Experimental and numerical evaluation of proposed precast concrete connections

In this article, the cyclic performance of an innovative precast beam-to-column connection is evaluated experimentally and numerically. Two full-scale beam-column cross-shape interior connection specimens named PF-1 and PF-2 are tested. By adding extra nuts to the connecting bolts, specimen PF-2 behaves in a more shear-dominant pattern and shows less pinching. Comparison of performance of these specimens with the numerical monolithic model in terms of stiffness, strength, ductility and energy dissipation capacity indicates the proposed system can provide conditions close to the monolithic connection. However, to reduce the pinching drawback, a minor modification was made leading to performance improvements in strength and equivalent viscous damping ratio up to 51 % and 29 % respectively. The results of this study have direct industrial relevance and may be used for the development of reliable seismic guidelines for precast concrete structures.

Keywords: Precast concrete connection, cyclic behaviour, experimental evaluation, numerical analysis, pinching, ductility and energy dissipation capacity.

1 Introduction

Past earthquakes throughout the world have taught valuable lessons and provided unique experience for engineers to improve construction methods by introducing new systems and techniques. In the seismic design of buildings, it is expected that the system will tolerate large successive cyclic displacements during earthquakes without experiencing collapse. Due to concentration of plastic hinges at the beam-to-column connections in the moment frames, these areas are the most susceptible locations for failure. Even though they have considerable ductility, loss of lateral stiffness of the building due to excessive plastic hinges and transferring from frame action to truss action results in large displacement which may cause collapse of the system, especially in tall buildings. Considering improper supervision of construction in some countries, which results in poor and unexpected seismic performance of reinforced concrete (RC) buildings, the reliability of these elements is very crucial. Simplicity of construction is another issue, which reduces the chances of negative effects of workmanship to a minimum. So proposing a more feasible connection system with acceptable seismic performance is of utmost importance in such countries.

Some past researchers in the realm of precast connections have mainly concentrated on providing a system identical to the monolithic connection. Ersoy and Tankut [1] carried out cyclic tests on seven full-scale dry-joint precast connections and stated that side plates are necessary in order to obtain performance close to monolithic connections. Khaloo and Parastesh [2] investigated the cyclic performance of four precast and one monolithic internal beam-column connections with the scale of 1 : 2.5. They concluded that extending the connection length reduces flexural cracks leading to performance improvement. The results indicated their proposed connection is almost the same as the monolithic one. Shariatmadar and Zamani Beydokhti [3] performed cyclic tests on three full-scale beam-column specimens with cast-in-place connection. They concluded that strength degradation was more pronounced in precast specimens than monolithic ones. The precast specimens also showed some pinching, i.e. necking of the cyclic force-displacement diagram partially due to the bond-slip of the rebars, which is not a desirable response characteristic.

Apart from these differences, other parameters of precast and monolithic specimens were close. Nakaki et al. [4] studied the performance of a precast and a monolithic connection by cyclic testing. In their experiments, the precast connection showed pinching but other behavioural factors were the same in monolithic and precast specimens. Restrepo et al. [5] performed several cyclic tests on precast systems including two beam-column connections. They reported good strength, ductility and energy dissipation capacity of the precast systems. Two full-scale cyclic tests were conducted by Alcocer et al. [6] on precast connections. Although pinching occurred in the hysteretic loops of the precast specimen, acceptable performance was achieved by precast connections with the strong column-weak beam concept.

Xue and Yang [7] studied the behaviour of different types of connections by performing cyclic tests on four full-scale connections and one half-scale frame. They also followed the strong column-weak beam concept and...
reported acceptable strength, ductility and energy dissipation capacity of the system. Moreover, the hysteretic results of their tests showed no pinching. Pillai and Kirk [8] conducted eleven full-scale cyclic tests on their proposed ductile moment-resisting precast connection. The tests results indicated that the proposed method developed adequate strength, stiffness, and ductility. Bhatt and Kirk [9] performed experimental analysis on a welded moment-resisting precast connection and showed the system can withstand large ductility demands. Seckin and Fu [10] also studied semi-rigid precast beam-column connections by performing four full-scale tests. The results of their work showed that the system exhibits acceptable performance in terms of ductility and energy dissipation capacity. Priestly et al. [11] and also Nakaki et al. [12] performed shaking table tests as the culmination of the PRESSS (Precast Seismic Structural Systems) research program on a 60% scale five-story precast/pre-stressed concrete building. The results of their experiments indicate that their proposed connection can tolerate 4.5% drift ratio with little pinching. Although there have been several research projects on precast concrete connections, few were successful in reducing the pinching phenomenon.

There are also some useful documents which encourage good practice in the design of structural connections in precast concrete structures through the introduction of philosophy in the design of structural connections in the serviceability and ultimate limit states and in accidental/abnormal design situations. Moreover, constructional details, durability and maintenance have been discussed in this reference, fib 45 [13]. Another good example of these documents is the state-of-the-art and practice report fib 27 [14], which intends to promote safe and economical applications of structural precast concrete among designers and constructors by providing and also allowing modernization in design and construction.

This research project aims at proposing a precast connection while maintaining good level of energy dissipation and strength compared to monolithic connection, making large-scale construction faster, more feasible and with higher quality. The aim of this study is to evaluate the effectiveness of innovative precast connections in RC buildings. It is noteworthy that a considerable number of RC buildings throughout the world have been built according to the old versions of the seismic codes, which cannot fully meet structural demands. Taking this into account, the proposed connection can safely concentrate cracks in the connection area while maintaining an acceptable level of strength compared to a monolithic connection.

2 Connection specifications

In recent years, several attempts have been made in order to propose a new precast connection system (Korkmaz and Tankut [15], Choi et al. [16], Parastesh et al. [17]). In this study, the seismic performance of a newly proposed precast connection system under cyclic loading is evaluated experimentally and numerically. Fig. 1 shows the details of the proposed connection, in which precast beams containing the stirrups are assembled in the frame and connected to the columns through a box-shaped connection system. This box-shaped connection zone is made of a wide-flange IPB240 cut into a U-shape section and three welded plates. This box-shaped part is anchored in the beam by two Ø6 rebars at the top surface. On one side of this box, there are four holes, through which the Ø20 longitudinal rebars of the beam are anchored. On the opposite side, there are two 32 mm holes for M-24 Type 9-10 bolts. These bolts also pass through the existing holes in the column and therefore connect the ends of the two adjacent beams on each side of the column. Type 9-10 means that the yield and ultimate strength of the steel of the bolts are 900 MPa and 1000 MPa respectively. The upper rebars of the beam also pass through the predefined holes in the column, so these rebars are continuous in the vicinity of the connection from one beam to the next.

After assembling the beams and the column, high strength grout (Master Flow 515) is poured in the beam-to-column connection and the concrete is also cast in the upper part of the beam. It is recalled that 50 cm of the beam section is manufactured and the rest of 15 cm is cast in-situ. The grout used in the connection area is cement-based Master flow 515. The length of the box-shaped connection system, which is the distance between column face to the end of the perforated section of the beam, is designed according to the recommendation of 7% of the beam span (Khaloo and Parastesh [2]).

3 Experimental program

3.1 Test specimens and material properties

The tests were conducted at the Structural Laboratory of the Building and Housing Research Center of Iran (BHRC) on two full-scale cross-shaped specimens as shown in Fig. 2. There are two specimens called PF-1 and PF-2, which are identical to each other except for PF-2 having two nuts on the M-24 bolts, which were welded to the bolts; specimen PF-1 only has a single nut on each bolt without welding. The dimensions of different parts of the specimens and the material properties are shown in Table 1.

The test specimens represent a beam-column connection of a typical four-story building located in areas with very high seismicity, and are designed according to Iranian National Building Codes and also ACI and PCI [43] codes with the aid of SAP2000 [42]. Fig. 3 shows the configuration of the proposed connection zone in the test specimen.

3.2 Test setup, instrumentation and loading protocol

Fig. 4 shows the test setup and instrumentation configuration for the specimens. The two ends of the column are hinged and the boundary condition of the beam at both ends is rolled. The cyclic displacement is applied to the upper end of the column. As shown, the displacements of important parts of the specimens are recorded by displacement sensors. The Linear Variable Differential Transformers (LVDTs) are installed at the locations where great non-linearities were anticipated and the string pots are installed elsewhere. The loading protocol is according to FEMA 461 [18] and a view of specimen PF-1 under cyclic testing is shown in Fig. 5.
Fig. 1. Details of the proposed connection (m)

Fig. 2. Geometrical characteristics of test specimens (cm)
Therefore the main failure concentrates in the connection system and there is negligible damage to concrete parts of the connection, indicating that the potential capacity of the specimen has not been exploited. The above-mentioned modification to specimen PF-2 improves the behaviour of the model and the crack pattern on the connection area changes to a more inclined one, confirming a more pronounced shear failure mode, as shown in Fig. 7.

The load-displacement curve of the test specimens is shown in Fig. 7. As observed, doubling the number of nuts in the connection system in specimen PF-2 achieves a relative improvement in terms of the energy dissipation ca-

### Table 1. Dimensions and material properties of different parts of the test specimens

<table>
<thead>
<tr>
<th>Member</th>
<th>Section dimensions</th>
<th>Rebar</th>
<th>Stirrup</th>
<th>Concrete strength (Mpa)</th>
<th>Steel tensile strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>40 × 40 cm</td>
<td>12φ20</td>
<td>φ10@7.5 cm</td>
<td>48.1</td>
<td>AIII (Grade 3)</td>
</tr>
<tr>
<td>Beam</td>
<td>45 × 40 cm</td>
<td>Top: 4φ20</td>
<td>φ10@7.5 cm</td>
<td>34.2</td>
<td>AII (Grade 2)</td>
</tr>
<tr>
<td></td>
<td>Bot: 4φ20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 3.3 Experimental results

Fig. 6 shows the failure mode of specimen PF-1. As the crack pattern indicates, this specimen failed in flexure, which results in detachment of the beams from the column. This behavior originates from cutting off the nuts shown in Fig. 3 from the connection system, leading to premature failure of the specimen, which accompanies the pinching phenomenon. This abrupt failure of the nuts occurred at load 176 kN, which is less than the expected value because of the premature failure mode compared to the intended one, yielding of longitudinal rebars in the beam. Therefore the main failure concentrates in the connection system and there is negligible damage to concrete parts of the connection, indicating that the potential capacity of the specimen has not been exploited. The above-mentioned modification to specimen PF-2 improves the behaviour of the model and the crack pattern on the connection area changes to a more inclined one, confirming a more pronounced shear failure mode, as shown in Fig. 7.

The load-displacement curve of the test specimens is shown in Fig. 7. As observed, doubling the number of nuts in the connection system in specimen PF-2 achieves a relative improvement in terms of the energy dissipation ca-
capacity. Moreover, there is a 15% increase in strength of the system and the curve is more symmetric, which indicates a more stable seismic response during successive cyclic displacement demand during earthquakes. This improvement of the strength and energy dissipation capacity of specimen PF-2 directly originates from the prevention of premature failure in the connection zone i.e. failure of the nuts as observed in specimen PF-1.

Both specimens exhibit pinching behaviour, which is not considered a desirable seismic response characteristic. It is noteworthy that no failure in the nuts occurs in specimen PF-2. However, like specimen PF-1, detachment of the beams from the column occurs during load application. As indicated in the next section, this failure mode results in pinching of the specimen, which is common in precast connections. Since detachment of the beams from the column necessitates considerable deformation in the connection zone, damage is accumulated in the box-shaped connection system, which prevented exploitation of the specimen’s full capacity. Therefore, the proposed connection was modified and the details and the results of this modification are presented in the next sections of this paper.

4 Numerical analysis

The experimental results indicate an undesirable failure mode of the proposed connection and hence the necessity of modifications to the system. The importance of a calibrated numerical model in this study is to perform numerous analyses without any cost. The assumptions and results of the numerical simulation are presented in this section.

There are several existing numerical studies on the behaviour of precast concrete structural systems (Hawileh et al. [19] and Moyo et al. [20]). The numerical simulation is performed using the general-purpose finite element program Abaqus 6.9 [44]. The concrete and the connection parts are modelled with continuum hexahedral, first or-

![Damage to experimental specimens](image)

(a) Detachment of the beams from the column (PF-1)
(b) Detachment of the nuts from the bolt (PF-1)
(c) Inclined shear cracks (PF-2)

Fig. 6. Damage to experimental specimens

![Load-displacement curve of the experimental specimens](image)

(a) PF-1
(b) PF-2

Fig. 7. Load-displacement curve of the experimental specimens
nder, reduced-integration elements and the rebars are simulated by first-order truss elements. The use of first-order reduced-integration elements is based on the sensitivity of the concrete part to the results of large element distortions. These elements, however, are susceptible to an “hour-glassing” phenomenon, in which the flexural stiffness of the system is dramatically under-predicted. To eliminate this problem, the size of elements is reduced further than the state of mesh convergence.

Drucker-Prager plastic flow (Drucker and Prager [21]) with Lubliner yield function (Lubliner et al. [22]) are used for the concrete. For unconfined concrete and confined concrete in the stirrups, Kent and Park model and Mander model are used respectively (Kent and Park [23] and Mander et al. [24]). Compressive and tensile damage to the concrete is also considered according to Eqs. (1) and (2). This damage controls the slope of the unloading branch of the stress-strain curve for the concrete. It is noteworthy that the tensile damage to the concrete has no effect on the compressive behaviour, while compressive damage to the concrete has full influence on concrete in the tensile phase.

\[
D_c = 1 - \frac{\sigma_e}{f'_c} \quad \text{and} \quad D_t = 1 - \frac{\sigma_t}{f'_t} \tag{1, 2}
\]

The behaviour of the steel members is assumed to be elastic-perfectly-plastic with the hardening phase for St-37 and without hardening for rebars (AIII) and stirrups (AII) with the Von-Mises plastic flow (Von Mises [25]). Table 2 presents the mechanical properties of concrete and steel in the numerical models. Values used for other parameters of the numerical model are based on the applied behavioural model and calibration process considering recommended ranges.

It is noted that successive opening and closure of cracks play an important role in pinching behaviour. Since Concrete Damaged Plasticity (CDP) material is incapable of rigorous simulation of the aforementioned behaviour, an attempt was made to account for pinching by setting a small value for stiffness recovery factor in compression. According to Fig. 8(a), \(\psi_c\) and \(\psi_t\) values are set to 0.01 and 0.5 respectively. The former parameter controls stiffness recovery from the compressive to tensile regime; while the latter parameter governs the cyclic stiffness recovery from tensile to compressive zone. Although a standard full-cyclic test is required to determine these parameters, it is generally assumed that \(\psi_c\) and \(\psi_t\) are zero and unity, respectively, although, the results of the numerical study are calibrated by assuming \(\psi_c = 0.5\) in this study, mainly because of the problem of simulating cyclic behaviour of the concrete.

The other solution is to introduce non-linear elastic springs as proposed by Ueda et al. [26] between the longitudinal bars and the adjacent concrete nodes. These springs can model the bond-slip behaviour between the bars and concrete, which partially contributes to the pinching phenomenon. The parameters necessary for introducing these springs are shown in Fig. 8(b). Note that the slippage force of these springs connecting each adjacent node of concrete and the bar in longitudinal direction of the bars depends on the surface area and the mesh size of the model.

The size of the mesh in different parts of the numerical model is decided after performing mesh sensitivity analyses. These analyses are performed on the model in order to reach the state of “mesh convergence”. It is noteworthy that areas with the highest strain gradient are the most critical, in which mesh should be locally refined. The final mesh state is depicted in Fig. 9. Considering the highly non-linear behaviour of the model, an explicit approach is used making use of a central difference numerical solver. In this approach, loads and boundary conditions are exerted “slowly” in order to provide a “pseudo-static” nature of the experimental conditions. The criterion for determination of the “pseudo-static” state is the ratio of kinetic energy of the model to its internal energy, which should be lower than 10% at each increment.

As observed in Fig. 10, the main failure mode in specimen PF-2 is tensile failure of the connection, which causes partial detachment of the connection system from the beam and hence the beam from the column. This phenomenon itself, which is caused by the connection failure, is the origin of the pinching behaviour of the hysteretic curve. Therefore the load-bearing mechanism in the connection in reverse loading is affected and pinched. Also the crack pattern of the numerical model representing PF-2 is shown in Fig. 11, which is in accordance with the experimental results. Fig. 12 compares the hysteresis curve of the numerical model and experimental specimen. The results suggest

**Table 2. Mechanical properties of concrete and steel in the numerical models**

<table>
<thead>
<tr>
<th></th>
<th>Concrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\rho \left( \frac{kg}{m^3} \right))</td>
<td>E (GPa)</td>
<td>(f'_{co} ) (MPa)</td>
</tr>
<tr>
<td>Beam</td>
<td>2450</td>
<td>22.3</td>
</tr>
<tr>
<td>Column</td>
<td>29.4</td>
<td>7850</td>
</tr>
</tbody>
</table>

**References**

4.1 Modification to the proposed connection

In an attempt to improve the behaviour of the proposed connection in terms reducing the pinching phenomenon, the connection system is altered from a box-shaped to two half-cylinder parts, as shown in Figs. 14 and 15. This cylindrical part is 12 mm in thickness and has two caps, each with 20 mm thickness. The length and radius of the half cylinder are 240 mm and 90 mm respectively. The connecting rod is 32 mm in diameter and 50 mm in length. These half cylinders are attached to the manufactured part of the beams by two embedded rebars welded to them. The capacity of the connection considering yielding of the two 22 mm rebars is 320 kN, considering an over-strength factor. Pinching is considerably reduced by this connection specification, mainly due to the higher tensile capacity and ductility of the half-cylinder parts compared to box-shaped part in specimens PF-1 and PF-2. In fact, it postpones connection failure and hence detachment of the beams from the column, leading to opening and closing of cracks which could result in pinching. In order to ensure improvement in the performance of the system, universal tests were performed on the modified part. The test specimens and the failure mode of the modified part are shown in Fig. 16. Fig. 17 shows the results of the tensile tests on the modified connection system. As can be seen, the failure mode of the models is shear-punching...
and their capacity is well beyond the design capacity. Although the main intention of these tests was the determination of the capacity of the connection system itself (replacing the two 22 mm rebars with two M22 Type 9-10 high strength bolts in the specimens), which is shear-punching. In reality, the failure model of the connection zone is yielding of longitudinal rebars in the beam (related to 320 kN of force capacity) and hence the shear-punching (related to 400 kN of force capacity as observed in the tests) is not the dominant failure mode. Also, the modified parts show ductile behaviour and the small drop in the first phases of loading originates from the bolts adjustment in the half-cylinder part.

The crack pattern of the modified connection is shown in Fig. 18. As observed, more pronounced shear failure associated with inclined cracks in the connection zone occurs and accumulation of these cracks causes more energy absorption, which postpones the flexural failure mode. The flexural mode is directly related to detachment of the beams from the column, opening of gaps and cracks and their closing without requiring considerable force in reverse cyclic loading which is associated with the

Table 3. Behavioural characteristics of the studied models

<table>
<thead>
<tr>
<th>Model name</th>
<th>Initial secant stiffness</th>
<th>Secant stiffness at strength capacity</th>
<th>Secant stiffness at 8.7% drift</th>
<th>Strength capacity (kN)</th>
<th>Ductility</th>
<th>Energy dissipation capacity at 8.7% drift (kJ)</th>
<th>$\xi_{eq,8.7%}$ (%)</th>
<th>a</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monolithic</td>
<td>8.25 8.08</td>
<td>2.33 2.38</td>
<td>1.37 1.32</td>
<td>296 304</td>
<td>4.7 3.9</td>
<td>98.68</td>
<td>6.2</td>
<td>0.91</td>
<td>0.80</td>
</tr>
<tr>
<td>PF-1</td>
<td>4.27 4.61</td>
<td>2.38 2.36</td>
<td>0.74 0.93</td>
<td>202 172</td>
<td>5.1 5.4</td>
<td>57.73</td>
<td>5.5</td>
<td>0.80</td>
<td>0.84</td>
</tr>
<tr>
<td>PF-2</td>
<td>4.17 4.45</td>
<td>1.61 2.02</td>
<td>1.03 1.10</td>
<td>237 216</td>
<td>5.3 4.8</td>
<td>76.60</td>
<td>5.2</td>
<td>0.89</td>
<td>0.65</td>
</tr>
<tr>
<td>Modified PF-2</td>
<td>15.92 15.48</td>
<td>2.29 1.75</td>
<td>1.72 1.75</td>
<td>358 354</td>
<td>5.4 5.1</td>
<td>145.79</td>
<td>6.7</td>
<td>0.95</td>
<td>0.69</td>
</tr>
</tbody>
</table>
pinching phenomenon, as observed in specimens PF-1 and PF-2. The hysteresis curve of the numerical model with modified connection is depicted in Fig. 19. It is obvious that the performance of the specimen has improved dramatically, indicating the effectiveness of the proposed modified connection system. The behavioural characteristics of the studied models are presented in Table 3. In this table, the initial secant stiffness has been calculated as the slope of the line from the origin to the points on the force-displacement diagram related to first yielding or first major cracks, which is associated with the first reduction in the slope of the diagram. The line slope from the origin to the point related to the peak force in the diagram is defined as the secant stiffness related to strength capacity. Also the line slope from the origin to the point related to drift 8.7%, as the final exerted drift ratio, is called the secant stiffness at 8.7% drift. The ductility is calculated by idealizing the diagram with an elastic-perfectly plastic curve. The maximum displacement of the new curve is equal to that of the initial diagram, while the yield point is determined by assuming equal areas under both diagrams. The energy dissipation capacity is calculated from the area enclosed in last cycle (average of positive and negative) of the hysteretic curve. In order to normalize this parameter for comparison purposes, the equivalent viscous damping ratio based on Eq. (3) (Chopra [27]) is used and the results are presented in Table 3. Also the relationship of secant stiffness degradation to drift ratio can be evaluated using Eq. (4), which can be found in other forms in Tomažević.

According to the results, the ductility of the precast connections is greater than that of the monolithic one. Although models PF-2 Modified and Monolithic show the greatest stiffness degradation in the first steps of loading, both models maintain their stiffness in the last loading steps; while the opposite conditions apply for specimen PF-1. The improvement in energy dissipation capacity in the modified model PF-2 is greater than strength capacity with 92% increase compared to 51% increase. This increase in the energy dissipation capacity is mainly due to yielding of the modified part in the mid steps of loading, which causes stable and large hysteretic loops; however, no increase in equivalent ductility demand of the modified model PF-2 is noticeable. Also no obvious improvement is achieved in ductility for the modified model PF-2.

The results of the ductility are very close to the previous experimental studies (Ersoy and Tankut [1] and Xue and Yang [7]). However, this parameter in the present study is slightly less than ductility factors of Khaloo and Parastesh [2]. This originates from the fact that the monolithic specimen also showed greater ductility than the monolithic model in this study. While there is no considerable increase in ductility in their precast connection, an improvement in ductility of the precast specimens has been achieved in this study. The energy dissipation capacity is also comparable with that of Xue and Yang [7]; while in this study the main focus was on the last loading steps. Another point is the displacement ductility capacity of the specimens in the present work compared with the similar previous studies; the specimens can tolerate up to 9% of inter-story drifts (neglecting non-structural performance and other considerations), while the average ultimate drift for other works are 4% (Khaloo and Parastesh [2] and Xue and Yang [7]) and 5% (Shariatmadar and Beydokhti [3]).
4.2 Determination of the Connection Rigidity

In order to determine the rigidity of the connection, \( a = \frac{k_f}{EI} \) criterion is used; where \( k_f = \frac{M}{\theta} \), \( M \) and \( \theta \) are moment and rotation respectively due to service loads (Fig. 20(a)). The service loads include both dead loads and live loads, which are determined according to Iranian National Building Codes, Loads in Buildings Part 6-1385 [30]. \( L \) and \( EI \) are length and flexural stiffness (multiplied by \( L^3 \)) of the beam. If “\( a \)” is greater than 20, the connection is rigid and if “\( a \)” is smaller than 2, the connection is flexible. The connection is semi-rigid and its stiffness, strength and ductility should be considered in design if “\( a \)” is between 2 and 20 based on AISC [31].

In the numerical analysis, rigidity of connection is determined form the slope of the moment-rotation curve of the model. Fig. 20(b) shows the moment-rotation diagram of the proposed modified connection system. The modified connection rigidity based on this figure is presented in Table 4.

4.3 Behavior Factor of the Proposed Connection

There are several formulas for determination of the behaviour factor; some of them are more practical and others are more sophisticated and rarely used (BSSC [32], Whitaker et al. [33], Uang [34], ATC-19 [35], Mwafy and Elnashai [36], EN 1990 [41] and NRC [37]). In this study, the behaviour factor of the proposed connection is calculated according to Eq. (5) (ATC-34 [38]):

\[
R = R_\mu \Omega
\]

where \( R \) is the behaviour factor, \( R_\mu \) is the behaviour factor due to ductility which is a function of ductility and natural period of the system, and \( \Omega \) is the over-strength factor. The natural period of the system is calculated using Eq. (6) based on Iranian National Building Codes, Design of Concrete Structures [39]:

\[
\text{Table 4. Calculation of the connection rigidity}
\]

<table>
<thead>
<tr>
<th>( L ) (m)</th>
<th>( f'_c ) (MPa)</th>
<th>( E = 4700\sqrt{f'_c} )</th>
<th>( I ) (cm(^4))</th>
<th>( M_s ) (kN.m)</th>
<th>( \theta_s ) (rad)</th>
<th>( K_S = \frac{M_S}{\theta_S} ) (kN.m)</th>
<th>( a = \frac{K_S L}{EI} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.6</td>
<td>30 (4.2)</td>
<td>25742</td>
<td>56731</td>
<td>100</td>
<td>0.001</td>
<td>( 1 \times 10^3 )</td>
<td>24.65</td>
</tr>
</tbody>
</table>

Fig. 16. The modified part under tensile loading

Fig. 17. Force-elongation of the modified parts

Fig. 18. Crack pattern of the modified connection
of the ductility of the system are derived from performing a push-over analysis (see Fig. 21) on a single-bay, single-storey frame with the proposed connections. Finally, the parameters related to calculation of the behaviour factor of the proposed system are shown in Table 5.

![Fig. 19. Hysteresis curve of the numerical model with modified connection](image1)

![Fig. 20. Moment-rotation diagram of the proposed modified connection system](image2)

![Fig. 21. Push-over curve of the proposed modified system](image3)

Fig. 19. Hysteresis curve of the numerical model with modified connection

**Fig. 20.** Moment-rotation diagram of the proposed modified connection system

**Fig. 21.** Push-over curve of the proposed modified system

\[ T = 0.07H^{3/2} \]  

(6)

where \( H \) is the total height of the system in m. Miranda and Bertero [40] proposed a formula for determining the behaviour factor due to ductility shown in Eq. (7):

\[
R_\mu = \mu - \frac{1}{\phi} + 1 \geq 1
\]

(7)

where \( \phi \) is a function of the ductility and natural period of the system which is calculated based on Eq. (8) for hard rock soil:

\[
\phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} \exp \left( -\frac{3}{2} \left( \ln T - \frac{3}{5} \right)^2 \right)
\]

(8)

The over-strength factor, which is based on Eq. (9), is the ratio of the force related to the first yielding \( (C_y) \) and the force related to first major yielding in the system \( (C_s) \).

\[
\Omega = \frac{C_y}{C_s}
\]

(9)

### 5 Implementation and Use

The proposed system has been formally accepted under the title of “Iran Frame Co. Semi Precast RC System” by the “Building and Housing Research Center of Iran (BHRC)” with the patent No. 72815. This centre is the only reference in Iran for patents and recognizes that the new structural system permits implementation of the proposed system with shear wall in buildings up to 10 stories. It is worth mentioning that the proposed system is now being implemented in mass construction throughout the country. The initial experimental results indicate that the new system can act as rigid flexural frame up to 4 stories (Fig. 22).

<table>
<thead>
<tr>
<th>H (mm)</th>
<th>( T_\delta ) (s)</th>
<th>( K = \frac{K_N}{mm} )</th>
<th>( C_y ) (kN)</th>
<th>( C_s ) (kN)</th>
<th>( \Delta_y ) (mm)</th>
<th>( \Delta_m ) (mm)</th>
<th>( \mu )</th>
<th>( \phi )</th>
<th>( R_\mu )</th>
<th>( \Omega )</th>
<th>( R )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3000</td>
<td>0.16</td>
<td>65.0</td>
<td>800</td>
<td>420</td>
<td>12.2</td>
<td>68.0</td>
<td>5.6</td>
<td>2.0</td>
<td>3.3</td>
<td>1.9</td>
<td>6.3</td>
</tr>
</tbody>
</table>
An investigation is underway to ensure the seismic performance of the system and make necessary modifications.

6 Summary and Conclusions

In this study, the performance of a proposed precast connection is evaluated by experimental and numerical analysis. It is shown that the connection can act as a sufficiently rigid connection. While the behaviour characteristics of the proposed connection, including the ultimate strength and ductility, are comparable to a numerically-simulated monolithic model, exactly duplicating the proposed specimen, the specimen suffered from a pinching phenomenon. Since in the first specimen, the failure was due to detachment of the nuts from the bolt in the connection area, there was a 15% increase in the strength of the second specimen through the doubling of the number of nuts. Moreover, construction of moment frame buildings using the proposed connection with consideration of the moment demand of the connection is simple. The modified connection exhibits stable hysteretic curves, hence reducing the required sections of the structural elements and postponing the failure mechanism. The results show the moment capacity of the connection system is maintained at 250 kNm up to a rotation of 0.02 rad. It is however worth mentioning that the results related to the modification of the proposed connection can only prove comparative improvement of the performance of the connection; i.e. a precise justification of the aforementioned results can only be made by performing further experimental studies on the modified connection.

Acknowledgments

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Notation

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>α and β</td>
<td>coefficient derived from regression analysis of secant stiffness and displacement of specimen at each cycle</td>
</tr>
<tr>
<td>ρ</td>
<td>mass density</td>
</tr>
<tr>
<td>E</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>ν</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$d_{c}$, $D_{t}$</td>
<td>compressive and tensile damages, respectively</td>
</tr>
<tr>
<td>$d_{Hmax}$, $d_{Hmax, i}$</td>
<td>maximum displacement at the i-th cycle</td>
</tr>
<tr>
<td>$d_{Hult}$</td>
<td>maximum displacement related to the maximum force</td>
</tr>
<tr>
<td>$E_{Hult}$</td>
<td>maximum displacement at the last cycle hysteretic dissipated energy of the last cycle compressive and tensile strength of concrete respectively</td>
</tr>
<tr>
<td>$f_{c}$, $f_{t}$</td>
<td>concrete compressive strength in unconfined and confined state respectively</td>
</tr>
<tr>
<td>$k_{s}$</td>
<td>secant stiffness related to the maximum force</td>
</tr>
<tr>
<td>$k_{s,i}$</td>
<td>secant stiffness at the i-th cycle</td>
</tr>
<tr>
<td>$\epsilon'_{cc}$</td>
<td>secant stiffness related to the last cycle strain related to concrete compressive strength in unconfined and confined state, respectively</td>
</tr>
<tr>
<td>$\psi_{co}$, $\psi_{cc}$</td>
<td>dilation angle of concrete in unconfined and confined states respectively</td>
</tr>
<tr>
<td>$f_{y-st37}$, $f_{u-st37}$, $f_{u-AIII}$</td>
<td>yield strength of St-37 steel strain related to ultimate strength of St-37 ultimate strength of St-37 and AIII steel, respectively</td>
</tr>
<tr>
<td>$\sigma_{c}$, $\sigma_{t}$</td>
<td>compressive and tensile stresses, respectively</td>
</tr>
</tbody>
</table>

References


