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EFFECT OF FRP–CONFINEMENT REHABILITATAION PATTERN ON THE SEISMIC PERFORMANCE OF NOMINALLY DUCTILE EXISTING RC FRAME STRUCTUES

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ABSTRACT

Many reinforced concrete (RC) frame structures designed according to older strength-based codes are susceptible to abrupt non-ductile strength deterioration once the shear capacity of the columns and/or beams is reached. Recently, performance-based seismic design methodology is being adopted by several codes. Ductility of main structural elements past initial steel bar yielding and inter-storey drift capacity became the target for good design. This approach is expected to decrease the probability of failure of the structure, and increase its energy dissipating capacity, when subjected to the design ground motions.

Fibre composites are used to increase the shear strength of existing RC columns and beams by wrapping or partially wrapping the members. Additional shear strength contribution is introduced by orienting the fibres normal to the axis of the member or to cross potential shear cracks. Increasing the shear strength can alter the failure mode to be more ductile with higher inter-storey drift ratio capacity. Shear strengthening using external confinement with fibre-reinforced polymer (FRP) may be provided at locations of expected plastic hinges by completely wrapping the column section or using FRP U-wraps near the beams ends.

The objective of this study is to analytically evaluate the effect of FRP-confinement rehabilitation pattern on the seismic performance of nominally ductile existing RC frame structures. The study investigates the performance of three RC frames with different heights –representing existing buildings designed according to pre-1970 strength based codes– when rehabilitated using different patterns of FRP confinement and subjected to three types of scaled ground motion records. The heights of the RC frames represent low, medium, and high-rise buildings. The ground motion records represent earthquakes with low, medium, and high frequency contents. The effect of the selected element's force-displacement backbone curve on the maximum capacity of the structures was investigated as well. The analyses results and conclusions are drawn by evaluating the seismic performance enhancement in terms of strength-deformation, ductility, inter-storey drift, and energy dissipation capacities of the studied structures.

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Introduction

In the recent few decades, the world had suffered from increasing hazards of earthquakes. Some of these earthquakes were of severe intensities which caused the failure of many existing buildings or at least left severe damage behind. Many of the existing reinforced concrete (RC) frame structures designed according to pre-1970 codes were not able to survive such ground motions as they were designed upon strength-based concept which didn't enforce ductility limits and energy dissipation capacity of the structure. The lack of appropriate reinforcement detailing of the frame columns, beams and joints lead to low shear capacity of the beams and columns and hence non-ductile strength deterioration 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, strengthening such structures is essential and can not be neglected.

Fibre-reinforced polymer (FRP) composite materials have received an increasing attention in the past decades as a potential method for rehabilitation or strengthening of existing structures due to their tolerable characteristics and ease of application. Fibre composites are used to increase the shear strength of existing RC beams and columns by wrapping or partially wrapping the members. Additional shear strength contribution is introduced by orienting the fibres normal to the axis of the member or to cross potential shear cracks. The wrapping pattern and the number of FRP layers used in the strengthening determine the additional strength and ductility of the members, and hence the ductility of the structure and its overall response when subjected to a specified seismic hazard level.

Performance-based (PB) seismic engineering is the modern approach to earthquake resistant design. 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 273/274, 1997 and Vision 2000, 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 drifts (as a damage parameter) to be: <0.2%, <0.5%, <1.5%, <2.5%, and >2.5%, respectively. The hazard level can be represented by the probability of exceedence of 50%, 10%, 2% in 50 years for low, medium and high intensities of ground motions, respectively. Fig. 1 shows the typical seismic performance of existing non-ductile structures versus structures designed according to performance-based seismic engineering. From the schematic it can be seen that upgrading the seismic performance of existing non-ductile structures can be achieved by increasing the capacity of the structure with or without reducing its drift. The former can be achieved by increasing the stiffness of the building, e.g. by using RC walls or FRP bracings. The later can be achieved by increasing the ductility capacity of the structural elements of the building without altering their stiffness, e.g. by using FRP confinement.

The objective of this study is to analytically evaluate the effect of FRP-confinement rehabilitation pattern on the seismic performance of nominally ductile existing RC frame structures. The study investigates the behaviour of three RC frames with different heights –representing existing buildings designed according to pre-1970 strength based codes– when rehabilitated using different schemes of FRP confinement and subjected to three types of scaled ground motion records. The heights of the RC frames represent low, medium, and high-rise buildings. The ground motion records represent earthquakes with different frequency contents. The studied FRP-confinement rehabilitation patterns include rehabilitating the columns and beams along the full height and lower half of the structure height. The effect of the selected members to be rehabilitated (i.e. columns or beams) is also investigated. The analyses results and conclusions are drawn by evaluating the seismic performance enhancement in terms of strength-deformation, ductility, inter-storey drift, and energy dissipation capacities of the studied structures.

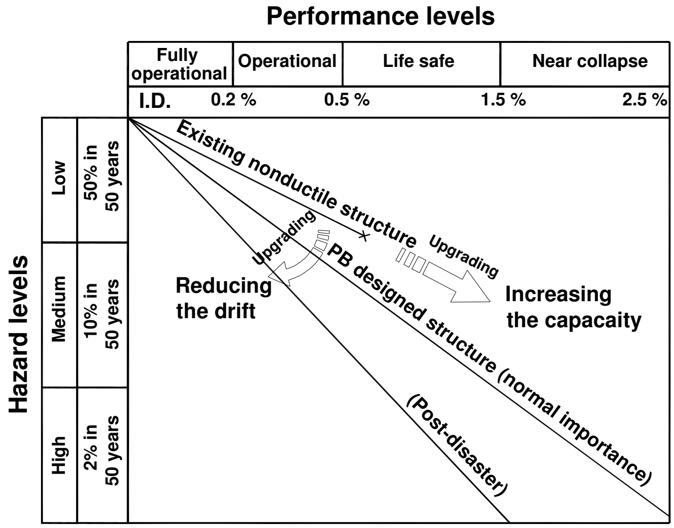

Figure 1. Seismic performance of existing non-ductile structures and possible ways of upgrading.

Properties of the Selected Ground Motions

Tso et al. (1992) examined the significance of A/V ratio as a parameter to indicate the dynamic characteristics of earthquakes. 3 sets of strong ground motion records were analyzed with low, medium and high A/V ratio. It was found that A/V ratio can be used as a simple indicator for the frequency content of the ground motion. In this study, a set of 9 far-field earthquake records was selected for the analysis (Tso et al. 1992 and PEER 2006). The ground motion records represent earthquakes with low, medium and high frequency contents. The properties of selected ground motions are summarized in Table 1.

Table 1. Properties of the selected ground motions (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

Properties of the Selected Buildings

Three buildings designed according to pre-1970 strength based code (ACI 1968) are selected for this study. The buildings have five, ten and fifteen storeys to represent low, medium and high-rise buildings, respectively. The three buildings have the same floor plan that consists of three symmetrical bays in both directions, where the bay width is 6 m. The floors are designed to carry their own weight and live load of 2 kPa, the floor height is 3.25 m and the total heights of the three buildings are 16.25, 32.5 and 48.75 m, respectively. The elevations of the three 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 varying 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.

Existing Structures and Different Rehabilitation Schemes

The performance of the three existing frames is evaluated when rehabilitated using FRP composites. In this study, two FRP contents and four different rehabilitation patterns were investigated. The columns were rehabilitated by wrapping them laterally with FRP composites, while the beams were rehabilitated using FRP U-wraps near their ends. The studied patterns include rehabilitation of both columns and beams (1) along the full height and (2) in the lower half of the structure height; and rehabilitation of columns only (3) along the full height and (4) in the lower half of the structure height. The lateral force-displacement ductility relationship of the columns and beams of the existing buildings is assumed to have limited displacement ductility, μ, equal to 2, which is followed by a quick reduction in the lateral load resistance. The two FRP contents used in the rehabilitation of the structural elements will increase their energy dissipation capacities and displacement ductility to be moderately and highly ductile with μ=4 and μ=6, respectively. The force-displacement ductility backbone curves for the existing, moderately ductile and highly ductile frames are shown in Fig. 3.

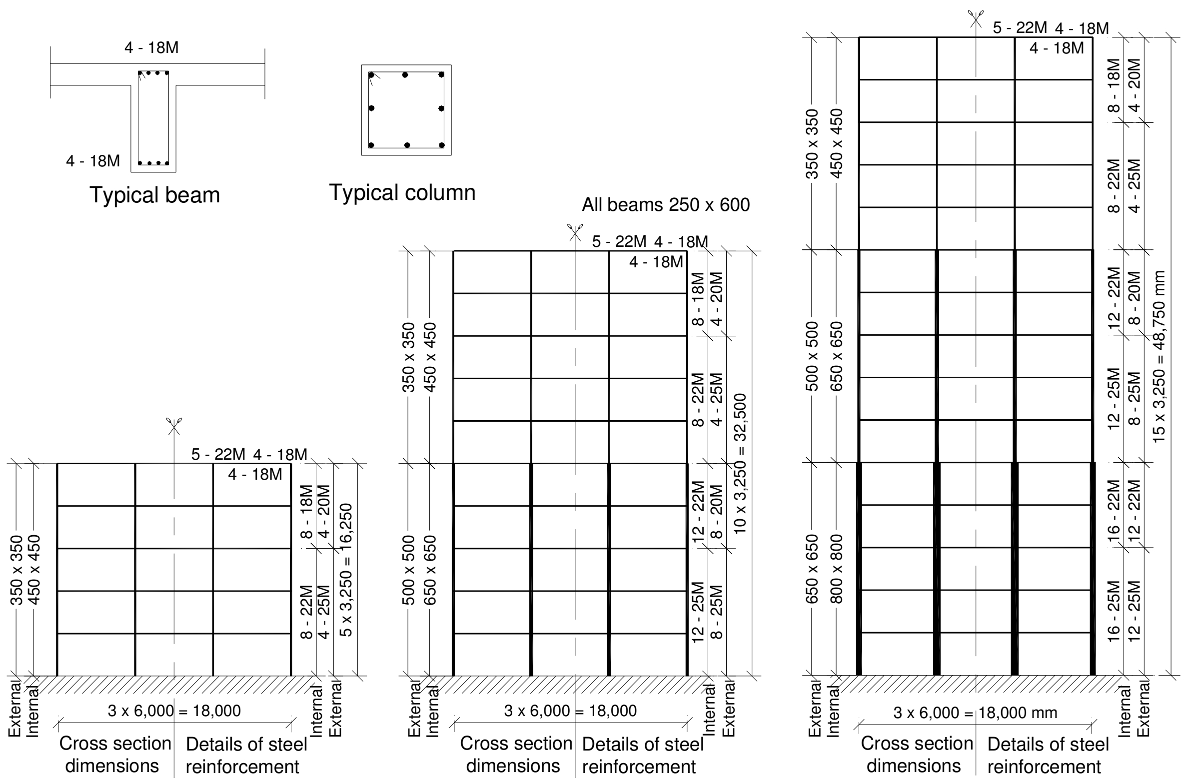

Figure 2. Elevations of 5, 10 and 15 storey studied frames.

The element's force-displacement ductility relationship is a combination of flexure and shear resistance. Ignoring the representation of the strength degradation in the overall response of the element once its shear capacity is reached would lead to erroneous response predictions. On the other hand, modeling the structural element using two independent non-linear springs representing flexure and shear hysteretic responses would result in an uncontrolled displacement ductility capacities of the element, which is a major parameter in the performance-based seismic design approach. Added to that is the difficulty in defining the hysteretic properties of the independent shear subhinge, and ignoring the flexure-shear interaction.

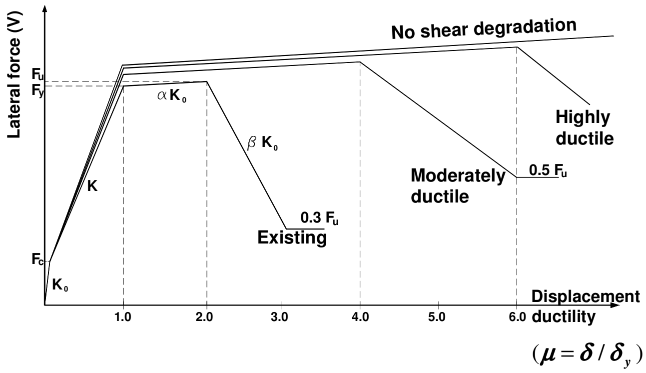

Figure 3. Lateral force-displacement ductility relationships of the existing, moderately ductile and highly ductile models.

Galal (2007) proposed a method to predict the backbone lateral force-displacement ductility relationship for non-ductile RC squat columns rehabilitated with FRP confinement. The model considered both the flexure and shear capacity of the column. The model considered that stiffness, yielding displacement and yielding capacity of columns didn't change after rehabilitation, while the ductility of the rehabilitated members increased significantly. Galal *et al.* (2005) studied the performance enhancement of RC columns when strengthened using glass or carbon composite materials, and they evaluated the effect of number of FRP layers on the behaviour of columns. Haroun *et al.* (2002) had experimental and analytical studies on half-scale bridge rectangular RC columns retrofitted using FRP jackets. They conclude that the as-build (existing) columns failed at displacement ductility equal to 2 due to bond deterioration of the lap-spliced longitudinal reinforcement. On the contrary, the jacketed columns were able to reach displacement ductility greater than 6.

In this study, strength degradation is considered in the element's lateral force-displacement ductility backbone curve which corresponds to the onset of shear failure. The columns and beams of the existing, the moderately ductile, and the highly ductile structure are assumed to reach displacement ductility of 2.0, 4.0 and 6.0, respectively. The post-peak degradation in strength occurs through a displacement equivalent to one and two times the yield displacement up to a residual force of 0.3 and 0.5 of the ultimate load capacity for existing and both ductile structures, respectively, as shown in Fig. 3.

Nonlinear Model Used in the Time History Analyses

Non-linear dynamic analyses for the three buildings with different rehabilitation schemes are conducted. A computer software for three dimensional nonlinear and dynamic structural analysis CANNY (Li 2006) is selected for the analysis. The model chosen to represent the force-displacement backbone curve for the members is the deterioration model CP4 (Li 2006), 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. The hysteretic behaviour of the model is shown in Fig. 4. The main input data for this model is the element's force-displacement backbone curve. The moment-curvature relationship was drawn for every element in the existing structure and consequently the input curves for the FRP-rehabilitated moderately ductile and highly ductile were estimated as a multiplier of the existing structure curve.

Table 2 shows the relative properties of the structural element's force-displacement backbone curves for the two rehabilitation schemes considered in the analysis as related to the existing structure properties. The beams and columns are modeled as linear elastic element with two inelastic single-component flexure-shear rotation springs located at the ends of the member. CANNY deterioration model CP4 is used to model the nonlinear flexure-shear rotation spring. The mass of each floor is lumped at the column joints according to the tributary areas. The frame joints are assumed to be rigid and rigid zones are applied at the ends of each member. P-delta effects are included in the analyses.

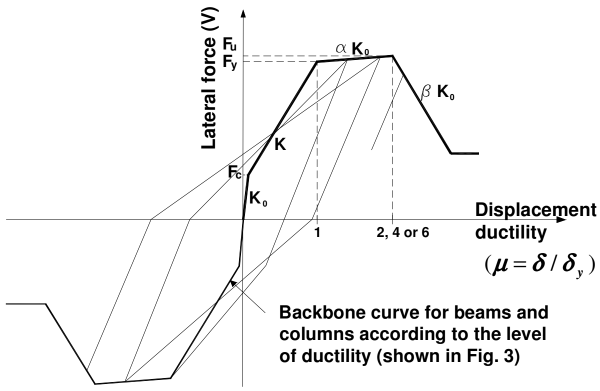

Figure 4. Hysteretic behaviour of the model CP4 (CANNY 2006).

Table 2. Properties of the FRP rehabilitated structural elements as compared to those of the existing structure.

Model properties	Existing structure	Moderately ductile	Highly ductile
Initial stiffness	K	1.1 K	1.15 K
Yield Capacity	Fy	1.1 Fy	1.15 Fy
Displ. Ductility Capacity	2.0	4.0	6.0

Table 3. Natural periods for the studied buildings (sec).

No. of storeys	Mode 1	Mode 2	Mode 3
5	1.01	0.32	0.18
10	1.74	0.59	0.33
15	2.49	0.86	0.47

Modal analysis is conducted for the three buildings, and 5% damping ratio was considered. Table 3 shows the natural periods for the 5, 10 and 15 storey buildings. The period of the existing structures and the rehabilitated ones have the same period. This is due to the fact that the FRP confinement increases the ductility of the rehabilitated elements with a negligible influence on their stiffnesses.

Analyses Results

a- Maximum applied PGA

9 earthquake records are applied on the three frames with different rehabilitation schemes. The records represent a sample of earthquakes with low, medium and high frequency contents. In the analyses, the maximum earthquake intensity that can be applied on the three structures was evaluated for different rehabilitation schemes. Fig. 5 shows the maximum PGA that can be resisted by low and high-rise frames.

From the figure it can be seen that, for low-rise building, the difference between rehabilitation scheme of the columns and beams (full height) and rehabilitation of columns only (full height) is not significant, which implies that rehabilitation of columns only will be effective in case of low-rise structure. On the other hand, the figure shows that, for high-rise building, rehabilitation of columns only was not as effective as rehabilitation of both columns and beams for the full height of the structure. The aforementioned results can be attributed to the mode of failure in the studied low- and high-rise frames. For the low-rise building, the failure was observed to occur in the columns, while the failure occurred in the beams for the high-rise building. This observation occurred for both FRP-rehabilitation patterns: columns only, and columns and beams. Therefore, it can be stated that the insignificant gain in PGA capacity for nominally ductile low-rise building when rehabilitating the beams (in addition to the columns) is due to the high relative strength of the beams to that of the columns of low-rise building (both designed to carry gravity loads). Consequently, as the relative strength of the beams to that of the columns of high-rise building reduces, the gain in the PGA capacity of the building increases with the rehabilitation of the beams. Also, it can be seen from the figure that the rehabilitation of the lower half (for the cases of rehabilitating both columns and beams) is not effective in increasing the PGA capacity for both low- and high-rise buildings. This can be attributed to the sudden change in the ductility capacity of the structural elements at the mid height of the building.

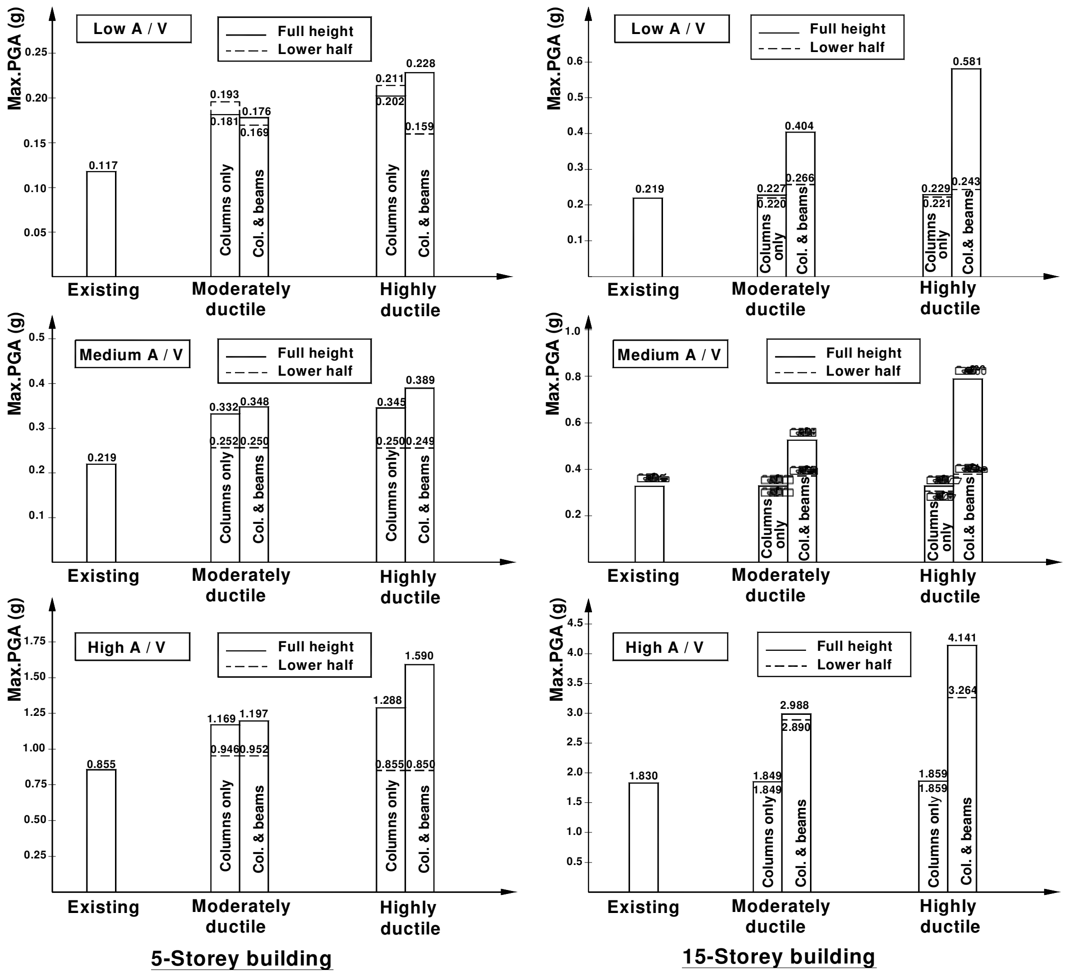

Figure 5. Maximum PGA resisted by low- and high-rise buildings.

b- Maximum inter-storey drift

Fig. 6 shows the maximum inter-storey drift (I.D.) ratio for the 5 and 15 storey buildings for the case of rehabilitating the columns and beams and the case of rehabilitating the columns only (full height or lower half) for the existing structure to be moderately ductile or highly ductile, when subjected to ground motions with different frequency contents (low, medium and high). For both frames, it can be noticed that the max. I.D. value increases with the increase of FRP content, which leads to a more ductile structure with higher inter-storey drift capacity. The figure shows that for low rise building, the difference in max. I.D. between rehabilitation scheme of the whole structure (columns and beams) and rehabilitation of columns only is not significant. On the other hand, rehabilitation of columns only in case of high-rise buildings did not result in a significant increase in the max. I.D. capacity compared to the case of rehabilitation of the whole structure. The same response was observed for the maximum PGA capacities for both cases, as mentioned earlier. From the figure, it can be noticed also that for a certain height, and dynamic and hysteretic properties of the building, the maximum I.D. capacity almost didn't change with respect to the change in the earthquake properties. This was not the case for the maximum PGA capacity of the structures which was affected by the frequency content of the ground motion record as shown in Fig. 5. This justifies the validity of using the maximum inter-storey drift as a uniform and reliable damage parameter that can be used to judge the performance of FRP-rehabilitated structures. Performance levels that represent the damage degree of structure in terms of inter-storey drifts as recommended by Vision 2000 (1995) and FEMA 273/274 (1997) are depicted in Fig. 6.

Fig. 7 shows the I.D. distribution along the height for 5 and 15 storey frames with different rehabilitation schemes when subjected to the maximum scaled El-Centro earthquake record that the building can resist. The figure shows that the maximum I.D. occurs at the first floor level for the 5 storey frame, indicating that the first mode of vibration governed the response of the low-rise building. On the other hand, for the high rise building, the maximum I.D. occurred about one-third of the height of the building. This can be attributed to the effects of higher modes.

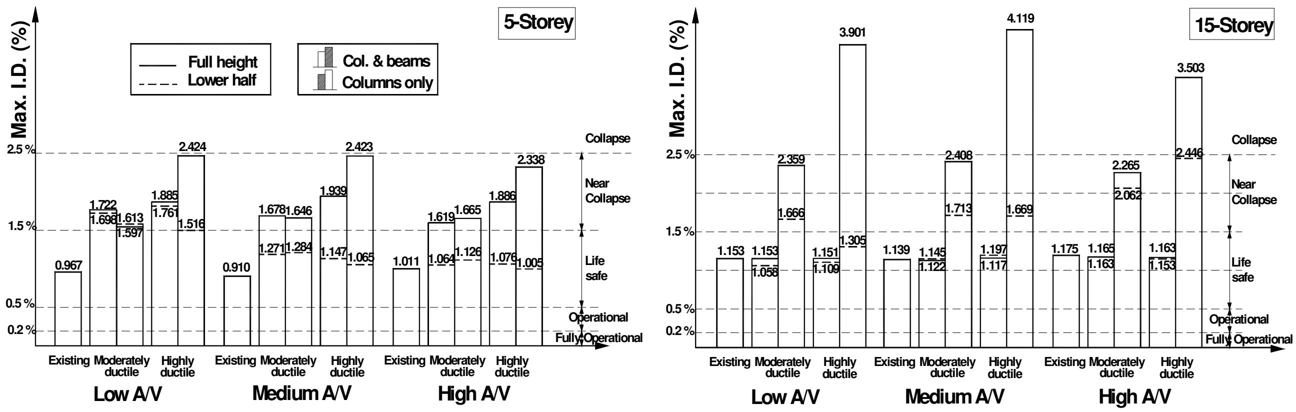

Figure 6. Maximum inter-storey drift ratio capacity for low- and high-rise buildings.

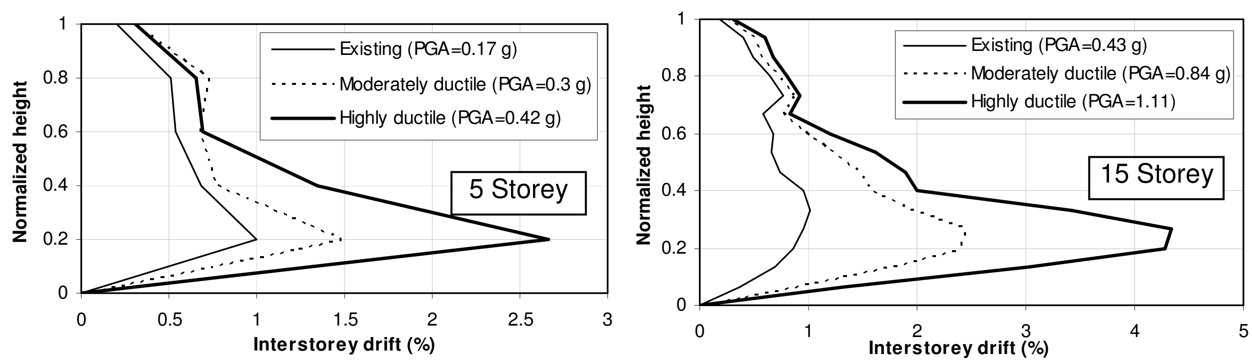

Figure 7. I.D. distribution along the height of 5 and 15 storey buildings subjected to the maximum scaled El Centro earthquake record that the building can resist.

c- Maximum storey shear

Fig. 8 shows the maximum storey shear-to-the total weight of structure ratio versus the maximum I.D. relationship (average capacity for the nine earthquakes) for both 5 and 15 storey buildings. It can be seen that the maximum storey shear demand increased when the structures were rehabilitated using FRP confinement. The effect of FRP rehabilitation in increasing the storey shear capacity is more significant for the high-rise building compared to the low-rise building. The result of the analysis showed that the value of the maximum storey shear for the 5 storey building is not influenced by the change of ground motion frequency content. On the other hand, the maximum storey shear value for the 15 storey building is affected by the frequency content of the earthquake record used.

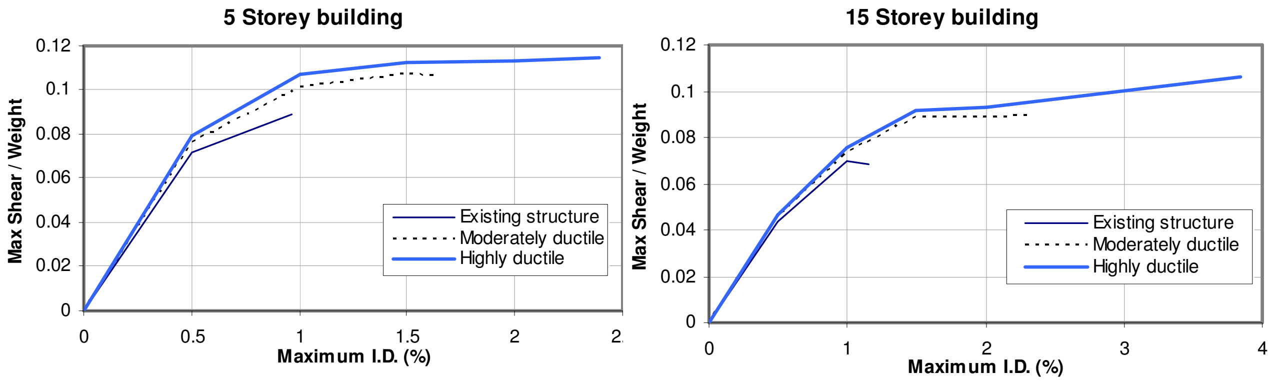

Figure 8. Maximum storey shear / weight with maximum I.D.

d- Energy dissipation capacity

Fig. 9 shows the maximum energy dissipated by both low- and high-rise frames. It can be seen that the energy dissipation capacity of the frames increased when they were rehabilitated using FRP confinement. The figure shows that rehabilitation of columns only (full height) was effective in case of low-rise structure, while it wasn't as effective as rehabilitation of both columns and beams for the full height of high-rise frames. This was observed previously from the results of maximum PGA capacities of the structures.

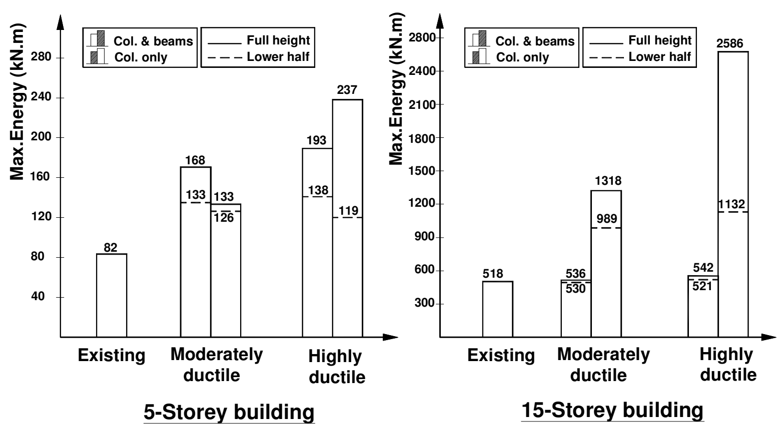

Figure 9. Maximum energy dissipated for low- and high-rise buildings.

Importance of accounting for strength degradation in the nonlinear model of columns and beams

The hysteretic response of RC members must experience a post-peak degradation, even when rehabilitated using FRP for a highly ductile targeted performance. This degradation in strength can arise from several mechanisms, such as: reaching the shear capacity, P-Δ effect, crushing of concrete in compression, buckling of compression rebars, or sudden tear of the FRP confinement. Ignoring the strength degradation beyond the peak capacity of the element would lead to erroneous response predictions of the structure performance. Fig. 10 shows the effect of inclusion of strength degradation in the analysis.

The figure compares the maximum PGA resisted by the highly ductile frames of 5 and 15 storey buildings and the maximum I.D. ratio in two cases; the first case when the element strength degradation is included in the analysis, while the second case does not account for the strength degradation. In the later case, the failure of the structure was assumed to occur when the structure reached a roof displacement equal to 5% of the structure height. The figure shows a major difference in the predicted results in both cases, which emphasizes on the significance of accounting for the strength degradation on the predicted performance of structures.

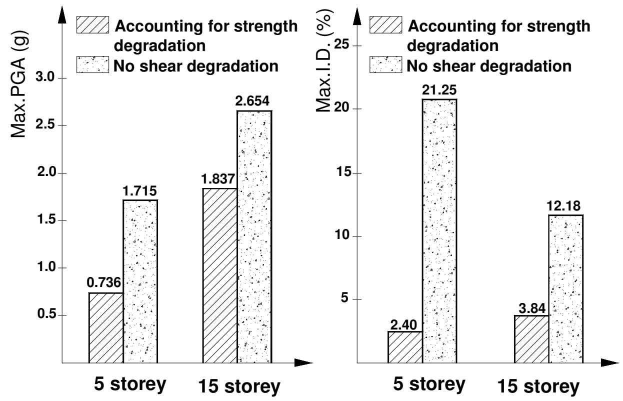

Figure 10. Accounting for strength degradation.

Rehabilitation of Damaged Members Only Versus the Selected Rehabilitation Schemes

In this study, the efficiency of rehabilitation of selected damaged members of the structures when subjected to a specific earthquake was examined. The 5- and 15-storey existing frames (case 1) are subjected to El Centro earthquake ground motion of maximum peak ground acceleration of 0.15 and 0.3g, respectively.

Table 4. Max. PGA capacities for the considered cases.

No. of storeys	Case 1	Case 2	Case 3
5	0.17 g	0.23 g	0.42 g
15	0.42 g	0.43 g	1.11 g

For the 5-storey building, four plastic hinges occurred at the base of the first floor columns, while for the 15-storey frame, the plastic hinges occurred at the end of the second span beams at the 3rd, 4th, 5th and 6th floors. These location were selected for rehabilitation with FRP composites for both structures, the PGA capacities are calculated for both rehabilitated frames (case 2) and are compared to the PGA capacities of the full rehabilitation pattern of beams and columns (case 3). Table 4 shows the PGA capacities for the three considered cases. From the table, it can be noticed that, the rehabilitation of the damaged members only was not effective in enhancing the seismic performance of the structures, and this ensure the importance of having a certain rehabilitation scheme (beams and columns, columns only, full height, lower half, etc).

Conclusions

The effect of FRP-confinement rehabilitation patterns on the seismic performance of existing RC frame structures was evaluated. Nonlinear dynamic analysis of three existing frames with different heights has been conducted. The frames were rehabilitated with different patterns and different FRP contents. The analyses were conducted on the existing frame, moderately ductile and highly ductile ones. Strength degradation was considered in the element's force-displacement ductility backbone curve to represent the shear failure. The existing, moderately ductile and highly ductile frames were modeled to have displacement ductility capacities of 2.0, 4.0 and 6.0, respectively. The selected ground motion records were selected to represent earthquakes with low, medium and high frequency contents. For the analyzed frames subjected to the selected ground motion records, the maximum applied PGA and PGV resisted by the frame, maximum inter-storey drift, maximum storey shear demands, and maximum energy dissipation capacity were evaluated. The importance of accounting for the strength degradation in the hysteretic model was also studied.

Based on the conducted analyses, the following conclusions were reached:

1 – For low-rise RC frame structures, FRP-rehabilitation of the columns only was effective in increasing the structures′ PGA and maximum interstorey drift demands. On the other hand, FRP-rehabilitation of all the columns and beams of the structure did not result in a significant increase in the seismic performance of the rehabilitated structure compared to the case of rehabilitating the columns only. This can be interpreted that, for low-rise buildings, the relative strength of the beams to that of the columns is high, which resulted in failure in the columns, so the gain in the PGA capacity of the building with the rehabilitation of the beams (in addition to the columns) is not significant.

2 – For high-rise RC frame structures, the rehabilitation of columns only was not as effective as rehabilitation of both columns and beams for the full height of the structure. This can be interpreted that, for high-rise buildings, the relative strength of the beams to that of the columns is low, which resulted in failure in the beams, so the gain in the PGA capacity of the building increased with the rehabilitation of the beams in addition to columns.

3 – The FRP-rehabilitation of the lower half of the structure was found to be not efficient in both low- and high-rise buildings.

4 – The maximum interstorey drift ratio capacity increases with the increase of FRP content, which leads to a more ductile structure.

5 – For a certain height and dynamic properties of the FRP-rehabilitated buildings, the maximum interstorey drift capacity is slightly influenced by the earthquake properties compared to the PGA capacity. This justifies the validity of using the maximum inter-storey drift as a uniform and reliable damage parameter that can be used to judge the performance of FRP-rehabilitated structures.

6 – Ignoring representing the post-peak strength degradation in the hysteretic nonlinear model of FRP-rehabilitated RC columns and beams would lead to erroneous over-estimation of the seismic performance of the structure.

It’s important to clarify that the drawn results are for the studied cases and the selected earthquakes. More earthquakes should be considered and more analysis and experimental work on existing, and rehabilitated, frame structures subjected to simulated ground motion excitations (e.g. Weber *et al.* 2004) is needed for the conclusions to be generalized.

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