses conducted on the selected frames rehabilitated using FRP-confinements showed that this rehabilitation scheme is very efficient in increasing the maximum PGA, I.D. and energy dissipation capacities, especially for medium- and high-rise frames. From Figure 12, we noticed that the FRP-confinement scheme did not reduce the value of I.D. at the same PGA level but it increased the PGA and I.D. capacities, and this was attributed to the fact that FRP-confinement does not contribute to the structure stiffness like other studied rehabilitation schemes do, but it increases the ductility of the rehabilitated structure. This led to the importance of investigating the behaviour of the studied rehabilitation schemes when two frames only of the studied building (e.g. the external frames) were rehabilitated using a particular scheme and the other frames remain without rehabilitation. In this study each 15-storey rehabilitated frame model (i.e. frame with RC wall, with steel X-bracings, with X-FRP bracings, and with FRP-confinements) with soft infill was added to the existing frame model. The two frames were connected using rigid links at the floor levels representing the RC slabs. The frames' models were subjected to the nine ground motion records selected for this study and the average values were calculated for each model.

Table 2 shows the results of the nonlinear dynamic analyses for each individual frame and for the combined frames (existing + rehabilitated frame) for the high-rise building with soft infill. From the table, it can be seen that rehabilitating the external frames only using RC walls, steel X-bracing or X-FRP bracing improved the behaviour of the whole structure. On the other hand, rehabilitating the external frames using FRP-confinement did not result in a significant enhancement in the structure response as it is expected. This can be attributed to the fact that FRP-confinements increase the ductility capacity of the frame members without altering their strength significantly. In that case rehabilitating the external frames only of the structure will not be efficient as the global behaviour of the structure will be controlled by the non-ductile (unrehabilitated) frames, and the FRP-confined frames will not reach their ultimate ductility capacity (i.e. the maximum I.D. capacity of the existing frame is much smaller than that of the FRP-confined frames). Consequently, in case of FRP-confinement rehabilitation pattern, all the structure’s existing frames should be rehabilitated in order to achieve the ductility level required for the seismic enhancement of the structure. On the other hand, rehabilitating the external frames only using RC structural wall, steel X-bracings, or X-FRP bracings was found to be efficient, as the maximum I.D. capacity of the existing frame is higher or equal to that of the rehabilitated frames, and hence the rehabilitated frames were able to reach their maximum capacities when they are connected to the existing frames.

Table 2 Behaviour of the individual and combined 15-storey frames with soft infill.

Analysis results	Individual systems					Combined systems			
	Existing	RC Wall	Steel bracing	X-FRP bracing	FRP confinement	Existing +RC Wall	Existing + Steel bracing	Existing + X-FRP bracing	Existing + FRP confinement
PGA (g)	0.94	1.19	1.15	1.14	2.18	1.11	1.25	1.03	0.96
I.D. (%)	1.16	0.99	1.04	1.21	4.42	0.98	1.06	1.18	1.16
Storey shear/weight	0.11	0.40	0.28	0.15	0.15	0.28	0.22	0.13	0.12
Energy dissipated (kN.m)	1180	3170	2670	1880	4350	5020	5010	3000	2520

CONCLUSIONS

The effectiveness of different rehabilitation patterns in upgrading the seismic performance of existing non-ductile RC frame structures was evaluated. Two RC frames with different heights were selected for the analyses representing low- and high-rise frames. The frames were rehabilitated using four techniques; the first is by introducing a RC wall, the second is by using X-steel bracings, the third is by using X-FRP strips (FRP bracings) in case of masonry-infilled frames, and the fourth technique is by rehabilitating the structural members using FRP confinements. Two different masonry infill types with different stiffness (soft and stiff infill) were considered in the analyses. The bare frames ignoring the effect of masonry infill were also studied. The ground motion records were selected to represent earthquakes with low, medium and high frequency contents. The seismic performance enhancement of the analyzed frames was evaluated based on the maximum applied peak ground acceleration resisted by the frames, maximum inter-storey drift ratio, maximum storey shear-to-total weight ratio and energy dissipation capacity. The importance of accounting for the masonry infill on the seismic behaviour of structures was also investigated.

The conducted analyses have resulted in the following conclusions:

1- For the studied low-rise frame, introducing a RC wall increased the PGA, storey shear, and energy dissipation capacities while rehabilitating the columns and beams of the structure using FRP confinement increased the I.D. ratio capacity.

2- For the studied medium- and high-rise frames, introducing a RC wall increased the storey shear demand, while rehabilitating the columns and beams of the structure using FRP-confinement increased the PGA, I.D., and energy dissipation capacities.

3- For the case of FRP-confinement rehabilitation technique, all the structure's existing frames should be rehabilitated in order to achieve the ductility level required for the seismic enhancement of the structure. On the other hand, for the other rehabilitation techniques, rehabilitating a limited number of the structure's frames was found to be efficient. The number of frames needed to be rehabilitated can be determined according to the seismic enhancement needed for the structure.

4- Accounting for the presence of masonry infill in the analytical model has decreased the maximum I.D. ratio and increased the energy dissipation capacity of the frames. Hence, ignoring the effect of masonry infill would lead to under-estimation of the seismic performance of structures.

ACKNOWLEDGEMENTS

The financial support of Le fonds Québécois de la recherche sur la nature et les technologies (FQRNT) through the team research project program and the Natural Science and Engineering Research Council (NSERC) of Canada are greatly appreciated.

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