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Performance of Exterior Column Beam Connections as built at site

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Abstract: ***Comparing performance of ill detailed and ductile detailed connections as per IS: 13920 was the prime aim of this study. It was by and large observed during literature review that, usually investigations are done at laboratory casting and curing conditions. How-ever, site conditions are grossly different from laboratory conditions. It is a general experience that the ductile detailing is rarely followed at site. Thus it was felt necessary to investigate performance of such ill detailed constructions at site testing conditions. Specimen for this experiment were manufactured by site people at site conditions and cured at site conditions. A 6-storied MRF constructed in Satara (IS: 1893, Zone 3) was analyzed. An exterior column-beam connection from first slab was chosen for assessment. Design for seismic requirements was carried out referring to suggestions from latest revisions of IS :1893 and IS: 13920. Four 0.3:1 scaled down specimen from actually site sourced concrete and steel were constructed. Out of the four, two were detailed as per actual site practice. Two specimen were detailed as per IS:13920. Specimen were subjected to reverse cyclic displacement loading protocol. It was observed that latest revisions from IS codes ensure that beam fails prior to the connection. Overall performance characteristics were seen improved in case of ductile detailed connections.***

Keywords: ***Exterior Column beam Connection, Ductile detailing, Ductility factor Experiment, IS:13920, Non-ductile detailing Reverse Cyclic loading, Scaled model, Site conditions, Stiffness degradation.***

I. INTRODUCTION

Deficiencies in the construction of structures are usually not known until they are tragically exposed by unfortunate testing events like earthquakes. Forensic investigation of past seismic events worldwide has revealed the poor performance of ill detailed structures and lack of "Capacity Design" principles. Moment resisting frames (SMRF) built in India are by-n-large vulnerable to lateral actions leading to brittle collapse. Column beam joints are subjected to large inelastic deformations when subjected to Severe reverse cyclic displacements during earthquakes. Ill-detailed joints significantly jeopardize response of frames designed as SMR frames. In order to reduce congestion, the detailing of reinforcement around connection region is more vital.

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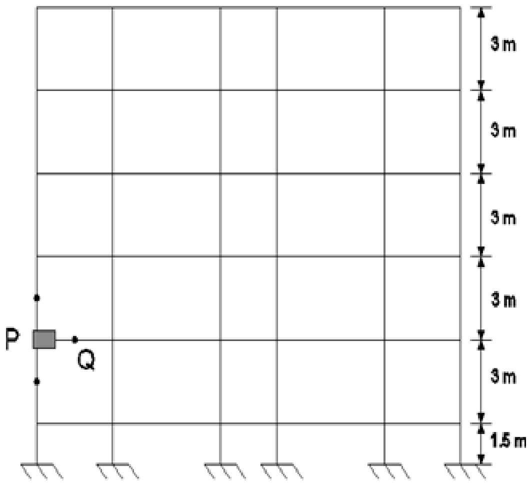
It is assumed during the analysis, that connections are perfectly rigid and are capable of transferring stresses from one structural element to other without failure. How-ever severe shear stresses developed during earthquake loads, tend to falsify this assumption regarding connection rigidity [9]. Recent earthquakes have revealed that failure of connection leads to catastrophic failure even if members such as beam and column are intact. It was also observed that exterior joints are most vulnerable for failure. Therefore, ensuring ductility as well as resilience of exterior connections is very important in the seismic design of SMRF [2]. Effect of poor design and constructional ill-detailing of connections is multiplied by excessive deformation demands imposed by connecting members during earthquakes. Resilience of members is derived through their capacity to undergo inelastic deformations without undergoing failure. Ill-designing and detailing of connections leads to impounding failure even if connecting members are properly designed and detailed. Experiments on column beam assemblies reveal that prevalent design and detailing practice on site leads to extensive deterioration of connections.

experimental assessment of interior joints subjected to various parameters like axial load, percentage of confining reinforcement, characteristic strength of concrete, effect of lateral ties, with respect to strength of connections was done by [6]. Similarly, three interior column beam connections with column loads varied between 10%, 5%, and 0% of its axial load capacities were tested by [7]. Results revealed that there was 22 % reduction in ultimate displacement when axial load was decreased from 10 % to 0 %. The prime requirement from the connection is that the plastic hinge shall be formed in the beam prior to connection or the column. Thus flexural strength of column should be more than that of beam to ensure formation of plastic hinge in beam. Moreover, connection should possess sufficient shear strength so that it does not fail prior to column or beam. Confining reinforcement in connection works as effective shear reinforcement, confining joint region and restraining diagonal cracks. Experiments have revealed that use of rectangular confining reinforcement significantly improves resilience of external column beam connections [18]. Hoop reinforcement and anchorage requirements as per code often lead to reinforcement congestion in and around the joint. More-over, the development length requirements ask for larger member sizes or smaller diameter bars, making the situation even worse. There are many reasons why code provisions are not being followed strictly by the construction industry as such, but practicality of the detailing requirements is one of them. It is interesting to note that, in spite of following all the code provisions, the intense diagonal cracking around joint cannot be prevented. Studying effect of axial load on connection capacity is not in purview of this study, how-ever it is worthwhile to note that increased axial loads increase shear capacity of the joint. Axial load reduces principal stresses induced in connection region and thus improving the shear capacity of the connection [1].

This work is an attempt to compare performance of ill detailed and ductile detailed connections, especially constructed as per site conditions.



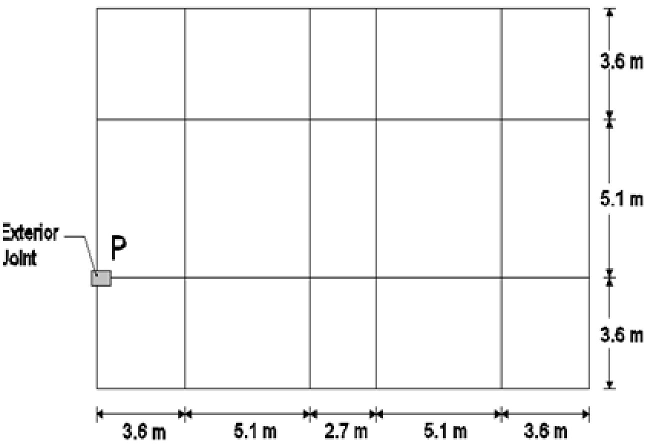


Fig. 1: Details of the frame analyzed

II. EVALUATION OF COLUMN BEAM CONNECTION

Length of columns is 3 m. Dimensions adopted were 600x300 mm. Beams are 3.6 m long with 300x600 mm size as per design. Live load was assumed to be 3 kN/m2 along with finishing load of 1 kN/m2. Wall thickness was assumed to be 230 mm. Grade of concrete assumed was M20 and grade of Steel used was Fe 415. Plane frame analysis was done for the frame shown in Fig.1. IS 456, IS 13920 and IS 1893 were referred to for design and ductile detailing of transverse reinforcement.

III. DESIGN OF BEAM COLUMN ASSEMBLY

A. Reinforcement details

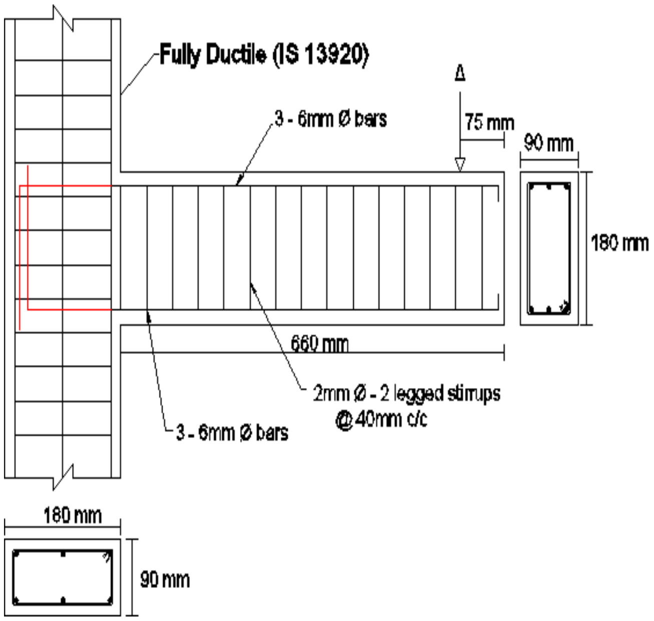
A 6-storied SMRF in Satara (zone 3) resting on class II soil was analyzed. Design parameters like axial load, bending moment and shear forces around selected connection were calculated. The joint marked "P as shown in Fig. 1 was selected for design. Design SF and BM for critical load combinations for beam PQ were 153.99 kN, 137.74 kN-m respectively. Shear reinforcement for beam PQ was calculated and detailed based on relevant codes and is shown in Table 1.It was practically not possible to construct and test, full sized assembly as per design. Hence it was decided to proportionately scale down the designed connection. Designed connection was scaled down to 0.3:1 of its size, as illustrated in Table 2. The details of scaled down specimen, reinforcement in connection and connecting members are depicted in Fig. 2 and Table 2. Specimens in Group 1 were cast with reinforcement ill- detailed as per site condition (HD 1,2). Second Group was detailed as per IS:13920 (FD 1,2). All specimens were subjected to cyclic displacement at the tip of beam as shown in Fig2, until failure.

Table 1: Reinforcement: Full size assembly

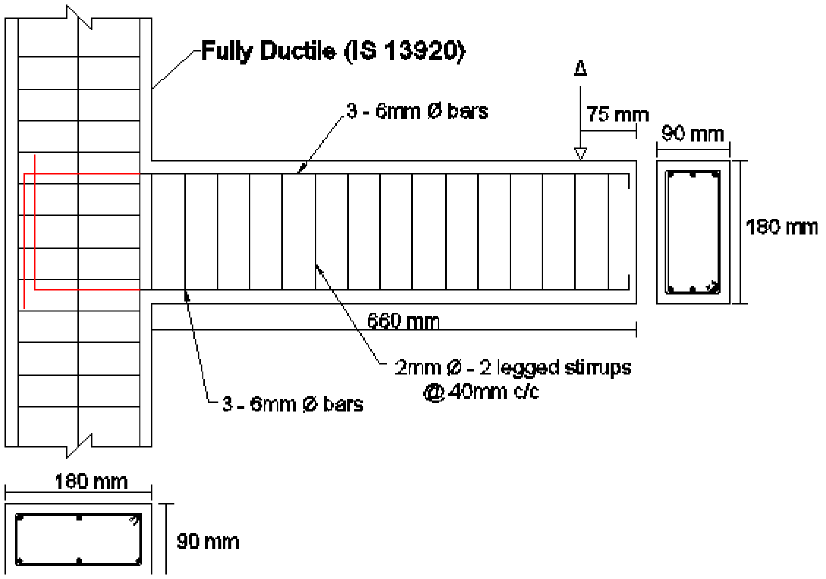
Column		Beam		Joint	Remarks
Longitudinal Reinforcement	Transverse Reinforcement	Longitudinal Reinforcement	Transverse Reinforcement	Transverse Reinforcement	
6 φ 20mm	φ 8mm @ 120.0 mm c/c	3 φ 20 mm (top and bottom)	φ 8mm @ 120.0 mm c/c	φ 8mm @ 120.0 mm c/c	as per IS: 13920
6 φ 20mm	φ 8mm @ 120.0 mm c/c	3 φ 20 mm (top and bottom)	φ 8mm @ 120.0 mm c/c	φ 8mm @ 120.0 mm c/c	as per built at site

Table 2: Reinforcement: Scaled Down Specimen

Specimen	Column		Beam		Joint	Remarks
	Longitudinal Reinforcement	Transverse Reinforcement	Longitudinal Reinforcement	Transverse Reinforcement	Transverse Reinforcement	
FD_1	6 φ 6mm[#]	φ 2mm @ 40mm c/c	3 φ 6mm (top and bottom)	φ 2mm @ 40mm c/c	φ 2mm @ 40mm c/c	Confining reinforcement as per IS: 13920
FD_2	6 φ 6mm	φ 2mm @ 40mm c/c	3 φ 6mm (top and bottom)	φ 2mm @ 40mm c/c	φ 2mm @ 40mm c/c	Confining reinforcement as per IS: 13920
HD_1	6 φ 6mm	φ 2mm @ 40mm c/c	3 φ 6mm (top and bottom)	φ 2mm @ 40mm c/c	φ 2mm @ 40mm c/c	Confining reinforcement as per built at site
HD_2	6 φ 6mm	φ 2mm @ 40mm c/c	3 φ 6mm (top and bottom)	φ 2mm @ 40mm c/c	φ 2mm @ 40mm c/c	Confining reinforcement as per built at site



(a).Reinf. details as per site condition



(b) Reinf. details as per IS 13920

Fig. 2: Scaled down Specimen.

IV MAUFACTURE OF SPECIMEN

A.Formwork and Casting

Wooden molds were prepared as shown in Fig. 3(b). Reinforcement was detailed as shown in Fig 3(a). Loading and end regions of reinforcement cage were strengthened in order to avoid edge disturbances. Casting was done using OPC (53 grade) cement. Crush sand (medium type) as available at site along with coarse aggregate as available at site was used for preparation of concrete. Site supervisor was instructed to prepare mix for M20 concrete as per their standard site practice. It was observed that water-cement ratio was used as 0.5 as per site conditions.150mm and 90 mm cubes were cast to ascertain characteristic strength of obtained mix. The 28-day average compressive strengths from 150 mm and 90 mm cube test was found out to be 24.62 N/mm2 and 28.11 N/mm2 respectively. Average yield stress of reinforcement tested was 432 N/mm2. All the specimens were cast either in horizontal or vertical position on the same day. Specimens were de-shuttered after 24 hours and cured at site along with other RCC elements at site conditions.

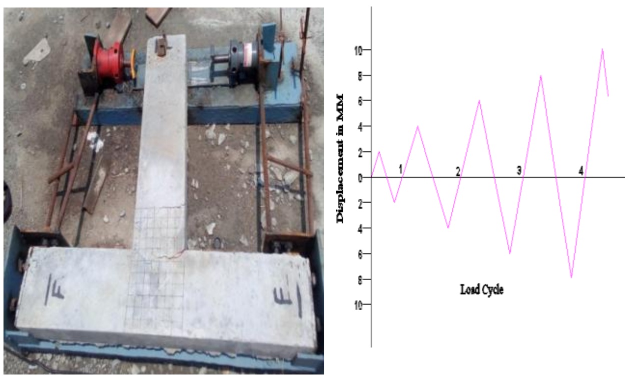
(a) Reinforcement details (b) Form work for casting

Fig. 3: Reinforcement and Setup for casting

B. Test procedure

Specimen was kept horizontal on specially fabricated load frame. The hand operated mechanical jack, load cell and digital venire caliper were used for testing. Each connection specimen was tested under reversed cyclic displacement loading. Both ends of the columns were fixed. The beam was loaded at tip as shown in Fig. 2 and as per displacement loading protocol as shown in Fig. 4(b).The connection region was marked with 25 mm x 25 mm grid to understand extent of cracking in the region.

Seismic capacity of RC element is measured in terms of Ductility i.e. capacity to undergo inelastic deformation without undergoing failure, strength to with stand loadings in elastic region and the resilience. Resilience is the capacity to dissipate energy without failure. With the increase in number of cycles and increase in amplitude of cycles, it was expected that all these performance markers will tend to decrease. [1],[18]. It was decided to load the connection to its ultimate capacity. Thus the displacement cycle was chosen to increase in multiples of 2 mm. The specimen was first displaced up to (+)2 mm and then in the reverse direction up to (-)2 mm. The amplitude of subsequent cycles was increased in multiples of 2 mm. To record the loads precisely, load cell with the least count of 1kg was used. The setup was instrumented with a digital Vernier with the least count of 0.1 mm to measure the deflection at the loading point. The readings were recorded in tabular form and various graphs were plotted with the use of spreadsheets.

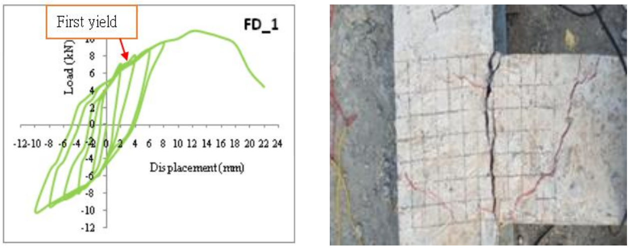


(a)Test Setup (b) Loading protocol

Fig. 4.

V. OBSERVATIONS AND REMARKS

A. FD 1: Specimen with bars detailed as per IS:1392

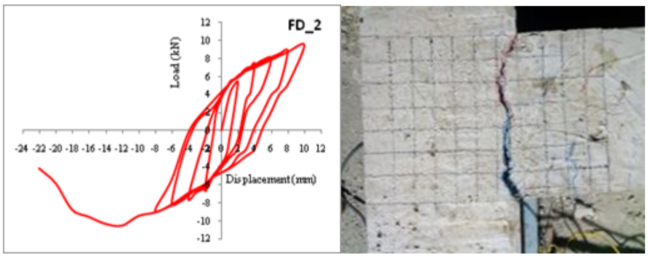


(a).Hysteresis loop of FD 1 (b) Failure pattern of FD 1

Fig. 5: Observations for FD 1

Fig. 5 (a) shows Hysteresis loop of FD 1 and Fig.5 (b) shows failure pattern of FD 1. First shear crack developed during first cycle of loading. Yield load for positive and negative direction was found to be 7.01 kN and 7.89 kN respectively. Ultimate load for positive direction was 10kN and for negative direction was 10.88kN. After yield load plastic deformation started. Specimen failed by vertical crack at connection. Also cracks in beam region were seen. How-ever it was noted that no crack formation occurred in joint region. A vertical cleavage can be clearly seen in Fig.5(b).

B. FD 2: Specimen with bars detailed as per IS:13920



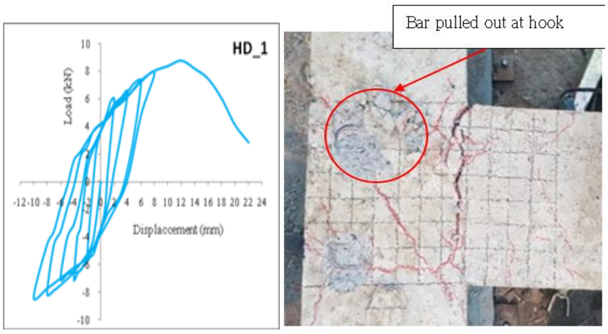
(a)Hysteresis loop of FD 2 (b) Failure pattern of FD 2

Fig. 6: Observations for FD 2

Fig 6. (a) shows Hysteresis loop of FD 2 and Fig.6 (b) shows failure pattern of FD 2. Yield load of positive and negative direction was 6.67 kN and 7.45 kN respectively. Ultimate load in positive direction was 9.46 kN and in negative direction was 10.59 kN After yield load, Specimen failed by vertical cracks at connection. There were no cracks in the connection region. Also, no spalling or cracking was observed around beam hooked bar region. So there was no slip between rebar and concrete.

C. HD 1 : Specimen with bars detailed as per site condition.

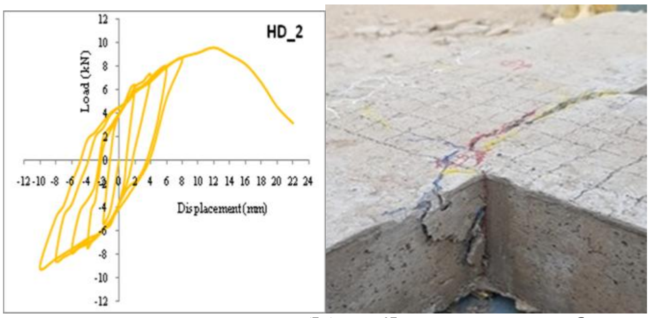
Fig. 7.(a) shows Hysteresis loop of HD 1 and Fig. 7(b) shows failure pattern for HD1. Yield load in positive and negative direction was 6.03 kN and 7.06 kN. respectively. Ultimate load in positive direction was 8.33 kN and in negative direction was 8.77 kN. progressive diagonal cracks were seen to develop in connection region. Concrete around hooked region of bar was seen cracking and spalling due to slip of bar. This was due to inadequate development length of hooked bar. Final failure was due to vertical separation at beam column interface



(a) Hysteresis loop of HD 1 (b) Failure Pattern of HD1

Fig. 7: Observations for HD1 .

D. HD 2: Specimen with bars detailed as per site condition.



(a)Hysteresis loop of HD (b) Failure pattern of HD2

Fig. 8: Observations for HD 2

Fig.8(a) shows Hysteresis loop of HD 2 and Fig. 8(b)shows failure pattern of HD 2. Yield load in positive and negative direction was 6.33 kN and 7.4 kN respectively. Ultimate load in positive direction was 9.12 kN and negative direction was 9.56 kN. Diagonal connection cracks and vertical cleavage at beam column interface were typical for this ill detailed specimen also.

VI. RESULT ANALYSIS

A. Hysteresis Loops

The displacement hysteresis loops for the various specimens are shown in Fig 5 thro 8. It can be observed from Table 3 that ultimate LCC (Load carrying capacity) for ductile detailed joints was more than those for ill detailed connections.

Table 3. Ultimate and Yield LCC

Specimen	Displacement (mm)			
	Yield Load		Ultimate Load	
	Positive Direction	Negative Direction	Positive Direction	Negative Direction
FD_1	7.01	7.89	10	10.88
FD_2	6.67	7.45	9.46	10.59
HD_1	6.03	7.06	8.33	8.77
HD_2	6.33	7.4	9.12	9.56

Energy dissipated by the specimen during a particular cycle is calculated from area under load displacement curve for that cycle.

The area under any loop can be calculated with the help of spread sheets.

Based on trapezoidal rule the area under curve is divided in multiple trapezoids of small widths. Total area under the curve can be calculated by a macro developed in spreadsheet as summation of all strip areas under the loop. Table 4 shows the step-wise and cumulative energy dissipation for all specimens. The highest energy dissipated was 92.77 kN-mm for Specimen FD1. Average Cumulative energy dissipation of FD specimen was 10.95% more than HD.

B. Energy Dissipation

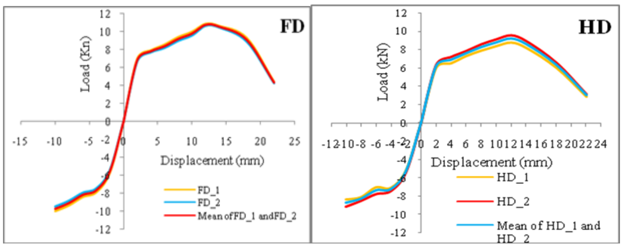
Energy dissipated by the specimen during a particular cycle is calculated from area under load displacement curve for that cycle. The area under any loop can be calculated with the help of spread sheets. Based on trapezoidal rule the area under curve is divided in multiple trapezoids of small widths. Total area under the curve can be calculated by a macro developed in spreadsheet as summation of all strip areas under the loop. Table 4 shows the step-wise and cumulative energy dissipation for all specimens. The highest energy dissipated was 92.77 kN-mm for Specimen FD1. Average Cumulative energy dissipation of FD specimen was 10.95% more than HD.

Table 4. Energy Dissipation

Specimen	Energy Dissipation/cycle					Cumulative Energy Dissipation	Mean	% rise
	Cycle_1	Cycle_2	Cycle_3	Cycle_4	Cycle_5			
FD_1	10.58	16.02	19.30	28.39	20.84	95.13	92.77	10.95
FD_2	10.27	15.25	18.73	26.38	19.79	90.42		
HD_1	9.76	13.63	16.30	24.00	16.73	80.42	83.62	
HD_2	10.17	14.76	17.68	25.84	18.36	86.82		

C. Envelope

Envelope is locus joining peaks of all cycles. Yield load and ultimate load can be observed from envelope. Fig 9(a) and (b) show envelope of specimen FD and HD. Peak and yield loads are mentioned in Table No. 3.

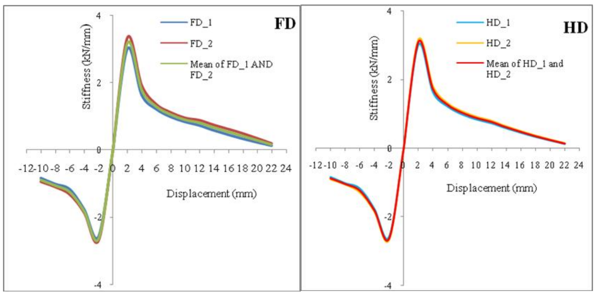


(a)Envelopes for FD (b) Envelopes for HD

Fig. 9: Envelopes for FD* and HD*

D. Stiffness degradation

Stiffness is load required to create unit deformation at the tip of beam. Slope of line joining peak of each cycle to origin is stiffness for that cycle. Stiffness thus calculated for each cycle is shown in Fig 10(a) and (b). It was observed that stiffness was highest for FD* (ductile detailed) specimens as compared to HD* (ill-detailed) specimens. Also stiffness degradation was comparatively more steep for HD specimens compared to FD specimens.



(a)FD Specimen (b) HD Specimen

Fig. 10. Stiffness degradation

E. Displacement Ductility

Ratio of Ultimate load to yield load is termed as displacement ductility. Table 5. depicts ductility ratios for FD* and HD* specimen. Mean ductility of FD and HD specimens was 1.42 and 1.34 respectively. Displacement ductility of FD specimen was 5.57% more as compared to those for HD specimens. Displacement ductility is measure of inelastic deformation capacity of the specimen.

Table 5. Displacement Ductility

Specimen	Displacement (mm)				Displacement Ductility		Average Ductility	Mean Ductility	% rise
	Yield		Ultimate						
	Positive Direction	Negative Direction	Positive Direction	Negative Direction	Positive Direction	Negative Direction			
FD_1	7.01	7.89	10	10.88	1.27	1.55	1.41	1.42	5.6
FD_2	6.67	7.45	9.46	10.59	1.59	1.27	1.43		
HD_1	6.03	7.06	8.33	8.77	1.18	1.45	1.32	1.34	
HD_2	6.33	7.4	9.12	9.56	1.23	1.51	1.37		

F. Joint Shear Stress

Capacity of a connection to resist horizontal shear stress is given by (1)[2].

$$\tau_{jh} = \frac{P}{A^{h}_{core}}\left[\frac{L_b}{d_b} - \frac{L_b + 0.5D_c}{L_c}\right] \quad \text{----------} \ (1)$$

τ_{ACI} is calculated as $0.083*\gamma*\sqrt{fc}$ Mpa, where fc is compressive strength in MPa and γ is 15 for exterior joints. Joint shear capacity of FD specimen was more than HD specimen by about 14.39% . Table 6 shows Joint shear stress for all specimens.

Table 6: Joint Shear Stress

Specimen	Positive Ultimate Load P_u kN	τ_{jh}	τ_{jh} / τ_{ACI}	Negative Ultimate Load P_u kN	τ_{jh}	τ_{jh} / τ_{ACI}	Avg. Shear Stress	Mean	% rise
FD_1	10	3.47	0.60	10.88	3.78	0.65	3.63	3.55	14.39
FD_2	9.46	3.68	0.63	10.59	3.29	0.57	3.48		
HD_1	8.33	2.89	0.50	8.77	3.05	0.53	2.97	3.11	
HD_2	9.12	3.17	0.55	9.56	3.32	0.57	3.24		

VII. CONCLUSIONS

Aim of this exercise was to evaluate performance of exterior connections as constructed at site conditions. Two discrete sets of connections were manufactured. One designed and ductile detailed as per the IS 13920 and other ill detailed as constructed at site.

All specimens ultimately failed due to the cracks at interface of beam and column.

– The connection region was free from cracks except for some hairline cracks in case of connections detailed as per IS 13920.

– Specimens having special confining reinforcement as per IS 13920 had improved resilience as compared to those with ill detailed reinforcement.

– Diagonal cracks were seen in joint region of ill detailed specimen indicating inadequate shear and rotation capacity of the joint.

– Concrete spalling and cracking was seen around hooked bar region indicating slip of reinforcement due to inadequate anchorage in ill detailed specimen.

– It was concluded from the observations that overall energy dissipation and ultimate load characteristics of a connection improves substantially due to ductile detailing of reinforcement as per IS 13920.

– Ill detailed connections as built at site have inadequate capacity to act as a rigid interfacing element for beam and columns.

– Ductile detailed connections have better inelastic deformation capacities and hence would perform better during seismic activity.

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