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BEHAVIOUR OF PRECAST BEAM-COLUMN MECHANICAL CONNECTIONS UNDER CYCLIC LOADING

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ABSTRACT

The present work focuses on comparing the performance of precast and monolithic beam-column joints subjected to cyclic loading. Experiments were conducted on 1/3 scale models of two types of precast beam-column connections and a monolithic connection. The precast connections considered are the beam-column connections in which beam is connected to column with corbel using (i) J-bolt and (ii) cleat angle. The specimens were subjected to reverse cyclic loading. The experimental results of the precast specimens were compared with those of the monolithic connection. Axial load was applied to the column using 400kN capacity actuator. The cyclic loading is applied in the beam using another two actuators, one for positive load cycle and the other for the negative load cycle. The hysteresis behaviour, load carrying capacity, energy dissipation capacity and ductility factor were measured and the performance for the precast and monolithic beam-column connections were compared.

Keywords: Cyclic loading; precast concrete; beam to column connection; J-bolt; cleat angle; monolithic

1. INTRODUCTION

The precast concrete has many advantages like reliability, durability, faster construction, higher quality and all weather construction. But this type of construction is more preferred for construction of flyovers around the world. In the International arena precast concrete sector has experienced reasonable growth in the recent years. But there is hesitancy in extensively using precast concrete in highly seismic areas. There was a clear evidence of failure of precast parking structures during the 1994 Northridge earthquake [1,2]. Failure in these earthquakes was mainly due to poor connections between the precast elements themselves and between the precast elements and lateral load-resisting system. Hence, a lot of research is required in this area. For the past four decades though a lot of research has been done in precast structures, a complete understanding of the behaviour of precast beam-column connections to various

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possible structural loadings has not been completely understood. Connections are one of the most essential parts in prefabricated structures as they constitute the weakest link in the structure. The behaviour of a precast structure, to a large extent, depends on the behaviour of the connections. A key aspect is the behaviour of joints that should have larger capacity than elements, or having a dissipative behaviour, should possess the necessary ductility resources. Therefore, the proper selection of the type of connection to be used and their design play a prominent role in the performance of precast structures. Hence, there is a necessity to carry out more research in this area which will help to improve the knowledge base and thus aid in arriving at improved codal provisions for construction of more durable precast structures.

2. LITERATURE SURVEY

Castro et al. [3] conducted tests on nine two-thirds scale beam-column joints including a monolithic specimen. It was concluded that precast concrete specimens can sustain inelastic deformations and can be ductile as cast-in-situ specimens.

Stone et al. [4] developed a hybrid precast system, which was designed to have the same flexural strength as a conventionally reinforced system with the same beam size. The hybrid system was self-centering and displayed essentially no residual drift. The hybrid system had a very large drift capacity. The hybrid system dissipated more energy per cycle than the conventional system for up to 1.5 percent drift. The concrete in the hybrid suffered negligible damage, even at drifts up to 6 percent.

Alcocer et al. [5] conducted experiments on two full scale beam-column precast concrete joints under uni-directional and bi-directional loading that simulated earthquake type loadings. The specimens exhibited ductile behaviour. The lateral load carrying capacity was maintained nearly constant up to drifts of 3.5 percent, which are larger than the maximum drift values allowed in most design codes around the world.

Joshi and Murty [6] performed experiments on two precast and corresponding monolithic exterior beam-column joint sub-assemblage specimens. The monolithic specimen with beam bars anchored into the column performed better than the monolithic specimen with continuous U-bars as beam reinforcement. The cumulative energy dissipation for the monolithic specimen with continuous U-bar reinforcement was more than the other monolithic specimen. Similarly,the precast specimens with beam bars anchored into the column performed better than the corresponding monolithic beam. The precast specimen with continuous U-bars as beam reinforcement performed worse than the corresponding monolithic specimen, due to high average strength and stiffness deterioration. Of the two precast specimens, the one with the beam bars anchored into the column with the welding of the lap splices performed better than the one with continuous U-bars as beam reinforcement.

Ertas et al. [7] presents the test results of four types of ductile, moment-resisting precast concrete frame connections and one monolithic concrete connection, all designed for use in high seismic zones. All tested precast concrete connections, except for one precast specimen were suitable for high seismic zones in terms of strength properties and energy dissipation. The hysteresis behaviors of precast specimens were similar to those of monolithic specimen. Most of the precast concrete connections, reached their calculated yield and ultimate flexural moment capacities.

Kulkarni et al. [8] proposed a precast hybrid-steel concrete connection detail and showed that the connection gave satisfactory flexural performance. It was concluded that the connecting plate thickness at the joint influenced the energy dissipation and deflections during the cyclic loading.

From the literature review reveals that the precast connections can be detailed as strong as that of the monolithic connections. It is also understood that the mechanical precast connections have better energy dissipation characteristics. Hence for the present study, two types of mechanical connection, in the form of J-bolt and cleat angle were adopted.

3. OBJECTIVE OF PRESENT STUDY

The objective of the present study is,

1. To identify a simple and suitable precast beam – column connection for an exterior beam-column joint of a moment resisting framed structure.
2. To conduct experimental investigations on two types of precast connections and a monolithic connection.
3. To identify the most suited connection for the precast elements.

4. MATERIAL CHARACTERISTICS

The specimens were cast with M30 concrete using 53 grade Ordinary Portland Cement and Fe 415 grade steel. The water-cement ratio was 0.443.The specific gravity of fine aggregate and coarse aggregate were 2.45 and 2.69 respectively. The fineness modulus of the fine aggregate and coarse aggregate used in the design mix were found to be 3.04 and 6.194 respectively. The average compressive strength of concrete on 28th day was 41.6 MPa.

5. DESIGN AND DETAILING OF SPECIMENS

The beam-column connection in a three storey reinforced concrete residential building in Chennai, India was considered for the present study. The building was modeled and analyzed using STAAD Pro software. The force resultants such as shear force, bending moment and axial force around the exterior beam-column joint due to various load combinations were computed. Seismic analysis was performed using equivalent lateral force method given in IS:1893-2002 [9]. The design and detailing of beam, column and exterior joint was carried out based on the guidelines given by in IS:456-2000 [10] and IS:13920-1993 [11]. One-third scaled models were developed for monolithic and precast specimens. The dimensions of the beam were 100 mm x 100 mm x 550 mm. The column was of size 100 mm x 100 mm x 1200 mm.

5.1 Monolithic connection (ML)

The monolithic reinforced concrete test specimen (ML) was designed according to IS:456-

2000 and detailed according to IS:13920-1993. The Flexural reinforcement for the beam consisted of four bars with one bar at each corner of the transverse reinforcement. Two numbers of 10 mm diameter bars were provided as tension reinforcement and two numbers of 10 mm diameter bars 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. 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. The schematic representation of the isometric view monolithic specimen is shown in Figure 1.

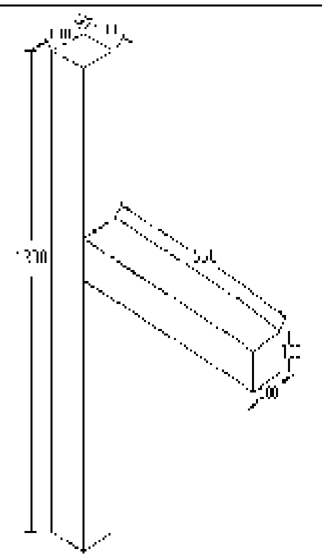

Figure 1. Monolithic beam-column connection

5.2 Beam to column connection using J-Bolt (PC 1)

In this connection the beam was supported on concrete corbel using J-bolt. This connection transmits vertical shear forces. J-bolt of diameter 16 mm was kept inside the corbel and cast by keeping its straight portion protruding outside. The beam was inserted on to the J-bolt and the nut tightened. Iso-resin grout was used to fill the gap between the J-bolt and the hole in the beam. The schematic representation of the isometric view of precast concrete column with corbel and the beam connected using a J-bolt is shown in Figure 2.

5.3 Beam to column connection with cleat angle (PC 2)

In this type of connection two 16mm diameter bolts were used, in which one bolt connects the cleat angle with the column and the other connects the cleat angle with both the beam and the corbel. Figure 3 shows the schematic representation of the isometric view of the precast beam-column connection using cleat angle. The cleat angle used for the connection is ISA 100x100x10. The bolts used are high tensile friction grip bolts. The gap between the bolts and the groove was filled using iso-resin grouts.

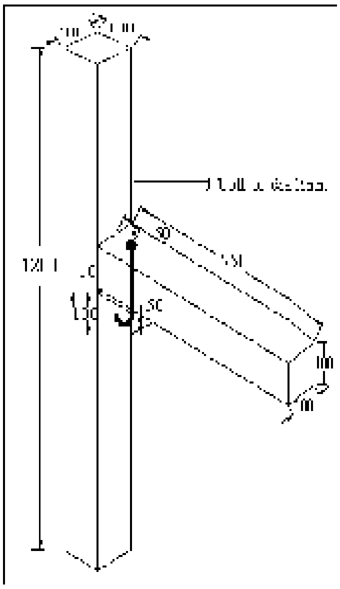

Figure 2. Precast beam-column connection using J-bolt

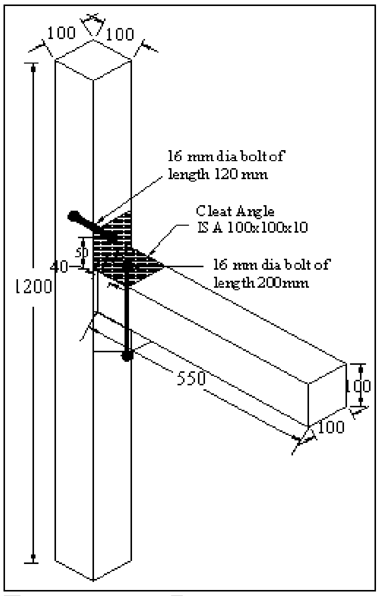

Figure 3. Precast beam-column connection using cleat angle

6. EXPERIMENTAL TEST SETUP

The experiments were carried out on a loading frame of 2000kN capacity. A hydraulic jack was fixed to the loading frame for the application of the axial load along the axis of the column. Two hydraulic jacks were used to apply the reverse cyclic loading. Displacement controlled loading system was adopted. The specimens were tested in an upright position with column in vertical and beam in horizontal position. The column was hinged at floor and was laterally restrained at the top. The schematic representation of the experimental test setup is shown in Figure 4.

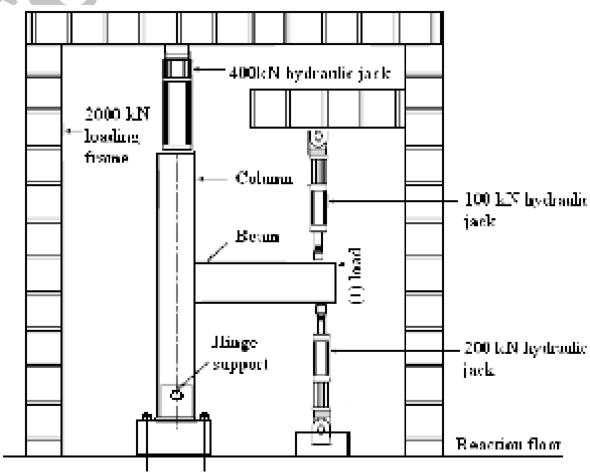

Figure 4. Schematic test setup

7. LOADING SEQUENCE

In order to account for the dead load transferred from upper floors, an axial load of equal to $0.1f_c' A_g$ was applied to the column at the beginning of the test and maintained throughout the test (Cheok and Lew [12]) using hydraulic jack of capacity 400kN. The loading history consists of displacement cycles as shown in Table 1 and Figure 5. Two hydraulic jacks of capacity 100kN and 200kN 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.

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

Sl. No.	Displacement (mm) Start	Displacement (mm) End	Increment
1	0.1	1.0	0.1
2	1.0	2.0	0.2
3	2.0	10.0	0.5
4	10.0	18.0	2.0
5	18.0	21.0	3.0
6	21.0	25.0	4.0
7	25.0	30.0	5.0

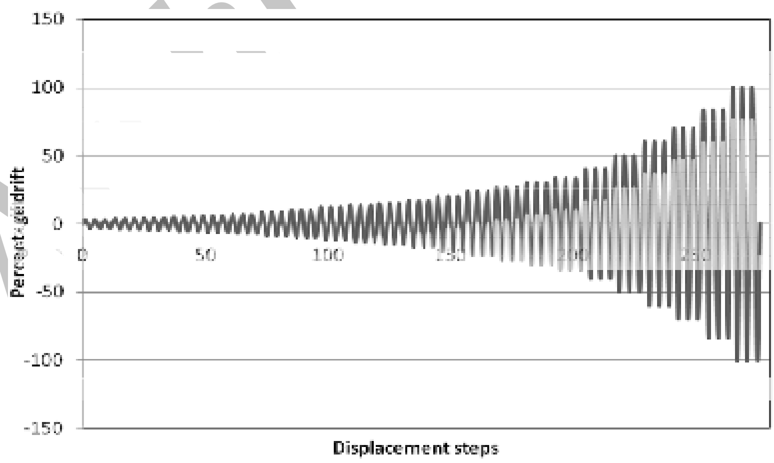

Figure 5. Cyclic loading history

The specimens were instrumented with dial gauges and strain gauges to monitor the behavior. Two dial gauges were fixed in the beam at a distance of 100 mm and 200mm respectively from the face of the column and the third one was fixed at a distance of 125mm from the free end of the beam. Strain gauge indicator was used to measure the strains. To measure the strain in the reinforcement, strain gauges were fixed at various positions in the specimen as shown in Figure 6, 7 and 8. Four strain gauges were fixed in the main reinforcement of beam at a distance *d* (effective depth of beam) from the face of the column. Two strain gauges were fixed in the longitudinal reinforcement of columns at the level of the corbel for the precast specimens. For the monolithic specimen two strain gauges were fixed in the longitudinal reinforcement of column at level with the soffit of the beam.

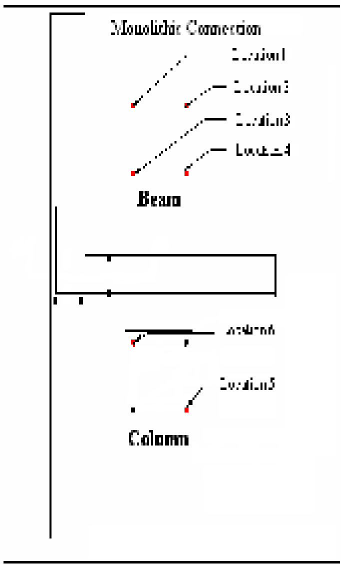

Figure 6. Strain gauge locations monolithic beam- column connection

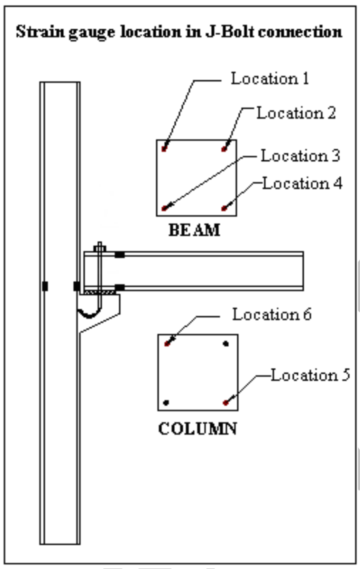

Figure 7. Strain gauge locations in J-bolt connection

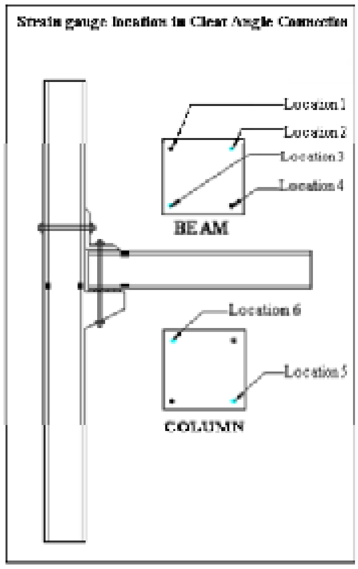

Figure 8. Strain gauge locations in cleat angle connection

8. RESULTS AND DISCUSSION

8.1 Strength

The ultimate load carrying capacity of the specimen ML was found to be 11.29kN and11.75kN in positive and negative directions respectively. For the specimen PC1, the ultimate load carrying capacity was found to be 5.42kN and 4.57 kN in positive and negative directions respectively whereas for the specimen PC2, the ultimate load carrying capacity was found to be 4.33 kN and 3.58 kN in positive and negative directions respectively which is very much lesser than the monolithic specimen. From the results, it is observed that the load carrying capacity of the specimen PC1 was 51.99% and 61.11% lesser than the monolithic specimen in the positive and negative direction respectively. Similarly, the load carrying capacity of specimen PC2 was 61.65% and 69.53% lesser than the monolithic specimen in the positive and negative direction respectively. Out of the two precast specimens the specimen PC1 performed better than specimen PC2. While comparing with the precast specimens the monolithic specimen performed better in resisting the load.

8.2 Crack pattern

All the specimens were subjected to reverse cyclic loading. For the specimen ML, the flexural crack initiated at the beam-column junction at 2 mm displacement cycle (5.13 kN) and propagated further. The flexural cracks in beams were initiated at 2.5 mm displacement cycle (6.15 kN) and were developed away from the beam-column junction. Shear cracks first occurred at the beam-column junction at 7 mm displacement cycle (9.92 kN) and cracks further propagated at 12 mm (10.61 kN), 15 mm (10.94 kN), 18 mm (10.95 kN), ±21 mm (11.29 kN), 25 mm (11.29 kN) displacement cycles. The failed monolithic specimen ML is shown in Figure 9.

For the specimen PC1, the first flexural crack initiated on the beam where the bolt has been fixed at 1.5 mm displacement cycle (2.7 kN). Further flexural cracks occurred at 3 mm (2.71 kN),-6 mm (3.09 kN), 8 mm (4.33 kN), and 18 mm (5.14 kN) displacement cycles. Cracks in the corbel occurred at 8 mm displacement cycle (4.33 kN) where the bolt had been fixed. Further cracks developed in the corbel at 18 mm displacement cycle (5.14 kN). All the cracks in the beam and corbel occurred at the position of J-bolt. No cracks were observed in the column except at the corbel region. The failed precast specimen PC1 is shown in Figure 10.

For the specimen PC2, the first flexural crack in the beam was initiated below the cleat angle at −2.5mm (3.64 kN) displacement cycle. Also flexural cracks occurs at −3mm (3.65 kN), −4 mm (4.18 kN), -7mm (4.56 kN),12 mm (2.01 kN) at the position where the recesses was provided for the bolt which connected the cleat angle with the column. Cracks occurred in the corbel at 1.4mm (2.7kN) displacement cycle and propagated at 2.5mm (217 kN) displacement cycle. Spalling of concrete was also observed at the position of bolts. The failed precast connection, PC2, is shown in Figure 11.

Figure 9. Failed monolithic specimen

Figure 10. Failed J-bolt connection

Figure 11. Failed cleat Angle connection

8.3 Load displacement relationship

The Load-displacement relations for the monolithic and the precast specimens have been obtained from the test results and presented in Figures 12, 13 and 14.

The load-displacement hysteresis loops for the cyclic loading at each displacement excursion level are shown in Figure 12, 13 and 14.The load displacement hysteresis curve of monolithic specimen ML shown in Figure 12 exhibited similar load displacement pattern in both positive and negative directions. The strength and stiffness degradation has been observed only after 25mm displacement cycle. From Figure 13, it is inferred that the energy dissipation in the positive direction is greater than that in the negative direction. This is because of the ductility offered by the J bolt. In the positive direction, strength degradation occurred beyond 18 mm displacement cycle whereas in the negative direction, the strength degradation occurred only beyond 25 mm displacement cycle. From Figure 14, it is observed that the strength degradation occurred beyond 10 mm displacement cycle in the negative direction, whereas in the positive direction, the strength degradation occurred only beyond 18 mm displacement cycle. For monolithic and the two precast specimens the test was stopped after completion of 30 mm displacement cycles, as the strength dropped below 80 percent of ultimate strength in positive and negative displacement direction.

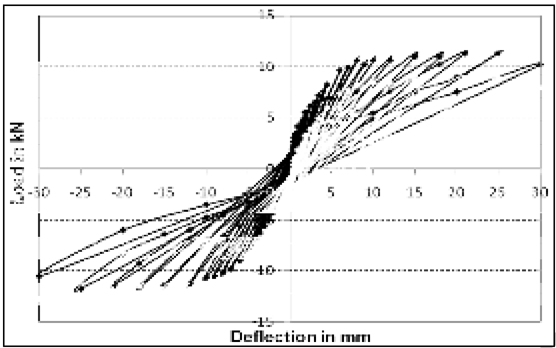

Figure 12. Hysteresis curve for monolithic specimen ML

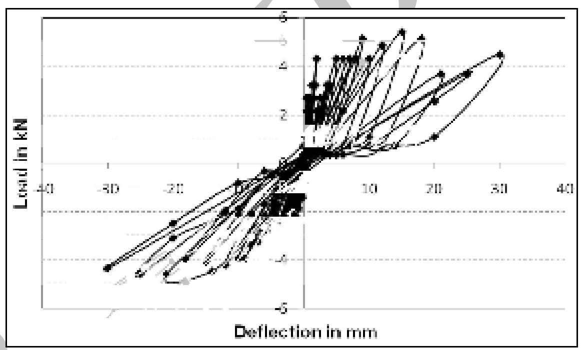

Figure 13. Hysteresis curve for precast specimen PC1

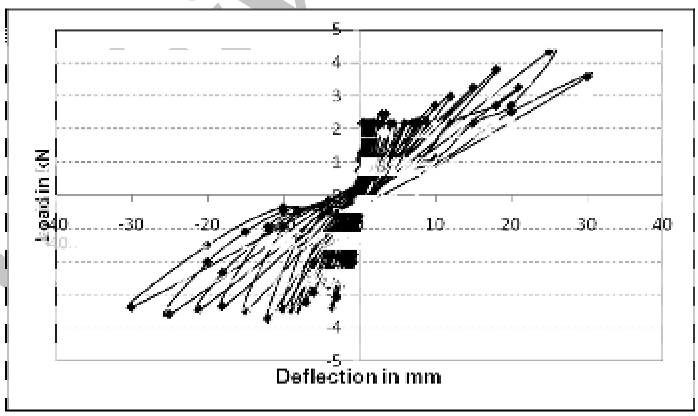

Figure 14. Hysteresis curve for precast specimen PC2

8.4 Energy dissipation

The area under the load displacement curve gives the energy dissipation of the specimen. Figure 15 shows the comparison of energy dissipation of the precast specimens with that of the monolithic specimen.

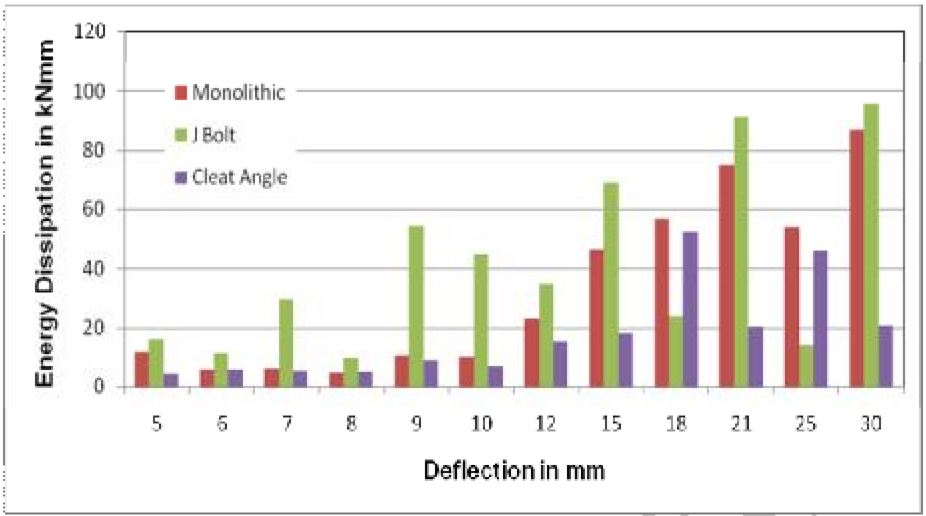

Figure 15. Comparison of energy dissipation from 5mm to 30mm

It can be observed that the cumulative energy dissipation for the specimen PC1 was 22.87% greater than the ML connection whereas the energy dissipation for the specimen PC2 was 41.78% lesser than the monolithic connection. The specimen PC1 exhibits better performance because the J bolt is properly embedded within the concrete medium and provides sufficient ductility to the system.

8.5 Ductility

The displacement ductility factor is determined as the ultimate displacement divided by the displacement at the occurrence of yielding of longitudinal steel bars. The ductility factor of the monolithic and precast specimens have been evaluated and given in Table 2.

Table 2: Ductility factor of the three specimens

Specimen	**Yield displacement**	**Ultimate displacement**	**Ductility factor**
Monolithic	5	25	5
Precast (PC1)	1	18	18
Precast (PC2)	9	25	2.78

It can be observed from Table 2 that the displacement ductility of the specimen PC1 was found to be more than that of monolithic specimen. Hence the specimen PC1 is more ductile when compared to the specimen ML. As energy dissipation and ductility are the characteristics which make the structure perform better under seismic forces, the results indicate that the precast specimen PC1 have favourable behaviour under seismic load whereas specimen PC2 does not have favourable behaviour under seismic load.

8.6 Strain in Reinforcement

8.6.1 Monolithic specimen(ML)

In this connection, totally six strain gauges were used to measure the strain in reinforcement under cyclic loading. The strain gauges were pasted at various locations given in Figure 6. Figure 16 gives the strain values corresponding to the deflection and it can be observed that the strain in the bottom left longitudinal reinforcement bar in the beam (strain gauge no.3) and the strain in the longitudinal bar at the outer edge of the column (strain gauge no. 5) experiences the maximum strain due to the cyclic loading.

8.6.2 Precast connection using J-bolt (PC1)

Similarly, the strain measured corresponding to deflections in the PC1 specimen for the strain gauges shown in Figure 7 have been measured and plotted in Figure 17. From Figure 17, it has been observed that the strain in the bottom left longitudinal reinforcement bar in the beam (strain gauge No.3) experiences the maximum strain due to the cyclic loading applied. In precast connection, the column reinforcements were free from strains compared to that of monolithic connection.

8.6.3 Precast connection using Cleat Angle

The strain measured corresponding to deflections in the PC2 specimen for the strain gauges shown in Figure 8 have been measured and plotted in Figure 18. From Figure 18, it has been observed that the strain in the bottom left (strain gauge No.3) and the bottom right (strain gauge No.4) longitudinal reinforcement bar in the beam experiences the maximum strain due to the cyclic loading applied. In precast connection, the column reinforcements were free from strains compared to that of monolithic connection.

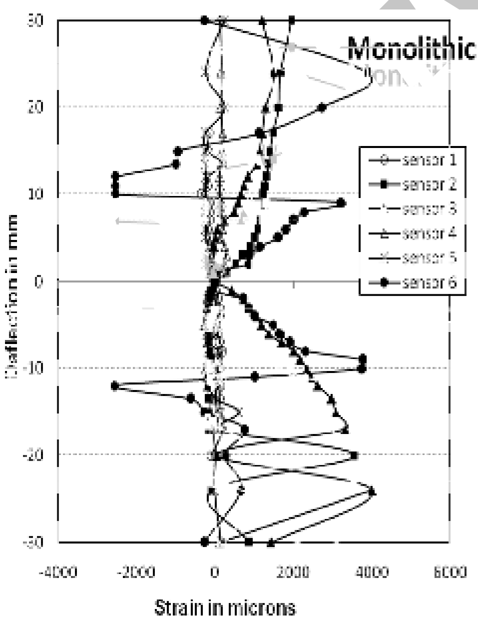

Figure 16. Strain in reinforcements in specimen ML

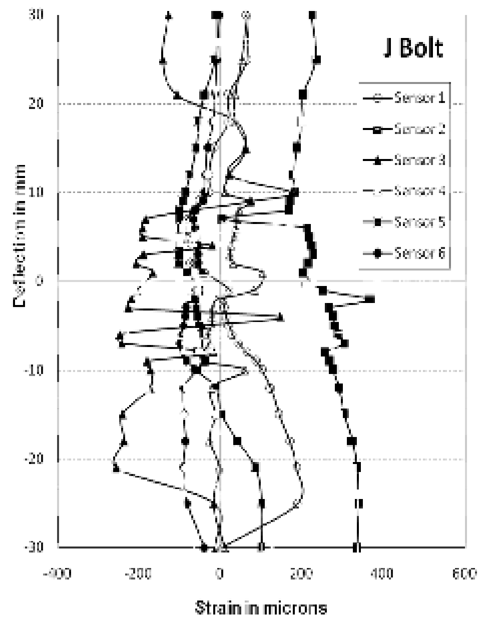

Figure 17. Strain in reinforcements in specimen PC1

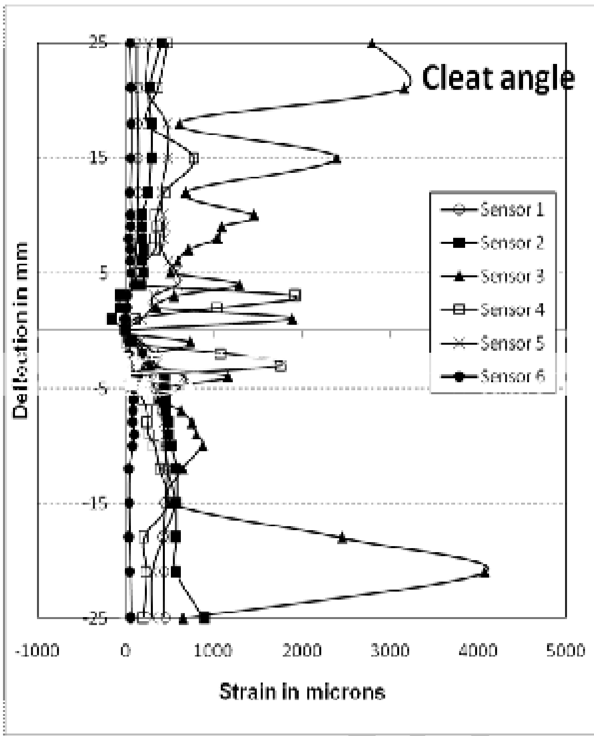

Figure 18. Strain in reinforcements in specimen PC2

9. CONCLUSIONS

Precast construction is most versatile form of construction and it provides high-quality structural elements, construction efficiency, and savings in time and overall cost of investment. In the design of earthquake resistant structures that incorporate precast concrete elements the main difficulty has been to find efficient and economical methods for connecting the precast concrete members together, and create connections that give adequate strength, stiffness and ductility. But lack of sufficient experimental data affects their application in high seismic regions. In this context, monolithic and precast specimens were cast and the behavior under cyclic loading was experimentally investigated.

From the results it was observed that the ultimate load carrying capacity of the monolithic specimen is more than the precast specimens PC1 and PC2. Precast specimen PC1 is more ductile and dissipates more energy compared to the monolithic specimen whereas precast specimen PC2 is less ductile and dissipates less energy compared to the monolithic specimen. Precast specimens showed increased stiffness in the negative direction due to the presence of corbel. The bottom left reinforcement bar in the beam experiences the maximum strain due to the applied cyclic loading in the both the precast specimens. In precast connection, the column reinforcements were free from strains compared to that of monolithic connection.

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