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FRP versus traditional strengthening on a typical mid-rise Turkish RC building

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Abstract. This paper investigates the limits and efficacies of the Fiber Reinforced Polymer (FRP) material for strengthening mid-rise RC buildings against seismic actions. Turkey, the region of the highest seismic risk in Europe, is chosen as the case-study country, the building stock of which consists in its vast majority of mid-rise RC residential and/or commercial buildings. Strengthening with traditional methods is usually applied in most projects, as ordinary construction materials and no specialized workmanship are required. However, in cases of tight time constraints, architectural limitations, durability issues or higher demand for ductile performance, FRP material is often opted for since the most recent Turkish Earthquake Code allows engineers to employ this advanced-technology product to overcome issues of inadequate ductility or shear capacity of existing RC buildings. The paper compares strengthening of a characteristically typical mid-rise Turkish RC building by two methods, i.e., traditional column jacketing and FRP strengthening, evaluating their effectiveness with respect to the requirements of the Turkish Earthquake Code. The effect of FRP confinement is explicitly taken into account in the numerical model, unlike the common procedure followed according to which the demand on un-strengthened members is established and then mere section analyses are employed to meet the additional demands.

Keywords: seismic retrofit; FRP strengthening; RC jacketing; confinement effect; deformation capacity

1. Introduction

Earthquakes pose a common threat for many European and Mediterranean countries, where the building stock is exposed to earthquake events of different magnitude, many of which have been proven quite destructive. The prohibitive cost of substituting all structures that suffered at least moderate damage, in conjunction with associated legal issues and complicated bureaucratic procedures, is the driving force for owners or authorities to proceed with strengthening of the building so that it meets the standards of safety set by the relative codes.

Several solutions have been developed for seismic strengthening of existing RC frame structures, usually based on conventional material and construction techniques. However, in the last years, due to the increased demands for ductility and durability and in a continuous effort to

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reduce the time and application cost, new techniques and materials have emerged offering comparatively advantageous solutions, especially in cases in which the architectural limitations govern.

Composite materials are commonly used in practice in repair and strengthening works due to their competitiveness in speed, cost, low profile and ease in application. Supported by advances in material science, the area of composite materials has shown a great improvement in the last two decades, leading thus to more developed and easy engineering applications in the most complicated real life problems. FRP and other relevant materials, such as ordinary and special epoxies, fire protection systems, blast and impact preserving systems, anchors and finishing materials, are widely used, in both the interior and exterior of structures, in ordinary or aggressive environments with high level of confidence in terms of strength and durability. Research and development studies on this field have assisted engineers to be more confident with possible uses of those materials and aware of their limitations.

In this context, this paper investigates the efficiency of FRP strengthening of columns and beams of an ordinary low-rise RC structure in a highly seismic area in order to define whether the FRP solution can be applied as a stand-alone method of strengthening or not.

2. Research significance

2.1 Aim of presented research

In this paper the merits of two different methods of strengthening, i.e. column jacketing and FRP application, are evaluated through the employment of a characteristic RC building commonly met in Turkey that needs to be demolished unless retrofitted in accordance to the current code requirements in vigor. The focus is placed on (i) assessing a RC frame structure representing a building category, numerous similar buildings of which have been exposed to several earthquakes, (ii) applying FRP and traditional strengthening methods to reach a specific performance level, and (iii) comparing the FRP solution with the conventional one in terms of technical features and feasibility through employment of the state-of-the art in FRP strengthening of RC structures against seismic actions.

Unlike done in this study, the effect of FRP is not represented in numerical models. Instead, a numerical model is created, the demand is established on un-strengthened members, and followed by section analyses employed to meet the additional demands. Number and configuration of FRP layers are determined in this way, but not placed into the numerical model to back-calculate the effect of FRP in the local and overall response and consequently confirm or revise if needed the initial estimation. In this study, the effect of FRP confinement is explicitly taken into account in the numerical model with its effect directly defined during the analysis steps.

2.2 Experiments on FRP-confinement under axial loading

The state-of-the art research on FRP confinement of RC members primarily centers on confinement of FRP-wrapped RC members that are axially loaded. Concrete samples are not even reinforced in some of the tests, while few published experiments focus on concrete confinement effect when the member is simultaneously under flexural and axial loading (see references below).

There has been extensive research on FRP-wrapped cylinder specimens, or on concrete filled

FRP tubes under axial loads by Nanni and Bradford (1995), Karbhari and Gao (1997), Samaan *et al*. (1998), Demers and Neale (1999), Toutanji (1999), Saafi *et al*. (1999), Fam and Rizkalla (2001), Becque *et al*. (2003), Lin and Liao (2004), Au and Buyukozturk (2005), Ilki *et al*. (2006).

Regarding the tests on columns of rectangular or square section, the work by Rochette and Labossiere (2000), Wang and Restrepo (2001), Shehata *et al*. (2002), Ilki *et al*. (2004), Rocca *et al*. (2006), Campione (2006) is referred. Attention is drawn to the specimens tested by Ilki *et al*. (2004), the compressive strength of which was around 10 MPa, a parameter that is important for the Turkish building stock investigated in this paper. In more recent studies, FRP-wrapped rectangular RC members with conventional and externally applied reinforcement of different types (Realfonzo and Napoli 2009, Bournas and Triantafillou 2013) have been tested too.

2.3 Design-oriented stress-strain models

Several stress-strain models to be used in design were proposed by researchers such as Xiao and Wu (2000), Ilki and Kumbasar (2002, 2003), Lam and Teng (2002, 2003a, b), Mandal *et al*. (2005). De Lorenzis and Tepfers (2003), however, state that none of the available models could predict the strain at peak stress with reasonable accuracy. Bisby *et al*. (2005) evaluated and modified available analytical models for confined concrete in order to achieve best fit to the experimental database. Matthys *et al*. (2005) tested large-scale cylinder columns and observed that most of the available stress-strain models based on small cylinder tests seem to predict the ultimate strength fairly accurately.

The aforementioned models, developed and calibrated based on tested specimens concentrically axially loaded, describe the uniaxial monotonic stress-strain relationship of confined members but do not take into account the different role of confinement in case of an earthquake when only part of the section is under compression while the rest is in tension. Though the concrete behavior, as function of confinement effect, is expected not to be the same as that obtained by testing elements under concentric compression, there is lack of experimental evidence to properly address this potential difference in confinement. The issue was recently addressed by Demir *et al*. (2015), who proposed a new model in addition to the existing available cyclic ones by Shao *et al*. (2006), and Lam and Teng (2009).

2.4 Analysis-oriented stress-strain models

One of the important issues in real-life applications of FRP confinement is the contribution of the existing stirrups into the confinement level combined with FRP-induced confinement. In RC columns, transverse steel reinforcement exists and provides confinement to concrete. When the amount of transverse reinforcement is limited, as in several old structures that do not comply with the modern seismic codes, the confinement provided by steel transverse reinforcement is rather small and can be conservatively ignored (Elsanadedy 2002, Teng *et al*. 2010, Lin *et al*. 2012). For such members, existing stress-strain models, devised for use in FRP-confinement only, can be directly used. For other columns properly designed and built with stirrups per modern codes and standards, the confinement provided by transverse reinforcement is significant and has to be taken into account in analysis (Eid *et al*. 2009, Lee *et al*. 2010, Wang *et al*. 2012, Hu and Seracino 2013).

A significant number of experimental studies have been conducted on FRP-confined concrete members, as explained above, and several stress-strain models have been proposed as design-oriented models according to the classification by Teng and Lam (2004). There are only three existing stress-strain models (i.e., Braga *et al*. 2006, Megalooikonomou *et al*. 2012, Hu and Seracino 2013) strictly analysis-oriented. Among these three models, the models proposed by Braga *et al*. (2006) and Megalooikonomou *et al*. (2012) employ an equation by Kupfer *et al*. (1969) for the lateral-to-axial strain ratio that introduces an upper limit value of 0.5, although the lateral-to-axial strain ratio of confined concrete can be above 0.5 as revealed by numerous tests (Jiang and Teng 2007). Therefore, a recent model has been proposed by Teng *et al*. (2014) in order to overcome this inconsistent limitation.

2.5 Experiments on FRP-confinement under axial loading combined with lateral cyclic loads

Despite the large variety of papers published on confinement effect under axial loading alone, few research papers, listed below in detail, have actually centered on axial and cyclic lateral loads.

Iacobucci *et al*. (2003) tested near full-scale concrete columns with axial load plus cyclic lateral loads. The displacement ductility of the columns was around 3.7 in the un-strengthened case, reaching up to 8.2 in case 1 to 7 layers of FRP confinement were applied.

Specimens with dimensions 150×300×1000 mm, designed for gravity loads, were tested under axial load combined with lateral cyclic loads by Harajli and Rteil (2004), concluding that wrapping the column’s critical zone with a relatively small area of CFRP flexible sheets increased the bond strength and strength capacity of the columns, significantly reduced concrete spalling and bond deterioration in the column’s end zone, and resulted, in effect, in considerable improvement in seismic performance of the columns. However, Harajli and Rteil (2004) do not provide an analytical model for predicting the entailed increase in strength and ductility.

Ghosh and Sheikh (2007) tested 16 columns with dimensions 510×760×810 mm. The columns were detailed with poor lap splices and inadequate transverse confinement reinforcement in the potential plastic hinge regions near beam-column joints, characteristic of pre-1970 design provisions. Their work was directed toward the evaluation of the effectiveness of carbon fiber-reinforced polymer (CFRP) jackets in strengthening and repair of such columns under simulated earthquake loading. The CFRP retrofitting technique was reported to be effective in enhancing the seismic resistance of the columns and resulted in more stable hysteresis curves with lower stiffness and strength degradations as compared with the un-retrofitted columns.

The research summarized above constitutes the basis for the development of confinement models for FRP confined RC members later included in several codes, guidelines and standards around the world. Besides, in practical applications, several RC members have been strengthened based on the aforementioned research work. Nevertheless, how the effect of FRP wrapping of RC members is translated in terms of overall performance in real applications remains to be answered. This very answer is expected to set the limitations for FRP wrapping determining whether it can be a stand-alone solution.

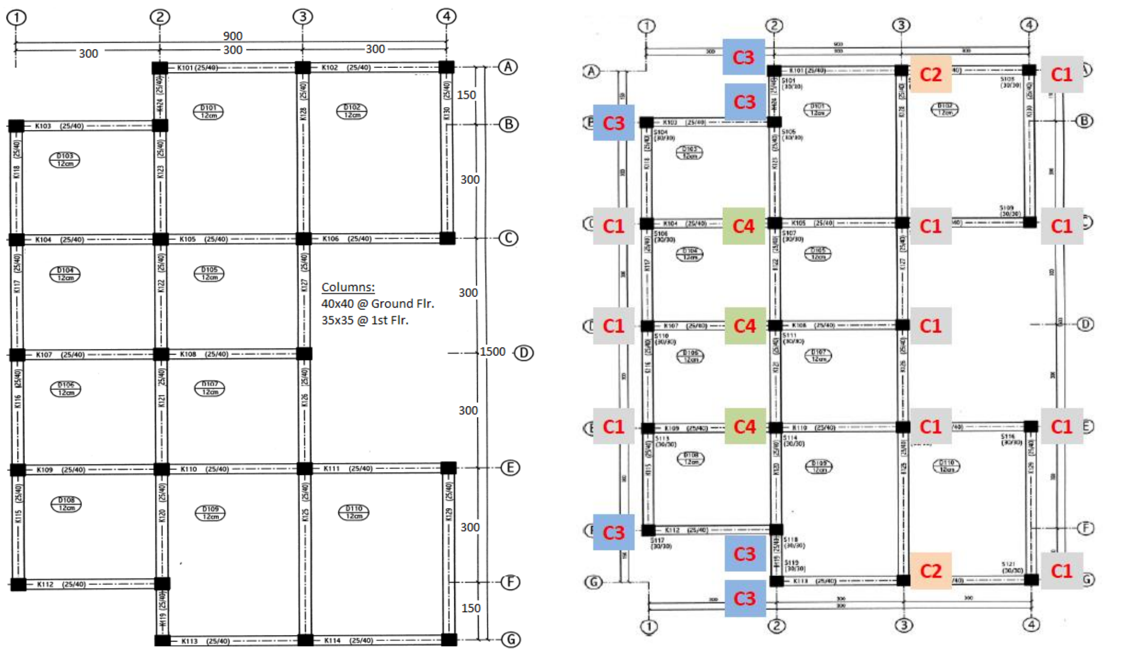
2.6 Shear strengthening of beams by using FRP materials

In an early experimental work by Triantafillou (1998) the use of composites for shear strengthening of beams is investigated providing an analytical model for the design of such members within the framework of modern code format and based on ultimate limit states. More recently Koutas *et al*. (2013) study the effectiveness of spike anchors used in shear strengthening of T-shaped beam sections, concluding that anchors placed inside the slab are several times more effective than those placed horizontally inside the web, and anchors of similar geometrical characteristics (e.g., embedment length) display similar effectiveness despite the difference in fiber type. Finally, Kim *et al*. (2014) report tests on reinforced concrete T-beams externally strengthened with carbon fiber-reinforced polymer (CFRP) laminates and anchors to increase the shear performance suggesting that the current design codes need to be updated.

3. Details of case-study structure

The case-study structure (Fig. 1a) was selected to be relatively simple and regular in plan and height in order to eliminate a number of uncertainties from analyses. It is a typical building possessing similar plan to some of the standard designs of ordinary buildings constructed in several regions of Turkey in large numbers during the last 40 years (Fig. 2). This fact entails that the selected, repeatedly built, ordinary RC building has experienced several different earthquakes so that any conclusions and result findings can be reasonably extended to many similar structures. Further details about the structure can be found in Basaran (2006).

The examined structure is designed according to the 1975 Code, like 65% of the RC structures in Turkey (Bal *et al*. 2008). The concrete and reinforcement quality are C16 and S420 respectively. The fundamental period corresponding to a lateral mode is estimated after eigenvalue analysis at 0.54 sec. The lateral load coefficient, i.e., the lateral load strength over the seismic weight, is found equal to 0.16, though the structure was initially designed for values around 0.08-0.10 according to the 1975 aseismic design code. Thus, some over-strength has taken place too.



(a) Formwork plan (b) Arrangement of the 4 different column sections

Fig. 1 Case-study structure (Başaran 2006)

Fig. 2 Some examples of the standard state structures built as residential apartments in similar dimensions to the structure examined here (courtesy of Fikret Kuran)

Table 1 Reinforcement details of the columns

Floor	Column (Fig. 1b)	Dimensions (cm)	Reinforcement	Reinforcement Ratio (%)
1	C1	40×40	4ϕ18+8ϕ14	1.41
	C2	40×40	4ϕ18+4ϕ14	1.02
	C3	40×40	4ϕ18	0.62
	C4	40×40	4ϕ18+4ϕ14	1.02
2,3	C1	35×35	4ϕ18+4ϕ14	1.33
	C2	35×35	4ϕ18	0.83
	C3	35×35	4ϕ18	0.83
	C4	35×35	4ϕ20	1.03
4	C1	35×35	8ϕ14	1.01
	C2	35×35	4ϕ16	0.66
	C3	35×35	4ϕ14	0.50
	C4	35×35	4ϕ16	0.66

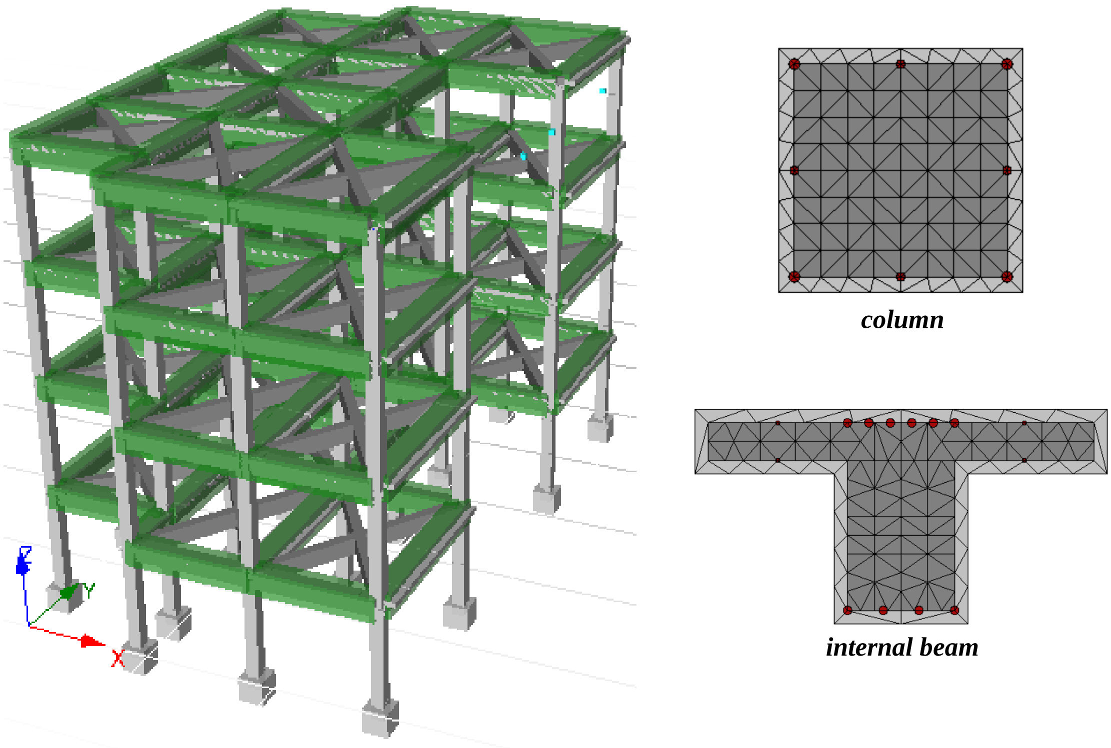
Table 2 Reinforcement details of the beams

Beam	Dimensions (cm)	Top Reinforcement	Bottom Reinforcement	Side Reinforcement in Slab Piece
Internal	25×40 effective slab width of 40 cm	6ϕ14	4ϕ14	2×2ϕ8
Perimeter	25×40 effective slab width of 77 cm	6ϕ14	4ϕ14	2×2ϕ8

The columns of the structure are 40×40 cm in the first floor and 35×35 cm in the upper floors with varying reinforcing details. The beams are 25×40 cm with a constant slab thickness of 12 cm. The rebar details of columns and beams are listed in Tables 1-2 respectively. The columns are categorized in four groups based on dimensions and reinforcement arrangement that vary from floor to floor (Fig. 1b and Table 1). The beams, on the other hand, are divided into two groups (Table 2). The effective slab width of the beams is calculated per Turkish RC design standard (TS500 2001).

4. Modelling and analyses parameters

The structure (Fig. 3) was modeled in SeismoStruct (SeismoSoft 2014), a distributed-plasticity structural analysis software that can successfully model the 3D nonlinear behavior of RC frame structures. The assessment of the structure is conducted by using the regulations of the Turkish Earthquake Code (TEC 2007). The seismic demand spectrum used corresponds to the 1st degree earthquake zone in Turkey with an effective ground acceleration of 0.4 g. Note that the PGA used for constructing the design spectrum is rather high as compared to the design PGA levels in other high-seismicity regions in Europe. The same parameter in Europe would range between 0.24 to 0.36 g, entailing thus lower displacement and ductility demand for the structure examined.



(a) 3D view (b) Example discretization of fiber sections

Fig. 3 Numerical model (SeismoStruct 2014)

Mander *et al*. (1988) confined concrete model is employed for defining the behavior of concrete. The Turkish Earthquake Code of 2007, however, defines zero confinement in cases the stirrups are not closed in 135^{o}. The 135^{o} closure of stirrups is a practice initiated after the Code of 1998, thus, concrete in structures built before 1998 is practically treated as unconfined. The unconfined/confined concrete model has ultimate strength of 16 MPa and strain at peak stress 0.002. The tensile strength of concrete is assumed as 10% of the compressive strength. The reinforcement is modeled with Menegotto-Pinto (1973) cyclic model where yield strength is 220 MPa and an isotropic hardening of 2.5% is assumed.

Force-based elements, available in SeismoStruct (2014), were employed for modeling the RC elements as they perform better in cases where the section-based response quantities, such as curvatures and strains, need to be measured. In-plane rigid diaphragm action is assıgned to the slabs. The mass and weight are distributed over the beam elements (Fig. 3a).

The RC sections are modeled by using fiber-based discretization of 200 fibers per section in average (Fig. 3b). The fiber-based elements have the advantage that the normal force - moment interaction is accurately evaluated since the section is analyzed in every step of the analysis by using the new updated set of axial force - moment values. The drawback of the fiber models, however, is that the shear deformations are not properly accounted for in the analyses. Shear is considered elastic in the model, and therefore a final check of the RC members against shear failure has to be conducted at the end of the assessment procedure.

Starting point for the methodology followed for the assessment of the structure is the determination of the target displacement accepting the fundamental rule of equal displacement or equal energy depending on the position of the fundamental period of the structure as compared to the corner period of the acceleration spectrum. Thus, when the target displacement is reached, the individual element performances will be defined, on which, in sequence, the overall performance level of the structure will be based. It should be noted that the Turkish Earthquake Code (2007) defines the member-level damages based on the material strains. The software offers the possibility of automatic checks of exceedance of the flexural limit-state values by monitoring the strain values of each section.

The modal mass participation ratios in two orthogonal directions were 85% and 71% for the long and the short direction, respectively. According to the mass participation ratio and the Turkish Earthquake Code, the structure is eligible for running a first-mode pushover analysis in order to

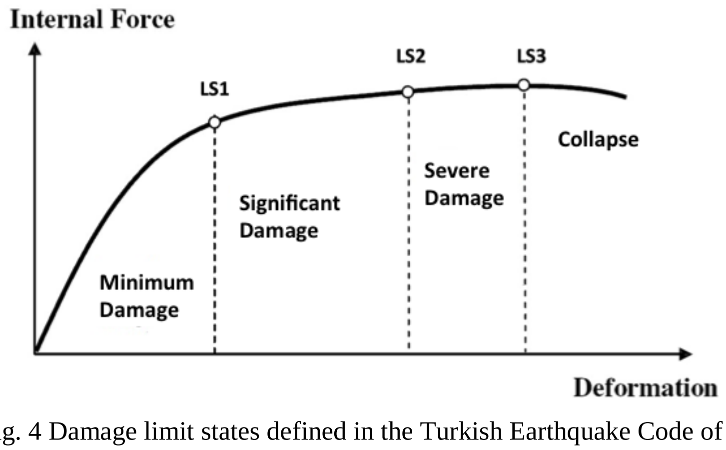


Fig. 4 Damage limit states defined in the Turkish Earthquake Code of 2007

obtain a capacity curve in base shear-top displacement format. The multi-degree-of-freedom response is then translated into the representative single-degree-of-freedom (SDOF) response by multiplying forces and displacements with the modal participation and displacement-at-effective-height coefficients. The SDOF response is then plotted over the Acceleration Displacement Response Spectrum (ADRS) in order to compare the demand with capacity. The procedure conceptually described here is illustrated later in Fig. 8, where the analysis results for the case-study structure are presented.

The sectional limit states for the ductile reinforced concrete load-bearing members that undergo the plastic deformation are defined in the Code as Minimum Damage Limit (LS1), Safety Limit (LS2) and Collapse Limit (LS3), as shown in Fig. 4. At LS1, the RC element is only slightly damaged. Structural elements have not reached significant yielding and have retained their strength and stiffness. LS1 corresponds to either a strain of 0.010 for reinforcing steel bar or a concrete strain at the most outer concrete fibers equal to 0.0035, whichever happens first. At LS2, RC element is significantly damaged, but still retains considerable strength and stiffness. The limit state strain values for core concrete and reinforcement in LS2 are

$$(\varepsilon_{cg})_{GV} = 0.0035 + 0.01 \cdot (\rho_s/\rho_{sm}) \leq 0.0135 \;\; ; \; (\varepsilon_s)_{GV} = 0.04 \quad (1)$$

where ε_{cg} is the core concrete outer fiber strain, ε_s is the reinforcing steel bar strain, ρ_{sm} and ρ_s are the minimum design and the existing volumetric ratio of the transverse reinforcement, respectively.

For the Collapse Limit state, the element is significantly damaged with very limited residual strength and stiffness. Wide flexural and/or shear cracks occur, buckling of longitudinal reinforcement may happen and the structure is not far from collapse. Concrete strain at the outer fibers of the core concrete and reinforcing steel bar strain are

$$(\varepsilon_{cg})_{GC} = 0.004 + 0.014 \cdot (\rho_s/\rho_{sm}) \leq 0.018 \;\; ; \; (\varepsilon_s)_{GC} = 0.06 \quad (2)$$

The assessment chapter of the most recent Turkish Earthquake Code of 2007 follows a realistic, yet conservative approach, by assuming that the limit state of the structure is reached if a percentage of beams and/or column has exceeded given strain limits. A short description of the flexural limit states used in the Turkish Earthquake Code of 2007 is quoted below:

• At each storey, in the considered direction, Minimum Damage limits for beams may be exceeded in 10% of the beams at most. However, all other vertical members should satisfy these limits.

• At each storey, in the considered direction, Life Safety limits for beams may be exceeded in 30% of the beams at most. In addition, again at each storey, in the considered direction, the shear forces taken by the columns, which cannot satisfy Life Safety limits, should be 20% of the storey shear force at most. All other vertical members should satisfy these limits. In this situation, the building is accepted for Life Safety Performance Level and retrofit is considered according to number and location of unacceptable elements.

• Collapse Prevention limit is exceeded if storey mechanism occurs in any of the floors with the columns, bearing more than 30% of the storey shear, attaining at both ends LS1 deformations. Additionally, at each storey Collapse Prevention Limit for beams may be exceeded in 20% of the beams at most, in the considered direction. In addition, again at each storey, in the considered direction, the shear forces taken by the columns, which cannot satisfy Collapse Prevention limit, should be 20% of the storey shear force at most. All other vertical members should satisfy these limits. In this situation, the building is accepted for Collapse Prevention Performance Level and retrofit is needed. However, retrofit should be discussed whether it will be economical or not.

5. Strengthening of columns with FRP wrapping

As already mentioned, the amount of steel lateral reinforcement (i.e., stirrups) plays an important role in deciding whether the overall member confinement will be affected by the confinement effect of the stirrups or the confinement will be dominated solely by FRP. A recent study by Teng *et al*. (2014), as shown in Fig. 5, supports that the confined confinement effect would be significant only if the volumetric ratio of stirrups is above 0.1%.

A study by Bal *et al*. (2008) indicates that stirrups of 8 mm diameter are placed in 20 to 40 cm distance in the majority of the existing RC pre-1998 building stock in Turkey, a situation that is very common in European-Mediterranean RC building stock constructed before the modern seismic design codes. The average numbers stated by Bal *et al*. (2008) are examined and it was found that the volumetric ratio in the majority of the existing Turkish RC building stock would be in the range of 0.03% to 0.12%, thus, neglecting the stirrups in FRP confinement calculations would not lead to significantly erroneous results.

The Turkish Earthquake Code of 2007 allows an increase in the concrete quality and deformability based on the level of confinement provided by the FRP material, the effect of which is approximately represented in Fig. 6. The enhancement of ductility in RC columns is defined as given in Eqs. (3), (4), (5)

$$f_{cc} = f_{cm}\left(1 + 2.4\left({f_l}/{f_{cm}}\right)\right) \geq 1.2 f_{cm} \tag{3}$$

In Eq. (3) f_{cm} is the compressive strength of the existing concrete and f_l is the lateral pressure provided by FRP and calculated according to Eq. (4)

$$f_l = \frac{1}{2}\kappa_a \rho_f \varepsilon_f E_f \tag{4}$$

where κ_a is the shape factor, calculated based on Eq. (5), ρ_f is the volumetric ratio of the FRP wrapped, ε_f is the design strain of FRP, and E_f is the modulus of elasticity of the FRP material

$$\kappa_a = \begin{cases} 1 & (circular\ sections) \\ \frac{b}{h} & (eliptical\ sections) \\ 1 - \frac{(b-2r_c)^2-(h-2r_c)^2}{3bh} & (rectangular\ sections) \end{cases} \tag{5}$$

where b, h and r_c are the width, height and the corner radius of the section, respectively. In an FRP-wrapped section, the strain value corresponding to the confined concrete strain is then

$$\varepsilon_{cc} = 0.002\left(1 + 15\left({f_l}/{f_{cm}}\right)^{0.75}\right) \tag{6}$$

It is noted that the sectional aspect ratio for the rectangular sections should be limited to 2.0 for the applicability of the aforementioned equations. The columns in the case-study structure are all

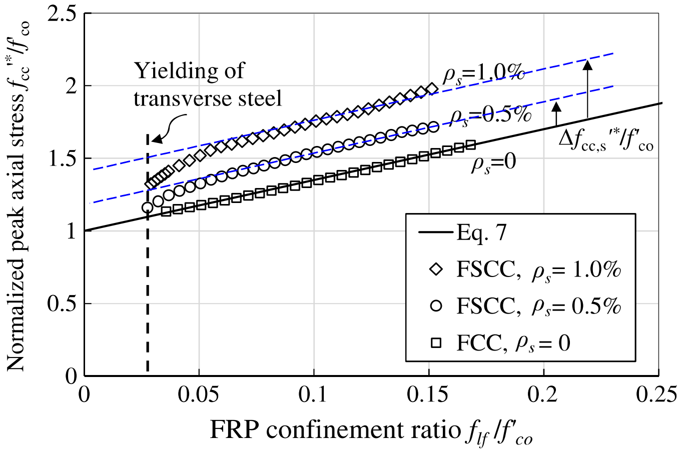


Fig. 5 Combined FRP and steel reinforcement confinement effects (Teng *et al*. 2014)

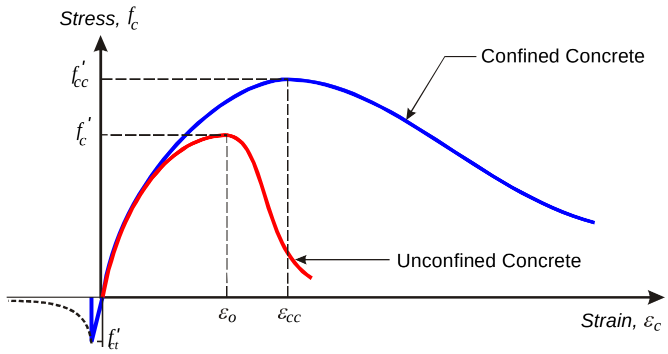


Fig. 6 Considered effect of the full FRP wrapping on concrete behaviour

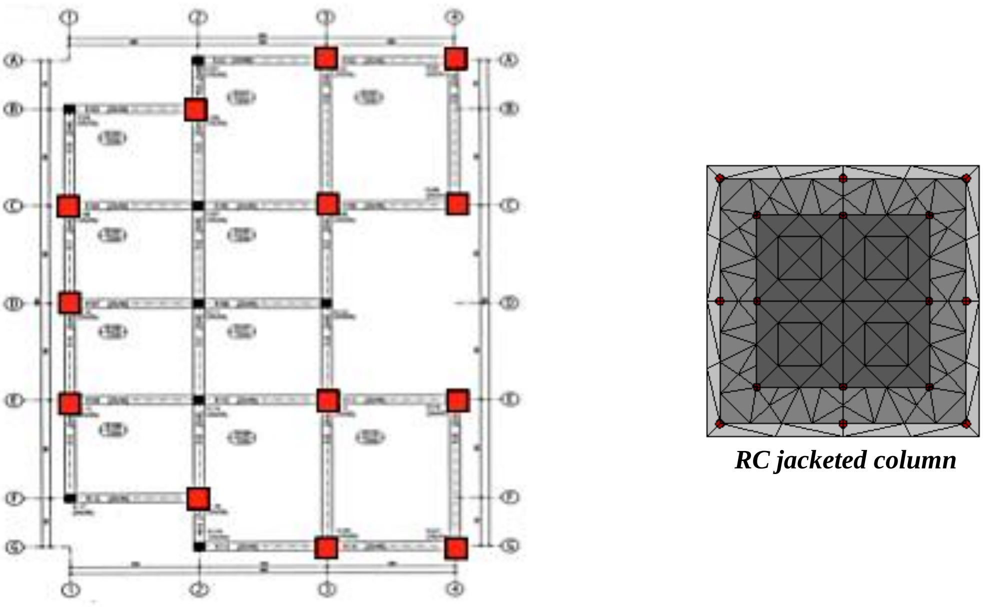
square, a fact that increases the efficiency of the FRP confinement. The FRP material used in this study is a carbon material available in the market as a commercial product. Wet-lay up system and relevant mechanical properties are assumed. Accordingly, the tensile modulus of elasticity of the material used, considering gross laminate properties, is 82 GPa. The nominal thickness of the cured material is 1 mm.

The confined concrete properties, as calculated above, have been inserted back into the model and analyses have been repeated with the columns and beams with updated strengthened properties. The overall response has then been monitored to find the actual and "new" target displacement and the corresponding performance at that displacement. Running a model with strengthened (or confined) element properties may be particularly important in terms of re-distribution potentially able to alter the expected damage mechanism.

6. Strengthening of columns with RC jacketing

The traditional strengthening method preferred in this study is the concrete jacketing of some of the columns, an application that is required in all floors as the analyses proved. The scheme of the column jacketing is shown in Fig. 7(a). In particular, applying 10 cm RC jacketing in 2/3 of the columns (total number per floor equal to 13) an equivalent result of performance to the FRP confinement is achieved, as long as the RC jacketing is continuous along the height of the structure. In case that the jacketing of the columns does not cover all the floors, a scenario that has been checked too, the soft-storey mechanism is simply pushed towards the upper floor at which the jacketing of the columns is interrupted. The assessment results, as obtained after analyses, showed that only four columns in the second floor would exceed LS1 entailing a performance of the structure falling within Minimum Damage level. It should be mentioned here that in order to ensure that the results obtained are fair and comparable, both strengthening options are applied up to the limit the structure is upgraded to the desired limit state response level. This is the reason why the structure is tightly close to the Minimum Damage level in both solutions. A wise engineering approach in practice would of course foresee a certain level of safety margin.

The jacketing is applied as 10 cm to all sides of the columns, with a single layer of reinforcement. The ready elements available in SeismoStruct (2014) for RC jacketed columns are employed. An example fiber discretization of a jacketed RC column is shown in Fig. 7(b), where the different colours represent the original column (inner), the core of the RC jacket (intermediate) and the cover of the RC jacket (outer). The existing column is treated as a single concrete material, it is also assumed that the cover of the existing concrete will be removed during the application and the concrete of the RC jacket starts where the first layer of reinforcement is placed, immediately below the concrete cover of 3 cm. A full strain compatibility between the old and the new concrete is assumed in the analyses. The stirrups of the RC jacket are considered to be sufficient preventing shear failure.



(a) Scheme of the concrete jacketed columns (b) Discretized fiber model

Fig. 7 Structure after retrofitting with RC jacketing

The RC jacket adds strength, ductility and stiffness to the system, therefore; as a consequence, the overall displacement demand decreases while the overall stiffness increases. This is the reason why the demand on the un-jacketed columns, both in terms of deformation and force demand, decreases drastically. The entire structure was RC jacketed as a first step and then the number of RC jackets is decreased iteratively in an optimization process, until a balance point is reached, at which further decrease in the number of columns with RC jacket would result in failure of non-jacketed columns.

7. Strengthening of beams

Another issue that needs to be examined is the shear capacity of beams. Though their lack of flexural capacity is overlooked, in line with most modern assessment codes that have adopted capacity design concepts, shear failure is not allowed in the beams. This is a major disadvantage for the common strengthening with RC walls or jackets since no feasible measures regarding beams can be taken.

The RC jacketing should be applied at all beam length for obvious application reasons. For this scenario, the application of shear strengthening with FRP is much easier and faster since tailor-made design is implemented. An early study by Triantafillou (1998) shows the effectiveness and ease of FRP strengthening of beams against shear actions. In strengthening of beams FRP materials are accompanied by appropriate anchors that are especially developed and certified after extensive research work and laboratory tests and are absolutely essential for avoiding a failure due to debonding.

The shear safety of the beams is calculated according to the TEC'07 where the shear safety is described by Eqs. (7)-(8) below

$$V_e \leq V_r \tag{7}$$

$$V_e \leq 0.22 b_w d f_{cd} \tag{8}$$

where b_w is the width of the section, d is the effective section height and f_{cd} is the design compressive strength of concrete. The shear resistance, V_r, is then calculated according to the TS500, the Turkish Standard for design of RC structures (2001). The relevant formulae in the TS500 provisions are given below in Eqs. (9) to (12). The cracking strength of the section under shear is calculated as

$$V_{cr} = 0.65 f_{ctd} b_w d \left(1 + \gamma \frac{N_d}{A_c}\right) \tag{9}$$

where N_d, the normal force on the section, is taken positive both in tension and in compression cases, A_c is the gross sectional area, and f_{ctd} is the design tensile strength of concrete. The γ factor is 0.07 in axial compression and -0.3 in axial tension cases. If the tensile strength on the section is below 0.5 MPa, the γ factor can be assumed as zero

$$V_r \leq V_c + V_w \tag{10}$$

$$V_c = 0.8 V_{cr} \tag{11}$$

The contribution of stirrups, V_w, is the calculated as

$$V_w = \frac{A_{sw}}{s} f_{ywd} d \tag{12}$$

where f_{ywd} is the design yield strength of stirrups. In the existing structures, under severe earthquake loading, the contribution of concrete is neglected. The design equations above for the shear strengthening of beams are valid for fully-wrapped beams, the application of which is practically impossible due to the presence of slab. Alternatively, as Kim *et al*. (2014) report, the design equation in the ACI 440 document (2008) for members fully wrapped with CFRP could be applied to the U-wrap CFRP laminates, provided that properly installed CFRP anchors are used.

For a sample beam in an internal frame of the structure, the existing shear capacity is based on V_w. The 8 mm diameter stirrups are placed in every 25 cm, leading thus to a V_w of 32.73 kN. The beams that exceed this shear force during the pushover analysis, before the target displacement is reached, require shear strengthening in accordance with the Turkish Earthquake Code of 2007. Consequently, in the existing un-strengthened case, for all beams in the first two floors, apart from four beams, shear strengthening is demanded. The number of beams requiring shear strengthening drastically decreases in the upper floors.

In the case of the RC jacketing option, in the first three floors 16 out of 30 beams at each floor (48 beams in total corresponding to 154 lm of jacketing) require shear strengthening, an application that is at least extremely difficult, if not unfeasible at all, when it is executed with conventional methods. As far as the FRP option is concerned, 26 in each floor in the first two floors and 24 out of 30 beams in the third floor (76 beams in total corresponding to 122 lm of jacketing) are estimated that demand shear strengthening. Note that FRP jacketing does not cause any change in beam member stiffness or flexural strength, thus can be limited only to the required length starting from the beam-ends and does not alter the strong column-weak beam balance.

8. Results and comparisons for the two alternative solutions

The case study structure is on soft soil, which corresponds to Type C according to the NEHRP classification (NEHRP 1997). Type C soil in the Turkish Earthquake Code (named Z3 in the TEC) has a corner period of 0.60 sec. Given the fact that the case-study structure belongs to the equal-energy range of periods (T=0.54 sec<0.60 sec) and for the demand spectrum of the Turkish Earthquake Code (2007) considered, the target displacement is estimated approximately at 0.10 m (Fig. 8) for the SDOF representative system, which corresponds to 12 cm top displacement in the real 3D structure.

As first analyses results showed, a failure mechanism is formed above ground in the first floor (Fig. 9). The reason why the mechanism did not occur in the ground floor but was shifted to the first floor is that the structure was designed mainly against dead loads, design that results in significantly larger ground floor columns (i.e., 35-40% more sectional area in average) rendering the first floor’s columns more vulnerable. This is a phenomenon commonly observed in the RC structures where the earthquake loads were either not taken into account or did not clearly govern the design. In reality, however, the presence of the masonry infill walls changes the picture and shifts the storey mechanism down to the ground floor in most of the cases. In this study, however, and in any assessment studies done in practice in Turkey, the masonry infill walls are neglected.

As per the damage distribution, in particular, the reinforcement of the columns of the first floor reaches LS1, while the concrete material has already attained LS2 and LS3 in the columns of the same floor. These strains occur in both ends of the columns resulting in a soft-storey mechanism, which is primarily attributed to the abrupt change in stiffness and strength caused by the reduction of column dimensions in the upper floors, a characteristic feature of frame structures designed for gravity loads in the countries of the European-Mediterranean region.

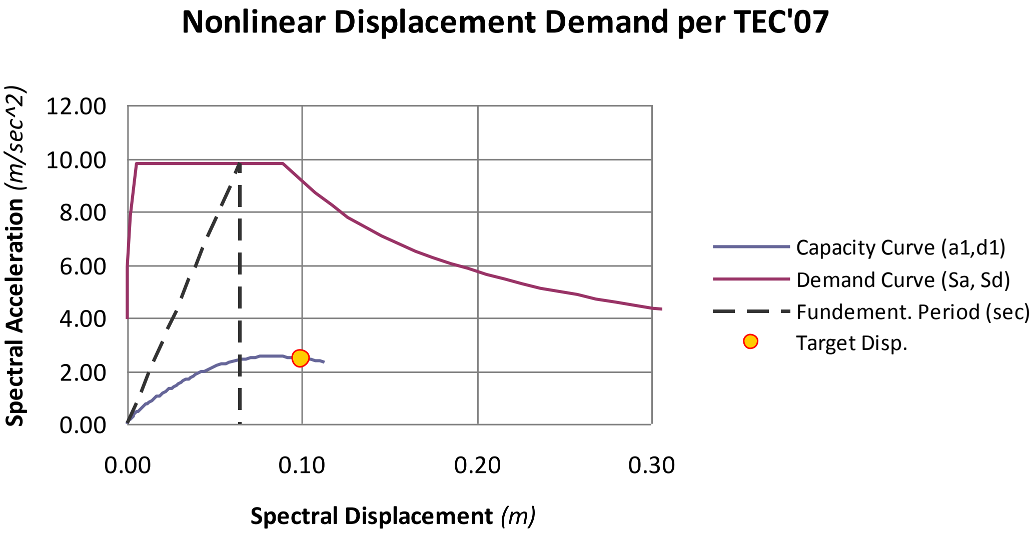
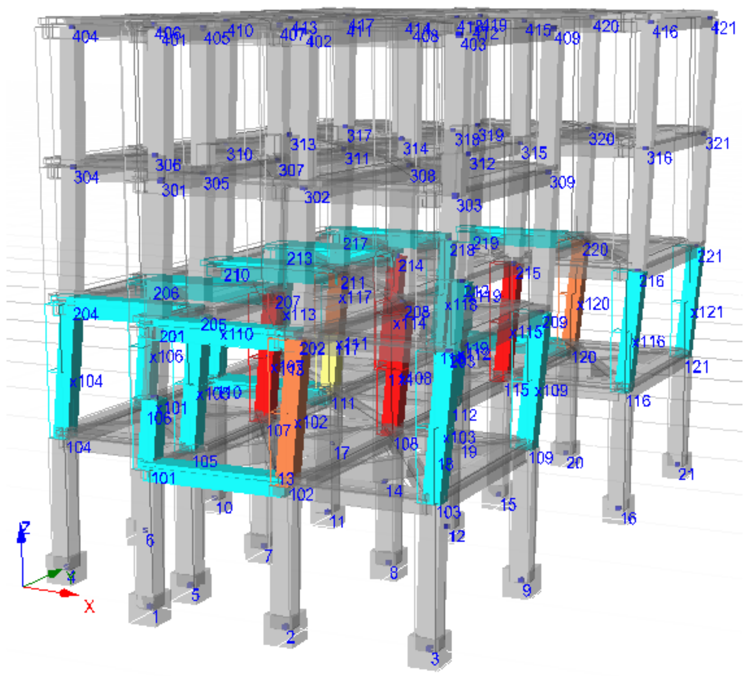
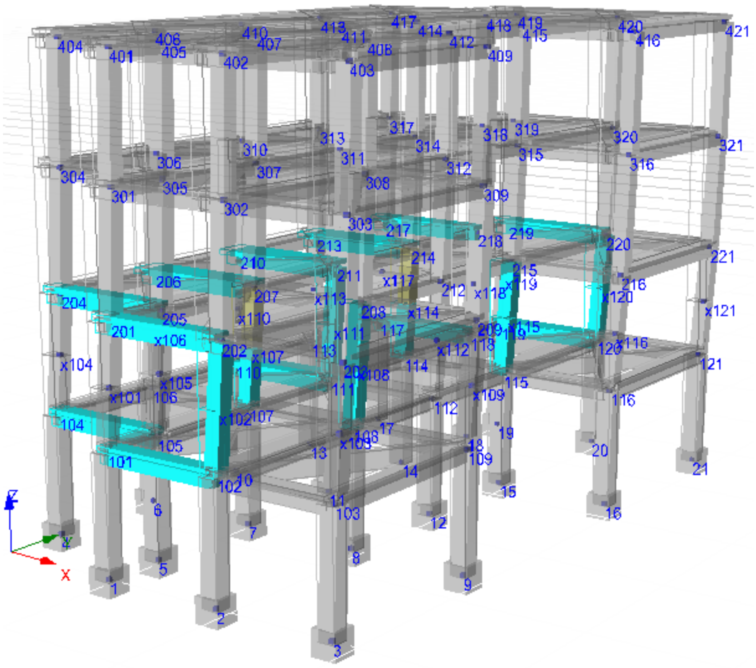


Fig. 8 The definition of the target displacement as per TEC before FRP wrapping



Blue: LS1 of rebars is reached
Orange: LS2 of concrete is reached
Red: LS3 of concrete is reached

Fig. 9 Damage distribution when the target displacement is reached, before FRP confinement



Blue: LS1 of reinforcement is reached

Fig. 10 Damage distribution when the target displacement is reached, after FRP confinement

As explained above, in the TEC (2007), the LS2 and LS3 for the core concrete is function of the level of confinement. The stirrup ends are closed by 90^o (not 135^o) in the old construction practice in Turkey, similar to applications met in many buildings of the European Mediterranean building stock. This fact leads to practically zero confinement according to the TEC regulations, entailing very small strain limits (i.e., 0.0035 and 0.0040) for core concrete for LS2 and LS3. The lack of proper confinement, in combination with the low quality of reinforcement, constitutes the reason for the low deformation capacity of the elements, enhancement of which would limit the problem in the first floor. An improvement can be achieved after application of FRP materials.

In case a FRP-based solution needs to be developed, a full wrapping of the column ends only in the first two floors will sufficiently increase the concrete confinement and consequently the concrete compressive strength, as well as the compressive strain limits of the confined concrete (Fig. 6). Thus, the damage on the columns can be eliminated or at least limited, pulling the structure to Minimum Damage performance level. The effectiveness of such a solution has been confirmed by analysis results that demonstrate that only few columns in the first floor pass to LS1 (Fig. 10).

Note that the finite element model of the structure is updated after the FRP confinement option was pursued. The concrete model of the core and cover concrete are changed, namely a more ductile concrete material with higher strength is assigned to the column members. Following this change in the model, analyses are repeated. No column is detected with LS2 or LS3 in the first floor of the structure after the change in column confinement. The FRP confinement alters the limit state of the columns preventing plastic hinges developing at the two ends and immediately shifting the structure from Collapse Prevention to Life Safety. It is reminded that the collapse

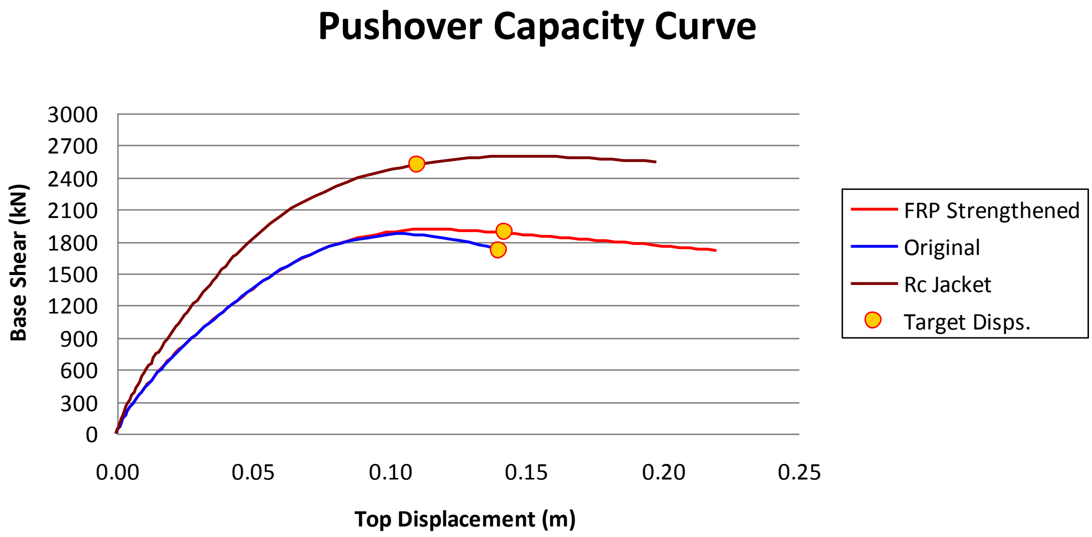


Fig. 11 Comparison of the pushover capacity curves for 3 cases

prevention in the Turkish Code is defined when certain number of columns (those bearing 30% of the storey shear force) reaches LS1 deformations at both ends, which are not attained after the application of FRP wrapping. Once FRP delays the formation of plastic hinges at column ends the structure reverts from collapse to lower and acceptable performance levels.

As per the updated overall base shear versus top displacement response, illustrated in Fig. 11 where pushover curves for the two solutions are compared, it should be noted that the target displacement has increased slightly after the FRP wrapping of the columns, signifying small improvement in the post-yield response of the structure, without, as expected, change in stiffness. In overall, the FRP confinement does not substantially alter the target displacement, but it enhances quite effectively the deformation capacity of the structure to respond to the deformation demand, namely ameliorating the ductility. On the contrary RC jacketing increased both stiffness and strength of the structure regardless, however, of whether it was needed or not. The base shear with the RC jacketing is 40% higher than the equivalent with FRP strengthening, resulting in 40% higher seismic forces induced in the building and its content. The level of induced seismic forces can be critical in industrial buildings, hospitals, facilities hosting telecommunication or other hardware, high-tech equipment, museums etc., where the value of the contents can be several times higher than the structure's itself.

The application of the FRP solution involves wrapping in the first two floors the total number of columns, i.e., 21 elements, at their ends for a length of 80 cm using 3 layers. Thus, the desired ductility will be achieved. U-shaped wrapping of the beams of the first three floors, i.e., 76 beams in total, until a length of 2*h* from the ends and two layers is also required in order to sufficiently upgrade the beams' shear capacity. It is estimated, given the calculated quantities and dimensions, and for current average prices for materials and workmanship, that de-installation and installation of the same window/door frames (no dimension changes), plastering, as well as some other small repair works, correspond to a reconstruction cost per plan surface square meter that is fraction of that for a new construction cost.

Similarly, in the case of RC jacketing, the application requires full RC jacketing of 12 columns per floor at all four floors, construction of small foundations for the column jackets, beam jacketing (in whole length) at the first three floors (48 beams in total) and changing of window/door frames (dimension changes due to RC jacketing), demolition and reconstruction of walls, new flooring, plastering, painting all interior and exterior surfaces of the building, and other repair works, turning into a reconstruction cost per plan surface square meter substantially lower than the cost of a new construction but approximately double of that for FRP wrapping, at least for the solution examined in this work.

In terms of cost, the interventions described previously should be taken into account in detail for each solution in order to reach a realistic estimation of the cost. To be more precise, indirect costs due to downtime or parameters such as disturbance should be also included in the overall evaluation of the alternative solutions. Though the purchase of FRP materials is more expensive as compared to the traditional materials, and their application presupposes the availability of specialized experienced crew, adding to that the confidence engineers feel with traditional methods, the final total cost for the FRP solution may be balanced considering that the quantities required are usually not excessive and the works of reconstruction can be considered local interventions. Specifically for the case study examined with the FRP solution, no work is predicted in the fourth floor, thus no reconstruction cost for the fourth floor is generated.

9. Conclusions

Strengthening of an existing RC structure may be realized in several ways, depending not only on technical parameters but also on other factors such as direct cost, legal issues, shutdown time and disturbance. In case additional stiffness is not required, something that may be the case for up to 3- to 4-storey ordinary RC frame structures, as shown in this study FRP can be used with competitive advantages in the light of the latest state-of-the-art research in seismic strengthening with FRP. FRP material is particularly effective in increasing the ductility by means of improvement of confinement and/or shear capacity, an option that can be used complimentary to other traditional interventions. At the same time, the stiffness of the structure is not altered, a slight strength increase is achieved in the post-peak region and the target displacement, i.e. the demand of the code-level earthquake, does not change since stiffness, strength and weight of the structure remain unchanged. The primary advantage by FRP strengthening is that the displacement capacity, i.e. the ability of the structure to meet larger displacements, is significantly augmented. Thus, the FRP effect is positively limited to the capacity without influencing the demand part. Note that the response of the upgraded strengthened structure was confirmed after repeating analysis with revised improved element properties explicitly accounting for the effect of FRP in confinement.

The use of FRP as a stand-alone solution in this work did not aim to endorse a very strengthening approach versus other ones, but simply attempted to delineate the FRP material application when employed as sole means of seismic strengthening. It can be inferred that, in small residential buildings with less than five floors, FRP can be used for seismic upgrading, not necessarily in combination with traditional interventions and obviously depending on the particular characteristics of each case. Besides that, it clearly constitutes a more plausible answer to the problem of shear strengthening of beams, as compared to traditional methods, RC jacketing included. A combination of confinement with FRP and application of targeted traditional methods appears to be the optimum solution with effective results in decreasing the deformations at structural members and increasing the stiffness of the structure.

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