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Comparing Repaired and Undamaged Specimens Test Results of Post-Tensioned Beam to Column Connections

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Abstract**—**Since, it is essential to provide homeless people by the earthquake with safe, habitable accommodation repairing medium and slight levels of damage at the connection parts should be undertaken. In order to prove that a repaired connection was sufficiently strong, a precast beam to column post tensioned connection was tested in three phases. In phase one, the middle level damage was observed at 6% drift at these connections. As a result of the extra loads applied, little damage was observed. In the last phase, the four connections tested in the first phase were repaired using epoxy resin and then retested. The results from the tests on the repaired precast and the undamaged specimens showed that the repaired specimens were sufficiently strong, thus proving that repair to damaged precast beam to column post tensioned connections can be undertaken.

Keywords**—**Precast beam to column connection, moment-resisting connection; post-tensioned connections, repair of precast connections.

I. Introduction

On 17 August 1999 a 7.4 magnitude earthquake, affected the whole Marmara Region is causing nearly 20,000 deaths, many more injuries and severe damage to industrial, commercial and domestic property. As a result of the studies performed by the Turkish Precast Union (TPB) just after the earthquake, it was stated that 24.50 % of the precast buildings constructed by member firms in Adapazarı, were damaged. Furthermore, one of the frequent sites of damage was observed in the beam to column connections of the precast structures [1]. In order to provide habitable accommodation and allow the industrial and commercial activities to resume as soon as possible, it is very important to safely repair medium and slight levels of damage in the connections. When we examine the literature, there was no evidence of any study on the repair of the precast beam to column post tensioned connections. However, although not directly related to repairs of these connections, in order to better understand the general behavior of post tensioned beam to column connections various studies are summarized below.

Blakeley and Park [2], [3] examined the behavior of beam - to - column connections, connected, post-tensioning, with anchorage and partial anchorage. In the first stage, all specimens showed satisfactory behavior in terms of high ductility and low residual displacement. In the second stage, [4] tested ten beam - to - column connections. In these studies, mild steel and high strength bars were used for the post-tensioning. They observed that mild steel increased the ductility of the specimens and decreased the loss of stiffness and strength until the concrete was crushed. Priestly and Tao [5] filled the duct with grout, which increased the ductility of the specimens. However, the losses were still observed in the anchoring force of the high strength reinforcing bars used in the post-tensioning and this kind of connection showed stiffness loss due to excessive stressing. In the second stage of the study, [6] analytically tested two pre-cast partially post-tensioned connections. In the studies at National Institute of Standards and Technology (NIST) [7]-[12] the type, location, anchorage of the mild steels used, the use of the bars and the amount of mild steel were taken as the parameters to be tested. It was understood from the experimental results, that the post-tensioned beam-to-column connections were as rigid and ductile as the cast-in-place specimen. The energy dissipation capacity was found to increase when the reinforcement was taken closer to the center or when a pre-stressing strand was used. The energy dissipation capacity of the specimens was increased when mild steel was used at the top and bottom of the beam with full anchorage. In the studies carried out in Precast Seismic Structural Systems (PRESSS) [13]-[16] four different types of connections were tested. The first type of connection involved high strength reinforcing bars without anchorage, the second utilized mild steel, the third used high strength reinforcing bars without anchorage in the middle of the cross-section and the fourth type of connection utilized special equipment, which dissipates energy through friction. In the second stage of the study, a pre-cast building was designed and tested under cyclic loads. In this stage, the four types of connections given above were tested in this building. The results showed that the performance of the hybrid and pre-stressed connections were quite good.

In the experimental study performed by [17]-[20] the effects on the stressing rates applied to the pre-stressed strands on the behavior of the connections were researched. The result was that load loss of these specimens was small and the specimens did not lose their load capacity. At the connection point of the precast specimens; it was observed that the column, beam and grout were not totally crushed.

In the last phase of the study, the beam to column post tensioned connections that had been damaged to a medium extent were repaired and tested. The results obtained from the repaired specimens were compared to the results obtained from the first phase specimens.

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II. Experimental Program

A. Test Specimens

In the columns of all the specimens, four Ø16 longitudinal mild steel and Ø8 stirrups were used. Four Ø12 longitudinal reinforcement with a total of 18 stirrups with two different heights were used in the precast beams [20]. The properties of the experiment specimens that were tested in the first (reference) and final (repaired) phase are given in Table I. In the first stage, the loading program was applied to the reference specimens. The same measurement mechanism was used for all specimens [17]-[21].

B. Materials

The yielding and tensile strengths of the pre-stressed strands and normal construction steel are given in [17]-[21]. The compressive strengths of the body, grouts and topping samples are given in [17]-[21].

The damaged specimens were repaired with Sikadur 42 Epoxy repair mortar , based on a combination of high strength epoxy resins and specially graded aggregates [22]. The properties of the epoxy resin are given in Table I.

TABLE I
Mechanical Properties of EPOXY

	Comp. stress (N/mm^2)	Flexural Stress (N/mm^2)	Concrete bond stress (N/mm^2)	Steel Bond Stress (N/mm^2)
1 day	90-100	15-30	-----	-----
7 days	100-110	-----	-----	-----
14 days	110-120	20-40	4	15-20

C. Test Setup and Instrumentation

The experiments were performed on a rigid platform comprising a rigid wall vertical to a rigid slab. The test specimens were connected to a table installed in the slab where the columns were placed horizontally and the beams were placed vertically. While applying load to the beams, a double effective lifting hydraulic jack was used to connect to the load cell. During the experiments, electronic displacement measurements (LVDT) were used [20], [21]. The same loading pattern applied to the reference specimens in the first phase was applied to the repaired precast specimens.

D. Repair of the Experimental Specimens

It was observed that the bottom edges of the topping concrete of the precast specimens, bottom edges of the beam and upper edges of the corbels were slightly crushed and the grout between the beams and columns was broken into pieces but not crashed [17]. Upon seeing that there was no serious damage to the precast specimens during the first phase of the experimental program, repair to these specimens was begun. The broken parts were removed from the area, then the connection regions were cleaned with a compressor. The cleaned connection area was filled with SİKADUR-42 a sand, polymer and hardener product. One day later, the hardboard molds were removed and the final phase of tests was performed when the epoxy resin had completely hardened.

III. Experimental Results

A. Experimental Results of the Repaired Specimens

In repetitive loadings, the loading capacity of the AP1R specimen reached average 52 % of the loading capacity of the AP1 when a 1.5 % drift was applied, and a maximum loading capacity of the AP1R specimen reached average 73,50 of the loading capacity of the AP1 specimen. The initial stiffness of the AP1R specimen was seen to be higher than the initial stiffness of the reference specimen. After a load of nearly 7.8 kN the epoxy filling between the column and beam from the surfaces of the column and beam began cracking and this caused a decrease in the stiffness of the specimen. The experimental specimen continued to carry the load during the later cycles at a lower stiffness. Excessive damage was not observed at the concrete side of the specimen during the test (Fig. 1).

Fig. 1 Cracking of the corbel of the AP1R test specimen

In repetitive loadings, the loading capacity of the BP1R specimen reached average 44 % of the loading capacity of the BP1 when a 1.5% drift was applied, and a maximum loading capacity of the BP1R specimen reached average 75,50 of the loading capacity of the BP1 specimen. The initial stiffness of the BP1R specimen was seen to be lower than the initial stiffness of the reference specimen. When the load applied to the specimen reached nearly 7.4 kN, the loss of the initial stiffness of the specimen increased. The reason for this loss was the cracking of the epoxy resin present in the connection from the bottom surface of the beam. Due to the cracking of the epoxy resin on this surface, a loss of stiffness was observed in the specimen. At the end of the test, no serious damage was observed in the connection of this specimen.

When the load applied to the specimen reached nearly 7.6 kN the loss of the initial stiffness of the specimen decreased due to the cracking of the epoxy resin at the bottom surface of the beam. In spite of observing a loss in the stiffness of the CP1R specimen, the test was completed without excessive damage at the connection (Fig. 2).

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Fig. 2 A view of corbel and bottom side of the beam at the CP1R specimen after experiment

Fig. 3 Fallen state of the parts of the concrete at the upper corner of the corbel of the DP1R specimen

In repetitive loadings, the loading capacity of the DP1U specimen reached average 54 % of the loading capacity of the DP1 when a 1.5 % drift was applied, and a maximum loading capacity of the DP1R specimen reached average 77,00 of the loading capacity of the DP1 specimen. The initial stiffness of the DP1R specimen was seen to be higher than the initial stiffness of the reference specimen. After a nearly 7.5 kN load, the cracking of the epoxy resin caused a decrease in the stiffness of the specimen. The experimental specimen continued to carry the load during the later cycles at a lower stiffness. The bottom edge of the beam touching the corbel at the corbel side was crushed during the backward loading of the last cycle. The cracking of the concrete at the bottom part of the beam caused a 6 kN load loss of the specimen at this loading (Fig. 3).

B. Strength and Behavior

Even though a 264 N/mm^2 stress, loss was observed at the strands of the AP1R specimen compared to the reference specimen, 1136 N/mm^2 stress remained in the strands of this specimen. In the strands of the BP1R specimen, 240 N/mm^2 stress, loss was observed compared to the reference specimen. However, in the strands of this specimen a 760 N/mm^2 stress remained.

IV. Evaluation of Test Results

In the strands of the CP1R specimen, a 314 N/mm^2 stress, loss was observed compared to the reference specimen and a 806 N/mm^2 stress remained. In the strands of the DP1R specimen a 386 N/mm^2 stress, loss was observed compared to the reference specimen where a 1294 N/mm^2 stress remained.

When the net stresses remaining in the strands of the specimens were compared the highest stresses remaining in the AP1R, and DP1R specimens' strands. Same story as the drift was applied to all the specimens; the maximum compressive stress was applied to the AP1R, and DP1R specimens. The compressive stresses at the bottom edges of the beams at the corbel side caused the crushing of the concrete in this region overcoming the compressive strength of the concrete.

At the end of the test, the maximum loading capacity of the AP1R specimen was 74% of the maximum loading capacity of the AP1 specimen, the maximum loading capacity of the BP1R specimen was 76% of the maximum loading capacity of the BP1 specimen, the maximum loading capacity of the CP1R specimen was 72% of the maximum loading capacity of the CP1 specimen, and the maximum loading capacity of the DP1R specimen was 77% of the maximum loading capacity of the DP1 specimen.

During the backward loadings of the AP1R, and DP1R specimens in the last cycle, crushing was observed at the bottom edge of the beam which resulted in losses in the loading capacities of the specimens.

The same loading pattern was applied to the BP1R, and CP1R specimens, however, serious damage was not observed on these specimens during the forward and backward loading.

A. Stiffness of the Experimental Specimens

The initial stiffness of the AP1R specimen were 108% of the initial stiffness of the AP1 specimen, the initial stiffness of the BP1R specimen were 91% of the initial stiffness of the BP1 specimen, initial stiffness of the CP1R specimen was 96% of the initial stiffness of the CP1 specimen and initial stiffness of the DP1R specimen was 113% of the initial stiffness of the DP1 specimen (Table II).

TABLE II
Initial Stiffness Ratio and Stiffness Ratio at 1.5 % Drift

		1.5 % Drift		Maximum
Stiffness ratio (rep*/ref*)	Forward (%)	Backward (%)	Backward (%)	Forward (%)
AP1R/AP1	110	106	73	67
BP1R/BP1	91	91	60	74
CP1R/CP1	91	93	71	59
DP1R/DP1	114	112	79	65

rep*: repaired specimen ref*: reference specimen

The stiffness of the AP1R specimen at 1.5% story drift rate was 70% of the stiffness of the AP1 specimen at this story drift rate, the stiffness of the BP1R specimen at 1.5% story drift rate was 67% of the stiffness of the BP1 specimen at the same rate, the stiffness of the CP1R specimen at 1.5% story drift rate was 65% of the stiffness of the CP1 specimen at this rate and stiffness of the DP1R specimen at 1.5 % story drift rate was 72% of the stiffness of the DP1 specimen at this story drift rate.

V. Conclusions

It was seen that loading capacity of the DP1R repaired specimen at the 1.5% story drift was 54% of the loading capacity of the reference specimen (DP1), the loading capacity of the same specimen at the end of the test was nearly 77% of the maximum load capacity of the reference specimen (DP1).

The energy dissipation capacities of the repaired specimens were seen to be nearly equal to the energy dissipation capacities of the reference precast specimens connected in a post tensioned way

The initial stiffness of the repaired specimens is more than the initial stiffness of the reference specimens. However, the stiffness of the DP1R specimen, which had the highest stiffness at a 1.5% story drift, was at the level of 73% of the stiffness of the reference specimen (DP1).

As a result, when the success of the repaired specimens in the experiments is considered, these specimens are seen to have performed very well. Even though a 6% drift was applied to the reference specimens in the first phase the loading capacity of the DP1R specimen reaching 77% loading capacity of the reference specimen (DP1) was an example of the success of the repair specimens. In this study, it was seen that the precast industrial constructions with columns and beams connected in a post tensioned way could be repaired to make the building safely useable after light or medium level damage from earthquakes.

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