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Behaviour and Retrofitting of Beam-Column Joint under Seismic Loading

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Abstract: This paper proposes a rapid rehabilitation scheme for repairing moderately damaged reinforced concrete (RC) beam-wide column joints. Reinforced concrete beam-column joints are commonly used in structures such as parking garages and road overpasses, which might be exposed to extreme weathering conditions and the application of deicing salts. The objective of this research project is to assess the seismic behaviour of concrete beam-column joints reinforced with glass (G) FRP bars and stirrups. In current codes and recommendations for seismic design and evaluation, simple expressions are used typically to design the joint, and a strut-mechanism approach has been adopted to assess the strength. However, prior and ongoing research has shown that joint behaviour is more complicated than implied by these documents and that defining failure by static strength alone is not sufficient to describe performance.

Keywords: Beam-Column joint, ductility, stiffness, retrofitting.

Introduction

In moment resisting reinforced concrete RC framed buildings, corner joints are generally found at the roof level. These joints, if designed only for gravity loads and based on preseismic codes, may suffer substantial damage during earthquakes due to inadequate shear reinforcement in the joint region. Several techniques of repairing and strengthening of RC joints, damaged by earthquakes, have been reported in earthquake prone countries such as Japan, Mexico, and China. Of the various repair techniques used, the most common involved were RC or steel jackets. The shear strength and confinement pressure provided by joint panel stirrups are crucial to preserve joint panels from premature brittle failures; a suitable amount of transverse reinforcement allows the action between the beams and columns to be appropriately transferred. However, the lack of joint panel transverse reinforcement is very common in structural systems designed for gravity load or according to obsolete seismic code especially in the Mediterranean area. For this reason, several surveys carried out in the aftermath of major recent earthquakes have shown that beam column joints represent one of the main sources of vulnerability in existing reinforced concrete (RC) constructions. Most previous studies have concentrated on joints that contain transverse reinforcement, with the objective of improving joint response. However, a large number of older reinforced concrete frames exist and these frames lack joint reinforcement. Limited research has been conducted on such joints, and data are needed to support the development and refinement of guidelines for evaluating them.

Fig. 1

Typical damaged structure, Figure 1, after an earthquake demonstrates that the failure of beam-column joints is the major contributor for the collapse of buildings due to earthquake excitation. It needs for engineering approach to adopt efficient and economical methods to improve the joint performance. The free-body diagram shown in Fig. 2 is used as the basis of design criteria for anchorage of reinforcement at interior beam column joints. When the capacity design philosophy is used the magnitude of the tensile stress is assumed to equal the over strengths tress of the reinforcing bar (i.e., the maximum stress likely to occur in the reinforcement, including an allowance for strain hardening and material property variation), and the magnitude of the compression stress is expected to be less than the over strength stress of the reinforcing bar. The equilibrium of the reinforcing bar must be maintained, and in the absence of mechanical anchors, the equilibrating force is generated by bond stresses along the length of the bar passing through the joint.

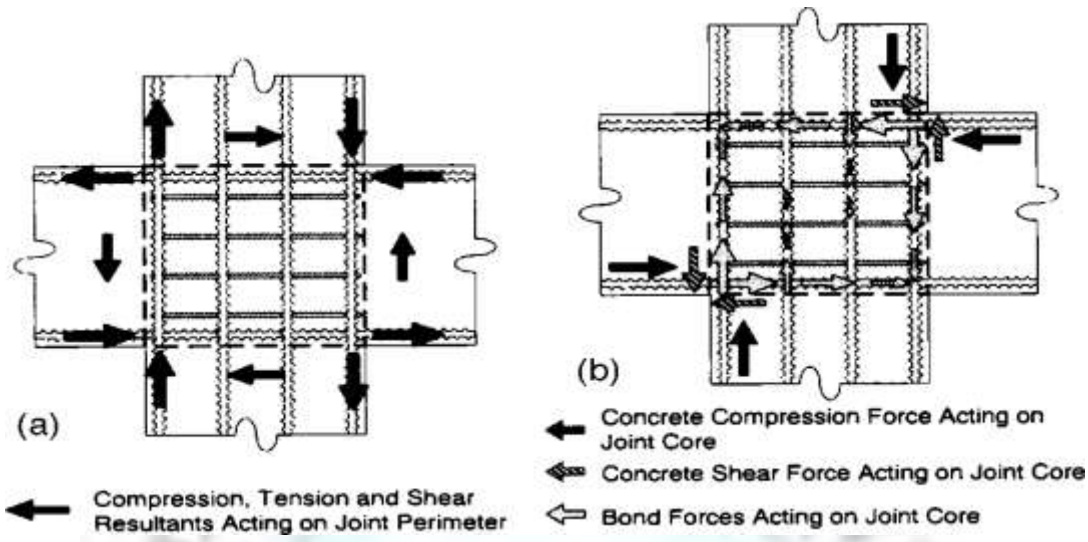

Fig. 2

1. Research Objectives

The overall objective of the current investigation is to provide a comprehensive treatment, within a capacity design framework, of the aspects of supply and demand in seismic shear performance of beam-column connections. In the context of studying the joint shear behaviour a realistic estimate of the shear capacity is deemed crucial. This will provide consistent reliability levels in nondissipative parts of the structure. The more detailed objectives of this paper are to (1) investigate the response of beam-column sub assemblages to vertical ground excitation; (2) provide a better understanding of the shear transfer mechanisms in joints through a controlled mode of failure-testing scheme; (3) quantify the effect of variation of axial column load, as an aspect of demand, on the PZ shear strength and deformability; and (4) emphasize the role of vertical ground acceleration as a potential design parameter.

2. Experimental and Analytical Investigation

The experimental study on exterior beam column joint of a multi-storey reinforced concrete building in falling under the seismic Zone – III has been analyzed using STADD.pro. The specimens were designed for seismic load according to IS 1893 (Part I):2002 & IS 13920: 1993. The structure is five storey two bay frames including 1.5 m foundation depth. The maximum moment is occurred at the ground floor roof level. We consider that particular joint for the experimental study.

2.1 Details of specimen

The test specimen was reduced to 1/5 th scale to suit the loading arrangement and test facilities. Prototype specimen having beam dimension of 305 X 460 including slab thickness and column dimension of 305 X 460. For testing model the dimension of beam was 120 X 170 mm without slab thickness and beam length of 450mm and that column size was 120 X 230 mm. Height of the column was 600mm.

2.2 Reinforcement details

The reinforcement details of beam column joint are shown in Figure 3 Main reinforcement provided in the beam was 10 mm diameter bars, 3 No's at top and 3 No's at bottom. The stirrups are 6 mm diameter bars at 30 mm c/c for a distance of 2d, i.e. 300 mm from the face of the column and at 60 mm c/c for remaining length of the beam. The longitudinal reinforcement provided in the column was 8 No's of 8 mm diameter bars equally distributed along four sides of column. The column confinements are 6 mm diameter bars at 30 mm c/c for a distance of 150 mm from the face of the column and at 60 mm c/c for remaining length of the column.

Fig. 3: Ductile Detailing of Beam Column Joint as per IS 13920; 1993.

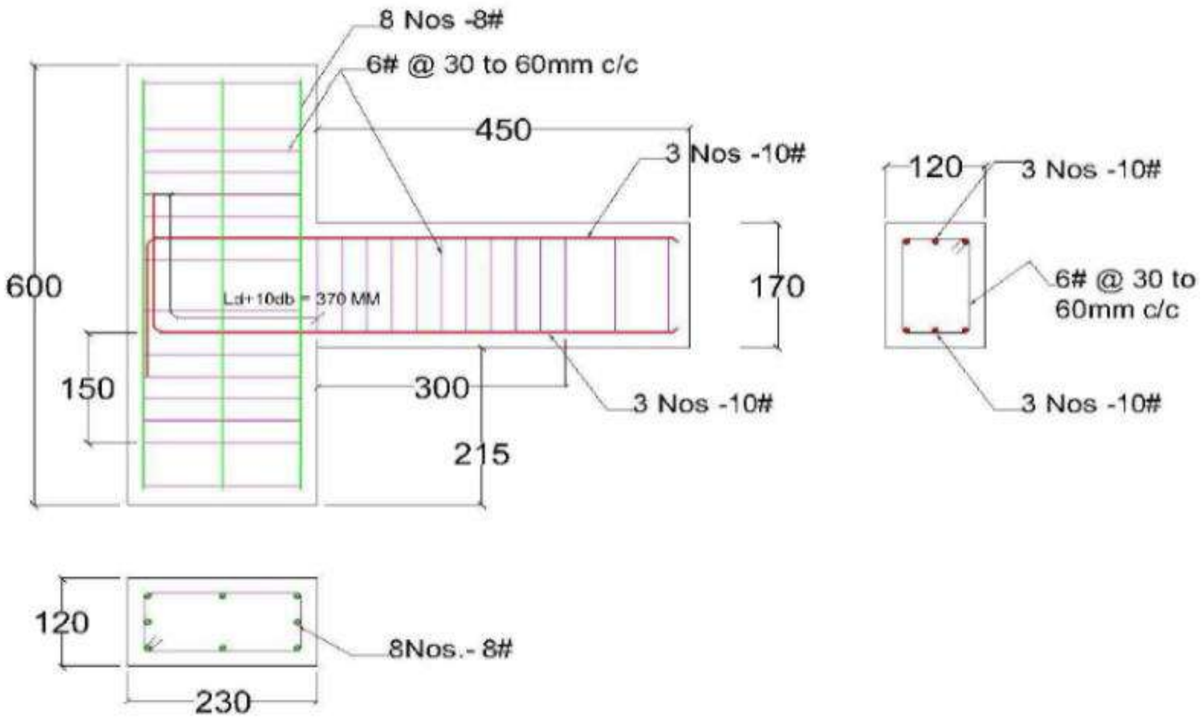

Fig. 3

2.3 Test setup and instrumentation

The specimen was tested in a reaction frame. A hydraulic jack was used to apply the axial load for column. To record the load precisely a proving ring was used. The load is applied forward cyclic and reverse cyclic and deflection measured from every 3 KN by using LVDT. The deflection was measured at the beam free end tip. Loading is applied gradually such as 3,6,9,12,15 KN respectively for forward direction and 3, 6, 9, 12, 15 KN respectively for reverse direction.

2.4 Preparation of Retrofitted Specimens

The Exterior beam column joint specimen named as SL1 (Single Layer) & DL2 (Double Layer) was tested subject to quasistatic cyclic loading simulating earthquake loads. The Load was applied by using screw jack totally 5 cycle were imposed. The beam column joint was gradually loaded by increasing the load level during each cycle. The load sequence consists of 3kN, 6kN, 9kN and upto 70% ultimate load. The deflection measured at tip during the cycle of loading. As the load level was increased in each cycle.

2.5 Load Carrying Capacity

The first crack was witnessed during 4 th cycle at the load level of 17.0kN. As the load level was increased, further cracks were developed in other portions. The 70% of ultimate load carrying capacity of the SL1 & DL 2 specimens was 20.0kN recorded at 5th cycle. After the testing of the two specimens the cracks are filled with the fiber glass paste mixed with unsaturated polyester resin. The surface area was covered with help of fiber glass paste in all direction. The Spalling of the concrete portion are filled with this fiber glass paste.

2.6 Loading and load deflection behaviour

The GFRP wrapped specimen was subjected to quasistatic cyclic loading simulating earthquake loads. The load was applied by using screw jack Totally 14 cycles were imposed. The cyclic load versus deflection was presented in Figure 4.

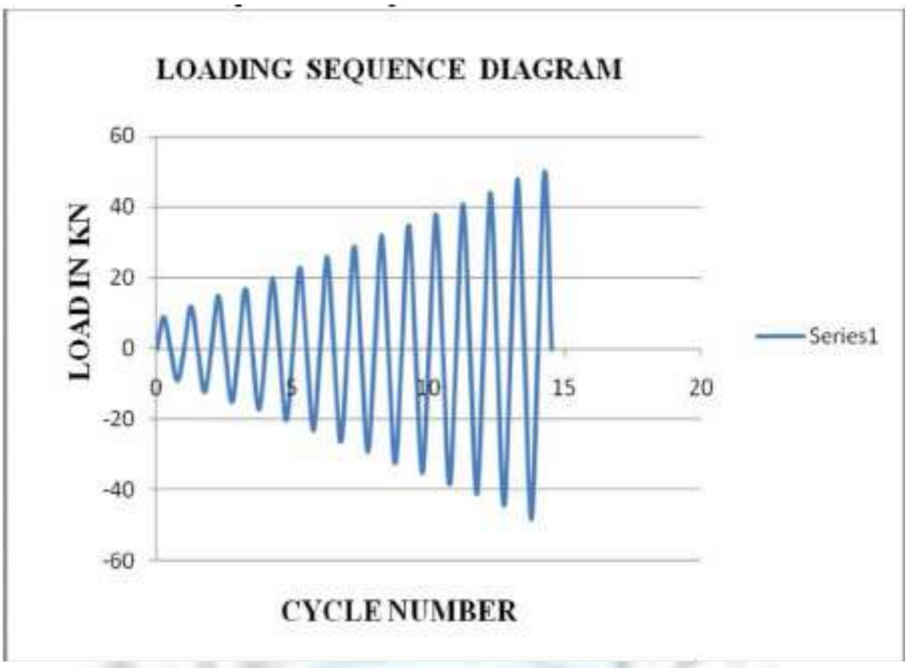

Fig. 4: Load sequence Diagram

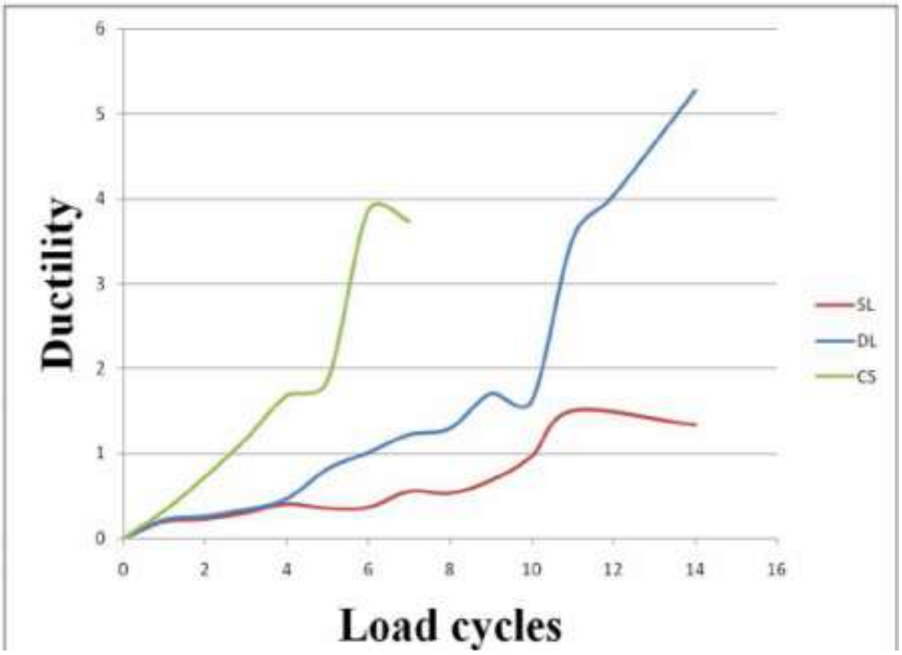

Fig. 5: Comparison of Ductility factor for Control & Retrofitted Specimens

3 Retrofitting Methods

3.1 Surface treatment

Surface treatment is a common method, which has largely developed through experience. Surface treatment incorporates different techniques such as ferrocement, reinforced plaster, and shotcrete. By nature this treatment covers the masonry exterior and affects the architectural or historical appearance of the structure.

3.2 Ferrocement

Ferrocement consists of closely spaced multiple layers of hardware mesh of fine rods (Fig. 6) with reinforcement ratio of 3-8% completely embedded in a high strength (15-30 MPa) cement mortar layer (10- 50 mm thickness). The mortar is troweled on through the mesh with covering thickness of 1-5 mm. The mechanical properties of ferrocement depend on mesh properties. However, typical mortar mix consists of 1 part cement: 1.5-3 parts sand with approximately 0.4 w/c ratio (the ferrocement network, Montes and Fernandez 2001). The behaviour of the mortar can be improved by adding 0.5-1% of a low-cost fiber such as polypropylene.

Fig. 6

In order to reduce the mortar cost, it is possible to replace 20% of cement by fly ash or rice-husk; this replacement increases durability and decreases overall porosity as well as makes the mortar more plastic with a limited effect on overall strength (the ferrocement network). Ferrocement is ideal for low cost housing since it is cheap and can be done with unskilled workers. It improves both in-plane and out-of-plane behaviour. The mesh Helps to confine the masonry units after cracking and thus improves in-plane inelastic deformation capacity. In a static cyclic test (Abrams and Lynch 2001), this retrofitting technique increased the in-plane lateral resistance by a factor of 1.5. Regarding out-of plane behaviour, ferrocement improves wall out-of-plane stability and arching action since it increases the wall height-to-thickness ratio.

3.3 Reinforced plaster

A thin layer of cement plaster applied over high strength steel reinforcement can be used for retrofitting (Sheppard and Tercelj 1980). The steel can be arranged as diagonal bars or as a vertical and horizontal mesh. In diagonal tension test and static cyclic tests, the technique was able to improve the in-plane resistance by a factor of 1.25-3 (Jabarov et al. 1980, Sheppard and Tercelj 1980). The improvement in strength depends on the strengthening layer thickness, the cement mortar strength, the reinforcement quantity and the means of its bonding with the retrofitted wall, and the degree of masonry damage.

3.4 Shotcrete

Shotcrete overlays are sprayed onto the surface of a masonry wall over a mesh of reinforcing bars (Fig. 7). Shotcrete is more convenient and less costly than castin- situ jackets. The thickness of the shotcrete can be adapted to the seismic demand. In general, the overlay thickness is at least 60 mm (Abrams and Lynch 2001, Tomazevic 1999, Karantoni and Faradis 1992, Kahn 1984, Hutchison et al. 1984). The shotcrete overlay is typically reinforced with a welded wire fabric at about the minimum steel ratio for crack control (Karantoni and Fardis 1992). In order to transfer the shear stress across shotcrete-masonry interface, shear dowels (6-13 mm diameter @ 25-120 mm) are fixed using epoxy or cement grout into holes drilled into the masonry wall (Abrams and Lynch 2001, Tomazevic 1999, Karantoni and Faradis 1992, Kahn 1984). Other engineers believe that a bonding agent like epoxy is required to be painted or sprayed on the brick so that adequate brick-shotcrete bond is developed (Kahn 1984). However, there is no consensus on brick-to-shotcrete bonding and the need for dowels.

Diagonal tension tests of single and double wythe URM panels (Kahn 1984) retrofitted with shotcrete showed that, dowels did not improve the composite panels response or the brick-shotcrete bonding; header bricks satisfactory joined the wythe of existing masonry panels. In addition, Tomazevic (1999) and Kahn (1984) recommended wetting the masonry surface prior to applying shotcrete. Kahn (1984) shows that such brick surface treatment does not affect significantly the cracking or ultimate load, it affects to limited extend the inelastic deformations. Retrofitting using shotcrete significantly increases the ultimate load of the retrofitted walls. Using a one-sided 90 mm thick shotcrete overlay and in diagonal tension test, Kahn (1984) increased the ultimate load of URM panels by a factor of 6-25. Abrams and Lynch (2001), in a static cyclic test, increased the ultimate load of the retrofitted specimen by a factor of 3. This retrofitting technique dissipates high-energy due to successive elongation and yield of reinforcement in tension (Fig. 8). Although in diagonal tension test (Kahn 1984) the improvement in the cracking load was very high, in static cyclic test (Abrams and Lynch 2001) the increment in the cracking load was insignificant. Typically, the shotcrete overlay is assumed to resist all the lateral force applied to a retrofitted wall with the brick masonry being neglected all together (Abrams and Lynch 2001, Hutchison et al. 1984). This is reasonable assumption for strength design since the flexural and shear strength of the reinforced shotcrete overlay can be many times more than that of the URM wall.

Fig. 7

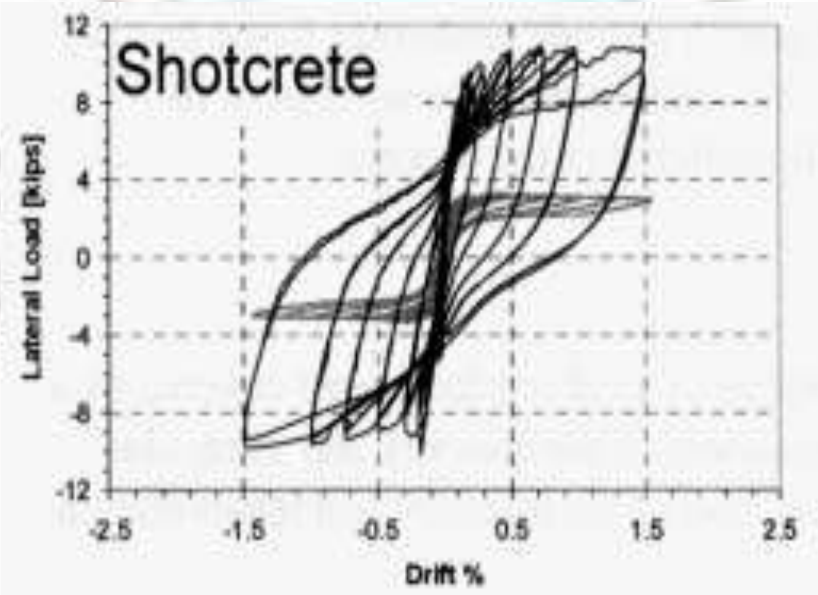

Fig. 8

This assumption may result in some cracking of the masonry as the reinforcement in the shotcrete strains past yield. This may violate a performance objective for immediate occupancy or continued operation.

3.5 Grout and epoxy injection

Grout injection is a popular strengthening technique, as it does not alter the aesthetic and architectural features of the existing buildings. The main purpose of injections is to restore the original integrity of the retrofitted wall and to fill the voids and cracks, which are present in the masonry due to physical and chemical deterioration and/or mechanical actions. For multi wythes masonry walls, injecting grout into empty collar joint enhances composite action between adjacent wythe. The success of a retrofit by injection depends on the injectability of the mix used, and on the injection technique adopted. The injectability of the mix influences by mixes mechanical properties and its physical chemical compatibility with the masonry to be retrofitted. For injection, epoxy resin is used for relatively small cracks (less than 2 mm wide); while, cement-based grout is considered more appropriate for filling of larger cracks, voids, and empty collar joints in multi-wythe masonry walls (Calvi and Magenes 1994, Schuller et al. 1994). However, Schuller et al. (1994) used a cement-based grout (100% type III Portland cement ASTM C150 with expansive admixture and w/c ratio of (0.75) to inject 0.08 mm wide cracks.

Conclusions

In this paper, two effective and economical FRP rehabilitation schemes have been developed for existing nonseismically detailed interior RC beam-wide column joints. Particular focus has been given to evaluate the effectiveness of two rehabilitation schemes on two series of beam-wide column joints. Based on the observations and the experimental results of this study, the following conclusions can be made:

• The axial compression loading did not significantly affect the lateral-resisting capacity and energy-dissipation capacity for the control specimens. The effect of axial-compression loading on the performance of the repaired specimens cannot be explicitly concluded because of the different repair techniques adopted and the axial-compression loading schemes utilized in each series of specimens.

• Fiber anchors can effectively prevent premature delamination of FRP sheets, although debonding of FRP L-wrap near the beam-column interface was observed in all of the repaired specimens at the final DR. To further improve the efficiency of fiber anchors in preventing premature delamination of the FRP L-wrap, it is recommended that additional fiber anchors be utilized in the L-wrap at the beam region, and at least one of the anchors be placed as close as possible to the beam column interface.

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