Experimental Evaluation of New Technique for Seismic Retrofitting of External RC Beam-Column Joint with Non Seismic Detailing

J. Shafaei, A. Hosseini, M. Marefat & A. Arzeytoon
School of Civil Engineering, University of Tehran, Tehran, Iran

SUMMARY:

An innovative seismic upgrading technique for existing reinforced concrete frame connections constructed prior to the 1970s based on enlargement of beam column joint using precast steel plate and post tensioning bars is proposed. Several strengthening techniques are currently available to upgrade the beam-column joint but these methods do not consider the real condition and physical constraint in joint regions. The proposed method not only considers the real condition but also is economical and architecturally acceptable.

Three half-scale beam column specimens were built and tested under quasi-static cyclic loading in University of Tehran Structural Laboratory.

According to test results, the control specimen showed brittle joint shear failure while retrofitted specimens showed beam flexural failure. The strength, stiffness, energy dissipation and ductility of retrofitted specimens were greatly improved. This new technique is effective to reduce joint shear stress and can be moved away the plastic hinge formation from column face, thus preventing joint shear failure.

Key Words: Substandard beam-column Joints, Seismic Retrofitting, new technique, Plastic hinge

1. INTRODUCTION

Several earthquakes have demonstrated many collapses of buildings due to the brittle failure of sub-standard beam-column joint. There are plenty of researches on experimental investigation of RC beam-column joints. The failure of beam-column joint can be very disastrous and the need to retrofit existing beam-column joints to resist earthquake excitation is therefore a critical consideration (Sasmal 2011 and Prasad 2005).

Fig 1.1 shows the exterior joint failure in a reinforced concrete building after the 1999 Izmit earthquake. Exterior beam–column joints are more vulnerable than interior joints, which are partially confined by beams attached to four sides of the joint and contribute to the core confinement. There are some differences between the shear response of interior and exterior joints when subjected to earthquake ground motion due to joint confinement by beams. However, the bond-slip mode of failure of exterior and interior joints is similar (Mady and El-ragaby 2011).

Several methods for retrofitting beam-column joints have been proposed in the past. Concrete jacketing is one of the common techniques. However, this method produces protruding parts of the concrete jacketing, which reduces usable floor space and may make it architecturally unacceptable in many cases. Other researchers have attempted to strengthen beam-column joints by steel plates and fiber-reinforced polymer (FRP) materials but these methods do not considers the real condition and physical constraint in joint regions (Pimanmas and Chaimahawan 2009,2010).

The objective of beam–column joint rehabilitation is to strengthen the shear and bond-slip resistance in order to eliminate these types of brittle failure and ensure instead that ductile flexural hinging in the beam will take place. Recent studies on the effect of shear and bond-slip rehabilitation on the behaviour of reinforced concrete frame have shown significant improvements in the overall frame ductility. It is important to develop effective and economic rehabilitation techniques for upgrading the vulnerable beam–column joints in existing structures (Said and Nehdi 2008)

Figure 1.1. Type of Failure Observed in Pre-1970's Building Structures (Izmit, Turkey Earthquake 1999)

In this paper, a new retrofitting technique with post tensioning bars and enlargement of beam column joint using precast steel plate is proposed. In this method, the beam-column joint is two-dimensionally enlarged by pre cast steel angle. The method is comparatively easy in application. No slab perforation is required. Since conventional materials are used, the method is cost-effective. In terms of architectural concern, the planar joint expansion can be hidden in infill walls or non-structural walls with less than 5% coverage area. The proposed scheme also offers a relatively easy installation. It uses short and stiff members which have a low contribution to the dead load and where give minimal disruption to the building's use.

Like other strengthening methods, the planar joint expansion has limitations. Due to shortened member length, beams and columns are more prone to shear failure. The member stiffness also increases, which may attract more lateral seismic force. Hence, it may be necessary to examine the likelihood of member shear failure and reanalyze the structure with increased stiffness. Moreover, the method is applicable to beam-column joint that lacks sufficient joint shear capacity and beam bar anchorage bond. In case that the frame is prone to column sway mechanism, the proposed technique may not be relevant. Other suitable strengthening methods such as shear wall addition may be considered.

2. EXPERIMENTAL PROGRAM

In this study, a conventional five-storey RC structure with 5 bay has been considered for analysis, design and detailing of exterior beam column joint. (see Fig. 2.1)

It is found that due to sudden discontinuity of the geometry, exterior joints are more vulnerable to Seismic loading than the interior one because it demands to explore additional parameters such as bond-slip of reinforcement. Hence, in the present study, exterior beam-column joint has been chosen for investigating the performance under seismic type loading.

Beam-column joints can be isolated from plane frames at the points of contra flexure. The beam of the current test unit is taken to the mid-span of the bay, while the column is taken from the mid-height of one storey to the mid-height of the next storey. All three specimens had identical dimensions. Specimens AB-2 and RS-P2-2 had a similar reinforcement configuration representing deficient structures from the pre-1970s era, whereas specimen AB-1 represents a standard beam-column joint designed to satisfy ACI 352R-08 (2008) requirements.
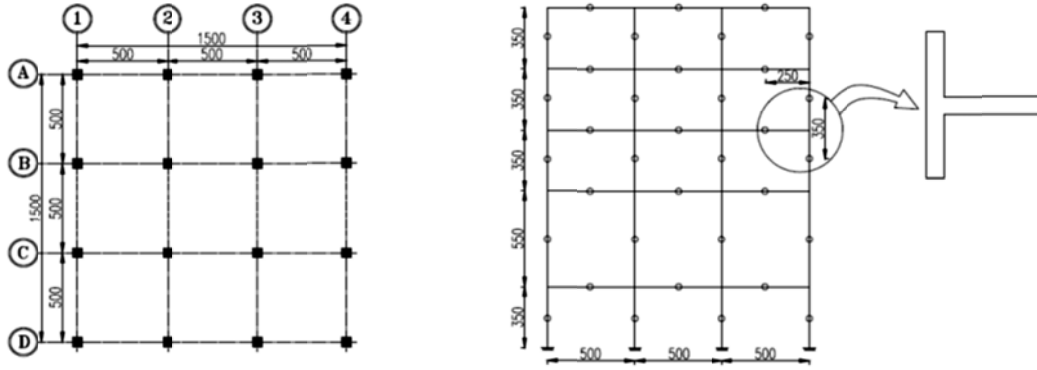
Figure 2.1. Conventional five-storey RC structure

2.1. Standard specimen (AB-1)

The first specimen constructed in this study is shown in Fig 2.2. It is a standard beam-column joint (AB-1) designed to satisfy ACI 318R-08 requirements. It has sufficient shear reinforcement in the joint area, in the column hinging area, and in the beam hinging area. The height of the column and the length of the beam represent the distance to the points of contraflexure in the frame. The column was 2100 mm high with cross section dimensions of 250x250 mm (gross cross sectional area Ag = 62500 mm^2). The beam's length was 1400 mm from the face of the column to its free end with a cross-section of 220x250 mm. The longitudinal reinforcement used in the column was 8ϕ14 bars, corresponding to a 2% reinforcement ratio. The transverse reinforcement in the column was one ϕ 8 closed rectangular ties. The column ties were spaced at 60mm inside the joint and 60 mm for the rest of the column's height. The top and bottom longitudinal reinforcements of the beam respectively were 4ϕ14 and 3ϕ14 bars each corresponding to a 1.22% and 0.91% reinforcement ratio. The transverse reinforcement of the beam is ϕ8 rectangular ties starting at 25 mm from the face of the column. The ties were spaced at 60 mm. The longitudinal rebar size and transverse reinforcement for the joint and hinging zones confinement satisfy current code requirements.
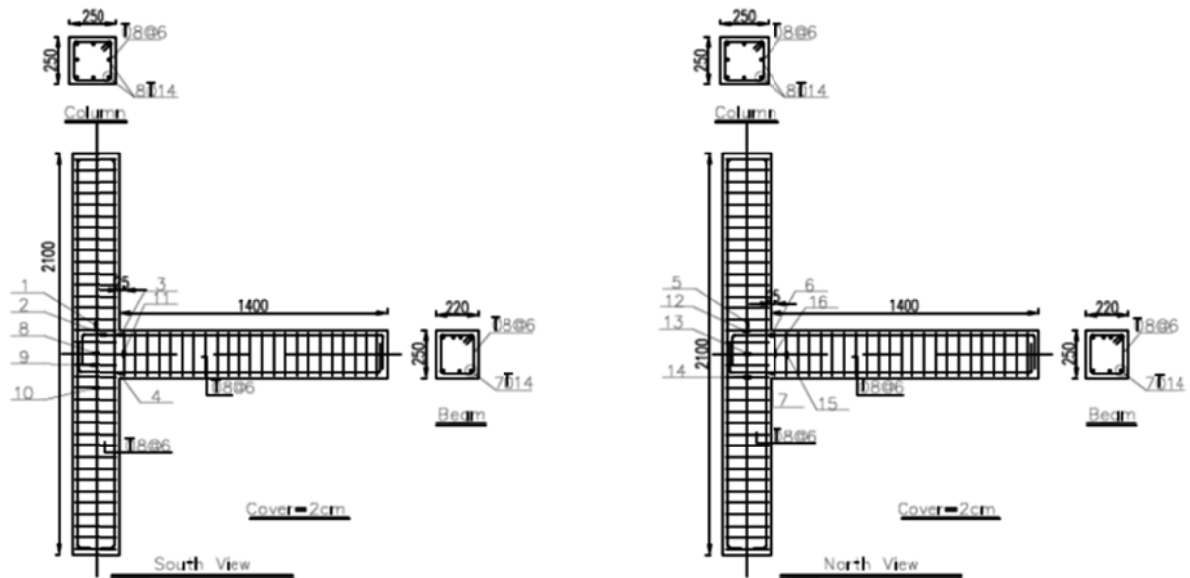

Figure 2.2. Reinforcement details and strain gauge locations for the standard specimen (AB-1)

2.2. Substandard specimen (AB-2) and Rehabilitated specimen (RS-P2-2)

The second and third beam-column joint specimen constructed in this study (AB-2 and RS-P2-2) represents a typical beam-column sub-assemblage designed according to the pre-1970s codes, with similar dimensions to specimen AB-1(Fig 2.3). The longitudinal reinforcement used in the column consisted of 8ϕ14continuous bars corresponding to a 2.2% reinforcement ratio. The transverse reinforcement in the column consisted of ϕ8 rectangular ties. Ties started at 30 mm above and below the beam and were spaced at 60 mm as shown in figure 2.3. Following the common practice prior to current seismic design codes, no transverse reinforcement was installed in the beam –column joint, The top and bottom longitudinal reinforcement of the beam was similar to specimen AB-1. The beam-column joint is expected to fail by joint shear before a plastic hinge is formed in the beam due to the lack of transverse reinforcement in the joint.
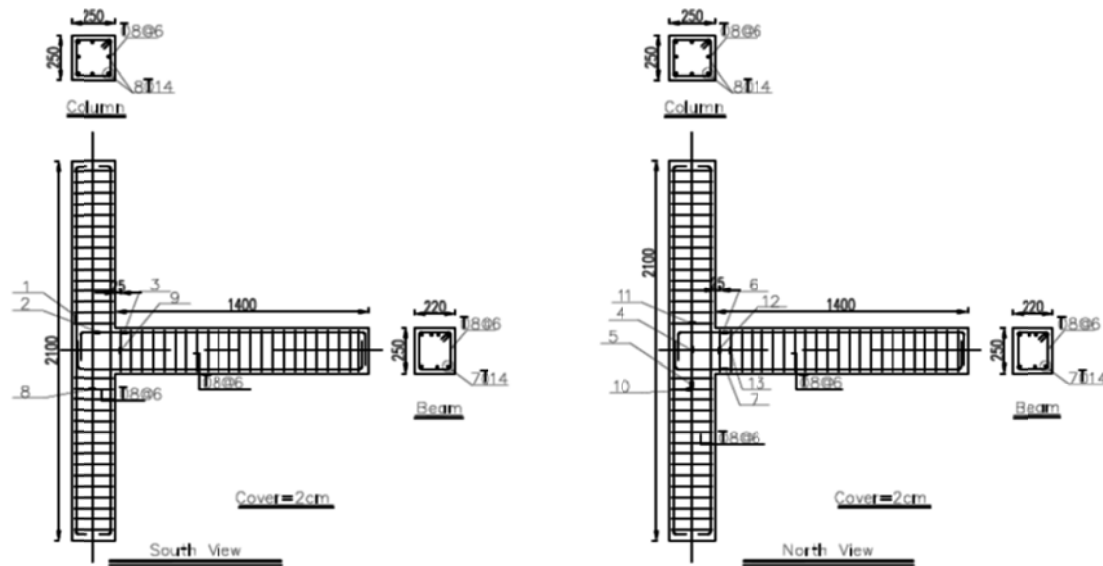

Figure 2.3. Reinforcement details and strain gauge locations for the substandard specimen (AB-2 and RS-P2-2)

2.3. Installation of the Steel Enlargement Member

The steel enlargement members installation process was performed after axially loading the column up to 220 kN in order to simulate the actual case of rehabilitating a deficient beam-column joint in a building in service. The level of axial load in the column affects the effectiveness of the direct strut mechanism of force transfer inside the joint. It can also improve the bond slip behaviour, so that the penetration length of beam bars into the column can be reduced. While some experimental results reported in the literature showed an enhancement to joint behaviour and capacity at higher axial load levels, other studies did not confirm such a benefit. However, joint specimens tested at an axial tensile force of 5% of the column's compressive strength capacity suffered an 18 –50% loss in shear strength. Statistical studies concluded that there was no discernible relationship between the level of column axial load and the shear strength of the joint and suggested that the effect on the deformability of the joint would be more significant.

The bracing system is divided into three steel units, as shown in Fig 2.4, in order to make the steel encasement of the beam and column possible. No holes were drilled through the specimen. The three steel units were then mounted and held in place using high tensile strength M16 rods.

The system proved to be versatile since it could be easily installed for exterior and interior beam-column joints, even in the presence of a slab, with additional simple perforations.
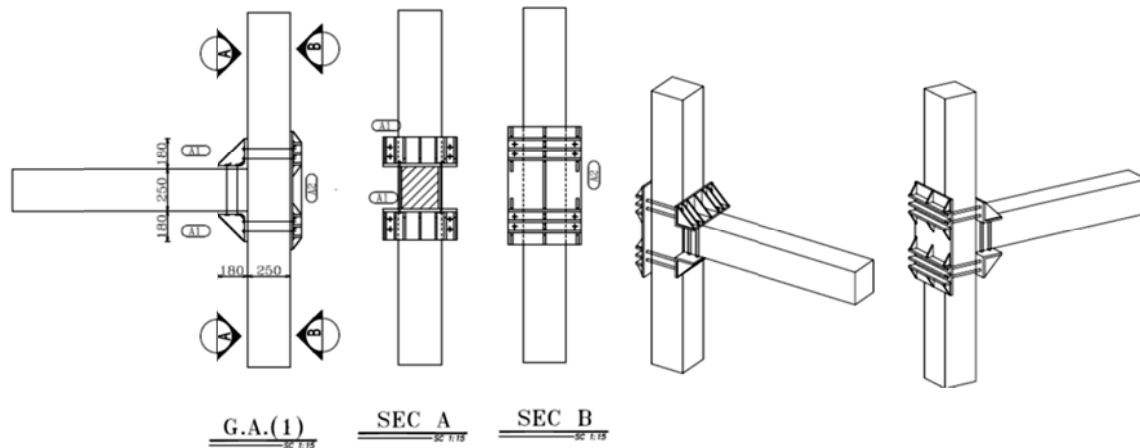
Figure 2.4. 3D view of joint enlargement: geometry and Dimensions of strengthened specimens.

2.4. Material properties and application

The concrete used in constructing the beam-column joint specimen had a compressive strength on the day of the test of 22, 23 and 22.5 MPa for specimen AB-1, AB-2 and RS-P2-2 respectively. The reinforcing steel had a yield stress of 450 and 400 MPa for ϕ14 and ϕ8 bars, respectively.

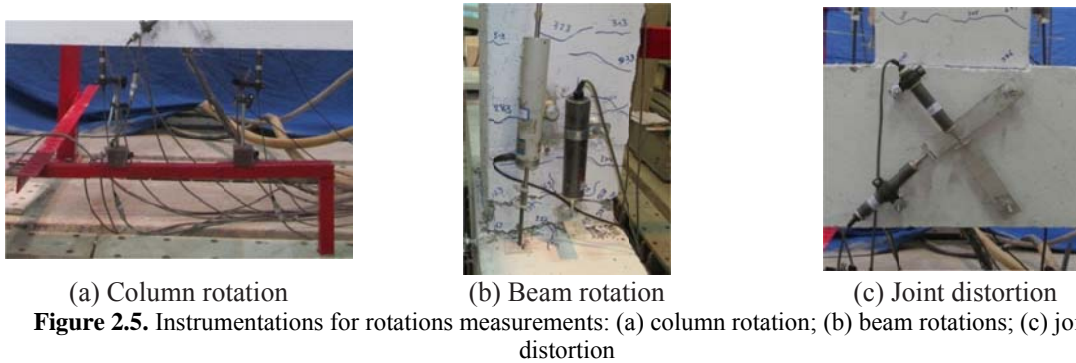## 2.5. Test set-up, instrumentation and load history

Fig 2.6 and Fig 2.7 illustrate the test setup with the specimen supports and other key components. All specimens were tested while the column was lying horizontally and the beam was standing vertically, 90-degree rotated from the actual position. The column was linked to a hinge connector at each end (i.e., top and bottom) and anchored to the strong floor. The end of the beam was linked to an actuator with a swivel connector. Thus, the end of the beam and the top and bottom of the column were all pin-connected in the loading plane, to simulate inflection points of a moment frame subjected to lateral earthquake loading. The column pin-to-pin story height (lc) was 1700 mm, while the beam length (lb) between the loading point and the beam-joint interface was 1250 mm.

Uni-axial reversed cyclic loading was statically applied at the end of the beam, parallel to the longitudinal direction of the column, by a hydraulic actuator with a +/− 100 kN loading capacity and a +/− 200 mm linear range. Positive and negative loading directions are indicated in Fig 2.6. Also, a 1000-kN-capacity hydraulic jack was positioned horizontally to apply a concentric load on the column. The two ends of the column were restrained against both vertical and horizontal displacements, whereas their rotations were allowed (hinged boundary conditions).

Instrumentation used in each specimen was as follows.

For each specimen, a total of 15 electrical resistance strain gauges were attached to the reinforcing bars and stirrups at critical locations to measure strains [Fig 2.2 and Fig 2.3]. Also, 13 linear variable displacement transducers (LVDTs) were used to measure the beam and column rotations, and the joint distortion (Fig 2.5). Furthermore, two load cells were used to monitor the column axial load and the vertical reaction at one of the column ends (Fig 2.6).

A microcomputer controlled data acquisition system is used to record the data.

(a) Column rotation   (b) Beam rotation   (c) Joint distortion

Figure 2.5. Instrumentations for rotations measurements: (a) column rotation; (b) beam rotations; (c) joint distortion

Each specimen was tested under reversed cyclic load applied at the beam tip. The selected load is intended to cause forces that simulate high levels of inelastic deformations that may be experienced by the frame during a severe earthquake. Fig 2.8 shows the pattern of cyclic lateral displacements applied by the actuator during each test.

A total of 13 displacement cycles were statically applied up to 6% drift ratio. Three consecutive same-drift cycles were tested to examine strength and stiffness degradations under reversed cyclic loading based on the performance acceptance criteria specified in ACI 374.1–05 (ACI T1.1).
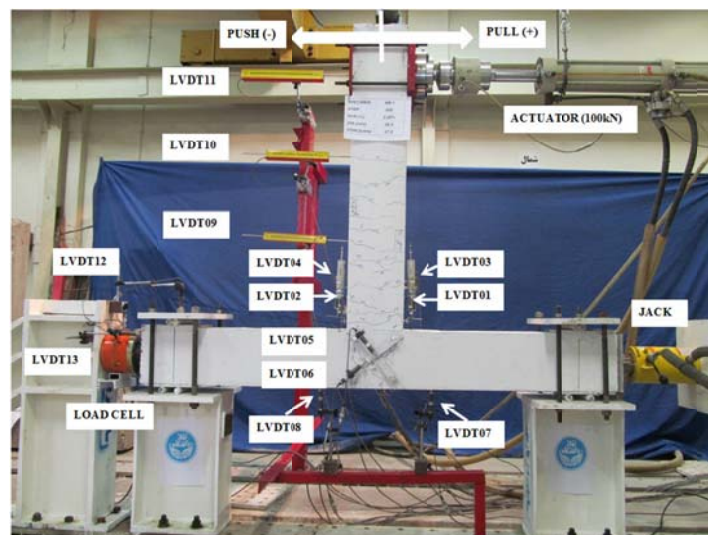

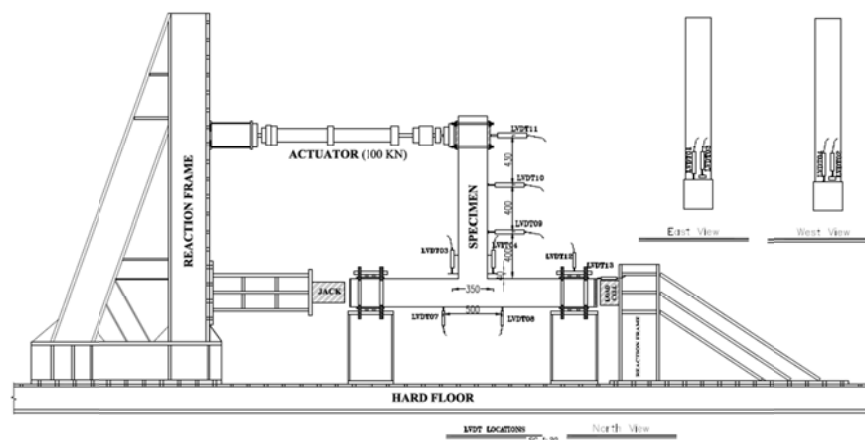**Figure 2.6.** Overview of AB-1 test specimen and setup

Figure 2.7. Test setup: schematic drawing and dimensions for instrumentation, front view;
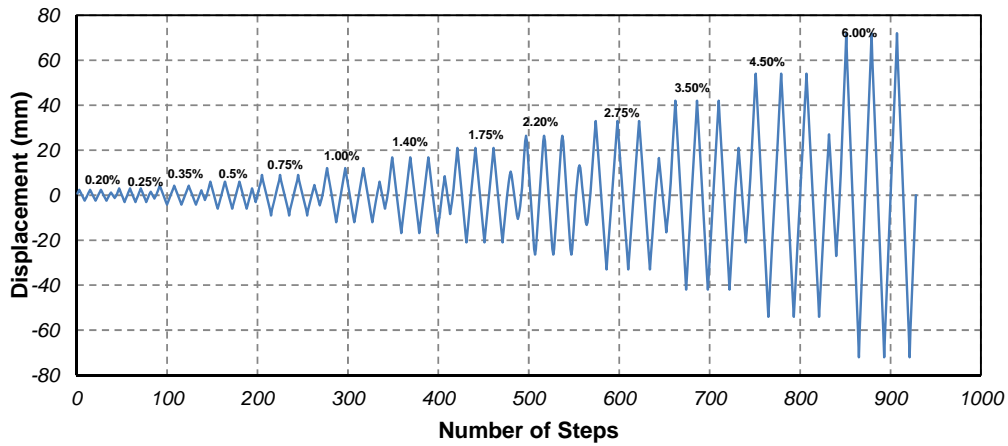
Figure 2.8. Load history for the reversed cyclic load test used in this study

2.6. Test Procedure

For all specimens, an axial load of 220 kN (16% of the column capacity) was applied to the column and maintained constant throughout the test. The loading procedures for all specimens were based on acceptance criteria specified in ACI 374.1–05.

After application of the 220 kN axial load to the end of the column, a quasi-static cyclic load was applied at the beam tip in a displacement-controlled mode. Quasi-static cyclic loading gives conservative estimate of the strength as the dynamic forces due to earthquakes increase the strain rate and, hence, the strength and stiffness.

3. EXPERIMENTAL RESULTS AND OBSERVATIONS

The analysis of results includes comparisons of damage patterns, load-displacements hysteretic responses, for the three half-scale beam-column joint specimens tested in this study. In this analysis, the performance of the standard specimen (AB-1) represents the desired target performance of joints designed using current code provisions. The behaviour of the deficient specimen (AB-2) is a benchmark representing the performance of joints designed as per pre-1970s design codes. The behaviour of the rehabilitated specimen (RS-P2-2) indicates how successful the proposed new technique rehabilitation scheme is in upgrading the deficient performance of (AB-2) to the target standard performance of (AB-1).

3.1. Standard specimen (AB-1)

The hysteretic load-displacement plot and damage pattern for the standard specimen is shown in Fig 3.1. First flexural cracking of the beam section subjected to maximum bending appeared at 0.2% drift. The yield of the beam's longitudinal steel was reached at an average beam tip load of 28 kN and the corresponding yield drift was 1% (based on an average of push up and pull down values). The onset of diagonal cracks in the joint area took place at a beam tip load of 33 kN, corresponding to a drift of 2.2%. Additional cracks in the joint area appeared thereafter as loading progressed, but remained within a very fine width throughout the test. At a ductility factor of 2, the beam became extensively cracked along a distance equal to its depth from the face of the column. At a ductility factor of 4, wide cracks developed in the hinge area of the beam and rubble started falling. The load was terminated at 6% drift but the column's axial load was maintained and the joint area remained intact, except for the presence of fine cracks.

The specimens exhibited ductile load-displacement behavior without showing notable strength drops by the end of testing (up to 6% drift ratio). The final crack pattern of the standard specimen is shown in Fig 3.1.
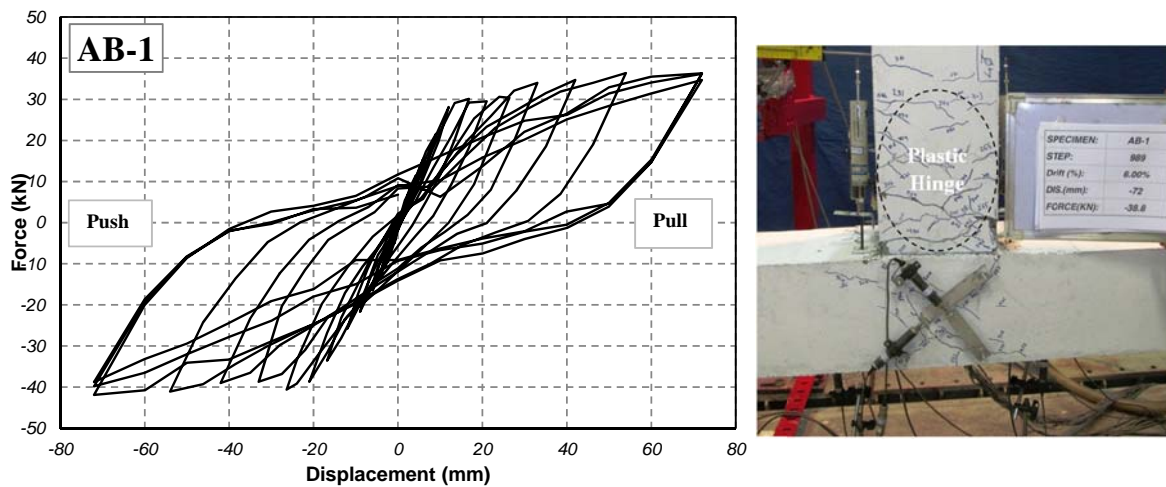

Figure 3.1. damage pattern and hysteretic load-displacement for specimen AB-1

3.2. Substandard specimen (AB-2)

The hysteretic load-displacement plot and damage pattern for the substandard specimen is shown in Fig 3.2.

The control specimen AB-2 failed by concrete crushing in the joint panel (Fig 3.2). In the experiment, the first diagonal crack occurred in the joint panel at 0.75% drift. The spalling of concrete in the joint panel started at 2% drift, indicating the imminent joint failure.

Here, the drift ratio (δ) was determined as the beam end displacement divided by the beam length from the loading point to the column face. The hysteretic responses were typical in that they exhibited pinching (the middle part of each hysteretic loop was relatively narrow), as well as stiffness and strength degradations during repeat same-drift cycles. The test was stopped as the beam shear force dropped at 4.5% drift.

The failure point is defined as the point where the strength of specimen drops more than 20% in the next cycle, i.e., the remaining strength in the next cycle is 80% of the strength at failure point.

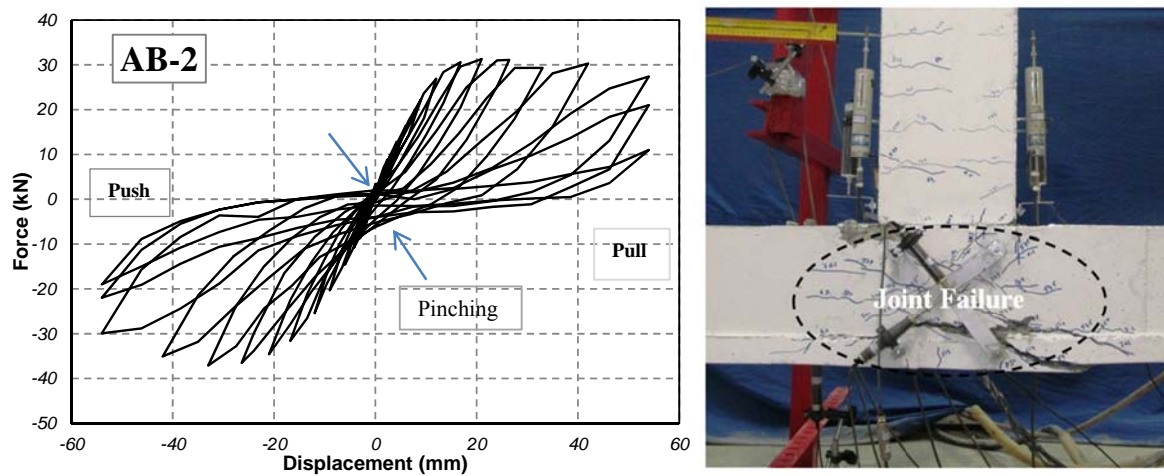The final crack pattern of the standard specimen is shown in Fig 3.2.

Figure 3.2. damage pattern and hysteretic load-displacement for specimen AB-2

3.3. Retrofitted specimen (RS-P2-2)

The hysteretic load-displacement plot for the Retrofitted specimen is shown in Fig 3.3.Retrofitted specimen (RS-P2-2) failed by flexural yielding in beam with moderately ductile performance (Fig. 3.3) The first flexural cracks occurred at the edge of enlarged areas in beams at 0.25% drift. Diagonal cracks formed in the joint panel at 1.5% drift but did not widen in subsequent cycles. The flexural cracks in beam started to actively grow at 2% drift. The concrete cover in the beam started to spall off at 2.5% drift, causing buckling of longitudinal beam bars. Stirrups in beams were broken at around 3.5% drift. The load was terminated at 6% drift. Retrofitted specimens failed by beam flexural failure with high ductility.
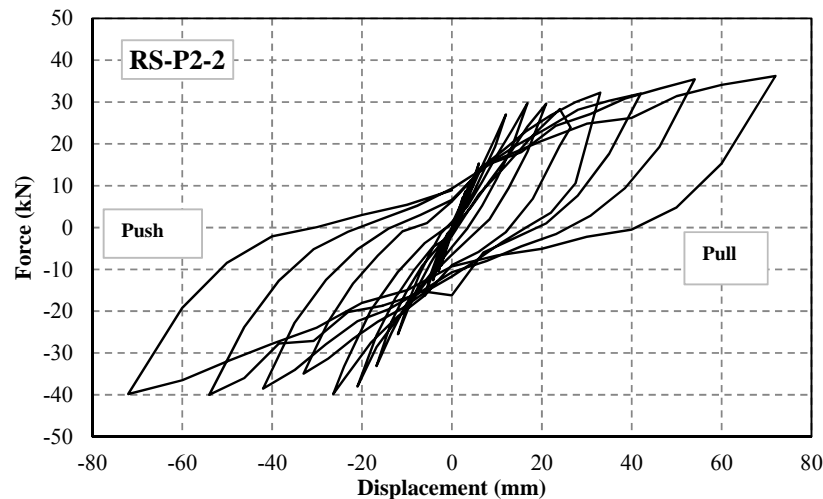

Figure 3.3. hysteretic load-displacement for specimen RS-P2-2

4. CONCLUSIONS

In this paper, a new seismic retrofitting method of beam-column joint is proposed. It is based on two-dimensional enlargement of beam column joint using precast steel plate and post tensioning bars. An experimental program was conducted to enhance the behaviour under cyclic loading of a pre-1970s beam-column sub-assemblage using an innovative and practical rehabilitation technique to demonstrate the performance of the method. The performances of a control deficient specimen (AB-2), a standard specimen (AB-1) and a rehabilitated specimen (RS-P2-2) under reversed cyclic loading were evaluated and compared. Based on the presented experimental observations' and test results the following conclusions are drawn:

1. Retrofitting substandard beam-column connection by enlargement of joint using precast steel plate and post tensioning bars is effective to increase stiffness, ultimate strength, energy dissipation and ductility.
2. The planar joint enlargement shifts the location of plastic hinge from column face to the edge of joint expansion. The failure mode is changed from brittle joint shear failure to bending failure in beam.
3. A significant reduction of horizontal joint shear stress is observed in retrofitted specimens due to the reduction in strains of longitudinal beam bars at column faces. A significant increase in column depth to bar diameter ratio is also advantageous for the anchorage bond performance.
4. The proposed technique significantly delayed brittle joint shear failure and beam bottom reinforcement slippage.
5. Proposed technique mobilized part of the beam and column to resist the shear input to the joint. Thus, shear failure of the beam column joint was substantially delayed.
6. The proposed upgrade technique can move the plastic hinge away from the column's face, thus enhancing the beam bottom reinforcement anchorage.

7. The original specimen with no reinforcement in the joint area showed a high rate of strength deterioration once yielding of the longitudinal steel bars in the beam reached. This was due to the brittle shear failure of the joint.

8. The main objective of retrofit design is to avoid joint shear failure and to encourage beam flexural failure. In terms of structural performance, post tensioning and the joint enlargement is designed to increase the joint shear capacity through increasing effective joint area, to improve the anchorage bond of longitudinal beam bar within the joint through increasing apparent column depth and to prevent lap splice failure of column longitudinal bars above the floor level.

ACKNOWLEDGMENTS

REFERENCES

ACI Committee 318 [2008] Building Code Requirements for Structural Concrete (ACI 318–08) and Commentary (ACI 318R–08), American Concrete Institute, Farmington Hills, Michigan, USA.

ACI Committee 374 [2005] Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary (ACI 374.1–05), American Concrete Institute, Farmington Hills, Michigan, USA.

Chaimahawan, P., & Pimanmas, Æ. A. (2009). Seismic retrofit of substandard beam-column joint by planar joint expansion. *Materials and Structures*, 443-459.

Mady, M., El-ragaby, A., & El-salakawy, E. (2011). Seismic Behavior of Beam-Column Joints Reinforced with GFRP Bars and Stirrups. Composites, (December), 875-886. doi:10.1061/(ASCE)

Pimanmas, A., & Chaimahawan, P. (2010). Shear strength of beam–column joint with enlarged joint area. *Engineering Structures*, 32(9), 2529-2545. Elsevier Ltd.

Prasad, R., Pan, T.-chien, Irawan, P., Tsai, K.-chyuan, Lin, K.-chun, & Chen, C.-hsin. (2005). Experimental study on the dynamic response of gravity-designed reinforced concrete connections. *Engineering Structures*, 27, 75-87.

Said, a, & Nehdi, M. (2008). Rehabilitation of RC frame joints using local steel bracing. *Structure and Infrastructure Engineering*, 4(6), 431-447.

Sasmal, S., Ramanjaneyulu, K., Novák, B., Srinivas, V., Saravana Kumar, K., Korkowski, C., Roehm, C., et al. (2011). Seismic retrofitting of nonductile beam-column sub-assemblage using FRP wrapping and steel plate jacketing. *Construction and Building Materials*, 25(1), 175-182. Elsevier Ltd.