# REPAIRING OF SHEAR-DAMAGED RC JOINT PANEL ZONE USING CHEMICAL EPOXY INJECTION METHODOLOGY

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**ABSTRACT :**

In this experimental study, the effectiveness of epoxy-injection repair methodology applied to a full-scale, T-shaped beam-column joint specimen was investigated. One full-scale reinforced concrete beam-column joint specimen was designed and built according to the old construction practice and previous Turkish Earthquake Code (TDY-75) with no seismic provisions including smooth reinforcing bars, inadequate transverse reinforcement, and low concrete compressive strength. The beam-column joint was subjected to a reversed cyclic lateral load and a constant axial load equivalent to forty percent of its maximum axial load capacity. The specimen experienced pure shear failure due to the large shear deformation in the joint region. The damaged specimen was then repaired using epoxy-injection technique to seal the large shear cracks and was tested again under the same loading condition. Although, the shear failure was observed again, the Lateral load capacity and consequently the shear capacity was enhanced significantly.

**KEYWORDS:** RC beam-column joint, seismic detailing, shear deficiency, repair, epoxy injection

## 1. INTRODUCTION

Reinforced concrete structures, which were built in Turkey especially before the devastating 1999 Kocaeli and Duzce earthquakes, which caused more than 20,000 lives, lack many seismic deficiencies such as usage of smooth rebars, insufficient amount of longitudinal reinforcement to resist the upgraded seismic forces and lengths of lap splicing, inadequate transverse reinforcement to prevent sudden flexural and shear collapses, low strength and quality of concrete material, strong beam-weak column design principle, and high degree of corrosion in the reinforcement (El-Amoury, 2002, Sezen, 2000, Aydan, 1992, Gur, 2005, Aschheim, 2000, Penzien, 1970). In addition, coupled with these shortfalls, lack of control in construction showed how vulnerable these structures are against the strong earthquakes.

Aftermath earthquake site investigation reports generally point to one particular weakest link in the buildings, which is a failure in the beam-column joint regions (Beres, 1994, Ghobarah, 1997, Antonopoulos, 2003). Figure 1.1 clearly shows this phenomenon. In the presence of transverse beams, column ends usually fail. However, in the case of exterior frames, beam-column joint panels fail before the column ends. Although infill walls provide some damping in the building to certain extend, when they are totally collapsed, the empty frames are exposed to lateral forces, and consequently, they cannot resist for longer periods.

After a moderate earthquake, the building may suffer some damage, and according to its overall structural integrity, it could be saved by upgrading or repairing only the damaged areas. The objective of this paper is to investigate the effectiveness of epoxy-injected repair technique to a damaged beam-column joint panel region. The scope of this experimental study is limited to testing one full-scale T-shaped beam-column joint specimen as control and then as repaired. The results showed considerable improvement in the overall behavior of the repaired specimen.


Figure 1.1 Typical beam-column joint failure in 1999 Kocaeli earthquake (El-Amoury, 2002 and Sezen, 2000)

## 2. TEST SPECIMEN AND SETUP

The specimen was produced with the reinforcing detailing as shown in Figure 2.1. It represents typical pre-1999 construction practice in Turkey. If the specifications were used according to 1975 Turkish earthquake building code (TDY-75), the detailing would have been somewhat different then the actual test specimen. This indicates a major lack of control by authorities in the design project and construction phases. If this specimen were designed and constructed strictly according to TDY-75, it would have been detailed as shown in Figure 2.2.

The dimensions of columns and beam were chosen such that strong beam-weak column design principle exists. Accordingly, the calculated column to beam flexural ratio came out to be around 1.0, which is much less than the minimum requirements indicated in recent codes (1.2 to 1.3) (ACI-318, 2002, TDY-07).

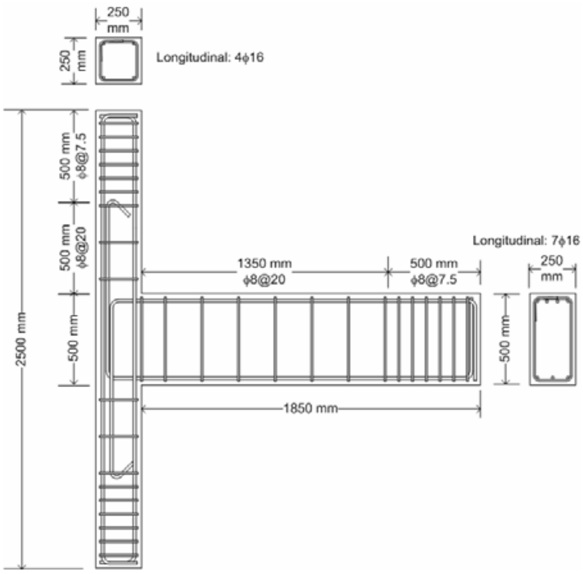



Figure 2.1 Reinforcement details for the test specimen

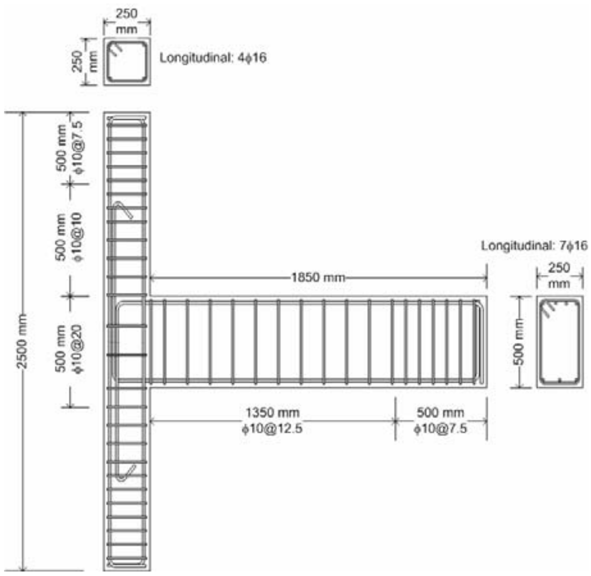



Figure 2.2 Reinforcement details according to TDY-75 code

Out of 3 concrete cylinder samples, an average compressive strength of 13.5 MPa was obtained. Smooth rebars were used for both longitudinal and transverse reinforcements. Tensile tests were performed on these rebars where an average tensile strength of 360 MPa was obtained.

The test set up is shown in Figure 2.3. The specimen was tested in a test rig recommended by ACI Innovation Task Group 1 and Collaborators (ACI T1.1, 2001) where the beam is placed parallel to the strong floor and hinged at its free end, simulating a roller support. The column was placed in vertical position and supported by a universal pin at the bottom end. Constant axial load was applied vertically by a load-control loading system with a 1000 kN-capacity actuator, while horizontal load was applied by a displacement-control loading system through a 250 kN-capacity dynamic actuator.

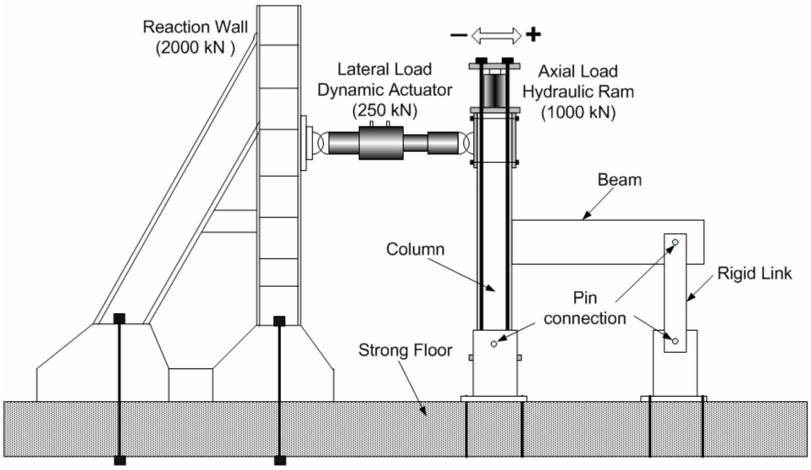



Figure 2.3 The test setup

The amount of the axial force applied was 40% $f'_c A_g$ where $f'_c$ is the characteristic strength of concrete and $A_g$ is the gross cross-sectional area of column. The applied lateral load was of displacement-control type. Three cycles of the same amplitude in every story drift were repeated before the subsequent displacement amplitude increased. The loading cycles are shown in Figure 2.4.

Set of strain gauges and linear variable displacement transducers (LVDT) were mounted on rebars and the specimen to collect data through a data acquisition system.

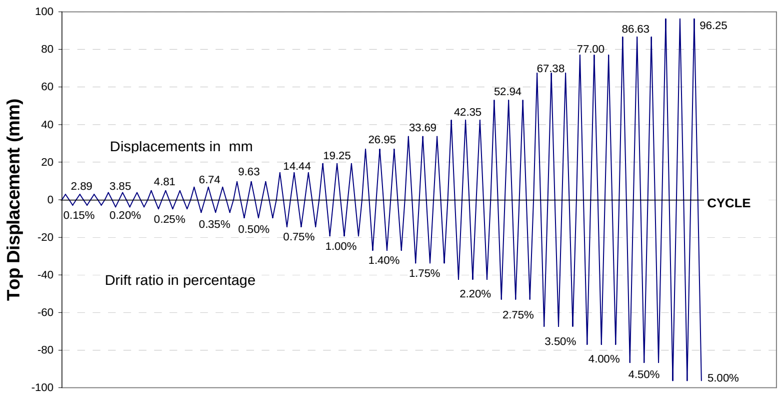



Figure 2.4 Loading pattern

Column tip displacement of specimen as well as moment-curvature and shear deformation values were calculated using LVDT measurements. Strains developed in longitudinal reinforcements of beam and columns, and in transverse reinforcements were also measured to mainly check whether the rebars were yielded.

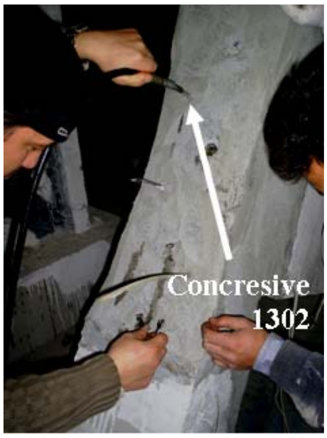
## 3. REPAIR TECHNIQUE

Repair procedure was performed to the already-damaged control specimen. Loose concrete was removed, the surface and the cracks were cleaned using wire brush, and then blown by high-pressure air until the area is free from dust particles. Epoxy injection procedure was started first and several holes were drilled and check valve nipples were placed along the cracks. Large visible cracks were filled with epoxy-based high strength repairing material, *Concresive 1495*, with a compressive strength of 90 MPa. Then, to replace the loose concrete voids, the joint region is covered with epoxy based material, *Concresive 1406* and a cement-based repairing material, *Emaco S88C*. After the cure of the material, low viscosity *Concresive 1302* epoxy was injected with 200-bar pressure through the nipples via special pumps. The repair material properties could be obtained from BASF material catalogue. An illustration of step-by-step repairing technique is shown in Figure 3.1.

Epoxy injection of large cracks

Replacement of loose concrete voids

Replacement of loose concrete voids



High-pressure epoxy injection

Figure 3.1 Repairing procedure

## 4. TEST RESULTS

### *4.1 Control Specimen*

At the first drift level of 0.15%, a flexural crack occurred at the top of the beam region in pull direction. In the following cycles of the same drift, the second crack was occurred at the bottom of the beam, in push direction. The maximum loads observed in this drift level were 14 kN in pull and 12 kN in push directions, respectively.

The first diagonal shear crack at the joint region was observed at a drift level of 0.35%. In pull direction, when the load reached to 23 kN, the crack started from the top corner of the beam-column joint region and extended towards the joint core.

After the 0.50% drift level, more visible damages were observed at joint region. At the drift level of 0.75%, the maximum loads observed in pull and push directions were 27 kN and 24 kN, respectively.

In the subsequent drift cycles, up to the 1.75% drift level, additional shear cracks occurred at the joint region and load carrying capacity started to decrease. When the drift level of 1.75% was applied, the column longitudinal reinforcements started to buckle, and thus, the cover concrete at the back side of the joint totally spalled off. At the same time, the diagonal cracks widened. The test ended due to the load carrying capacity of the specimen decreased below 80% of maximum load. In Figure 4.1, the crack pattern at end of the tests could be observed. Also, force – displacement relationship of this test is provided in Figure 4.2.

Figure 4.1 The cracks occurred in the joint region at the end of the test

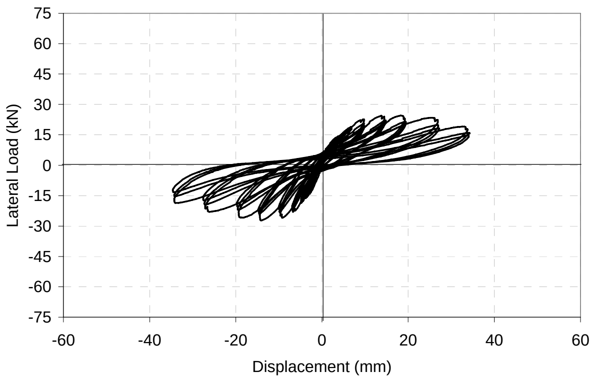



Figure 4.2 Force – displacement relationship of the control specimen

### *4.2 Repaired Specimen*

The repaired specimen was tested in the same manner as the-Control specimen. The first crack was observed at the top of the beam, 90 mm away from the column surface, in the pull cycles. The drift level was 0.25% and the load carrying capacities were 16.5 kN and 24.1 kN for push and pull directions, respectively. Up to drift level of 0.75%, flexural cracks occurred on the beam only.

During the 1.00% drift cycles, the first diagonal crack occurred at the top corner of the beam-column joint region. The recorded loads were 45.3 kN and 63.2 kN for push and pull directions, respectively. After this drift level, the load carrying capacities started to decrease. The first crack on the bam widened further to about 2.5 mm at the drift level of 1.40%. During this drift level, the vertical cracks were observed at the back side of the column at joint region. The reason of these cracks was due to buckling of column longitudinal reinforcements.

The test was ended at 2.75% drift level when the cover concrete at the back side of the column spalled off completely and the column longitudinal reinforcing bars were buckled. Figure 4.3 shows the observed damage and Figure 4.4 illustrates the force – displacement relationship.

Figure 4.3 Cracking of joint region at the end of the test

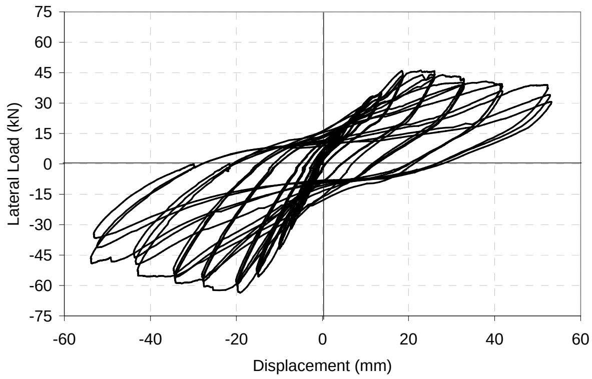



Figure 4.4 Force – displacement relationship of the repaired specimen

## 5. CONCLUSIONS

It has been shown that the lateral load carrying capacity and thus the energy dissipating capacity of the beam-column joint region are increased dramatically with the applied repair technique. The force-displacement relationship is improved from brittle to a more ductile behavior and the pinched shape of the hysteresis curve is also improved. The reason of this is replacing the poor strength and crushed concrete material with high strength repairing materials around the joint region. As a consequence, the applied repair technique increased the performance of the damaged beam column joint significantly.

## ACKNOWLEDGEMENTS

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