Research Article

Behaviour of Precast Column Foundation Connection under Reverse Cyclic Loading

L. Hemamathi 1 and K. P. Jaya 2

1Department of Civil Engineering, R.M.K Engineering College, Gummidipoondi, Tamilnadu 601206, India
2Structural Engineering Division, Department of Civil Engineering, Anna University, Chennai 600025, India

Correspondence should be addressed to K. P. Jaya; kpjaya@nayan.co.in

Received 19 November 2020; Revised 3 March 2021; Accepted 23 March 2021; Published 7 April 2021

Academic Editor: Andreas Lampropoulos

Copyright © 2021 L. Hemamathi and K. P. Jaya. This is an open access article distributed under the Creative Commons Attribution License, which permits unrestricted use, distribution, and reproduction in any medium, provided the original work is properly cited.

Precast column foundation connection is one of the critical connections under reverse cyclic loading, and the present study focuses on this connection. Three types of connections were considered, such as (i) base plate connection, (ii) pocket connection, and (iii) grouted sleeve connection. All the above connections were designed, and experimental investigation was carried out on 1:2 scaled models by subjecting the column to lateral reverse cyclic loading. Displacement-controlled loading pattern has been adopted for the testing of the specimens. The structural response of the connection was studied for their (i) load-displacement hysteresis behaviour, (ii) stiffness degradation, (iii) energy dissipation, and (iv) ductility. The results were then compared with that of the monolithic connection. The precast connection was more ductile, and the energy dissipated by the pocket connection was high compared to the base plate and grouted sleeve connection. The ductility and the load-carrying of grouted sleeve connection were small compared to other connections. The results of the study showed the precast column foundation can be used in seismic prone areas.

1. Introduction

Rapid growth in the construction industry necessitates quality construction along with a reduction in construction time and cost-effectiveness of structural elements and materials. This is achieved by precast concrete construction which has been used widely internationally due to better quality control, compared to onsite constructions. Despite of many of its advantages, precast concrete structures have failed during an event of an earthquake, and the failure is attributed to the improper connection between the structural elements [1]. The connection between the various structural elements such as beams, columns, slabs, and walls must integrate effectively to ensure safety, serviceability, and durability [2]. The response of precast concrete structures is driven by the most critical connections which include external and internal beam-column connections, wall-to-wall connections, wall-to-slab connection, and column-to-foundation connections. The seismic response greatly depends on the behaviour of the connection system, and the key role was played by proper design and detailing of the joints [3]. The problem associated with the external beam-column connection is lack of ductility and low shear strength. Number of research has been focused on this connection to improve its flexural strength, shear strength, and ductility by providing a variety of wet and dry connections [4, 5]. The behaviour of precast shear wall and slab connection has been studied using in situ concrete and dowel bars [6], and the precast connection showed superior behaviour concerning ultimate load and ductility compared with monolithic connection [7]. In high seismic regions, the lateral load acting on the structure due to earthquake or wind would damage the entire structure if not properly designed. Out of all the structural connections considered, not much research has been carried out for column-to-foundation connection. The design of column footing is based on the assumption that a plastic hinge may develop at the column base during seismic action. The typical structural
layout consists of moment-resisting frames with a plastic hinge occurring at the column base [8].

Metelli and Riva [9] proposed an Edilmatic connection system for the column-to-foundation joint composed of threaded bars with bushes embedded in the column and tied to the column reinforcement; plastic ducts deactivate the bond of longitudinal high strength steel bars to provide adequate ductility and dissipative capacity of the joint in case of cyclic action. They have studied that the connection evinced localized damage allowing easier postseismic column repair. An evaluation of the response of welded connection in steel column shoe connected to the foundation through anchor bolts was examined by Bianco et al. [10], and it was studied that the collapse mechanism is governed by the behaviour of anchor bolts without significant damage to the column specimen. Testing of grouted sleeve connection for the seismic load was carried out by Buratti et al. [11], where it was observed that high values of rotation were registered at the column base, while deformation does not propagate over the height of the column. Also, a stable hysteretic behaviour was observed up to 5% drift in comparison with the cast-in-place connection. A study was conducted by Aboukif et al. [12] on pocket base connections using the Leonhardt and Monnig model. The experimental results proved that the connection is of the closest type to monolithic connection, where no failure occurred in the pocket itself.

2. Research Significance

When precast concrete structures are considered for seismic behaviour, the most crucial is the connection between the structural members. Generally, the different types of connections in precast structures are wet, emulative, dry, welded, and bolted connection. Research on external and internal beam-column connection, wall-to-wall connection, wall-to-slab connection, column-to-column connection, beam-to-beam connection, and column-to-foundation connection has been carried out by researchers all over the world. This research paper focuses on the connection between precast column and foundation for three different types of connections.

The generally adopted column-to-foundation connections are as follows: (i) bolted base plate embedded in the foundation, (ii) foundation pockets in which the column is inserted and grouted, (iii) grouted sleeves, and (iv) mechanical splices. This paper presents the experimental investigation of precast column connected to the foundation through simple base plate (PCBJ) and pocket connection (PC) and by using grouted sleeve (GS) subjected to reverse cyclic loading. Four specimens comprising of precast column and foundation were cast on a 1:2 scale, and the specimens were subjected to reverse cyclic loading. The test results were then compared with that of the monolithic specimen of the same size subjected to the same loading condition.

3. Experimental Program

To arrive at the force components for the experimental investigations, a four-storied structure has been modelled and analysed using structural software. The analysis results have been utilised for the testing of the specimens. The methodologies and procedure have been discussed in the following sections.

4. Modelling of Prototype

A four-storied structure was considered for the study with five bays of 6.0 m each along X-direction and four bays of 4.0 m each along Y-direction. The total height of the structure was 12.2 m with the height of ground storey as 3.2 m and other storeys as 3.0 m each [13]. The structure is designed to be located in Chennai which falls under zone 3 as per IS 1893:2002 with moderately stiff soil condition.

The structure was modelled and analysed using SAP 2000 software. Figures 1 and 2 show the modelled view of the structure to locate the critical column. The structure was analysed for various load combinations as per IS 1893:2002. The critical column was identified based on the resultant axial force and bending moment, and the same has been marked in Figure 2. The critical column and its connection to the foundation were considered for the study.

The force resultants acting on the critical column are shown in Table 1 and were considered for designing the connection. Specimen with a 1:2 scaled-down model has been considered for conducting the experimental study.

The dimensions of the prototype and model are given in Table 2.

The reinforcement details of the column and foundation used in this research paper for the prototype and the model are given in Table 3.

5. Design of Connection and Elements

5.1. Design of Monolithic Connection. The column and foundation base was designed for a design load of 480 kN and was detailed for ductility [14, 15]. The dimension of the foundation block was calculated considering medium-stiff soil with a safe bearing capacity of 200 kN/m². The structural members were designed as per IS 456 (2000) and detailed as per IS 13920 (1993). The design of the precast column was done similar to the monolithic column. The design and detailing of various precast connections are discussed as follows.

5.2. Precast Column and Base Plate Connection (PCBJ). The base plate was fixed to the column by welding it to the main reinforcing bars of the column using a 6 mm fillet weld. The base plate was designed for erection load as well as the resultant forces. It is subjected to biaxial bending from the resultant compressive forces acting on the surface. The
The thickness of the base plate depends on the overhang projection from the column face [16].

The base plate of 300 mm × 300 mm and 12 mm thick was used to connect the column to the foundation through anchor bolts embedded in the foundation. Nuts and washers were used to connect base plate and anchor bolts permit the control of the vertical position and ensure fixity of the connection. Anchor bolts used to connect the base plate to the foundation was designed for the compressive forces acting on them.

| Critical load on column | Prototype | Model |
|-------------------------|-----------|-------|
| Axial load (kN)         | 1920      | 480   |
| Moment (kN-m) (uniaxial)| 142.5     | 17.8  |
| Shear force (kN)        | 420       | 105   |

**Table 1: Critical load on column.**

| Dimension                  | Prototype     | Model   |
|----------------------------|---------------|---------|
| Size of column             | 400 mm × 400 mm | 200 mm × 200 mm |
| Height of column           | 3.5 m         | 1.725 m |
| Size of square footing     | 2.7 m × 2.7 m | 1.35 m × 1.35 m |
| Thickness of footing       | 650 mm        | 325 mm  |

**Table 2: Dimensions of the prototype and model.**
The compressive force on the bolt is calculated from
\[
F = 0.4 f_{cu} b \Psi - N, \tag{1}
\]
where \( f_{cu} \) is the grade of concrete, \( b \) is the width of base plate, \( \Psi \) is the compressive stress block depth, and \( N \) is the axial force on the column.

The area of holding down bolts is calculated using
\[
A_b = \frac{F}{\sum n \times f_{yb}}, \tag{2}
\]
where \( \sum n \) is the number of bolts and \( f_{yb} \) is the ultimate tensile strength of bolt.

The bolts were fabricated from steel stud in the shape of J with a length of 410 mm and 12 mm diameter. Holes in the plate are normally oversized to offset construction setting out and production tolerances. Figure 3 shows the force distribution in the column base of the precast connection. POWERGROUT-NS3, a cement-based nonexpanding polymer enriched high performance, high early strength, and high-quality binder for precision grouting applications, was used to grout the portion between the base plate and foundation. The compressive strength of the grout tested as per IS 4031 Part 6 used for grouting the specimen as provided by the supplier was 60 N/mm² at 28 days at 10% water ratio.

5.3. Pocket Connection (PC). In the pocket foundation connection, the precast column is rigidly fixed to the foundation, and the loads are transmitted in the pocket by friction and end bearing. To ensure total fixity, the column is inserted into the pocket by 1.5D, where D is the largest cross-sectional dimension of the column as recommended by the PCI Connection details committee [17]. Additional links are provided in the precast column to avoid bursting pressure generated by end bearing forces. The gap between the pocket wall and column should be at least 50 mm to 75 mm all round, and it should be filled in with grout. The distribution of forces in the column pocket is shown in Figure 4.

Pocket is exerted by the horizontal forces as follows [14].

Horizontal force \( H_A \) is obtained from
\[
H_A = 1.14 \left( \frac{M}{h} \right) - 0.15N + 1.03H_D, \tag{3}
\]
where \( M \) is the moment about point A, \( h \) is the height of pocket wall, \( N \) is the axial force on the column, and \( H_D \) is the horizontal force at the surface of the transverse wall.

Horizontal force \( H_A \) at point A is obtained by the following equilibrium equation:
\[
H_A = H_B - H_D. \tag{4}
\]

The reinforcement in the socket is worked out from
\[
A_{SB} = \frac{H_B}{0.87 f_y}, \tag{5}
\]
\[
A_{SA} = \frac{H_A - \mu R}{0.87 f_y}, \tag{6}
\]
where \( A_{SA} \) is the ring reinforcement at \( H_A \) level, \( A_{SB} \) is the ring reinforcement at \( H_B \) level, \( R \) is the vertical reaction, \( \mu \) is coefficient of friction, and \( f_y \) is the yield strength of steel rebar.

Vertical reinforcement in pocket wall is calculated using
\[
A_{SV} = \frac{M + H_D + h}{0.87 f_y Z}. \tag{7}
\]

The surface of the column and inner walls of the pocket was roughened to transmit the axial forces from the column to the foundation. The pocket connection has been detailed in two different methods and was designated as PC I and PC II. The foundation was designed by considering the frictional forces and horizontal reaction acting on the walls of the foundation pocket [16].

Detailing of the transverse walls of the pocket was focused, and a design model proposed by Canha et.al [18] was used for PC I. Compressive forces \( H_B \) and \( H_A \) act on the top and bottom of transverse walls along with frictional forces \( \mu H_D \) (Figure 4). To resist these forces, the transverse reinforcements \( A_{SA} \) and \( A_{SB} \) were provided on the pocket walls. The corners of the wall are the regions of high-stress concentration, and the main vertical reinforcement \( A_{svm} \) was designed to resist this stress. Also, secondary reinforcement \( A_{sv} \) was provided at the midportion of the walls. The analysis of this connection is based on bending theory [18]. The pressure caused by the column on the joint would produce bending in the wall of the foundation pocket, and it would be transmitted to corners. In order to resist such forces, the secondary reinforcing bars were wound around the main

---

**Table 3: Reinforcement details of the column and foundation.**

| Reinforcement details | Prototype | Model |
|-----------------------|-----------|-------|
| **Column**            |           |       |
| Main reinforcement    | 8 # 20 mm dia bars | 8 # 10 mm dia bars |
| Transverse reinforcement | 8 mm bars @ 225 mm c/c | 6 mm bars @ 100 mm c/c |
| Top and bottom 500 mm was provided with 16 mm bars @120 mm c/c for ductility | Top and bottom 240 mm was provided with 8 mm bars @ 50 mm c/c for ductility |
| **Foundation**        |           |       |
| Main reinforcement    | 20 mm bars @ 150 mm c/c | 10 mm bars @ 75 mm c/c |
| Transverse reinforcement | 12 mm bars @ 300 mm c/c | 8 mm bars @ 100 mm c/c |
Figure 3: Force distribution in base plate of precast column.

Figure 4: Distribution of forces in column pocket.
reinforcement of the pocket walls. In addition to that, the corners were strengthened by providing dowels bent along the corners at each layer of horizontal reinforcement.

In the second type of connection pocket connection, PC II, the detailing was done considering each of the transverse walls separately, as proposed by Canha et al. [19]. Reinforcement $A_{SA}$ and $A_{SB}$ were provided around the main reinforcement of each wall separately and were tied at corners of the wall. An additional link is provided in the tension anchorage zone for 300 mm at the base of the column to resist the bursting pressure generated by end bearing forces. The main vertical reinforcement of the pocket was extended to the foundation base and tied with its main reinforcement.

5.4. Grouted Sleeve Connection (GS). This is one of the economical precast connections, where the starter bars projecting from the foundation are housed into the sleeve provided in the column. The column is positioned on the packing shims which provide the fixing tolerance. The design of the column is based on the assumption that a full bond is provided to the starter bar enabling their full strength through the grout and sleeve.

The column and foundation base is designed similar to a monolithic connection. A flexible corrugated polyvinyl sleeve with wire reinforcement was placed on four sides of the column. The sleeves were tied close to the main reinforcing bars of the column before concreting, for a length equal to the development length of the bars to be housed.

The diameter of the sleeve used was 25 mm, and it was placed for a length of 475 mm from the base of the column. One end of the sleeve was bent in such a way that it was flushed to the face of the column so that grout could be pumped into it. Four bars of 10 mm diameter were made to protrude from the foundation, and they were inserted into the column while placing the column on the foundation base. NS3, nonshrink grout was used to connect the column and foundation through the sleeve arrangement. Grout was also placed between the column and foundation for a thickness of about 10 mm.

6. Connection Details

The reinforcement details of monolithic connection, precast column with base plate (PCBJ), pocket connections PC I and PC II, and grouted sleeve connection (GS) are shown in Figures 5–9. Special confining reinforcement [14] in the form of closely spaced links is provided for a length of 250 mm from the top and base of the column towards midspan. This is the region where flexural yielding may occur under the effect of earthquake forces. In monolithic connection special confining reinforcement of the column extend into the foundation.

7. Test Set up and Instrumentation

The experimental set up was done to test the monolithic and precast specimen of column and foundation connection under reverse cyclic loading condition. The entire program was displacement controlled [20]. A loading frame of 2000 kN capacity was used for the study. Axial load to simulate the gravity load on the column was applied at the top face of the column using a 400 kN capacity load cell. Reverse cyclic loading was induced at the column top on two opposite faces using a load cell of 100 kN capacity. Two LVDTs were placed on either side of the column, and it can measure lateral displacement up to 50 mm on each side. The set up was connected to “Dewesoft 7.1.1” software to measure the displacement and its corresponding load. The specimen was anchored to the strong reaction floor by fixing the foundation rigidly to the floor. The test set up is given in Figure 10.

8. Loading Protocol

A displacement controlled loading protocol has been adopted for the experimental investigation. Reverse cyclic loading was applied using two load cells which were mounted on the side face of the column at the top on the opposite sides. Three cycles of loading were applied for each displacement level. The loading protocol considered for the investigation consisted of displacement $\pm 1$ mm, $\pm 2$ mm, $\pm 3$ mm, $\pm 5$ mm, $\pm 10$ mm, $\pm 15$ mm, $\pm 20$ mm, $\pm 25$ mm, $\pm 30$ mm, $\pm 35$ mm, and $\pm 40$ mm with a maximum drift of 2.5%. An axial load of $0.1 f_A g$ was applied to the column before starting of cyclic load, and the same was maintained throughout the test using a load cell of capacity 400 kN [21]. Figure 11 represents the loading history for testing the specimen. The specimens were subjected to cyclic loading according to ACI 374.1-05, and cycles shall be of predetermined drift ratios [22].

9. Results and Discussion

Constant displacement was applied on the specimen for both positive and negative cycles, and the corresponding load was noted for each cycle. The test was continued until a displacement value of 40 mm was reached. The specimens were studied for their structural response due to reverse cyclic loading, and the results are compared and discussed below.

9.1. Ultimate Load-Carrying Capacity. The ultimate load-carrying capacity of each specimen, both in the positive direction and negative direction, has been arrived at from the experimental investigation. The same has been plotted in Figures 12(a) and 12(b).

It is observed that the ultimate load-carrying capacity of the monolithic specimen was higher when compared to all other specimens. In the positive direction, the ultimate load-carrying capacity of the monolithic specimen was 33.5%, 28.88%, 85.2%, and 244.04% greater than the PC I, PC II, PCBJ, and GS specimen whereas in the negative direction, the ultimate load-carrying capacity of the monolithic specimen was 48.28%, 51.11%, 53.2%, and 291.02% greater than the PC I, PC II, PCBJ, and GS specimen. The load-displacement envelope is shown in Figure 13.
Figure 5: Reinforcement details of monolithic specimen.

Figure 6: Reinforcement details of PCBJ specimen.
Figure 7: Reinforcement details of PC I connection.

Figure 8: Reinforcement details of PC II connection.
Figure 9: Reinforcement details of grouted sleeve specimen (GS).

Figure 10: Test set up.
9.2. Observations. Initially, the cracks were visible at the junction of the column and foundation. As the loading increased, the cracks developed along with the height of the specimen approximately up to 1 m. The crack pattern was observed in each of the specimens throughout the test. All the specimens started developing horizontal cracks in the column as the load reached yielding load. The first crack developed in the column on its loading face. Once the plastic hinge got developed at the junction, no new cracks were formed but the existing cracks started to widen in each of the displacement cycles and a well-established crack pattern was visible on the junction between the column and foundation.

In the monolithic connection, visible cracks developed for a height of 1.0 m from the base of the column, as the specimen was loaded in both positive and negative direction. The plastic hinge was developed at a displacement of 32 mm,
beyond which no new cracks were developed, but the existing cracks widened for each of the displacement cycle up to 40 mm. Figures 14(a)–14(e) show the visual cracks developed in the test specimens.

A similar observation was made in the precast column. In PCBJ, crack started to develop at various locations along with the height of the column. The load gets transmitted to the foundation through the connection between the base plate and the foundation. As the displacement was incremented, the grout between the base plate and the foundation started to peel off and the connection started to fail. This was because of the yielding of anchor bolts. When the displacement was about 35 mm in the positive direction, complete failure of the anchor bolt was seen. At this point, the load-carrying capacity of the connection gradually decreased and the main bars of the column started to resist the loading for the further increment of displacement up to 40 mm. The experiment was stopped at this point, and the visual cracks were noted.

In pocket connection, which was similar to monolithic connection in many aspects, the plastic hinge was developed in the column at 26 mm displacement cycle for PC I connection and 32 mm displacement cycle for PC II connection. At this time, the grout between the column and pocket started to fail because of bearing pressure in both PC I and PC II. In PC I, few cracks were noticed diagonally along with the corners of the socket wall. Few visible hairline cracks were noticed from the face of the column towards the edges of the wall. In the case of PC II connection, grout between the column and wall failed before the development of cracks in the walls of the pocket. Visual observation indicated the detailing provided for PC II performed better than the detailing of PC I.

In grouted sleeve connection, visible cracks started to develop as the displacement incremented and they started to form on the column face. As the load reached a value of 16 mm, the grout between the column and foundation started to fail. The crack propagated along the sleeve representing that the grout has failed and the load was transferred to the concrete before being transmitted to the bar inside the sleeve. The column was no longer able to take up any more load beyond a displacement of 20 mm, but it was able to displace beyond the ultimate load representing its ductile nature and energy dissipation capacity.

### 9.3. Post-Elastic Strength Enhancement Factor (Load Ratio)

Post-elastic strength enhancement factor or load ratio [20] is calculated as the 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. Table 4 gives the value of load ratio of all specimens.

From the table, it is observed that the load ratio increases for a monolithic specimen as displacement increases. In the case of the precast specimen, the load ratio was found to increase up to a displacement of 26 mm for PC I specimen, 32 mm for PC II specimen, 30 mm for PCBJ, and 20 mm for GS specimen, respectively, beyond which the values started to decrease. The value of the yield load of the precast specimen is lower than that of the monolithic specimen. The observation on load ratio helps to assess the load-carrying capacity of precast specimen beyond yield value, and it is seen that all the specimen were able to withstand the load up to the maximum considered displacement of 40 mm. Figure 15 provides the comparison of the load ratio of all connections.

### 9.4. Hysteretic Behaviour

Load-displacement hysteretic loop for monolithic, PCBJ specimen, PC I, PC II, and GS are shown in Figures 16(a)–16(e). The column top was subjected to reverse lateral loading using a load cell of 100 kN capacity. Simultaneously, displacement was measured from the LVDT connected on the face of the column that can measure displacement up to 100 mm. The entire setup was connected to “Dewesoft version 7.1.1” and the plot of load-displacement envelope was obtained from the same. Hysteresis behaviour characterizes the pinching effect of reinforced concrete structural elements. The wider the loops, the larger the energy dissipation capacity will be and the performance in case of an earthquake event will be better. Also, wider loops indicate good bonding between reinforcement and concrete. From the hysteresis loop of all specimens, it is evident that the pinching effect is greater for precast specimen when compared to the monolithic specimen.

In the PCBJ connection, the anchor bolts provided in the connection paved way for good energy dissipation. The load applied to the column was resisted by the base plate and the bolts protecting the column without causing any damage to the column. The grout between the column and foundation was responsible for good energy dissipation. There was a good pinching effect that was witnessed in this connection beyond a displacement of 24 mm. The same effect was felt in the case of pocket connection PC I and PC II. The load applied to the column was transferred to the pocket walls through the grout. The grout between the column and foundation was responsible for good energy dissipation. Beyond displacement of 26 mm in the case of PC I and 22 mm in PC II, the grout started to fail and the loops started to widen indicating good energy dissipation. Observation of grouted sleeve specimen (GS) showed not much pinching effect, and the energy dissipated was also less when compared to the monolithic specimen. Less pinching in the monolithic specimen was due to flexural cracking at the junction of column and foundation.

### 9.5. Energy Dissipation Capacity

The satisfactory performance of a structure in the inelastic range is measured by its energy absorption capacity. Under cyclic loading, a joint region will be ductile, if a sufficient amount of energy is dissipated without substantial loss of strength and stiffness. The area enclosed by a hysteretic loop in a given cycle represents the energy dissipated by the specimen during that cycle. The cumulative energy dissipated was computed by summing up all the energy dissipated in the consecutive cycles throughout the test. Figure 17 gives the comparison of...
Figure 14: (a) Crack pattern on monolithic column, (b) crack pattern on PC I, (c) crack pattern on transverse of PC I and PC II, (d) development of visible cracks and damage of grout in precast specimen PCBJ, and (e) visible cracks in grouted sleeve connection.
**Table 4: Load ratio of all specimens.**

| Displacement (mm) | Monolithic | PC I | PC II | PCBJ | GS  |
|-------------------|------------|------|-------|------|-----|
| 2                 | 0.365      | 0.453| 0.350 | 0.423| 0.678|
| 4                 | 0.424      | 0.692| 0.523 | 0.529| 0.978|
| 6                 | 0.498      | 0.805| 0.780 | 0.642| 1.15 |
| 8                 | 0.579      | 0.919| 0.843 | 0.769| 1.264|
| 10                | 0.669      | 0.993| 0.935 | 0.824| 1.385|
| 14                | 0.768      | 1.197| 1.148 | 0.844| 1.426|
| 16                | 0.812      | 1.320| 1.232 | 1.117| 1.463|
| 20                | 0.858      | 1.391| 1.291 | 1.179| 1.495|
| 26                | 1.015      | 1.597| 1.317 | 1.249| 1.319|
| 28                | 1.038      | 1.552| 1.443 | 1.341| 1.316|
| 30                | 1.098      | 1.559| 1.455 | 1.398| 1.258|
| 32                | 1.251      | 1.481| 1.496 | 1.335| 1.138|
| 34                | 1.395      | 1.395| 1.290 | 1.238| 1.044|
| 38                | 1.501      | 1.395| 1.272 | 1.112| 0.967|
| 40                | 1.585      | 1.394| 1.266 | 1.069| 0.695|

**Figure 15: Comparison of load ratio of all specimen.**

**Figure 16: Continued.**
cumulative energy dissipated in the monolithic and precast specimen.

From the graph, it is evident that the precast specimen was able to dissipate more energy when compared to the monolithic specimen. Both the pocket connection PC I and PC II served to dissipate more energy followed by the PCBJ connection. The energy dissipated by PC I was greater than 26.76% of PC II, 59.21% of PCBJ, 90.46% of monolithic, and 137.6% of GS specimen.

9.6. Ductility. The ratio of the maximum displacement that a structure or an element can undergo without significant loss of maximum load-carrying capacity to the initial yielding deformation is defined as displacement ductility. From the load versus displacement envelope, the yield and ultimate displacement were taken using the concept of reduced stiffness equivalent elastoplastic yield [23]. The ultimate displacement corresponded to 85% of the peak load [24]. 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 [25]. Displacement ductility and average ductility factor are tabulated in Table 5.

It is evident from Table 3, specimen PC I is more ductile compared to all other specimens. Also, precast specimen proved to more ductile than monolithic specimen as the connection between the column and the foundation is semirigid. From the above table, it is seen that the ductility of specimen PC I is 44.2% greater than PC I, 132.7% greater than GS, 136.25% greater than PCBJ, and 184.32% greater than the monolithic specimen.

9.7. Stiffness Degradation. All the structural components and system exhibit some stiffness degradation of some level when subjected to reverse cyclic loading. Stiffness is one factor that helps to study the structure’s response due to seismic forces. Due to reverse cyclic loading, damage accumulates in the specimen which leads to stiffness degradation. Stiffness degradation is measured as peak-to-peak stiffness. The secant value for each cycle was calculated, and that gives the stiffness degradation. The peak-to-peak stiffness is defined as the slope of the line that connects the peak positive and negative response during a load cycle [26]. The level of stiffness degradation depends on the characteristics of the structure such as material properties, geometry, and level of ductile detailing, as well as the loading history. The variation of secant stiffness in each displacement cycle is calculated and is shown in Figure 18.

As the displacement increases, the connection joint between column and foundation exhibited damage and there was a reduction in stiffness. The figure represents the degradation of the peak-to-peak stiffness. It can be seen that the stiffness reduced suddenly in the monolithic specimen as the displacement cycle increases, whereas the stiffness decreased gradually in the case of the precast specimen. The stiffness decrease from 9.16 kN/mm to 0.88 kN/mm in the
Figure 17: Cumulative energy dissipation of all specimen.

Table 5: Displacement ductility and ductility factor of all specimens.

| Specimen | Yield displacement ($\Delta_y$) (mm) | Ultimate displacement ($\Delta_u$) (mm) | Displacement ductility factor $\mu = \Delta_u / \Delta_y$ | Average ductility factor ($\mu$) |
|----------|--------------------------------------|----------------------------------------|----------------------------------------------------------|---------------------------------|
| Monolithic | 28.91 Positive 21.28 Negative | 39.35 Positive 35.94 Negative | 1.36 Positive 1.69 Negative | 1.524 |
| PCBJ | 22.75 Positive 21.75 Negative | 41.04 Positive 40.56 Negative | 1.80 Positive 1.86 Negative | 1.834 |
| PC I | 8.04 Positive 10.77 Negative | 40.00 Positive 39.19 Negative | 5.02 Positive 3.64 Negative | 4.333 |
| PC II | 15.57 Positive 11.6 Negative | 40.00 Positive 39.89 Negative | 2.57 Positive 3.44 Negative | 3.003 |
| GS | 23.2 Positive 20.00 Negative | 40.03 Positive 40.00 Negative | 1.73 Positive 2.0 Negative | 1.862 |

Figure 18: Stiffness degradation of all specimens.
10. Conclusions

The experimental results concerning the precast column to the foundation using a base plate and anchor bolts, pocket connection, and grouted sleeve connection under reversed cyclic loading helped to understand the behaviour of the connection. The test results indicated that the precast column foundation connection can be used in regions of medium to the moderate earthquake. The preexisting cracks at the interface of the column and foundation block allow the damage to be localized with easy postrepair at the connection. The experimental results of all tested specimens concluded the following:

(1) The ultimate load-carrying capacity of the monolithic specimen was higher when compared to all other specimens. The connection between the column and foundation is rigid in monolithic whereas complete rigidity cannot be provided in the precast specimen. They were semirigid, and they were not able to resist load in comparison with monolithic specimen.

(2) Visual observation indicated that the failure of the precast specimen was due to grout failure. In PCBJ, the failure of anchor bolts was followed by the failure of the grout without much damage to the column and the foundation. In the case of pocket connection failure of the grout was due to bearing pressure exerted by the column on the walls of the pocket. For the same displacement controlled loading, the detailing proposed for PC II was better than that of PC I. In the case of PC I, the pocket wall started to develop diagonal cracks at the corners whereas no such cracks developed in PC II. In grouted sleeve (GS) connection the bars inside the sleeve started to yield once the grout failed inside the sleeve. This reduced the load-carrying capacity of the specimen.

(3) The load ratio for monolithic specimen kept increasing as displacement increased. In the case of specimen PCBJ, PC I, PC II, and GS, the load ratio was found to increase up to 30 mm, 26 mm, and 32 mm, and 20 mm displacement cycles, respectively, beyond which there was a decline in the value.

(4) From the hysteresis loop, it is evident that the pinching effect is greater for precast specimen when compared to the monolithic specimen. All precast specimen except for GS connection had good energy dissipation capacity than a monolithic specimen, which proved the usage of the precast specimen in seismic region.

(5) Precast specimen evinced to be more ductile than conventional monolithic connection. The ductility of specimen PC I is 44.2% greater than PC I, 132.7% greater than GS, 136.25% greater than PCBJ, and 184.32% greater than the monolithic specimen.

(6) The stiffness degradation in the precast specimen reveals the gradual pace of decrease indicating good behaviour of the connection during a seismic action.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

Acknowledgments

The research was carried out to study the behaviour of precast column foundation connection under reverse cyclic condition, as the precast construction is in boom in India. So, the financial support was self-borne by the authors.

References

[1] R. Vidjeapriya, V. Vasanthalakshmi, and K. P. Jaya, “Performance of exterior precast beam-column dowel connections under cyclic loading,” *International Journal of Civil Engineering*, vol. 12, no. 1, 2014.
[2] A. A. Vee, “Design considerations for precast prestressed concrete building structures in seismic areas,” *PCI Journal*, vol. 36, pp. 40–55, 1991.
[3] E. Brunesi, R. Nascimbene, D. Bolognini, and D. Bellotti, “Experimental investigation of the cyclic response of reinforced precast concrete framed structures,” *PCI Journal*, vol. 60, 2015.
[4] T. Ahmed, S. Allam, E. Etman, and A. Mohamed, “Behavior of precast reinforced concrete beam-column external connections under cyclic loading, earthquake resistant engineering structures,” *WIT Transactions on the Built Environment*, vol. 185, 2019.
[5] R. Vidjeapriya, K. P. Jaya, and B. Praveenkumar, “Cyclic behaviour of precast beam to column connections with joint in beam region,” in *Proceedings of the 16th World Conference on Earthquake*, 16WCEE 2017, Santiago, Chile, January 2017.
[6] S. Arthi and K. P. Jaya, “Hysteresis behaviour of precast shear wall–slab connection under reverse cyclic loading,” in *Proceedings of the International Conference on Materials, Mechanics and Structures 2020 (ICMMS2020)* Department of Civil engineering, National Institute of Technology (NIT) Calicut, India, July 2020.
[7] S. Arthi and K. P. Jaya, “Seismic performance of precast shear wall–diaphragm connection: a comparative study with monolithic connection,” *International Journal of Civil Engineering*, vol. 18, no. 1, pp. 9–17, 2020.
[8] A. Belleri and P. Riva, “Seismic performance and retrofit of precast concrete grouted sleeve connections,” *PCI Journal*, vol. 57, 2012.
[9] G. Metelli and P. Riva, “Seismic behaviour of precast column to foundation joint,” in *Tailor Made Concrete Structures*, J. C. Walraven and D. Stoelhorst, Eds., Taylor & Francis Group, London, UK, 2008.
L. Bianco, S. Santagati, D. Bolognini, and R. Nascimbene, *Seismic Response of Column Connected to the Foundation through a Fastening Technique*, Peikko Group, Lahit, Finland, 2012.

N. Buratti, L. Bacci, and C. Mazzotti, "Seismic behaviour of grouted sleeve connections and foundations and precast columns," in *Proceedings of the Second European Conference on Earthquake Engineering*, Istanbul, Turkey, August 2014.

M. A. Aboukifa, K. H. Reyad, and F. A. Saad, "Behaviour and design of precast column/base pocket connection with smooth surface interface," *Al-Azhar University Civil Engineering Research Magazine (CERM)*, vol. 39, no. 3, 2017.

IS:1893, “Code of practice for criteria for earthquake resistant design of structures,” *General Provisions and Buildings*, Bureau of Indian Standards, New Delhi, India, 2002.

IS: 13920, *Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces*, Bureau of Indian Standards, New Delhi, India, 1993.

IS: 456, *Indian Standard Code of Practice for Plain and Reinforced Concrete*, Bureau of Indian Standards, New Delhi, India, 2000.

Singapore Standard, *On Code of Practice for Structural Use of Concrete*, Singapore Standard, Singapore, 1999.

PCI Connection Details Committee, *PCI Connection Manual for Precast and Prestressed Concrete Construction*, PCI, Chicago, IL, USA, 1st edition, 2008.

R. M. F. Canha, K. de Borja Jaguaribe Jr, A. L. H. de Cresce El Debs, and M. K. El Debs, “Analysis of the behavior of transverse walls of socket base connections,” *Engineering Structures*, vol. 31, no. 3, pp. 788–798, 2009.

R. M. F. Canha, G. M. Campos, and M. K. El Debs, “Design model and recommendations of column-foundation connection through socket with rough interfaces,” *Revista IBRACON de Estruturas e Materiais*, vol. 5, no. 2, pp. 182–218, 2012.

F. Alameddine and M. R. Ehsani, “High-strength RC connections subjected to inelastic cyclic loading,” *Journal of Structural Engineering*, vol. 117, no. 3, pp. 829–850, 1991.

G. S. Cheok and H. S. Lew, “Model precast concrete beam-to-column connections subject to cyclic loading,” *PCI Journal*, vol. 38, no. 4, pp. 80–92, 1993.

ACI Committee 374, *Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary*, American Concrete Institute, Farmington Hills, MI, USA, 2005.

R. Park, “Ductility evaluation from laboratory and analytical testing,” in *Proceedings of the Ninth World Conference on Earthquake Engineering*, pp. 605–616, Tokyo, Japan, August 1988.

R. Park, T. Paulay, *Reinforced Concrete Structures*, John Wiley & Sons, New York, NY, USA, 1975.

R. Park, “Evaluation of ductility of structures and structural assemblages from laboratory testing,” *Bulletin of the New Zealand Society for Earthquake Engineering*, vol. 22, no. 3, pp. 155–166, 1989.

E. I. Saqan, “Evaluation of ductile beam column connections for use in seismic resistant precast frames,” Doctoral dissertation, University of Texas, Austin, TX, USA, 1995.