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STATIC NONLINEAR ANALYSIS OF A RETROFITTED TYPICAL REINFORCED CONCRETE BUILDING IN TURKEY

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ABSTRACT

In this study, an attempt has been done for investigating the seismic behavior of a typical existing building in Turkey by performing static pushover analysis before and after retrofitting the columns either by reinforced concrete or carbon fiber reinforced polymer (CFRP) composite jackets. The selected model building represents nearly all typical construction deficiencies of buildings constructed before recent earthquake resistant design codes. According to the results of the structural analysis, significantly larger lateral displacement and slightly higher lateral strength with respect to original performance are possible by jacketing the columns of the building with CFRP sheets. On the other hand, in the case of reinforced concrete jacketing, the lateral strength and stiffness were significantly increased, as well as remarkable improvement of the lateral displacement capacity. Similarly, cross-sectional analysis performed through derivation of moment-curvature relationships showed that, an increase in ductility was much pronounced for CFRP jacketing while improvement in lateral strength was remarkable for reinforced concrete jacketing.

Introduction

Throughout the world many existing reinforced concrete structures, constructed prior to up to date earthquake resistant design procedures, suffer from the inability to supply adequate ductility during earthquakes. Particularly, brittle columns without adequate transverse reinforcement may cause total collapse of this type of structures due to lack of sufficient deformation capacity. Retrofit of this type of columns by means of forming an additional jacket layer may supply the required transverse reinforcement and enhance the seismic performance by providing additional ductility, and reducing seismic force demand. In this study, the seismic behavior of a typical existing building in Turkey is investigated by using pushover analysis before and after retrofitting its columns with CFRP or reinforced concrete jackets. The selected building contains most of the typical construction deficiencies that are common for the buildings

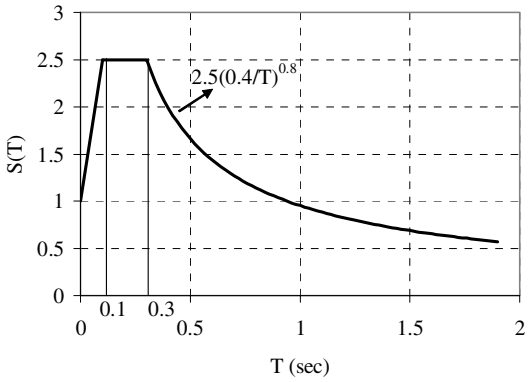
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constructed before recent earthquake resistant design codes. In addition to the insufficiency observed in the planning of the structural system, deficiencies such as low quality of concrete (f'_c: 10 MPa), inadequate transverse reinforcement (6 mm diameter bars with 300 mm spacing), and usage of plain bars with relatively lower yield strength (f_y: 220 MPa) also exist. Due to low concrete strength and relatively smaller column dimensions, level of axial stresses are generally greater than 50% of the axial load capacity of the sections. Moment-curvature relationships of the potential plastic hinging zones, which are essential for performing pushover analysis, were derived by using the fiber element analysis approach. During the analysis of the original structure, unconfined concrete stress-strain relationship was used for determining the contribution of concrete. For the analysis of the structural members retrofitted by CFRP jacketing, a stress-strain model which was specially proposed for CFRP jacketed low strength concrete, was used. Finally, analysis of the members retrofitted by reinforced concrete jacketing was carried out by using a trilinear confined concrete stress-strain model. Both for original and retrofitted members a trilinear stress-strain relationship, that took the effect of strain hardening into account, was used for reinforcing steel. The analytical results showed that CFRP and reinforced concrete jacketing of this type of deficient columns enhanced the overall structural seismic performance. However, the enhancement in ductility was much significant for CFRP jacketing whereas the enhancement in strength and increase in lateral stiffness was more significant for reinforced concrete jacketing.

Features of the Building

The reinforced concrete frame building, which was constructed around 1970s, is in Anatolian part of Istanbul on the highest seismic risk zone. Turkish Earthquake Resistant Design Code (Ministry of Public Works and Settlement, 1998) states the design horizontal acceleration as 0.4g for such zones with high seismicity. The footings of the six story building are located on stiff rock, which can be classified as the strongest local site type according to the Turkish Earthquake Resistant Design Code (Ministry of Public Works and Settlement, 1998), Fig.1. Note that, a seismic load reduction factor of 4 is taken into account as mostly done in practice for this type of existing RC frame structures. Consequently, base shear coefficients can be determined as 0.087 and 0.092 for x and y directions considering the periods of original structure. A typical floor plan of the building, which is used for residential purposes, is given in Fig.2. According to Fig.2 it can be seen that, all columns are rectangular, structural system is not symmetric in any of the directions, many columns are not connected to each other, and the columns and their orientations are not distributed evenly. Consequently, an irregular structural system is formed due to these deficiencies. 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 older existing 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 supplying the adequate confinement required for a ductile behavior. Regarding the results of elastic analysis carried out according to the Turkish Earthquake Resistant Design Code (Ministry of Public Works and Settlement, 1998), the lateral drifts exceed the prescribed limits (relative drifts should be less than 0.0035 and 0.02/R) and almost all of the columns are found to be inadequate in terms of flexure. Since lateral stiffness of the structure is quite low due to small cross-sectional areas of its columns, periods of first two modes are 1.13 and 1.05 seconds for x

and y directions, respectively. It should also be noted that high level of axial stresses on columns also reduces the ductility.



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

V_t : Base shear force
W : Total weigth of the structure
A_o : Effective ground acceleration coefficient (0.4g in this case)
I : Building importance factor (1 for this case)
S(T) : Spectrum coefficient
$R_a(T)$: Seismic load reduction factor (4 for this case)

Figure 1. Calculation of base shear force according to the Turkish Earthquake Resistant Design Code.

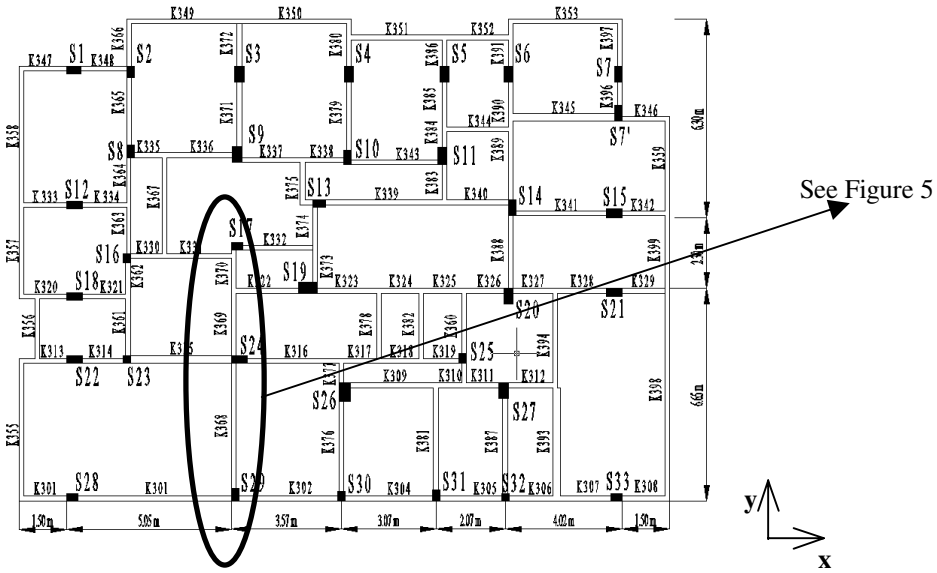


Figure 2. Typical floor plan of the building.

Pushover Analysis for Original and Retrofitted Buildings

The pushover analysis can be considered as a series of incremental static analysis carried out to examine the nonlinear behavior of structure, including the deformation and damage (plastic hinging pattern). The procedure consists of two parts. First, a target displacement for the structure is established. The target displacement is an estimate of the seismic top displacement of the building, when it is exposed to the design earthquake excitation. Then, a pushover analysis is carried out on the structure until the displacement at the top of the building reaches the target displacement. The extent of damage experienced by the building at the target displacement is considered to be representative of the damage experienced by the building when subjected to design level ground shaking. A judgment is formed as to the acceptability of the structural behavior for the level of damage of an existing building for evaluation purposes. The

structure is assumed to be subjected to a monotonically increasing lateral load. Gravity loads are also included in the mathematical model and combined with the lateral loads. The lateral load variation is assumed to represent the distribution of inertia forces in a design earthquake, proportional to Equivalent Seismic Load distribution as defined in Turkish Earthquake Resistant Design Code (Ministry of Public Works and Settlement, 1998). During analysis, the nonlinear load-deformation characteristics of individual members of the structure are incorporated in the form of moment-rotation relationships. The effect of changes in column axial load level due to the lateral loads is neglected for column plastic hinge properties. Moment-rotation relationships of columns are obtained at an axial load level of gravity loads. The shear reinforcement of columns and beams is sufficient for preventing premature shear failure and sustaining shear stresses until flexural capacity is exceeded. All the beams are assumed to have one typical moment-curvature relationship, representing weak column strong beam cases. Observations from on-site damage surveys after earthquakes in Turkey show that beams are generally stronger than the columns.

Moment-Curvature Relationships

The well known fiber element approach was used in order to obtain the moment-curvature relationships. During the analysis, sections were divided into a number of fibers of either concrete or reinforcement. After estimating the axial deformations for given curvatures, the axial deformations, which satisfy the balance requirements, were predicted by an iterative procedure. Finally, the resisted moments were calculated by using the deformation distribution on the section.

Original Columns

Hognestad model (Hognestad, 1951) was used for the stress-strain relationships of concrete and a trilinear stress-strain relationship with strain hardening considering buckling was used for reinforcing steel.

CFRP Jacketed Columns

Potential plastic hinging zones of all columns of the structure were assumed to be externally confined with one ply of CFRP sheets in transverse direction. The characteristics of the CFRP sheets, which behave linear elastic until failure, are given in Table 1. In this table f^*_{fu}, E_f, ε^*_{fu} and t_f are the tensile strength, elasticity modulus, ultimate tensile elongation and effective thickness of the CFRP sheets as given by the manufacturer.

Table 1. Characteristics of CFRP sheets.

f^*_{fu}, MPa	E_f, GPa	ε^*_{fu}	t_f, mm	Unit weight, kg/m^3
3430	230	1.5%	0.165	1820

The analytical moment-curvature relationships of CFRP jacketed column sections were also obtained by fiber element approach similar to the original columns. The stress-strain relationships of CFRP jacketed concrete were obtained by utilizing the model introduced by (Ilki

et al., 2004a). Although there are numerous stress-strain models for CFRP jacketed concrete available in the literature, this model was selected due to the reliability and applicability of the model for low strength concrete, (Ilki et al., 2006). Though combined contribution of CFRP jacket and internal transverse reinforcement were taken into account, the contribution of internal transverse reinforcement was minor compared with the CFRP's. The stress-strain relationship of the reinforcement was modeled with a trilinear strain hardening elasto-plastic relationship.

Reinforced Concrete Jacketed Columns

As the second retrofitting alternative, all columns of the building were assumed to be enlarged with the well known reinforced concrete jacketing technique. A jacket thickness of 100 mm was considered for this purpose. Although, sometimes due to boundary conditions all sides can not be jacketed, this situation was neglected in this case. Both longitudinal and transverse reinforcement were selected in a way that the minimum requirements stated by the Turkish Earthquake Resistant Design Code (Ministry of Public Works and Settlement, 1998) were satisfied. The characteristic compressive strength of the jacket concrete was selected as 25 MPa. The longitudinal bars were deformed bars of 14 mm diameter and the transverse reinforcement in the potential plastic hinging zones were deformed bars of 8 mm diameter with 100 mm spacing. The characteristic yield strength of jacket reinforcement was 420 MPa.

Fiber element approach was carried out by considering the materials and features of the original and jacket sections. The effect of unconfined cover concrete spalling was also taken into account in the analysis. Hognestad (Hognestad, 1951) and (Ilki et al., 2004b) models were used for the stress-strain relationships of unconfined and confined concrete, respectively. An elasto-plastic trilinear stress-strain relationship with strain hardening was used for reinforcing steel. During the analysis, moment-curvature relationships were ceased when top compression fiber of the confined jacket concrete reached to a strain level that corresponds to 85% of the confined jacket concrete strength on the descending branch.

Evaluation of Original and Retrofitted Performances

The dominant deficiencies of the building were poor flexural performance due to plain longitudinal bars, low concrete quality, inadequate transverse reinforcement and small cross-sectional sizes of the columns. As a result of these critical deficiencies, the building, which was representing the typical features of existing structures in Turkey, exhibited a poor performance both in cross-sectional and structural scale.

Cross-Sectional Analysis

Cross-sectional analysis were carried out in terms of moment-rotation relationships assigned to potential plastic hinges under the assumption that all columns in the building were flexure critical members either before or after being retrofitted. Rotations in sections were derived from curvature values considering that the plastic hinge lengths were half of the section depths. Moment-curvature relationships obtained for sample columns before and after being retrofitted are presented in Figs.3a and 3b for CFRP jacketed columns and in Figs.4a and 4b for reinforced concrete jacketed columns. Main characteristics of the sample columns are given in Tables 2 and 3 for original and reinforced concrete jacket retrofitted columns, respectively. In

these tables, n is the ratio of applied axial load to the axial capacity and ρ is the geometric ratio of longitudinal reinforcement. As seen in Figs.3a and 3b, CFRP jacketing of square and rectangular columns provided significant enhancement in flexural behavior, particularly in terms of ductility. For reinforced concrete jacketed columns, although development in strength was much significant, deformation capacities remained similar to that of original columns. As expected, while higher axial loads led to higher flexural strengths, ductility was significantly reduced with increasing level of axial load even for the jacketed columns. It should also be noted that axial stress levels of reinforced concrete jacketed columns dropped dramatically due to increases in section areas and concrete quality, while axial load acting on the column did not alter too much. The corresponding stress-strain curves for concrete and reinforcing steel of the original columns before and after CFRP jacket retrofitting are also presented in Figs.3c and 3d, respectively. CFRP jacketed concrete stress-strain relationships given in Fig.3c stand for the ultimate curvature values given in moment-curvature relationships. In Fig.3d, the marked points correspond to tensile strains of longitudinal reinforcement at ultimate curvature, where external CFRP composite sheets ruptured and member failed suddenly for different columns. Similarly, in Figs.4c and 4d, marked points on confined jacket concrete and jacket steel stress-strain relationships stand for the ultimate curvature of given reinforced concrete jacketed sections.

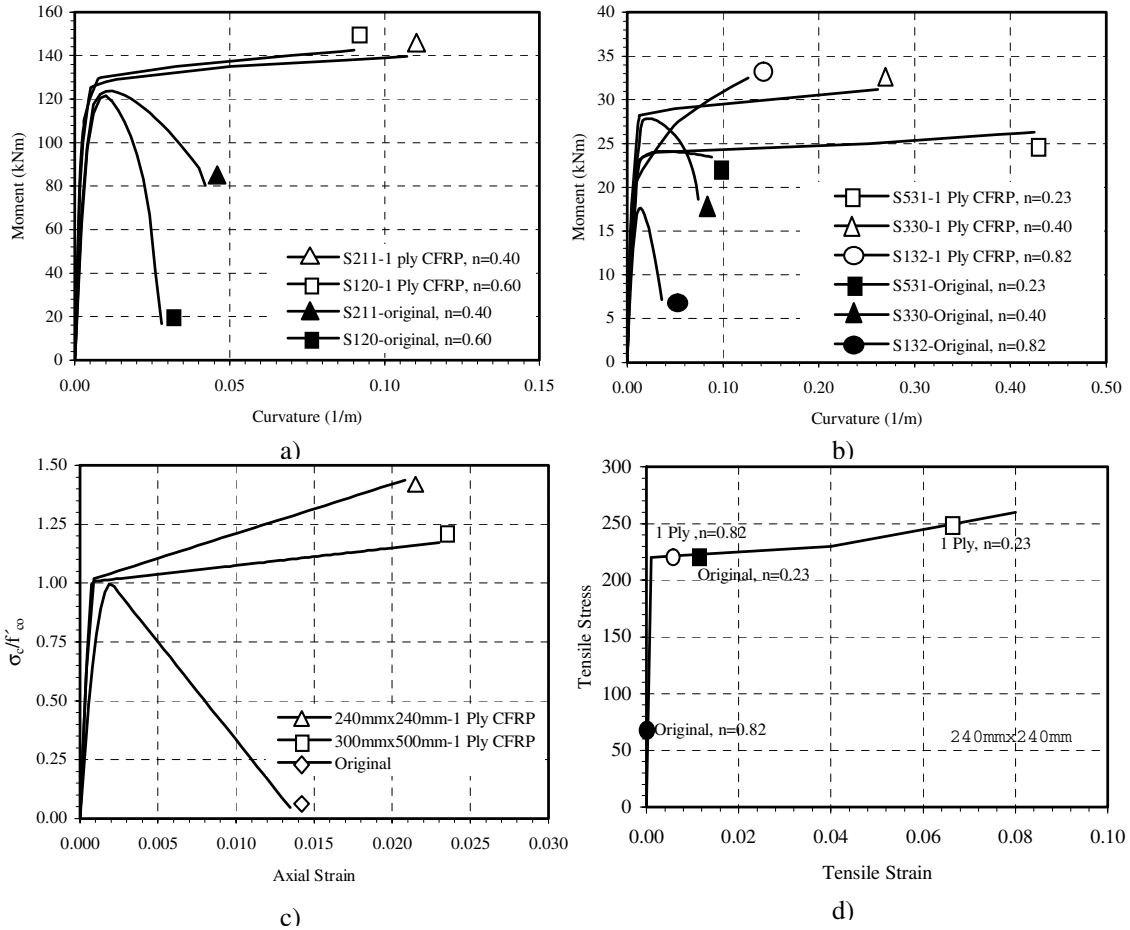


Figure 3. Sample moment-curvature relationships of original and CFRP retrofitted columns and corresponding stress-strain curves for concrete in compression and reinforcing bars in tension.

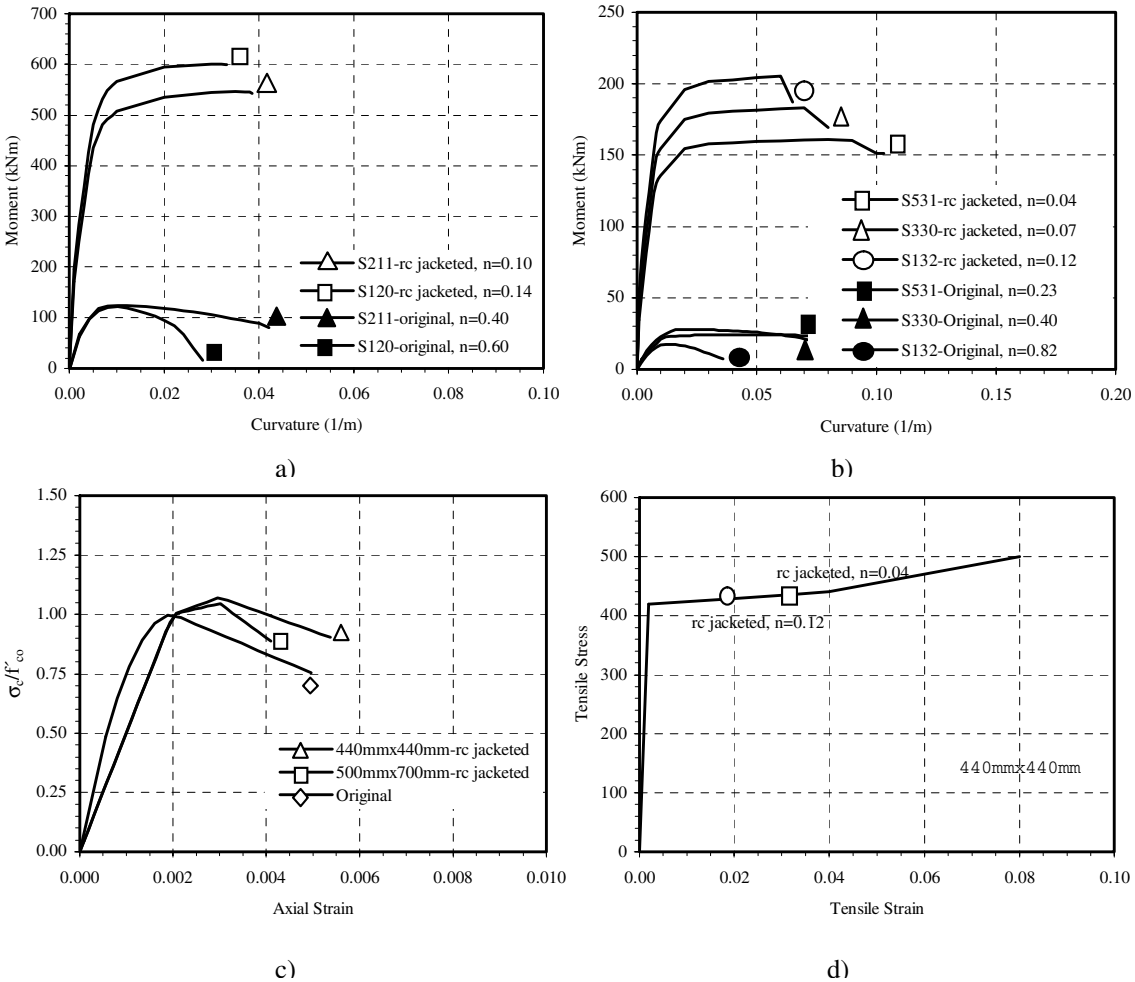


Figure 4. Sample moment-curvature relationships of original and reinforced concrete jacket retrofitted columns and corresponding stress-strain curves for concrete in compression and reinforcing bars in tension.

Table 2. Characteristics of original and CFRP jacketed columns.

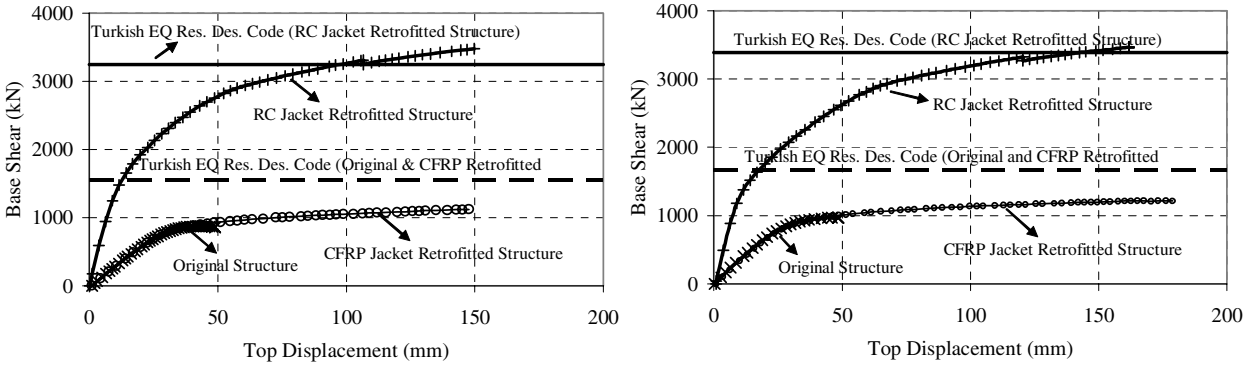
Column	**Story**	**Section (mm)**	**n (%)**	**Longitudinal Reinforcement**	**ρ**
S211	2	300x500	40	6R16	0.0080
S120	1	300x500	60	6R16	0.0080
S531	5	240x240	23	4R14	0.0107
S330	3	240x240	40	4R14	0.0107
S132	1	240x240	82	4R14	0.0107

Table 3. Characteristics of reinforced concrete jacketed columns.

Column	Story	Section (mm)	n (%)	Longitudinal Reinforcement	ρ
S211	2	500x700	10	6R16+14D14	0.0096
S120	1	500x700	14	6R16+14D14	0.0096
S531	5	440x440	4	4R14+8D14	0.0095
S330	3	440x440	7	4R14+8D14	0.0095
S132	1	440x440	12	4R14+8D14	0.0095

Structural Analysis

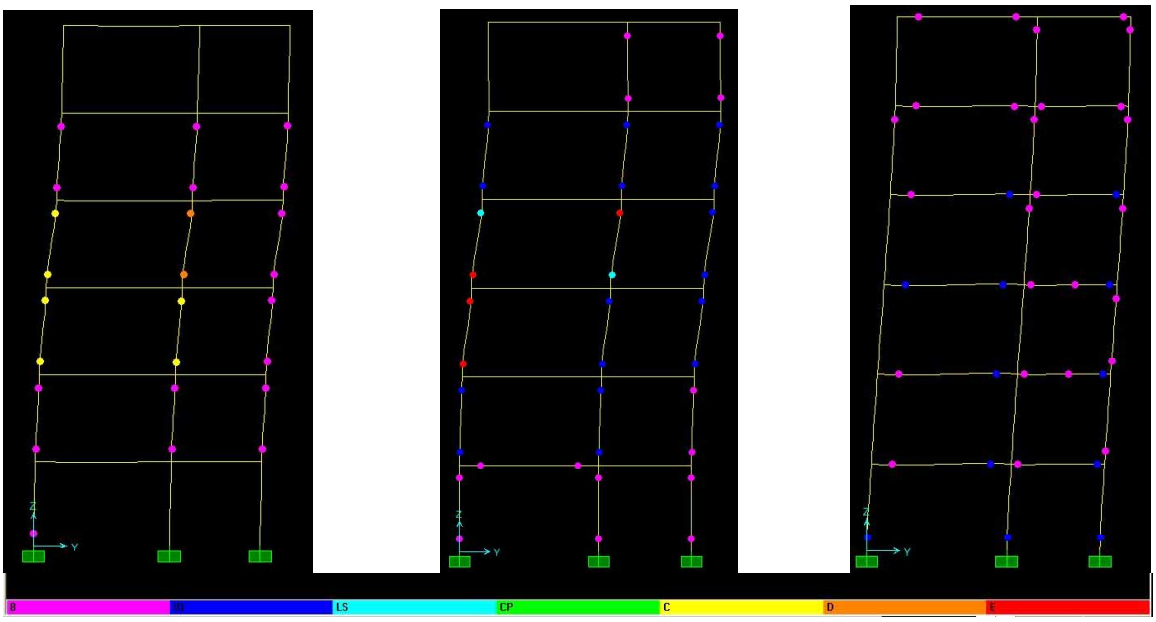
The top displacement-base shear relationships obtained by pushover analysis for original and retrofitted structures in x and y directions are presented in Fig.5 for CFRP jacketing and reinforced concrete jacketing cases. In this figure, the base shear forces calculated according to Turkish Earthquake Resistant Design Code (Ministry of Public Works and Settlement, 1998) are also plotted. It should be noted that, the seismic load reduction factor is taken into account as four. In comparison with the original structure, both retrofit techniques enhanced the strength and ductility characteristics of the building. The occupant friendly CFRP jacketing retrofit technique supplied more displacement capacity but less lateral strength than reinforced concrete jacketing alternative. On the other hand, the structure retrofitted by reinforced concrete jacketing exhibited a more rigid behavior so that structural and non-structural elements could suffer less damage. As a result of increased lateral stiffness, first two natural vibration periods of the original structure, which was not altered by CFRP jacketing, was reduced to 0.57 (x direction) and 0.54 (y direction) seconds from 1.13 (x direction) and 1.05 (y direction) seconds. CFRP jacket retrofitted structure could not maintain a lateral strength as much as the base shear defined by the Turkish Earthquake Resistant Design Code (Ministry of Public Works and Settlement, 1998) while reinforced concrete jacket retrofitted one could do. Even though the structural members of the building were flexure critical, the applied retrofitting techniques increased shear strengths of the members significantly, as well.



(a) x direction (b) y direction

Figure 5. Base shear-top displacement relationships for original and retrofitted structures.

The deformed shapes and plastic hinges of the frame indicated in Figure 2 are presented in Figure 6. In this figure the colors of hinges with colors of pink, blue and yellow correspond to yielding rotation, 25% of the ultimate rotation and 80% of the ultimate rotation, respectively. It should be noted that this frame is parallel to direction y and there are other frames, which experienced larger deformations and caused the failure of the structure. From Figure 6a, it can be seen that in the original structure, none of the beams experienced plastic deformation due to very low strength and ductility of columns. This is a clear demonstration of weak column-strong beam configuration. However, as seen in Figure 6b, as the ductilities of the columns were increased by the CFRP jackets, the structure exhibited larger displacements without strength loss and there were plastic hinges on few sections of the beams. As seen in Figure 6c, when the columns were retrofitted with reinforced concrete jackets, due to increase in ductility and particularly strength of the columns, many beam sections experienced plastic deformations and the structure resisted to relatively higher lateral loads.



(a) Original Structure (b)Retrofitted Structure RC Jacket (c) Retrofitted Structure CFRP Jacket

Figure 6. Distribution of plastic hinges for original and retrofitted structures.

Conclusions

The following conclusions are obtained after doing an attempt for analyzing the inelastic behavior of a typical existing reinforced concrete structure with various deficiencies before and after retrofitting its columns either by wrapping one ply of CFRP composite sheets in transverse direction or enlarging all columns by forming reinforced concrete jackets. Both of the techniques improved the ductility characteristics of the structure. Besides the ductility enhancement, since CFRP jacketed concrete could experience significantly large compressive strains without loss of strength, the reinforcing bars could also exhibit significantly large tensile strains making strain hardening possible. Consequently, in the case of CFRP jacketing in

addition to significant enhancement in ductility, flexural strength also increased slightly due to increase in tensile strength of the reinforcement. The columns retrofitted with reinforced concrete jacketing developed the overall structural performance in terms of ductility and lateral strength, strength being more pronounced due to larger cross-sections and additional longitudinal reinforcement. Dynamic characteristics of the structure was not affected by CFRP jacketing while reinforced concrete jacketing changed them significantly due to the increase in lateral stiffness. Consequently, reinforced concrete jacketing may be more preferable when lateral drifts are needed to be limited, which in turn limits the damage as well. However, when less disturbance is required and relatively a higher level of damage is acceptable against severe earthquakes, CFRP jacketing may be more preferable. In this technique, since the lateral stiffness of the columns are not changed, the seismic demand is not increased either. It should be noted that for reaching more general results, more detailed retrofit schemes should be examined.

References

Hognestad, E., 1951. A Study of Combined Bending and Axial Load in RC Members, *Engineering Experimental Station Bulletin Series No. 399*, University of Illinois.

Ilki, A., Kumbasar, N. and Koc, V., 2004a. Low Strength Concrete Members Externally Confined with FRP Sheets, *Int. J. Str. Eng. Mech*., 18(2), 167-194.

Ilki, A., Kumbasar, N., Ozdemir, P. and Fukuta, T., 2004b. A Trilinear Stress-strain Model for Confined Concrete, *Int. J. Str. Eng. Mech.,* 18(5), 541-563.

Ilki, A., Bedirhanoglu, I., Demir, C., Kumbasar, N., 2006 (to be submitted). Prediction of the Behavior of FRP Confined RC Columns under Axial Load and Flexure, *Proc. of Second fib Congress*, Naples.

Ministry of Public Works and Settlement, 1998. *Turkish Earthquake Resistant Design Code*, Ankara.