CYCLIC BEHAVIOR OF EXTERIOR BEAM COLUMN JOINT SPECIMENS UNDER QUASI-STATIC TESTING

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ABSTRACT

Cyclic behaviour of two reinforced concrete exterior beam-column joints has been carried out under quasi-static testing with the objectives to investigate the hysteresis behavior of the beam-column connection, its failure mechanism, and evaluation of strength and ductility. The specimens have been fabricated in T shape and details are as follows (a) without confining the joint reinforcement and b) confining the joint reinforcement i.e. as per Indian Standard Code of practice on seismic design detailing. The size of the beam (275mm x 275mm) has been kept smaller than the column size (300x 300mm) but the main reinforcement (4 @ 20mm) is the same in both the components. The amount of transverse reinforcement in unconfined specimen is 10mm Ø @ 180mm in beam and 10mm Ø 300mm in column. In the confined specimen transverse reinforcement is 10mm Ø @ 100mm in beam and 10mm Ø @ 150mm in column excluding the joint region where confining reinforcement is 10mm Ø @ 100 mm. It has been observed that the strength in both the specimens is nearly the same while ductility of the confined joint is more as compared to unconfined specimen. The energy dissipation is more pronounced and also stable in confined specimen. It has also been observed that bond failure is leading to failure in attaining design beam strength due to insufficient anchorage.

KEYWORDS: R. C., BEAM-COLUMN JOINT, QUASI-STATIC, HYSTERESIS, DUCTILITY

1. INTRODUCTION

Earthquake resistant design of a structure depends on the design strength and ductility of its major structural elements like columns, beams and beam-column joints. The performance of all these members up to the anticipated level is doubtful if there is lack of understanding between practical behavior and analytical design. Structures that are even designed and detailed for earthquake forces using modern codes turns out to be futile due to misconception still prevailing in the detailing issues and understanding the behaviour of structural members in realistic manner. The experimental testing results of exterior beam column joints by the current design procedures could some times result severe damage to the joint, despite the use of strong column-weak beam philosophy (Tsonos. 2006). From the visual examination of the devastated performances of building stock in the past earthquakes, no where it can been seen that a rebar yielded until fracture in any of the locations either it may be a plastic hinge location at the base of any column, or at the top of column, or at the ends of any beam, or in the beam column joint region. Most failures of R.C. framed structures look like a pack of cards with the columns thrown away intact representing pullout of Rebars from the joint region. Hence there is a need to stress and address the requirements of attaining bond in the beam column joint regions and also its confinement affect to improve its performance and making it more ductile. Within the two types of joint configurations that occur in a structure i.e. 1) Interior beam-column joints and 2) Exterior beam column joints, more emphasis is to be laid on the behavior of the latter than the former due to the absence of space to anchor the rebar in the opposite face of the beam. In this case the possibility is to anchor the beam rebar with an L-bend at the end into the column. In the present study experimental behavior of exterior beam column joint with the provision of full development length Ld + 10 d_b in the form of L-bend from the inner face of column under cyclic loading has been presented. Two beam column joint subassemblages; one with unconfined beam column joint region and the other with confined beam column joint region have been tested under cyclic loading at quasi-static testing facility in the Department of Earthquake Engineering, IIT Roorkee, India. The results indicate that the current guidelines regarding detailing of exterior beam column joints specified in IS 13920: 1993 needs improvement

2 DETAILS OF BEAM COLUMN JOINT SPECIMENS

Two exterior beam column joint specimens were designed and constructed with column and beam sizes 300mm × 300mm, 275mm × 275mm respectively. Both the columns and beams are reinforced with 4 No. 20 mm diameter rebar of S50 grade TMT as main reinforcement with yield Stress of 500Mpa and 10mm diameter rebar of S50 Grade TMT as shear reinforcement The sections of the beam and column are as shown in figure.1. M20 grade of concrete has been used in making of specimens

Figure 1.0 Cross Sectional Details of Beam and Column

Two types of reinforcement detailing i.e. 1) Unconfined beam-column joint, where the shear reinforcement is provided according to IS 456: 2000 Code of practice representing gravity load design and 2) Specially confined beam-column joint, where the shear reinforcement is provided as per IS 13920: 1993 Code of practice representing earthquake resistant design are considered in this study. The reinforcement detailing of the unconfined and confined specimens with detailed specimens in the inset of the figures are shown in figure. 2.0 (a), (b), respectively. The required development length $L_d + 10d_b$ where d_b is the bar diameter as per IS13920 has been provided from the inner face of column through joint using L-bend in both the specimens

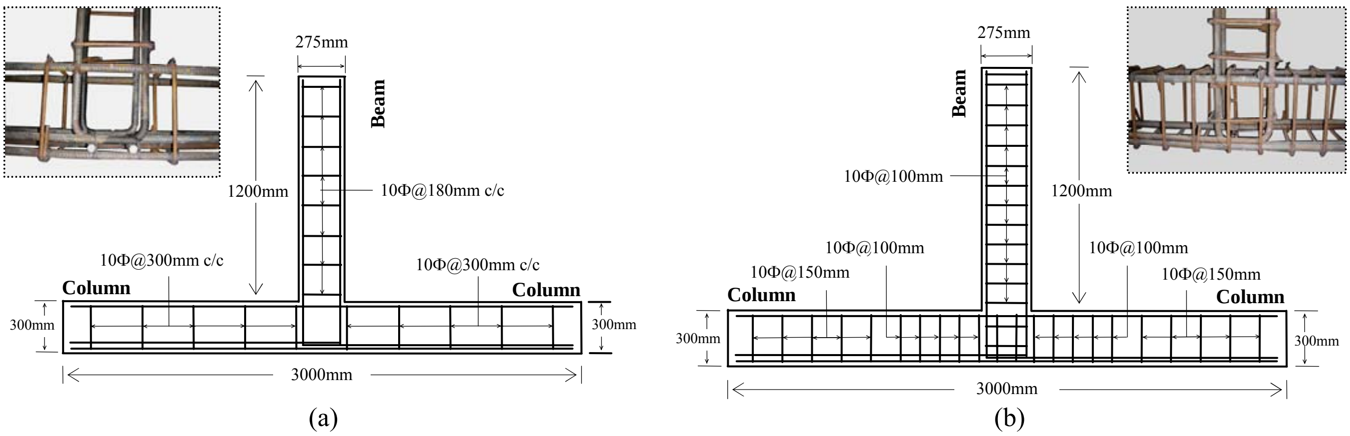


Figure 2.0 Reinforcement detailing of (a)Unconfined and (b) Specially Confined Beam-Column Joint

2.1 Detailing Requirements As per IS 13920 Code

According to IS 13920: 1993, "Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces- Code of Practice", anchorage of beam bars in an external joint is given in clause 6.2.5 and is as presented below, and details is shown in figure 3.0

"Clause 6.2.5: *In an external joint, both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90 degree bend(s)"*

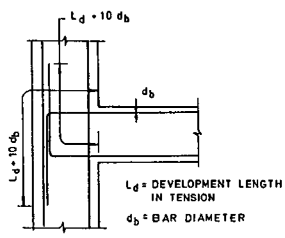


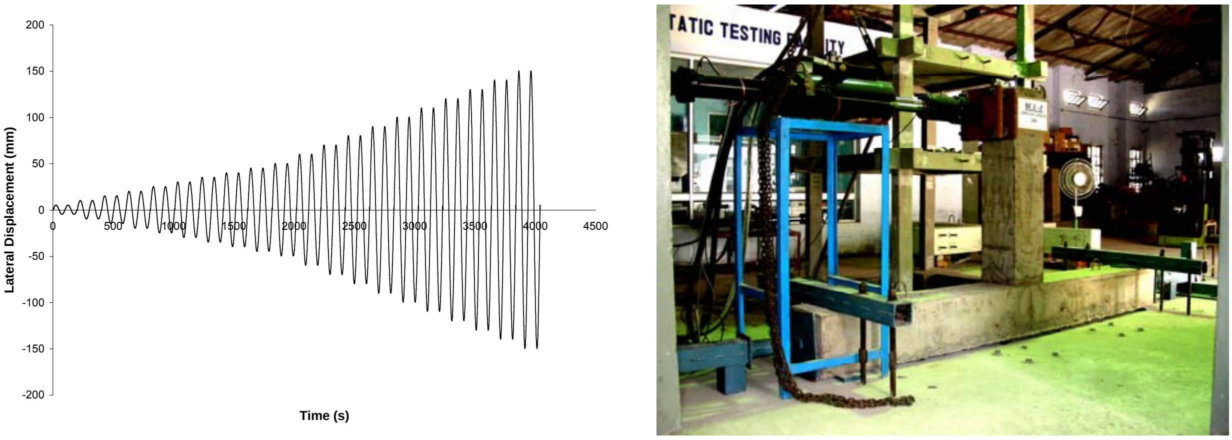
Figure 3.0 Anchorage of beam bars in an external joint (IS 13920:1993)

3 QUASI-STATIC TESTING FACILITY AND TESTING SEQUENCE:

The experimental setup comprises 1) strong-floor of the testing platform where the models can be tied firmly to the ground so as to resist overturning of the specimen in the absence of acting gravity load on the Beam column joint assemblage, 2) a reaction wall to apply lateral load, and 3) computer controlled servo hydraulic actuator of capacity 100 kN, with maximum permissible stroke of 300mm 4) two mechanical jacks which are used to restrain the column member on both ends restraining its translation in both horizontal and vertical directions, 5) Fastenings like nut , bolts, box sections, and two base plates as supports etc. 6) a camera with interval time shooting has been used to capture the failure pattern at various instance of loading, which are used in this testing. The loading data from the load cell inbuilt at the front of the actuator and displacement data from the inbuilt LVDT placed at the back is acquired during the test. The loading is applied under displacement controlled with sine waves of amplitude at the free end of the beam are given in the table 3.1 and load history shown in figure 4.0(a). Two loading cycles are performed at each displacement cycle. For convenience of applying loading and testing, the T -shaped beam- column joint specimen is rotated 90 degrees, so that the column member is in the horizontal position and the beam member is in the vertical position as shown in figure 4(b), and a schematic diagram of the test setup is given in figure 5.0.

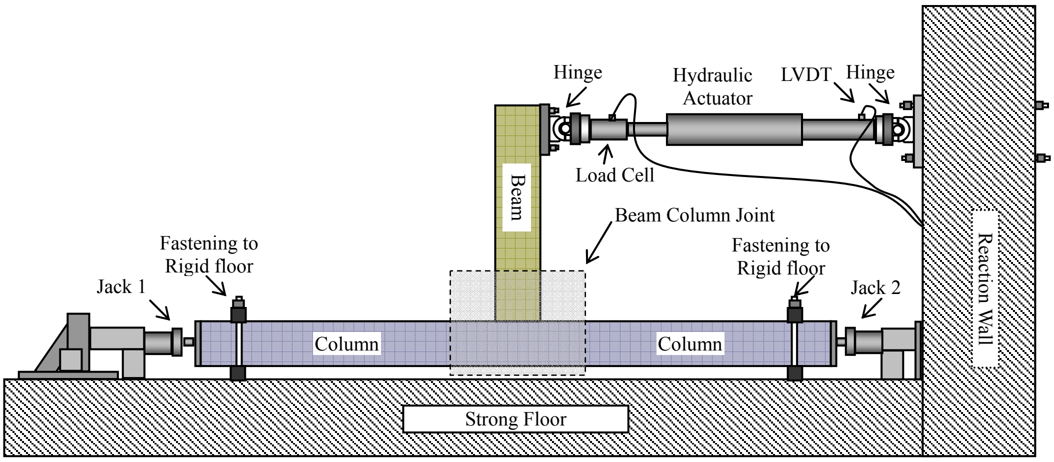
Table 3.1 Loading sequence under displacement controlled Quasi-static testing of beam-column joint

Loading Cycle	1	2	3	4	5	6	7	8	9	10	11	12	15	14	15	16	17	18	19	20
Amplitude (mm)	5	10	15	20	25	30	35	40	45	50	60	70	80	90	100	110	120	130	140	150



(a) (b)

Figure 4.0 (a) Lateral displacement history, (b) Beam-Column joint under Quasi-Static Test Set-up,



Note: Not to Scale

Figure 5.0 Schematic Quasi-Static Test Setup of Beam Column Joint

4 EXPERIMENTAL RESULTS:

4.1 Unconfined Beam Column Joint

Initial specimen and first flexural cracks have been seen in the beam during 10mm cycle as shown in figure 6 (a), (b) respectively, their depth increases further diagonally with appearance of first shear crack diagonally at the joint region in 15mm cycle as shown in figure 6 (c).During 20mm Cycle flexural cracks in beam on the opposite face along with diagonal joint shear cracks have appeared representing starting of strength degradation of joint figure 6(d). In 25mm cycle more widening of shear cracks in the joint has been observed denoting a shear failure of beam column joint figure 6 (e). The horizontal crack at base is opening wider in 30mm cycle representing loss of bond of anchored beam rebar in column as in figure 6(f). Formations of secondary diagonal shear cracks at the joint have been seen during 35mm cycle, catering the action of diagonal compression strut of concrete in the process swelling of concrete cover also seen. Failure of the strut action at side face of column results in swelling in 40mm cycle as shown in figure 6(g) and total side face cover of column swells more in 45mm cycle and finally spalls in 50mm cycle and rebars exposed as shown in figure 6(h), from 60mm to 150 mm cycle degradation of concrete surrounding rebar due to abrasion is observed. The corners at the base of the beam get crushed and beam bends to take the displacement excursion without any increment of load, instances of 60mm, 90mm, 120mm and 150mm are shown in figures 6(i), to (l) respectively.

(a) Initial (b) 10mm cycle (c) 15mm cycle (d) 20mm cycle

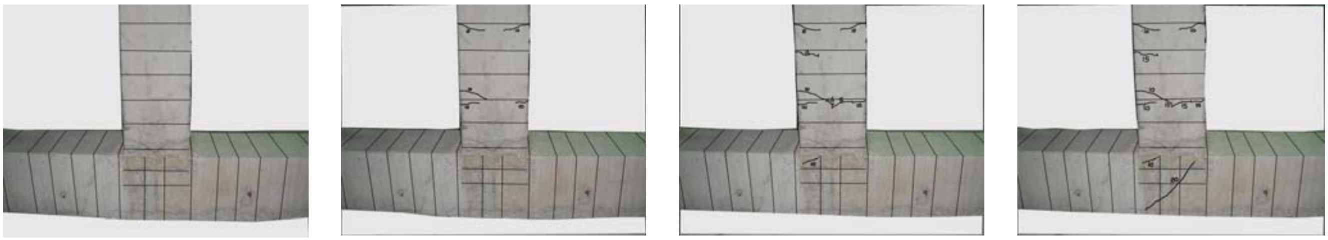
(e) 25mm cycle (f) 30mm cycle (g) 40mm cycle (h) 50mm cycle

(I) 60mm cycle (J) 90mm cycle (k) 120mm cycle (l) 150mm cycle

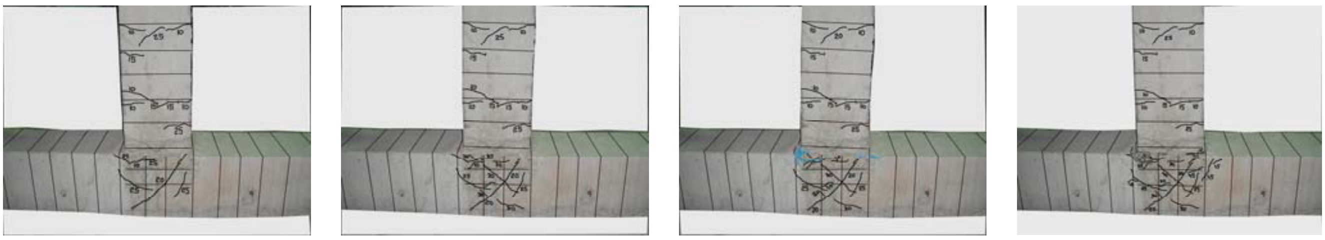
Figure 6.0 Sequential failure of Unconfined Beam-Column joint along with cracking at various loading cycles

4.2 Confined Beam Column Joint

No cracks are seen in 5mm cycles shown in figure 7(a). Initial flexural cracks are seen in the beam during 10mm cycle on both the faces of beam as shown in figure 7 (b), and their depth increased further in 15mm cycle with small shear crack at the joint region as shown in figure 7 (c).During 20mm cycle major diagonal shear crack along with widening of flexural cracks in beam appears representing starting of shear failure of joint as shown in figure 7(d). In 25mm cycle more shear cracks in the opposite direction have been observed in beam column joint as shown in figure 7 (e). The horizontal crack at base is opening wider along with opening of shear cracks in 30mm cycle representing loss of bond of anchored beam rebars in column as in figure 7(f). Formations of secondary diagonal shear cracks at the joint are seen during 35mm cycle, catering the action of diagonal compression strut of concrete, in the process, swelling of concrete cover has also been seen. Crack at the base of beam is widening along with the diagonal shear cracks at joint without any widening of flexural crack in the beam in 40mm cycle as shown in figure 7(g) and total side face cover of column swells more and compression of concrete at corners of beam started in 45mm cycle. More swelling of side face cover concrete of column and clear separation of joint region into wedges as shown in figure 7(h),.from 60mm cycle onwards the shear strength degradation is due to the abrasion of rebar with surrounding concrete with still action of the compressive strut of confined core concrete of joint region. The corners at the base of the beam is getting crushed in every cycle and the beam bends to take the displacement excursion without retaining previous cycle load and is shown in figures 7(i) to (k),.finally cover concrete spalls in 120mm cycle.



(a) Initial (b) 10mm cycle (c) 15mm cycle (d) 20mm cycle



(e) 25mm cycle (f) 30mm cycle (g) 40mm cycle (h) 50mm cycle

(I) 60mm cycle (J) 90mm cycle (k) 120mm cycle (l) 150mm cycle

Figure 7.0 Sequential failure of Confined Beam-Column joint along with cracking at various loading cycles

4.3 Hysteresis Behaviour of Beam Column Joints

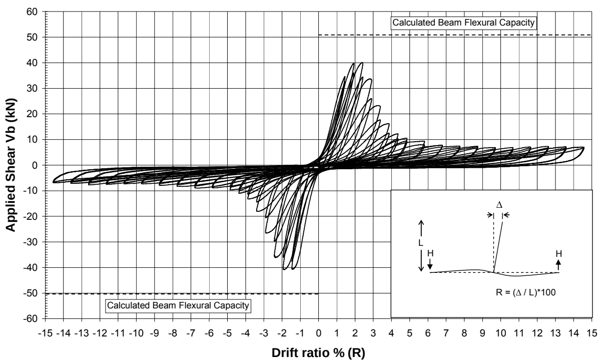
The behaviour of the two beam column joint subassemblage specimens is presented and discussed in terms of applied shear (V_b) versus drift ratio (%). Drift ratio (R) plotted in the above figures, is defined as the beam tip displacement Δ divided by the beam span L, and is expressed as a percentage and can be seen in the inset of the figures 8(a), (b). Hysteresis plots of applied shear versus drift ratio % plots for both the assemblages; Unconfined and Confined are shown in figure 8(a) and 8(b) respectively

4.3.1 Member ductility

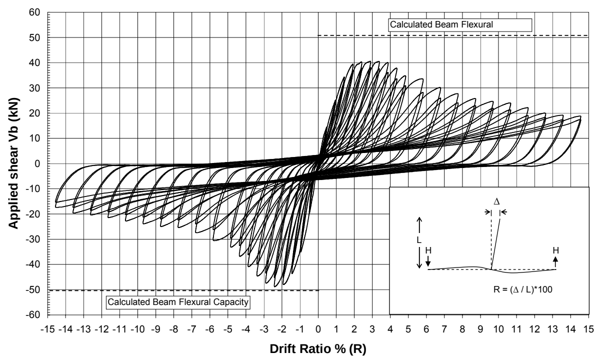
Assuming beam column joint assemblage as a member, the ability to deform beyond yield is obtained in the form of displacement ductility given as the ratio of the maximum displacement to the yield displacement from the load deformation envelop plotted from the points of the maximum load and corresponding displacement of all cycles of excursion. With the location of yield displacement on the experimental envelop the various levels of ductility have been specified on envelop curve to evaluate their performance at the instance of each ductility level as the failure is premature shear failure in both the cases . The envelop curves of both unconfined and confined beam-column joint specimens is shown in figure 9.0.

4.3.2 Performance of exterior beam column joints at various ductility levels

Performance of both the unconfined and confined specimens' up to the yield (ductility level-1) i.e. upto the drift ratio of 1.94% is more or less same even with different shear force detailing. From this point onwards the confined specimen showed stable behaviour with the onset of yielding upto around ductility level-2 i.e. drift ratio3.88%, where as the unconfined specimen shows a steep degradation of shear strength i.e. 64% of its yield strength where as the former degraded only upto 7.5% of its yield, representing a stable behaviour. At ductility level-3, -4, -5,-6, and -7 the strength degradation of unconfined and confined joints are (78%, 27%), (81%, 34%), (83%, 44%), (83%, 50%), (83% 55%) respectively of their yield strengths as shown in figures 8(a), (b). The details of cyclic loading at various levels of ductility has been given in table 4.1



(a)



(b)

Figure 8.0 Hysteresis behavior of (a)Unconfined (b) Confined beam- column joint

Table 4.1 cyclic loading behaviour of BCJ specimens at various levels of ductility

	Ductility Level		μ=1	μ=2	μ=3	μ=4	μ=5	μ=6	μ=7
Applied shear Vb (kN)	BCJ-Unconfined	+ve	39.79	16.17	9.33	7.37	6.84	7.11	7.12
		-ve	-38.37	-11.64	-7.39	-6.84	-6.04	-5.97	-5.46
	BCJ-Confined	+ve	39.08	39.64	33.05	27.28	24.24	21.82	18.81
		-ve	-47.49	-40.42	-30.05	-25.01	-23.58	-21.16	-19.34

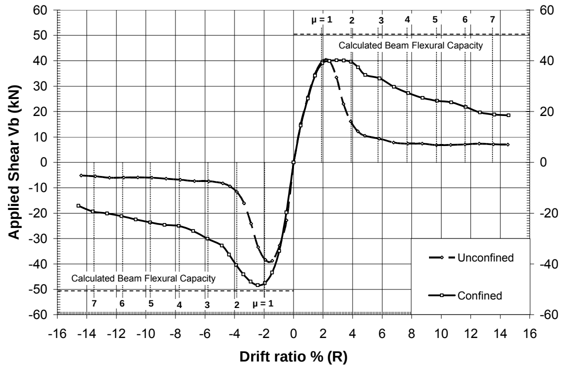


Figure 9.0 Envelop curves of unconfined and confined beam column joints

4.4 Comparative Performance of Beam Column Joints

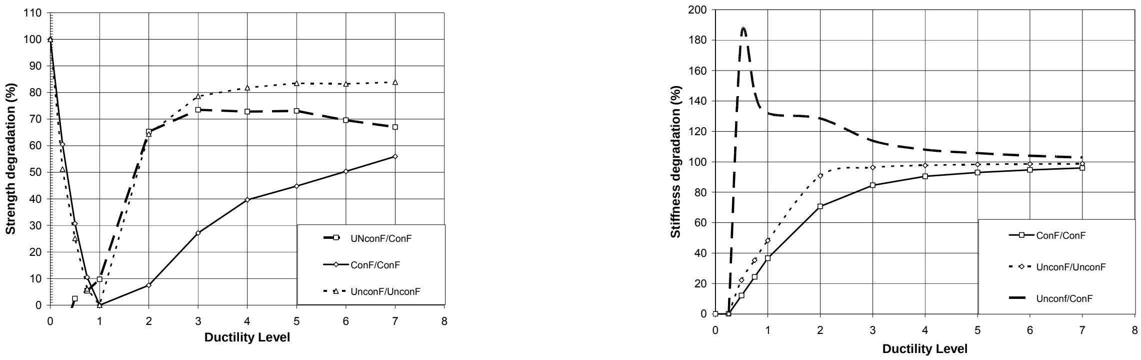
The observed variation in behaviour of both the exterior beam column joints shows the significance of confinement in the region of plastic hinge locations, majorly the beam column joint locations. Hence the comparative performance of its strength and stiffness has been carried out to quantify the amount of damage that can be reduced by the practice of ductile detailing for earthquake safety of structures in these locations. In this process the average envelop of both the positive and negative envelop of individual specimens of a selected parameter is taken and compared as 1) unconfined behaviour over its yield represented as *"UnconF/UnconF"*, 2) confined behaviour over its yield represented as *ConF/ConF* 3) Unconfined behaviour over Confined behavior represented as *UnconF/ConF* at various ductility levels in percentage as shown in figure 10 (a), (b)

4.4.1 Strength degradation

Under comparative performance with respect to strength degradation shown in figure 10(a), the negative slope represents that there is no degradation upto yield level-1, in both the *unconf/unconf* and *conf/conf* curves. But from level-1 to level-2 in the *unconf/unconf* and *unconf/conf* curves there is a very steep hike in the slope representing high degradation on the onset of yield, i.e. representing shear failure of joint where as in case of *conf/conf* it is very mild representing less degradation, similarly the graph can be studied in the other levels of ductility also. In case of *Unconf/Conf* there is only positive slope from the initial point indicating the behavioral difference of unconfined over the confined joint specimen at various ductility levels.

4.4.2 Stiffness degradation

As shown in figure 10(b) the stiffness degradation under cyclic loading of unconfined i.e. "*Unconf/Unconf*" and confined joint i.e. "*Conf/Conf*" over their initial stiffness, is 48% and 36% at ductility level-1, similarly at ductility level-2 they are 90%and, 70%, at ductility level-3, 96%,and 84% and at ductility level four 97% and 90% respectively. The comparative performance of unconfined over confined i.e. "*Unconf/Conf*" is 131% at ductility level-1, similarly for ductility level 2, 3and 4 it is 128%, 113% and 107% respectively Hence the drastic variation of stiffness degradation from confined beam column joint to unconfined makes the confined joint a stable hinge dissipating energy.



(a) (b)

Figure 10.0 (a)Strength degradation (b) Stiffness degradation of Beam column joints under cyclic loading

5 CONCLUSIONS

Two exterior beam-column joint subassemblages designed and detailed according to 1) IS 456: 2000 and 2) IS 1893(part1): 2002 and IS 13920: 1993, have tested under quasi-static cyclic loading upto failure. Based on the test results the following conclusions are drawn

1 The performance of unconfined exterior beam-column joint behave as expected with premature shear failure at the joint region with out reaching its flexural capacity to push the formation of plastic hinge in beam

2 Despite the fact that the exterior beam-column joint which is specially confined at the joint region does not performed as expected/codified, premature shear failure at the joint region as in unconfined specimen was observed, hence no plastic hinge formation in adjacent beam in this case also, but joint confinement with the L bend anchorage of rebar in column improved core concrete's ability to transfer shear through the diagonal concrete strut action not leading to drastic strength degradation as evident from the hysteresis curves

3 This study stressed the need to improve IS 1893:2002 and IS 13920:1993 codes requirements towards the achievement of global ductility of moment resisting frames incorporating strong column-weak beam actions not only in design but also in its detailing with more emphasis on beam-column joints with a clear perspective of factors influencing its behaviour, as the present design and detailing clauses are lacking expected design performances.

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