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Performance of exterior precast concrete beam-column dowel connections under cyclic loading

R. Vidjeapriya[1], V.Vasanthalakshmi[2], K.P.Jaya[3,*]

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Abstract

The present study focuses on the performance of precast concrete beam-column dowel connections subjected to cyclic loading by conducting experiments. In this study, one-third scale model of two types of precast and a monolithic beam-column connection were cast and tested under reverse cyclic loading. The precast connections considered for this study is a beam-column connection where beam is connected to column with corbel using (i) dowel bar and (ii) dowel bar with cleat angle. The experimental results of the precast specimens have been compared with that of the reference monolithic connection. The sub-assemblage specimens have been subjected to reverse cyclic displacement-controlled lateral loading applied at the end of the beam. The performance of the precast connections in terms of the ultimate load carrying capacity, post- elastic strength enhancement factor, load-displacement hysteresis behaviour, moment-rotation hysteresis behaviour, energy dissipation capacity, equivalent viscous damping ratio and ductility factor were compared with that of the monolithic beam-column connection. The monolithic specimen was found to perform better when compared to the precast specimens in terms of strength and energy dissipation. In terms of ductility, the precast specimen using dowel bar and cleat angle showed better behaviour when compared to the reference monolithic specimen.

Keywords: *Beam-column connection, Precast concrete, Cyclic loading, Dowel bar, Cleat angle.*

1. Introduction

In the recent years, there is an enormous infrastructural growth in India. The rapid infrastructural growth together with increasing demand for quality buildings necessitate the construction industry to continuously seek for improvement. This will lead to industrialization in the building industry, which can be achieved in the form of precast concrete construction. In the International arena precast concrete sector has experienced reasonable growth in the recent years. This is because precast concrete provides high-quality structural elements, construction efficiency, and savings in time and overall cost of investment. The other advantages include reduced requirement of formwork, scaffolding and less reliance on wet concrete. Also the time taken for construction at the site is up to 20% less for precast concrete structure when compared to the construction of a similar cast-in-situ concrete structure [1].

* *Corresponding author: kpjaya@nayan.co.in*

1 Assistant Professor (Sr.), Department of Civil Engineering, Anna University, Chennai – 600025, India

2 Post Graduate Student, Department of Civil Engineering, Anna University, Chennai – 600025, India

3 Associate Professor, Department of Civil Engineering, Anna University, Chennai – 600025, India

Though it has many advantages over the cast-in-situ concrete construction, still there is hesitancy in extensively constructing precast concrete structures in highly seismic areas.

It can be observed that many precast concrete structures have failed in the past during earthquakes. Failure of the structures in these earthquakes was mainly due to the poor connections between the precast elements themselves and between the precast elements and lateral load-resisting system. Hence, there is a necessity to carry out more research in this area. For the past four decades though a lot of research has been carried out on the behavior of precast structures, a complete understanding of the behaviour of precast beam-column connections to dynamic loading has not been completely understood.

In India, most of the construction utilises cast-in-situ technique. Precast concrete construction is still in its early stage. Generally, precast concrete construction is more preferred for construction of large span bridges. As India is a fast developing country, there is a large scope for improvement in the construction sector, especially towards development and utilization of factory made quality controlled precast units that provides for faster construction leading to economy.

In general, precast concrete structural systems displaying non-linear response characteristics can be broadly classified into two main categories as Equivalent Monolithic Systems and Jointed Systems. In the equivalent monolithic systems, the structural systems are designed to emulate the cast-in-situ reinforced concrete construction. The jointed systems are systems where precast elements are separated from each other but are connected with special jointing details like welded or bolted plates [2]. For the present study, precast concrete with dry mechanical connections were considered. The behavior of two types of precast beam-column dowel connections and a reference monolithic connection were investigated.

2. Literature Survey

Several Studies were conducted to evaluate the performance of precast beam-column moment resisting frames under cyclic loading. Ochs and Ehsani tested five precast beam to column subassemblies under simulated earthquake type loading [3]. The columns included steel plates or angles embedded in the columns and beams. It was concluded that precast concrete specimens performed similarly to that of monolithically cast concrete connection. The precast column was strong enough to force a plastic hinge away from the column face. The critical part of the precast connections was the welded beam bars as they initiated the failure of specimens.

Loo and Yao conducted experimental investigations on eighteen half scale interior connection models to evaluate their strength and ductility properties under static and repeated loading [4]. It was concluded that under both static and repeated loading, the precast connections attained a higher flexural strength and larger energy absorbing capacities than monolithic connections.

Khaloo and Parastesh carried out an experimental study to investigate a simple moment-resisting precast concrete beam-column connection under cyclic inelastic loading [5]. The variables examined were the connection length of reinforcements and presence of transverse bars at mid height of connection. It was concluded that the reduction in connection length reduced strength, ductility and energy absorption. The failure mode changed toward partial separation and slippage of bond between the precast concrete beam and the cast-in-place grout. The presence of transverse bars in the connection length enhanced the seismic behavior of the precast connection system.

Chun et al assessed the effectiveness of headed bars terminating in exterior beam-column joints under reversed cyclic loading [6]. The primary test parameters were the anchorage type, size and arrangement of the beam bars and the heads and the detailing provided for roof joints. The test results indicated that hysteretic behaviour of exterior joints constructed with headed bars was similar or superior to joints constructed and tested with hooked bars. It was also concluded that in addition to providing vertical U-bars at roof joints, heads on column bars should extend beyond the beam top bars to provide improved behavior.

Li et al conducted experimental and analytical investigations of hybrid-steel concrete connections under cyclic load reversals [7]. The precast specimen's performance was good at exhibiting adequate ductile behavior under seismic loading and it also agreed well with cast-in-place specimen. Embedment of the steel sections in the joint greatly enhanced the strength of the joint core with the specimens carrying storey shears up to a ductility factor of 3.5.

Xue and Yang studied the behavior of precast concrete connections in a moment resisting frame under cyclic loading [8]. The connections studied were exterior connection, interior connection, T connection and knee connection. It was observed that Knee connections were less effective when compared to other connections. All the connections exhibited strong column - weak beam failure mechanism. It was concluded that all the connections performed satisfactorily in seismic conditions with respect to strength, ductility and energy dissipation capacity.

Novelty of the approach

From the literature, it is observed that the precast connections can be detailed as strong as that of the monolithic connections. It can also be widely observed in the literature that a lot of research on precast structures emulating the behaviour of reinforced concrete cast-in-situ seismic resistant frames had been carried out. However limited research work has been carried out on jointed systems for use in seismic regions. Hence, the present study aims at developing dry connections using dowel bars and cleat angle. It is also aimed to replace the bearing pad with iso-resin grout. The main focus of the work is to develop optimum dry connection for the low rise moment resisting frames.

3. Objective of Present Study

The objective of the present study is,

- To identify a simple and suitable dry precast beam-column connection for an exterior beam-column joint of a three storey moment resisting framed structure.
- To conduct experimental investigations on two types of precast connections and a reference monolithic connection.
- To compare the behaviour of precast beam-column connections with that of the monolithic connection.
- To identify the most suited connection for the precast elements.

4. Experimental Programme

For the present study an exterior beam-column joint of a three storey reinforced concrete building was considered. Six test specimens of 1/3 rd scale model were cast and tested under reverse cyclic loading.

4.1. Material testing

The 28th day average cube compressive strength of the concrete (f_{cu}) was 41.6 MPa. The cylinder compressive strength has been evaluated based on the relationship, f_c' =0.8 f_{cu} and was observed as 33.28 MPa. The split tensile strength of the concrete had been observed as 3.06 MPa by testing three cylinders of 150 mm diameter and 300 mm height. Three beams of 100 mm x100 mm x500 mm were cast and tested for the flexural strength. The flexural strength of the concrete was 5.95 MPa.

4.2. Design and detailing of specimens

For this study a typical exterior beam-column connection of a three-storey reinforced concrete building has been analyzed and considered. Figure 1 shows the plan and elevation of the building showing the exterior beam-column joint considered (joint A). For the various load combinations, shear forces, bending moments and axial forces in the exterior beam-column joint A in the first floor had been calculated. Seismic analysis had been performed using equivalent lateral force method recommended by IS: 1893-2002 [9]. The design and detailing of beam, column and exterior joint had been done based on the guidelines given in IS: 456-2000 [10] and IS: 13920-1993 [11], respectively. One-third scaled models had been developed for monolithic and precast specimens with cross-sectional dimensions 100 mm x 100 mm for both beam and column. The clear span of the beam was 550 mm. The height of the column was 1200 mm. The cover thickness of monolithic and the two precast beam and column specimens were 10 mm. Figure 2 (a) and 2(b) shows reinforcement detailing of specimen ML and precast specimens PC-DW and PC-DWCL respectively.

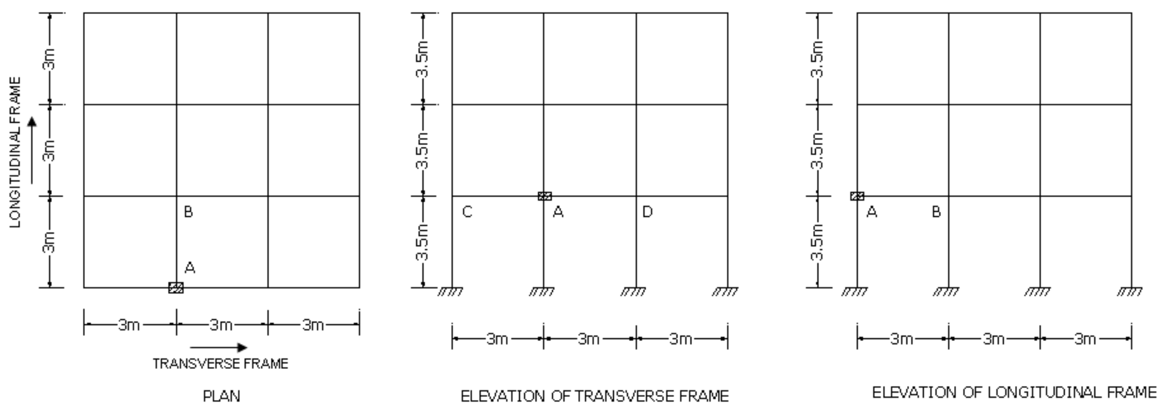
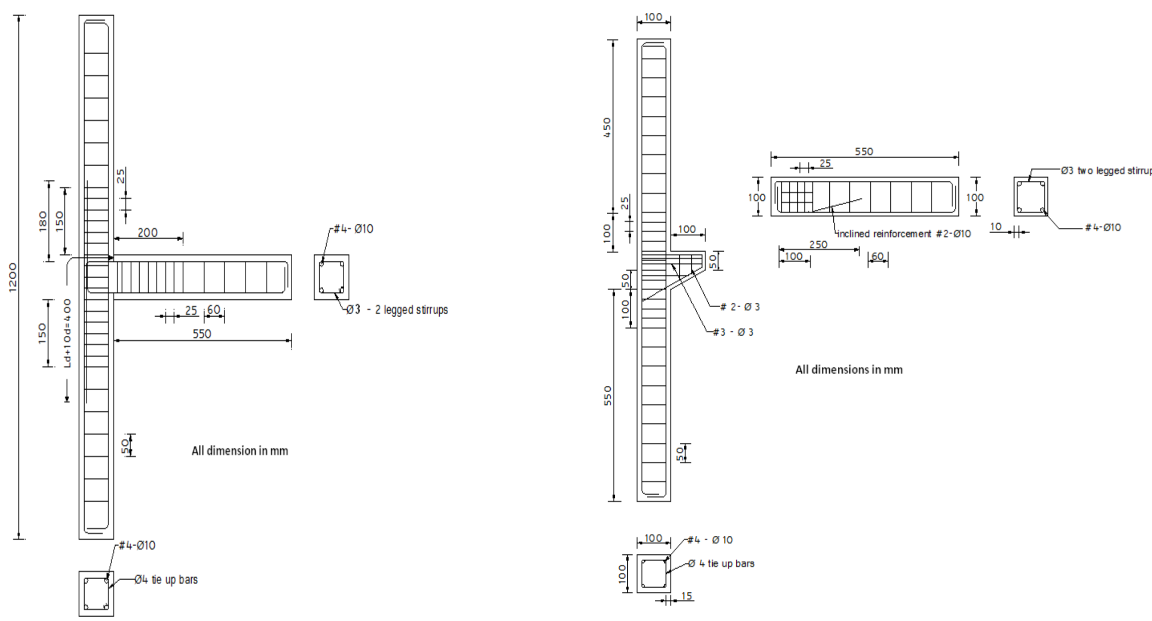


Fig. 1 Plan and elevation of the building



a) Specimen ML b) Specimen PC-DW and PC-DWCL

Fig. 2 Reinforcement detailing of specimen PC-DW and PC-DWCL

4.3. Test specimen and connection details

4.3.1. Monolithic connection (ML)

The monolithic reinforced concrete test specimen (ML) was designed according to IS:456-2000 and detailed according to IS:13920-1993. Two bars of 10 mm diameter each were provided as tension reinforcement and two bars of 10 mm diameter each were provided as compression reinforcement. The shear reinforcement consisted of 3 mm diameter two legged stirrups spaced at 60 mm. For a distance of 100 mm from the column face the spacing of the lateral ties were decreased to 25 mm. The column reinforcement arrangement also consisted of four 10 mm diameter bars. Along the column height excluding the joint region, the lateral ties were spaced at 50 mm. At the joint region the spacing of the lateral ties were reduced to 25 mm.

4.3.2. Precast concrete dowel connections

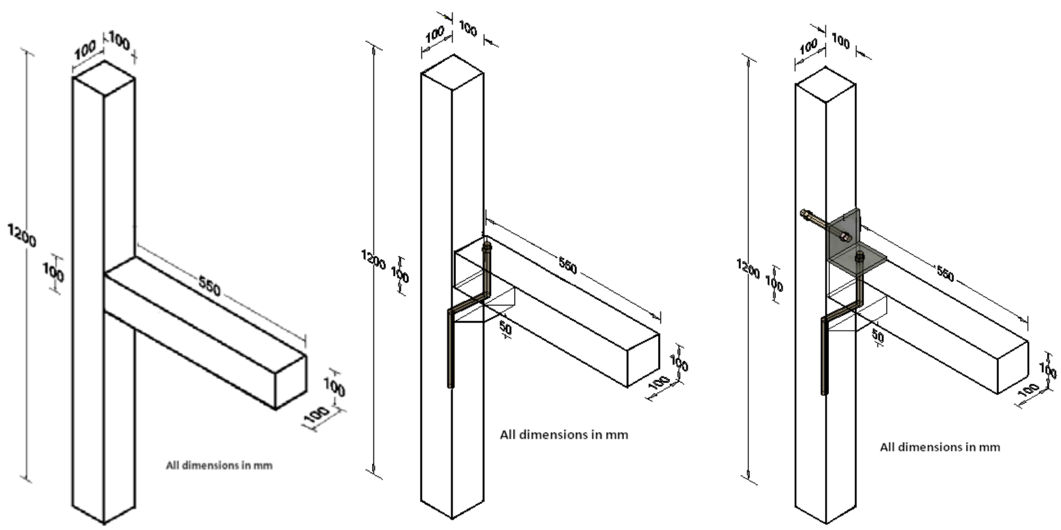
The precast concrete elements were designed according to IS: 456-2000 and detailed according to IS: 13920-1993. The design of cleat angle and bolts which were the connecting elements in the precast concrete connections were designed according to IS: 800-2007 [12]. Additional horizontal stirrups were provided for a distance 100 mm to cater to the confinement of concrete at region of the bolt hole in the beam. For the precast beam column connection, two mechanical connections were considered for this study as detailed as follows

4.3.2.1. Precast connection using dowel bar (PC-DW)

In this connection the beam was supported on concrete corbel using a dowel bar. The dowel bar of 16 mm diameter was embedded in the column to a length equal to the development length and cast with the bar projecting from the corbel. The precast beam with 21 mm diameter sleeve hole which was cast inside the beam was inserted into the projecting dowel bar. The gap between the dowel bar and the hole was filled with iso-resin grouts.

4.3.2.2. Precast connection using dowel bar and cleat angle (PC-DWCL)

In this connection the beam was supported on concrete corbel using a dowel bar and cleat angle. The dowel bar is of 16 mm diameter and cast in the column in a similar fashion as that of precast connection PC-DW. The cleat angle used for the connection was ISA 100x100x10. A sleeve of 21 mm diameter was cast inside the column and beam to facilitate the connectivity between precast elements. A part of the dowel was projecting outside the corbel for connection with the beam using cleat angle and nuts. A bolt of 16 mm diameter of grade 4.6 was used to connect the cleat angle and the column. The gap between the dowel bar and the groove was filled with iso-resin grouts. Figure 3(a), 3(b) and 3(c) shows the schematic representation of the isometric view of specimens ML, PC-DW and PC-DWCL respectively.



a) Specimen ML b) Specimen PC-DW c) Specimen PC-DWCL

Fig. 3 Isometric views of a) Specimen ML, b) Specimen PC-DW and c) Specimen PC-DWCL

4.4. Construction of sub-assemblages

The precast test specimens were cast in two stages. First the beam and column were cast in the horizontal position. Then the beam and column were assembled together using mechanical connecting elements such as dowel bar, cleat angle and bolts.

4.5. Test and loading setup

An experimental programme consisting of six beam–column connection subassemblies under reverse cyclic loading was conducted to evaluate the seismic behaviour of two types of precast connections and a reference monolithic specimen. The boundary conditions for the test specimens were chosen in order to model actual conditions where the moments were approximately zero at mid-span of column when subjected to lateral loading. The lower end of the column for both the monolithic and precast specimen was connected to the strong reaction floor by a hinge support while the upper end of the column was free to move and rotate by a roller support. The schematic representation of the test setup used for testing the monolithic and two precast connections is shown in Figure 4. A loading frame of capacity 2000 kN was used for conducting the experiments. Axial load was applied at the upper end of the column by a 400 kN capacity hydraulic jack. The axial load was applied to simulate the gravity load on the column. The vertical displacement of the beam was measured by three dial gauges which were placed at a distance of 100 mm, 200 mm and 425 mm from the face of the column.

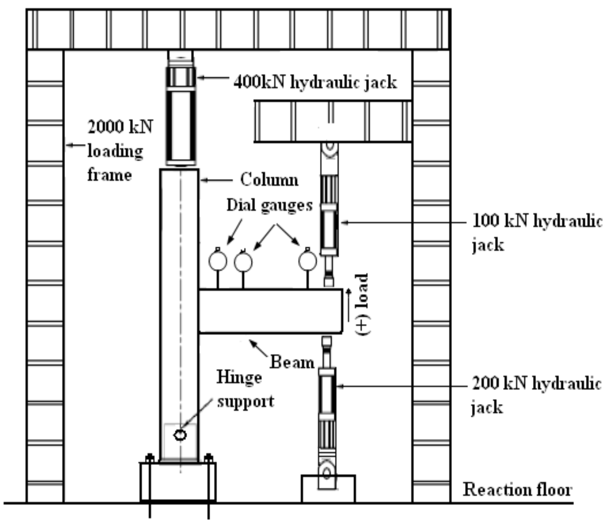


Fig. 4 Schematic Experimental Test Setup

4.6. Loading history

The loading history consists of displacement cycles as shown in Table 1 and Figure 5.

Table 1 Displacement sequence for the displacement based loading of the specimens

Sl.No.	Displacement (mm)		Drift (%)			
	Start	End	Start	End	Increment (mm)	Increment (%)
1	0.0	1.0	0.0	0.24	0.1	0.02
2	1.0	2.0	0.24	0.47	0.2	0.05
3	2.0	5.0	0.47	1.18	0.5	0.12
4	5.0	10.0	1.18	2.35	1.0	0.24
5	10.0	12.0	2.35	2.82	2.0	0.47
6	12.0	21.0	2.82	4.94	3.0	0.71
7	21.0	25.0	4.94	5.88	4.0	0.94
8	25.0	30.0	5.88	7.06	5.0	1.18

The loading system was displacement controlled. The drift has been calculated as the ratio of beam displacement to the length of the beam measured from the joint to the position of dial gauge. Two hydraulic jacks were mounted on top and bottom face of the beam end, respectively, to apply the cyclic loading. Three cycles were applied at each of these displacement levels. An axial load equal to $0.1f_c' A_g$ was applied to the column before starting of cyclic load and maintained throughout the test using hydraulic jack of capacity 400 kN [13].

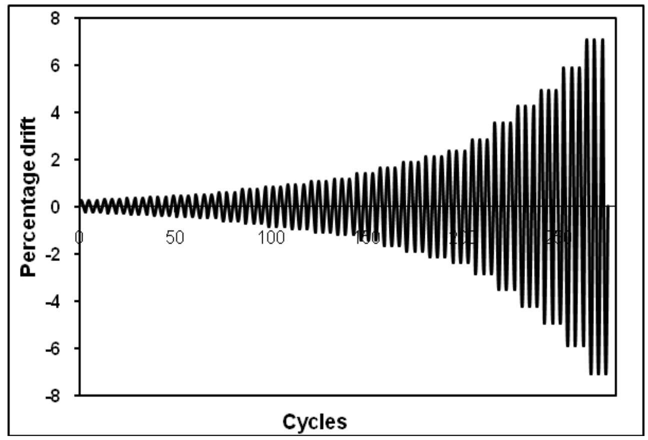


Fig. 5 Cyclic loading history

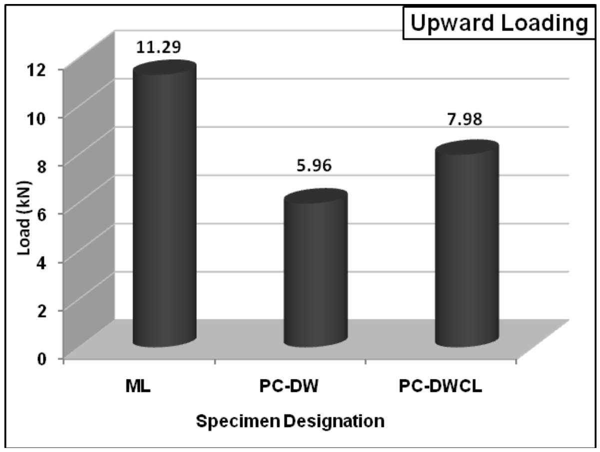
5. Test Results and Discussion

5.1. Ultimate load carrying capacity

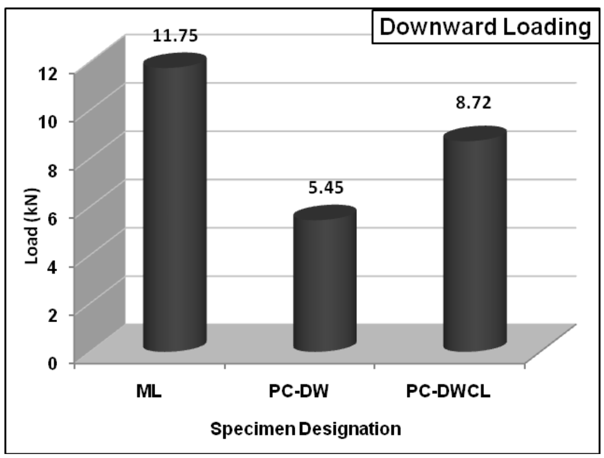
Comparison of experimental yield and ultimate loads for all the specimens is given in Table 2. The specimen with dowel bar and cleat angle PC-DWCL performed better than specimen with dowel bar PC-DW. The specimen PC-DWCL exhibited 25% and 38% greater load carrying capacity than the specimen PC-DW in the positive and negative direction respectively. The load carrying capacity of specimen PC-DW was 47% and 54% lesser than the monolithic specimen ML in the positive and negative direction respectively, whereas, specimen PC-DWCL was 29% and 26% lesser than the monolithic specimen in the positive and negative direction respectively. The ultimate load carrying capacity for specimen PC-DWCL in the positive direction was greater when compared to specimen PC-DW because more resistance was offered by the cleat angle. The ultimate load carrying capacity of the monolithic specimen was found to be 11.3 kN and 11.75 kN in the positive and negative directions respectively. Figures 6 (a) and 6 (b) compare the measured strength of the three specimens in the positive and negative direction respectively.

Table 2 Comparison of experimental yield and ultimate loads for all the specimens

S.No	Specimen	Experimental Yield Load(kN)		Experimental Ultimate Load(kN)	
		Upward direction	Downward direction	Upward direction	Downward direction
1	ML	8.80	9.40	11.29	11.75
2	PC-DW	4.90	4.18	5.96	5.45
3	PC-DWCL	6.17	7.20	7.98	8.72



a) positive direction



b) negative direction

Fig. 6 Comparison for measured strength of all specimens

5.2. Visual observations

The crack pattern development of all the specimens is given in Table 3. As the precast concrete connections had predetermined crack locations at the beam column interface because of imposed cold joints, a predetermined crack opening or closing type of response was observed in the connection region during the load cycles. In all the precast specimens, the column damage was minor which was consistent with a strong column weak beam system. Figure 7 (a) , 7 (b) and 7 (c) gives the failure mode photos of specimen ML, PC-DW and PC-DWCL respectively.

Table 3 Crack pattern development in all the specimens

Concrete component	Types of	Specimen ML	Specimen PC-DW	Specimen PC-DWCL
Beam-column junction	Shear	Crack has been initiated at 7 mm displacement cycle and cracks further propagated at	-	-

		12mm, 15mm, 18mm, ± 21 mm and 25mm displacement cycles.		
Beam	Flexure	First crack has been initiated at 2mm displacement cycle near the joint and propagated at ±2.5mm, ±3mm displa-cement cycles. Additional crack has been initiated away from the joint at 2.5 mm displacement cycle, also at ± 3mm, ± 4mm, 6mm, 7mm, -8mm, -10 mm displacement cycles.	First flexural crack occurred in the beam region at -3.5 mm displacement cycle at a distance of 12 cm from the face of the column. Also flexural cracks occurred in the beam at 5mm displacement cycle and propagated at 7mm displacement level. On the rear face, cracks started in flexural mode from the bottom of the beam at 7mm displacement cycle and started propagating in the shear mode towards the dowel.	First flexural crack occurred at beam bottom at 1.6mm displacement cycle and further propagated at 2mm and 2.5mm displacement cycles in the flexural mode.
	Shear	Crack has been initiated at -8mm,-10mm, -25mm, 30mm displacement cycle.	Crack further propagated in shear mode to the top towards the dowel at 15mm displacement cycle.	At the beam bottom, shear cracks developed at 2.5mm and propagated at 3mm displacement cycle.
Column	Shear	Crack has been initiated at ± 21 mm,25mm and -30mm displacement cycles.	Free of cracks.	Free of cracks.
Corbel	Shear	-	Cracks first occurred in the corbel at the top at -4mm displacement cycle and cracks started at the bottom at 15mm displacement cycle at the rear side.Similarly cracks occurred on the front face, at 9mm displacement cycle and propagated in shear mode at -10mm displacement cycle.	Cracks started in the corbel at -4mm and propagated at -6mm displacement cycle in shear mode.

a) Specimen ML b) Specimen PC-DW c) Specimen PC-DWCL

Fig. 7 Crack patterns of all the three specimens

5.3. Post elastic strength enhancement factor (load ratio)

Post Elastic Strength Enhancement Factor or Load ratio [14] is calculated as ratio between the average maximum load obtained during each cycle and the yield load of the specimen. The load ratio gives the development of the load carrying capacity beyond yield as well as the degree of deterioration of the three specimens. The maximum load ratio for the specimens ML, PC-DW and PC-DWCL were 1.27, 1.24 and 1.22, respectively. For the entire duration of the displacement cycles, the precast specimen with dowel PC-DW and monolithic specimen ML were able to maintain their yield loads. This trend was not observed for the specimen with dowel and cleat angle PC-DWCL. It showed a decreasing trend beyond 25 mm displacement cycle. In comparison with the precast specimens, monolithic specimen ML exhibited lesser deterioration. This behaviour was due to the good confinement of the joint core by closely spaced stirrups provided as per the guidelines of IS: 13920-1993.Though closely spaced stirrups were provided for the two precast specimens -for the beam and column element- it exhibited lesser load ratio due to the lack of confinement of the joint. Table 4 provides the comparison of Load ratio of the three specimens.

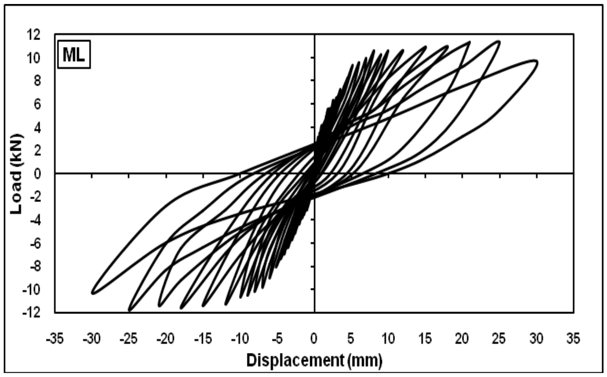
Table 4 Comparison of load ratio of the three specimens

Average of positive and negative displacement cycles (mm)	Load ratio			Average of positive and negative displacement cycles (mm)	Load ratio		
	ML	PC-DW	PC-DWCL		ML	PC-DW	PC-DWCL
1	0.4	0.43	0.47	9	1.14	1.00	1.19
2	0.54	0.48	0.75	10	1.17	0.99	1.19
3	0.67	0.54	0.87	12	1.20	1.10	1.19
4	0.79	0.75	0.92	15	1.23	1.22	1.22
5	0.93	0.83	1.01	18	1.24	1.18	1.21
6	1.02	0.86	1.09	21	1.25	1.24	1.17
7	1.08	0.95	1.13	25	1.27	1.23	1.06
8	1.14	0.98	1.16	30	1.14	1.02	0.76

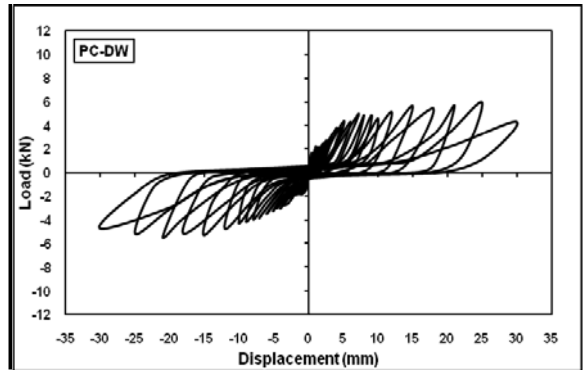
5.4. Hysteretic behaviour

The hysteretic behavior of the joint with respect to load-displacement and moment-rotation relationships has been discussed in this section. Figure 8(a), 8(b) and 8(c) gives the hysteretic load-displacement relationship for the monolithic (ML), precast specimen (PC-DW) and precast specimen (PC-DWCL) respectively. The lateral load at the beam tip and the displacement at 425 mm away from the joint (the position of the farthest dial guage) have been plotted in the load-displacement hysteretic curve. Similarly, figures 9(a), 9(b) and 9(c) gives the moment-rotation relationship of the joint subjected to cyclic loading of specimen ML, specimen PC-DW and specimen PC-DWCL respectively. The moment in the beam near the interface of the sub-assemblage and the rotation of the beam element has been plotted in the moment-rotation hysteretic curve. At the early stage of loading, the three connections exhibited a stable load versus displacement hysteretic response and then pinching had been observed in the hysteresis loops of all the three connections. Figure 8 (a) and 9(a) show the load-displacement and moment-rotation hysteretic response of the monolithic specimen (ML). These figures exhibited fat hysteresis loops with very less pinching, due to good bonding between reinforcement and joint concrete. The slight pinching was due to diagonal cracking in the joint region and flexural cracking in the beam. The areas of the hysteresis loops gradually became larger as the displacement cycle increased, which indicated good energy dissipating capacity.

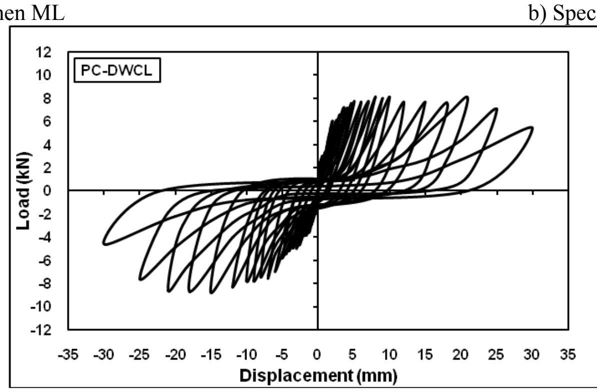
Figure 8 (b) and 8(c); show the load-displacement hysteretic response of the precast specimens PC-DW and PC-DWCL respectively. Also, Figures 9(a) and (c) show the Moment-rotation hysteretic response of the precast specimens PC-DW and PC-DWCL respectively. Greater pinching was observed for the specimens PC-DW and PC-DWCL, because of predefined gap opening at the connections, which indicated minimal energy dissipation. For specimen PC-DW and PC-DWCL, cracks were observed around the recess provided for the accommodation of the dowel in the beam region and corbel, whereas no cracks were observed in the column. The load-displacement envelopes of the specimens ML, PC-DW and PC-DWCL is shown in Figure 10.



a) Specimen ML

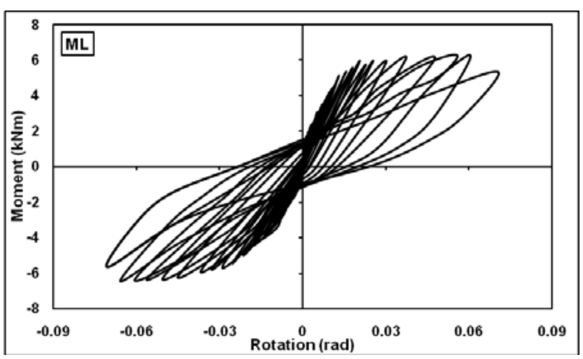


b) Specimen PC-DW

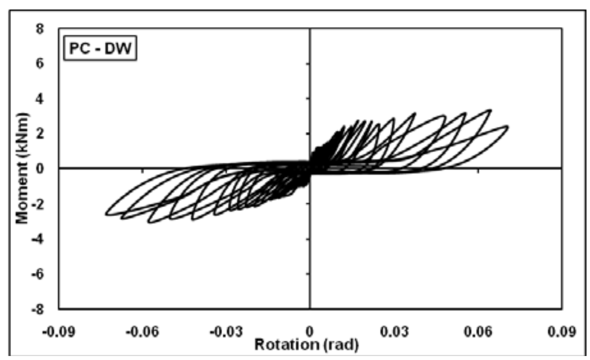


c) Specimen PC-DWCL

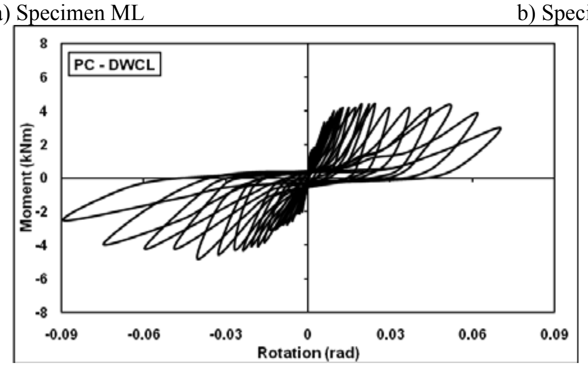
Fig. 8 Hysteresis Load- Displacement curves of all the specimens



a) Specimen ML



b) Specimen PC-DW



c) Specimen PC-DWCL

Fig. 9 Hysteresis Moment-Rotation curves of all the specimens

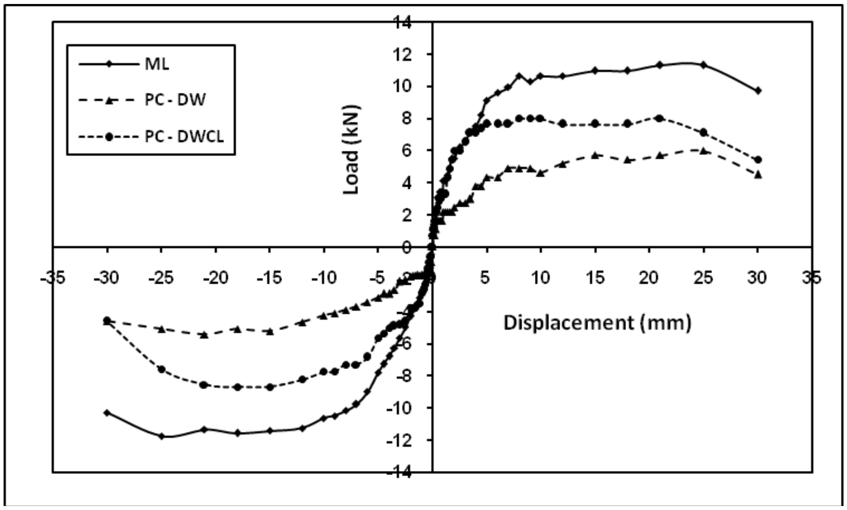


Fig. 10 Load-displacement envelopes of specimens ML, PC-DW and PC-DWCL

5.5. Energy dissipation capacity

When sufficient amount of energy is dissipated by the connection without any substantial loss of strength and stiffness under cyclic loading, the beam-column joint will behave in a ductile manner. Good energy dissipating capacity of a connection is an indication of the satisfactory performance of the connection in the inelastic stage. To determine the energy dissipated by the test specimen during each cycle, the area enclosed by the hysteretic loop in a given cycle is calculated. It is obvious that the larger the area occupied, the larger the dissipated energy and the larger the damping effect. The cumulative energy dissipated is computed by summing up the energy dissipated in the consecutive cycles throughout the test. Figure 11 provides a comparison of the cumulative energy of all the specimens. In comparison with the precast specimens, it is observed that the monolithic specimen ML dissipated greater energy of 857.05 kNmm. At the drift ratio of 7.1% the precast connection with dowel PC-DW showed 25 % reduction in the cumulative energy dissipation when compared to monolithic specimen whereas specimen PC-DWCL had exhibited only 16% reduction.

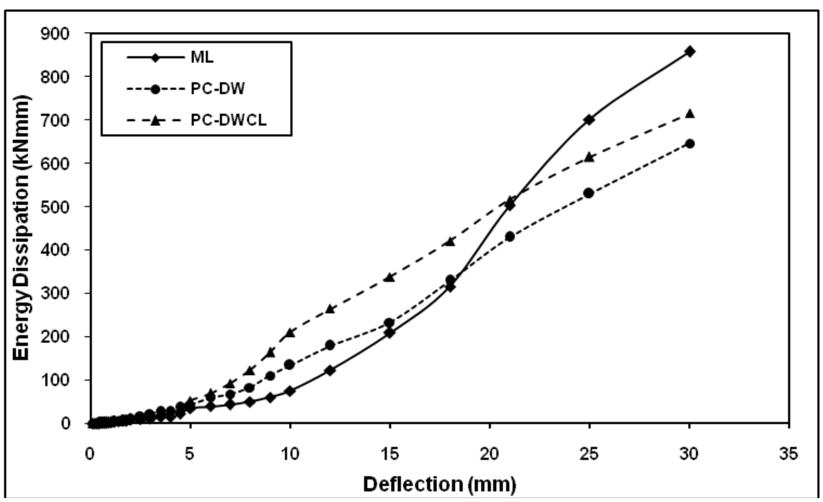


Fig. 11 Cumulative energy dissipation curves of the three specimens

5.6. Equivalent viscous damping ratio

The Equivalent Viscous Damping ratio (H_{eq}) is the ratio of energy dissipated in a half cycle to the strain energy of an equivalent linear system divided by the constant 2Π [15]. The Equivalent Viscous Damping ratio gives the ability to reduce the peak response amplitudes due to inelastic deformation caused by earthquake excitations. Hysteretic damping associated with inelastic deformations in structural connections or members, usually accounts for most of the energy dissipation in the structure. The Equivalent Viscous Damping ratio versus drift ratio of specimens ML, PC-DW and PC-DWCL for one percent drift ratio and higher are shown in Figure 12.

The ratios at 1 percent drift for specimens ML, PC-DW and PC-DWCL were nearly the same. This was because upto 1 percent drift all specimens behaved elastically. The ratios for specimen PC-DWCL started at the same level as PC-DW but increased at a much higher rate upto 2.35 percent drift. Beyond 3.5 percent drift, the ratios remained more or less constant. The ratios for specimen PC-DW and ML started increasing at a higher rate beyond 1.88 percent drift and remained constant beyond 3.5 percent drift and 5 percent drift respectively.

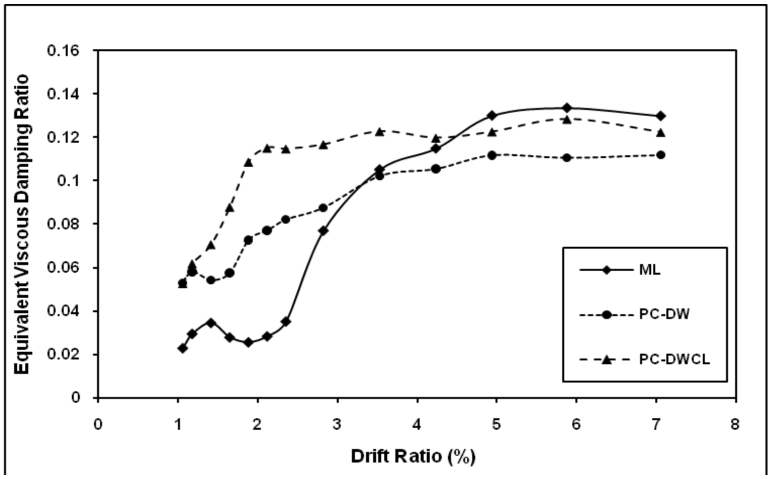


Fig. 12 Equivalent Viscous Damping Ratio curves of the three specimens

5.7. Ductility

The ratio of the maximum displacement that a structure or an element can undergo without significant loss of initial load carrying capacity to the initial yielding deformation is defined as displacement ductility. The concept of reduced stiffness equivalent elasto-plastic yield was used to define the yield and ultimate displacement from the load versus displacement envelope [16]. The ultimate displacement corresponded to 85% of the peak load [17]. The first yield displacement was found by extrapolating the measured stiffness at 75% of the theoretical flexural strength of the specimen up to the theoretical strength of the specimen [18].The displacement ductility factor calculated for monolithic and the two precast beam – column connections is shown in Table 5. The average displacement ductility of the specimens indicated that all the three connections behaved in a ductile manner. The average displacement ductility factor of precast specimen PC-DWCL was slightly greater than the monolithic specimen ML. Specimen PC-DWCL showed about 38% and 17% increase in ductility when compared to specimen PC-DW and specimen ML respectively.

Table 5 Comparison of Displacement Ductility Factor

Specimen	Yield displacement Δ_y, (mm)		Ultimate Displacement Δ_u (mm)		Displacement Ductility factor(μ)		Average Displacement Ductility factor(μ)
	Positive	Negative	Positive	Negative	Positive	Negative	
ML	6	7.6	30	30	5	3.947	4.474
PC-SS	7	11	27.8	29.4	3.971	2.673	3.322
PC-DS	3.4	7.4	25.8	23.2	7.588	3.135	5.362

5.8. Stiffness degradation

As the test specimens were subjected to reverse cyclic loading, it resulted in the accumulation of damage which led to stiffness degradation. Stiffness degradation of the test specimens were determined based on the secant stiffness changes. The secant stiffness is peak to peak stiffness that is defined as the slope of the line that connects the peak positive and negative response during a load cycle [19].The variation of secant stiffness in each displacement cycle is calculated and is shown in Figure 13.

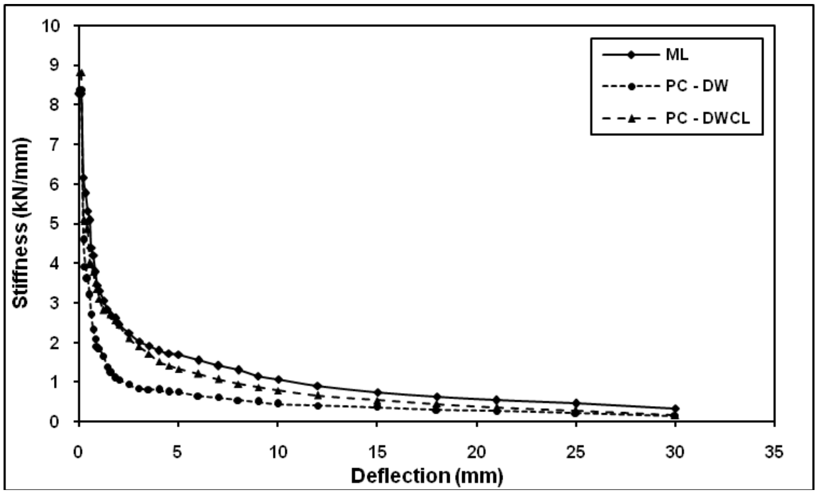


Fig. 13 Stiffness Degradation of three specimens

As the displacement levels increased, it was observed that the cumulative damage in the beam-column joint region for the monolithic specimen ML and beam and corbel for the precast specimens PC-DW and PC-DWCL increased. This led to reduction in stiffness of the test specimens. The stiffness degradation was observed to be faster in the initial displacement cycles (i.e) upto 5 mm displacement cycle due to concrete cracking. Due to the additional stiffness offered by the cleat angle, the precast specimen PC-DWCL exhibited highest initial stiffness in comparison with specimen ML and precast specimen PC-DW. The initial stiffness for the specimen ML and specimen PC-DW was found to be nearly the same. For comparing the stiffness of all the test specimens, each secant stiffness value of a specific specimen was normalized (k_{norm}) with respect to the secant stiffness measured at 5 mm displacement level. The normalized stiffness degradation versus deflection of the three specimens is shown in Figure 14. It was observed that at the end of 30 mm displacement cycle, the precast specimen with dowel PC-DW and monolithic specimen ML retained about 20% of the initial stiffness followed by specimen PC-DWCL. The loss of initial stiffness for the specimens ML, PC-DW and PC-DWCL were 80%, 80% and 87% respectively. The specimen with dowel and cleat angle PC-DWCL showed greater loss of initial stiffness when compared to specimens ML and PC-DW.

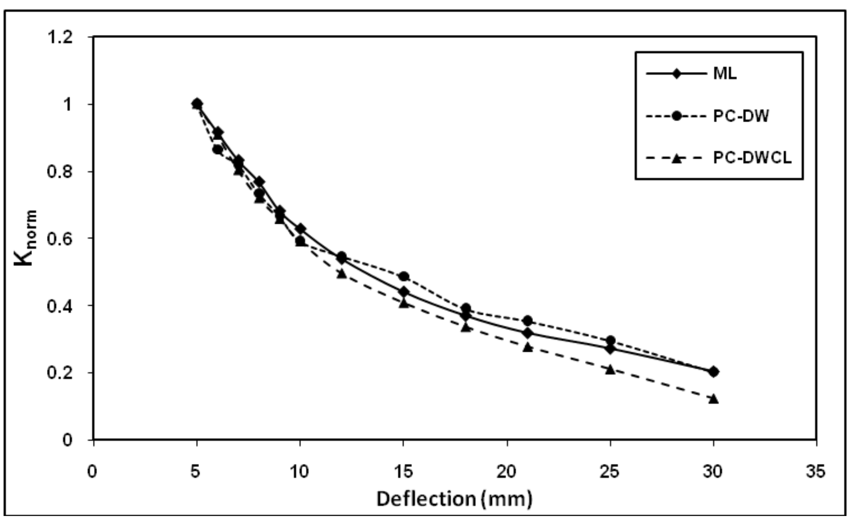


Fig. 14 Normalized Stiffness Degradation of the three specimens

6. Conclusions

Two types of simple precast concrete dowel beam-column connections subjected to reverse cyclic loading were investigated by conducting experiments. The results were then compared with the performance of a reference monolithic beam-column connection. The types of precast concrete connections considered for the present study are (i) Dowel Bar (PC-DW) and (ii) Dowel Bar with Cleat Angle (PC-DWCL). The parameters considered for the present study are load carrying capacity, energy dissipation and ductility. The summary of the observations are as follows;

a) The ultimate load carrying capacity of the connection with dowel bar and cleat angle PC-DWCL exhibited 25% and 38% greater load carrying capacity than the specimen with dowel bar PC-DW in the positive and negative direction respectively. This is due to the additional stiffness and strength developed due to the presence of cleat angle. Compared to the monolithic specimen ML, the specimen PC-DWCL exhibited lesser load carrying capacity. The variation is 29% and 26% in the positive and negative direction respectively.

b) All the specimens were loaded upto 30mm displacement cycle. There were no cracks developed in the column for both the precast specimens. Only minor cracks were developed in the corbel of the specimen PC-DW and PC-DWCL. The monolithic specimen developed cracks in the column region.

c) The precast connection PC-DWCL exhibited wider hysteretic curves compared to specimen PC-DW. Both the precast specimens experienced pinching due to the predefined gap opening at the connections. The monolithic specimen ML showed comparatively better hysteretic behaviour because of the confinement in the joint core. Both the precast concrete connections PC-DW and PC-DWCL has shown similar trend as that of monolithic specimen ML in the post elastic range upto the ultimate load.

d) Considering the energy dissipation, the specimen PC-DWCL performed better than the specimen PC-DW and dissipated 11% higher energy than specimen PC-DW. The energy dissipation of specimen PC-DWCL is about 16% lesser than the monolithic specimen ML.

e) The precast specimen with dowel and cleat angle PC-DWCL showed better ductility than that of precast specimen with dowel PC-DW and monolithic specimen ML. Specimen PC-DWCL showed about 38% and 17% increase in ductility when compared to specimen PC-DW and specimen ML respectively.

f) Out of the two precast specimens, specimen PC-DWCL performed much better than the specimen PC-DW.

g) Also, it is observed that the precast specimen, PC-DWCL exhibited satisfactory behaviour in comparison with the monolithic specimen ML.

7. Summary

The observation made from the performance of various precast connections, it is proposed that the precast concrete connection with dowel bar and cleat angle PC-DWCL is a simple dry connection that can be used for the construction of low rise moment resisting frames.

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References

[1] Precast Concrete Frames Guide. Irish Precast Concrete Association (IPCA), Dublin, 2005.

[2] American Concrete Institute (ACI), ACI 550.1 R-01. Design Recommendations for Precast Concrete Structures Reported by ACI-ASCE Committee, 2001.

[3] Ochs J.E, Ehsani M.R. Moment resistant connections in precast concrete frames for seismic regions, PCI Journal, 1993, No. 5, Vol. 38, pp. 64-75.

[4] Loo Y.C, Yao B.Z. Static and repeated load tests on precast concrete beam-to-column connections, PCI Journal, 1995, No. 2, Vol. 40, pp. 106-115.

[5] Khaloo A, Parastesh H. Cyclic loading of ductile precast concrete beam-column connection, ACI Structural Journal, 2003, No. 3, Vol. 100, pp. 291-296.

[6] Chun S, Lee S, Kang T. Mechanical anchorage in exterior beam-column joints subjected to cyclic loading, ACI Structural Journal, 2007, No. 1, Vol. 104, pp. 102-112.

[7] Li B, Kulkarani S.A, Leong C.L. Seismic performance of precast hybrid-steel concrete connections, Journal of Earthquake Engineering, 2009, Vol. 13, pp. 667-689.

[8] Xue W, Yang X. Seismic tests of precast concrete, moment resisting frames and connections, PCI Journal, 2010, No. 2, Vol. 55, pp. 102-121.

[9] IS: 1893-2002. Code of practice for Criteria for earthquake resistant design of structures Part 1 General provisions and buildings, Bureau of Indian Standards, New Delhi.

[10] IS: 456-2000. Indian Standard code of practice for plain and reinforced concrete, Bureau of Indian Standards, New Delhi.

[11] IS: 13920-1993. Code of practice for Ductile Detailing of reinforced concrete structures subjected to seismic forces, Bureau of Indian Standards, New Delhi.

[12] IS: 800-2007.Code of Practice for General Construction in steel, Bureau of Indian Standards, New Delhi.

[13] Cheok G.S, Lew H.S. Model precast concrete beam-to-column connections subjected to cyclic loading, PCI Journal, 1993, No. 4, Vol. 38, pp. 80-92.

[14] Alameddine F, Eshani M.R. High strength RC connections subjected to inelastic cyclic loading, ASCE Journal of Structural Engineering, 1991, No. 3, Vol. 117, pp. 829-850.

[15] Clough R, Penzien J. Dynamics of Structures, John Wiley & Sons, New York, 1975.

[16] Park R. Ductility evaluation from laboratory and analytical testing, Proceedings of Ninth World Conference on Earthquake Engineering, 2-9 August, Tokyo-Kyoto, Japan (VIII), 1988, pp. 605-616.

[17] Park R, Paulay T. Reinforced Concrete Structures, John Wiley & Sons, New York, 1975.

[18] Park R. Evaluation of ductility of structures and structural subassemblages from laboratory testing, Bulletin of the New Zealand National Society for Earthquake Engineering, 1989, No. 3, Vol. 22, pp. 155-166.

[19] Saqan E.I. Evaluation of Ductile Beam Column Connections for use in seismic Resistant Precast Frames, Thesis (PhD), University of Texas, Austin, 1995.