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Repair of post-tensioned precast beam to column connections

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SUMMARY

Earthquakes can cause immense damage to precast structures, rendering them inhabitable. After the 1999 Marmara earthquake in Turkey, research by the Turkish Precast Union stated that 24.50% of the precast structures were damaged with some of this damage being observed in the beam-to-column connections of the structures. Since it is essential to provide those rendered homeless 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, middle level damage was observed at 6% drift at these connections. In phase two, 1.2, 2.4 and 4.8% drifts were applied three times to the test specimens. 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 reference 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. Copyright © 2010 John Wiley & Sons, Ltd.

1. INTRODUCTION

On 17 August 1999, a 7.4 magnitude earthquake, lasting for 45–50 s, affected the whole Marmara Region, causing nearly 20 000 deaths, many more injuries and severe damage to industrial, commercial and domestic property. This damage was observed at various levels on both sides of İzmit Bay and in the industrial zones between İzmit-Adapazarı. 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 structures produced 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 (Earthquake and Prefabrication, 2000). 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 connection parts of the beams. 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 behaviour of post-tensioned beam-to-column connections, various studies are summarized below.

Blakeley and Park (1971, 1973) examined the behaviour of beam-to-column connections, connected, post-tensioning, with anchorage and partial anchorage. In the first stage, all specimens showed satisfactory behaviour in terms of high ductility and low residual displacement. In the second stage, Park and Thompson (1997) tested 10 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 (1997) filled the duct with grout, which increased the ductility of the specimens. However, losses were still observed in the anchorage force of the high strength reinforcing

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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, Priestly and MacRae (1996) analytically tested two precast partially post-tensioned connections. In the studies at the National Institute of Standards and Technology (Cheok and Lew, 1991, 1993; Cheok *et al.*, 1994, 1998; Stone *et al.*, 1995, 1997), 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 centre 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 (Priestly, 1996; Palmieri *et al*., 1997; Nakaki *et al.*, 1999; Priestly *et al*., 1999), 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 precast building was designed and tested under cyclic loads. In this stage, the four types of connections given above were tested on this building. The results of the experiments showed that the performance of the hybrid and pre-stressed connections were quite good.

In a three-phase experimental study performed by Kaya (2007) and Kaya and Arslan (2009a, 2009b, in press), the effects of the stressing rates applied to the pre-stressed strands ensuring the connection at the precast beam-to-column connections which are post-tensioned and the effect of the diameter of the strands on the behaviour of the connections were researched. In the second phase of the study, in order to determine the level of the damage on these experimental specimens, 1.2, 2.4 and 4.8% drifts were applied three times. 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 and, in addition, there was no yielding at the pre-stressing strands.

In the third 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 with the results obtained from the first phase specimens.

2. EXPERIMENTAL PROGRAMME

2.1. Test specimens

In the columns of all the specimens, four Ø16 longitudinal mild steel and Ø8 stirrups with 100-mm spacing were used (Figure 1). Four Ø12 longitudinal reinforcement, with a total of 18 stirrups with two different heights, were used for the precast beams (Figure 2). The properties of the experiment specimens that were tested at the first (reference) and final (repaired) phase are given in Table 5. In the first stage, the loading programme was applied to the reference specimens (Kaya, 2007; Kaya and Arslan, 2009a, 2009b, in press). The same measurement mechanism was used for the repaired specimens and the reference specimens (Figure 3).

2.2. Materials

The yielding and tensile strengths of the pre-stressed strands and normal construction steel are given in Tables 1 and 2. The compressive strengths of the body, grouts and topping samples are given in Table 3.

The damaged specimens were repaired with Sikadur 42 Epoxy resin; this is a solvent-free, three component, pourable grout, based on a combination of high strength epoxy resins and specially graded aggregates (SIKA Construction Chemicals, 1910). The properties of the epoxy resin are given in Table 4 and the characteristic properties of the repaired and reference specimens are given in Table 5.

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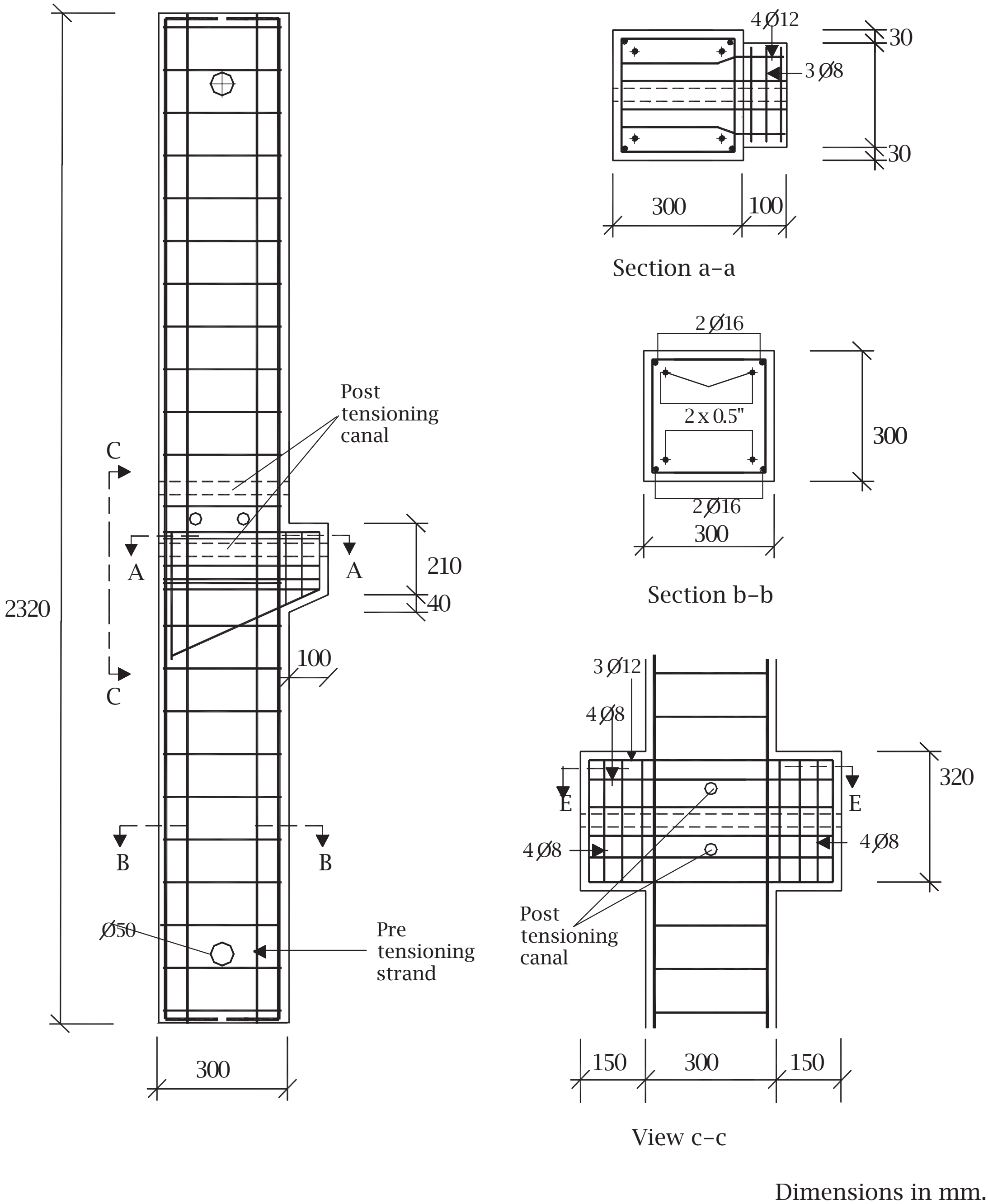

Figure 1. Reinforcement detail of the precast column.

2.3. *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 on a table installed on 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 connected to the load cell. The load readings were recorded in a computer storage. During the experiments, electronic displacement measurements (LVDT) were used (Figure 3). The data obtained from these displacement measurements were entered into a computer storage. The same loading pattern applied to the reference specimens in the first phase was applied

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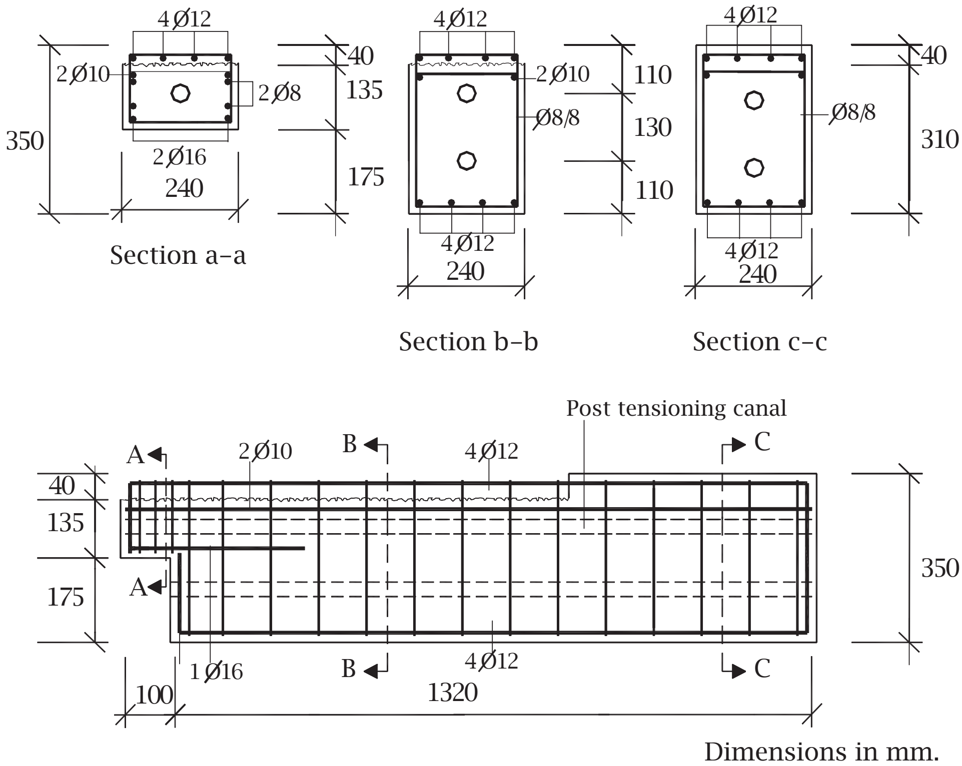

Figure 2. Reinforcement detail of the precast beam.

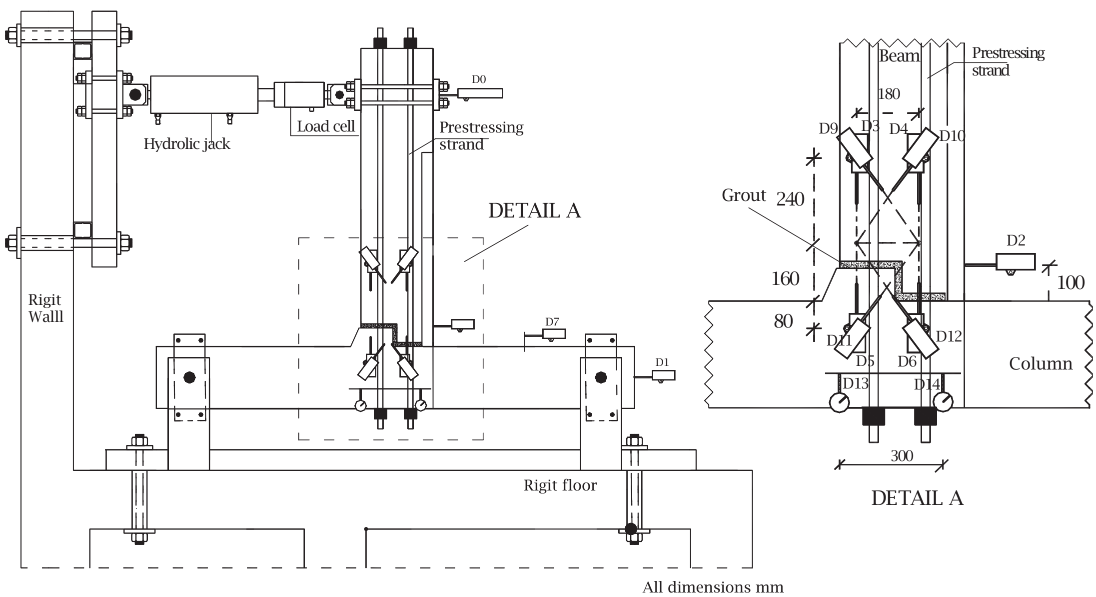

Figure 3. Test setup and measurement mechanism.

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Table 1. Yielding and tensile strengths of the pre-stressing strands.

Strand (mm)		Area (mm^2)	Yield stress (N/mm^2)	Ultimate stress (N/mm^2)	Yield strength (kN)	Ultimate strength (kN)	Elastic modulus $\times 10^3$ (N/mm^2)
12, 70	Minimum	95.25	1589	1710	151	187	182 280
	Maximum	107.78	1738	1870	180	210	209 720
15, 20	Minimum	137.7	1589	1710	266	266	182 280
	Maximum	152.60	1738	1870	290	290	209 720

Table 2. Yielding and tensile strengths of the mild steel used at the experiments.

Reinforcement	Class	Yield stress (N/mm^2)	Ultimate stress (N/mm^2)
Ø 5	S420a	–	454
Ø 8	S500bs	585	690
Ø 10	S500bs	505	593
Ø 12	S500bs	522	625
Ø 14	S500bs	540	733
Ø 16	S500bs	485	590

Table 3. Compressive strengths of the experiment specimens.

Specimen	Body (MPa)	Grout* (MPa)	Grout† (MPa)	Surface (MPa)
AP1	49.1	46.0	41.3	40.5
BP1	51.1	45.7	39.6	38.3
CP1	50.6	44.3	42.1	41.2
DP1	50.3	45.1	39.3	38.6

*filled with the joints.
†filled with the ducts.

Table 4. Mechanical properties of epoxy.

Day(s)	Compressive stress (N/mm^2)	Flexural stress (N/mm^2)	Bond stress on concrete (N/mm^2)	Bond stress on steel (N/mm^2)
1	90–100	15–30	–	–
7	100–110	–	–	–
14	110–120	20–40	4	15–20

Table 5. Properties of test specimens.

Reference specimen	Repetitive specimen	Repaired specimen	Pre-stressing strand diameter (mm)	Stress ratio (%)
AP1	AP1U	AP1R	15.24	50
BP1	BP1U	BP1R	12.70	50
CP1	CP1U	CP1R	15.24	40
DP1	DP1U	DP1R	15.24	60

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to the repaired precast specimens; furthermore, the same measuring combination was used for the reference specimens as with the repaired specimens (Kaya, 2007; Kaya and Arslan, 2009a, 2009b, in press).

2.4. *Repair of the experimental specimens*

It was observed that the bottom edges of the topping concrete of the precast specimens, bottom edges of the beams and upper edges of the corbels were slightly crashed, and the grout between the beams and columns was broken into pieces but not crashed (Kaya, 2007; Kaya and Arslan, 2009a, 2009b, in press). Upon seeing that there was no serious damage to the precast specimens during the first phase of the experimental programme, repair to these specimens was begun. First, the beams were made to stand vertically on the columns. Later, the beams were fixed on the columns at three points using the steel profiles. The broken parts were removed from the area then the connection regions were cleaned with a compressor. The cleaned connection area was surrounded by chipboard then the connections were filled with SİKADUR-42, a sand, polymer and hardener product. The sand, polymer and hardener were mixed at the recommended proportions of 3 sand, 1 polymer and ½ hardener. One day later, the chipboard moulds were removed and the final phase of tests was performed when the epoxy resin had completely hardened.

3. EXPERIMENTAL RESULTS

3.1. *Experimental results of repetitive loadings*

In repetitive loadings, the loading capacity of the AP1U specimen reached 28% of the loading capacity of the AP1 when a 1.2% drift was applied; when a 2.4% drift was applied to the AP1U specimen, it reached 44% of the loading capacity of the AP1; and when a 4.8% drift was applied, the loading capacity of the beam reached 58% of the loading capacity of the AP1 (Figure 4).

Through a cycle of repetitive loadings, the loading capacity of the BP1U specimen reached 35% of the loading capacity of the BP1 when a 1.2% drift was applied; when a 2.4% drift was applied to the same specimen, it reached 43% of the loading capacity of the BP1; and when a 4.8% drift was applied, the loading capacity of the beam reached 60% of the loading capacity of the BP1 (Figure 4).

Over repetitive loadings, the loading capacity of the CP1U specimen reached 25% of the loading capacity of the CP1 when a 1.2% drift was applied; when a 2.4% drift was applied to the same specimen, it reached 41% of the loading capacity of the CP1; and when a 4.8 % drift was applied, the loading capacity of the beam reached 61% of the loading capacity of the CP1 (Figure 4).

Through the repetitive loadings, the loading capacity of the DP1U specimen reached 34% of the loading capacity of the DP1 when a 1.2% drift was applied; when a 2.4% drift was applied to the

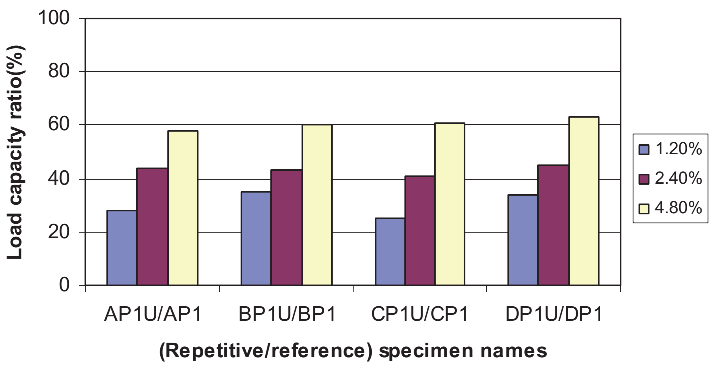

Figure 4. Rate of the load capacities of the repetitive specimens to the load capacity of the reference specimens.

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same specimen, it reached 45% of the loading capacity of the DP1;and when a 4.8% drift was applied, the loading capacity of the beam reached 63% of the loading capacity of the DP1 (Figure 4).

When the results from the repetitive loadings were examined, the loading capacity of the specimens was seen to increase in a directly proportional way to the drifts applied. These results show that excessive damage did not occur on the specimens in the first phase of the experiment.

3.2. *Experimental results from the repaired specimens*

The loading programme applied to the reference specimens was also applied to the repaired specimens. First, two controlled loading cycles were applied, then later cycles were applied with the displacement controlled.

The loading capacity of AP1R specimen at 1.5% drift in the forward direction was 1.4 kN more than its load capacity in the backward direction. At the end of the test, the loading capacity of AP1R specimen in the forward direction was 9 kN more than its loading capacity in the backward direction. The initial stiffness of the AP1R specimen in the backward and forward loadings was seen to be higher than the initial stiffness of the reference specimen. After 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. The bottom edge of the beam touching the corbel at the corbel side was crashed. The cracking of the concrete at the bottom part of the beam and grout caused 5-kN load loss in the specimen at this loading (Figure 5).

The loading capacity of BP1R specimen at 1.5% drift in the forward direction was 1.9 kN less than its load capacity in the backward direction. At the end of the test, the loading capacity of the BP1R specimen in the forward direction was 1.3 kN more than its loading capacity in the backward direction. The initial stiffness of the BP1R specimen in the backward and forward loadings 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 (Figure 6).

The loading capacity of the CP1R specimen at 1.5% drift in the forward direction was 0.2 kN more than its load capacity in the backward direction. At the end of the test, the loading capacity of the CP1R specimen in the forward direction was 6.2 kN more than its loading capacity in the backward

Figure 5. Cracking of the corbel of the AP1R test specimen.

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Figure 6. View of conection of the BP1R test specimen after experiment.

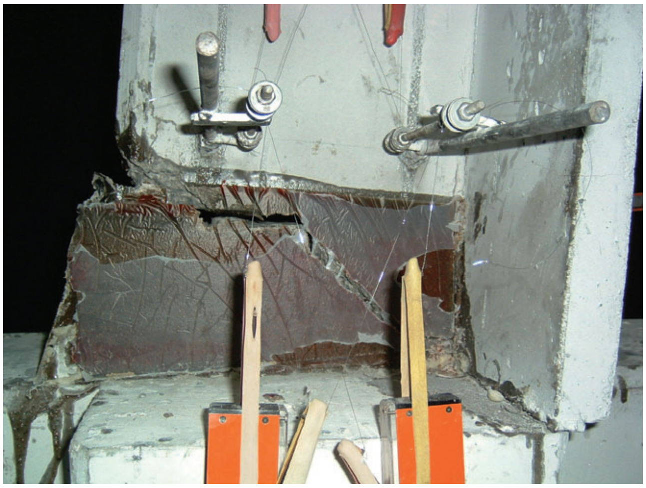

Figure 7. A view of corbel and bottom side of the beam at the CP1R specimen after experiment.

direction. The initial stiffness of the CP1R specimen in backward and forward loadings was seen to be lower than the initial stiffness of the reference 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 (Figure 7).

The loading capacity of DP1R specimen at 1.5% drift in the forward direction was 4.9 kN more than its load capacity in the backward direction. At the end of the test, loading capacity of DP1R specimen in the forward direction was 6.3 kN more than its loading capacity in the backward direction.

The initial stiffness of the DP1R specimen in the backward and forward loadings 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 crashed 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 (Figure 8).

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Figure 8. Fallen state of the parts of the concrete at the upper corner of the corbel of the DP1R specimen.

Table 6. Load ratio at 1.5% drift and maximum drift.

	Load ratio at 1.5% drift		Load ratio at maximum drift	
Load ratio (rep*/ref*)	Load ratio at forward loading (%)	Load ratio at backward loading (%)	Load ratio at backward loading (%)	Load ratio at forward loading (%)
AP1R/AP1	54	50	78	69
BP1R/BP1	44	44	68	83
CP1R/CP1	29	29	79	66
DP1R/DP1	54	54	83	71

rep*, repaired specimen; ref*, referans specimen.

4. EVALUATION OF TEST RESULTS

4.1. Strength and behaviour

Even though a 264 N/mm^2 stress loss was observed at the strands of the AP1R specimen compared with the reference specimen, a 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 with the reference specimen. However, in the strands of this specimen, a 760 N/mm^2 stress remained. In the strands of the CP1R specimen, a 314 N/mm^2 stress loss was observed compared with 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 with the reference specimen where a 1294 N/mm^2 stress remained.

When the net stresses remained in the strands of the specimens are taken into consideration, the highest stresses remained in the strands of the AP1R and DP1R specimens. As the same storey 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.

During the forward and backward loadings at the 1.5% storey drift, the loading capacity of the AP1R specimen was 52% of the loading capacity of the AP1 specimen; the loading capacity of the BP1R specimen was 44% of the loading capacity of the BP1R; the loading capacity of the CP1R specimen was 29% of the loading capacity of the CP1 specimen; and the loading capacity of the DP1R specimen was 54% of the loading capacity of the DP1 specimen (Table 6).

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Table 7. Initial stiffness ratio and stiffness ratio at 1.5% drift.

	Initial stifness ratio		Stiffness ratio at 1.5% drift	
Stiffness ratio (rep*/ref*)	Stiffness ratio at forward loading (%)	Stiffness ratio at backward loading (%)	Stiffness ratio at backward loading (%)	Stiffness ratio at forward loading (%)
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*, referans specimen.

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 (Table 6).

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 loadings.

4.2. Energy dissipation capacities of the experimental specimens

When the energy dissipation ratio of the repaired specimens is taken into consideration, they seem to have dissipated rates of energy equal to the post-tensioned precast reference specimens (Kaya, 2007; Kaya and Arslan, 2009a, 2009b, in press).

4.3. Stiffness of the experimental specimens

The initial stiffness of the AP1R specimen was 108% of the initial stiffness of the AP1 specimen; the initial stiffness of the BP1R specimen was 91% of the initial stiffness of the BP1 specimen; the initial stiffness of the CP1R specimen was 96% of the initial stiffness of the CP1 specimen; and the initial stiffness of the DP1R specimen was 113% of the initial stiffness of the DP1 specimen (Table 7).

The stiffness of the AP1R specimen at 1.5% storey drift rate was 70% of the stiffness of the AP1 specimen at this storey drift rate; the stiffness of the BP1R specimen at 1.5% storey drift rate was 67% of the stiffness of the BP1 specimen at the same rate; the stiffness of the CP1R specimen at 1.5% storey drift rate was 65% of the stiffness of the CP1 specimen at this rate; and the stiffness of the DP1R specimen at 1.5% storey drift rate was 72% of the stiffness of the DP1 specimen at this storey drift rate (Table 7).

5. CONCLUSIONS

In the experimental studies, a storey drift nearly four times as much as the life security limit stated in the ATC40 specification (Applied Technology Council, 1996) for the precast structures was applied. It was seen that loading capacity of the DP1R repaired specimen at the 1.5% storey 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).

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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 was seen to be 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% storey drift, was at the level of 73% of the stiffness of the reference specimen (DP1).

When the results of the experimental studies are taken into general consideration, the performances of the repaired specimens (loading capacity and stiffness) are seen to be proportional to the force applied to the pre-stressed strands used in the post-tensioned connections of these specimens.

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 (Kaya, 2007) and 1.2, 2.4 and 4.8% storey drifts were applied to the reference specimens three times in the second phase (Kaya, 2007), in particular, 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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