

Analytical investigation of the seismic performance of RC frames rehabilitated using different rehabilitation techniques

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ABSTRACT

The objective of this study is to investigate analytically the effectiveness of different rehabilitation patterns in upgrading the seismic performance of existing non-ductile reinforced concrete (RC) frame structures. The study investigates the performance of two RC frames (with different heights representing low- and high-rise buildings) with or without masonry infill when rehabilitated and subjected to three types of ground motion records. The ground motion records represent earthquakes with low, medium and high frequency contents. Three models were considered for the RC frames; bare frame, masonry-infilled frame with soft infill, and masonry-infilled frame with stiff infill. Four rehabilitation patterns were studied, namely: (1) introducing a RC shear wall, (2) using steel bracing, (3) using diagonal FRP strips (FRP bracings) in the case of masonry-infilled frames, and (4) wrapping or partially wrapping the frame members (columns and beams) using FRP composites. Incremental Dynamic Analysis was conducted for the studied cases. The seismic performance enhancement of the studied frames is 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.

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1. Introduction

The seismic behaviour of multistory frame structures has been studied extensively in the last few decades due to their efficiency as earthquake resisting systems. The multistory framed systems can be categorised as reinforced concrete (RC) frames, masonry-infilled frames, and wall-frame systems.

A large number of pre-1970 designed buildings used masonry infill panels in their construction. Although the masonry infill panels can be beneficial or detrimental to the seismic response of the structure, they are not usually considered in the analysis of frame structures. Moreover, pre-1970 design codes adopted a strength-based philosophy, and hence once the ultimate strength of the structure is reached, a non-ductile deterioration follows, which reduces the energy dissipated by the structure and results in a brittle failure. Performance-based (PB) seismic engineering intends to decrease the probability of the brittle failure of structures, and increase their energy dissipation capacity when subjected to the design ground motions. Therefore, structures designed according to old codes need to be strengthened to meet the requirements of the performance-based design approach.

The 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 [1,2]. 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. The hazard level can be presented 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 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. Increasing the capacity by reducing the structure's drift at the same peak ground acceleration (PGA) level [depicted by arrow (A) in Fig. 1] can be achieved by increasing the stiffness of the building, e.g. by using RC walls or steel bracings. Increasing the structure's ductility capacity without reducing the drift at the same PGA level [depicted by arrow (B) in Fig. 1] 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 for columns and/or beams.

RC structural walls and steel bracings are effective lateral force resisting systems that are considered as efficient methods for

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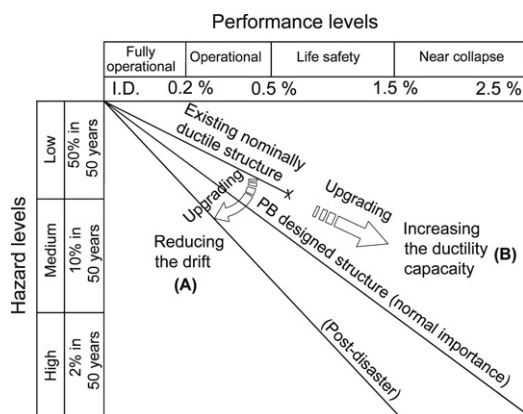


Fig. 1. Seismic performance of existing non-ductile structures and possible ways of upgrading.

retrofitting existing structures. RC walls can provide the required lateral stiffness and strength for resisting lateral loads due to wind or earthquakes, and hence several research studies and experimental work were done to investigate their behaviour under lateral loads, and to enable the designers to predict their seismic response in a certain building when subjected to a severe ground motion. A steel bracing system also has several advantages which make it another effective alternative to provide additional lateral stiffness and strength to the structure. Steel bracings are easy to apply, and they can be applied externally without disturbance to the building occupants. The steel has high strength-to-weight ratio, thus the additional mass added to the structure is less compared to the case of introducing a RC structural wall.

Fibre-reinforced polymers (FRP) composite materials are considered to have a high potential in strengthening and rehabilitation of existing structures due to their high strength and ease of application. Several FRP rehabilitation schemes have been proposed and used for frame structure elements [3]. FRP composites can be used by wrapping or partially wrapping the structural elements, such as columns and beams, to increase their ductility and shear strength capacity [e.g. [4–10]]. In the case of masonry-infilled frames, epoxy-bonded FRP overlays can be applied on the whole surface of the infill or by using diagonal FRP strips (bracing) that are anchored to the infill and frame elements [e.g. [11,12]].

The objective of this study is to investigate analytically the effectiveness of different rehabilitation techniques in upgrading the seismic performance of existing non-ductile RC frame structures designed according to pre-1970 strength-based codes. The study investigates the performance of two RC frames (with different heights) with or without masonry infill when rehabilitated and subjected to three types of ground motion records. The heights of the RC frames represent low- and high-rise buildings. The ground motion records represent earthquakes with low, medium and high frequency contents. Three models were considered for the RC frames; bare frame, masonry-infilled frame with soft infill, and masonry-infilled frame with stiff infill. The studied rehabilitation patterns include (1) introducing a RC shear wall, (2) rehabilitating using steel bracing, (3) rehabilitating using diagonal FRP strips (FRP bracings) in the case of masonry-infilled frames, and (4) wrapping or partially wrapping the frame members (columns and beams) using FRP composites. The study also investigates the effect of the infill stiffness on the seismic performance of the structures. Incremental Dynamic Analysis was conducted for the studied cases to evaluate the seismic performance enhancement of the rehabilitated frames. The seismic performance is assessed 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.

2. Properties of the selected ground motions

Tso et al. [13] 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 analysed 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 the current study, a set of nine far-field earthquake records was selected for the analysis [13,14]. The ground motion records represent earthquakes with low, medium and high frequency contents. The properties of the selected ground motions are summarised in Table 1. A similar set of ground motion records were used to study the effectiveness of eccentric steel bracings in the rehabilitation of low-rise RC frames by Ghobarah and Abou-Elfath [15].

3. Properties of the selected buildings

Two RC frames designed according to a pre-1970 strength-based code [16] are selected for this study. The frames are of heights five and fifteen storeys to represent low- and high-rise buildings, respectively. The frames are designed to carry a 6 m wide slab, assuming a frame spacing of 6 m and a live load of 2 kPa. The storey height is 3.25 m and the total heights of the two frames are 16.25 and 48.75 m, respectively. The elevations of the two frames, the concrete dimensions for the beams and columns, the steel reinforcement and the reinforcement ratios are shown in Fig. 2. The dimensions of the columns' section and the steel reinforcement ratios varied along the height of the frames according to the change of axial load acting on each group of columns, while the beam dimensions and reinforcement were assumed to be the same for the entire frame. For existing frames, 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 as 20 GPa and that of steel was taken as 200 GPa. The concrete density was 24 kN/m³ and concrete Poisson's ratio ν was taken to be 0.15.

4. Existing structures and different rehabilitation schemes

5- and 15-storey existing frames representing low- and high-rise buildings, respectively, were studied. For both frame heights, bare and masonry-infilled frames were considered in the analysis. To investigate the effect of infill stiffness on the seismic response of the frames, two different masonry infill types with different stiffnesses were considered. The infill stiffnesses represent soft and stiff infills. The three types of existing frames (bare, with soft masonry infill, and stiff masonry infill) were rehabilitated using four techniques; the first was by demolishing the masonry panel in the middle bay and introducing a RC wall. The RC wall used was 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 steel reinforcement ratio was taken 0.015 for both walls. The wall dimensions and steel reinforcement ratio were assumed to remain constant along the wall height. The second rehabilitation technique was by introducing steel X-bracings in the middle bay along the full height of the frames. The 5- and 15-storey frames were rehabilitated using HSS 219 × 6.4 and HSS 273 × 11 braces, respectively. The maximum tensile strength for the steel material F_y was taken as 350 MPa.

The third rehabilitation technique was by applying FRP bracings on the three panels of the frame along the full height, in which they are anchored to the infill and to the concrete members. The case of applying the X-FRP bracings to the intermediate panel only was also studied. The fourth rehabilitation technique was by wrapping

Table 1
Properties of the selected ground motions [14].

No.	Earthquake	Site	Date	A (g)	V (m/s)	A/V		Duration (s)	Soil condition
						(g/s/m)	Level		
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

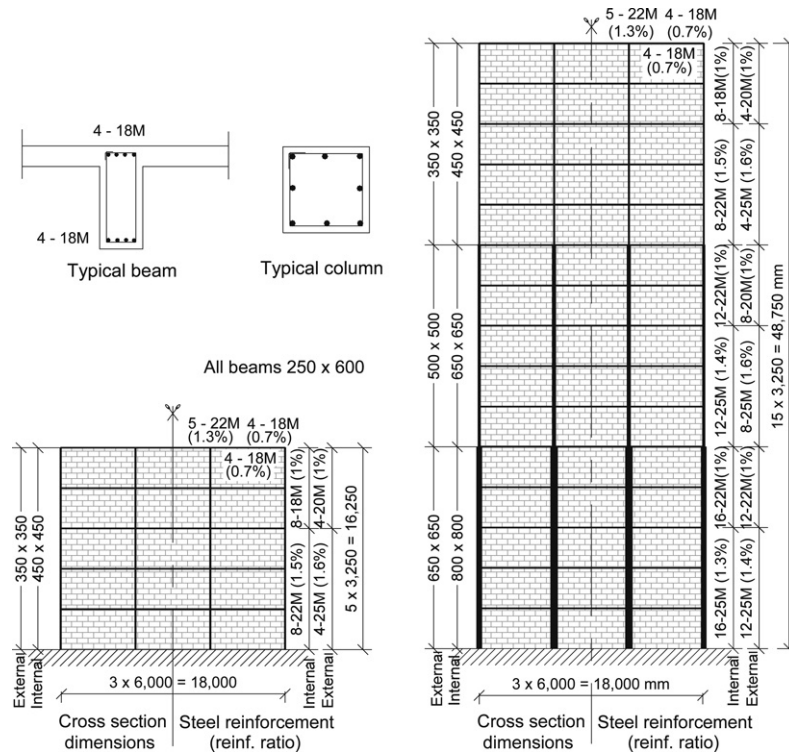


Fig. 2. Elevations of the 5- and 15-storey studied frames.

the columns laterally with FRP composites, while the beams were rehabilitated using FRP U-wraps near their ends. A total of fourteen cases were studied for each of the 5- and 15-storey frames; the cases of existing, rehabilitated with RC wall, rehabilitated with steel bracings, rehabilitated with FRP bracings, and rehabilitated columns and beams with FRP wrapping were studied for the bare frame, soft infill, and stiff infill frame models. Fig. 3 shows the four studied rehabilitation schemes assembled in one figure.

5. Nonlinear models used in the time history analyses

Non-linear dynamic analyses were conducted for the 5- and 15-storey frames with different rehabilitation schemes. A computer software for three dimensional nonlinear and dynamic structural analysis CANNY [17] was selected for the analyses. The mass of each floor was lumped at the column joints according to the tributary areas. The frame joints were assumed to be rigid and rigid zones were applied at the ends of each member. P-delta effects were considered in the analyses. Fig. 4 shows the idealisation of different members of the studied frames assembled in one figure.

5.1. Modelling of existing beams and columns

The beams and columns were modelled as linear elastic elements with two inelastic single-component flexure rotation springs located at the ends of the member. The Deterioration Model CP4 [17] was used to model the nonlinear flexure rotation spring, which allows a representation of the combined flexural and shear backbone curves with a parameter that controls the displacement ductility capacity after which the postpeak degradation of the element occurs. The lateral force–displacement ductility relationship of the columns and beams of the existing frames was assumed to have limited displacement ductility, μ_{Δ} , equal to 2, which is followed by a quick reduction in the lateral load resistance due to the onset of shear failure. The post-peak degradation in strength occurs through a displacement equivalent to the yield displacement up to a residual force of 0.3 of the ultimate load capacity. The hysteretic behaviour of the model CP4 is shown in Fig. 5.

5.2. Modelling of masonry infill

Madan et al. [18] and Dolsek and Fajfar [19] studied the behaviour of masonry infilled RC frame structures. In their

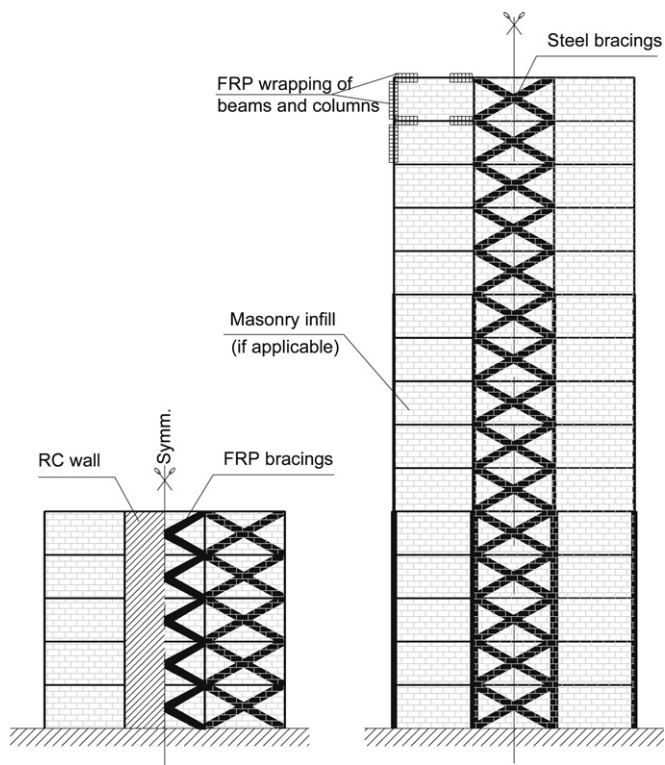


Fig. 3. The four studied rehabilitation schemes.

models, the masonry infill panel was represented by two diagonal compression struts, while the tensile strength of masonry was neglected. The hysteretic behaviour of the compression strut was defined using a trilinear model that is able to represent the cracking and the ultimate strength followed by strength degradation. In the current study, the unrehabilitated infill panels were modelled using the compression strut model. The properties of the compression strut were chosen based on Özcebe et al. [11], and they were scaled to match the panel dimensions of the studied frames. Two different masonry infill types with different stiffnesses were considered, representing soft and stiff infill, to investigate the effect of infill stiffness on the response of structures. Fig. 6 shows the axial stress–axial strain relationship for the compression struts of the two types of infill. The hysteretic parameters used for the infill models are shown in Table 2.

5.3. Modeling of RC wall

The RC wall was modelled using the CANNY panel element. The panel element has four nodes at the corners in addition to a node

Table 2
Hysteretic parameters for the masonry infill models.

Parameter	Soft infill	Stiff infill
Maximum strength (MPa)	3.30	3.70
Initial stiffness (GPa)	9	27
Maximum strain	0.003	0.003
Unloading stiffness degradation (γ)	0.5	0.1
Inner loop stiffness reduction (ζ)	1.0	1.0
Post-peak stiffness (GPa)	−0.072	−0.072

Table 3a
Parameters used for the concrete model CS2.

Parameter	Value
Strain at maximum compressive strength	0.002
Compression post-peak residual/max capacity ratio λ	0.4
Ultimate strain/strain at maximum compressive strength ratio μ	1.75
Post-peak unloading stiffness parameter γ	0.2
Tension descending part after tension crack τ	3.0

Table 3b
Parameters used for the steel model SS3.

Parameter	Value
Skeleton curve parameters ν and κ	1.0, 1.0
Post-yielding parameter β	0.01
Parameter ϕ to direct unloading	0
Unloading stiffness degrading parameter γ	0.2
Unloading control parameter θ	0.75

at the mid points of the top and bottom boundaries (Fig. 4). The adjacent panels have compatible deformations at their common three nodes that are connecting them. A multi-axial spring model was used to represent the flexural and axial tension/compression interaction of the panel elements. A bilinear model CS2 [17] was used to represent the force–deformation relationship for the concrete springs, while the trilinear/bilinear model SS3 was used to represent the steel material. Fig. 7 shows the hysteretic behaviour for the concrete model CS2 and the steel model SS3. Tables 3a and 3b show the values for the parameters used for the models CS2 and SS3, respectively. For the modelling of the RC walls, the shear deformations were assumed to be linear. A similar representation for RC walls was used in the analysis of lightly reinforced concrete walls subjected to near-fault and far-field ground motions [20].

5.4. Modelling of steel bracing

The steel braces were modelled as axial tension/compression struts. The Buckling Model STB [17] 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 due to increasing the number of load cycles as was observed by

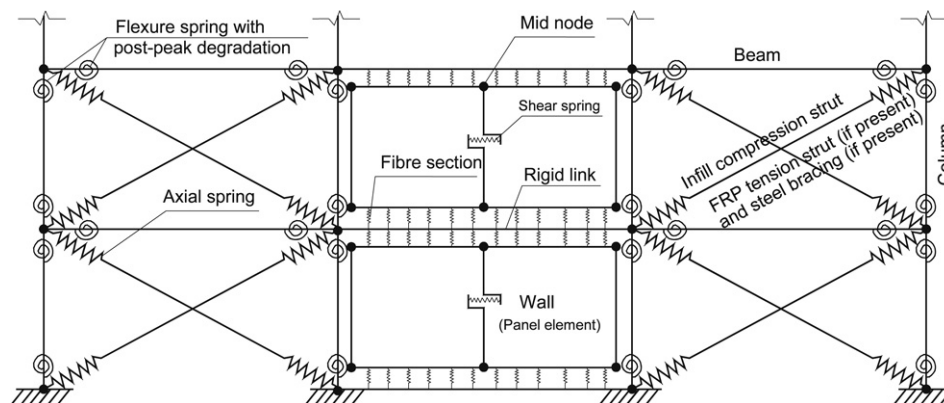


Fig. 4. Idealisation of different members of the studied frames (generic).

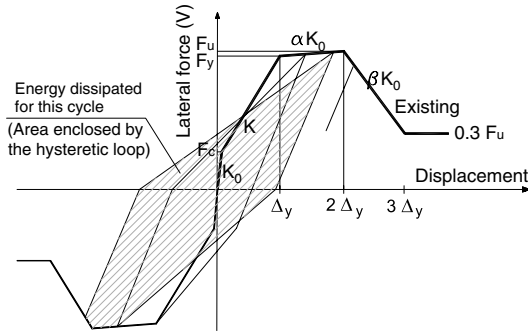


Fig. 5. Hysteretic behaviour of the Deterioration Model CP4 [17].

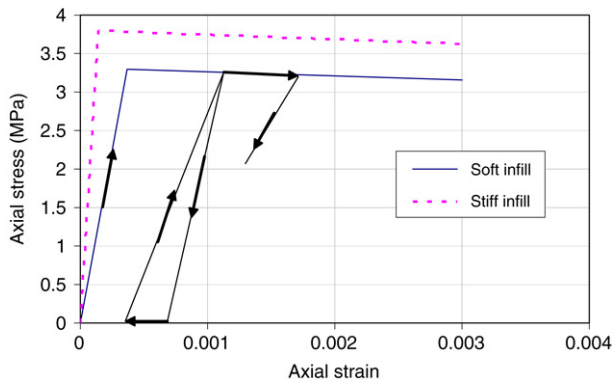


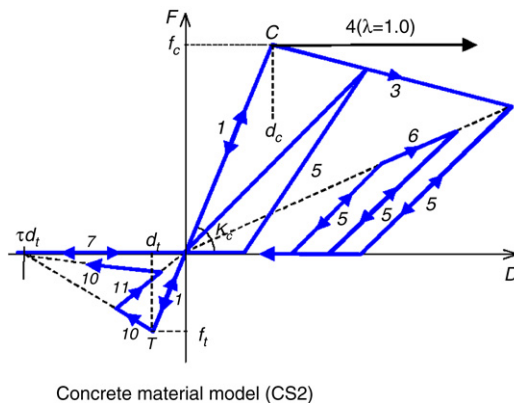
Fig. 6. Strut model for masonry infill.

Table 4

The properties of steel braces used for low- and high-rise frames.

	5-storey	15-storey
Section	HSS 219 × 6.4	HSS 273 × 11
Area (mm ²)	4240	9160
Slenderness ratio (λ)	90	74
F _y (kN)	1696	3664
F _c /F _y	0.335	0.396

Jain et al. [21] in their experimental tests on steel braces. The main input data for the model is the yield tensile strength of the member and the effective slenderness ratio which controls the value of maximum compressive strength. The hysteretic behaviour of the Buckling Model STB is shown in Fig. 8. Table 4 shows the properties of the steel braces used in the analysis. A similar model was used by Ghobarah and Abou-Elfath [15] to investigate the effectiveness of the eccentric steel bracing in rehabilitation of RC frame structures.



5.5. Modelling of FRP bracing

Özcebe et al. [11] proposed an analytical representation to predict the behaviour of masonry-infilled frames when rehabilitated with FRP bracings. The analytical model was correlated to the experimental tests carried out at Middle East Technical University (METU, Turkey) on a number of two-storey masonry-infilled frame specimens rehabilitated with different patterns of FRP, and subjected to cyclic displacement excitations at the storey levels. A similar uniaxial model for the FRP bracings was used in the current study. Fig. 9 shows the axial stress–axial strain relationship for the tension strut of FRP bracings. The FRP bracings were modelled as uniaxial tension struts with maximum axial strain of 0.003 and maximum axial stress of 190 MPa. These values took into account the characteristics of carbon FRP, plaster, infill, as well as the effect of FRP delamination and failure of anchor dowels that was observed during the tests at METU. The models used for the masonry infill and FRP bracing were verified using the results of the experimental tests carried out at METU, and the results of the analytical model using CANNY were matching the experimental results for the unrehabilitated and rehabilitated specimens, and for both static pushover and dynamic analyses. Similar models for the masonry infill and FRP bracings were used in the analytical study conducted by Binici et al. [22] to simulate the behaviour of FRP strengthened infill walls.

5.6. Modelling of rehabilitated columns and beams with FRP wrapping

For this rehabilitation scheme, the columns were rehabilitated by wrapping them laterally with FRP composites, while the beams were rehabilitated using FRP U-wraps near their ends. The force–displacement ductility backbone curves for the existing and rehabilitated frames using FRP wrapping are shown in Fig. 10. The number and thicknesses of FRP wraps needed for columns and beams to reach the targeted displacement ductility level can be designed according to the equations proposed by Monti and Liotta [23] for the FRP U-wrapped beams, and by Galal [24] for the FRP wrapped columns. The FRP content used in the rehabilitation of the structural elements intended to increase their energy dissipation capacities and displacement ductility, μ_{Δ} , to be equal 6.0 (compared to 2.0 in case of existing members). The displacement ductility values of 2.0 and 6.0 were chosen based on the experimental work done by Memon and Sheikh [25] in which they found that the displacement ductility for the existing columns ranged between 1.3 and 3.7, while the displacement ductility for columns rehabilitated using glass FRP ranged between 4.7 and 6.8. Also, it is worth noting that the product of the ductility and overstrength factors, $R_d \cdot R_o$, defined by the National Building Code of Canada [26] to relate the linear elastic to nonlinear ductile

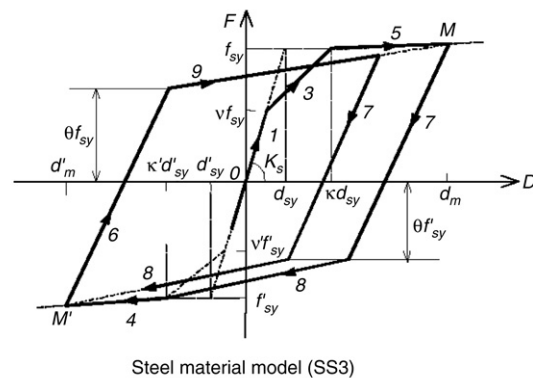


Fig. 7. Hysteretic models of the concrete and steel fibres for the wall element [17].

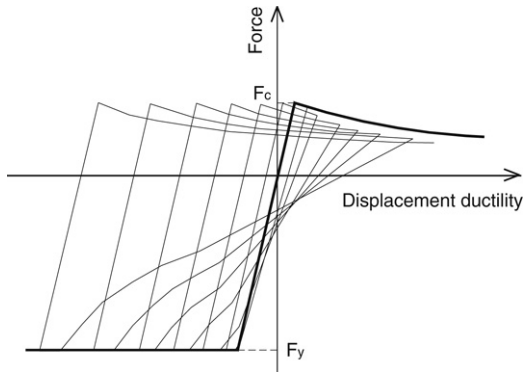


Fig. 8. Hysteretic behaviour of the Buckling Model STB [17].

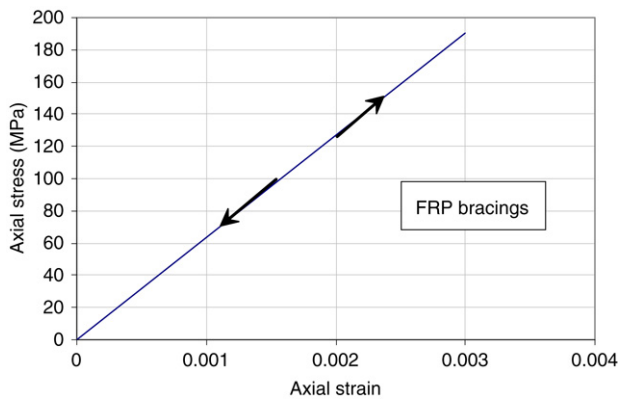


Fig. 9. Strut model of FRP bracings.

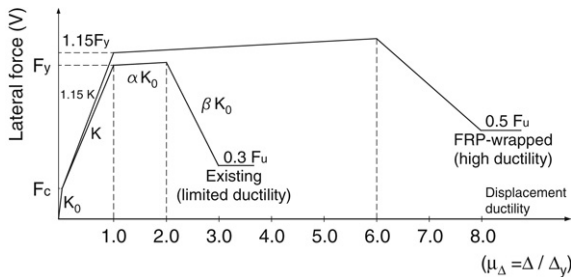


Fig. 10. The force–displacement ductility relationships of the existing and FRP-wrapped columns or beams.

response- is equal to 2.6 and 7.5 for the limited ductility and ductile RC moment resisting frames, respectively.

Fig. 10 also shows that strength degradation due to the shear failure was considered in the element's lateral force–displacement ductility backbone curve. For the rehabilitated frame members, the post-peak degradation in strength occurs through a displacement equivalent to two times the yield displacement up to a residual force of 0.5 of the ultimate load capacity. For the models of the existing frame members and the rehabilitated ones using FRP wrapping, the unloading stiffness degradation factor, γ , was taken as 0.1, while the inner loop stiffness reduction factor, ξ , was assumed to be 1.0. It is worth noting that the equations proposed by Monti and Liotta [23] for the design of FRP U-wrapped beams took into account the FRP delamination, as these equations were obtained based on their experimental work in which FRP debonding were observed at beam failures.

Modal analysis was conducted for the 5- and 15-storey frames for different rehabilitation schemes, and 5% damping ratio was considered. Table 5 shows the modal analysis results for the studied cases. It should be noted that, for the same building

height, the existing structure and the rehabilitated one using FRP wrapping have the same natural period. This is due to the fact that the initial stiffness of the member before cracking was assumed to be the same for the existing and rehabilitated members.

6. Analysis results

6.1. Maximum applied peak ground acceleration (PGA)

Nine earthquake records were applied to 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 both 5- and 15-storey frames were evaluated for different rehabilitation schemes and earthquake records. Fig. 11 shows the maximum PGA resisted by different rehabilitation patterns and different infill stiffnesses for low- and high-rise 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 both low- and high-rise frames, 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-wrapping is an efficient technique in increasing the PGA capacity of the frames, especially for high-rise buildings. For the effect of the earthquake frequency content on the seismic behaviour of RC structures, it can be noted that the PGA capacity was significantly dependent on the frequency content of the ground motion. The maximum PGA resisted by the 5- and 15-storey frames increased with the increase of A/V ratio of the selected ground motions, and this correlated well with the mean acceleration response spectra for the three A/V ratio groups studied by Tso et al. [13].

6.2. Maximum inter-storey drift

In the analytical model of the studied frames, a supernode was defined at each floor level which represents a rigid diaphragm in the horizontal plane (RC slab). This node would control the lateral displacement of other frame nodes in the same floor [17]. The lateral displacement of the supernode at each level was obtained (master displacement), and the interstorey drift (I.D.) ratio was calculated at each floor by dividing the difference in the lateral displacement of two successive floors over the storey height. Fig. 12 shows the maximum I.D. ratio capacity (defined as the maximum I.D. ratio calculated when the frame was subjected to the maximum PGA that can be resisted) for the 5- and 15-storey frames for different rehabilitation schemes and different infill stiffnesses. From the figure, it is noted that FRP bracings have a negligible effect on the maximum I.D. ratio capacity, which matches the experimental test results carried out in METU [11]. On the other hand, rehabilitating the frames using 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 bracing 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 frame. It can be seen from the figure that, the maximum I.D. ratio capacity is high for the rehabilitated frames using FRP wrapping for both low- and high-rise frames, which leads to a more ductile structure with a higher I.D. ratio capacity.

The figure also shows that for a certain building height (i.e. low- or high-rise), and certain dynamic and hysteretic properties of the

Table 5
Natural periods for the studied frames (s).

No. of storeys	Bare frame				Soft infill frame					Stiff infill frame				
	Existing	+ RC wall	+ Steel bracing	+ FRP-wrapping	Existing	+ RC wall	+ Steel bracing	+ FRP bracing	+ FRP-wrapping	Existing	+ RC wall	+ Steel bracing	+ FRP bracing	+ FRP-wrapping
5	1.01	0.38	0.59	1.01	0.39	0.28	0.35	0.38	0.39	0.22	0.21	0.21	0.21	0.22
15	2.49	1.52	1.75	2.49	1.38	1.19	1.28	1.36	1.38	1.04	1.02	1.04	1.04	1.04

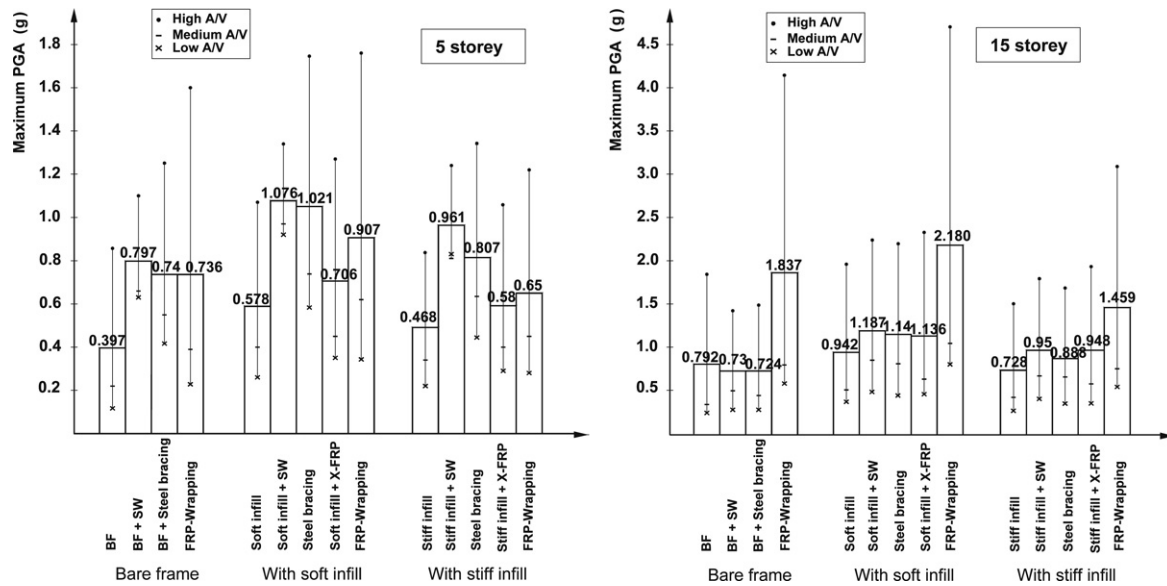


Fig. 11. Maximum PGA resisted by low- and high-rise frames.

building, the maximum I.D. capacity slightly changed with respect to the change in the earthquake properties (A/V ratio). This was not the case for the maximum PGA capacity of the structure which was affected by the earthquake frequency content as shown in Fig. 11. This justifies the validity of using the maximum inter-storey drift ratio as a uniform and reliable damage parameter that can be used to judge the performance of rehabilitated structures. The performance levels that represent the damage degree of a structure in terms of inter-storey drifts as recommended by SEAOC [1] and FEMA [2] are also depicted in the figure.

Fig. 13 shows the $PGA-I.D._{max}$ capacity curves of the existing 5- and 15-storey frames with soft infill when rehabilitated using different rehabilitation patterns. From the figure, it can be seen that rehabilitating the frames with FRP bracings resulted in a decrease in the maximum I.D. ratio for a certain PGA level when compared to the existing frames. While for the same PGA level, rehabilitating the frames using RC wall or steel bracing decreased the value of maximum I.D. significantly 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 might be needed. The figure also shows that rehabilitating the frames using a FRP wrapping did not decrease the value of maximum I.D. ratio at the same PGA level significantly but it increased the maximum PGA and maximum I.D. ratio capacities for the structure.

Fig. 14 shows the distribution of inter-storey drift ratio along the height of the 5- and 15-storey frames with soft infill when

subjected to the scaled El Centro earthquake record. The figure shows the effectiveness of each rehabilitation scheme in reducing the value of the maximum I.D. ratio. From the figure, it could be seen that rehabilitating the existing frame by introducing a RC wall or steel X-bracings resulted in a different profile of I.D. compared to other rehabilitation techniques. This could be attributed to the fact that the deformations of the moment resisting frame are characterised by shear beam behaviour, while the RC wall and steel X-bracings deformations are characterised by flexural beam behaviour. It should be noted that, the I.D. profile associated with the rehabilitation scheme of introducing a RC wall or using steel X-bracings has the advantages of having lower maximum I.D. (in comparison to other rehabilitation schemes), and that the maximum I.D. occurs at a higher level of the building. The latter would reduce the additional nonlinearities resulting from $P-\Delta$ effects.

6.3. Maximum storey shear

Fig. 15 shows the maximum storey shear-to-the total structure's weight ratio (average demand for the nine earthquakes) obtained for low- and high-rise frames. It can be seen that rehabilitating the frames using RC wall or steel bracing attracts higher forces due to the increase of stiffness, which results in a reduction in the natural period of the structure. The figure shows also that the presence of masonry infill leads 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 also be seen that the rehabilitation using FRP wrapping did not increase the storey shear demand significantly. This can be attributed to the fact that FRP wraps do not contribute to the structure stiffness, but they increase the ductility capacity of the structural elements of the building without altering their stiffness.

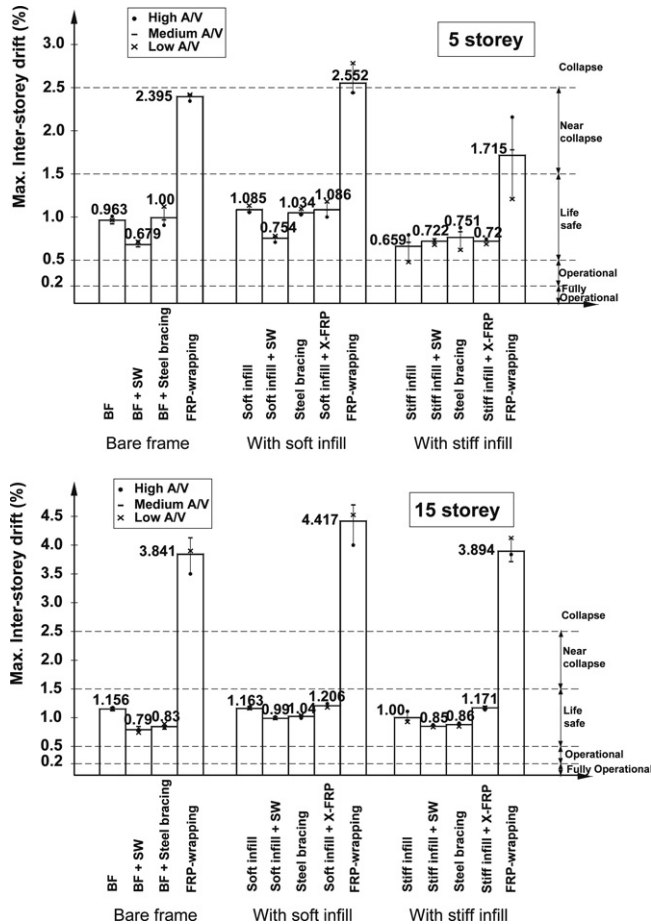


Fig. 12. Maximum inter-storey drift ratio capacity for low- and high-rise frames.

6.4. Energy dissipation capacity

Energy dissipation capacity is an important indicator of the structure's ability to withstand severe ground motions. This parameter can be determined from the area enclosed by the hysteretic loops of the load-deformation relationship as shown in Fig. 5. Fig. 16 shows the maximum energy dissipated by the 5- and 15-storey frames for different rehabilitation patterns and the two infill stiffnesses. It can be seen that for low-rise frame, rehabilitating the frame using a RC wall or steel bracing dissipated higher energy than the case of FRP bracings or FRP wrapping, which indicates that the introduction of a RC wall will be more efficient in resisting the lateral loads than the use of FRP bracing or FRP wrapping, yet this solution will be impractical if the building was occupied while rehabilitation. In that case the steel bracing, FRP bracing or FRP wrapping could be alternative solutions. The figure also indicates that for the studied high-rise frame, wrapping the columns and beams using FRP will dissipate higher energy than introducing a RC wall, steel bracing or FRP bracings.

7. Importance of accounting for the masonry infill in the analysis

Masonry infill can significantly improve the seismic response of structures or, on the contrary, they can cause unexpected damage. Therefore, the influence of masonry infill on the seismic performance of structures should not be overlooked. In this study, the selected frames were analysed for the cases of neglecting (bare frame) and accounting for the presence of the masonry infill (soft infill and stiff infill), and the response was compared

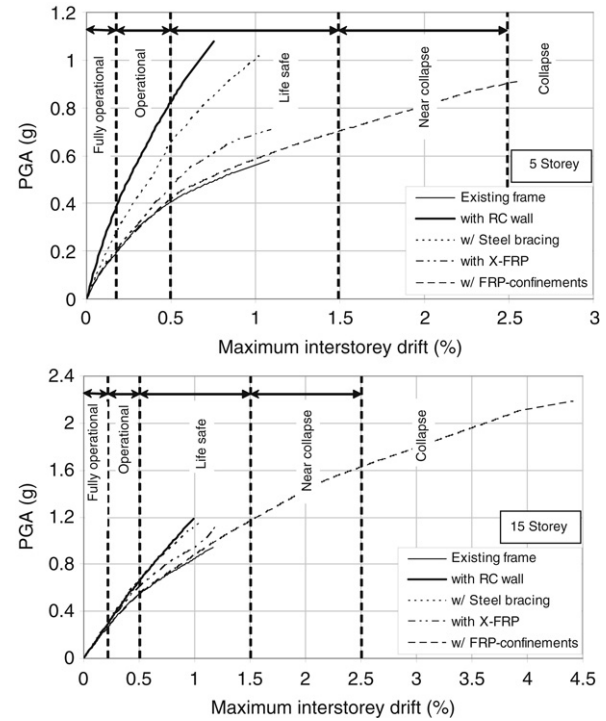


Fig. 13. Capacity curves of the 5- and 15-storey frames with soft infill.

in both cases. Fig. 17 shows the PGA-I.D._{max} curve for the 15-storey frame rehabilitated using FRP bracings and using a RC wall in case of ignoring or accounting for the masonry infill. From the figure, it can be seen that at the same level of PGA, the presence of masonry infill reduces the I.D. ratio which represents a better seismic performance of the structure according to the SEAOC [1] criteria. Figs. 11, 12, 15 and 16 show the bare frame's response compared to that of the masonry infilled frames. It can be seen that accounting for the masonry infill in the analyses has improved the performance of the 5- and 15-storey frames, especially for energy dissipation capacity. On the other hand, it should be noted that the current model is not capable of representing some of the non-ductile failures that can occur due to the presence of infill, such as short column mechanism at the column ends.

8. Effect of number of rehabilitated bays for the case of FRP bracings

For the case of rehabilitation using FRP bracing, the effect of the number of rehabilitated bays was investigated. The selected frames were studied in case of rehabilitating the intermediate bay only, compared to rehabilitating the three bays using FRP bracing. Table 6 shows the performance parameters of the 5- and 15-storey frames with different infill stiffnesses, when rehabilitated using one bay or three bays of FRP bracings. It can be seen that rehabilitating the whole frame resulted in a better performance than rehabilitating one bay only. In that case, cost analysis should be considered in order to decide on the most economical choice.

9. Effectiveness of rehabilitating a limited number of the structure's frames versus rehabilitation of all frames

Figs. 13 and 16 show that the FRP wrapping rehabilitation scheme is efficient in increasing the maximum PGA, I.D. and energy dissipation capacities, especially for high-rise frames. From Fig. 13, it was noticed that the FRP wrapping scheme did not reduce the value of maximum I.D. at a certain PGA level (as was the case

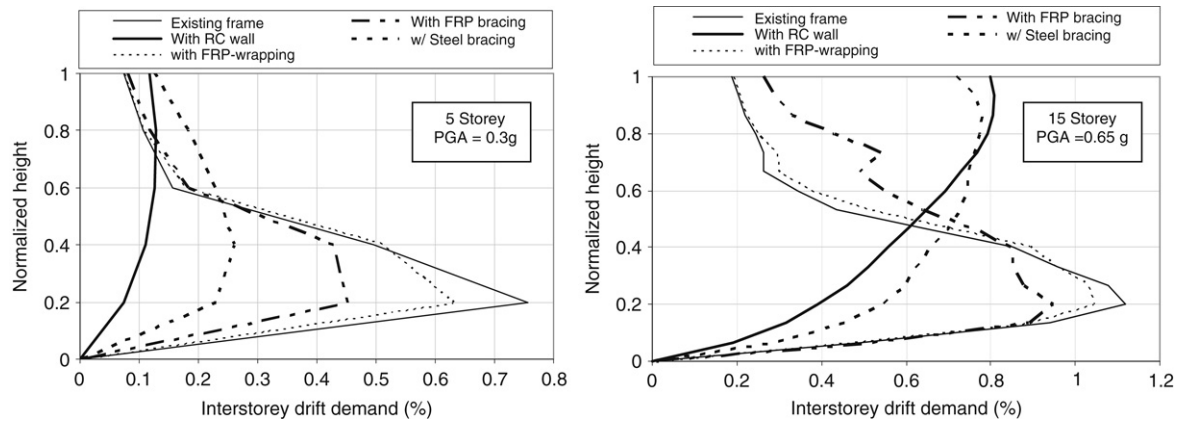


Fig. 14. I.D. distribution along the height of 5 and 15 storey frames when subjected to the scaled El Centro earthquake record.

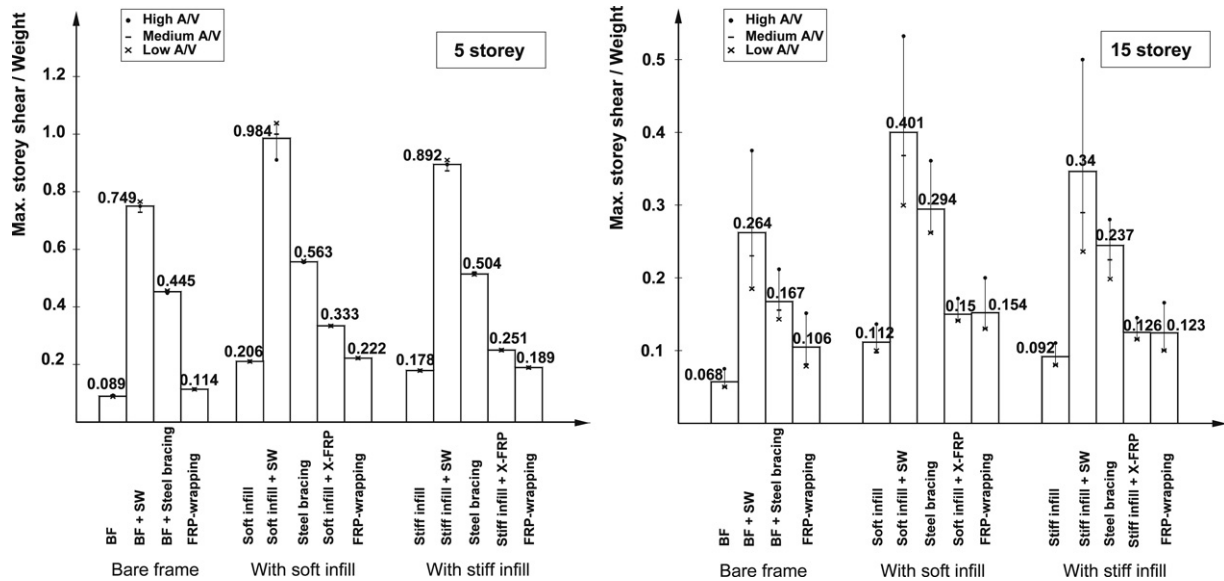


Fig. 15. Maximum storey shear-to-the total weight for low- and high-rise frames.

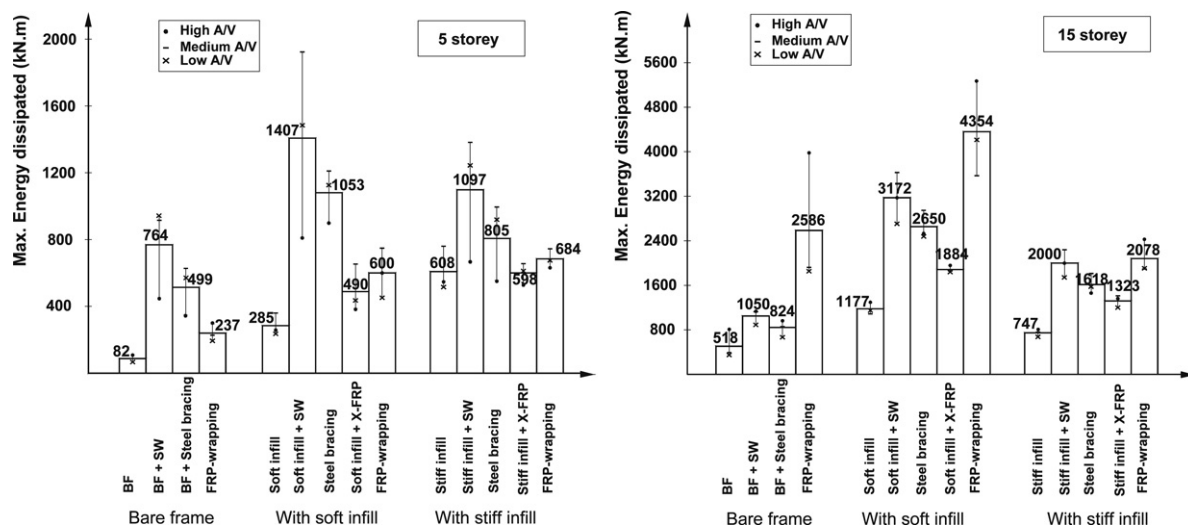


Fig. 16. Maximum energy dissipated for low- and high-rise frames.

for other rehabilitation schemes) but it increased the PGA and I.D. capacities of the rehabilitated buildings. This behaviour was observed more in the 15-storey building. This could be attributed to the fact that FRP wrapping did not contribute to the stiffness of

the structure like the other three studied rehabilitation schemes did, but it rather increased the ductility of the rehabilitated structure. This led to the importance of investigating the effect of the rehabilitation scheme on the seismic performance of the

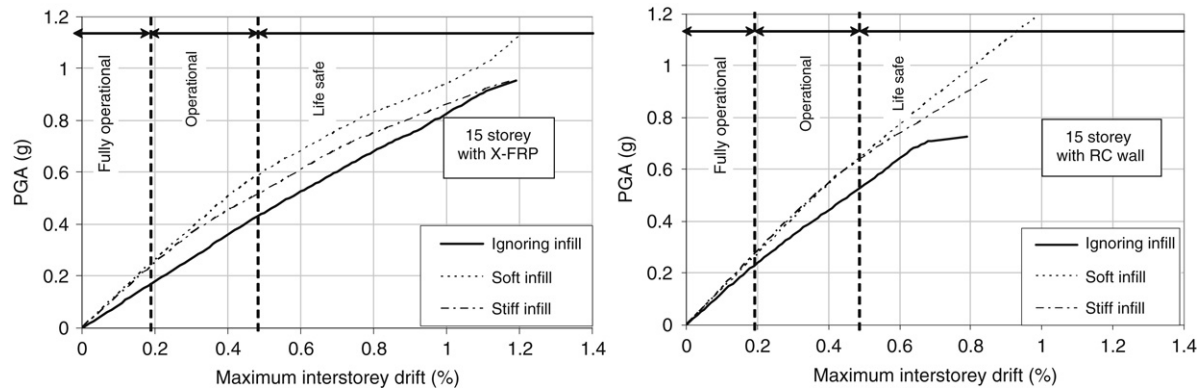


Fig. 17. Performance of the 15 storey frame in cases of ignoring or accounting for the masonry infill.

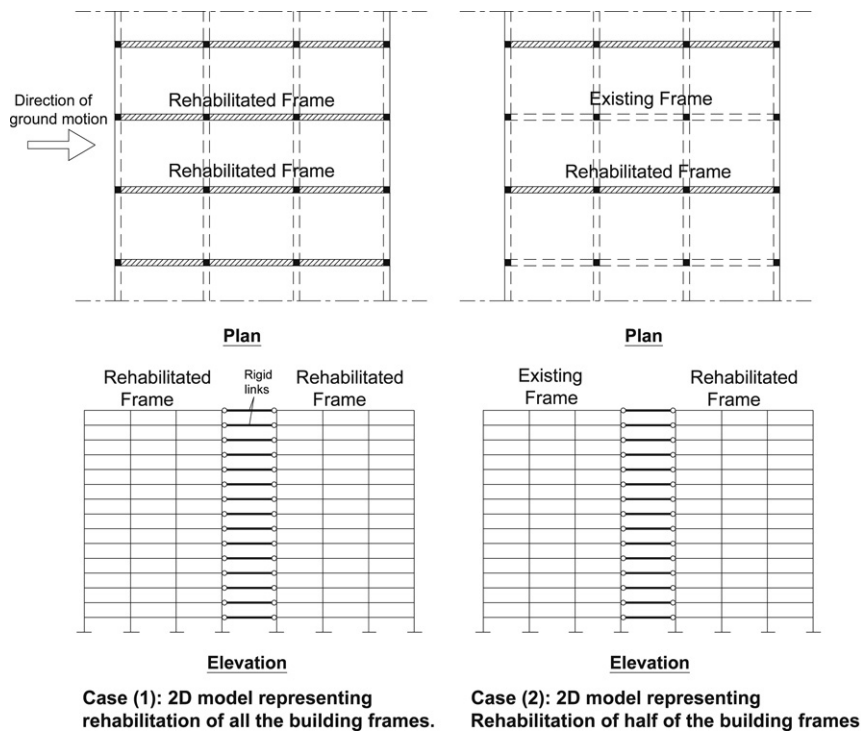


Fig. 18. 2D models representing cases of rehabilitating all frames and rehabilitating half of the frames.

Table 6

Effect of number of bays rehabilitated with FRP bracing on the seismic performance of the frames.

No. of storeys	Infill type	No. of rehabilitated bays	Max. PGA (g)	Max. I.D. (%)	Storey shear /weight	Max. Energy (kN m)
5 Storey	Soft	1 bay	0.64	1.09	0.245	361
		3 bays	0.71	1.09	0.333	490
	Stiff	1 bay	0.50	0.79	0.202	684
		3 bays	0.58	0.72	0.251	598
15 Storey	Soft	1 bay	0.98	1.18	0.12	1391
		3 bays	1.14	1.21	0.15	1884
	Stiff	1 bay	0.79	1.14	0.10	964
		3 bays	0.95	1.17	0.13	1323

studied 15-storey building when a limited number of frames (in the direction of ground motion) were rehabilitated using one of the studied four rehabilitation schemes while the other frames remained without rehabilitation. In this section, two different cases were considered as shown in Fig. 18. The first case is when all the building frames were rehabilitated using one of the studied four rehabilitation schemes. For modelling of this case, two identical frames with soft infill were connected using rigid links

at the floor levels representing the diaphragm action of the RC slabs. The second case is when every other frame of the building frames was rehabilitated. In this case, each frame model of the 15-storey rehabilitated building (i.e. frame with RC wall, with steel bracing, with FRP bracing, and with FRP wraps) with soft infill was linked to the existing frame model. The frames' model for the above mentioned two cases was subjected to the nine ground motion records selected for this study.

Table 7

Rehabilitation of all the building frames versus rehabilitation of half the building frames.

Analysis results (per two frames)	Case 1: Rehabilitation of all building frames					Case 2: Rehabilitation of half the building frames			
	Existing	RC structural wall	Steel bracing	X-FRP bracing	FRP wrapping	Existing + RC structural Wall	Existing + steel bracing	Existing + X-FRP bracing	Existing + FRP wrapping
PGA capacity (g)	0.94	1.19	1.15	1.14	2.18	1.11	1.25	1.03	0.96
I.D. capacity (%)	1.16	0.99	1.04	1.21	4.42	0.98	1.06	1.18	1.16
Max. 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)	2360	6340	5340	3760	8700	5020	5010	3000	2520

Table 7 shows the results of the nonlinear dynamic analyses for the two studied cases (case of rehabilitation of all the building frames, and case of rehabilitation of half the building frames) for the studied high-rise building. From the table, it can be seen that rehabilitating half of the building frames using RC walls, steel bracing or FRP bracing improved the behaviour of the whole structure to different degrees. On the other hand, rehabilitating half of the building frames using FRP wrapping did not result in a significant enhancement in the whole structure response. This was expected due to the fact that FRP wraps increase the ductility capacity of the frame members without significantly altering their stiffness. Therefore, rehabilitating individual frames of the structure using FRP wraps will not be efficient since the global behaviour of the structure will be controlled by the nominally ductile (unrehabilitated) frames. In this case, the FRP rehabilitated frames will not be able to reach their capacity because the maximum I.D. capacity of the existing frame is considerably smaller than that of the FRP-wrapped frames. Consequently, it can be concluded that in case of rehabilitation using FRP wraps, all the structure's existing frames should be rehabilitated in order to effectively enhance its seismic performance (i.e. PGA, I.D., and energy dissipation capacities). On the other hand, rehabilitating half of the building frames using RC wall, steel X-bracing, or X-FRP bracing was found to be efficient in enhancing the seismic performance of the studied building.

10. Conclusions

The effectiveness of different rehabilitation patterns in upgrading the seismic performance of existing non-ductile RC frame structures was evaluated. The frames were rehabilitated using four different techniques; (1) introducing a RC wall, (2) introducing steel X-bracings, (3) using FRP X-bracings, and (4) by wrapping the structural members (beams and columns) using FRP composites. Two different masonry infill types with different stiffnesses (soft and stiff infill) were considered in the analyses. The bare frames ignoring the effect of masonry infill were also studied. Non-linear dynamic analysis was conducted for the existing frames and the rehabilitated ones when subjected to three types of ground motion records. The ground motion records used were selected to represent earthquakes with low, medium and high frequency contents. The seismic performance enhancement of the analysed frames was evaluated based on 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. 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:

- For the studied low-rise frame, introducing a RC wall increased the PGA capacity, storey shear demand, and energy dissipation capacity while rehabilitating the columns and beams of the structure using FRP wrapping increased the I.D. ratio capacity.
- For the studied high-rise frame, introducing a RC wall increased the storey shear demand, while rehabilitating the columns and beams of the structure using FRP wrapping increased the PGA, I.D., and energy dissipation capacities.
- For a certain building height (i.e. low- or high-rise) and certain dynamic properties of the building, the maximum inter-storey drift capacity is slightly influenced by the earthquake properties. 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 rehabilitated structures.
- 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.
- 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 structure.
- 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.

From the above conclusions it can be seen that the choice of the most suitable rehabilitation scheme should consider the properties of the structure, the seismic hazard, and the desired performance parameter needed to be enhanced. It is important to clarify that the results drawn are for the studied cases and the selected earthquakes. More ground motion records should be considered and more analysis including cost analysis is needed for the conclusions to be generalised.

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References

- Structural Engineers Association of California (SEAOC). Performance-based seismic engineering of buildings. In: Proc., vision 2000 committee. 1995.
- Federal Emergency Management Agency (FEMA). Commentary on the NEHRP guidelines for the seismic rehabilitation of buildings. In: FEMA 273/274, Washington (D.C.), FEMA; 1997.
- Bakis C, Bank L, Brown V, Cosenza E, Davalos J, Lesko J. Fiber-reinforced polymer composites for construction—state-of-the-art review. Compos Construct 2002. ASCE 6 (2) 73–87.
- Lu X, Teng J, Ye L, Jiang J. Bond-slip models for FRP sheets/plates bonded to concrete. Eng Struct 2005;27(6):920–37.
- Khalifa A, Gold W, Nanni A, Abdel Aziz M. Contribution of externally bonded FRP to shear capacity of RC flexural members. Compos Construct ASCE 1998; 2(4):195–202.

- [6] Pellegrino C, Modena C. FRP shear strengthening of RC beams with transverse steel reinforcement. *Compos Construct ASCE* 2002;6(2):104–11.
- [7] Pellegrino C, Modena C. FRP shear strengthening of RC beams: Experimental study and analytical modeling. *ACI Struct J* 2006;103(5):720–8.
- [8] Harajli M, Hantouche E, Soudki K. Stress-strain model for fiber-reinforced polymer jacketed concrete columns. *ACI Struct J* 2006;103(5):672–82.
- [9] Teng J, Huang Y, Lam L, Ye L. Theoretical model for fiber-reinforced polymer-confined concrete. *Compos Construct ASCE* 2007;11(2):201–10.
- [10] Rocca S, Galati N, Nanni A. Review of design guidelines for FRP confinement of reinforced concrete columns of noncircular cross sections. *Compos Construct ASCE* 2008;12(1):80–92.
- [11] Özcebe G, Keskin U, Tankut T. Strengthening of brick-infilled RC frames with CFRP. Technical report no. 2003/1. Structural Engineering Research Unit. Middle East Technical University (METU). 2003.
- [12] Saatcioglu M. Research on seismic retrofit and rehabilitation of reinforced concrete structures. In: *Proceedings of 31st annual conference of Canadian Society for Civil Engineering CSCE*; 2003. p. 2403–12.
- [13] Tso W, Zhu T, Heidebrecht A. Engineering implication of ground motion A/V ratio. *Soil Dynam Earthq Eng* 1992;11(3):133–44.
- [14] Pacific Earthquake Engineering Research Center (PEER). Strong Motion Database, PEER. Berkeley (CA): Univ. of California at Berkeley; 2008. <http://peer.berkeley.edu/smcat/>.
- [15] Ghobarah A, Abou Elfath H. Rehabilitation of a reinforced concrete frame using eccentric steel bracing. *Eng Struct* 2001;23(7):745–55.
- [16] American Concrete Institute (ACI). *Manual of concrete practice*, part 2. Committee 318-1R-68, Detroit (USA); 1968.
- [17] Li K. Three dimensional nonlinear static and dynamic structural analysis. Computer program. User's manual. CANNY Structural Analysis. Vancouver, B.C. (Canada); 2008.
- [18] Madan A, Reinhorn A, Mander J, Valles R. Modeling of masonry infill panels for structural analysis. *Struct Eng* 1997;123(10):1295–302.
- [19] Dolsek M, Fajfar P. Mathematical modeling of an infilled RC frame structure based on the results of pseudo-dynamic tests. *Earthq Eng Struct Dyn* 2002;31(6):1215–30.
- [20] Galal K. Modeling of lightly reinforced concrete walls subjected to near-fault and far-field earthquake ground motions. *Struct Design Tall Spec Build* 2008;17(2):295–312.
- [21] Jain A, Goel S, Hanson R. Hysteretic cycles of axially loaded steel members. *Struct Div* 1980;106(8):1777–95.
- [22] Binici B, Özcebe G, Özelik R. Analysis and design of FRP composites for seismic retrofit of infill walls in reinforced concrete frames. *Composites Part B: Eng* 2007;38(5–6):575–83.
- [23] Monti G, Liotta M. Tests and design equations for FRP-strengthening in shear. *Construct Build Mater* 2007;21(4):799–809.
- [24] Galal K. Lateral force–displacement ductility relationship of non-ductile squat RC columns rehabilitated using FRP confinement. *Struct Eng Mech* 2007;25(1):75–90.
- [25] Memon M, Sheikh S. Seismic resistance of square concrete columns retrofitted with glass fibre-reinforced polymers. *ACI Struct J* 2005;102(5):774–83.
- [26] National Building Code of Canada (NBCC). *National building code of Canada*. Associate Committee on the National Building Code, Ottawa, Ontario (Canada): National Research Council of Canada; 2005.