First Middle East Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures
8 – 10 February 2011, Dubai, UAE

SMAR 2011

Monitoring Damage of Rehabilitated Beam-Column Joints under Simulated Seismic Loading

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ABSTRACT: Multi-storey RC frames built in the 1960’s are generally considered deficient according to seismic requirements of current design codes and may behave in a non-ductile manner. The lateral load capacity of these structures is often insufficient due to non-ductile reinforcement detailing which includes either inadequate or no transverse reinforcement in the beam-column joint area. Several major earthquakes have demonstrated that the joint capacity is an important factor in maintaining the integrity of the entire structure. Due to the large extent of the problem, it is necessary to develop economic means to upgrade the joint’s capacity to avoid brittle failures and instead shift the frame failure towards beam flexural hinging mechanism. The latter mechanism is a more ductile type of failure and is associated with significant energy dissipation. A simple and effective rehabilitation technique will provide safety to the occupants of the structure. In this study, the behavior of a beam-column joint representative of old constructions is investigated before and after the application of a rehabilitation scheme using FRP. The monitoring of the FRP jacket is used to assess its contribution to the strength of the joint as well as the damage progression during the test.

1 INTRODUCTION

When the 1970’s design codes implemented new beam-column joint design recommendations, existing joints became classified as deficient based on new criteria. Such deficient joints have mainly inadequate or absent joint transverse reinforcement. These deficient RC frames are also referred to as gravity load-designed frames. In the event of an earthquake, reversed loading on a RC frame produces an alternating state of shear stress in the joint and a reversing moment on the beam framing into the joint. Several recent earthquakes made it clear that beam-column joint deficiencies may cause a total collapse of concrete structures since joints are vital elements in keeping structural integrity. Evidence from previous earthquakes such as the 1980 El-Asnam (Algeria) and the 1989 Loma Prieta (California, USA) (Bertero and Shadh, 1980; EERI, 1989) as well as recent earthquakes, such as the 1995 Hanshin-Awaji (Kobe, Japan) and the 1999 Kocaeli (Turkey) earthquakes, shows that in many cases a brittle failure in the frame joints was the major factor behind the total collapse of many structures (Anderson et al., 1996; Mugurama et al., 1995 and Saatcioglu et al., 2001). This created a need for innovative rehabilitation techniques that can handle beam-column joint deficiencies and be easily implemented under different joint configurations and circumstances.

Several beam-column joint rehabilitation schemes emerged since the appearance of new joint design provisions in the post 1970’s design codes. These techniques often involve the use of traditional materials to enhance the performance of joints through jacketing. Several studies have been conducted in order to develop rehabilitation schemes for deficient beam-column joint subassemblages using conventional materials such as reinforced concrete and steel. Work performed by Estrada (1990) and Beres et al. (1992) addressed joint deficiencies using bolted steel plates.

2 EXPERIMENTAL PROGRAM

2.1 Test Specimens

Beam-column joints can be isolated in plane frames at the points of contraflexure. These points are generally represented as the beam's mid-span of the bay and the column's mid-height of one storey to the mid-height of the next storey.

The tested specimen was constructed with the dimensions and reinforcement details shown in Figure 1. The height of the column and the length of the beam represent the distance to the points of contraflexure in the moment resisting frame. Following common practices before seismic design codes availability, the beam-column joint had no transverse reinforcement. With this reinforcement configuration, the beam-column joint specimen is expected to fail in joint shear prior to the formation of a plastic hinge in the beam. The specimen was first tested as-is as a control specimen, then the joint concrete was removed and repaired, then retested. During both tests, the specimen was subjected to constant axial load throughout the test to simulate gravity loads.

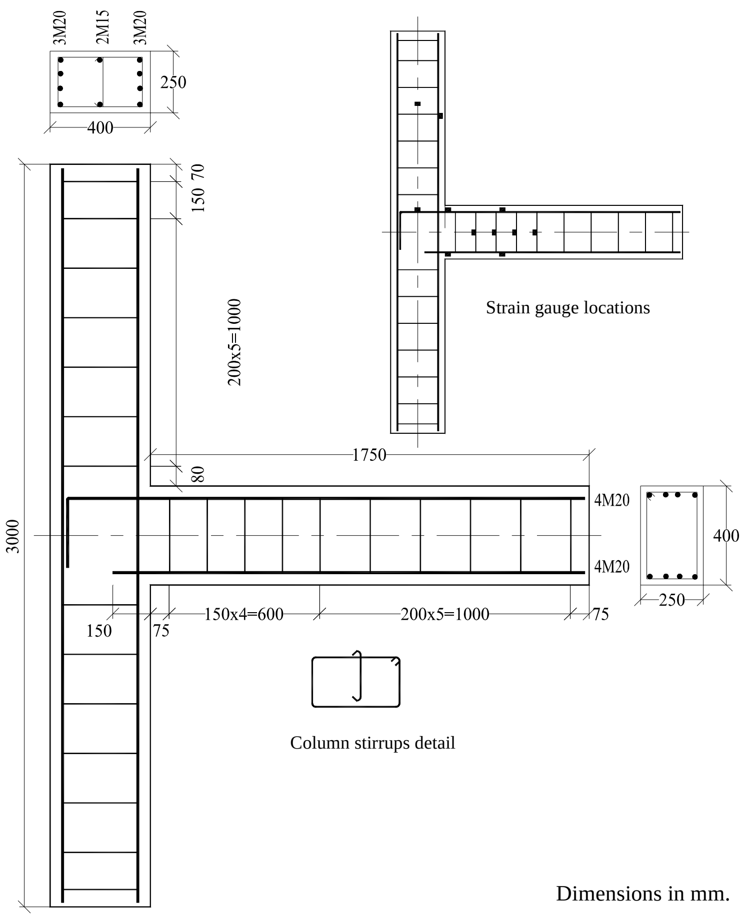

Figure1. Reinforcement details for the tested specimen.

2.2 Rehabilitation scheme

The rehabilitation scheme applied to the specimen (shown in Figure 2) consists of wrapping the reinforced concrete joint area with two U-shaped layers of bi-directional FRP laminates. One layer covered the joint area, and the second layer extended 30 cm above and below the joint area. The fiber directions were placed to correspond with the direction of diagonal tension forces in the joint at 45º with the vertical. A final layer of unidirectional FRP covered an L/6 distance of the column. This configuration provided adequate confinement to the joint and prevented its shear strength deterioration. Subsequently, a steel plate was added using threaded steel bolts to allow the laminate to develop its full capacity through the tying of the free ends of the U-shaped laminates.

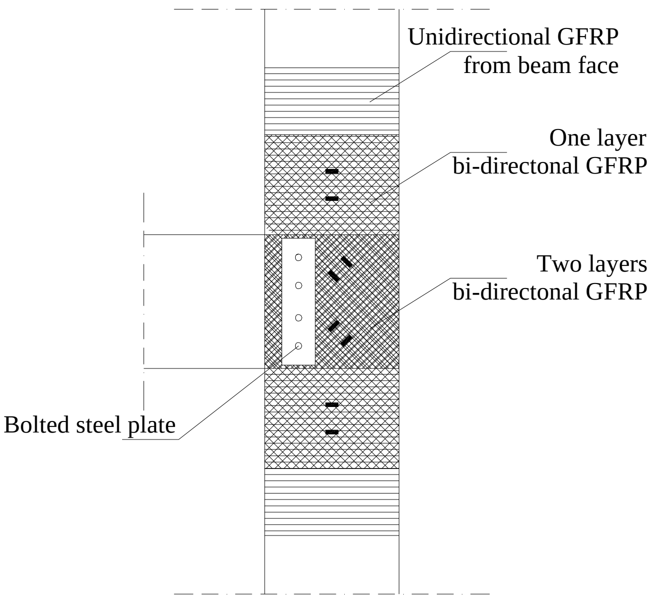

Figure 2. Layout of the rehabilitation scheme.

2.3 Load History

The specimens were tested under constant axial load applied on the column and reversed quasi-static cyclic load applied at the beam tip. The selected loading pattern was intended to induce forces that simulate high levels of inelastic deformations that RC frames undergo during severe earthquakes. The applied load history consisted of two phases. The first phase was load-controlled followed by a displacement-controlled loading phase. In the load-controlled phase, two load cycles were applied until the beam flexural cracking occurred. Subsequently, two cycles were performed at the load causing initial yield of the bottom longitudinal steel bars in the beam. The displacement at initial yield of the steel, δ_y, was recorded and used in the subsequent displacement-controlled phase of loading. In the displacement-controlled phase incremental multiples of the yield displacement are applied (previously recorded at initial yield). Two load cycles were applied at each ductility level to verify the stability of the specimen.

3 RESULTS

3.1 Control and rehabilitated specimens behavior

For the control specimen, the first crack was recorded at the column face at a tip load of 15 kN in both directions. Cracking in the beam increased gradually in the pre-yield cycles. Before first yield of longitudinal steel in the beam, a diagonal shear crack was noted in joint area in each loading direction forming an X-shaped pattern. The specimen was then loaded to yield. The joint shear capacity deteriorated as the specimen underwent deformation equivalent to $1\delta_y$ to $1.5\delta_y$ to $2\delta_y$. At failure, these cracks extended to the back of the column. A considerable degradation in strength occurred at $2\delta_y$, which caused the termination of the test as major chunks of rubble fell from the joint and back of the column area. The specimen failed in classical joint shear failure pattern. The final crack pattern for the control specimen is shown in Figure 3 while the load-displacement relationship is shown in Figure 4.

For the rehabilitated specimen, the first crack was observed at the interface between old concrete and grout. The beam was already cracked from to first test. These cracks widened progressively during the test. At a displacement of $2\delta_y$, the beam became extensively cracked for a distance equal to its depth from the face of the column. Cracking sounds of the jacket were heard but noting detected visually. At a displacement of $2.5\delta_y$, limited delamination was detected mainly on one face of the joint and the top part of the back of the column. In the following cycles, very wide cracks developed in the beam hinge area and rubble started falling. At a displacement of $4\delta_y$, the flexural hinge area lost most of it's concrete and the beam longitudinal reinforcement buckled. The test was stopped as the beam capacity dropped but the axial load and the joint areas were intact. The final crack pattern for the rehabilitated specimen is shown in Figure 3 while the load-displacement relationship is shown in Figure 5.

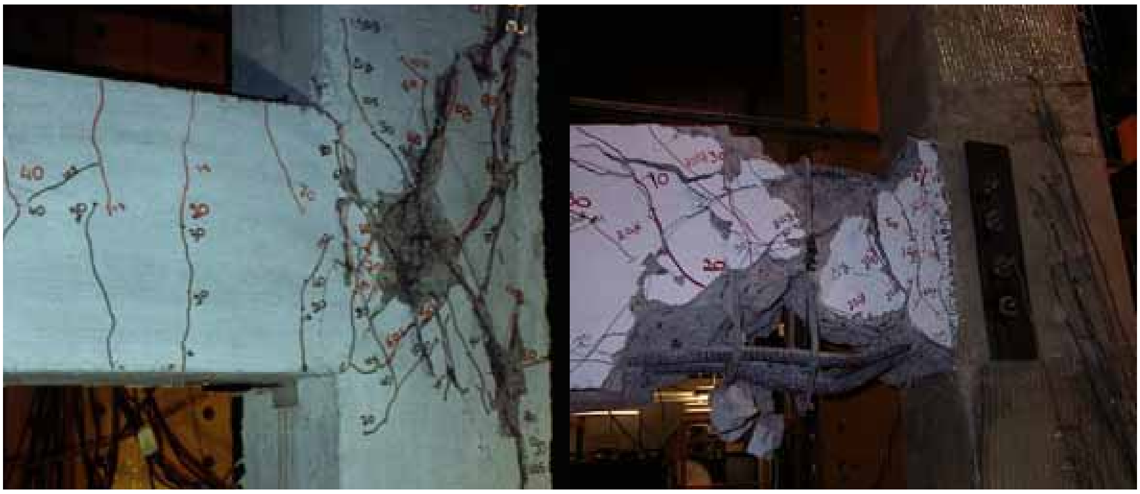

Figure 3. Final crack pattern for the control (left) and rehabilitated (right) specimens.

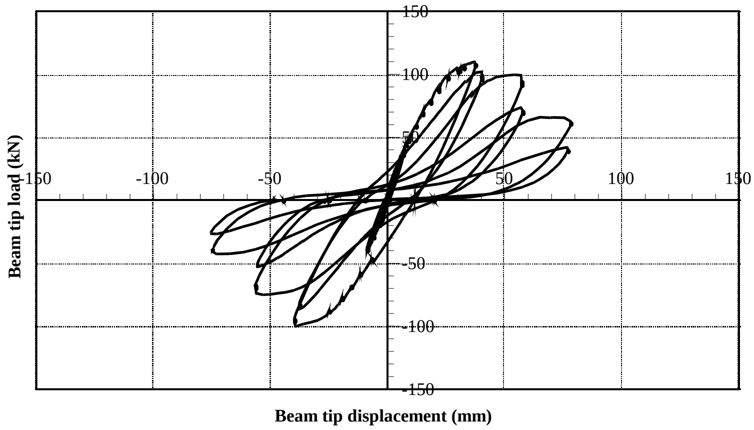

Figure 4. Beam tip load-displacement of the control specimen.

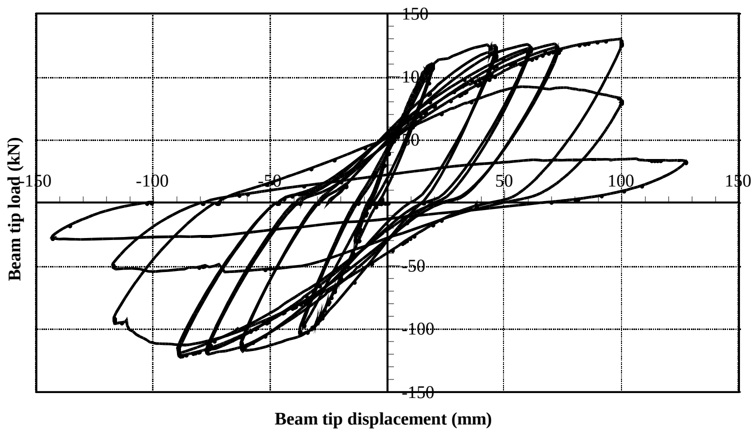

Figure 5. Beam tip load-displacement of the rehabilitated specimen.

3.2 Secant stiffness relationship

Secant stiffness is calculated as the peak-to-peak stiffness of the beam tip load-displacement relationship. Its magnitude represents the specimens' damage through stiffness degradation from one cycle to the subsequent cycle. Loss of stiffness of RC elements during cyclic loading is due several internal damage mechanisms (Priestly et al., 1996). An examination of the secant stiffness plots for the tested beam-column joint specimens, shown in Figure 6, indicates that the rehabilitated specimen had higher initial stiffness. Furthermore, the specimens had initially similar rates of stiffness deterioration. However, as the rehabilitated specimen's performance surpassed the control specimen, the rate of degradation decreased as the joint was able to maintain its integrity. However, the rehabilitated specimen degraded as the beam underwent plastic hinging.

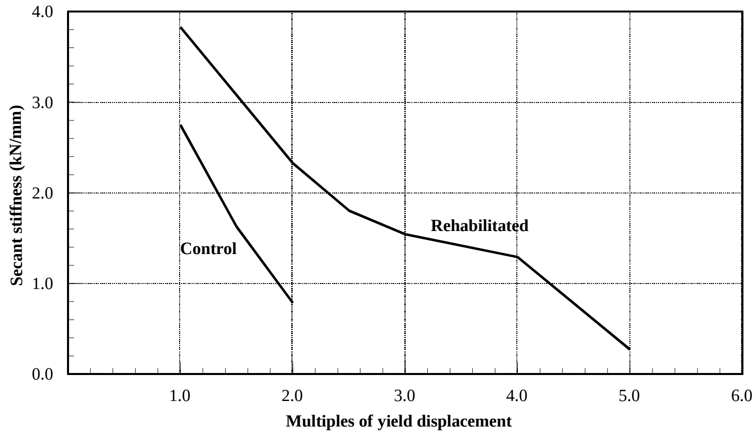

Figure 6. Secant stiffness-multiples of yield displacement plot for the control and rehabilitated specimens.

3.3 Cumulative dissipated energy

The ability of a structure to survive an earthquake depends on its ability to dissipate the energy input from the ground motion. Despite the difficulty in estimating such energy input during a ground movement event, a satisfactory design should have a larger energy dissipation capability of the structure than the demand. The cumulative energy dissipated by the beam-column joint specimens during the reversed cyclic load test was calculated by summing up the energy dissipated in consecutive load-displacement loops throughout the test. The energy dissipated in a cycle is calculated as the area that the hysteretic loop encloses in the corresponding beam tip load-displacement plot. Figure 7 shows plots of the cumulative energy dissipation versus storey drift for the tested specimens. Results show that the investigated joint repair technique enhanced the energy dissipation by about 5 times compared to that of the control specimen, which is a substantial increase.

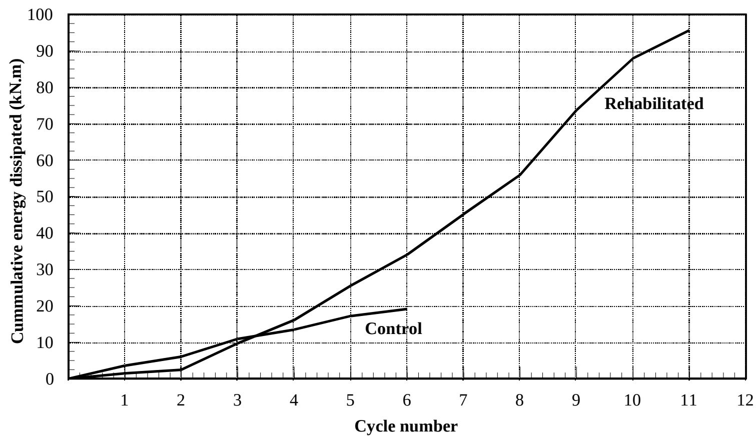

Figure 7. Energy dissipation at different cycles for the control and rehabilitated specimens.

4 CONCLUSION

A seismic rehabilitation scheme using GFRP for seismically deficient beam-column joints has been investigated. A shear deficient beam-column joint subassemblage was tested under cyclic loading. Subsequently, the specimen was retested after repair and the application of the investigated rehabilitation scheme using GFRP overlays was applied. The rehabilitation scheme was successful in critically changing the overall behavior of the specimen since the mode of failure was shifted from joint shear to beam hinging. Other improvements include enhancements to energy dissipation and stiffness deterioration.

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