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STRENGTHENING OF SHEAR DEFICIENT RC BEAM-COLUMN JOINTS IN MRFS UNDER SEISMIC LOADING

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Abstract: Reinforced concrete (RC) moment resisting structures built during the early 1950's through 1970's are vulnerable for earthquake loads due to lack of adequate strength and ductility. Beam-column joint, the common region between the framing beams and columns, is a crucial zone to ensure global response of such moment resisting structures. Many of such structures all over the world need immediate measures for upgrading their performance level to withstand the seismic loading effects. Several methods have been attempted over the years by many civil engineers and practitioners for strengthening of deficiently detailed RC beam-column joints. In this paper, an emphasis has been made to understand the joint vulnerability against lateral loads and review of various retrofitting methods and their efficiency for RC beam-column joints. Further, some experimental investigations on the performance of joints strengthened with haunch elements have been reported. The numerical studies show that at the location of 0.2 times the span of the beam from the center of the column at a orientation angle of 45^0 produced the highest reduction of shear stress in the joint region. The experimental investigations show that the RC beam-column joints designed with haunch elements exhibited better performance in terms of significant shear strength, ductility, less stiffness degradation and energy absorption under cyclic loading.

1. INTRODUCTION

Many reinforced concrete (RC) buildings, such as non-ductile RC frames, designed during the 1950s through 1970s existing today in many parts of the world do not satisfy the current seismic design requirements. These buildings generally do not possess adequate ductility due to poor detailing of reinforcement. Observations made on the failures of the existing structures due to earthquakes reveal that strengthening or retrofitting is necessary due to (i). poor detailing of joint reinforcement, (ii). deficient materials and inadequate anchorage length of beam reinforcement, (iii). improper confinement of joint region by transverse reinforcements, (iv). changes in the current design detailing requirement and (v). changes of loads due to frequency of earthquakes and alterations of earthquake zones.

Figure 1: Beam-column joint shear failure in RC buildings [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 need for study of earthquake effects on structures was realized when earthquakes occurred through the 1960s and 1970s causing irreparable damage and human loss. The design of joints was not given importance in the framed structures designed for gravity loads or gravity and routine live loads only. This causes severe problem in the event of an earthquake. Several studies led to the development of ASCE-ACI 352 Committee [2].recommendations for the design of reinforced concrete beam-column joints (connections) in the year 1976. But there is a lot that has still not been understood about beam-column joint and research needs to highlight these issues.

2. SHEAR TRANSFER MECHANISM

For the design purposes, the horizontal component of the joint shear stress can be calculated from the combined effect of: (i). diagonal strut mechanism, to consider the contribution of concrete in the joint; and (ii). truss mechanism, to consider the contribution of the joint shear reinforcement. Figure 2 shows the forces in the beam bars, the joint mechanism and the force components in the joint for calculating the joint shear strength.

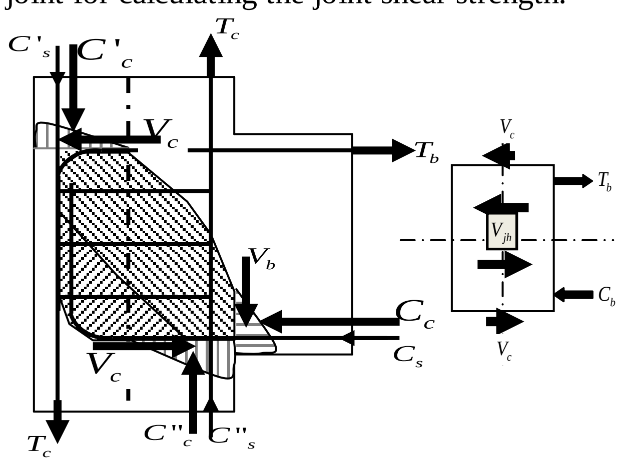


Figure 2: Shear mechanism in exterior joint.

As shown in Figure 2, the equilibrium of forces acting above the horizontal plane passing through the centroidal axis of the exterior beam-column joint is as follows

a) In terms of external forces: $V_{jh} = T_b - V_c$
b) In terms of internal force: $V_{jh} = V_{ch} + V_{sh}$

Horizontal component of joint shear force,

$$V_{jh} = V_{ch} + V_{sh} \tag{1}$$

Where, V_{ch} is the horizontal component of diagonal compression strut

$$V_{ch} = D_c \cos\alpha \; = C_c + \Delta T_c - V_{col} \tag{2}$$

V_{sh}=Horizontal joint shear force resisted by horizontal reinforcement by truss mechanism

$$V_{sh} = A_{jh} f_{yt} \tag{3}$$

Horizontal component of the joint shear stress can be calculated by;

$$\tau_{jh} = \frac{V_{jh}}{A^h_{jcore}} \tag{4}$$

Where,
D_c = diagonal compression strut at angle "α" to horizontal axis of joint
C_c = concrete compression force
ΔT_c = force in steel transmitted through bond to strut, over depth "c" of the flexural compression zone in the column
V_{col} = shear force in column
A_{jh} = horizontal joint reinforcement
f_{yt} = yield strength of joint reinforcement
A^h_{jcore}= horizontal c/s area of the joint

3. STRENGTHENING METHODS

Several techniques were adopted to strengthen beam-column joints such as use of concrete jackets, bolted steel plates and jacketing using corrugated steel sheets [3,4]. The joints strengthened using various steel-plate and angle rehabilitation systems were varied from simple to complex and were shown to be satisfactory in improving the joint shear strength and ductility. *Ghobarah et al.* [5] proposed use of mechanical anchors to prevent the bulging problems associated with flat steel jackets. *Ghobarah et al.* [4]. Investigated a retrofitting using corrugated steel jackets to encase the joint for prevention of bulging of the jacket and upgrading the shear strength of joint.

Figure 3: Joint failure of GRP rehabilitation *(Gobarah and Said, 2002)*

Ghobarah and Said (2002) used GFRP composites, as shown in Figure 3 to develop effective rehabilitation schemes for reinforced concrete beam-column joints. GFRP jacket increased the shear resistance of the joint and enhanced the performance of the connection from ductility point of view. Anchoring of FRP is important to provide confinement to the joint because the joint area is limited, and there is a need to develop the full strength of FRP with adequate anchorage.

Diagonally applied carbon fibre unidirectional strips outperformed the vertical ones. In a similar study *(Spadea et al. 1998)*, an emphasis was made on the importance of FRP anchorage in order to develop its full strength. One-third scale exterior beam-column joints with different wrapping configurations using FRP showed limited improvements in the overall performance such as peak load, ductility and energy-dissipation capacity. Only limited success has been achieved using FRP, due to problem associated with confinement of beam-column joints.

The conventional retrofitting schemes such as addition of RC and/or steel jackets were used for strengthening of joints and joint assemblies [3,7]. Joints enhanced strength regardless of reinforcement detailing and damage state. The joints with adequate anchorage length exhibited ductile behaviour with long plastic zones and the joints without proper anchorage resulted in pullout of bars from the joint.

Hakuto et al. [8] tested interior and exterior beam-column joints without transverse reinforcement and inadequate anchorage of longitudinal bars. By adopting concrete jacketing and using current detailing of reinforcement, the performance of beam-column joints was improved. The exterior beam-column joints similar to pre-seismic code or gravity load only design were tested for effectiveness of reinforcement detailing in the joints [9]. As it was expected, the joint suffered shear failures and poor energy dissipation capacity. The reinforcement detailing adopted as per ACI 318 provisions resulted in improved performance of the joint. By providing longitudinal beam bar anchorages and lateral reinforcement details, the seismic performance of the joint can be improved. The detailing of reinforcement may be adopted to shift the predetermined location of the plastic hinge by bending longitudinal bars away from the column face.

The effect of amount of reinforcement bars, the ratio of column–to-beam flexural capacity and the joint shear stress are studied [10]. A significant improvement of the joints reinforced with inclined bars is observed. The influence of size of beam-column joints on the general behaviour has been verified [11]. A higher rate of stiffness deterioration was occurred in small size joints due to weak bond between model reinforcement and mortar. Under large shear stress reversals, the beam-column joints constructed with 1.5% polyethylene fibers improved the joint strength without any lateral joint reinforcement [12]. The joint shear strength is comparable with the ACI Committee 352 shear stress limits. Excellent bond between longitudinal bars and surrounding HPFRCC has been observed though the joint was not provided with adequate development length as per ACI 318 provisions. *Geng et al.* [13] adopted CFRP jacketing for retrofitting of weak beam-column joint models without sufficient development length and ductility by wraping the CFRP sheets on the beam-column joints. The deficient detailed joints showed slipping and pulling out of tensile reinforcement in the joint, while ductility and capacity of CFRP retrofit joints were improved. The techniques prevented the crushing of concrete and shear cracking in the joint with significant ductility.

The retrofit schemes enumerated above have issues like effectiveness, resources, invasiveness, cost and practical implementation to overcome. All these strengthening methods aim at improving the strength of member which may be degraded after some cycles of loading.

A new and non-evasive retrofit strategy introducing haunch elements close to the beam-to-column joints as a means of enhancing the seismic response of joint sub-assemblages was suggested by *Pampanin and Christopoulos* [14].

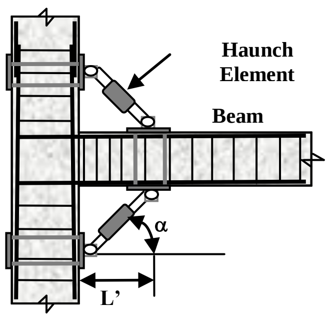


Figure 4a: Haunch retrofit for exterior Joint.

The basic idea of proposing haunch retrofit is to transfer critical joint shear damage while enhancing the global response of non-seismically detailed joints. Figure 4 shows a typical haunch element scheme.

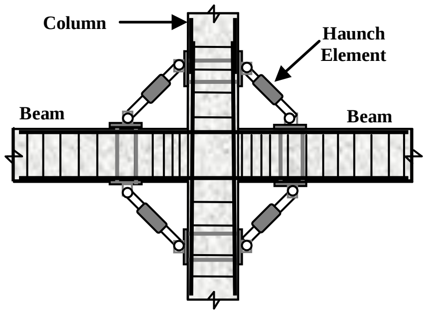


Figure 4b: Haunch retrofit scheme for interior joints.

Assuming inflexion points at the mid points of the span in columns and beams under applied lateral load, the bending moment diagram in members of an exterior joint is shown in Figures 5 and 6. The maximum moment in the beam M_{bc} occurs at the face of the column, while moments M_c represent moments along the centerline of the columns located at a distance $d_c/2$ from the face of the column, "d_c" is depth of the column. When the moment in the beam at the face of the column, M_{bc} reaches a critical value M_j, cracking and failure under cyclic loading occur if no other mechanisms such as plastic hinging of the beam occurs first. The value of M_j depends on the principal stresses in the joint, which are dependent on the axial force and shear in the column.

The interstorey shear in the joint is:

$$V_c = M_j^* \frac{\left(1 + \frac{d_c}{L_b}\right)}{H_c} \tag{5}$$

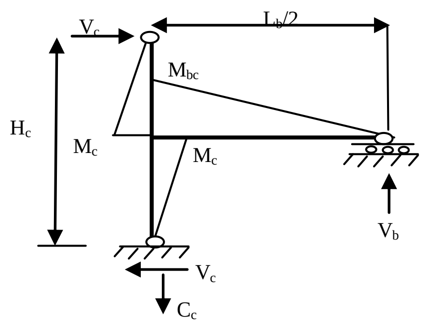


Figure 5a.BMD without haunch elements.

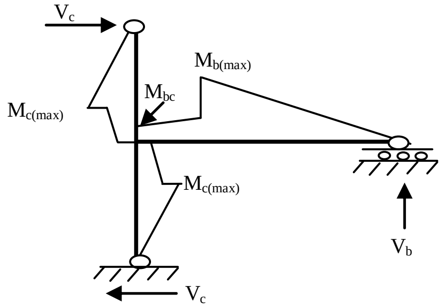


Figure 5b.BMD with haunch elements.

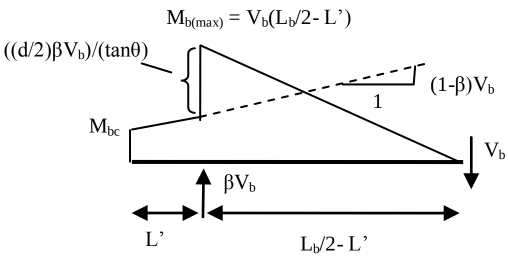


Figure 6a:BMD in beams with haunch elements.

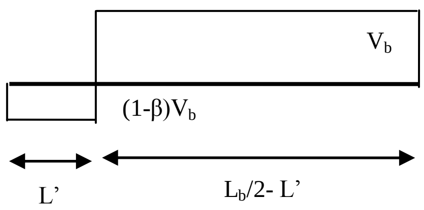


Figure 6b: Shear force diagram in beam.

4. NUMERICAL STUDIES

The numerical model is a 2D frame with 4 bays and 6 storeys including ground floor. The ground floor height is 4.0m and other floors are each 3.0m height. The length of the beams is 4.0mm. The plan dimensions of the floor of the building are 16m x 16m. The materials and sectional dimensions adopted for the structural members in the frame are shown in Table 1. Gravity loads include self weight of members, wall loads and floor finishes and live load is 4.0 kN/m^2. Seismic loading is as per Indian code of practice corresponding to Zone-V. The design parameters adopted for the seismic analysis are as follows: Zone Factor, (Seismic Zone-V), Z = 0.36, Importance Factor, I = 1.00, Response Reduction Factor, R= 5.0

In order to obtain the necessary data, various combinations of location and orientations of haunch are used. Location of the haunch, designated as L' from the centre of the column was (10, 12.5, 15, 20, 25, 40, 50) % L (L = effective length of beam), and orientation, α, of haunch with the axis of the column was 30^0, 45^0, and 60^0.

5. JOINT SHEAR FORCE

The joint shear force at the centre of the joint on the horizontal plane is the algebraic sum of the forces acting above or below the horizontal plane..

Table 1. Materials and Dimensions of members.

Compressive strength of concrete	f_{ck}	30 MPa
Yield strength of steel	f_y	415 MPa
Width of beam	b	300 mm
Depth of beam	D	400 mm
Width of column	b	500 mm
Depth of column	D	500 mm
Effective depth	d	360 mm
Cover to reinforcement	d'	40 mm
Area of tension reinforcement	A_{st}	600 mm^2
Area of compression reinforcement	A_{sc}	600 mm^2

Figure 7 shows the percentage reduction of joint shear force. The joint shear force decreases as the distance of the location of the haunch along the beam increases. Similar trend has been observed with different orientation angles of the haunch. The highest reduction of joint shear force has been observed when the distance of the location of haunch is about 0.2L. Beyond this location, there has not been much reduction in the joint shear force.

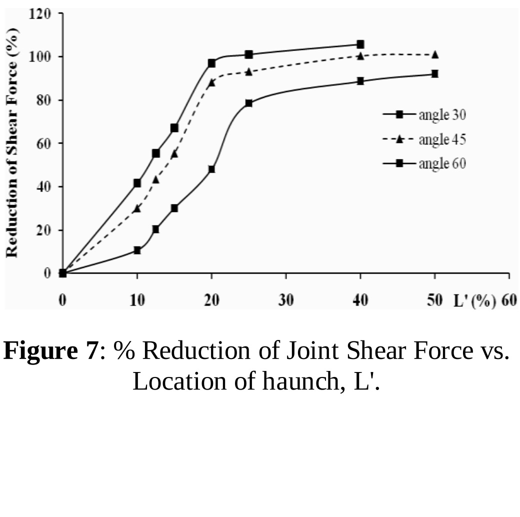


Figure 7: % Reduction of Joint Shear Force vs. Location of haunch, L'.

6. EXPERIMENTAL PROGRAMME

A T-shaped beam and column assembly has been identified to represent the essential components of a beam-column sub-assemblage in a 2D RC building frame subjected to lateral cyclic loading. The inflection points in a moment resisting frame are assumed at the mid-heights of columns and the mid spans of the beams. The assemblages were designed for gravity loading and the detailing was typical of pre-seismic design code. The beam-column joints tested are designated to study (i). the effect of transverse beam stub (BCJ-BE-RE and BCJ-BE-HE), (ii). the effect of joint reinforcement; (BCJ-JR-MN and BCJ-JR-CY), and (iii). the effect of eccentricity (BCJ-00-RE, BCJ-00-EN, and BCJ-00-HE).

The joints BCJ-BE-HE and BCJ-00-HE are provided with haunch elements and the joint BCJ-00-EN has eccentricity. The details of the joints are as given in Table 2. The column is 1800 mm long and beam span is 1500 mm for all sub-assemblages. The beams have same amount of top and bottom reinforcement.

6.1.Experimental Set-up

For loading the joint, system was designed for simulating quasi-static push-pull experiment. A reaction frame of 200 tonnes capacity was used to support the test set-up. The column in the beam-column assemblage was hinged at the top and bottom and was supported by the reaction frame from top. At the bottom, additional support was given to restrict the translation of the column. The assembly at bottom was connected to the strong testing floor via high strength bolts. One dimensional rollers were seated beside the column to allow in-plane rotation at both ends of the column. The column was subjected to constant axial load along its longitudinal axis using two hydraulic jacks placed below the column. To uniformly apply the axial load across the column, a capping box made of 25 mm thick steel plate of internal dimensions 400mm × 250 mm × 150 mm was used at both top and bottom ends. A hydraulic actuator supported vertically from the reaction frame was arranged at the beam end to apply cyclic loading at the beam-tip. A hinge swivel was attached with the actuator so that the load on the beam always remained vertical. The set-up is as shown in Figure 8.

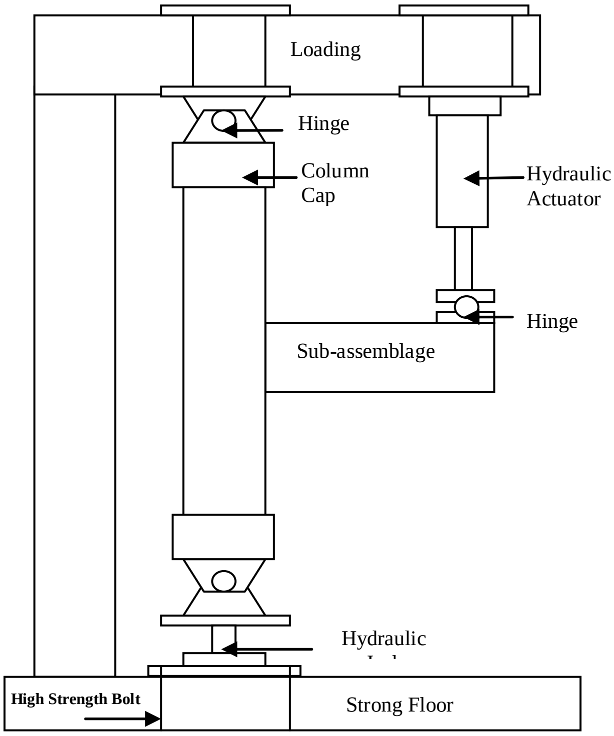


Figure 8: Diagram of the Experimental Set-up.

6.2. Loading and Measurement

Displacement control system was adopted for testing of all the beam-column joints. All the beam-column sub-assemblages except joint BCJ-JR-MN were tested under cyclic loading. The joint BCJ-JR-MN was loaded monotonically at the beam end. The axial load on the columns (P_{col}) was applied by load controlled hydraulic jack of capacity 750 kN. The axial load, P_{col} applied on the column was 10% of the capacity of the column. The column axial load was applied first and then the same was maintained constant throughout the testing. A hydraulic actuator of 1000kN capacity was adopted with displacement control to apply varying displacement cycles over the joint at the beam end. The loading was applied at increasing amplitudes of displacements, varying from 1.0mm to 60 mm. Each displacement was applied over two cycles of loading and unloading.

6.3.Design of Haunch Element

The haunch element was designed according to the capacity design concept intended to develop a proper strength hierarchy. This is so that the system is effective in preventing hinge formation in the joint region and also to allow plastic hinging in the beam.

Double-angle steel sections placed back-to-back were used as the haunch element. These were connected to a gusset plate which in turn was connected to an anchor plate. The plates were held in position with the help of high strength bolts. The haunch element was designed for both compression and tension. The assembly is made such that no slip should occur. To avoid slip of anchor plate, extra bolts have been drilled through the plates to the concrete (in BCJ-00-HE). The haunch element assembly is as detailed in Figure 9.

7. RESULTS AND DISCUSSION

7.1. Failure Pattern

In most of the joints, initial cracks appeared on the beams at around 30kN beam tip loading. The cracks in the beam-column interface and diagonal cracks in the joints started forming at higher displacement cycles. The interface crack was the main crack observed in joints BCJ-BE-RE, BCJ-JR-MN and BCJ-JR-CY. The joints BCJ-00-RE and BCJ-00-EN underwent significant shear cracking in the joint region. The haunch fitted joints BCJ-BE-HE and BCJ-00-HE showed an altered crack pattern due to the effect of the haunch. The beam underwent shear cracking at higher displacement cycles and there was local crushing and spalling of concrete observed near the beam where haunch was connected. A comparison of crack patterns of joints BCJ-BE-RE and BCJ-BE-HE is shown in Figure 10.

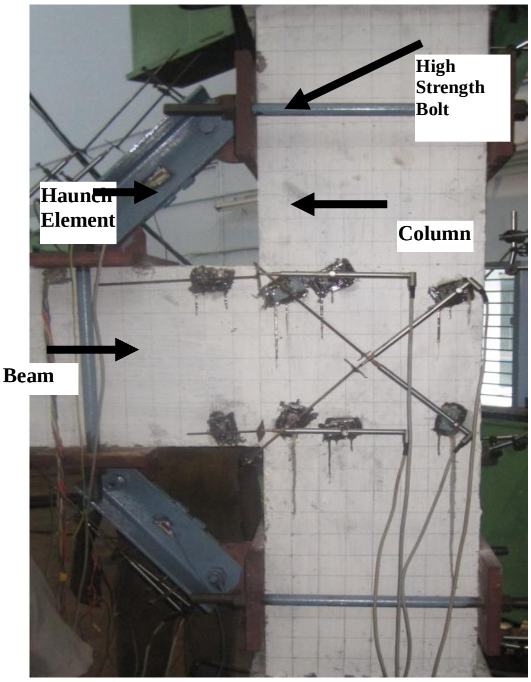
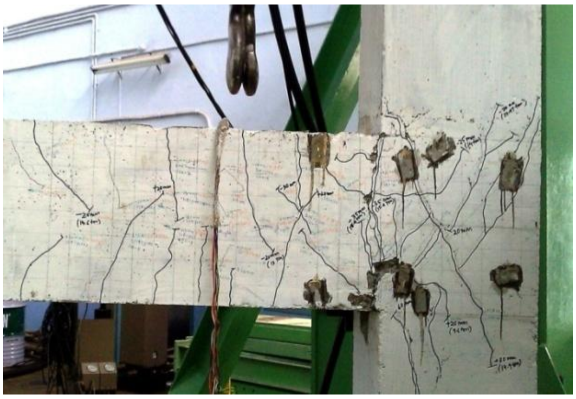


Figure 9: Experimental assembly for haunch element.



(a). BCJ-BE-RE

(b). BCJ-BE-HE

Figure 10: Crack Pattern in Joints.

Table 2: Joint Dimensions and Reinforcement Details

S. No	Beam Details			Column Details			Stirrups	Transverse beam			Joint stirrups
	Width (mm)	Depth (mm)	Reinforcement	Width (mm)	Depth (mm)	Reinforcement		Width (mm)	Depth (mm)	Reinforcement	
BCJ-BE-RE	250	400	5-20 mm φ	250	400	10-20 mm φ	8mm @ 150 mm c/c	400	400	6- 20 mm φ	
BCJ-BE-HE	250	400	5-20 mm φ	250	400	10-20 mm φ	8mm @ 150 mm c/c	400	400	5-20 mm φ	
BCJ-JR-MN	200	300	3-20 mm φ + 2-16mm φ	200	300	8-20 mm φ	8mm @ 150 mm c/c				8mm @ 80 mm c/c
BCJ-JR-CY	200	300	3-20 mm φ + 2-16mm φ	200	300	8-20 mm φ	8mm @ 150 mm c/c				8mm @ 80 mm c/c
BCJ-00-RE	200	400	4-20 mm φ + 2-16mm φ	250	400	12-20 mm φ	8mm @ 150 mm c/c				
BCJ-00-EN	200	400	4-20 mm φ + 2-16mm φ	250	400	12-20 mm φ	8mm @ 150 mm c/c				
BCJ-00-HE	200	400	4- 20 mm φ + 2-16mm φ	250	400	12-20 mm φ	8mm @ 150 mm c/c				

Table 3: Details of the Load and Displacement Response

Joint	Loading	Haunch	Transverse Beam	Joint stirrups	Eccentricity	Ultimate Load (kN)		Displacement at ultimate load (mm)	
						+ve	-ve	+ve	-ve
BCJ-BE-RE	Cyclic	-	YES	-	-	123	-157.3	30	-30
BCJ-BE-HE	Cyclic	YES	YES	-	-	177.01	-226.73	55	-55
BCJ-JR-MN	Mono	-	-	YES	-	-	-96.6	-	-60
BCJ-JR-CY	Cyclic	-	-	YES	-	100.45	-84.23	50	-45
BCJ-00-RE	Cyclic	-	-	-	-	117.25	-123.05	35	-30
BCJ-00-EN	Cyclic	-	-	-	YES	89.12	-104.95	35	-30
BCJ-00-HE	Cyclic	YES	-	-	-	140.35	-148.10	40	-45

Table 4: Estimation of Joint Shear Strength

Joint	Design shear strength $0.85V_n$ (kN)	Maximum shear force in joints, V_{jh} (kN)	$\frac{Predicted}{Experimental} = \frac{0.85V_n}{V_{jh}}$
BCJ-BE-RE	579.627	524.33	1.105
BCJ-BE-HE	579.627	739.05	0.703
BCJ-JR-MN	347.776	448.12	0.776
BCJ-JR-CY	347.776	474.29	0.733
BCJ-00-RE	380.380	355.48	1.070
BCJ-00-EN	344.153	303.19	1.135
BCJ-00-HE	380.380	427.83	0.889

7.2. Load vs. Displacement

The measured load at beam end (P_b) versus the corresponding applied displacement (Δ_b) was used to develop the Load vs. Displacement response of sub-assemblage. The observed maximum loads and the corresponding beam end displacements are shown in Table 3.

The load versus displacement response showed typical hysteresis properties and a comparison of responses of joints BCJ-BE-RE (control) and BCJ-BE-HE (haunch fitted) are shown in Figures 11.

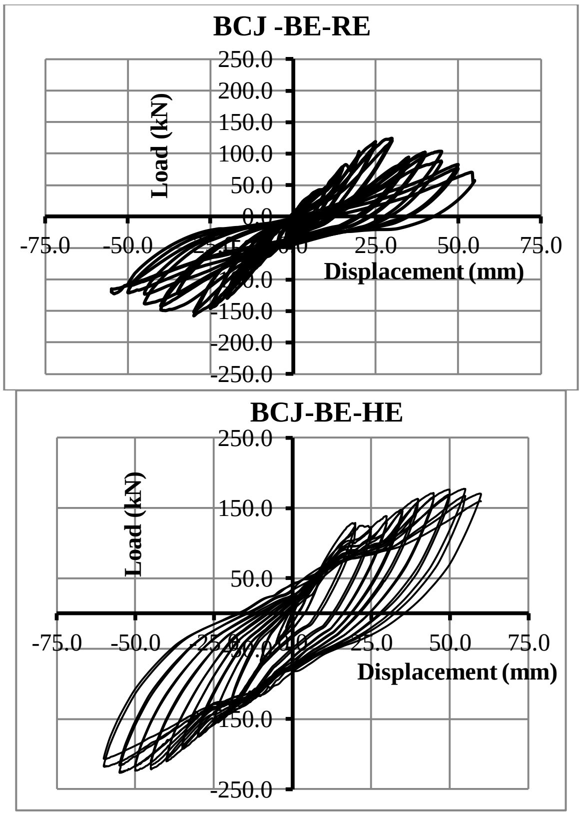


Figure 11: Load vs. Displacement for Joints.

The performance of BCJ-BE-HE is superior to BCJ-BE-RE in terms of ultimate load carrying capacity. The addition of haunch element is responsible for an increase of 44% in the maximum load carrying capacity of the joint BCJ-BE-HE as compared to the joint BCJ-BE-RE. Similar observations have been made in the joints BCJ-00-RE and BCJ-00-HE. Due to premature failure of joint BCJ-00-HE, full capacity of the joint could not be achieved. Nevertheless, an increase of 20% in the maximum load carrying capacity has been attained. The effect of eccentricity in the joint region has shown pronounced influence in the joint BCJ-00-EN. The load carrying capacity of the eccentric joint is low (15% decrease) as compared to the concentric joint BCJ-00-RE. Also, the failure has occurred at a much lower value of displacement. Joint stirrups were responsible for increased shear capacity of joints and better energy dissipation

7.3. Shear Strength of Joints

The shear stress in joints is calculated as per ASCE-ACI 352 report. The horizontal component of joint shear force V_{jh} is given as

$$V_{jh} = T_b - V_{col} \tag{6}$$

Where T_b is the tension in beam (kN) and V_{col} is the shear force in column (kN).

The nominal shear strength of the joint V_n according to ACI 352 [15].depends up on the strength of concrete, joint dimensions, confinement from various framing members can also be calculated as:

$$V_n = 0.083\gamma\sqrt{f_c'}b_j h_c \tag{7}$$

Where, f_c' is compressive strength of concrete (N/mm^2), b_j is effective joint width (mm) and h_c is depth of the column (mm). γ is a constant taken according to ACI 352 as 15 for all the joints. A constant 0.85 is the shear reduction factor on V_n taken for design. A summary of the predicted V_n and experimental V_{jh} for all the joints is shown in Table 4.

From Table 4, it can be observed that joints BCJ–BE-HE, BCJ-JR-MN, BCJ-JR-CY and BCJ-00-HE exhibited higher shear capacity than the predicted. This is due to the addition of haunch in joint BCJ-BE-HE and BCJ-00-HE and presence of joint lateral reinforcement in joints BCJ-JR-MN and BCJ-JR-CY. The joint BCJ-BE-HE experienced maximum shear stress and also underwent significant amount of shear deformation. But the failure pattern did not suggest excessive damage in the joints, indicating increased shear capacity.

7.4. Energy Dissipation

Energy dissipation in the structure is a measure of its seismic performance. The more is the energy dissipation, the better the seismic resistance of a structure. The energy dissipated by a structure is calculated from the area under the load-displacement curve. For comparison purposes, the cumulative energy dissipation is normalized by dividing it by the volume of the joint and grade of concrete. Typical energy dissipation for joint BCJ–BE-RE over subsequent cycles is shown in Figure 12 and the comparison of energy dissipated across the joints is shown, Figure 13.

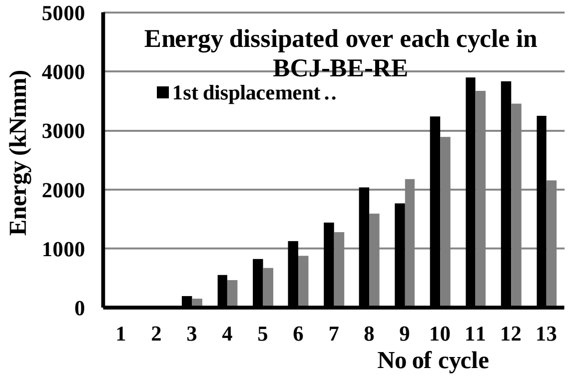


Figure 12: Energy Dissipation in Joint BCJ-BE-RE.

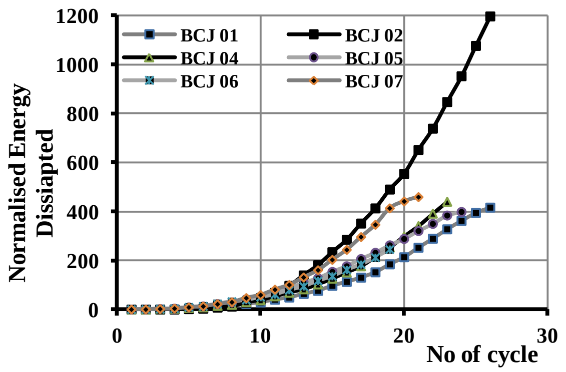


Figure 13: Cumulative Energy Dissipation.

Larger hysteresis loops give way to greater energy dissipation and this is evident in the case of joints BCJ-BE-HE and BCJ-00-HE. The presence of joint stirrups has contributed to an improvement in the energy dissipation capacity as indicated by joint BCJ-JR-CY. The eccentricity has affected the energy dissipation capacity of joint BCJ-00-EN, which reports low energy dissipation. The joint BCJ-BE-RE exhibited low energy dissipation, owing to the presence of a transverse beam. A 180% increase in overall energy dissipation in joint BCJ-BE-HE as compared to control joint BCJ-BE-RE. Joint BCJ-00-HE showed 51% increase in overall energy dissipation over joint BCJ-00-RE.

7.5. Stiffness Degradation

The stiffness of a sub-assemblage is calculated from the load-displacement response. The peak-to-peak stiffness is deduced and the degradation is shown over subsequent cycles. The slope from the positive peak to negative peak in the load-displacement response gives stiffness for a cycle. For comparison, the stiffness is normalised by dividing with the initial stiffness. The stiffness degradation in the beam-column joint sub-assemblage is shown in Figure 14.

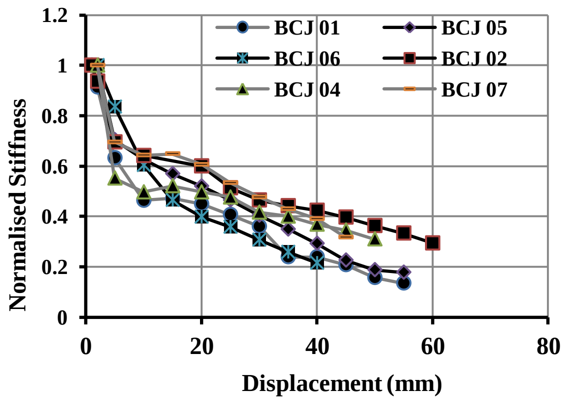


Figure 14: Stiffness Degradation in Sub-assemblages.

The stiffness degradation responses from various sub-assemblages show similar trend. The deterioration is more at increased displacements and this has resulted in pinching of hysteresis loops. The comparison of normalized stiffness indicates that the degradation is high in the case of joint BCJ-

00-EN. This can be attributed to the effect of eccentricity causing additional torsional moments in the joint. Both joints BCJ-BE-HE and BCJ-00-HE show comparatively lesser degradation, which is due to the effect of the haunch element. The joints BCJ-BE-RE and BCJ-BE-HE show high initial stiffness. This may be due to better confinement of the joint from the transverse beam framing in to the joint. The trend line for joint BCJ-JR-CY shows marginal improvement over the joints without joint reinforcement.

7.6. Ductility Ratio

Ductility ratio (cyclic) (D): The ratio of the ultimate displacement (Δ_{ult}) and the yield displacement (Δ_{yield}) of the joint observed in cyclic test.

Envelope curve: The locus of extremities of the load-displacement hysteresis loops, which contains the peak loads from the first cycle of each phase of the cyclic loading and neglects points on the hysteresis loops where the absolute value of the displacement at the peak load is less than that in the previous phase.

The ductility is calculated from the envelope curve by developing an equivalent energy elastic-plastic (EEEP) curve. EEEP curve is an ideal elastic-plastic curve circumscribing an area equal to the area enclosed by the envelope curve between the origin, ultimate displacement and the displacement axis as shown in Figure 15.

$$\text{Ductility Ratio, } D = \frac{\Delta_{ult}}{\Delta_{yield}} \tag{4}$$

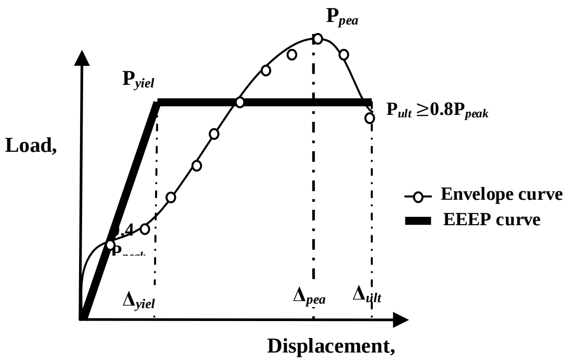


Figure 15: Development of EEEP curve.

Ductility ratios of various joints are given in Table 5. It can be inferred that the confinement in the joint improves the ductility as the joints BCJ-BE-RE and BCJ-BE-HE reported higher ductility ratios. Both the joints BCJ-BE-HE and BCJ-00-HE with haunch elements exhibited improvement in the ductility compared with the control joints BCJ-BE-RE and BCJ-00-RE respectively. The eccentric joint BCJ-00-EN showed lower ductility due to influence of additional shear stresses developed due to torsion in the joint due to eccentricity. Joint BCJ-BE-HE reported a higher ductility ratio of 10.21 over a ductility ratio of 7.42 for joint BCJ-BE-RE, whereas joint BCJ-00-HE exhibited 6.32 over a ductility of 5.8 of joint BCJ-00-RE.

Table 5: Ductility values of the sub-assemblages

Joint	Ultimate displacement $\Delta_{ul,}$ (mm)	Yield displacement Δ_{yield} (mm)	Ductility Ratio $D = \frac{\Delta_{ult}}{\Delta_{yield}}$
BCJ-BE-RE	55	7.41	7.42
BCJ-BE-HE	85	8.32	10.21
BCJ-JR-MN	70	13.3	5.26
BCJ-JR-CY	55	7.76	5.80
BCJ-00-RE	40	9.56	4.18
BCJ-00-EN	55	8.70	6.32

8. CONCLUSIONS

The following conclusions have been drawn from the study:

1) The haunch-fitted joints showed the maximum load carrying capacity over control joint.
2) The addition of haunch elements resulted in higher energy dissipation, less stiffness degradation and large ductility ratio.
3) The eccentricity induced additional torsion in joints, which caused pre-mature failure in the joints.
4) Confinement of joints was found to marginally improve the joint performance.
5) The transverse beam enabled higher ductility and high initial stiffness to joints.

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