

Analytical Evaluation of Seismic Performance of RC Frames Rehabilitated Using FRP for Increased Ductility of Members

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Abstract: Many reinforced concrete (RC) frame structures designed according to pre1970 strength-based codes are susceptible to abrupt strength deterioration once the shear capacity of the columns is reached. Fiber composites are used to increase the shear strength of existing RC columns and beams by wrapping or partially wrapping the members. Increasing the shear strength can alter the failure mode to be more ductile with higher energy dissipation and interstorey drift ratio capacities. The objective of this study was to analytically evaluate the effect of varying distributions of fiber-reinforced polymer (FRP) rehabilitation on the seismic performance of three existing RC frames with different heights when subjected to three types of scaled ground motion records. The FRP wrapping is designed to increase the displacement ductility of frame members to reach certain values representing moderate ductility and high ductility levels. These values were assumed based on previous experimental work conducted on members wrapped using FRP. The study also investigates the effect of the selected element's force-displacement backbone curve on the capacities of the structures with respect to maximum interstorey drift ratio, maximum peak ground acceleration, or peak ground velocity resisted by the frames, maximum storey shear-to-weight ratio and maximum energy dissipation. It was found that for low-rise buildings, the FRP rehabilitation of columns only was effective in enhancing the seismic performance; while for high-rise ones, rehabilitation of columns only was not as effective as rehabilitation of both columns and beams. Ignoring representing the postpeak strength degradation in the hysteretic nonlinear model of FRP-rehabilitated RC members was found to lead to erroneous overestimation of the seismic performance of the structure.

DOI: 10.1061/(ASCE)0887-3828(2008)22:5(276)

CE Database subject headings: Reinforced concrete; Fiber reinforced polymers; Frames; Ductility; Rehabilitation; Seismic effects; Dynamic analysis; Degradation.

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 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 cannot be neglected.

Fiber-reinforced polymer (FRP) composite materials have received increasing attention in the past decades as a potential

method for rehabilitation or strengthening of existing structures due to their high strength, light weight, and ease of application. Fiber 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 fibers 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 gain in strength and/or ductility of the members, and consequently, the ductility of the structure and its overall response when subjected to a specified seismic hazard level.

Performance-based (PB) seismic engineering is the recent 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 safety, near collapse, or collapse (FEMA 1997; SEAOC 1995). Overall lateral deflection, ductility demand, and interstorey drift (ID) are the most commonly used damage parameters. The five qualitative performance levels are related to corresponding five quantitative maximum interstorey 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, and 2% in 50 years for low, medium, and high intensities of ground motions, respectively. Fig. 1 shows the typical seismic performance of existing nominally ductile structures versus structures designed according to performance-based seismic engineering. From the schematic it can be seen that up-

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Note. Discussion open until March 1, 2009. Separate discussions must be submitted for individual papers. The manuscript for this paper was submitted for review and possible publication on March 8, 2007; approved on November 19, 2007. This paper is part of the *Journal of Performance of Constructed Facilities*, Vol. 22, No. 5, October 1, 2008. ©ASCE, ISSN 0887-3828/2008/5-276-288/\$25.00.

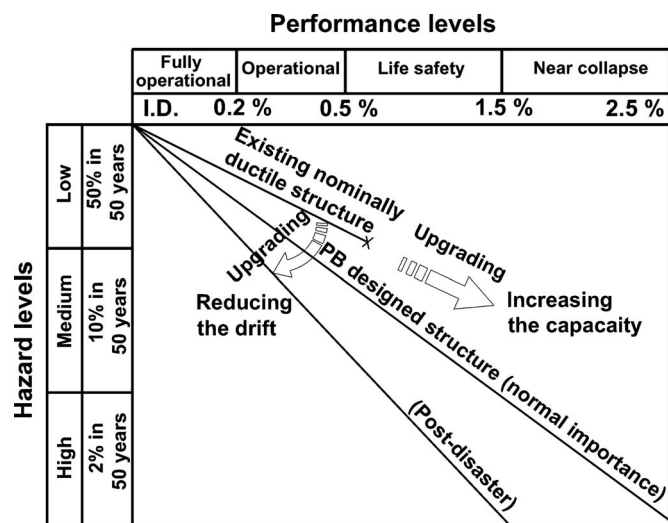


Fig. 1. Seismic performance of existing nominally ductile structures and possible ways of upgrading

grading 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 by reducing the drift can be achieved by increasing the stiffness of the building, e.g., by using RC walls or FRP bracings. Increasing the capacity without reducing the drift 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 was to evaluate analytically the effect of varying distributions of FRP rehabilitation on the seismic performance of nominally ductile existing RC frame structures. The study investigated the behavior of three RC frames with different heights—representing existing buildings designed according to pre1970 strength based codes—when rehabilitated using different distributions of FRP wrapping 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 wrapping rehabilitation patterns include rehabilitating the columns and beams along the full height and lower half of the structure height. The effect of rehabilitating selected members (i.e., certain columns or beams) versus rehabilitating a group of columns and/or beams was also investigated. The conclusions are drawn by evaluating the seismic performance enhancement in terms of the increase in the maximum peak ground acceleration (PGA) or peak ground velocity (PGV) resisted by the frames, maximum ID ratio, maximum story shear-to-weight ratio, and energy dissipation capacity of the studied structures.

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 summarized in Table 1.

Properties of the Selected Buildings

Three buildings designed according to the pre1970 strength-based code (ACI 1968) were selected for this study. The buildings had five, 10, and 15 stories to represent low-, medium-, and high-rise buildings, respectively. The three 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 three buildings were 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(a). 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. 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/m³ and concrete Poisson's ratio ν was taken to be 0.15. Fig. 2(b) shows the plan of the studied buildings.

Existing Structures and Different Rehabilitation Schemes

The performance of the three existing frames was evaluated when rehabilitated using FRP composites. Two FRP thicknesses were designed to increase the nominal ductility of the existing structural elements (i.e., beams and columns) to reach moderate and high ductility levels. The columns were rehabilitated by wrapping them laterally with FRP composites, while the beams were rehabilitated using FRP U-wraps near their ends. In this study, four different rehabilitation patterns were investigated. The studied patterns included 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. Fig. 3 shows the force-displacement ductility backbone curves for the existing, moderately ductile and highly ductile members. The lateral force-displacement ductility relationship of the columns and beams of the existing buildings was assumed to have limited displacement ductility, μ_Δ , equal to 2, which was followed by a quick reduction in the lateral load resistance, as shown in Fig. 3. The two FRP thicknesses used in the rehabilitation of the structural elements increased their energy dissipation capacities and displacement ductility to be moderately and highly ductile with $\mu_\Delta = 4$ and $\mu_\Delta = 6$, respectively. These values were assumed based on the experimental work done by Memon and Sheikh (2005), in which they found that the displacement ductility for the existing columns ranged between 1.3 and 3.7, while for the rehabilitated ones using glass FRP (GFRP), the displacement ductility ranged between 4.7 and 6.8. It is also worth noting that the product of the ductility and overstrength factors, $R_d \cdot R_o$ —defined by the National

Table 1. Properties of the Selected Ground Motions (Adapted from PEER 2006)

Number	Earthquake	Site	Date	Acceleration (g)	Velocity (m/s)	Acceleration/velocity [(g) (s/m)]		Duration (s)	Soil condition
						Level			
1	Lower California	El Centro	December 30, 1934	0.160	0.209	0.766	Low	90.36	Stiff soil
2	San Fernando, Calif.	2500 Wilshire Boulevard, Los Angeles	February 9, 1971	0.101	0.193	0.518	Low	25.32	Stiff soil
3	Long beach, Calif.	Los Angeles subway terminal	March 10, 1933	0.097	0.237	0.409	Low	99.00	Rock
4	San Fernando, Calif.	234 Figueroa Street, Los Angeles	February 9, 1971	0.200	0.167	1.198	Medium	47.10	Stiff soil
5	Kern County, Calif.	Taft Lincoln School Tunnel	July 21, 1952	0.179	0.177	1.011	Medium	54.42	Rock
6	Imperial Valley, Calif.	El Centro	May 18, 1940	0.348	0.334	1.042	Medium	53.76	Stiff soil
7	Lytle Creek, Calif.	6074 Park Drive, Wrightwood, Calif.	September 12, 1970	0.198	0.096	2.063	High	16.74	Rock
8	Parkfield, Calif.	Cholame, Shandon, Calif.	June 27, 1966	0.434	0.255	1.702	High	44.04	Rock
9	San Francisco	Golden Gate Park, Calif.	March 22, 1957	0.105	0.046	2.283	High	39.88	Rock

Building Code of Canada (NBCC 2005) to relate the linear elastic to nonlinear ductile responses—is equal to 2.6, 5.25, and 7.5 for the limited ductility, moderately ductile, and ductile *RC* moment resisting frames, respectively. From the above-mentioned values, the displacement ductility of existing, moderately ductile, and highly ductile members were chosen to be 2, 4, and 6, respectively. A sample of the data for the force–displacement ductility backbone curves of the ground floor columns and beams of the existing five-, 10-, and 15-story frames is shown in Table 2. Definitions of the input data are given in the Notation section.

Haroun et al. (2002) conducted experimental and analytical studies on half-scale bridge rectangular *RC* columns retrofitted using FRP jackets. They concluded that the as-built (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.

Galal (2007) proposed a method to predict the backbone lateral force–displacement ductility relationship for nonductile *RC* squat columns rehabilitated with FRP confinement. The model considered both the flexure and shear capacities of the column. The model considered that stiffness, yielding displacement, and yielding capacity of columns did not 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 the number of FRP layers on the behavior of columns.

Zou et al. (2007) presented an optimization technique for seismic design of *RC* frames rehabilitated using FRP confinements. The thickness of the FRP layers used in the confinement of columns was considered as a design variable. The study concluded that the FRP increased the ultimate capacity of the rehabilitated frame by about 9%, but it had an insignificant effect on the member stiffness, which is considered an advantage of using FRP as it will lead to lower seismic forces (demands). The study showed also that the backbone curves for the existing and rehabilitated frames were coincident before the cracking point.

Monti and Liotta (2007) conducted an experimental study on 24 beam specimens representing existing structures that were rehabilitated using different schemes of FRP. The rehabilitation schemes included side bonding, U-jacketing, and wrapping. An analytical study followed in which they proposed design equations for beams retrofitted using different patterns of FRP. The proposed equations were found to be in a good correlation with the experimental tests. The design equations were included in the Italian FRP-strengthening code (CNR 2005).

Table 3 shows the properties of FRP sheets used in the rehabilitation (Fyfe Fibwrap Company, <http://www.fyfeco.com/>). Table 4 shows the number of FRP layers that should be provided [in the case of glass or carbon FRP (CFRP)] to reach the targeted displacement ductility levels for the ground floor columns and beams of the 15-story building to be moderately ductile and highly ductile. The number of FRP layers and their thicknesses were designed according to the design equations proposed by Monti and Liotta (2007) for the beams, and by Galal (2007) for the columns. The Appendix shows a sample calculation of the number of FRP layers needed for wrapping the interior ground floor column of the 15-story frame and a typical beam.

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 er-

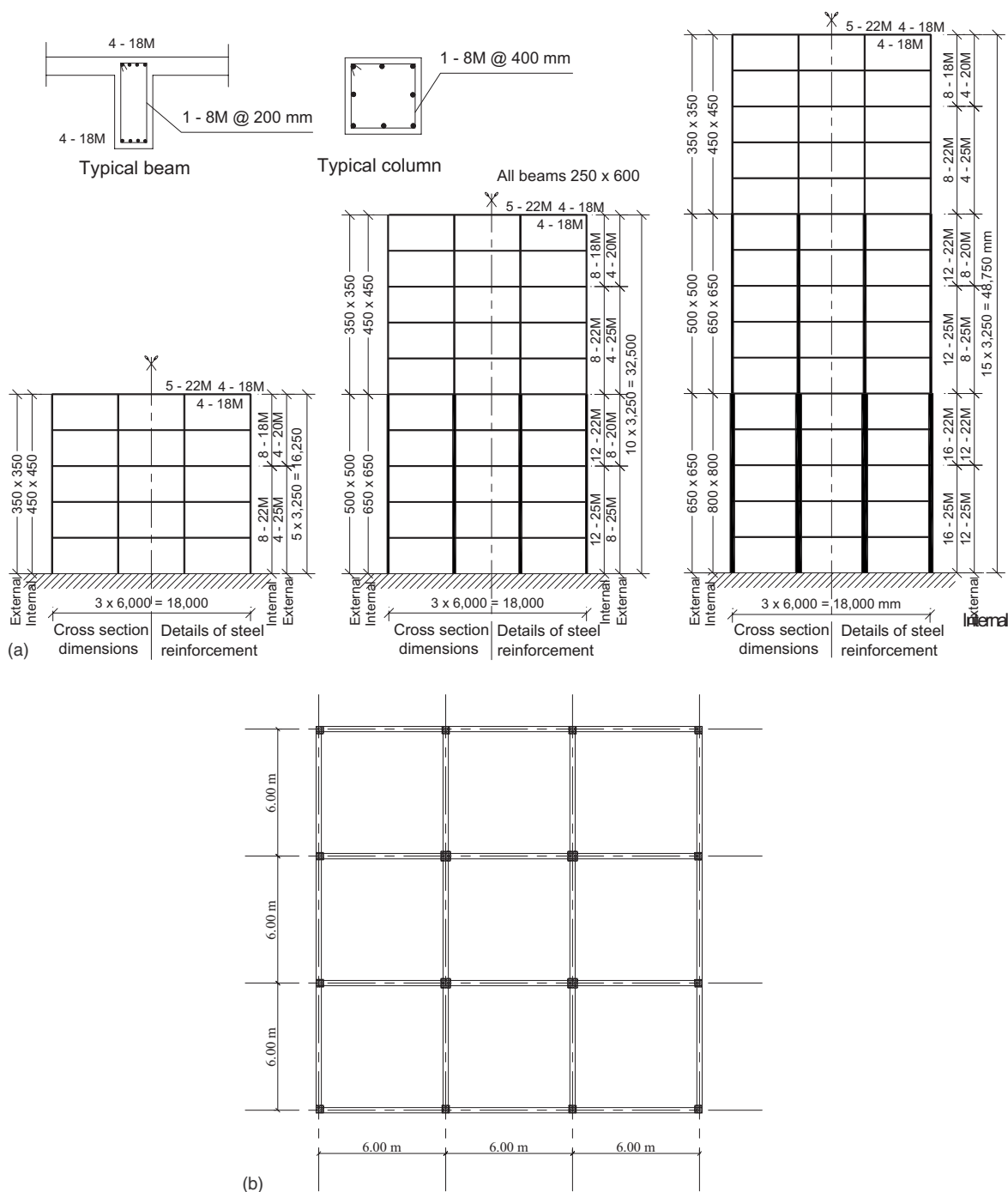


Fig. 2. (a) Elevations of the five-, 10-, and 15-story studied frames; (b) plan of the studied buildings

roneous response predictions. On the other hand, modeling the structural element using two independent nonlinear springs representing flexure and shear hysteretic responses would result in an uncontrolled displacement ductility capacity 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.

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 postpeak degradation in strength occurs through a displacement equivalent to one and two times the yield displacement up to a residual force

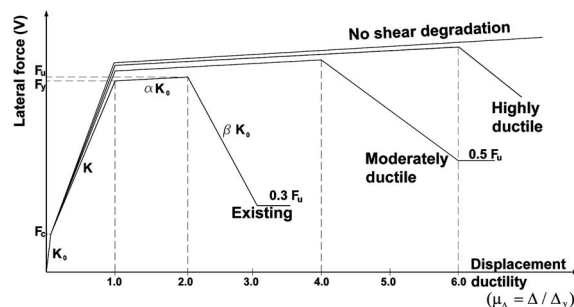


Fig. 3. Lateral force–displacement ductility relationships of the existing, moderately ductile, and highly ductile models

Table 2. Sample Data of the Lateral Force–Displacement Ductility Backbone Curves for Existing Columns and Beams

Building (number of stories)	Member	Location	F_c (kN)	F_y (kN)	Δ_{cr} (mm)	Δ_y (mm)	μ_Δ	K_o (kN)	K (kN)	α	β
5	Ground floor Columns	Internal	29.88	141.66	1.014	15.492	2	456.45	119.61	0.0063	−0.2375
		External	16.16	70.14	1.531	18.132	2	191.32	58.96	0.0146	−0.2859
10	Ground floor Columns	Internal	101.16	437.53	0.845	15.492	2	1,854.54	355.79	0.0050	−0.1807
		External	46.44	215.20	1.074	18.220	2	787.91	179.33	0.0080	−0.2108
15	Ground floor Columns	Internal	186.30	769.23	0.704	15.844	2	4,191.78	610.04	0.0016	−0.1388
		External	101.16	437.53	0.845	15.492	2	1,854.54	355.79	0.0050	−0.1807
All	Beams	External	18.75	73.33	2.940	32.400	2	206.63	60.03	0.0234	−0.2837

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 Time History Analyses

Nonlinear dynamic analyses for the three buildings with different rehabilitation schemes were conducted. A computer software for three-dimensional nonlinear and dynamic structural analysis, CANNY (Li 2006) was selected for the analysis. The model chosen to represent the force–displacement backbone curve for the members was Deterioration Model CP4 (Li 2006), which allows representation of the combined flexural and shear backbone curves with a parameter that controls the displacement ductility capacity after which the postpeak degradation occurs. The hysteretic behavior of the model is shown in Fig. 4. The main input data for this model were the element's force–displacement backbone curve. The force–displacement relationship was drawn for every element in the existing structure, and consequently, the input curves for moderately and highly ductile FRP-rehabilitated elements were estimated as a multiplier of the existing structure curve. Table 5 shows the relative properties of the rehabilitated structural element's (i.e., columns and beams) lateral force–displacement ductility relationship (shown in Fig. 3) for the two ductility levels considered in the analysis as related to those of the unrehabilitated elements in the existing structure. From Table 5

Table 3. Properties of the FRP Sheets to Be Used in the Rehabilitation^a

Property	Carbon FRP	Glass FRP
Ultimate tensile strength in primary fiber direction (MPa)	903	417
Elongation at break (%)	1.05	1.76
Tensile modulus (GPa)	86.9	20.9
Laminate thickness (mm)	0.28	0.635

^aAdapted from the Fyfe Fibwrap Company (<http://www.fyfec.com/>).

Table 4. Number of FRP Layers Needed for Wrapping of the Ground Floor Columns and Beams of the 15-Story Building to Reach the Targeted Ductility Levels

Element type	Location	Ductility level	Carbon FRP	Glass FRP
Column	Ground floor (interior)	Moderate	1	2
		High	3	5
Beam	All floors (exterior)	Moderate	2	3
		High	3	4

we can see that FRP wrapping of beams and columns was assumed to increase the postcracking force–displacement ductility slope, K , and yielding capacity, F_y , by 10% for the moderate ductility level and 15% for the high ductility level, respectively. These values were assumed based on the experimental work done by Memon and Sheikh (2005) in which they noticed that wrapping columns with GFRP sheets increased their lateral load capacity by 11–25% compared to the unrehabilitated columns. The insignificant increase in stiffness and strength of the structural members when wrapped with FRP sheets perpendicular to the member's longitudinal axis was due to the fact that FRP wrapping increases the shear capacity, and hence, increases the ductility capacity of the member without having a significant contribution to the member's stiffness or strength. On the other hand, application of FRP layers in the longitudinal direction of the member would increase the member's stiffness and strength significantly; such a rehabilitation pattern was not considered in this study.

The beams and columns were modeled as linear elastic elements with two inelastic single-component flexure–shear rotation springs located at the ends of the member. CANNY Deterioration Model CP4 was used to model the nonlinear flexure–shear rotation spring. The mass of each floor was lumped at the column joints according to the tributary areas. The frame joints are assumed to be rigid and rigid zones were applied at the ends of each member. P -delta effects were included in the analyses.

Modal analysis was conducted for the three buildings, and a 5% damping ratio was considered. Table 6 shows the natural periods for the five-, 10-, and 15-story buildings. For the same building height, the existing structure and the rehabilitated ones

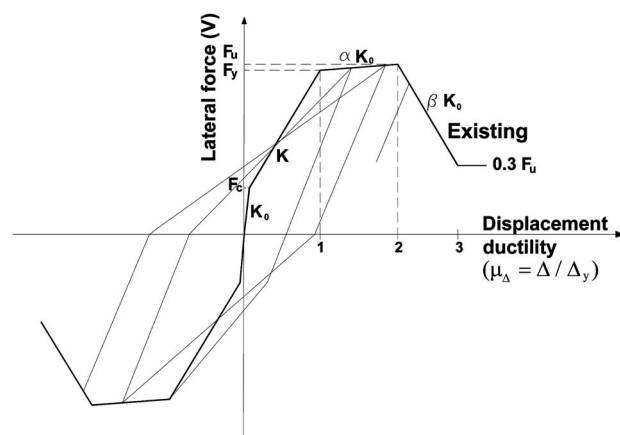
**Fig. 4.** Hysteretic behavior of Model CP4 (Li 2006)

Table 5. Properties of the FRP-Rehabilitated Structural Elements as Compared to Those of the Existing Structure

Model properties	Existing structure	Moderately ductile	Highly ductile
Postcracking force–displacement ductility slope	K	$1.1K$	$1.15K$
Yield capacity	F_y	$1.1F_y$	$1.15F_y$
Displacement ductility capacity	2.0	4.0	6.0

have the same period. This is due to the fact that the initial stiffness of the member before cracking is the same for the existing and rehabilitated members.

Analyses Results

Maximum Applied PGA

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

From Fig. 5 it can be seen that, for low-rise building, the difference between rehabilitation schemes 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 the case of a low-rise structure. On the other hand, Fig. 5 shows that, for medium- and high-rise buildings, 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-, medium-, 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 medium- and high-rise buildings. 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 buildings 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 a low-rise building (both designed to carry gravity loads). Consequently, as the relative strength of the beams to that of the columns of medium- and 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 Fig. 5 that the rehabilitation of the lower half (for the cases of rehabilitating columns only, and both columns and beams) is not effective in increasing the PGA capacity for the rehabilitated buildings compared to the PGA capacity of the existing ones. This can be attributed to the sudden change in the ductility capacity of the structural elements at the midheight of the building, which resulted in the failure occurring in the unrehabilitated part of the structure just above the mid-height level.

Maximum Interstory Drift

Fig. 6 shows the maximum interstory drift (ID) ratio for the five-, 10-, and 15-story buildings for the case of rehabilitating the columns and beams and the case of rehabilitating the columns only

Table 6. Natural Periods for the Studied Buildings (s)

Number of stories	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

(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). From Fig. 6, it can be noticed that the maximum ID value increases with the increase of FRP total thickness, which leads to a more ductile structure with higher ID capacity. Fig. 6 shows that for a low-rise building, the difference in maximum ID between the 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 cases of medium- and high-rise buildings did not result in a significant increase in the maximum ID 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. It is worth noting that FRP wrapping resulted in an insignificant reduction in the maximum ID at the same PGA level compared to the existing structure, but it increased the maximum ID capacity significantly. This is due to the fact that FRP wrapping did not contribute to the structure stiffness significantly, but it increased the ductility of the structural members. From Fig. 6, it can be noticed also that for a certain height, and dynamic and hysteretic properties of the building, the maximum ID capacity almost did not 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 interstory 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 a structure in terms of interstory drifts as recommended by SEAOC (1995) and FEMA (1997) are depicted in Fig. 6.

Fig. 7 shows the ID distribution along the height for five- and 15-story frames with different rehabilitation schemes when subjected to the maximum scaled El Centro earthquake record that the building can resist. Fig. 7 shows that the maximum ID occurs at the first floor level for the five-story 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 ID occurred about one-third of the height of the building. This can be attributed to the effects of higher modes.

Maximum Story Shear

Fig. 8 shows the maximum story shear-to-the-total weight of structure ratio (average capacity for the nine earthquakes) for the five-, 10-, and 15-story frames. From Fig. 8, it can be seen that FRP wrapping did not result in a significant increase in the maximum story shear. This is due to the fact that FRP wrapping does not contribute to the structure's stiffness significantly, but it increases the structure's ductility. Fig. 9 shows the maximum story shear-to-the-total weight of the structure ratio versus the maximum ID relationship for both five- and 15-story buildings. It can be seen that the effect of FRP rehabilitation in increasing the story

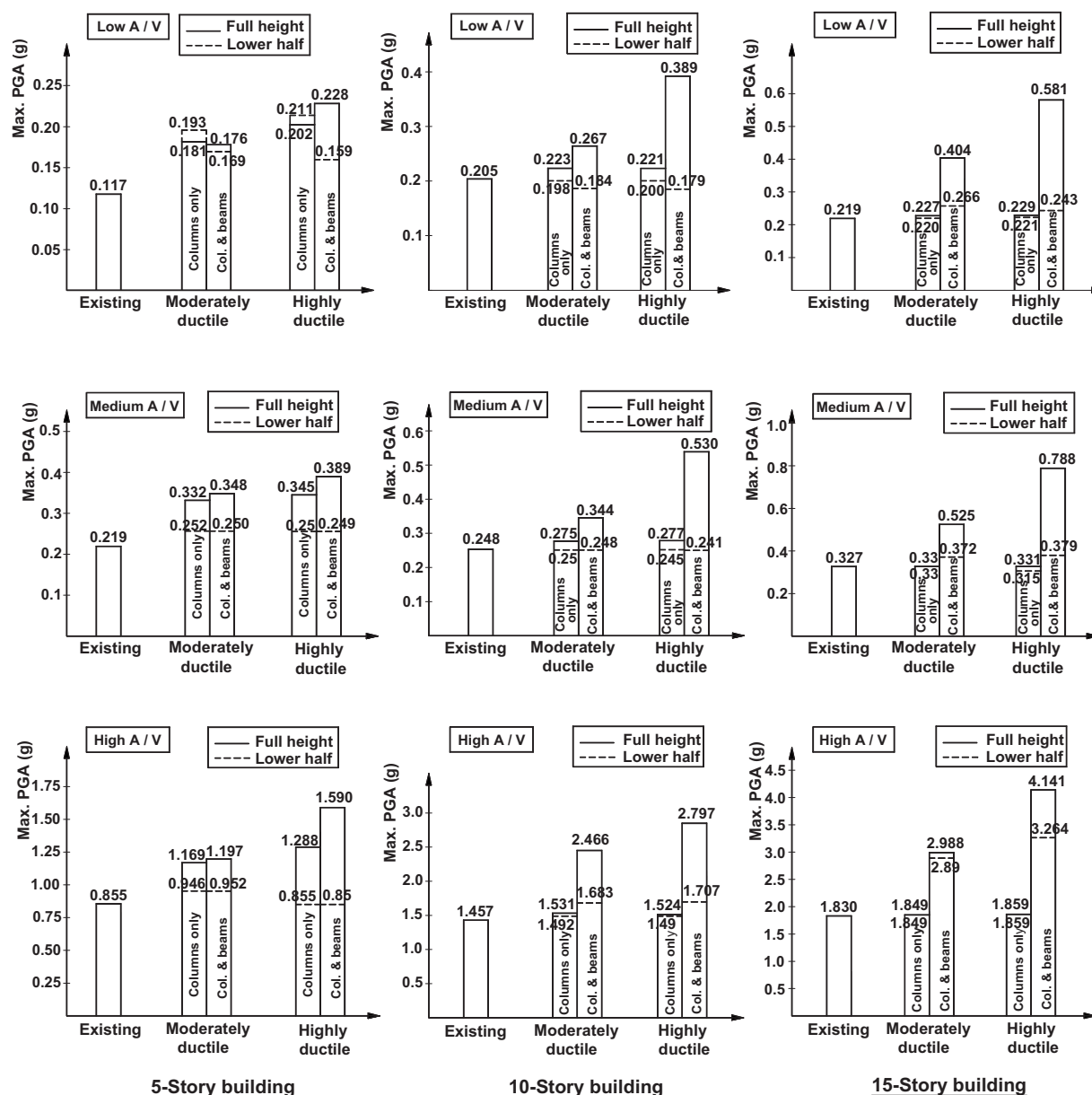


Fig. 5. Maximum PGA resisted by low-, medium-, and high-rise buildings

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 story shear for the five-story building is not influenced by the change of ground motion frequency content. On the other hand, the maximum story shear value for the 15-story building is affected by the frequency content of the earthquake record used.

Energy Dissipation Capacity

Fig. 10 shows the maximum energy dissipated by low-, medium-, and high-rise frames. It can be seen that the energy dissipation capacity of the frames increased when they were rehabilitated using FRP wrapping. Fig. 10 shows that rehabilitation of columns only (full height) was effective in the case of the low-rise structure, while it was not as effective as rehabilitation of both columns and beams for the full height of medium- and high-rise frames. This was observed previously from the results of maximum PGA capacities of the structures.

Maximum PGV Capacity versus Maximum PGA Capacity

Table 7 shows the maximum PGV that can be resisted by the low-, medium-, and high-rise frames. From Table 7, it can be noticed that the values of maximum PGV are similar for earthquakes with low and medium A/V ratios, while for high A/V earthquakes, a higher PGV can be resisted by the frames, which indicates a better seismic performance for earthquakes with high-frequency content.

From Fig. 5, it can be seen that the maximum PGA resisted by five-, 10-, and 15-story frames increased with the increase of A/V ratio of the selected ground motions. This trend is different from the trend for maximum PGV, but both trends correlate well with the mean acceleration response spectra for the three A/V ratio groups studied by Tso et al. (1992). In this study it was found that when the ground motion records were scaled to a common peak acceleration of 1g, the mean spectral acceleration decreased with

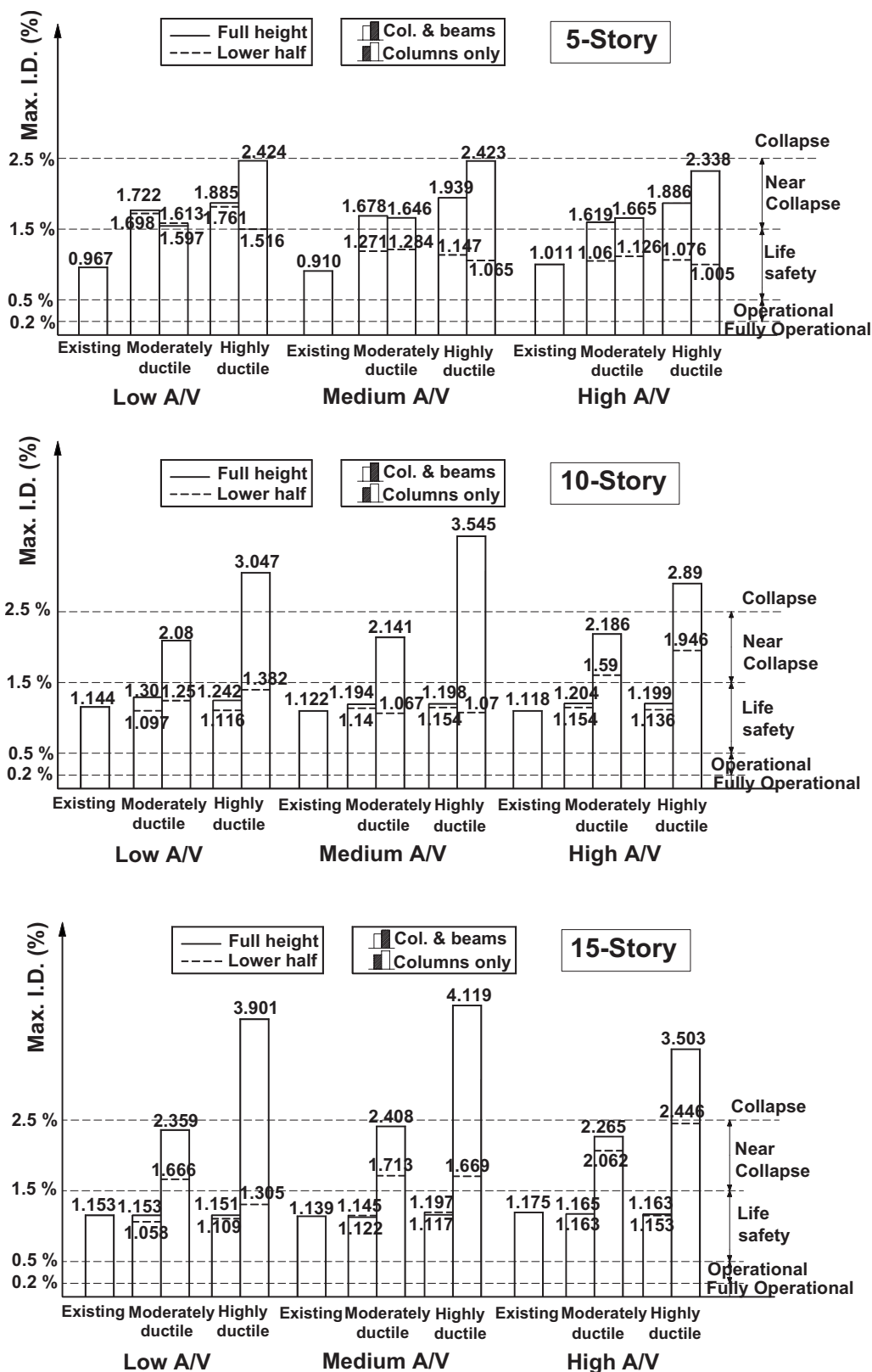


Fig. 6. Maximum ID ratio capacity for low-, medium-, and high-rise buildings

the increase of A/V ratio for periods longer than 0.5 s, which implies that the maximum PGA resisted by the frames increased with the higher-frequency content of the ground motion. This correlates well with the results for the maximum PGA capacity of the rehabilitated frames shown in Fig. 5. On the other hand, when

the earthquake records were scaled to a common peak velocity of 1 m/s, the mean spectral acceleration for low and medium A/V ratio were similar and higher than that of high A/V ratio for periods longer than 1 s. This correlates as well with the results for the maximum PGV capacity shown in Table 7.

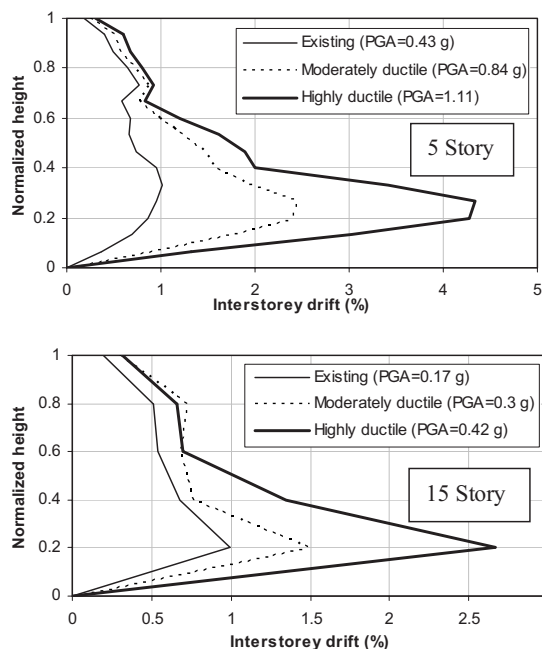


Fig. 7. ID distribution along the height of five- and 15-story buildings subjected to the maximum scaled El Centro earthquake record that the building can resist

Rehabilitation of Damaged Members Only versus Rehabilitation of a Group of Columns and/or Beams

In this study, the efficiency of FRP wrapping of selected damaged members of the structure when subjected to a specific earthquake was examined. The five- and 15-story existing frames were subjected to El Centro earthquake ground motion of maximum peak ground accelerations of 0.15 and 0.3g, respectively.

For the five-story frame, four plastic hinges occurred at the base of the first floor columns, while for the 15-story frame, the plastic hinges occurred at the end of the second span beams at the third through sixth floors. These locations were selected for rehabilitation with FRP wrapping for both structures. The PGA capacities were calculated for the case where the damaged members only were rehabilitated and were compared to the PGA capacities of the full rehabilitation pattern of beams and columns. Table 8 shows the PGA capacities for the three considered cases. From

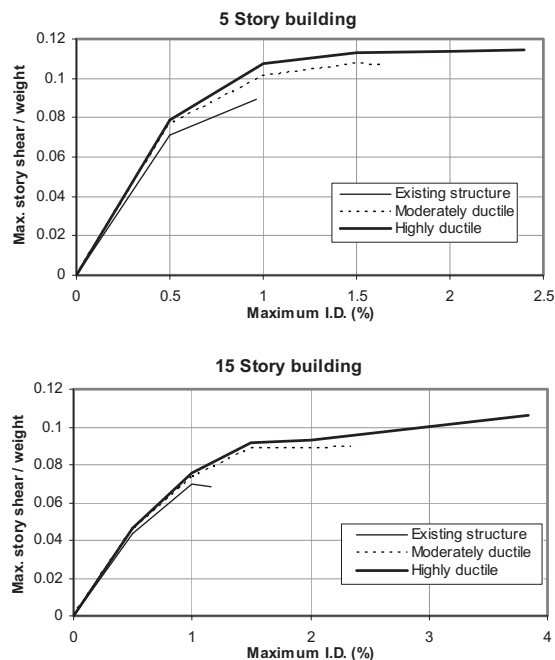


Fig. 9. Maximum story shear/weight with maximum ID for the five- and 15-story frames

Table 8, it can be noticed that the rehabilitation of the damaged members only was not effective in enhancing the seismic performance of the structures significantly. This was due to the fact that the overall capacity of the structure was dominated by the weakest structural member, which was transferred from one location (upon its rehabilitation) to another, etc.

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, $P \Delta$ 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

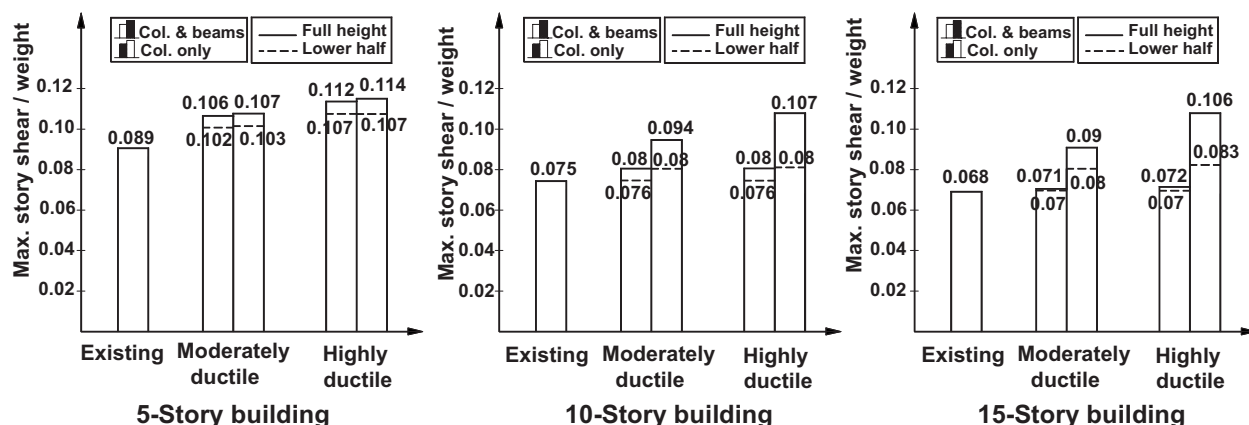


Fig. 8. Maximum story shear/weight for the five-, 10-, and 15-story frames

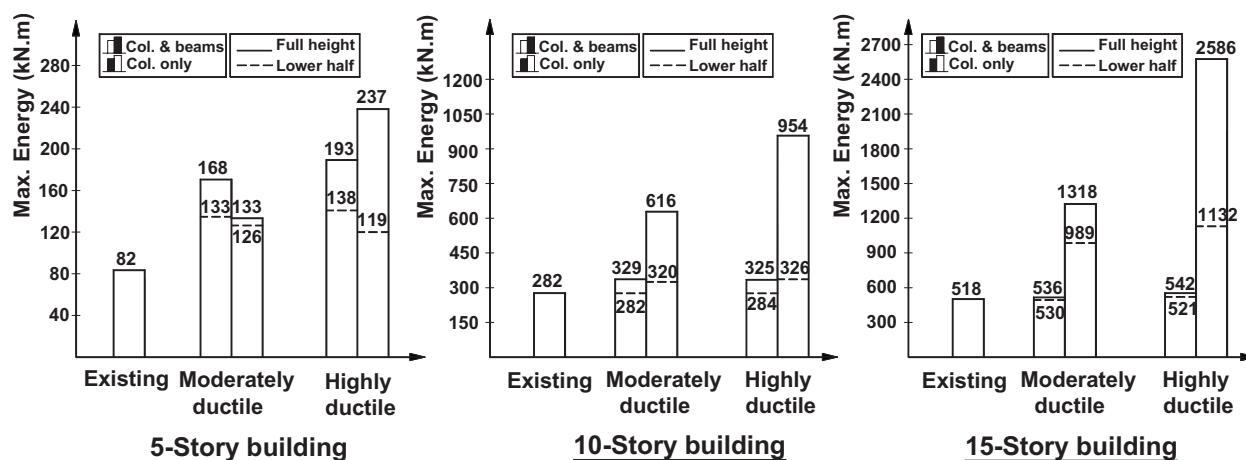


Fig. 10. Maximum energy dissipated for low-, medium-, and high-rise buildings

element would lead to erroneous response predictions of the structure performance. Fig. 11 shows the effect of inclusion of strength degradation in the analysis. Fig. 11 compares the maximum PGA resisted by the highly ductile frames of five- and 15-story buildings and the maximum ID ratio in two cases. In the first case the element strength degradation was included in the analysis, while the second case did not account for the strength degradation. In the latter case, the failure of the structure was assumed to occur when the structure reached a roof displacement equal to 5% of the structure height. Fig. 11 shows a major difference in the predicted results in both cases, which emphasizes the significance of accounting for the effect of strength degradation on the predicted performance of structures.

Rehabilitation of Selected Frames versus Rehabilitation of All Frames

The nonlinear dynamic analyses conducted on the selected frames rehabilitated using FRP wrapping showed that this rehabilitation scheme was very efficient in increasing the maximum PGA, ID, and energy dissipation capacities, especially for medium- and high-rise frames. As it was mentioned earlier, the FRP wrapping did not reduce the value of ID at the same PGA level but it increased the PGA and ID capacities, and this was attributed to

the fact that FRP confinement did not contribute to the structure stiffness, but it increased the ductility of the rehabilitated structures. This led to the need for investigating the behavior of studied rehabilitated buildings using FRP when rehabilitating all the building frames, compared to the case of rehabilitation of selected frames of the structure [e.g., the external frames only shown in Fig. 2(b)]. In this study the 15-story rehabilitated frame model 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' model was subjected to the nine ground motion records selected for this study and the average values were calculated.

Table 9 shows the results of the nonlinear dynamic analyses for the rehabilitated frame only (representing the case of rehabilitation of all frames) and for the combined rehabilitated and existing frames for the high-rise building [representing the case of rehabilitation of the two exterior frames only without rehabilitating the interior two frames shown in Fig. 2(b)]. From Table 9, it can be seen that rehabilitating the external frames only using FRP wraps did not result in a significant enhancement in the structure response. This can be attributed to the fact that FRP wraps increase the ductility capacity of the frame members without altering their stiffness significantly. Therefore, rehabilitating the external frames only of the structure will not be efficient since the global behavior of the structure will be controlled by the nomi-

Table 7. Maximum PGV (in m/s) Resisted by Low-, Medium-, and High-Rise Buildings

Number of stories	Acceleration/velocity ratio	Existing frame	Moderately ductile				Highly ductile			
			Beams and columns		Columns only		Beams and columns		Columns only	
			Full height	Lower half	Full height	Lower half	Full height	Lower half	Full height	Lower half
5	Low	0.204	0.294	0.285	0.300	0.315	0.370	0.271	0.331	0.342
	Medium	0.202	0.323	0.233	0.307	0.234	0.361	0.232	0.318	0.233
	High	0.414	0.58	0.460	0.566	0.456	0.760	0.415	0.626	0.416
10	Low	0.338	0.441	0.309	0.371	0.326	0.616	0.297	0.367	0.33
	Medium	0.232	0.319	0.23	0.258	0.233	0.489	0.224	0.26	0.23
	High	0.69	1.153	0.791	0.725	0.705	1.309	0.806	0.721	0.703
15	Low	0.346	0.63	0.433	0.359	0.346	0.928	0.382	0.363	0.347
	Medium	0.309	0.49	0.341	0.312	0.312	0.74	0.347	0.313	0.297
	High	0.884	1.433	1.376	0.893	0.895	2.019	1.551	0.897	0.898

Table 8. Maximum PGA Capacities for Existing and Rehabilitated Frames

Number of stories	Existing frame (g)	Rehabilitated frame	
		Damaged members only ^a (g)	All beams and columns (g)
5	0.17	0.23	0.42
15	0.42	0.43	1.11

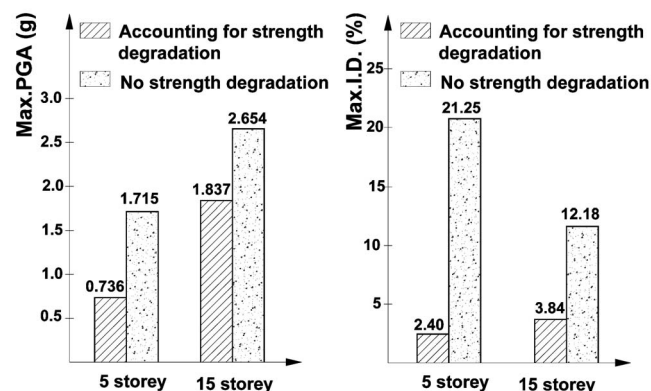
^aDamaged after being subjected to El Centro ground motion with PGA of 0.15 and 0.3g for five- and 15-story frame, respectively.

nally ductile (unrehabilitated) interior frames. In this case, the FRP-rehabilitated frames will not be able to reach their ultimate capacity (i.e., their maximum PGA and ID ratio capacities). Consequently, it can be concluded that in the case of rehabilitation using FRP wraps, all of the structure's existing frames should be rehabilitated in order to achieve the ductility level required for the seismic enhancement of the structure.

Conclusions

The effect of varying distributions of FRP rehabilitation on the seismic performance of existing nominally ductile RC frame structures was evaluated. Nonlinear dynamic analysis of three existing frames with different heights has been conducted. The frames were rehabilitated with different FRP distributions and different FRP contents. The analyses were conducted on the existing frames, 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 frame members were modeled to have displacement ductility capacities of 2.0, 4.0, and 6.0, respectively. The 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 frames, maximum interstory drift, maximum story 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:

**Fig. 11.** Comparison between the PGA and maximum ID capacities in cases of accounting for, or ignoring, strength degradation**Table 9.** Maximum Responses of the 15-Story Rehabilitated Frame Only, and Combined Rehabilitated and Existing Frames

Analysis results (frame)	Interstory		Story shear/weight	Energy dissipated (kN m)
	PGA (g)	drift (%)		
Existing frame	0.79	1.16	0.068	518
Rehabilitated frame	1.84	3.84	0.106	2,586
Existing+rehabilitated frame	0.81	1.16	0.070	1,098

1. For low-rise RC frame structures, FRP rehabilitation of the columns only was effective in increasing the structures' PGA and maximum interstory drift capacities. 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 was 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) was not significant.
2. For medium- and 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 medium- and high-rise buildings, the relative strength of the beams to that of the columns was 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.
3. The FRP rehabilitation of the lower half of the structure was found to be inefficient for the three studied buildings.
4. The maximum interstory drift ratio capacity increased with the increase of FRP content, which led to a more ductile structure.
5. For a certain height and dynamic properties of the FRP-rehabilitated buildings, the maximum interstory drift capacity was slightly influenced by the earthquake properties. This justified the validity of using the maximum interstory drift as a uniform and reliable damage parameter that can be used to judge the performance of FRP-rehabilitated structures.
6. Rehabilitation of selected frame members (beams and columns that were expected to have damage for a specified earthquake hazard) was not as effective as rehabilitation of both columns and beams for the whole structure.
7. Ignoring representing the postpeak strength degradation in the hysteretic nonlinear model of FRP-rehabilitated RC columns and beams would lead to erroneous overestimation of the seismic performance of the structure.
8. For the case of the FRP wrapping 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 whole structure.

It is important to clarify that the drawn results are for the studied cases and the selected earthquakes; more earthquakes should be considered and more analysis is needed for the conclusions to be generalized.

Acknowledgments

The financial support of Le fonds Québécois de la Recherche sur la Nature et les Technologies (FQRNT) through the team research

Appendix

Design of Required FRP Thicknesses for the Beams

The geometrical and mechanical properties of the beams were as such: $b=250$ mm; $h=600$ mm; $d=550$ mm; $A_s=5-22$ M = 2,000 mm²; 8 mm U-shaped stirrups at 200 mm; $f'_c=25$ MPa; and $f_y=400$ MPa. According to CSA (2004), the nominal shear resistance of the existing beam, V_r , will be equal to 192.96 kN.

NBCC (2005) requires that the shear resistance of the beams (and columns) designed to be ductile (i.e., $\mu_\Delta=6$) should be greater than the shear corresponding to the probable moment M_{pr} . In our case, $M_{pr}=451.2$ kN m, and the corresponding shear force $V_{req}=282.88$ kN, which is higher than V_r of the existing beams. Therefore, a shear contribution from the FRP, V_{FRP} , is needed to increase the ductility of the beam, where $V_{FRP,\mu=6}=282.88 - 192.96=89.92$ kN.

On the other hand, for the beams to be moderately ductile (i.e., $\mu_\Delta=4$), V_{req} would correspond to nominal moment M_n , where $M_n=376.8$ kN m, and the corresponding shear force $V_{req}=258.1$ kN. Therefore, $V_{FRP,\mu=4}=258.1 - 192.96=65.14$ kN m.

In this study, the beams were wrapped using U-wraps. The number and thickness of FRP layers to provide $V_{FRP,\mu=6}$ and $V_{FRP,\mu=4}$ were designed according to Monti and Liotta (2007), where the additional shear contribution of one layer of CFRP is given as

$$V_{Rd,f} = \frac{1}{\gamma_{Rd}} \times 0.9d \times f_{fed} \times 2t_f = 35.2 \text{ kN}$$

where design factor $\gamma_{Rd}=1.5$; beam depth $d=550$ mm; effective debonding strength $f_{fed}=190.5$ MPa; and laminate thickness $t_f=0.28$ mm (using carbon FRP with properties shown in Table 3).

Therefore, the number of CFRP wrap layers needed to make the beam ductile $= V_{FRP,\mu=6} / V_{Rd,f} = 89.92 / 35.2 \approx 3$ layers. Similarly, the number of CFRP wrap layers needed to make the beam moderately ductile $= V_{FRP,\mu=4} / V_{Rd,f} = 65.14 / 35.2 \approx 2$ layers.

Design of Required FRP Thicknesses for the Interior Ground Floor Column of 15-Story Frame

The geometrical and mechanical properties of the column were as such: $b=800$ mm; $t=800$ mm; $d=750$ mm; clear story height (h) = 2.65 m; $A_s=16-22$ M, bar spacing (w) = 175 mm; 8 mm ties at 400 mm; $f'_c=25$ MPa; and $f_y=400$ MPa. For that column, the axial load, $P=5,300$ kN; moment to shear ratio (M/V) = 1.325 m; nominal moment capacity $M_n=580$ kN m; and corresponding shear $V_{flex}=580 / 1.325=437.7$ kN.

In this study, the columns were confined using CFRP wraps with properties shown in Table 3. The number and thickness of the CFRP layers to provide moderate and high ductilities were designed according to Galal (2007).

To achieve a displacement ductility of 6, i.e., $\mu_\Delta=6$, we ensured that the nominal shear resistance of the rehabilitated column $V_{\mu=6} \geq V_{flex}$, where $V_{\mu=6}=V_s+V_f$ such that V_s =contribution of the steel ties to the shear resistance of the column; and V_f =contribution of the CFRP wraps. In our case, $V_s=75$ kN ($< V_{flex}$). Assuming three layers of CFRP, $V_f=416.1$ kN. Therefore, $V_{\mu=6}=491.1$ kN $\geq V_{flex}=437.7$ kN.

To achieve a displacement ductility of 4, i.e., $\mu_\Delta=4$, we ensured that the nominal shear resistance of the rehabilitated column $V_{\mu=4} \geq V_{flex}$, where $V_{\mu=4}=[1/3(V_c+V_p)]+V_s+V_f$ such that V_c and V_p =contributions of the concrete and axial force to the shear resistance of the column, respectively. In our case, $V_c=482.6$ kN and $V_p=800$ kN. Thus, assuming one layer of CFRP wrap, $V_f=138.7$ kN. Therefore, $V_{\mu=4}=641.2$ kN $\geq V_{flex}=437.7$ kN.

Notation

The following symbols are used in this paper:

- A/V = acceleration to velocity ratio of the ground motion record;
- E_f = Young's modulus of FRP material;
- F_c = cracking strength;
- F_y = yielding strength;
- F_u = ultimate strength;
- ID = interstory drift ratio;
- K_o = initial slope of the force-displacement ductility curve;
- K = postcracking slope of the force-displacement ductility curve;
- PGA = peak ground acceleration;
- PGV = peak ground velocity;
- α = postyielding stiffness parameter;
- β = postpeak stiffness degradation parameter;
- μ_Δ = displacement ductility;
- Δ_c = member cracking displacement; and
- Δ_y = member yield displacement.

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