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UPGRADING THE SEISMIC PERFORMANCE OF EXISTING MASONRY–INFILLED RC FRAMES USING FRP BRACINGS

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ABSTRACT

Reinforced concrete (RC) buildings located in seismically active zones are expected to deform well into their inelastic range when subjected to a seismic load. Prior to the enforcement of the ductile design philosophy of the 1970's, structures were designed on the basis of strength. Once the ultimate strength of a structure is reached, abrupt non-ductile deterioration follows, which reduces the energy dissipating capacity of that structure and results in brittle failure. Performance-based seismic engineering is the modern approach to earthquake resistant design. Seismic performance is described by designating the maximum allowable damage state indices for an identified seismic hazard. Overall, lateral deflection, ductility demand, and inter-storey drift are the most commonly used damage state indices.

A large number of pre-1970 designed buildings used masonry infill panels in their construction. Although representing the masonry infill panels can be beneficial or detrimental to the seismic response of the structure, they are not usually considered in the analyses of frame structures. Traditional rehabilitation methods of masonry-infilled RC frames include adding anchored steel mesh with plaster to the panel or by demolishing the masonry panel and introducing a RC infill. Such methods can be labor intensive, add considerable mass, and cause significant impact on the occupants. The use of fibre-reinforced polymer (FRP) composites offers a faster and easier rehabilitation alternative using epoxy-bonded FRP overlays applied on the whole surface of the infill or by using diagonal FRP strips that are anchored to the infill and frame members.

The objective of this study is to analytically investigate the effectiveness of FRP bracing in upgrading the seismic performance of existing masonry-infilled RC frame structures. This study investigates the performance of two masonry-infilled RC frames when rehabilitated using FRP bracing and subjected to three types of ground motion records. The heights of the RC frames represent low- and high-rise buildings and the ground motion records represent earthquakes with low, medium, and high frequency contents. The enhancement in the seismic performance using FRP bracing is compared to that provided by demolishing a masonry panel and introducing a RC infill that changes the panel to a RC wall.

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The analyses results and conclusions are drawn by evaluating the seismic performance enhancement in terms of strength-deformation, inter-storey drift, and energy dissipation capacities of the studied structures.

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 categorized 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 representing the masonry infill panels can be beneficial or detrimental to the seismic response of the structure, they are not usually considered in the analyses 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. Therefore, structures designed according to old codes need to be strengthened in order to follow up with the requirements of the performance-based design approach, this approach is expected to decrease the probability of brittle failure of the structures, and increase the energy dissipation capacity when subjected to the design ground motions.

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. Many FRP rehabilitation schemes are proposed and used for frame structures. FRP composites can be used by wrapping the structural elements, such as the columns and beams, to increase their ductility and shear strength capacity. 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 (Özcebe, et al. 2003).

The objective of this study is to analytically evaluate the effectiveness of FRP bracing in upgrading the seismic behaviour of existing masonry-infilled RC frame structures designed according to pre-1970 strength based codes. This study investigates the performance of two masonry-infilled frames with different heights, representing low- and high-rise buildings, when rehabilitated with FRP bracings and subjected to three types of ground motion records. The ground motion records represent earthquakes with low, medium and high frequency contents. The enhancement in the seismic performance when using FRP bracing is compared to that provided by demolishing a masonry panel and introducing a RC structural wall. The study also investigates the influence of inclusion of masonry-infill and the effect of the infill strength on the seismic performance of the structures. The analyses results and conclusions are drawn by evaluating the seismic performance enhancement in terms of strength-deformation, inter-storey drift, and energy dissipation capacities of the studied structures.

Properties of the Selected Frames

Two RC frames were designed according to pre-1970 strength based code (ACI 1968) are selected for this study. The frames are of heights five and fifteen storeys that represent low- and high-rise buildings, respectively. The frames are designed to carry a 6 m wide slab, assuming frames′ spacing of 6 m and live load of 2 kPa, the floor height is 3.25 m and the total heights of the two frames are 16.25 and 48.75 m, respectively. Hollow–clay bricks of thickness 75 mm were used as the infill material, which represents a soft infill, with plaster on both sides each of thickness 10 mm. The elevations of the two frames, the concrete dimensions for the beams and columns as well as the steel reinforcement are shown in Fig.1. The dimensions of the column section and the steel reinforcement ratios were varying along the height according to the change of axial load acting on each group of columns while the beam dimensions were assumed to be the same for the entire frame.

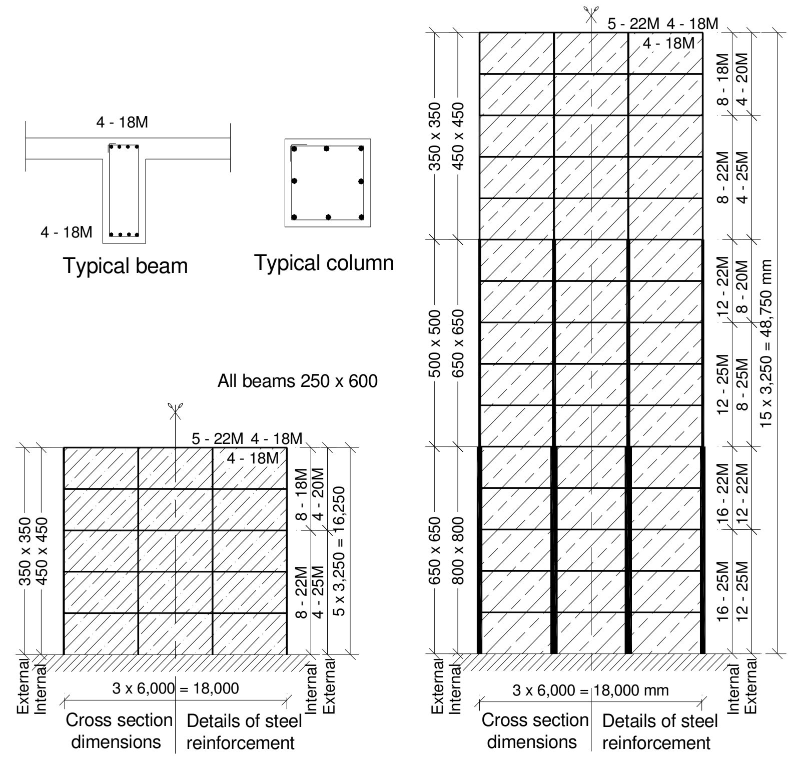


Figure 1. Elevation of 5 and 15 storey frames.

Properties of the Selected Ground Motions

In this study a set of 9 far-field earthquake records was selected for the analyses. The ground motion records represent earthquakes with low, medium and high frequency contents. The properties of selected ground motions are summarized in Table 1.

Table 1. Selected ground motions.

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

Existing structure and different rehabilitation schemes

The 5 and 15 storey existing masonry-infilled frames were rehabilitated using two techniques; one is by using FRP bracings along the full height of the frames, while the other is by the introduction of RC wall in the middle bay. The RC structural wall is added after demolishing the masonry panel in the middle bay. The wall is 6 m long, and has a thickness of 200 mm for the 15-storey frame, and a thickness of 100 mm for the 5-storey frame that represents a RC wall every 12 m. 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. In the study, the performance of the two frames was evaluated in case of not modeling the masonry infill (i.e. bare frame) and in case of modeling it. Moreover, in order 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. A total of nine cases were studied for each of the 5 and 15 storey frames; the three cases of existing, rehabilitated with FRP bracings, and rehabilitated with RC wall were studied for the bare frame, soft infill, and stiff infill frame models. Fig. 2 shows the two rehabilitation schemes considered in this study.

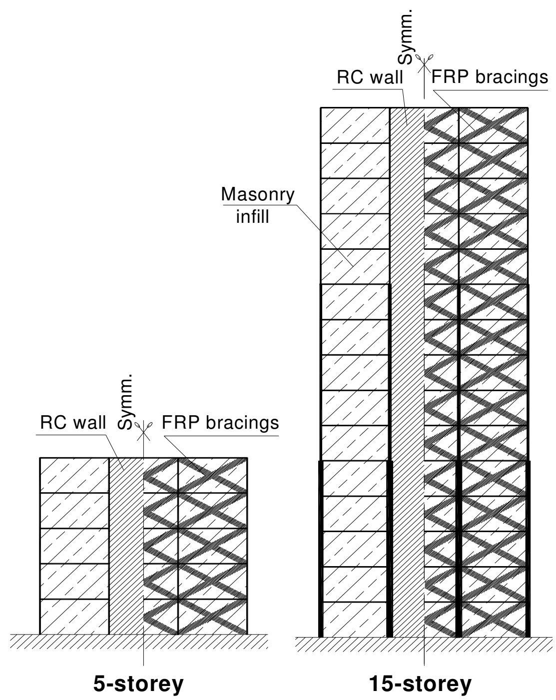


Figure 2. The two studied rehabilitation schemes.

Nonlinear Time History Analysis Using Recorded Ground Motions

a) Nonlinear analysis model:

A non-linear dynamic analysis was conducted for the 5 and 15 storey frames with the two rehabilitation schemes, a computer software for three dimensional nonlinear and dynamic structural analysis (CANNY) was selected for the analyses. The mass of each floor is lumped at the column joints according to the tributary areas. The frame joints are assumed to be rigid and rigid zones are applied at the ends of each member. P-delta effects were considered in the analyses.

1-Modeling of beams and columns: The beams and columns were modeled as linear elastic element with two inelastic single-component flexure rotation springs located at the ends of the member, CANNY deterioration model CP4 was used to model the nonlinear flexure rotation spring. The lateral force-displacement ductility relationship of the columns and beams of the existing frames was assumed to have limited displacement ductility, μ, equal to 2, which is followed by a quick reduction in the lateral load resistance. For the model the unloading stiffness degradation γ was taken 0.1, while the inner loop stiffness reduction ξ was assumed to be 1.0. The force-displacement ductility backbone curves for the existing frames are shown in Fig. 3.

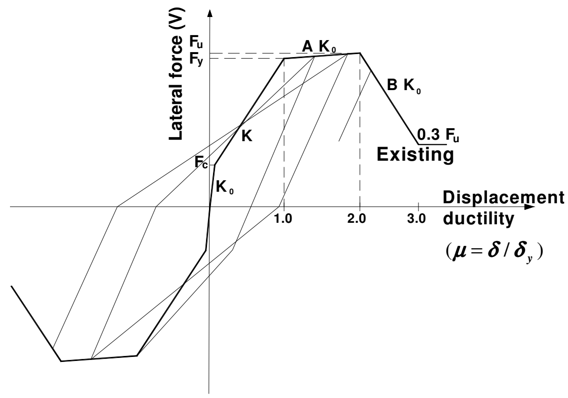


Figure 3. Lateral force-displacement ductility relationships for the columns and beams.

2- Modeling of FRP bracing: (Özcebe, *et al.* 2003) proposed an analytical representation to predict the behaviour of masonry-infilled frames when rehabilitated with FRP bracings. The analytical model was correlated to experimental tests carried out at Middle East Technical University (METU) on a number of two-storey masonry-infilled frame specimens rehabilitated with different patterns of FRP, and subjected to cyclic displacement excitations at the storey levels. A similar uniaxial model for the masonry infill and FRP bracings was used in the current study. The FRP bracings were modeled as uniaxial tension strut with maximum axial strain of 0.003 and maximum axial stress of 190 MPa, these values takes into account the characteristics of CFRP, infill, and the anchor dowels.

3- Modeling of masonry infill: The unrehabilitated infill panels were modeled as compression struts, the properties of the compression strut was chosen based on (Özcebe, et al.2003), and it was scaled to match the panel dimensions of the studied frames. The tensile resistance of the masonry infill was neglected in the analyses.

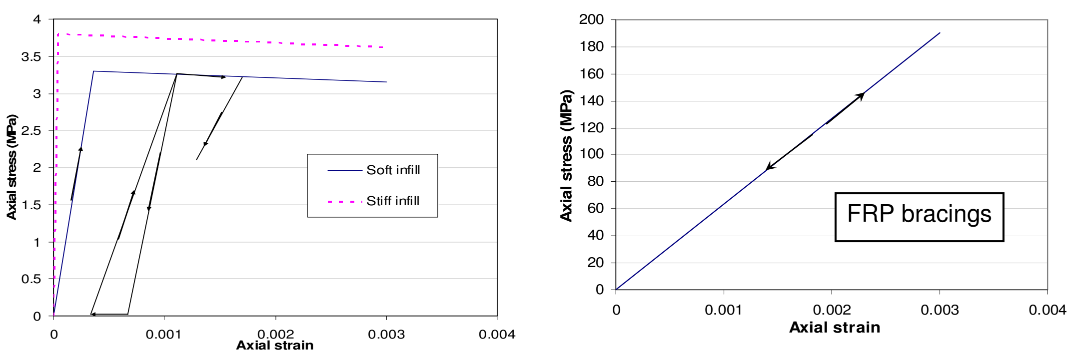
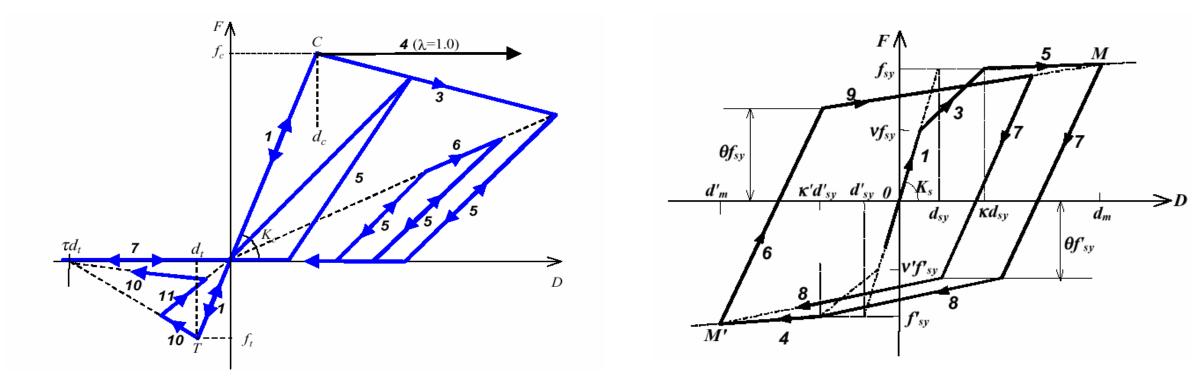


Figure 4. Strut model of masonry infill and FRP bracings.



Concrete material model

Steel material model

Figure 5. Hysteretic models of the concrete and steel fibres for the wall element.

In this study, two different masonry infill types with different stiffness were considered, representing soft and stiff infill, in order to investigate the effect of infill stiffness on the response of structures. Fig. 4 shows the axial stress- axial strain relationship for the tension strut of FRP and the compression strut of the infill.

4- Modeling of RC wall: The RC wall was modeled using CANNY panel element. The panel element has four nodes at the corners in addition to a node at the mid points of the top and bottom boundaries. The adjacent panels have compatible deformations at their common three nodes that are connecting them. Multi-Axial spring model is used to represent the flexural and axial tension/compression interaction of the panel elements. Multi-linear curves are used to represent the force-deformation relationship for steel and concrete springs. In the current analysis linear shear deformation were assumed. Fig. 5 (a) and (b) shows the concrete and steel spring models, respectively. Fig. 6 shows the analytical model for the studied frames.

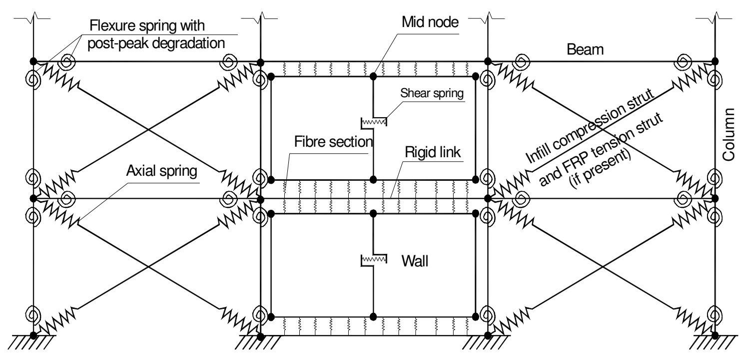


Figure 6. Idealization of the studied frames.

Verification of the analytical model using experimental results
The models used for the masonry infill and FRP bracing were verified using the results of the experimental tests carried out in METU (Özcebe, *et al.* 2003), and the results of the analytical model using CANNY was matching the experimental results for the unrehabilitated and rehabilitated specimens, and for both static push over and dynamic analyses.

b) Analysis results:
1- Dynamic properties of the structures:
Modal analysis is conducted for the 5 and 15 storey frames for different rehabilitation schemes, and 5% damping ratio was considered. Table 2 shows the modal analysis results for the frames.

Table 2. Natural periods for the selected frames

No of Storeys	Bare frame			Soft infill frame			Stiff infill frame		
	Existing	+ FRP bracing	+RC Wall	Existing	+ FRP bracing	+ RC Wall	Existing	+ FRP bracing	+ RC Wall
5	1.01	0.75	0.38	0.39	0.38	0.28	0.22	0.21	0.21
15	2.49	2.06	1.52	1.378	1.36	1.19	1.04	1.04	1.02

2- Maximum applied PGA:
In the analyses, the selected 9 earthquake records with different frequency contents were applied on the 5 and 15 storey existing and rehabilitated frames. The maximum earthquake intensities that can be resisted by the frames were evaluated, and the average values were obtained. Fig. 7 shows the relationship between the earthquake PGA applied and the corresponding maximum inter-storey drift ratio for the 15 storey frame when subjected to the selected ground motion records.

Fig. 8 shows the maximum PGA resisted by different rehabilitation patterns for the 5 and 15 storey frames. It can be seen from the figure that the use of FRP bracings increases the maximum PGA resisted by both the 5 and 15 storey frames, while the introduction of a RC wall has resulted in a higher value of PGA especially for the 5 storey frame.

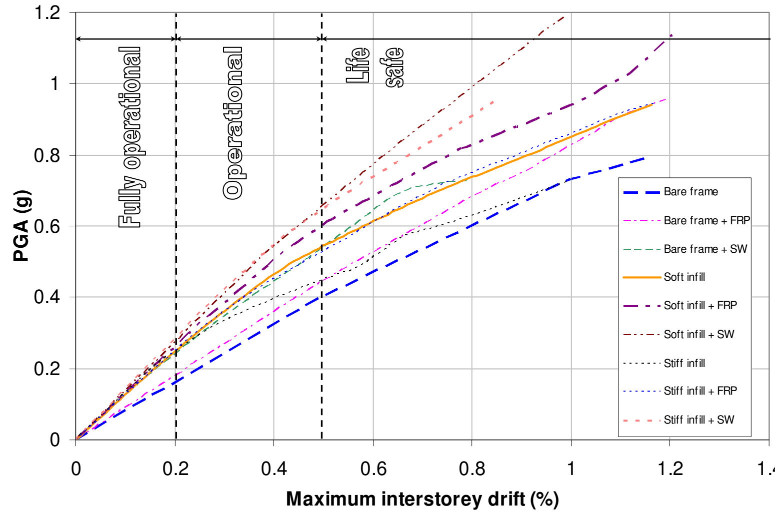


Figure 7. Performance of the 15 storey frame

3- Maximum inter-storey drift:

Fig. 9 shows the average maximum inter-storey drift (I.D.) ratio for the 5 and 15 storey frames for different rehabilitation schemes and different infill types. From the figure, it is noticed that FRP bracings has negligible effect on the maximum I.D. capacity, which matches the experimental test results carried out in (METU 2003). On the other hand, introducing a RC wall resulted in a reduced maximum I.D. capacity. This can be attributed to the fact that adding the RC wall increased the stiffness of the structure. It should be noted that in both rehabilitation schemes, the failure of the structure occurred in the non-ductile columns and beams of the existing structure. The figure also shows the performance levels that represent the damage degree of structure in terms of inter-storey drifts as recommended by (Vision 2000, 1995) and (FEMA 273/274, 1997). Fig. 10 shows the distribution of inter-storey drift ratio along the height of 5 and 15 storey frames when subjected to the scaled El Centro earthquake record. In the figure, the value of maximum inter-storey drift (I.D.) at the shown PGA levels represents the effectiveness of the rehabilitation techniques on the reduction of I.D. ratio, which is regarded as an indicator for the damage state in the structure. The figure shows that, for the studied frames, introducing a RC wall resulted in decreasing the I.D. ratio more than the FRP bracings.

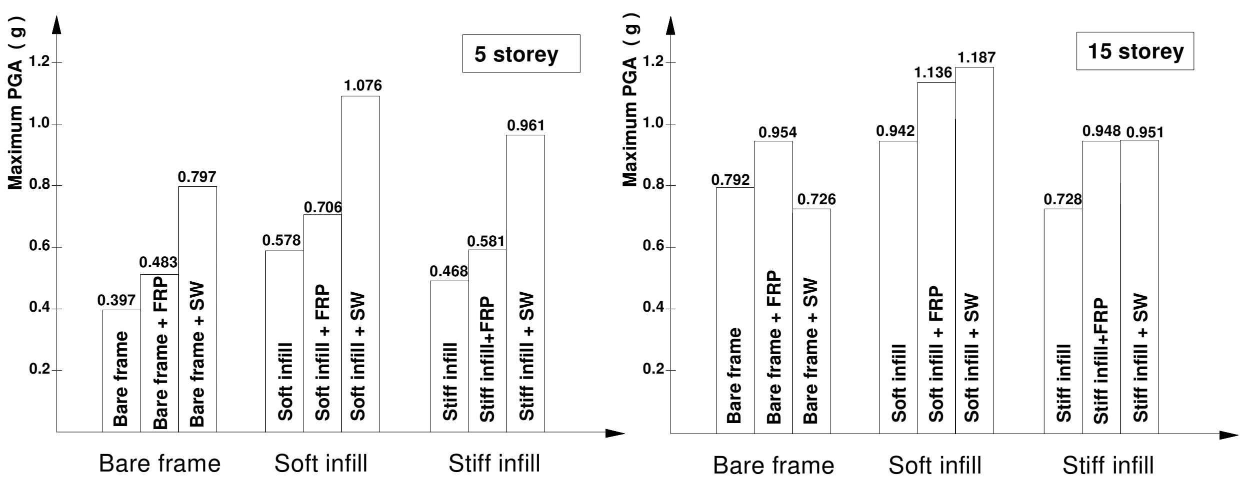


Figure 8. Maximum average PGA resisted by 5 and 15 storey frames.

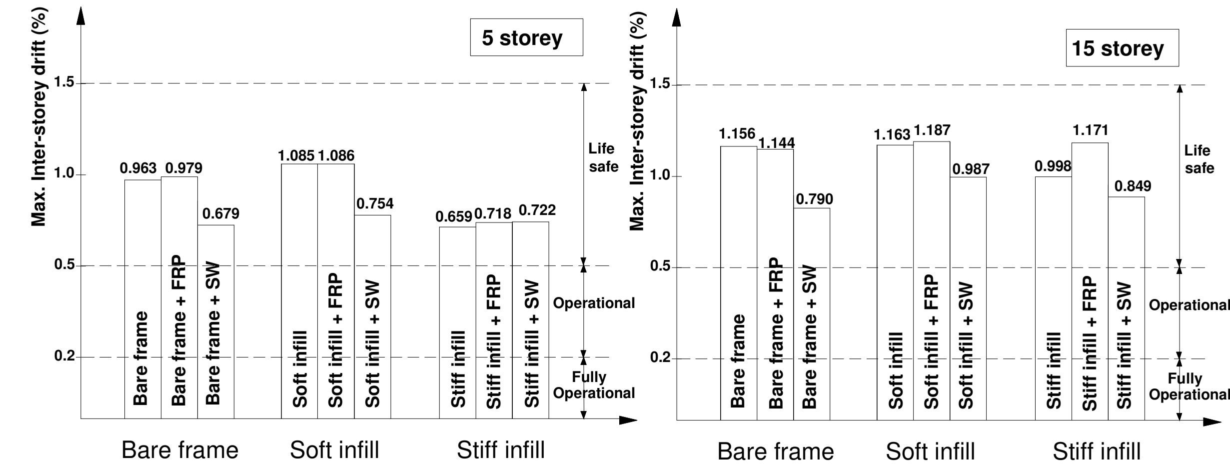


Figure 9. Maximum average inter-storey drift ratio.

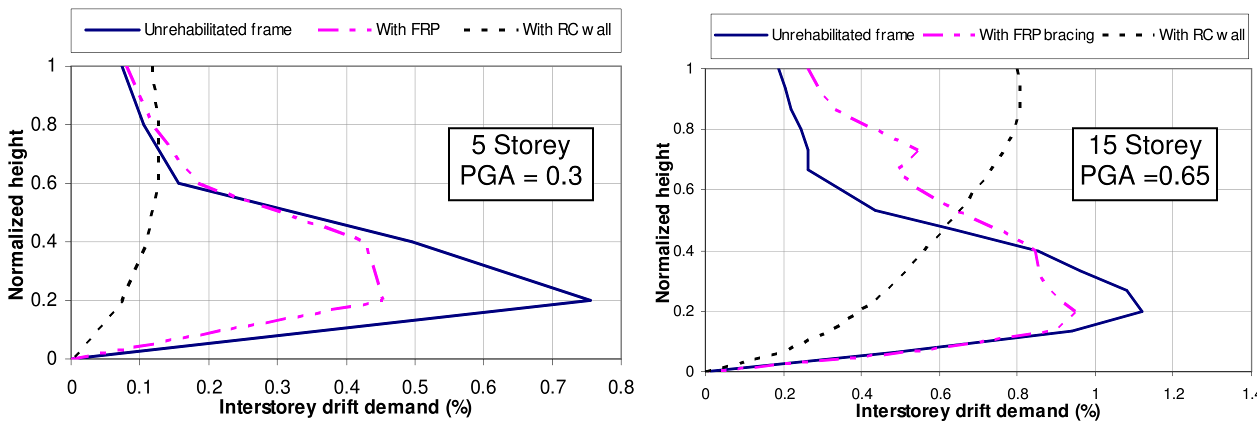


Figure 10. I.D. distribution along the height of 5 and 15 storey frames subjected to scaled El-Centro earthquake record.

<u>4- Maximum storey shear:</u>

Fig. 11 shows the maximum storey shear-to-the total structure weight ratio obtained for the two rehabilitation schemes. It can be seen that introducing the RC wall attracts higher forces due to the increase of stiffness and hence 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.

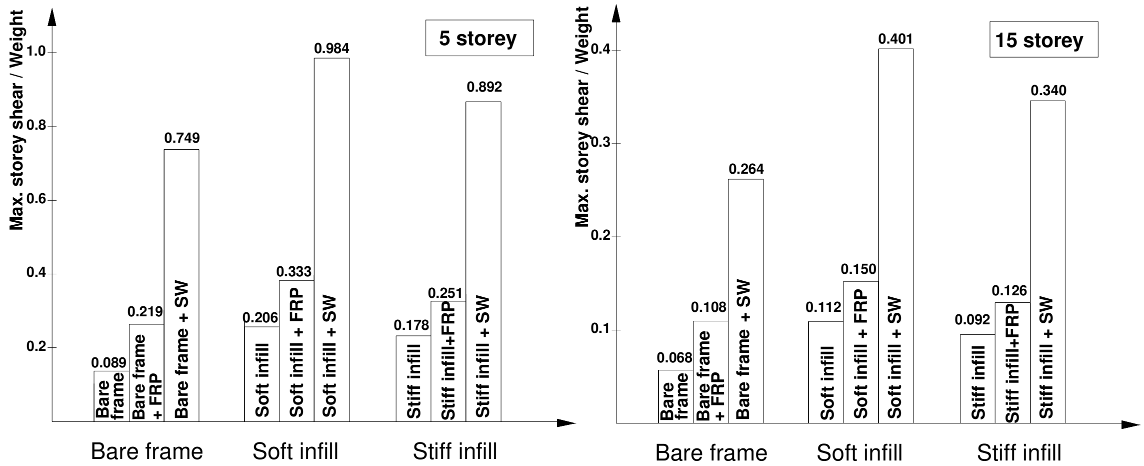


Figure 11. Maximum average storey shear.

5- Energy dissipation capacity:

Energy dissipation capacity is one of the most important indicators of the structure ability to stand against a severe ground motion, and it can be determined from the area enclosed by the hysteretic loops of the load deformation curve. Fig. 12 shows the maximum energy dissipated by the 5 and 15 storey frames for the two rehabilitation patterns and the two infill types. It can be seen that for the both the 5 and 15 storey frames, the RC wall rehabilitated frames dissipated higher energy than the case of FRP. The figure indicates that the introduction of RC wall in the low- and high-rise building will be more efficient in resisting the lateral loads than the use of FRP bracing, yet this solution will be impractical if the building was occupied while rehabilitation, in that case the FRP bracing will be a preferable solution. The figure shows also the effect of stiffness of infill on the response of the frames. It can be seen that the stiff infill leads to higher energy dissipation capacity in case of low-rise building, while in the high-rise building, the stiffer infill resulted in a lower ductility and hence a reduction in the energy dissipation capacity.

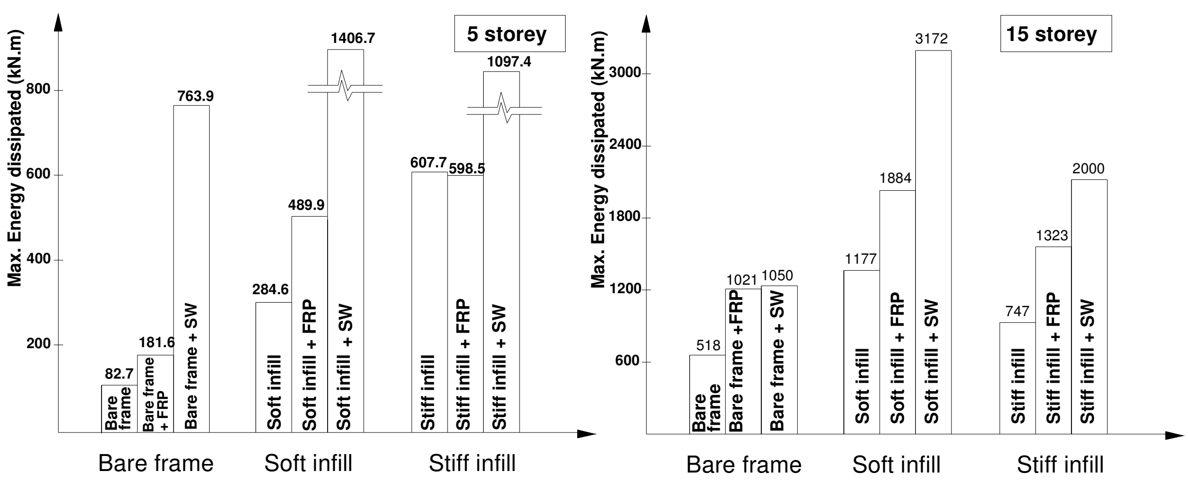


Figure 12. Maximum average energy dissipated at the capacity of each system.

Importance of Inclusion of Infill in the Analysis

Masonry infills can significantly improve the seismic response of structures or, in contrary, they can cause unexpected damages. Therefore the influence of masonry infill on the seismic performance of structures should not be over looked. In this study the selected frames were analyzed for case of neglecting (bare frame) and accounting for the presence of the masonry infill (soft infill and stiff infill), and the response was compared in both cases. From Fig. 6 it can be seen that at the same level of PGA the presence of masonry infill reduces the I.D. which represents a better seismic performance of the structure according to the Vision 2000 criteria. Figures 8, 9, 11 and 12 show the bare frame response compared to that of the masonry infilled frame. It can be seen that the inclusion of infill in the analyses has improved the performance of the 5 and 15 storey frames, especially for the maximum storey shear and 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 columns′ ends.

Conclusions

The effectiveness of FRP bracings in upgrading the seismic behaviour of existing masonry-infilled RC frame structures was evaluated. Nonlinear dynamic analysis of two existing masonry-infilled frames with different heights has been conducted. The frames were strengthened using FRP bracings along the full height of the frames. Two different masonry infill types with different stiffness were considered in the analyses. The bare frames neglecting the effect of masonry infill was also studied. The enhancement in the seismic performance using FRP bracings was compared to that provided by demolishing a masonry infill panel and introducing a RC infill. The analyses were conducted on the existing frames and the rehabilitated ones. The selected ground motion records were selected to represent earthquakes with low, medium and high frequency contents. The seismic performance enhancement of the analyzed frames was evaluated based on the maximum applied PGA resisted by the frames, maximum inter-storey drift (I.D.) ratio, maximum storey shear and energy dissipation capacity. The importance of accounting for the effect of masonry infill on the seismic behaviour of structures was also investigated.

The conducted analyses have resulted in the following conclusions:

1- Rehabilitating masonry-infilled RC frames with FRP bracings will result in a decrease in the maximum I.D. for a given PGA level when compared to the existing frame. However, the FRP bracings have negligible effect on the maximum I.D. capacity of the frame.

2- The frames rehabilitated with FRP bracings were able to dissipate higher energy, especially for the low-rise building, which indicates a higher efficiency in resisting the lateral loads.

3- For the same PGA level, the introduction of a RC wall decreased the value of maximum I.D. significantly compared to the use of FRP bracings.

4- The introduction of a RC wall increases the shear force’s demand due to the reduction in the natural period of the frame.

5- The introduction of a RC wall in the studied frames lead to a higher energy dissipation capacity of the structure compared to the case of using FRP bracing. Yet, it is worth mentioning that FRP bracings are considered to be a more viable intervention, especially when the building is occupied during its rehabilitation where minimum disturbance is required.

6- The presence of masonry infill 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 erroneous under-estimation of the seismic performance of the structure.

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

Acknowledgements

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