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SEISMIC BEHAVIOR OF EXTERIOR BEAM-COLUMN JOINT USING MECHANICAL ANCHORAGE UNDER REVERSAL LOADING: AN EXPERIMENTAL STUDY*

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Abstract– In reinforced concrete structures, beam-column joints are one of the most critical regions in the areas with moderate and severe seismic prone areas. Proper reinforcement anchorage is essential to enhance the performance of beam-column joints. Congestion of reinforcement and construction difficulty is one of the critical problems while using conventional reinforcement detailing in beam-column joints of concrete structures. An effort has been made to study and evaluate the performance of beam-column joints. The joints are detailed for higher seismic prone areas as per ACI-352 (Mechanical Anchorage), ACI-318 (Conventional Hooks Bent) and IS-456 (Full Anchorage Hooks Bent) along with confinement as per IS-13920 and proposed X-cross plus hair clip bar joint reinforcement. Apart from finding the solution to these problems, significant improvements in seismic performance, ductility and strength were observed while using mechanical anchorage in combination with X-cross plus hair clip bars. To assess the performances of anchorages and joint details, the specimens were assembled into two groups of three specimens each. The specimens were tested under reversal loading and test results were evaluated and presented in this paper.

Keywords– Reinforced concrete structure, beam-column connection, mechanical anchorage, reversal loading

1. INTRODUCTION

Beam-column connections are critical regions in reinforced concrete framed structures in seismic prone area. Proper anchorage of reinforcement is essential to enhance the performance. Innovative joint designs that can reduce congestion of reinforcement without compromising strength, stability, stiffness is desirable. ACI-352 [1] recommends research on use of T-headed bar in the design of beam-column connections in concrete structure. The investigation of the beam-column connection using longitudinal beam reinforcement bar with 90^0 standard bent hooks anchorage and mechanical anchor for joint core under reversal loadings has been a research area for many years. Some of the analytical studies and experimental studies carried out in this area so far are indicated below.

Park and Paulay [2] recommended the detailing of joints for the earthquake resistant structures using bent-up bars, stub-beam with bent-up bars and mechanical anchorage for serving as anchorage as well as effective ties for confinement in the joint core of the exterior beam-column joints.

Tsonos et al. [3] suggested that the use of crossed inclined bars in the joint region was one of the most effective ways to improve the seismic resistance of exterior beam-column joints.

Wallance et al. [4] suggested that use of headed reinforcement had eased specimen fabrication, pouring of concrete and improved the behavior equal to that of specimens with standard 90^0 hooks for beam-column corner joint.

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Murty et al. [5] reported that the standard hooks for anchorage of the longitudinal beam bar with hair clip-type transverse joint reinforcement as per ACI were more effective and such combination of anchorage with joint reinforcement is easy to construct and can be used in locations demanding ductility moderately.

Uma et al. [6] in their review of codes of practices considered ACI-318, NZS-3101: Part-1 and Eurocode-8 EN-1998-1 regarding the design and detailing aspects of interior and exterior beam-column joint.

Chutarat et al. [7] reported that the use of straight-headed bars in the exterior beam-column joints was very effective in relocating potential plastic regions.

Lee et al. [8] proposed extension of ACI design methods to cover the use of mechanical anchorage for eccentric beam-column joints. They also reported that cyclic behavior of exterior beam- column joints can be significantly improved by attaching double mechanical device on each beam bar within the joint.

Bindhu et al. [9] in their experimental investigations, validated with analytical studies and concluded that additional cross bracing reinforcement improves the seismic performance of the exterior reinforced concrete beam-column joints.

Sagbas et al. [10] in their FEA Computational analysis compared the experimental test results of seismically and non-seismically designed joint detailing for the shear deformations.

Baglin and Scott [11] compared the experimental test results with nonlinear behavior finite element analysis. Parametric study was conducted to evaluate the effect of the main variable of both joint capacity and joint behavior.

Hegger et al's [12] Non-linear finite element analysis exterior beam-column joint behavior has been calibrated using the third author's tests.

Park and Mosalam [13] presented an analytical model to predict the shear strength of reinforced concrete exterior beam-column joints without transverse reinforcement.

The use of headed bars has become increasingly popular for relatively large reinforced concrete (RC) structures that are exposed to extreme loads such as strong earthquakes or blasts, often providing an adequate solution to steel congestion (Chun et al., [14]; Kang et al., [15-17]).

It is noted that the anchorage requirements for the beam longitudinal reinforcement and the joint confinement are the main issues related to problems of congestion of reinforcement in the beam-column connections. An attempt has been made to evaluate the performance of the exterior beam- column joint by replacing the 90^0 standard bent bar anchorages by T-type mechanical anchorage and additional X-cross bar with U-bar in the beam- column joint core for the moderate and severe seismic prone zones. Zones are followed as per IS-1893[18] and IS-13920[19]. It is found that these combinations were effective in reducing the congestion of reinforcement in joint core and eased pouring of concrete without compromising the strength, ductility and stiffness of beam-column joints under reversal loading.

2. RESEARCH SIGNIFICANCE

The experimental study has been carried out for different types of anchorages and joint details in the exterior beam-column joint. The T-type mechanical anchorage (headed bar) in combination with additional X-cross bar with hair clip (U-bars) as joint detail is found to be marginally higher lateral load carrying capacity. This type of anchorage and the joint core detail improves the ductility, without compromising the strength and also reduces congestion of reinforcement in the joint core, pouring of concrete and makes fabrication easier at site.

3. TESTING PROGRAM

The test involves six numbers of specimens simulating the exterior beam-column joint for the experimental program. The specimens have been divided into two groups, each group having three specimens, with different anchorages. The anchorage details are designated as A, B and C and joint details are designated as 1 and 2. Anchorage detail-A is T-type headed bar followed as per ACI-352 [1]. Anchorage detail-B is a standard conventional 90^0 bent hook followed as per ACI-318 [20] and anchorage detail-C is full anchorage followed as per IS-456 [21]. Joint detail-1 has the proposed additional X-type cross bar with hair clip (U-bar) reinforcement and joint detail-2 has standard conventional shear ties arrangement in the joint core.

4. TEST SPECIMENS

a) Details of test specimens

All the six test specimens of beam-column assemblage are identical in size. The size of the beam is 200mmX300mm (width by depth). The column cross-section is 300mmX200mm as shown in Fig.2. The length of the beam is 1200mm from the column face and the height of the column is 1500mm. The various types of anchorages of reinforcement at joint used are shown in Figs. 1a, 1b and 1c and the joint details are shown in Fig.3. In Group-I, the anchorages A, B and C are combined with joint detail-1. These specimens are named as A1, B1and C1. In Group-II, the anchorages A, B and C are combined with joint detail-2. These specimens are named A2, B2 and C2.

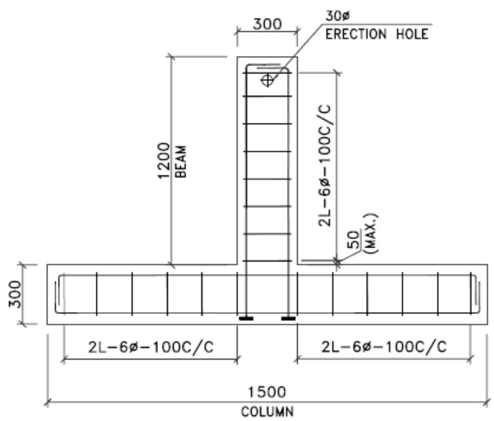

Fig. 1a. Specimen Type-A(ACI-352)

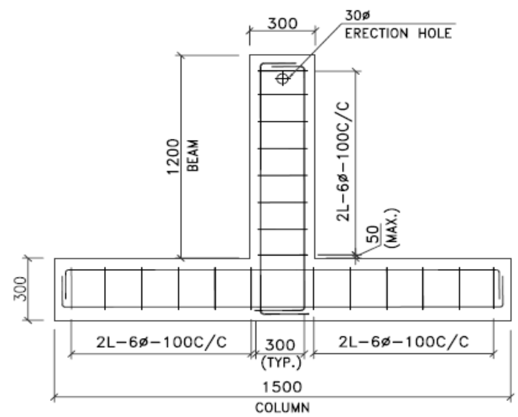

Fig. 1b. Specimen Type-B(ACI-318)

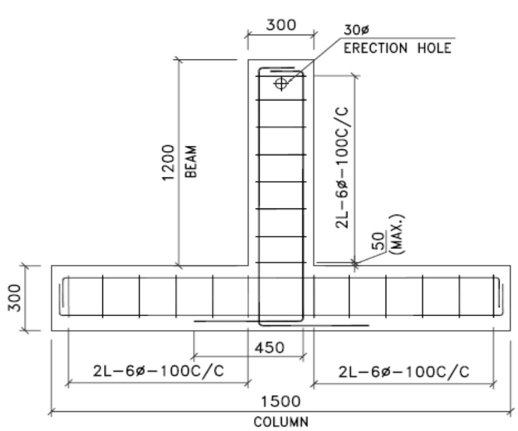

Fig. 1c. Specimen Type-C(IS-456)

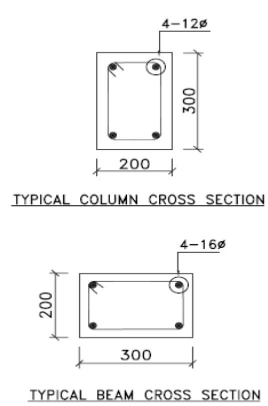

Fig. 2. Beam and Column size

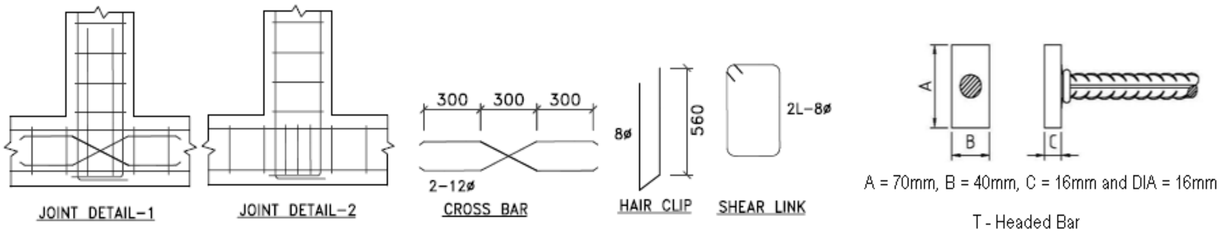

Fig. 3. Joint core detail arrangement

b) Materials used

Concrete mix was made with cement (43 N/mm^2) with river sand and 20mm downgraded coarse aggregate. The quantities of materials per cubic meter of concrete used were: Cement=435.45 Kg/m^3, Fine aggregate= 626.673 Kg/m^3, Coarse aggregate = 1188.22 Kg/m^3, Water = 191.6 Kg/m^3, Water/Cement ratio=0.45, the 28-day average cube compressive strength was 28.30Mpa. The reinforcement bars used were 6,8,12 and 16mm diameter of grade Fe-415 and the grade of welded T-type headed bar (mechanical anchorage) used was E410 as shown in Fig. 3.

5. JOINT BEHAVIOR AND JOINT MECHANISM DETAILS

A particularly severe ground shake situation can arise in certain beam-column joints of plane multistory frames when these are subjected to high seismic loading. The external action and the corresponding internal forces generated around such a joint are indicated in Fig. 4.

The following notations refer to the stress resultants.

T-Tensile force in the reinforcement, C_c - compressive force in the concrete, C_s-compressive force in reinforcement and V-shear force, subscript '*b*' for beam and '*c*' for column. From the position of the stress resultants it is apparent that diagonal tensile and compressive stress (f_t and f_c) are induced in the shear panel zone [2] of the joint.

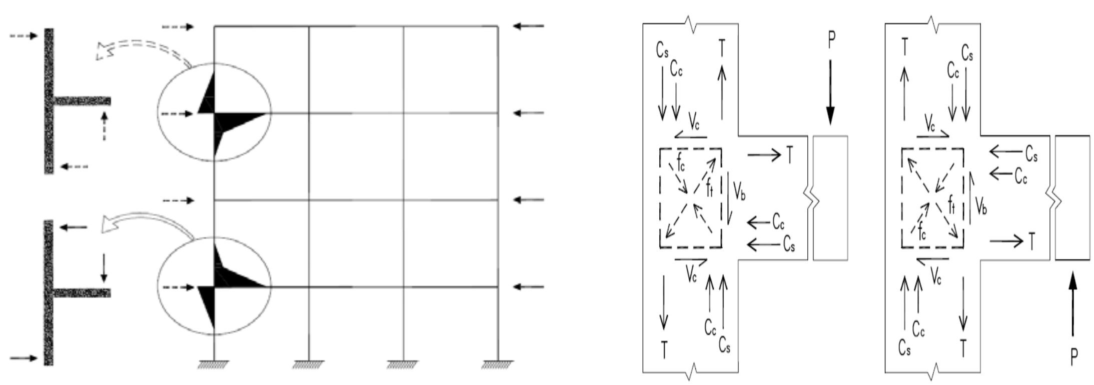

Fig. 4. Test specimen definition with forces acting on a joint core

6. BEAM-COLUMN JOINT CORE REINFORCEMENT

a) Joint core reinforcement anchorage

The ACI-352[1] report specifies that for beams with Type-2 connections, the critical section for development length of reinforcement, either hooked or headed should be taken at the outside edge of the column core. The development length *(L_{dh})* measured from the critical section should be computed as follows.

$$L_{dh} = \frac{\alpha * f_y d_b}{6.2\sqrt{f_c^{'}}} \tag{1}$$

Where,

L_{dh}= Development length for a hooked bar, measured from the critical section to the outside edge of the hook extension *(L_{dh} =267.75mm <272mm provided),* α = Stress multiplier for longitudinal reinforcement at joint-member interface for Type-2, $\alpha \geq 1.25$, f_y = Specified yield stress of reinforcement (415N/mm^2), d_b = Nominal diameter of bar, $f_{c'}$ = Compressive strength of concrete.

The development length L_{dt} of a headed bar should be taken as 3/4 of the value computed for hooked bars using the above equation of $L_{dh.}$

$$L_{dt} = \frac{3}{4} * \left(L_{dh}\right) \tag{2}$$

Where,

L_{dt} =development length for a headed bar measured from the critical section to the outside end of the head. In headed bar, the bar head should be located in the confined core within 2 in. (50 mm) from the back of the confined core. The minimum development length L_{dt} should not be less than 8 d_b or 6 in. (150 mm), for Type-1 and Type-2 connections (This provision defines Type-1 connection as frame members that are designed to satisfy strength requirements without significant inelastic deformation and Type-2 connection as frame members that are designed to have sustained strength under reversals into the inelastic range).

As per IS-456[21] the development length (L_d) of the hooked reinforcement bar should be computed as follows.

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}} \tag{3}$$

Where,

L_d = Development length, (L_d= 644.73mm < 710mm provided), Ø = Nominal diameter of the bar, σ_s = Stress in bar (0.87*f_y) at the section considered at design load, τ_{bd} =Design bond stress of concrete (can be increased by 60% for deformed bars).

b) Transverse reinforcement within the joint core

The ACI-352[1] committee report recommends adequate lateral confinement of the concrete in the joint core for the shear demand in the form of spirals or rectangular hoops for both Type-1 and Type-2 joints. For Type-2 joints, the total cross sectional area of transverse reinforcement within the joint in each direction should be at least equal to A_{sh} but not less than A_{sh} as given in the Eq. (4).

$$A_{sh} = 0.3\frac{S_h b_c^{"} f_c^{'}}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) \geq 0.09\frac{S_h b_c^{"} f_c^{"}}{f_{yh}} \tag{4}$$

The center to center spacing between layers of transverse reinforcement s_h should not exceed 1/4 of the minimum column dimension, six times the diameter of the longitudinal column bars to be restrained, or 150mm.

Where,

A_{sh} = Total cross-sectional area of all legs of hoop reinforcement (301.6mm^2 provided > 228.82mm^2 ≥71.57mm^2), including crossties, crossing a section having core dimension, $b_c^{"}$, S_h = Center-to-center spacing of hoops or hoops plus crossties, $b_c^{"}$ = Core dimension of tied column, outside to outside edge of transverse reinforcement bars perpendicular to the transverse reinforcement area A_{sh} being designed, f_c'=

Compressive strength of concrete, f_{yh}= Yield stress of spiral, hoop, and crosstie reinforcement, A_g = Gross area of column section, A_c =Area of column core measured from outside edge to outside edge of spiral or hoop reinforcement.

As per IS-13920[19] the area of cross section, A_{sh}, of the bar forming rectangular hoop, to be used as special confining reinforcement shall not be less than

$$A_{sh} = 0.18 * S * h \frac{f_{ck}}{f_y} \left(\frac{A_g}{A_k} - 1.0 \right) \tag{5}$$

Where

A_{sh} =Area of the bar cross section in mm^2 (241.30mm^2 < 301.6mm^2 provided), S= pitch of spiral or spacing of hoop, (the spacing of hoops used as special confining reinforcement shall not exceed ¼ of minimum member dimension but need not be less than 75mm nor more than 100mm), h=Longer dimension of the rectangular confining hoop measured to its outer, f_{ck}=Characteristic compressive strength of concrete cube, f_y=Yield stress of steel, A_g=Gross area of the column cross section, A_k=Area of confined concrete core in the rectangular hoop measured to its outside dimensions.

c) Joint shear strength

The ACI-352[1] requirements for joint shear strength are based on

$$\phi V_n = \phi * 0.083 \gamma \sqrt{f_c^{'}} b_j h_c \geq V_u \tag{6}$$

Where

φ = 0.85, V_n=Nominal shear strength of the joint, ($\Phi\ V_n$ = 253.98kN ≥ *128.60kN* = V_u), γ = Shear strength factor reflecting confinement of joint by lateral member (referred from Table-1 ACI-352), $f_c^{'}$=Specified compressive strength of concrete in the connection, b_j=Effective width of the joint transverse to the direction of shear, h_c=Depth of the column.

The horizontal joint shear demand V_u is calculated based on the amount of beam reinforcement as

$$V_u = T - V_{column} = \alpha A_s f_y - V_{column} \tag{7}$$

Where, T =Tension force in the reinforcement, As=Area of tension reinforcement, f_y =Nominal yield stress of the tension reinforcement and V_{column} = Shear in the column. Typically, inflection points are assumed at beam mid span and column middle height to compute the column shear, α = Stress multiplier to account for over-strength and strain hardening of the reinforcement. Values of α =1.00 and 1.25(minimum) are recommended for Type-1 and 2 joints, respectively.

7. EXPERIMENTAL SETUP

The testing of half-scale exterior beam-column joint specimen was carried out at MEPCO Engineering College, Sivakasi, India. The joint assemblage was subjected to reversal loading using hydraulic jack of 25 Ton capacity. The specimen column is kept in horizontal direction and beam is kept vertical as illustrated in Fig. 5b. Both ends of the RCC columns are restrained in vertical and also in both horizontal directions by using strong built up steel boxes which in turn are connected to the reaction floor using anchor bolts. To facilitate the application of reversal load (Left Hand Side-LHS and Right Hand Side-RHS) on either side of the RCC beam, hydraulic jacks are used which are connected to the strong steel frame with mechanical fasteners. The RCC beam was loaded as shown in Fig. 5a. Linear Variable Differential Transducers (LVDT) were placed on either side of the specimen to monitor the displacements. The test is a load controlled with a load increment of 1-ton. The specimen was tested till it reaches its maximum failure capacity.

Fig. 5a. Experimental setup

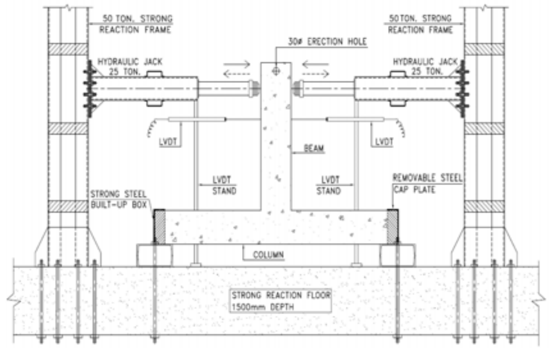

Fig. 5b. Schematic diagram of test setup

8. RESULTS AND OBSERVATION

a) Lateral load versus lateral displacement

The hysteresis loops obtained from the experimental test results of lateral load versus displacement are shown in Fig. 6a, 6b and 6c, and the corresponding peak load versus displacement is shown in Fig. 7. It is observed that in Group-I, the average ultimate load carrying capacity of the specimens A1, B1 and C1 are 89.50kN, 90.00kN and 89.00kN with the corresponding lateral displacement of 47.50mm, 47.50mm and 44.30mm respectively. Among these, B1 exhibits the maximum load carrying capacity which is just marginally higher than A1 by 0.5% and C1 by 1.1%.

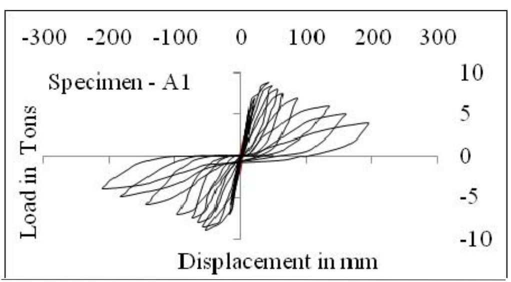

Fig. 6a. Load vs displacement

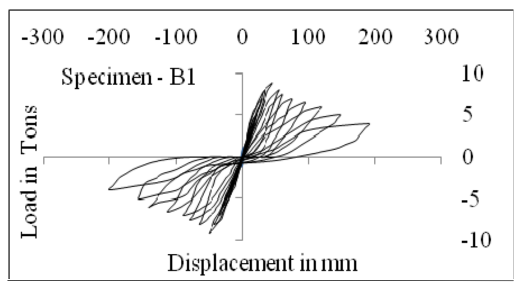

Fig. 6b. Load vs displacement

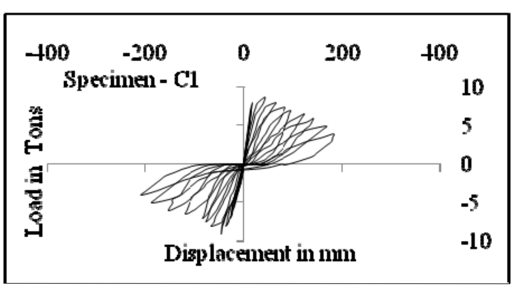

Fig. 6c. Load vs displacement

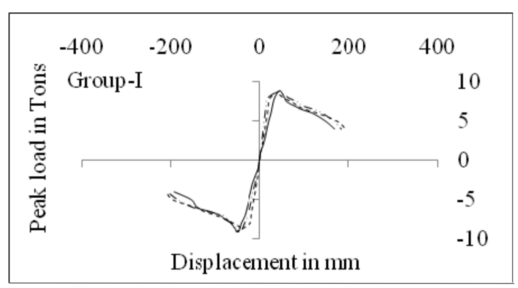

Fig. 7. Peak load vs displacement

The hysteresis loops (of Group-II) obtained from the experimental test results of lateral load versus displacement are shown in Fig. 8a, 8b and 8c, and the corresponding peak load versus displacement is shown in Fig. 9. It is observed that in Group-II, the average ultimate load carrying capacity of the specimens A2, B2 and C2 are 80.50kN, 79.00kN and 79.50kN with the corresponding lateral displacement of 45.37mm, 35.55mm and 48.12mm respectively. Among these A2 exhibits the maximum load carrying capacity which is just marginally higher than B2 by 1.86% and C2 by 1.24%.

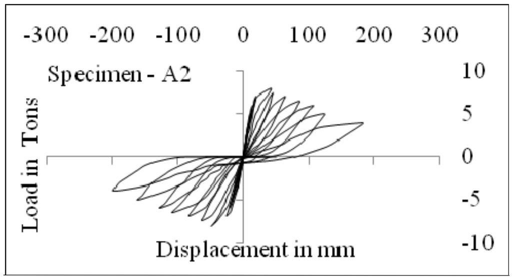

Fig. 8a. Load vs displacement

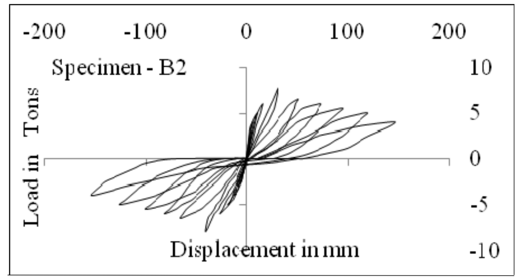

Fig. 8b. Load vs displacement

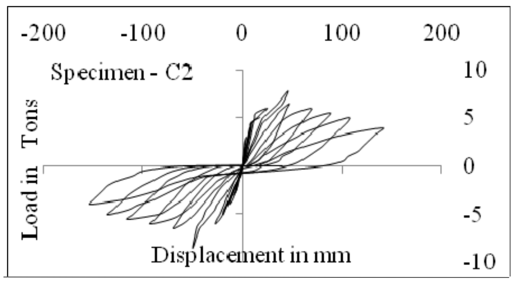

Fig. 8c. Load vs displacement

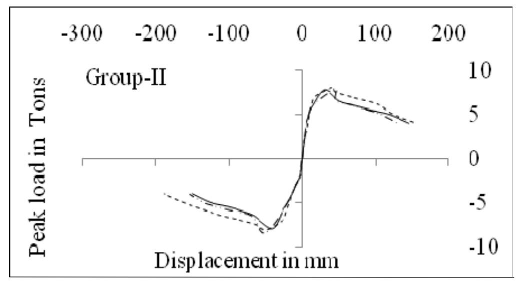

Fig. 9. Peak load vs displacement

Table 1. Lateral strength of test specimens

Specimen Name & Group	Yielding Displacement in mm (δ_y)	Ultimate Load in kN (P_u)		Average Ultimate Load in kN(P_u)	Ultimate Displacement in mm (δ_u)		Average Displacement for Ultimate load in mm (δ_u)
		Left Side	Right Side		Left Side	Right Side	
A1-I	2.15	89.00	90.00	89.50	42.00	53.00	47.500
B1-I	2.40	89.00	91.00	90.00	45.00	50.00	47.500
C1-I	2.20	88.00	90.00	89.00	43.50	45.36	44.430
A2-II	2.30	80.00	81.00	80.50	42.15	48.60	45.375
B2-II	2.85	78.00	80.00	79.00	30.85	40.25	35.550
C2-II	3.00	78.50	80.50	79.50	45.63	50.60	48.115

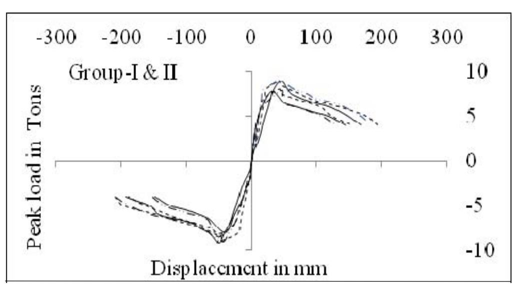

Fig. 10. Peak load vs displacement

The test specimens' ultimate load carrying capacity was assessed from Table-1 and Fig.10. It was observed that the ultimate load carrying capacity of Group-I specimens, namely A1, B1 and C1 exhibit higher load carrying capacity than Group-II specimens, namely A2, B2 and C2 by 10%, 12.2% and 10.67% respectively. From the above test results it can be inferred that the proposed additional X-cross bar with hair clip joint details marginally increases the ultimate strength.

b) Ductility behavior

It is essential that an earthquake resistant structure be capable of deforming in a ductile manner when subjected to several cycles of lateral loads in the inelastic range. Ductility is the property which allows the structure to undergo large deformation beyond the initial yield deformation without losing its strength abruptly. Ductility factor (μ) can be defined as the ratio of ultimate deflection (δ_u) to initial yielding deflection (δ_y). $\mu = (\delta_u/\delta_y)$

From Table 2, it is observed that Group-I specimens namely A1 (mechanical anchorage), B1 (ACI - 318, 90^0 bent hook anchorage) and C1 (IS-456 full anchorage) exhibit higher ductility than Group-II specimens, namely A2, B2 and C2 by 10.70%, 36. 97% and 20.58% respectively, wherein additional X-cross bar with hair clip joint details are used in Group-I and standard conventional shear ties are used as joint confinement in Group-II specimens. Among these six specimens, A1 exhibits better performance. This combination of anchorage and joint core details can be used in moderate and severe ductility demanding situations.

Table 2. Displacement ductility factor of test specimen

Specimen number & groups	Yielding displacement in mm (δ_y)	Ultimate displacement in mm (δ_u)		Ductility Factor for the Ultimate Load $\mu=(\delta_u/\delta_y)$		Average displacement ductility factor (μ)
		Left hand side (δ_u)	Right hand side (δ_u)	Left hand side (LHS)	Right hand side(RHS)	
A1-I	2.15	42.00	53.00	19.535	24.651	22.093
B1-I	2.40	45.00	50.00	18.750	20.833	19.792
C1-I	2.20	43.50	45.36	19.773	20.618	20.195
A2-II	2.30	42.15	48.60	18.326	21.130	19.728
B2-II	2.85	30.85	40.25	10.825	14.123	12.474
C2-II	3.00	45.63	50.60	15.210	16.867	16.038

c) Stiffness behavior

In the case of reinforced concrete beam-column joints, stiffness of the joint gets degraded when the joint is subjected to reversal loading. During the reversal loading, concrete and reinforcement steel bars are subjected to several loading, unloading and reloading cycles. The joints initially develop micro cracks inside, which leads to the lowering of the energy limit of the materials, and thereby results in the increase of deformation inside the joints. This may consequently cause the reduction in the stiffness. Therefore it becomes essential to assess the degradation of stiffness in the beam column joints subjected to reversal loading.

The stiffness versus average displacement is shown in Fig. 11. To obtain the stiffness, the average load(P) which is based on the peak values of each hysteresis loop was divided by the corresponding average displacement (δ). That is, the stiffness (K) was calculated from the relation, $K= (P/ \delta)$. From the stiffness versus displacement graph, between Groups-I and II, specimens A1 and A2 have higher values than specimens B1, C1, B2 and C2.

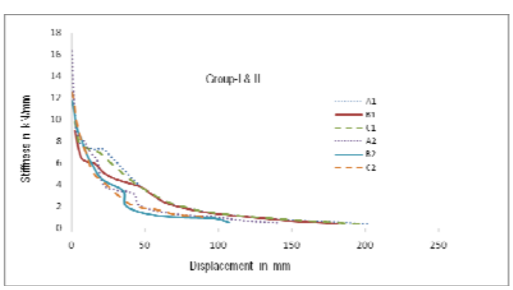

Fig. 11. Stiffness Vs displacement

Table 3. Stiffness of test specimen

Specimen number & groups	Yielding displacement in mm(δ_y)	Ultimate load in kN(P_u)		Average stiffness in kN/mm ($k=P_u/\delta_y$)
		Left hand side (LHS)	Right hand side (RHS)	
A1-I	2.15	89.00	90.00	41.627
B1-I	2.40	89.00	91.00	37.500
C1-I	2.20	88.00	90.00	40.454
A2-II	2.30	80.00	81.00	35.000
B2-II	2.85	78.00	80.00	27.719
C2-II	3.00	78.50	80.50	26.500

Table 3 shows only the average Initial stiffness (Initial stiffness $K= P_u/ \delta_y$, wherein P_u is the Ultimate load and δ_y is the yielding displacement). It has been observed from the experimental results that in Group-I, specimen A1 has the higher stiffness than specimens B1 and C1 and in Group-II, specimen A2 has the higher stiffness than specimens B2 and C2. The specimen A1 which had the proposed additional X-cross bar with hair clip joint detail exhibited better performance among these six specimens against stiffness degradation (stiffness of A1 is higher than A2 by 15.92%). Between the two Groups, Group-I has the higher stiffness.

d) Crack study

Fig. 12. Crack Pattern of Group-I (A1, B1, and C1)

The anchorages and joint details of specimens A1, B1 and C1 are shown in Figs. 1a, 1b, 1c, and Fig. 3 respectively. It can be seen from Fig. 12, shear cracks have developed on the beam-column junction in all the specimens where the plastic hinge formed at the face of the column. Further, diagonal cracks have also developed in the column shear panel area of the specimens B1 and C1. Besides the wide open cracks in the junction, the concrete had also spilled out from the specimens B1 and C1. In Group-I, the specimen with mechanical anchorage with X-Cross plus U-bars (A1) shows the lesser cracks and much better control of crack capacity than the other specimens. It can therefore be concluded that these types of joint core details are much more effective in controlling beam-column joint than conventional joints. It is apparent that the use of mechanical anchored bars is a viable alternative to use of standard 90^0 hooks in exterior beam-column joints in combination with the moderate and higher seismic prone area.

The anchorages and joint details of specimens A2, B2 and C2 are shown in Figs. 1a, 1b, 1c, and Fig. 3 respectively. It can be seen from Fig. 13 that shear cracks have developed in the beam-column junction and diagonal cracks have developed in the column shear panel area of all the specimens and the wide open crack pattern can be observed only in the specimens B2 and C2. In addition, the concrete had spilled out from the specimens B2 and C2 with buckling of beam longitudinal reinforcement. In Group-II, specimen A2 (mechanical anchorage) has the lesser crack pattern with only a few diagonal cracks were formed at joint core than specimens B2 and C2. In this performance study towards cracks of all these specimens, the specimen A1 shows an excellent performance with few shear cracks.

Fig. 13. Crack Pattern of Group-II (A2, B2, C2)

The specimens have 90^0 bent tensile anchorage bars which induce a compressive stress in the joint diagonally forming a compression strut due to contact pressure under the bend. Tension tie developed in the joint perpendicular to the direction of the strut induces a tensile stress. Diagonal cracks are developed perpendicular to the direction of the diagonal tension tie in the joint shear panel area. The specimens A1 and A2 with mechanical anchorage shows a lesser crack pattern than other specimens using conventional joints details in Group-I and II without losing the strength, however, specimen A1 with mechanical anchorage in combination with U-bar plus X-Cross bar, shows lesser cracks and much better control of crack capacity than other specimens. The X-cross bar is provided to control tensile failure in concrete of the joint shear panel area due to strut and tie action. Stranded conventional shear links are replaced with U-bar for easier fabrication. It can therefore be concluded that mechanical (headed bar) types of anchorages with proposed joint core details are much more effective in controlling beam-column joint. It is apparent that the use of mechanical anchored bars is a viable alternative to use of standard 90^0 hooks in exterior beam-column joints in seismic prone area. In addition, it effectively reduces the reinforcement congestion and is easier to repair using FRP composite wraps techniques to restore the flexural strength, ductility of earthquake damaged concrete beam-column joints (Mostofinejad & Talaeitaba [22]; Eshghi & Zanjanizadeh [23]; Esfahani et al., [24]; Sharbatdar et al. [25]).

9. CONCLUSION

The following suggestions for the detailing of reinforced cement concrete T-type exterior beam-column connections are made from the knowledge gathered through the experimental test results of beam-column joints.

1. It has been observed from the experimental test results that the T-type mechanical anchorage as per ACI-352 (specimens A1 and A2) offer better performance than the specimens reinforced with conventional 90^0 standard bent hooks anchorage as per ACI-318(specimens B1and B2) and full anchorage as per IS-456(specimens C1and C2). In addition, significant improvement in the ductility was observed in that Group-I exhibit higher ductility than Group-II specimens A2, B2 and C2 by 10.70%,36.97% and 20.58% respectively.

2. The specimen A1 and A2 with mechanical anchorage shows lesser crack pattern than other specimens using conventional anchorage and joint details. However, specimen A1 with mechanical anchorage (ACI-352, mechanical anchorage) in combination with X-cross bar plus U-bar shows lesser cracks and much better control of crack capacity with improvement in seismic performance for higher seismic prone areas where moderate and severe ductility is in demand.
3. The T-Headed bar anchorage and the joint details in the beam-column joints not only reduces the congestion of reinforcement in the joint core area but also eases the pouring of concrete and helps in faster construction at site. In addition to improvement in seismic performance, it is apparent that the use of mechanical anchored bars is a viable alternative to use of standard 90^0 hooks in exterior beam-column joints.
4. In Indian design practice, beam-column joints are given less attention. The above finding, recent research and suggestions by various national and international codes for using the mechanical anchorage systems may be accounted for in the upcoming revisions.

NOMENCLATURE

A_c	area of column core measured from outside edge to outside edge of spiral or hoop reinforcement
A_g	gross area of column cross section
A_{sh}	area of the bar cross section (IS-Code)
A_{sh}	total cross-sectional area of all legs of hoop reinforcement, including crossties, crossing a section having core dimension $b_c^{"}$
A_k	area of confined concrete core in the rectangular hoop measured to its outside dimensions
A_s	area of tension reinforcement
b	width of the compression face
$b_c^{"}$	core dimension of tied column, outside to outside edge of transverse reinforcement bars, perpendicular to the transverse reinforcement area A_{sh} being designed
b_j	effective width of the joint transverse to the direction of shear
d	effective depth
d_b	nominal diameter of bar
f_c'	compressive strength of concrete
f_{ck}	characteristic compressive strength of concrete
f_y	yield stress of reinforcement
f_{yh}	yield stress of spiral, hoop, and crosstie reinforcement
h	longer dimension of the rectangular confining hoop measured to its outer
h_c	depth of the column
h_{st}	height of the column
L_d	development length
L_{dh}	development length for a hooked bar, measured from the critical section to the outside edge of the hook extension
L_{dt}	development length for a headed bar, measured from the critical section to the outside end of the head
S_h	center-to-center spacing of hoops or hoops plus crossties
S	pitch of spiral or spacing of hoop, (the spacing of hoops used as special confining reinforcement shall not exceed ¼ of minimum member dimension but need not be less then 75mm nor more than 100mm)
T_b	tension force in the reinforcement
V_n	nominal shear strength of the joint
V_{col}	shear in the column calculated based on M_{pr} for beam.
σ_s	stress in bar ($0.87*f_y$) at the section considered at design load
τ_{bd}	design bond stress of concrete (can be increased by 60% for deformed bars)
α	stress multiplier for longitudinal reinforcement at joint-member interface for Type-2, $\alpha \geq 1.25$
γ	shear strength factor reflecting confinement of joint by lateral member
Ø	nominal diameter of the bar

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