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# EXPERIMENTAL INVESTIGATION OF SUBSTANDARD RC BEAM-COLUMN JOINTS RETROFITTED WITH EXTERNAL POST-TENSION RODS

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## *Abstract*

The efficiency of the proposed strengthening method using externally applied post-tension rods in reinforced concrete external beam-column joints, which do not comply with any code requirements, is investigated. Five full-scale specimens were tested in the laboratory. They have specific deficiencies resulting from lack of shear reinforcement in the joint and poor material properties including low strength concrete and the presence of plain round reinforcement bars. While all specimens were built with column and in plane beam, one of the tested specimens consists of transverse beam to demonstrate the applicability of presented retrofit technique. All specimens were subjected to cyclic quasi-static loading up to 8% drift ratio to observe different levels of structural damage. Two post-tension rods, which were mounted diagonally at each side of the joint, are utilized as a local retrofit technique. The reference specimen displayed a brittle behavior with the concentration of shear cracks mostly in the joint while the rest of the RC components were almost in their elastic range. The ultimate lateral load capacity was increased considerably for all retrofitted specimens. However, a brittle type of failure mechanism was observed such as joint shear failure or beam-joint failure in the three retrofitted specimens. A relatively ductile response was observed in the specimen with transverse beam, although the axial force in the post-tension rods was the same with the specimens without transverse beam. After testing all specimens, it is found that the lateral force capacities of the beam-column assemblies can be improved up to code requirements by the proposed retrofitting method.

*Keywords: post-tension; beam-column joint; seismic retrofit; low strength concrete; plain bar*

# 1. Introduction

The beam-column joint region deserves special interest in the improperly designed and constructed reinforced concrete (RC) structures, as it can be the critical and possibly the weakest link according to the capacity design principles or hierarchy of strength considerations [1]. Therefore, the joints suffer major damage as well as brittle failure in the buildings with inadequate seismic performance. Some of the main reasons for this appear to be use of low strength concrete, plain round bars and lack of shear reinforcement in the joints. Under these circumstances, developing a feasible local retrofit strategy became more of an issue to eliminate the brittle type of joint failure.

There exists limited number of contributions on the seismic behavior of joints that were constructed according to pre-1970s construction practice [2-4]. However, the tested specimens in these studies still did not fully represent the deficient RC buildings in Turkey even though they did not comply with former building standards. Nevertheless, Bedirhanoglu et al. [5] and Coskun et al. [6] studied on the existing deficient joints in Turkey that were built with smooth bars, low strength concrete and no shear reinforcement in the joint. They emphasized to take necessary precautions for buildings with the previously indicated deficiencies. Several attempts have been made to retrofit the joints through conventional materials. More recent studies have mostly focused on strengthening of non-seismically designed joints by using fiber-reinforced polymers (FRP) [7-17]. In addition, some contributions presented retrofit strategies through conventional construction materials. Shafaei et al. [18] studied on joint enlargement by using pre-stressed steel angles. The presented retrofit method in this study relocated the plastic hinges away from the joint panel by enlarging the joint with pre-stressed steel angles. Kam and Pampanin [19] proposed a local retrofit technique called as selective weakening.

In this paper, an efficient and practical strengthening solution was conceptually proposed through retrofit of substandard RC beam-column joints with externally applied post-tension rods. For this purpose, five full-scale test specimens were constructed with low strength concrete, plain round bars and no transverse reinforcement bars in the joint region. The presented method involves implementation of post-tension rods. The rods were mounted diagonally and relied on built-up steel angles in the joint panel. The seismic forces were thus compensated inside the joint panel by the rods. This technique is considered to be effective as no additional force is developed in the rest of the members. Furthermore, a cost effective and practical solution was presented by retrofitting the deficient beam-column joints by post-tension rods.

# 2. Experimental program

## 2.1. Description of test specimens

The experimental program consists of five full-scale test specimens. None of the specimens complied with the design principles of both current and former earthquake codes. The dimensions and reinforcement scheme of the specimens were presented in Fig. 1. Plain round bars were used in all specimens as transverse and longitudinal reinforcement. The measured mean values of yield and ultimate strength of reinforcement bars were 292.5 MPa and 437.5 MPa, respectively. Since the seismic behavior of the deficient RC beam-column joints were investigated, the minimum value of axial load for the columns which is $0.1.A_c.f_c$ according the Turkish Earthquake Code 2007 (TEC2007) [20] was selected in all specimens. By this way, the contribution of axial load on the shear capacity of the test specimen is minimized.

Quasi-static cyclic lateral displacement was applied at the top of the column (up to 8% drift ratio) under the combined action of constant axial load (see Fig. 2). The out of plane displacements were restricted by the ball bearing restraining systems applied both at the top of the column and at the beam end close to roller support. This out-of-plane restraining system worked properly such that no apparent displacement was observed in lateral direction. More detailed information about the test setup and loading pattern was presented by Yurdakul and Avşar [21-22].

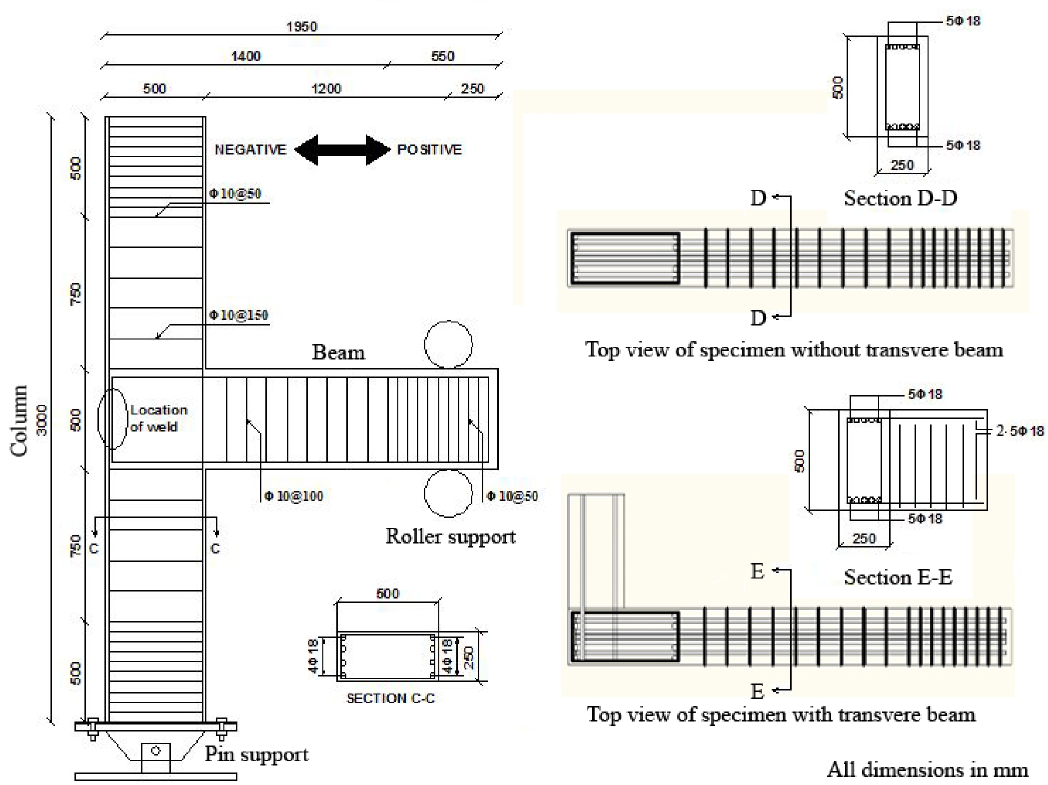


Fig. 1. Dimensions and reinforcement details

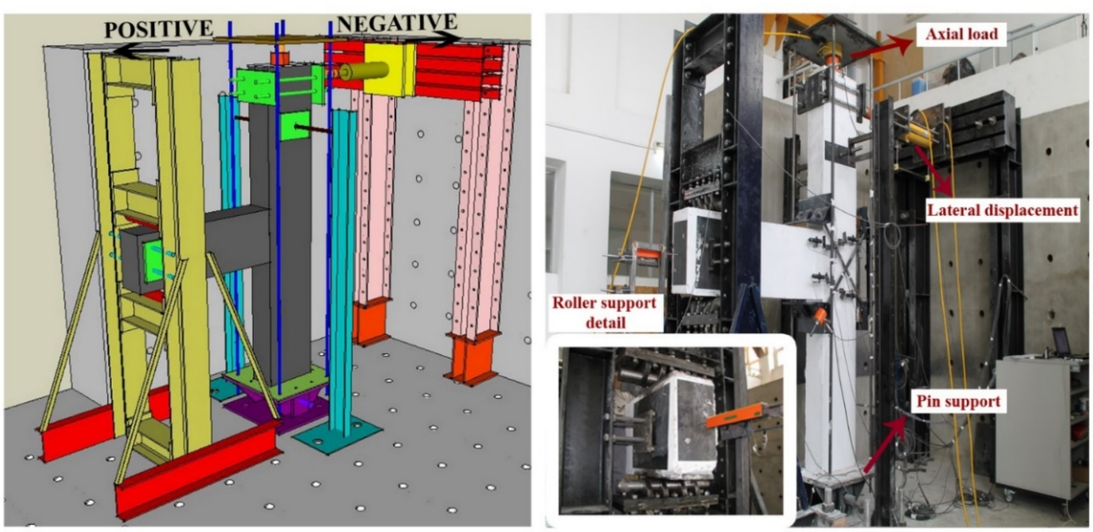


Fig. 2. 3D view of the test setup

The proposed strengthening method is very effective to improve the strength and energy dissipation capacity of the substandard beam-column joints provided that the slippage problem is prevented. In the case of retrofitting a substandard beam-column joint with plain round bars, slippage problem needs to be eliminated. This can be achieved even in the existing buildings by a procedure presented by Ilki et al [8]. The slippage failure modes can therefore be switched to the other failure modes. In the proposed retrofitting method, externally applied post-tension rods are not very effective in reducing the bar slip deformations of the plain bars. For this reason, occurrence of slippage problem is intended to be minimized by welding the beam hooks and column longitudinal reinforcement bars for all specimens.

In order to investigate feasibility of the proposed retrofit technique, one of the tested specimens was built with transverse beam, as discussed before. By this way, a viable solution is intended to be suggested for the existing structures (see Fig. 3c).

Post-tensioning was applied by four diagonal rods with a diameter of 24 mm and a length of 1000 mm. In order to fix the rods to the joint region, four equal angle built-up sections with dimensions of 225x15 mm were used. The angles had two holes at corners with a diameter of 25 mm (Fig. 3a and c). Two of the angles were placed to the beam-joint interface without using anchor bolts, whereby rest of them were doweled to the back side of the column by steel anchor bolts. The post-tension rods were mounted diagonally and fastened to the angles by nuts. Post-tension was applied to the nuts by torque wrench from the upper story column level while the nuts of the lower story column level were fixed. The torque was exerted one by one to the rods to minimize the eccentricity between two sides of the joint. In addition, the value of axial load in post-tension rods was measured by both torque wrench and load cell. In order to distribute the load evenly to the joint, a plate with dimensions of 20x250x950 was mounted to the steel angle in one of the tested specimen (see Fig. 3b).

### 2.2. Retrofit design

In the reference specimen, shear capacity of the joint is only limited by the tensile strength of concrete due to the lack of shear reinforcement in the joint. In the retrofitted specimens, the joint shear capacity is enhanced by post-tensioning. Therefore, the joint shear capacity is the sum of the contribution of the concrete tensile strength and the load provided by post-tension rods. In order to achieve a ductile behavior for a beam-column assembly, the beam is supposed to reach its flexural capacity before the joint undergoes shear failure. Moreover, the joint shear force demand is considered to be the maximum when the beam reaches its flexural capacity. The difference between the maximum joint shear force corresponding to beam flexure capacity and the joint shear capacity limited by the concrete tensile strength is the necessary horizontal force to be applied to the post-tension rods. Since there are two post-tension rods mounted diagonally at each side of the joint, the necessary post-tension load in each rod is the half of the calculated value as obtained in Eq. (1).

$$P = \frac{V_{jmax} - V_j}{\sin 45°} \times \frac{1}{2} \tag{1}$$

All necessary information about the specimens was summarized in Table 1.

Table 1 - Test specimen summary

| Specimen | Description | Mechanical Properties: Concrete Compressive Strength $f_c$ (MPa) | Mechanical Properties: Axial Force in One Rod P (kN) | Application | Transverse beam | Column Back Plate |
|---|---|---|---|---|---|---|
| EJ-R | Reference | 8.05 | - | - | N/A | N/A |
| EJ-P-1 | Without post-tension | 9.10 | 0 | Fig. 3a | N/A | N/A |
| EJ-P-2 | Post-tensioned | 9.47 | 100 | Fig. 3a | N/A | N/A |
| EJ-BP-1 | Post-tensioned, back plate | 9.92 | 100 | Fig. 3b | N/A | Present |
| EJB-P-3 | Post-tensioned with transverse beam | 10.41 | 100 | Fig. 3c | Present | N/A |

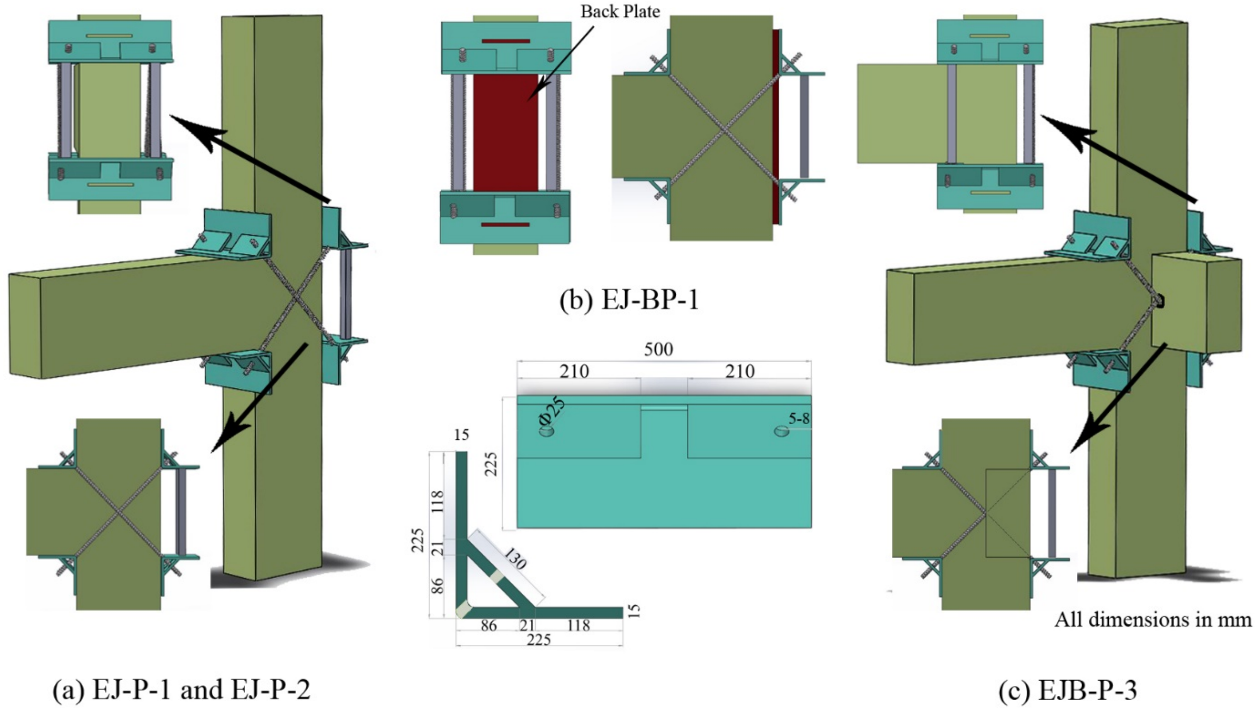


Fig. 3. Schematic representations of post-tension detail

## 3. Experimental results

### 3.1. Hysteretic response of the specimens

The reference specimen (EJ-R) displayed a non-ductile behavior with the concentration of shear cracks mostly in the joint panel (see Fig. 4a). The capacity of the specimen was therefore limited by joint shear strength. The first inclined crack in the joint, the first flexural crack in the beam and beam-joint interface were observed at 0.2% drift ratio, which corresponds the 29.5% of the ultimate load capacity of EJ-R. As the imposed displacement increased, new cracks were formed in the joint panel parallel to the beam longitudinal bar and parallel to the column reinforcement at 0.5% and 1.5% drift ratios, respectively. In the subsequent drift levels, spalling of concrete cover in the joint was observed which corresponds to severe damage. Beam hooks forced to split the concrete cover at the joint back because of push out forces created by movement of beam longitudinal bars. Therefore, spalling of concrete cover at the joint back due to anchorage push out failure was observed at 3% drift ratio. This indicates that the slippage of beam longitudinal bars was not fully prevented. In addition, anchorage push out failure induced the local buckling of column reinforcement since the beam hooks were welded to the column longitudinal bars. However, both bond slip and anchorage push out failure modes did not dominate the overall failure mode of the specimen. Presence of plain round bars in the beam lead to vertical splitting cracks in the beam-joint interface while the crack propagation was not uniformly distributed to the rest of the beam. The lateral load capacity of the test specimen is calculated as 68.9 kN for the initiation of plastic hinge at the beam-end. As shown in Fig. 5a, the experimentally obtained ultimate lateral load is less than 68.9 kN. This indicates that the joint panel reaches to its shear capacity before the beam attains its flexural capacity.

Shear failure of the joint panel dominates the hysteric response of EJ-P-1 (see Fig. 4b). The first joint inclined crack was observed at 0.5% drift ratio, which corresponds to 62% of maximum lateral load (see Fig. 5b). Diagonal cracks in the joint developed in the form of X-pattern and spread all over the joint panel in the subsequent drift levels. The first crack due to splitting at the joint back initiated at 0.5% drift level. As the drift ratio increases, enlargement in the existing cracks and deterioration in concrete were monitored in the joint panel. As a consequence of such severe damage, spalling of concrete cover at joint back as well as joint panel was observed at 3% drift ratio. The calculated flexure capacity of the beam was 76.19 kN, which has not been reached during the experiment. The first flexural crack in the beam-joint interface was occurred at 0.5% drift ratio. Then, new hairline cracks spread over the beam length at which the steel angles are connected. Formation of flexural cracks in the beam was continued up to 2.5% drift ratio, after which the joint shear failure dominates the overall response. As the shear failure took place in the joint panel, no more damage was observed at the beam. Damage was mostly localized in the joint panel, while the rest of the structural members were relaxed.

The beam-joint failure and joint shear failure were observed in negative and positive direction of EJ-P-2, respectively. The beam-joint failure was begun by yielding of longitudinal beam reinforcement bar. Shortly after beam yielding, severe joint shear cracks were appeared and then joint exposed to shear failure as also indicated by Hassan [4]. Therefore, it can be assumed that the behavior was relatively satisfactory once compared with the first two specimens in terms of strength (see Fig. 5c). The first beam flexural cracks initiated at 0.25 % drift ratio. Formation of new cracks and widening of existing cracks in the beam stabilized after 3% drift ratio when the joint shear failure dominated the overall response. While, the beam flexural capacity (76.3 kN) was almost reached in negative direction of EJ-P-2, the beam did not attain its flexure capacity in positive direction. Up to 1.5% loading level, no significant crack was formed in the joint region of specimen EJ-P-2. As the drift ratio increases, damage concentrates mostly in the joint panel (see Fig. 4c). Vertical cracks in the joint were observed parallel to the column reinforcement near the beam-joint interface at 0.3% drift level. The first inclined crack in the joint panel and the first crack at the joint back were developed at 0.4% and 0.75% drift ratio, respectively. Concrete cover in the joint was partially spalled at 2.5% drift ratio, which corresponds to severe damage. At 1.5% drift ratio, some hairline cracks occurred in the lower story column under the steel angle. Such column cracks have not been observed in the previous test specimens. Nevertheless, crack widths in the lower story column were not so critical that they appeared to be constant in further loading steps.

A beam-joint failure was monitored in the specimen EJ-BP-1. Therefore, a rapid strength deterioration and partially ductile behavior were observed (see Fig. 5d). The first flexural cracks in the beam appeared between the steel angles at 0.2% drift ratio. Formation of hairline flexural cracks in the beam continued up to 3% drift ratio. Then, no more cracks were developed in the beam when the crushing of concrete took place in the joint panel. It was also found that vertical splitting cracks were formed in the beam-joint interface. Nevertheless, the cracks were mostly concentrated in the joint after yielding of the longitudinal reinforcement of the beam. The splitting of concrete cover at joint back due to anchorage push out failure was restricted by the back plate (see Fig. 4d). More confined joint panel was thus achieved. However, crack propagation from the joint panel to the column was observed. Further cracks formed in the column, which differed from the previous specimens. Moreover, the first diagonal crack in the joint panel appeared at 0.3% and then, it continued to develop until merging the cracks in the beam-joint interface. After 3% drift ratio, joint shear failure became more critical and severe damage was monitored in the joint panel.

The overall response of the last specimen (EJB-P-3), which has a transverse beam connecting at the joint, was quite satisfactory compared to other specimens. A ductile behavior was monitored by the strain gauge measurements as the longitudinal reinforcement bars of the beam yielded before any type of failure. The computed plastic flexural capacity of the beam corresponding to the lateral force capacity of the specimen is 75.9 kN. The beam reached that value with the global yielding of the assembly (see Fig. 5e). The first flexural crack in the beam occurred at 0.20% drift ratio, which corresponds to 18.7% of maximum lateral load. Most of the propagated cracks in the beam occurred in the plastic hinge zone. They did not spread over the rest of the beam. Formation of cracks stabilized in the beams and diagonal cracks continued to widen in the joint after 5% drift level. The first observed inclined joint crack was formed at 1% drift ratio. During the 0.3% drift ratio, the first column crack, which was parallel to the beam and nearly perpendicular to the steel angles, was formed. As the drift ratio increases, some hairline cracks developed in the column between the steel angles but the behavior of the specimen has not been affected considerably. Due to exposed deformation, the unbalanced force in the rods was transmitted to the transverse beam, which causes the torsion. Diagonal cracks thus occurred in the transverse beam. The first shear crack was observed at 1.5% drift and new cracks continued to develop and spread over the whole length of transverse beam as the imposed displacement increased.

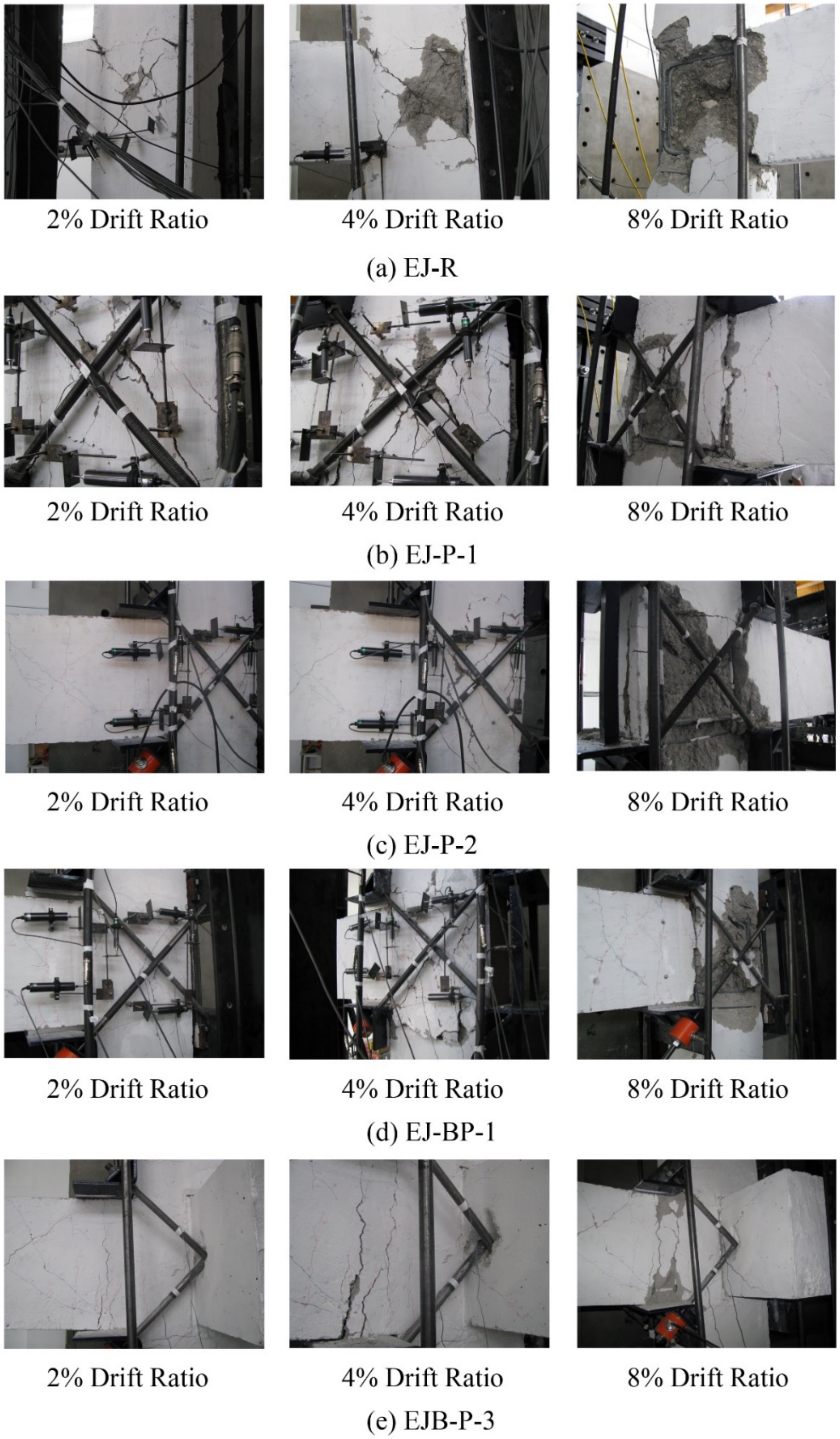


Fig. 4. Photographs of the damaged specimens at 2%, 4% and 8% drift ratios

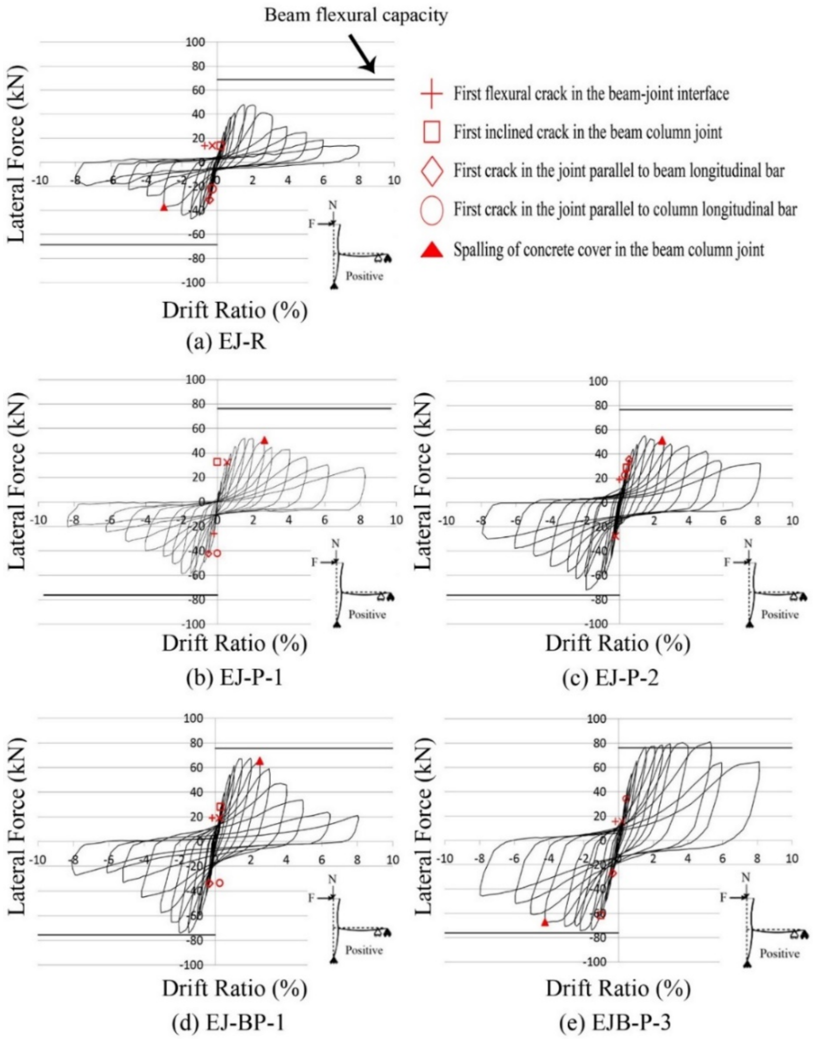


Fig. 5. Hysteresis curves

### 3.2. Effect of retrofit schemes

Improvement in the ultimate lateral load (strength), which verifies the effectiveness of proposed the local retrofit technique, can be compared by the backbone curves (see Fig. 6). Once the results were investigated in detail in terms of the average of strengths in positive and negative direction, considerable amount of strength improvement (64%) was achieved in the specimen EJB-P-3 with respect to reference specimen. It is certain that transverse beam provides additional confinement to the joint which increases its capacity. Therefore, observed ultimate lateral load is the maximum in the EJB-P-3. Sufficient amount of strength enhancement in EJ-BP-1 and EJ-P-2 was also observed, which are 50% and 34%, respectively. Even though the strength improvement was almost similar in negative direction of EJ-P-2 and EJ-BP-1, significant difference was found in positive direction of EJ-P-2, which decreases the mean value of strength improvement in EJ-P-2. A possible explanation for the difference in the results may be the confinement provided by the column back plate, which kept the strength in both directions almost the same. As the column back plate distributed the load provided by the post-tension rods uniformly, more confined joint area was achieved in the EJ-BP-1. It can be inferred that more confined joint kept the lateral load bearing capacities in both loading directions at nearly the same value. It also prevented the end anchorage failure and spalling of concrete cover. In addition, the results reported here appear to support the assumption that the strength enhancement was provided by not only confinement of the transverse beam but also the torsional stiffness of the transverse beam in the specimen EJB-P-3. During the deformation of the EJB-P-3, post-tension rods constrain the movement of the transverse beam. In addition, the unbalanced force in the rods was transmitted to the transverse beam. Due to restriction in the relaxation of the axial force in the rods, a relatively less decrement in the axial load of the post-tension rods (especially in the positive direction) was observed. This could not be monitored for the negative direction. This inconsistency in the behavior of the test specimen can be attributed to the uneven distribution of the structural damage in both directions around the joint region.

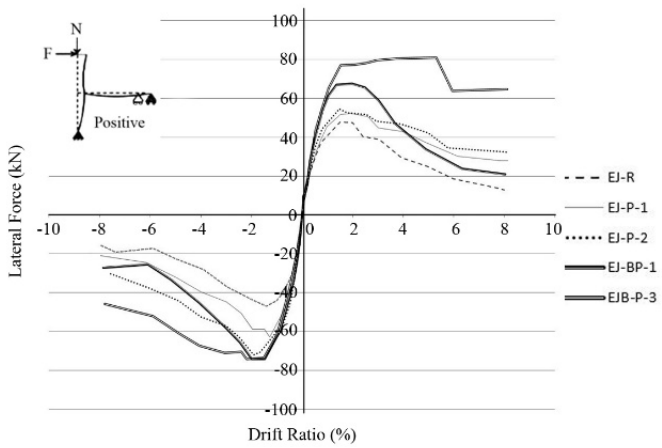


Fig. 6. Envelope curves of each tested specimen

Another parameter indicating the efficiency of the proposed method could be the variation in the axial force in the post-tension rods during the experiment. Relaxation in the post-tension rods was observed as the drift ratio increases (Fig. 7). The relaxation decreases the axial load in the rods. The trend in the axial load variation in the rods (Fig. 7) and the hysteretic behavior of the three specimens (Fig. 5c, d and e) are in good agreement. In test specimens EJ-P-2 and EJ-BP-1, a ductile behavior cannot be achieved. In a similar way, an average of 50% decrement is observed in the axial load of the rods for the 2% drift ratio in these test specimens. For the test specimen EJB-P-3, a very ductile response is observed in positive direction, where the decrement in the axial load of the rods is not more than 15%. On the other hand, in negative direction, a considerable amount of decrement in the axial load of the rods was observed for EJB-P-3. In a similar fashion, the hysteretic behavior of EJB-P-3 in negative direction is not as ductile as in the positive direction (Fig. 5e). These observations indicate that the effectiveness of the retrofit scheme and the axial load of the post-tension rods are related with each other. Post-tension force in the rods should be kept constant as much as possible for the effectiveness of the proposed retrofit scheme.

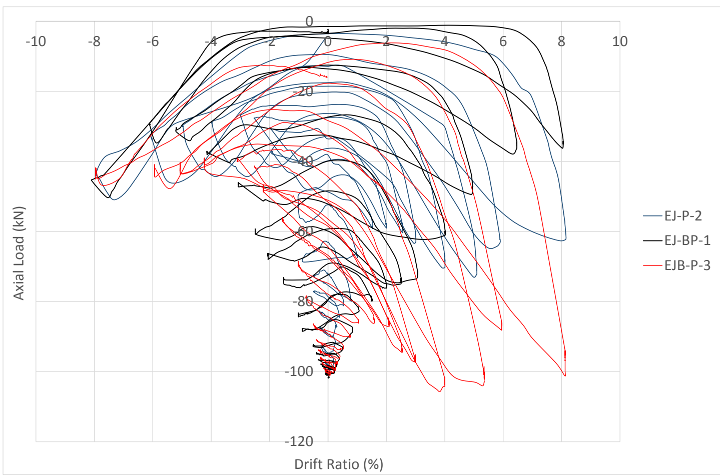


Fig. 7. Change in the axial force in the rods

The viability of the proposed method was investigated with specimen EJB-P-3, where the transverse beam could have been drilled to connect the post-tension rods. However, suggested method can disturb the other non-structural and structural components like infill walls and slab. It is therefore required to remove the infill wall during retrofitting process. Infill wall can be reconstructed after retrofitting. The slab can be drilled by the same procedure as suggested for the transverse beam (in EJB-P-3). Durability could be the another aspect of the viability of the proposed retrofit method. Necessary precautions against corrosion should be taken for protecting the post-tension rods and steel angles. There are several possibilities to avoid the corrosion, which can be the (i) use of stainless steel (ii) use of protective layer (iii) coating. These, of course, increase the cost but protecting the post-tension rods against corrosion is vital for the effectiveness of the proposed retrofitting solution. Moreover, relaxation in the nuts should be prevented for long-term usage. It may be achieved either use of self-locking nuts or welding of the nuts to the rod after application of the post-tension force.

## 3.3. Ductility

Even though the capacity of the EJB-P-3 reached the beam flexural capacity, the ductility still remains the most challenging part for the rest of the specimens. However, comparing only the ductility in the shear critical members could lead misinterpretations without considering the yield and ultimate displacement capacities. Comparing the ultimate displacement may be a reasonable approach in such specimens. Ultimate horizontal displacement, $\Delta_u$, which corresponds to the displacement when maximum lateral load reduces to 20% of the peak lateral strength as well as the yield displacement ($\Delta_y$) were presented in terms of global drift ratio in Table 2. Ultimate drift ratio, $\Delta_u^*$, of EJB-P-3 is almost twice the $\Delta_u^*$ of the reference specimen. This implies that the ultimate displacement capacity of EJB-P-3 was improved considerably compared to the reference specimen.

## 3.4. Stiffness degradation

The sustained stiffness at each loading cycle of the retrofitted specimens is higher than the stiffness of the reference specimen due to the presence of post-tension bars. As presented in Fig. 8, the higher the confinement in the joint through post-tensioning, the lower the rate of decrease in the peak-to-peak stiffness, $K^p$, was calculated. Almost 50% drop in the stiffness was observed within 0.5% and 1.5% drift ratios in the specimens EJ-R, EJ-P-1 EJ-P-2 and EJ-BP-1. The recorded value that corresponds to the same amount of decrement in the EJB-P-3 was between 0.5% and 2% drift ratios. After 3% drift ratio, decrease in the stiffness was almost linear. Then, for the drift ratios greater than 5%, the retrofitted specimens could sustain almost the same level of peak-to-peak stiffness with the reference specimen due to reduction in the contribution of post-tension rods as a result of relaxation.

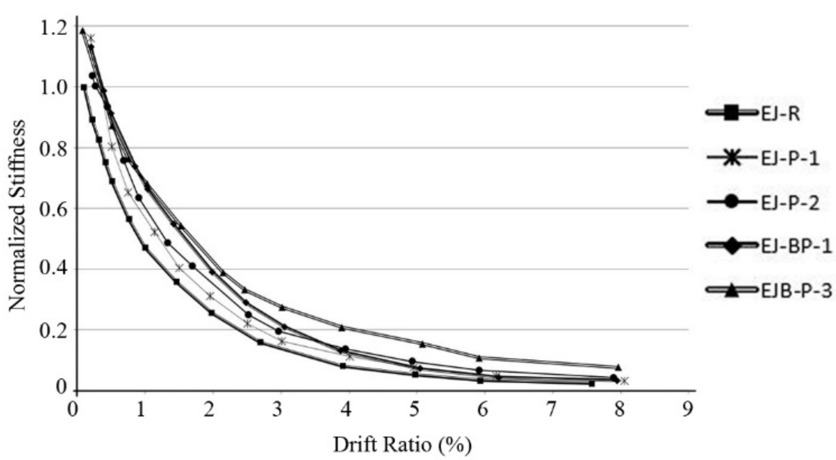


Fig. 8. Stiffness degradation curves

3.5. Energy dissipation capacity

Dissipated energy in the retrofitted specimens was more than the dissipated energy by the reference specimen as shown in Fig. 9. Therefore, proposed retrofit technique is considered to be effective in terms of energy dissipation. Energy dissipation capacity of tested specimens is very similar up to 1.5% drift ratio even though the specimens were in post-yield region. However, energy dissipation in the retrofitted specimens undergoes a sudden increment after 2.5% drift ratio. It should be noted that 2.5% drift ratio corresponds to the drift level when concrete crushing at the joint was potentially critical in the benchmark specimen. The cumulative dissipated energy for each test specimen is presented in Table 2.

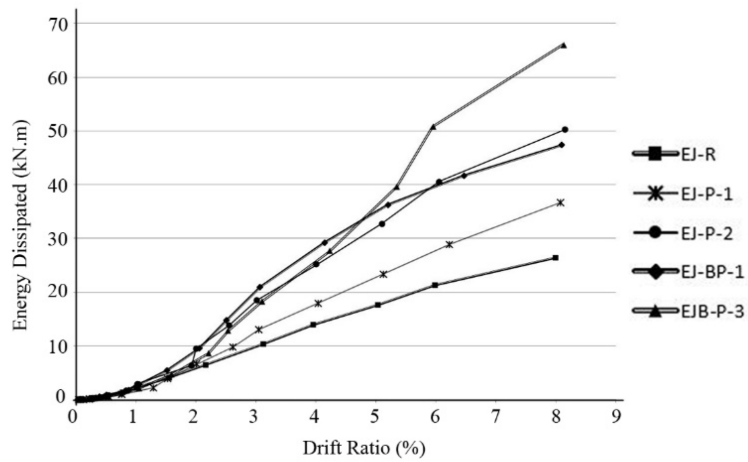


Fig. 9. Cumulative dissipated energy curves

Table 2. Summary of experimental results

| | | Strength | Yield Properties | | | Stiffness | Energy | Joint Panel Results | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Specimen | DOL | $V_{max}$ (kN) | $V_y$ (kN) | $\Delta_y^*$ (%) | $\Delta_u^*$ (%) | (kN/mm) | (kN.m) | $\tau_j$ (MPa) | $\sigma_t$ (MPa) | $\sigma_c$ (MPa) | Failure Type |
| EJ-R | + | 47.00 | 42.95 | 0.72 | 3.12 | 2.08 | 26.45 | 2.37 | 1.94 | 2.90 | Joint Failure |
| | - | 46.80 | 42.27 | 0.52 | 2.88 | 2.84 | | 2.31 | 1.88 | 2.84 | |
| EJ-P-1 | + | 52.05 | 47.47 | 0.79 | 4.20 | 2.09 | 36.66 | 2.62 | 2.18 | 3.14 | Joint Failure |
| | - | 62.70 | 56.66 | 0.59 | 2.43 | 3.36 | | 2.83 | 2.39 | 3.35 | |
| EJ-P-2 | + | 54.60 | 48.88 | 0.71 | 4.64 | 2.39 | 50.26 | 2.55 | 2.11 | 3.07 | Joint Failure |
| | - | 71.85 | 63.02 | 0.64 | 2.98 | 3.44 | | 3.39 | 2.95 | 3.91 | |
| EJ-BP-1 | + | 67.50 | 63.57 | 0.81 | 3.29 | 2.73 | 47.36 | 3.20 | 2.76 | 3.72 | Beam-Joint Failure |
| | - | 73.95 | 68.40 | 0.81 | 2.94 | 2.97 | | 3.46 | 3.02 | 3.98 | |
| EJB-P-3 | + | 81.00 | 77.46 | 1.01 | 5.90 | 2.66 | 66.05 | 3.52 | 3.07 | 4.03 | Beam Failure |
| | - | 73.95 | 68.68 | 0.96 | 5.07 | 2.51 | | 3.46 | 3.01 | 3.97 | |

DOL: Direction of loading

## 4. Conclusion

This study sets out to determine the response of five full-scale non-ductile beam-column joints under the combined effect of axial load and quasi-static cyclic loading up to 8% drift ratio. All specimens contained several deficiencies resulting from the lack of transverse reinforcement in the joint and poor material properties including low strength concrete and presence of plain round bars. Such deficiencies can result in exposure to brittle type of shear failure, which adversely affect the overall seismic behavior of the RC structures. A kind of post-tension strengthening technique, which is diagonally placed post-tension rods to the joint, was employed in this study. The response quantities such as lateral strength, energy dissipation capacity were considerably enhanced with the use of proposed retrofit strategy. The applicability of the proposed retrofitting scheme with post-tension bars was tested successfully by a test specimen with a transverse beam (EJB-P-3). The transverse beam was drilled diagonally without damaging any longitudinal reinforcement bars. In actual applications, there will be slab over the beams. Although it is not tested in the laboratory, the drilling of slab can be also done easily, as is the case for the transverse beam. For the maintenance of the proposed retrofitting solution, the post-tension rods should be protected against corrosion by taking necessary precautions such as the use of protective layer or coating around the rods. Moreover, relaxation in the nuts should be prevented for long-term usage. It may be achieved either use of self-locking nuts or welding of the nuts to the rod after application of the post-tension force.

## Acknowledgment

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## Notation

$A_c$= Gross cross-sectional area of column, beam and joint
$V_j$= Joint shear force corresponds the concrete tensile strength
$V_{jmax}$= Joint shear force corresponds the beam plastic flexure capacity
$V_{max}$ = Maximum lateral load that observed during experiment
$\Delta_u^*$= Ultimate drift ratio that corresponds 20% reduction of maximum lateral load
$\Delta_y^*$ = Yield drift ratio of specimens
$\sigma_t$= Experimental value of principal tensile strength in joint
$\sigma_c$= Experimental value of principal compression strength in joint
$\tau_j$= Experimental joint shear stress

## 5. References

[1] Pampanin S, Christopoulos C, Chen TH (2006): Development and validation of a metallic haunch seismic retrofit solution for existing under- designed RC frame buildings. *Earthquake Engineering Structural Dynamics*, **35**, 1739-66.

[2] Pantelides CP, Hansen J, Nadauld JD, Reaveley LD (2002): Assessment of reinforced concrete building exterior joints with substandard details. *Technical Report PEER 2002-18*, Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, CA.

[3] Wong HF (2005): *Shear strength and seismic performance of non-seismically designed reinforced concrete beam-column joints.* PhD Thesis, The Hong Kong University of Science and Technology.

[4] Hassan WM (2011): *Analytical and experimental assessment of seismic vulnerability of beam-column joints without transverse reinforcement in concrete buildings*. PhD Thesis, University of California, Berkeley, USA.

[5] Bedirhanoglu I, Ilki A, Pujol S, Kumbasar N (2010): Seismic behavior of joints built with plain bars and low-strength concrete. *ACI Structural Journal*, **107**, 300-10.

[6] Coskun C, Comert M, Demir C, Ilki A (2012): FRP retrofit of a full-scale 3d RC frame. *6th International Conference on FRP Composites in Civil Engineering (CICE 2012)*, Rome, Italy.

[7] Parvin A, Altay S, Altay C, Kaya O (2010): CFRP rehabilitation of concrete frame joints with inadequate shear and anchorage details. *Journal of Composites and Construction*, **14**, 72-82.

[8] Ilki A, Bedirhanoglu I, Kumbasar N (2011): Behavior of FRP-retrofitted joints built with plain bars and low-strength concrete. *Journal of Composites and Construction*, **15**, 312-26.

[9] Del Vecchio C, Di Ludovico M, Balsamo A, Manfredi G, DolceM (2014): Experimental investigation of exterior RC beam-column joints retrofitted with FRP systems. *Journal of Composites and Construction*, DOI: 10.1061/(ASCE)CC.1943-5614.0000459, 04014002

[10] El-Amoury T, Ghobarah A (2002): Seismic rehabilitation of beam-column joint using GFRP sheets. *Engineering Structures*, **24,** 1397-407.

[11] Hadi NSM, Tran TM (2014): Retrofitting nonseismically detailed exterior beam–column joints using concrete covers together with CFRP jacket. *Construction and Building Materials*, **63**, 161-73.

[12] Le-Trung K, Lee K, Lee J, Lee DH, Woo S (2010): Experimental study of RC beam–column joints strengthened using CFRP composites. *Composites Part B: Engineering*, **41**, 76-85.

[13] Lee WT, Chiou YJ, Shih MH (2010): Reinforced concrete beam–column joint strengthened with carbon fiber reinforced polymer. Composites Structures, **92**, 48-60.

[14] Topcu I (2008): *Experimental research on seismic retrofitting of R/C corner beam-column-slab joints upgraded with CFRP sheets*. MSc Thesis, Graduate Program in Civil Engineering, Bogaziçi University.

[15] Engindeniz M (2008): *Repair and strengthening of pre-1970 reinforced concrete corner beam-column joints using CFRP composites.* PhD Thesis, Georgia Institute of Technology.

[16] Akguzel U, Pampanin S (2010): Effects of variation of axial load and bidirectional loading on seismic performance of GFRP retrofitted reinforced concrete exterior beam-column joints. *Journal of Composites and Construction*, **14**, 94-104.

[17] Bedirhanoglu I, Ilki A, Kumbasar N (2013): Precast fiber reinforced cementitious composites for seismic retrofit of deficient rc joints - a pilot study," *Engineering Structures*, **52**, 192-206.

[18] Shafaei J, Hosseini A, Marefat MS (2014): Seismic retrofit of external RC beam–column joints by joint enlargement using prestressed steel angles. *Engineering Structures*, **81**, 265-88.

[19] Kam WK, Pampanin S (2009): Experimental validation of selective weakening approach for the seismic retrofit of exterior beam-column joints. *New Zealand Society of Earthquake Engineering (NZSEE) Conference*, Christchurch, New Zealand.

[20] Turkish Earthquake Code (2007): *Specification for structures to be built in disaster areas (TEC 2007).* Turkey: Ministry of Public Works and Settlement Government of Republic of Turkey.

[21] Yurdakul Ö, Avşar, Ö (2015): Structural repairing of damaged reinforced concrete beam-column assemblies with CFRPs. *Structural Engineering and Mechanics*, **54**, 521-43.

[22] Yurdakul Ö, Avsar Ö. (2016): Strengthening of substandard reinforced concrete beam-column joints by external post-tension rods. *Engineering Structures*, **107**, 9.22, DOI 10.1016/j.engstruct.2015.11.004.