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Analytical Perfomance of One Way Moment Resisting Connections of Precast Beam to Concrete Column

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Abstract: In this study, two proposed connections of beams to precast concrete columns were analyzed via application of nonlinear finite element analysis Software ABAQUS 6.10. Connection responses associated with lateral resistance, lateral stiffness, ductility and energy dissipation were compared to an equivalent monolithic connection. Achieved lateral resistance, lateral stiffness and ductility of the proposed connections was approximately 95, 80 and 70% of the equivalent monolithic connectionm, respectively. According to calculation results, these connections are appropriate substitutions for completely resisting beam to column connections in precast concrete structures.

Key words: Finite elements, precast concrete, energy dissipation, lateral stiffness, ductility, lateral resistance

INTRODUCTION

Advantages of precast concretes have increased worldwide usage of precast concrete structures. Connections in frame systems affect significantly the constructability, stability, strength, flexibility and residual forces in the structure. Furthermore, connections play an important role in the dissipation of energy and redistribution of loads as the structure is loaded. However, due to discontinuity, precast connections are considered disadvantageous for structures that require special attention to design and performance. Also, the creation of totally rigid connections of beams to columns in precast structures is very difficult and time-consuming, there by negating advantages gained using precast features. Studies have shown that if a semi rigid connection stiffness is >80% of an equivalent monolithic connection stiffness, seismic behavior of the system will not significant change (Sucuoglu, 1995). Therefore, connections must develop expected mechanical features and demonstrate a low-cost fast-construction system for multi-storey buildings.

In 1987, the research and development department of the precast concrete Institution conducted a laboratory investigation of 16 samples of precast connections (eight simple and eight moment resisting connections). The aim of the project was to obtain connections behavior such as resistance, ductility, energy dissipation and sustainability and economical performance (Dolan *et al*., 1987). Laboratory investigation of connections is very difficult, time-consuming and expensive. In addition, laboratory studies have shown that acquisition of information about connection behavior is not enough. In (Dolan and Pessiki, 1989) created laboratory models with scales of one-half and conducted an analytical study of connection in the PCISFRAD project No. 1/4 in order to demonstrate that a computer model can be an appropriate and acceptable method for analysis of precast concrete connection behavior. Bull and Park (1986), created laboratory models in order to evaluate seismic behavior of one type precast moment resisting beam to column connection in New Zealand. This connection was made by placing a U-shaped precast concrete beam at the junction and the connection was completed using in situ concrete and slabs. The connection has been used to make frame structures with small height. They adopted relations for connection design also. French *et al.* (1989) proposed an alternative hypothesis regarding precast connection systems. They design that the connection can be placed far from of the plastic hinge section (Dolan *et al*., 1987). In fact structures were designed to develop a plastic hinge in the partially prestressed portion of the beam outside of the connection region. Researchers such as Restrepo *et al.* (1995), Khoo *et al.* (2006) proposed beam to beam connections. The specimens exhibit ductility and energy dissipation characteristics similar to those of ordinary reinforced concrete elements as the connection regions are designed to emulate ordinary reinforced concrete in a precast system. Parastesh *et al.* (2014) conducted experiments on one type precast concrete connections with a scale of 0.4 under different bar ratio and stirrup distances in

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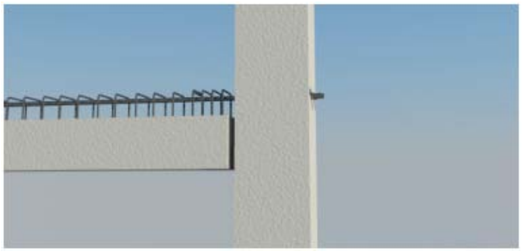


Fig. 1: Schematic exterior precast connection

beam. The primary objective of their research was to develop flexural resistant connection of precast beam to precast column for high seismic zones. In these experiments, cruciform specimens were made of continuous columns and beams that were separated. A gap was created in the columns to provide enough bearing area for sitting the beams and transferring the construction loads before *in-situ* concrete becomes structural. The cross-section of the beams was U-shaped at near connection while the rest of cross-section was rectangular. Forces resulting from flexure occurred due to overlapping of protruding bars from columns and beams buried with in situ concrete (Khaloo and Parastesh, 2003; Parastesh *et al*., 2014). Shariatmadar and Zamani studied three specimens of precast concrete connections. In two proposed connections, column was discontinuous at connections and performance of this area was completed by grouting. In the third connection, the column was continuous and the seated of the beam was created by welding available pending plates in the beam and column (Shariatmadar and Beydokhti, 2014). Elliot *et al.* (2003) tested 4 semi rigid connections. Supports for beams are provided by means of steel corbel or solid section on each side of the columns which also transferred shear forces. The top bars passed from the columns and provided connection continuity. This can lead to a low-cost fast-construction system for multi-storey buildings where multiple stories can be constructed at once (Elliot *et al*., 2003; Gorgun, 1997).

In this study two proposed moment-resisting precast beam to column connections were proposed and their lateral performance is illustrated. In both proposed connection systems the prefabricated concrete columns are cast continuously in elevation with steel corbel embedded in the connection core to connect beam elements in the proposed systems there is no need for using formwork and temporary vertical supports for beam and slab elements. This can lead to a low-cost fast-construction system for multi-storey buildings where multiple stories can be constructed at once.

Proposed two precast connections: Design and performance of completely rigid connections in precast concrete structures is difficult and time-consuming. For some connections, the need for framing or large volumes of in-situ concrete undermines the inherent beneficial features of prefabrications. Therefore, the industry of precast concrete structures seeks connections with easy quick installation that provide required mechanical features.

Two moment-resisting precast connections are proposed and their performance is assessed in the presented study. Figure 1 illustrates details of the exterior moment-resisting connections for precast concrete frames before cast-in-place concreting. In both proposed connection systems, the prefabricated concrete columns are cast continuously in elevation with steel corbel embedded in the connection core to connect beam elements. Four vertical bars are welded to the corbel in the connection zone of the precast columns (i.e., beam-column joint core) to provide adequate shear strength and stability during the installation and prevent the slip of corbel during lateral loading. The steel corbel provide enough bearing area for sitting the Reinforced Concrete (RC) beams and transferring the construction loads before in-situ concrete becomes structural. Consequently, in the proposed systems, there is no need for using formwork and temporary vertical supports for beam and slab elements. This can lead to a low-cost fast-construction system for multi-storey buildings where multiple stories can be constructed at once.

In both connections, the precast beam is placed on the steel corbel and steel bars below the beam were connected to the steel corbel using bolts or welding and provide continuity. Two top continuous bars of beams passed through the holes in the columns and develop negative moment. Cast-in-place concreting provides connection continuity. Followings, the other details of the two proposed connections are explained in more details.

MATERIALS AND METHODS

First Precast Connection (specimen PC-1): In the first proposed Precast Connection (PC-1) the precast concrete beam was placed on the embedded steel corbel in the continuous column and bottom threaded bars of the beam were tightened between the grooves of the corbel by two nuts and steel gaskets with thicknesses of 10 mm. The empty space of the connection area was filled with expandable grout. After grouting, two top bars were passed through two holes in the column. Those holes were also grouted and the connection was completed by slab concreting. The schematic of this connection is illustrated in Fig. 2. The details of the first beam-to-column connection is shown in Fig. 3.

Specimen PC-2: In the second Proposed Connection (PC-2) a steel box section of size 100×100×10 mm

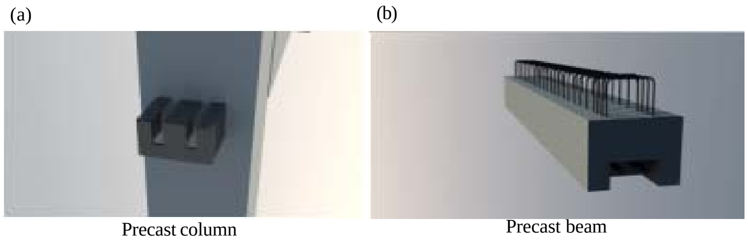


Fig. 2: a, b) Connection schema of first Precast Connection (specimen PC-1)

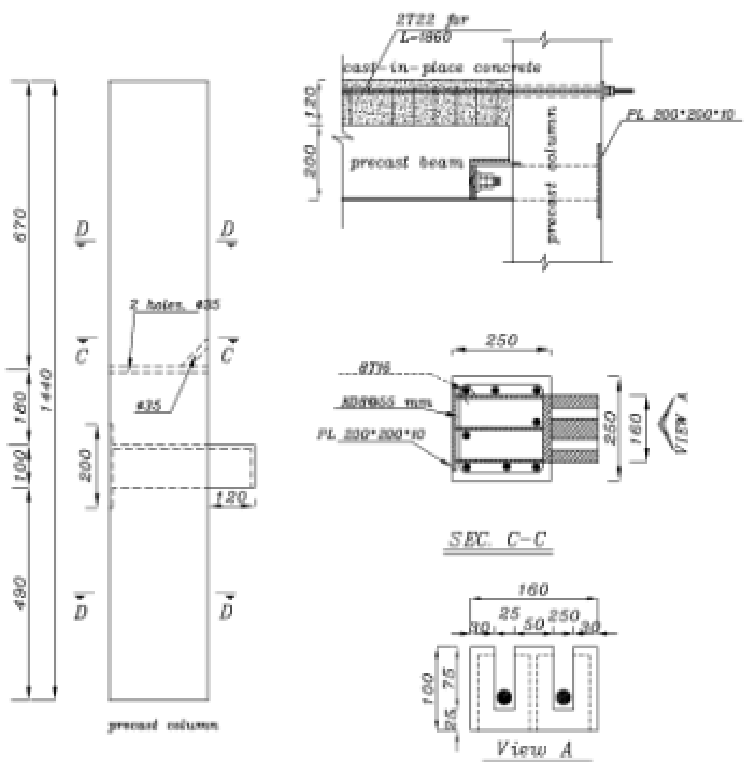


Fig. 3: Beam to column connection details of specimen PC-1

embedded in the column was used as a corbel to connect the beam and joint core. Bars below the beam transferred their force to columns through welding to a channel cross-section and then through channel-shaped welding to corbel. Rebar welding to the channel cross-section is typically carried out during manufacturing in factory. The beam-to-column connection details and schematic of the specimen PC-2 are shown in Fig. 4 and 5, respectively. After welding the channel to the box, the empty space was filled with expandable high-strength grout, completing the connection. Similar to the previous connection, two bars with diameters of 25 mm were passed from the columns then the holes were filled with grout and connection was completed by slab concreting.

Specimen MC: For comparing the result a cast-in-place specimen MC was designed and constructed according to ACI318-2008 for. The beam section was 250 mm by 320 mm, the column section was 250 mm by 250 mm, longitudinal top bar and bottom bar of beams and columns exactly were similar to specimen PC2. The specimens were designed according to the strong column-weak beam design concept so that the inelastic damage of the column did not occur. The net beam span was 1.375 m and the height of the column was 1.5 m.

Loading and boundary conditions: Two proposed PC-1 and PC-2 connections and monolithic connection MC were loaded in lateral form and axial force on the column. The lateral force was applied to the system step by step

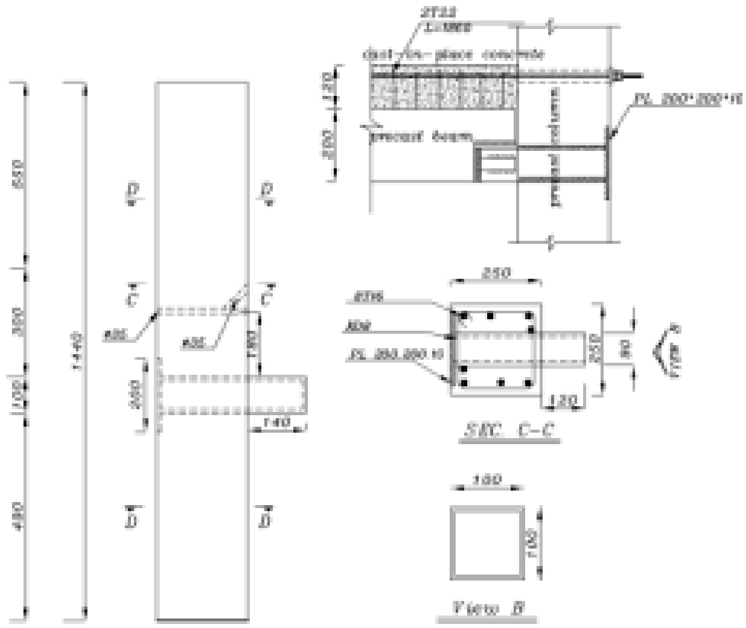
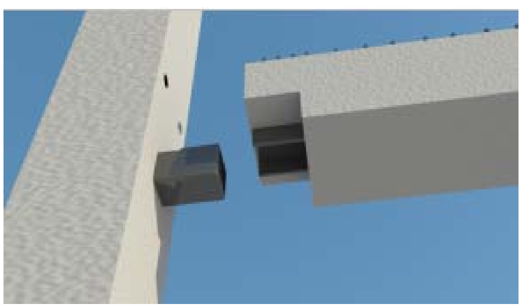


Fig. 4: Beam to column connection details of specimen PC-2



Precast beam-to-column connection

Fig. 5: Chematic of second precast connection (specimen PC-2)

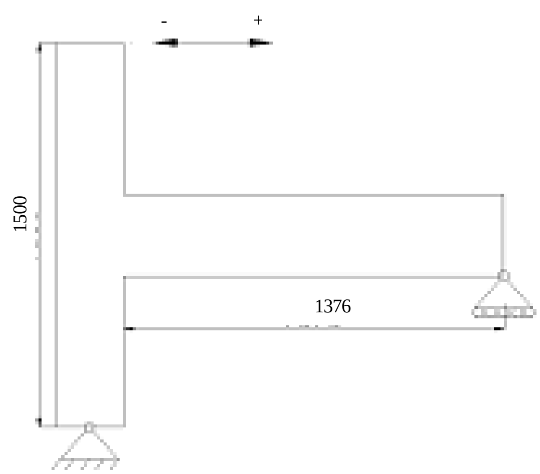


Fig. 6: Boundary condition and loading of specimens

to specimens fail in positive direction or negative direction. The column was supported by pined connection at its base and free on top. Roller supported was modeled to end of beams. Various axial loads was applied to the columns at each specimens. The boundary conditions of specimens are shown in Fig. 6.

Finite element modeling: To model the behavior of the two proposed moment-resisting connections a finite element model was developed using ABAQUS Software. The beams have 250×320 mm rectangular cross-section and 3 m length. The columns have square cross-section with dimension of 400 mm. The longitudinal reinforcement of the beam and column was deformed bars of Grade 420 while the beam stirrups and column transverse ties were applied with Grade 300 bars. The compressive strength of concrete and grout used for all specimens was 30 and 45 MPa, respectively. Steel plates, U sections and box sections used in the constructions were of st 37 with F_y = 240 MPa and f_u = 370 MPa in which f_y and f_u are yield and ultimate strengths, respectively. The weld material is of E60 electrode having as strengths f_y = 240 MPa and f_u = 370 MPa. Modulus of elasticity and poisson's ratio are considered as 200 Gpa and 0.3 for steel and 25 GPa and 0.2 for concrete, respectively.

Concrete: ABAQUS provides 6 types of three Dimensional (3D) stress/displacement elements for

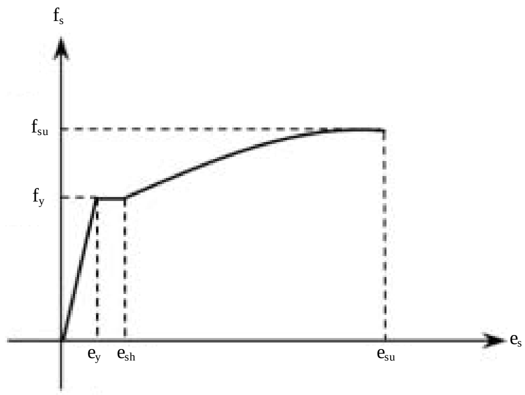


Fig. 7: Proposed stress-strain model for reinforcing steel by Mander

modeling concrete including the 4-node linear tetrahedron, the 6-node linear triangular prism, the 8-node linear brick, the 10-node quadratic tetrahedron, the 15-node quadratic triangle and the 20-node quadratic brick. To cover the concrete behavioral properties and minimizing problematic convergence of 3D Models, the 8-node linear brick element, titled C3D8R was chosen for 3D Modeling. The suffix "R" at the title of the element indicates the reduction of integration points to decrease the required run time. This element has three degree of freedom at each node.

In this study, the stress-strain behavior of concrete material were given to the software using the modified Hognestad curve. This constitutive stress-strain model considers two major mechanisms of rapture in concrete materials tensile cracking and compressive fracture. The compressive behavior has a parabolic function up to the maximum compressive stress. After ultimate compressive point, the post cracking behavior is taken into account by specifying a post linear stress-strain relation up to the maximum strain. The tensile behavior is linear initially followed by a strain softening after the ultimate tensile point. Modulus of elasticity of the concrete materials in this modeling was calculated using the ACI 318-2011 equation for normal strength concrete:

$$E_c = 4700\sqrt{f_c'}\ \text{MPa}$$

Reinforcing steel bars: In ABAQUS, rebar can be specified as smeared layers in membrane, shell or surface elements or they can be included in continuum elements by embedding rebar surface or membrane elements into continuum element. Here, steel bars and interaction between bars and concrete were specified using embedded capability in the software. This displacement capability interpolates the node related to the bar with the node related to the concrete element. This feature can simulate the interaction of rebar and concrete to an acceptable limitation. Adhesion between concrete surfaces with steel sheets and surfaces between precast and in situ concretes was disregarded as insignificant. Only tangential behavior between surfaces was taken into account. Friction between concrete surfaces was 0.7 and 1.0 if the surfaces were ridged (Rabbat and Russell, 1985; Lee *et al*., 2011). Proposed friction between the steel and concrete surfaces was 0.65. For steel mat erial, modulus of elasticity of 200 Gpa and poisson's ratio of 0.3 were used.

The monotonic behavior of longitudinal steel was modeled using (Mander, 1983) model. The model of Mander was developed as a result of many tension and compression coupon tests. This model which takes into account elastic behavior, yield plateau and strain hardening of steel material has three main regions as shown in Fig. 7. The first region is a linear function with slope equal to steel's modulus of elasticity; the region ends at the yield point with stress equal to yield stress of steel. The second region simulates yield plateau and the third region is an ascending curve up to the maximum strength of steel, simulating the strain hardening region of steel behavior. The post-ultimate stress region is not considered in Mander (1983) Model (Shirmohammadi, 2015).

Welds connections in steel parts of the connection are modeled 3 dimensionally in order to observe the stresses in the welds. This gives us the advantage of observing the stress in welds and adjusting the sizes and locations in order to minimize the stress concentration. Concrete to concrete or steel interfaces are considered to interact only tangentially, since the normal contact strength is reasonably small. The concrete element was the linear 8-node element with dimensions of 15 mm. Elements related to linear two-point rebar with dimensions measuring less than half of the concrete element were also considered. Numerical integration used in the software was the Gauss method and the method of nonlinear system solution of the software was the Newton Raphson method.

RESULTS AND DISCUSSION

Software analysis results of the proposed connections were presented in diagrams showing lateral load-displacement of the top of the column, energy dissipation, lateral resistance and lateral stiffness versus lateral displacement. Results were compared to an equivalent monolithic connection in identical conditions.

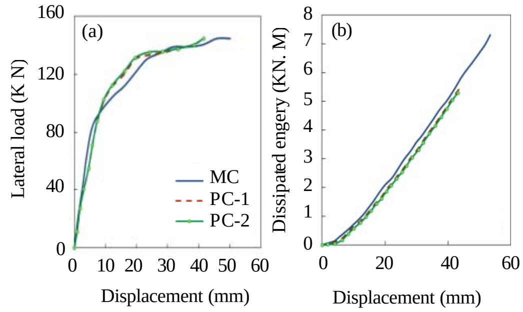


Fig. 8: Lateral load-displacement and energy dissipation curves of specimens under compressive load of 0.2 f_c A_g: a) Lateral load capacity and b) Energy dissipation)

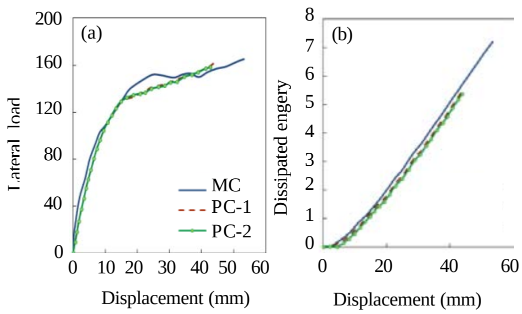


Fig. 9: Lateral load-displacement and energy dissipation response of specimens under compressive load of 0.5 f_c A_g: a) Lateral load capacity and b) Energy dissipation)

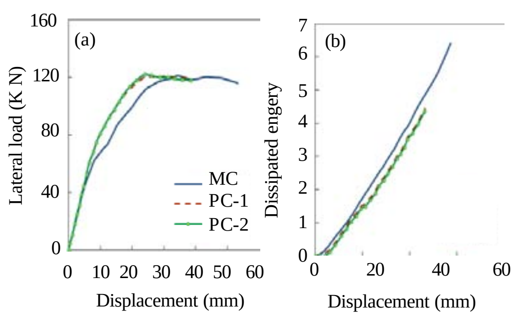


Fig. 10: Lateral load and energy dissipation versus displacement of specimens under tensile load of 0.5 f_y A_s: a) Lateral load capacity and b) Energy dissipation

Figure 8-10 show load-displacement curves and energy dissipation of two way connections under various axial loads including comparison to an equivalent monolithic connection. Initial lateral stiffness of the connection was equal to the graphed slope load displacement at the beginning of the load-displacement curve. Energy dissipation of the investigated system was

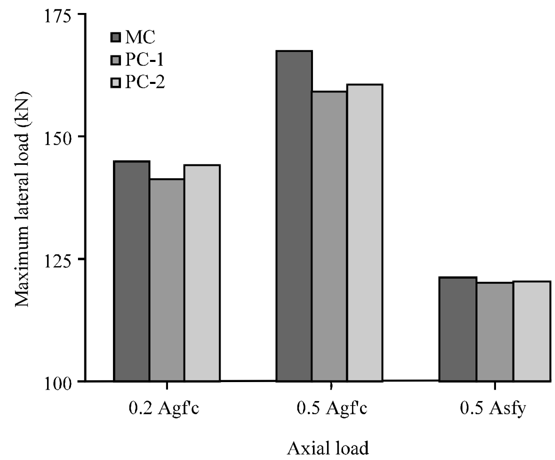


Fig. 11: Lateral stiffness of specimens under various axial loads

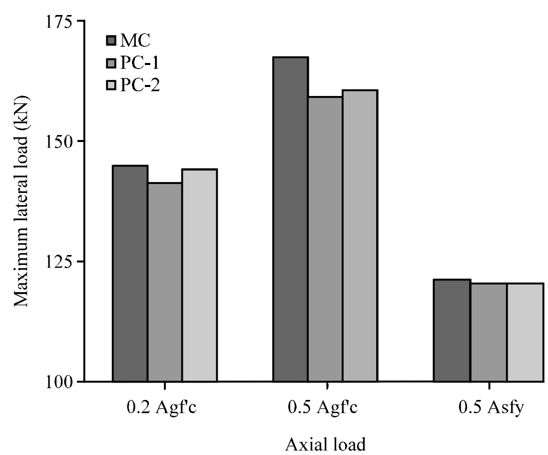


Fig. 12: Lateral resistance capacity of beams under various axial load levels

equal to the surface under the load-displacement graph. Results indicated that lateral resistance and stiffness of the system increased when the axial load of the column increased as demonstrated by a decreased number and severity of cracks. A change in axial load to tensile load significantly decreased the mentioned parameters. A comparison of results showed that the precast connections demonstrated less sensitivity to change of axial load compared to monolithic connections. The cracking distribution in specimens PC-1 Changes in lateral stiffness and lateral resistance of connections are shown in Fig. 11 and 12. Lateral resistance and stiffness of precast connections showed acceptably similar results to an equivalent monolithic connection. Because modeling and analysis of concrete compressive damage is difficult, the compressive strain standard was considered to be 0.0038 for the end of analysis. Therefore, calculation of ductility and energy dissipation was conducted based on that suppose and methods proposed by Park (1989). However, results achieved from laboratory investigations will be different. Increased compressive load caused >20% increase in energy dissipation of precast connections.

CONCLUSION

The present study investigated the performance of two proposed precast connections of precast beam to column connections. The proposed connections were analyzed under lateral load and various levels of axial force using nonlinear finite element model developed using ABAQUS. The following conclusions were drawn from this study.

According to axial load of columns, the proposed connections showed lateral resistance between 120-167 kN, up to 5% more than an equivalent monolithic connection.

Lateral stiffness, ductility and energy dissipation of the connections increased with increased compressive load of the column; however, sensitivity of the precast connections was less than the monolithic connections.

Connections weakened when tensile load was applied. Therefore, the decreasing effect of tensile load should be considered when designing structures and their connections.

Regardless of the limitations of software ABAQUS, it can simulate the mechanical behavior of concrete materials. Although, modeling many parameters in the software is not possible, study results offered an appropriate and acceptable estimation of proposed connection behavior.

Stiffness ratio of the precast connections to monolithic connections ranged between 0.8-0.9 depending on the axial load level of the column.

The difference of lateral stiffness and resistance of the proposed connections compared to an equivalent monolithic connection could be attributed to the difference of force transfer mechanism and position of the connection source(corbel presence, manner of force transfer of bars, grouted connection source, etc).

Ductility and energy dissipation of the proposed connections were measured at approximately 70-80% of an equivalent monolithic connection. The final strain standard for the end of the analysis was considered to be 0.0038.

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