Seismic analysis of a RC frame building with FRP-retrofitted infill walls

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ABSTRACT: Although they are not taken into account as load bearing elements, hollow brick infill walls contribute lateral load resistance of existing reinforced concrete buildings, in terms of strength and stiffness. In this study, after a brief summary of an experimental work on reinforced concrete frames with FRP retrofitted infill walls, an existing typical rc frame building characterized by low quality concrete, insufficient confinement of structural members, smooth longitudinal bars, insufficient stiffness and irregular frames was analyzed before and after retrofitting its infill walls using FRP composite sheets. The nonlinear behavior of the building, before and after retrofitting its infill walls with FRP composite sheets, is predicted by push-over analysis. A brief explanation of this retrofitting method as given in the recently published version of Turkish Seismic Design Code is also included as well.

1 INTRODUCTION

Although they are not taken into account as load bearing elements, hollow brick infill walls contribute lateral load resistance of existing reinforced concrete frames, in terms of strength and stiffness. Laboratory tests, as well as on-site observations of structural damages after earthquakes demonstrate the significant contribution of hollow brick infill walls to seismic resistance. For existing structures to benefit from the contribution of infill walls during earthquakes, the walls must be kept in their place and the out-of-plane failure should be prevented. It is clear that any other measure that may enhance the weak tensile properties of the infill walls may further increase the contribution of infill walls to the overall seismic behavior of reinforced concrete frames. For preventing out-of-plane failures and enhancing the tensile characteristics of hollow brick walls, retrofitting the infill walls with fiber reinforced polymer (FRP) composites and connecting infill walls to the reinforced concrete frame using FRP anchorages is a new retrofitting technique. This new retrofitting technique has also been included in the recently published version of the Turkish Seismic Design Code (TSDC 2006).

In this study, after a brief summary of an experimental work on one bay two story reinforced concrete frames with FRP retrofitted infill walls reported by Yuksel et al. 2006 and Erol et al. 2006, an existing typical six story reinforced conrete frame residential building characterized by low quality of concrete, insufficient confinement of structural members, usage of smooth longitudinal bars, insufficient stiffness and irregular frames was analyzed before and after retrofitting its infill walls using FRP composite sheets. The nonlinear behavior of the building, before and after retrofitting the infill walls with FRP composite sheets, is predicted by push-over analysis. A realistic and actually applicable retrofitting scheme is planned during the selection of the infill walls to be retrofitted not to hinder the effective usage of the building. During the analysis, the retrofitted infill walls are represented with diagonal struts, of different characteristics in tension and compression. The stiffness and strength characteristics of these struts used during modeling are taken from TSDC (2006). The push-over behavior of the retrofitted building is then compared with the behavior of original building. It is seen that the investigated retrofit technique, which was proven to be effective experimentally in element basis (Erdem et al. 2006, Yuksel et al. 2006, Binici & Ozcebe et al. 2006, Ozden et al. 2006), is successful in structural basis too. Particularly, by retrofitting the infill walls of reinforced concrete frame structures, it is possible to increase stiffness and lateral strength of the structure, leading to smaller drift and less residual damage during earthquakes. It should be noted that while it depends on the selected retrofitted scheme, the enhancement in lateral strength is more pronounced in the case of current study. Naturally, the increase in stiffness may pose an increase in seismic demand; therefore, similar to other retrofitting techniques, the optimum retrofitting scheme should be sought for adjusting the desired values of strength and stiffness. Besides the significant potential enhancement in stiffness and strength, the easy application of this retrofitting technique is an other major advantage. As expected, while stiffness and strength are enhanced, ductility of the structural system is affected negatively due to higher participation of brittle infill walls to the seismic behavior. However since the ductility of the original building is already very poor, the reduction in ductility has marginal effect on the overall behavior. It should be noted that the nonlinear analysis of this building for other different retrofitting schemes were investigated elsewhere, (Ilkı et al. 2005a and Goksu et al. 2006).

2 PREVIOUS EXPERIMENTAL WORK

The contribution of FRP retrofitted infill walls to the performance of two story, one bay frames were demonstrated experimentally by (Erdem et al. 2006, Yuksel et al. 2006, Binici & Ozcebe 2006 and Ozden et al. 2006). In the study carried out by Yuksel et al. (2006), six reinforced concrete frames including two bare, two infilled frames and two frames with FRP retrofitted infill walls were tested under constant axial load and reversed cyclic lateral loads. The idea was to understand the behavior of FRP retrofitted infilled frames experimentally and collect data to be used in theoretical work. At the end of the tests, it was seen that retrofitting of infill walls with FRP composites in diagonal direction provided significant enhancement in lateral strength and stiffness.

Figures 1 and 2 are the photographs from experimental work carried out in Istanbul Technical University through a joint project with Middle East Technical University and Bogazici University under a NATO Science for Peace Project. As it can be seen in Figure 2, diagonal FRP on both sides of infill were connected to each other by means of anchors made of FRP sheets and FRP diagonals helped the infill wall to be intact even after a considerable damage. Therefore dissipating significant amount of energy, the infill walls may provide an excellent damping effect against the seismic actions. Base shear versus lateral displacement envelopes from the experimental work is given in Figure 3.

As it can be seen in Figure 3, even the frame with infill walls without any retrofit perform better than the bare frame due to significantly increased lateral strength. Introduction of FRP diagonals further increases the lateral strength as well as providing a less steep descending branch in the base shear–displacement relationship.

Figure 1. General appearance of the specimen and the testing setup (Erol et al. 2006).

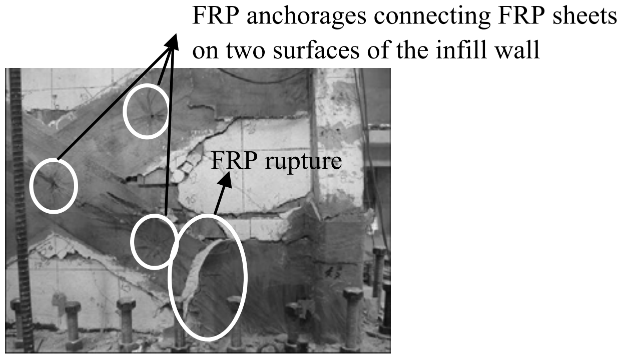


Figure 2. A retrofitted specimen after experiment.

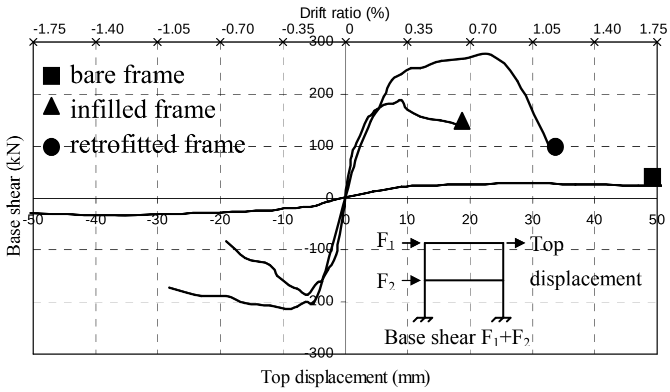


Figure 3. Base shear versus lateral displacement envelopes from experimental work (Yuksel et al. 2006).

3 OUTLINE OF THE EXISTING BUILDING

The nonlinear behavior of a typical six story reinforced concrete building is investigated by pushover analysis before and after retrofitting. The appearance of the building is given in Figure 4.

The reinforced concrete frame building, which was constructed around 1970s, represents all deficiencies of typical reinforced concrete buildings in Turkey. The building is located in Anatolian part of Istanbul on the highest seismic risk zone and on stiff rock, Figure 5.

Figure 4. Appearance of the existing building.

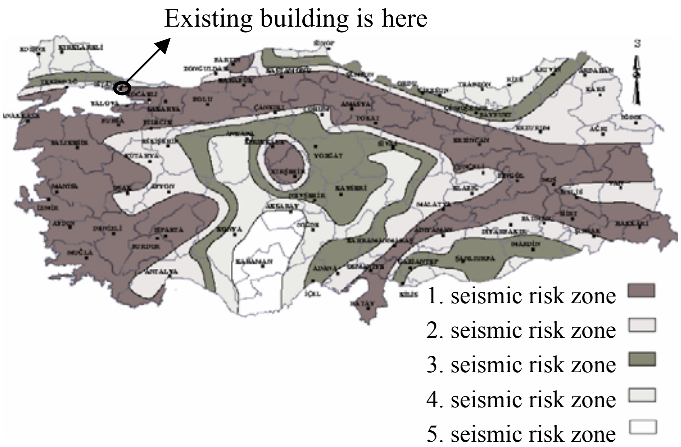


Figure 5. The distribution of seismic risk in Turkey.

The typical floor plan of the building is presented in Figure 6.

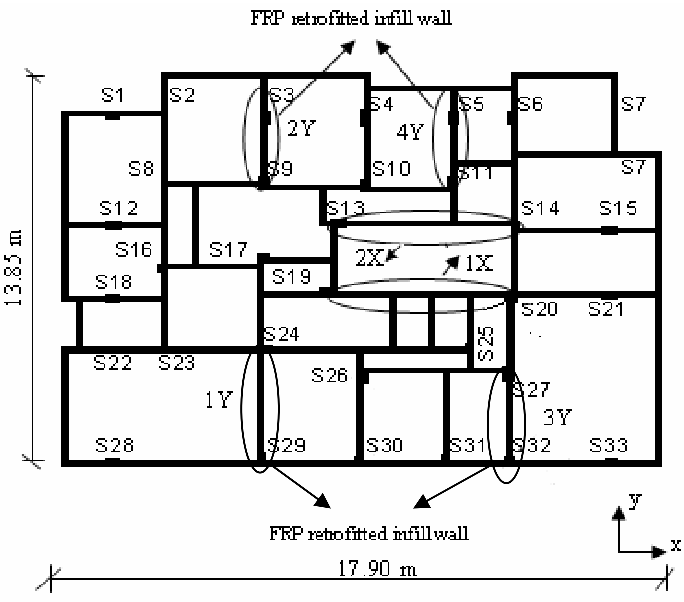


Figure 6. Typical floor plan of the existing building.

All columns are rectangular in cross-section as shown in Figure 6. The structural system is not symmetric in any of the principal directions, many columns are not connected to each other by beams, and the columns and their orientations are not distributed evenly. Cross-sections of beams are 150 mm × 600 mm and cross-sections of columns vary from 240 mm × 240 mm to 240 mm × 600 mm. In addition to these irregularities, the characteristic compressive strength of concrete is as low as 10 MPa, which is a commonly accepted mean value for relatively old existing reinforced concrete structures in Turkey. Both longitudinal and transverse reinforcement are plain bars with characteristic yield strength of 220 MPa. The transverse reinforcement of the original structure, consisting of 6 mm bars at 300 mm spacing is far from maintaining an adequate confinement required for a ductile behavior.

According to the results of elastic analysis carried out considering the TSDC (1998), the lateral drifts exceed the prescribed limits (relative drifts should be less than 0.0035 and 0.02/R, where R is the seismic load reduction factor based on the ductility and over strength of the structural system) and almost all of the columns are found to be inadequate in terms of flexure in both principal directions. Since lateral stiffness of the structure is quite low due to small cross-sectional areas of columns, poor connectivity of the columns with beams and low concrete quality, the periods of first two modes are found as 1.13 and 1.05 seconds for x and y directions, respectively.

According to TSDC (1998) and TSDC (2006), the design horizontal acceleration is 0.4g for zones with such high seismicity. While determining the equivalent static seismic load, the load reduction factor due to ductility and over strength is taken into account as 4, as mostly done in practice for this type of existing reinforced concrete frame structures. Base shear coefficients can be determined as 0.087 and 0.092 for x and y directions considering the periods of original structure (T_x = 1.13 sec, T_y=1.05 sec). It should also be noted that high level of axial stresses on columns reduces the ductility.

The equivalent static seismic base shear force according to TSDC (1998) and TSDC (2006) is calculated by Equation 1:

$$V_t = \frac{W A_o I S(T)}{R_a(T)} \qquad (1)$$

where V_t = base shear force (in this case 1560 kN and 1658 kN for x and y directions, respectively); W = total weight of the structure considering the live load reduction factor (18000 kN for this case with live load reduction factor of 0.3); A_o = effective ground acceleration coefficient (0.4 in this case); I = building importance factor (1 for this case); $S(T)$ = spectrum coefficient (0.87 and 0.92 for x and y directions for this case); $R_a(T)$ = seismic load reduction factor (4 for this case). The spectrum coefficient can be calculated as shown in Figure 7.

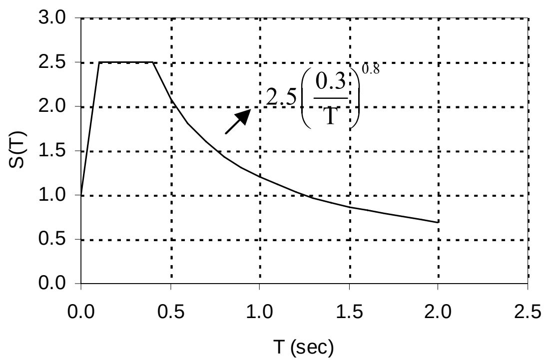


Figure 7. Calculation of spectrum coefficient according to TSDC (1998) and TSDC (2006) for Z_1 type very stiff soil.

4 RETROFITTING SCHEME

Totally 6 infill walls are retrofitted using FRP diagonals, 2 in x direction and 4 in y direction. The retrofitted infill walls which do not have any openings are shown in Figure 6.

The FRP diagonals are assumed to be applied over these infill walls without removing the plasters as also done by Yuksel et al. (2006) in the experimental study. The length, height and thickness of the retrofitted infill walls are shown in Table 1.

Table 1. Dimensions of retrofitted infill walls.

All stories	l mm	h_m mm	t_m mm
1X	5750	2200	200
2X	5650	2200	200
1Y	4100	2200	200
2Y	2400	2200	200
3Y	2760	2200	200
4Y	2450	2200	200

where l = length of retrofitted infill wall; h_m = height of retrofitted infill wall; t_m = thickness of retrofitted infill wall.

It is assumed that one ply of FRP sheets are applied over the infill walls in diagonal directions on both faces and FRP sheets are extended over the frame members and sufficiently anchored to them. The width of the FRP sheets is 400-500 mm as a function of the effective width of the compression strut of the infill wall. The properties of FRP sheets are shown in Table 2.

Table 2. Properties of FRP sheets (given by the manufacturer).

Fiber type	t_f mm	T_f N/mm^2	E_f N/mm^2	ε_f %
Carbon	0.12	4100	231000	1.7

where t_f = effective thickness of fabric; T_f = tensile strength of FRP sheet; E_f = tensile elastic modulus of FRP sheet; ε_f = rupture strain of FRP sheet.

5 ANALYSIS

5.1 Outline of recommendations of TSDC (2006)

The most recent version of TSDC (2006) permits retrofitting of reinforced concrete frame buildings using FRP composites through strengthening the infill walls in between the frame members when the ratio of wall length/height is between 0.5 and 2. The connection of retrofitted infill walls to the surrounding reinforced concrete frame is essential since the FRP reinforcement has to prevent the out-of-plane failure of the infill walls. Consequently, the contribution of infill walls to the seismic capacity can be maintained as well as the contribution of FRP reinforcement. In this technique, the retrofitted infill walls are assumed to behave as diagonal compression struts, while FRP sheets are assumed to act as diagonal tension struts. The schematic view and the static model of a retrofitted infill wall are shown in Figures 8 and 10, respectively.

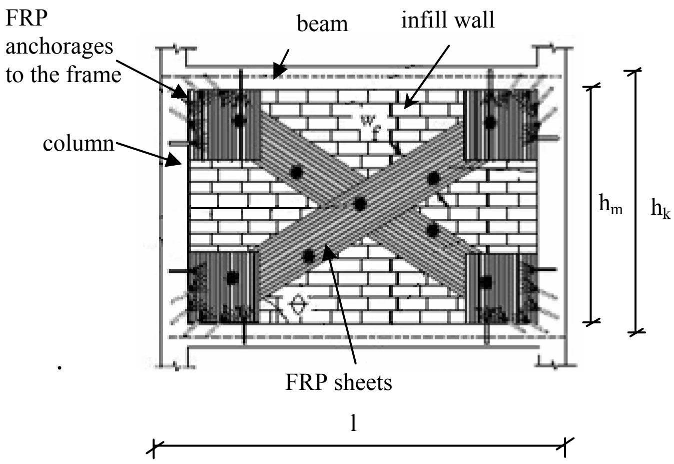


Figure 8. The schematic view of a retrofitted infill wall (Al-Chaar et al. 2002 and TDSC 2006).

As seen in Figure 8, the FRP composites on both faces of the wall should be connected to each other using the anchors made of FRP sheets. The spacing between these FRP anchors should be less than 600 mm. The connection of the retrofitted infill wall to the surrounding reinforced concrete frame is also to be made by means of FRP anchors as shown in Figure 9.

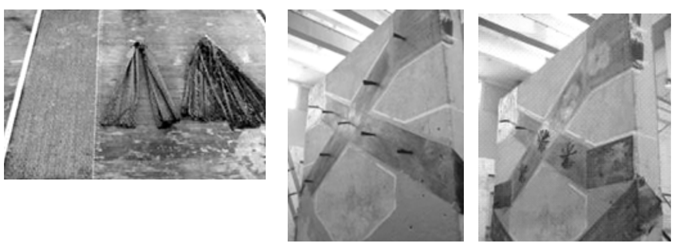


Figure 9. FRP anchors connecting the diagonal FRP sheets on two sides of the wall (Yuksel et al. 2006).

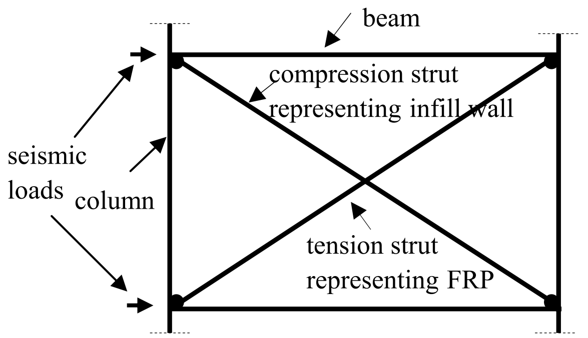


Figure 10. Static model of retrofitted infill walls.

According to Al-Chaar et al. (2002) and TSDC (2006) the effective width of the compression strut can be calculated by Equation 2:

$$a_m = 0.175(\lambda_m h_k)^{-0.4} r_m \tag{2}$$

where h_k = the height of column; and r_m = the diagonal length of infill wall (mm).

The λ_m coefficient can be obtained by using Equation 3:

$$\lambda_m = \left[\frac{E_m t_d \sin 2\theta}{4 E_c I_k h_m}\right]^{0.25} \tag{3}$$

where E_m = the elasticity modulus of infill wall (MPa); E_c = the elasticity modulus of concrete (MPa); h_m = the height of retrofitted infill wall (mm); I_k = moment of inertia of column (mm^4) and θ = angle of diagonal sheets with respect to the horizontal (degree).

The tensile strength of the tension strut is to be calculated by Equation 4:

$$T_f = 0.003 E_f w_f t_f \tag{4}$$

where E_f = the elasticity modulus of FRP sheet; w_f = the width of FRP sheet; t_f = the effective thickness of FRP sheet.

Dimensions of the compression and tension struts and mechanical properties of the infill are given in Table 3.

Table 3. Properties of retrofitted infill walls.

All Stories	a_m mm	w_f mm	t_m mm	E_m N/mm^2	f_m N/mm^2
1X	883	500	200	550	1
2X	741	500	200	550	1
1Y	561	500	200	550	1
2Y	432	400	200	550	1
3Y	401	400	200	550	1
4Y	462	400	200	550	1

where a_m = the equivalent width of compression strut, w_f = the width of FRP sheet (should be less than a_m according to TSDC (2006)); E_m = the elasticity modulus of infill wall taken as $550 f_m$ (FEMA356 2000); f_m = the compressive strength of the infill wall.

5.2 Assumptions for the analyzed building

Three pushover analyses are carried out for the examined building. The first analysis was carried out for original bare frame structure. In the second analysis, the infill walls, which are to be retrofitted, are included in the structural model without any FRP retrofit. In the third analysis, the FRP retrofit of these infill walls were also taken into account.

Although the TSDC (2006) permits retrofitting of the infill walls, when the length/height ratio is between 0.5 and 2, since the architecture of the analyzed building was not convenient, the length /height ratio of the walls retrofitted in x direction was slightly out of the permitted ranges (between 2.48 and 2.61). The infill walls, which are retrofitted using FRP sheets in diagonal directions, are shown in Figure 6. During the nonlinear seismic analysis, the behavior of FRP sheets are modeled as tension struts with nonlinear axial hinges at their connections to the reinforced concrete frame. Hinge length has been accepted as half of the length of the tension strut for each hinge. The immediate occupancy (IO), life safety (LS) and the collapse prevention (CP) levels of FRP diagonals are set at tensile strains of 0.00295, 0.0030 and 0.0032, respectively. All three levels are set so close to each other because of very brittle nature of FRP sheets. It should be noted that ultimate tensile strain for FRP sheets given by TDSC (2006) for such applications is 0.003. The stress-strain relationship used for the nonlinear axial hinges representing FRP sheets in tension is shown in Figure 11. The detailed information on the linear elastic stress-strain relationship can be found elsewhere (fib, 2001).

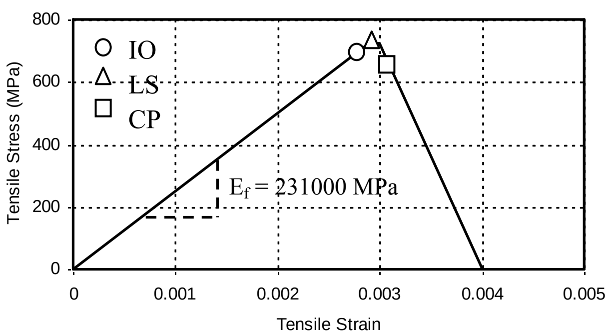


Figure 11. Stress-strain relationship assumed for the FRP diagonals in tension.

The infill walls are also modeled as diagonal members. However, infill walls are assumed to resist only compressive forces. The behavior of diagonal compression struts formed by infill walls are modeled by axial nonlinear hinges at their connections to reinforced concrete frames. Hinge length has been accepted as half of the length of the compression strut for each hinge. Stress-strain relationships of the infill walls are modeled using the relationship given in Figure 12. The IO, LS and the CP levels of infill walls are set as 0.002, 0.003 and 0.004, respectively.

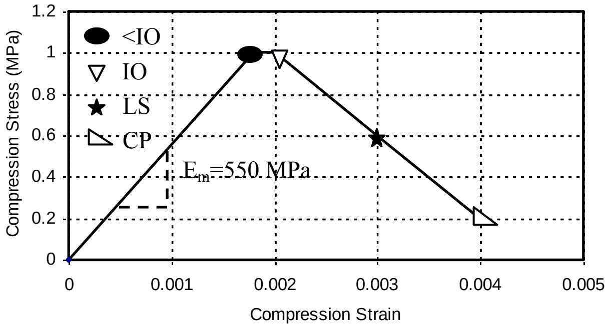


Figure 12. Stress-strain relationship for the infill wall in diagonal compression.

Application of the equations given in TSDC (2006) for a typical retrofitted infill wall of the building, namely retrofitted wall 1Y is done as follows:

$$\lambda_m = \left[\frac{550 \cdot 200 \cdot \sin 2(28.71)}{4 \cdot 24277 \cdot 54.10^7 \cdot 2200}\right]^{0.25} = 0.0009 \text{ mm}^{-4}$$

$$a_m = 0.175(0.0009 \cdot 2900)^{-0.4} \cdot 4788 = 561 \text{ mm}$$

Plastic hinge characteristics of reinforced concrete columns and beams are obtained through cross-sectional moment-curvature analysis using fiber approach. The details of this approach can be seen elsewhere (Bedirhanoglu & Ilki 2004). While converting the moment-curvature relationship into moment-rotation relationships, plastic hinge lengths of columns and beams are assumed as half of the member depth. The details of moment-curvature analysis of the members of original structure can be found elsewhere (Ilki et al. 2005a, b). Moment-rotation relationship of a typical column is presented in Figure 13. The IO, LS and the CP levels for this column is set to 0.0022, 0.0032 and 0.0096, respectively.

It should be noted that the contribution of FRP reinforcement in compression and the contribution of infill walls in tension are neglected.

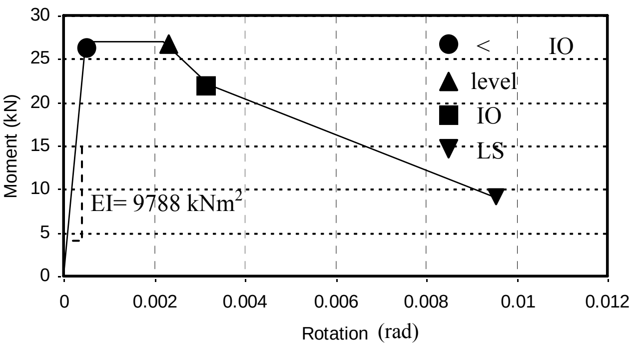


Figure 13. Moment-rotation relationship assigned for a typical column (S28 column-basement).

5.3 Analyses results

The top displacement-base shear relationships obtained by push-over analysis for original bare frame, frame with infill walls and frame with retrofitted infill walls in x and y directions are presented in Figure 14. In this figure, the design base shear forces calculated according to TSDC (1998 and 2006) are also plotted. While calculating the design base shear forces, the seismic load reduction factor is taken into account as four. It should be noted that while analyzing the frame with infill walls, only the infill walls, which are to be retrofitted are included in the model for determining the contributions of infill walls and FRP diagonals separately. The other infill walls, which are not retrofitted, are not included in the model assuming that they may prematurely fail due to out of plane effects. Summary of the analyses for all cases is given in Table 4.

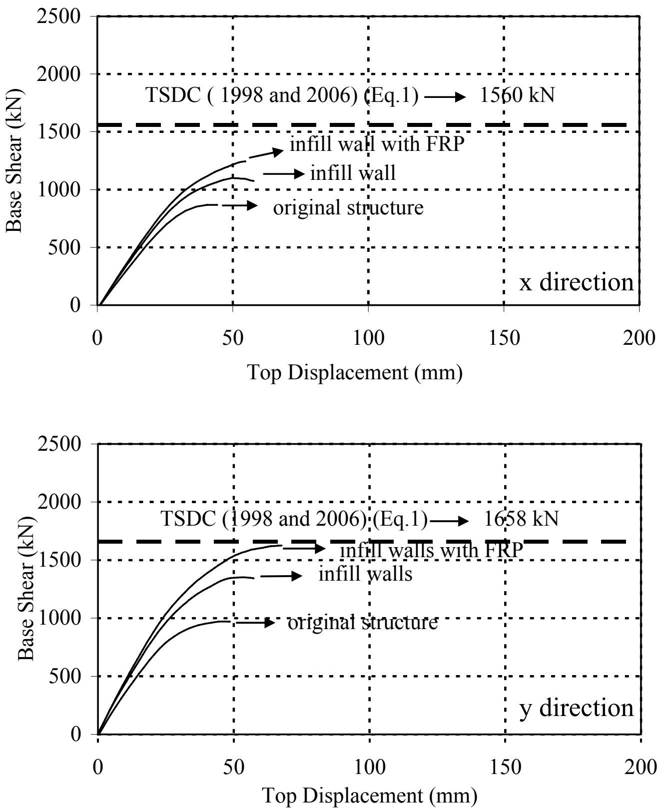


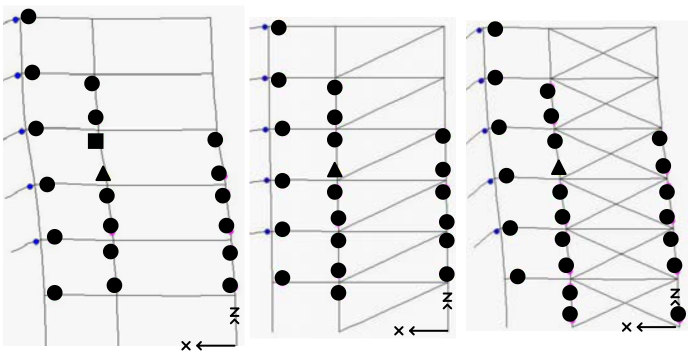
Figure 14. Base shear-top displacement relationships for original and retrofitted structures

Considering the results of the analyses, it is seen that the contribution of infill walls significantly increases the stiffness and lateral load capacity of the structure. Still being below the base shear demand required by TSDC (1998 and 2006), usage of FRP diagonals extends this base shear capacity increase to a further point by increasing the tensile capacity of the infill walls.

Table 4. Summary of the analysis.

	Bare frame	Infilled frame	Retrofitted frame
Base shear capacity (x direction)(kNm)	872	1101 (26% increase)	1243 (43% increase)
Ultimate disp. (x direction)(mm)	46	58 (26% increase)	55 (20% increase)
Period (x direction)(sec)	1.13	1.07 (4% decrease)	1.05 (6% decrease)
Base shear capacity (y direction) (kNm)	970	1351 (39% increase)	1622 (67% increase)
Ultimate disp. (y direction)(mm)	49	57 (16% increase)	65 (33% increase)
Period (y direction)(sec)	1.05	0.98 (6% decrease)	0.95 (9% decrease)

As seen in Figure 15, the structure with retrofitted infill walls can resist a higher base shear force with a relatively less damage at the ultimate displacement capacity (global drift ratio of 0.28%) of the original structure. Cross-sections of the columns of external frames and the columns of the third and fourth stories tend to experience larger deformations since the columns of these regions have smaller cross-section areas.



(a) Bare frame (b) Infilled frame (c) Infill walls retrofitted with FRP

δ = 44 mm $V_{resisted}$ = 885 kN

δ = 44 mm $V_{resisted}$ = 1064 kN

δ = 44 mm $V_{resisted}$ = 1177 kN

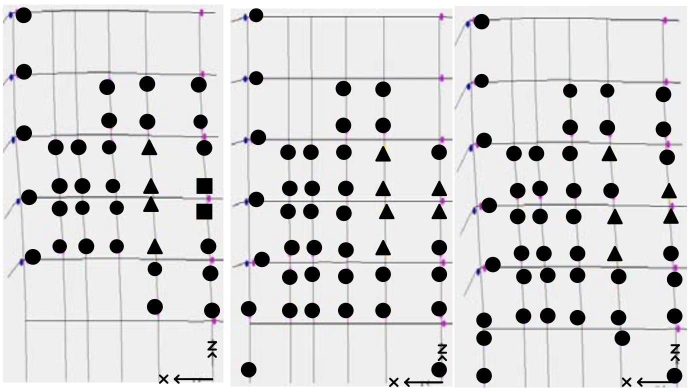
●: < IO level for columns
▲ : >IO and <LS levels for columns
■ : >LS and <CP levels for columns
⬬ :< IO level for infill walls (compression strut)

Figure 15. Damage mechanisms of a retrofitted frame at the ultimate displacement capacity of the original structure (global drift ratio of 0.28%).

where δ = top displacement; $V_{resisted}$ = resisted base shear force.

Since the stiffness of the retrofitted frames has increased considerably compared to the other frames, the distribution of the plastic hinges is not uniform throughout the structure. Consequently, larger internal forces are exerted to the columns and beams of the retrofitted frames while almost all critical sections of the remaining frames experience less damage. Damage distribution of an unretrofitted frame axe at the global drift ratio of 0.28% is presented in Figure 16.

Damage mechanisms at ultimate displacement capacities of the structure with infilled frames and the structure with FRP retrofitted infilled frames are also shown in Figure 17. As seen in the given representative frames in Figure 17, the structure with retrofitted infill wall, experience less damage with respect to the structure with unretrofitted infill walls.



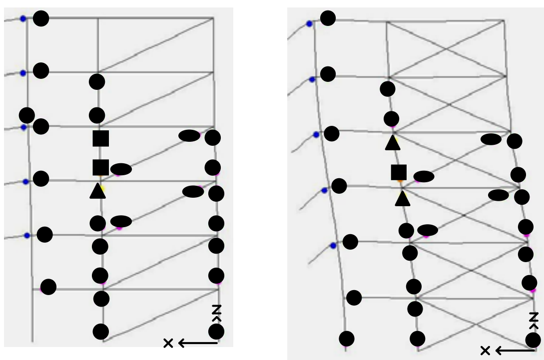
(a) Bare frame (b) Infilled frame (c) Infill walls retrofitted with FRP

δ = 44 mm $V_{resisted}$ = 885 kN

δ = 44 mm $V_{resisted}$ =1064 kN

δ = 44 mm $V_{resisted}$ = 1177 kN

Figure 16. Damage mechanism of an unretrofitted frame at the ultimate displacement capacity of the original structure (global drift ratio of 0.28%).



(a) Infilled frame (b) Frame with FRP retrofitted infill wall

δ = 57 mm $V_{resisted}$ = 1074 kN

δ = 54 mm $V_{resisted}$ = 1243 kN

Figure 17. Damage mechanism at the ultimate displacement capacities of the analyzed cases with infills and retrofitted infills.

6 CONCLUSIONS

Attempting to analyze the non-linear behavior of a typical existing reinforced concrete structure with various deficiencies, retrofitted with an experimentally verified retrofitting technique, the following conclusions are reached. The investigated retrofitting technique aims to benefit from the existing infill walls against seismic actions, by keeping them in place and preventing out of plane failure by using diagonal FRP sheets applied on both sides of the infill walls as well as introducing a diagonal tension strength to the infill walls through FRP sheets. During the design process of retrofitting the formulations and details mandated by TSDC (2006) were followed.

In order to investigate the effect of infill walls on the overall behavior, two different cases apart from the original reinforced concrete frame structure; namely the frame structure only with infill walls and the frame structure with infill walls retrofitted by FRP diagonal sheets, were analyzed.

Nonlinear push-over analysis showed that the lateral strength and stiffness of the structure with infill walls increased significantly due to the contribution of the infill walls, which are generally neglected due to their tendency to premature strength loss during seismic events. Structural analysis of the structure with FRP retrofitted infill walls exhibited even more increase by means of lateral strength and stiffness, resulting with a better structural performance due to the additional tensile capacity of the FRP sheets.

As a result of the increased stiffness, it is clear that the lateral drifts are limited with respect to the original structure, which in turn limits the residual damage as well. It should be noted that, the alteration of the dynamic characteristics of the structure due to the presence of retrofitted infill walls should be handled carefully since the increase in stiffness may cause an increase in the seismic demand as well.

The investigated retrofitting technique is an easy to apply and occupant friendly technique, which causes less disturbance than many of the available retrofitting techniques. However, it should also be noted that the number of walls suitable for retrofitting is generally limited by architectural reasons such as balconies and door or window openings in the infill walls.

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