Article:

Parastesh, H., Hajirasouliha, I. and Ramezani, R. (2014) A new ductile moment-resisting connection for precast concrete frames in seismic regions: An experimental investigation. Engineering Structures, 70. 144 - 157. ISSN: 0141-0296

https://doi.org/10.1016/j.engstruct.2014.04.001

Parastesh H, Hajirasouliha I & Ramezani R (2014) A new ductile moment-resisting connection for precast concrete frames in seismic regions: An experimental investigation. Engineering Structures, 70, 144-157.

A NEW DUCTILE MOMENT-RESISTING CONNECTION FOR PRECAST CONCRETE FRAMES IN SEISMIC REGIONS: AN EXPERIMENTAL INVESTIGATION

Hossein Parastesh [a], Iman Hajirasouliha [b*], Reza Ramezani [c]

[a] *Department of Civil Engineering, University of Science and Culture, Tehran, Iran*

[b] *Department of Civil & Structural Engineering, The University of Sheffield, Sheffield, UK*

[c] *Department of Civil Engineering, K. N. Toosi University of Technology, Tehran, Iran*

* Corresponding author: i.hajirasouliha@sheffield.ac.uk; Tel: +44 1142225710; Fax: +44 1142225700

ABSTRACT

A new ductile moment-resisting beam-column connection is developed for precast reinforced concrete (RC) frames in high seismic zones. The proposed connection provides good structural integrity in the connections and can reduce construction time by eliminating the need for formworks and welding, and minimizing cast-in-place concrete volume. A series of cyclic loading tests were carried out on six full-scale interior and exterior precast connections and two monolithic connections, all designed for use in high seismic zones. Test variables included the type of stirrups (open and closed) and the stirrup spacing in the beam connection zone. Test specimens were subjected to a constant axial load and a reverse cyclic loading based on a given displacement history. Flexural strength, ductility, strength degradation and energy dissipation capacity of the precast and monolithic connections are compared. The proposed interior and exterior moment-resisting connections proved to be efficient at improving the seismic performance of precast concrete frames in high seismic zones. While the precast connections provided adequate flexural strength, strength degradation and drift capacity, they exhibited considerably higher ductility and energy dissipation compared to similar monolithic specimens.

Keywords: Precast Concrete; Seismic Design; Cyclic Loading; Ductility; Energy Dissipation; Moment-resisting Connections

1- INTRODUCTION

With population growth and increasing demand from higher standards, construction industry is facing new major challenges. Societal and legal pressures mean that the structural systems need to comply with modern requirements such as cost efficiency, fast construction, resilience and mass production. Prefabricated building systems have advantages such as high quality control, fast construction and cost efficiency by reducing the amount of waste material [1]. Therefore, precast buildings are replacing conventional cast-in-place systems, especially by dominance of mass construction in developing countries. However, these structural systems may be vulnerable to strong earthquakes. Extensive damage and catastrophic failure of precast RC structures in major earthquakes was mainly due to failure of joints and inadequate ductility, which highlighted the importance of ductile connections in precast structures [2-5].

Moment-resisting beam-column connections are, in general, suitable systems to maintain lateral stability and structural integrity of the multi-storey precast concrete structures in high seismic zones. Park and Bull [6] conducted full-scale tests on three exterior precast concrete beam- column connections, and showed that the specimens detailed for seismic loads have adequate strength, ductility, and energy dissipation for ductile moment-resisting frames. Similar results were reported by French et al. [7, 8] on three different types of precast concrete connections. The results of their study indicated that the precast connections can provide adequate strength and energy dissipation with respect to monolithic concrete specimens.

Cheok and Lew [9] experimentally investigated the performance of precast concrete beam-column connections subjected to cyclic inelastic loading. The objective of their study was to develop a moment resistant precast connection that is economical and can be easily assembled at site. Similarly, Castro et al. [10] studied the seismic performance of eight two-thirds scale precast beam-column joints and a monolithic specimen with the same beam size. Their results showed that precast connections can sustain large inelastic deformations under cyclic loads, if they are designed properly. Stone et al. [11] developed a new hybrid moment-resisting beam-column connection for precast frames in regions with high seismicity. Their proposed system utilised mild steel bars to improve energy dissipation and post-tensioning clamps to provide adequate shear resistance. The precast connections were designed to have the same flexural strength, energy dissipation and drift capacity of typical monolithic connections. In another study, Priestley et al. [12] reported the satisfactory seismic performance of two un-grouted post-tensioned precast concrete beam-column joint subassemblies under cyclic loading.

SCOPE systems are framed structures consisted of precast prestressed concrete components. Cai et al. [13] showed that these structural systems can provide full hysteretic loops and good energy dissipation capacity under low cyclic and reciprocal loading. Khaloo and Parastesh [14, 15] developed a moment-resisting precast concrete connection for high seismic zones. Experimental tests on eight beam-column interior connections under inelastic cyclic loading demonstrated the good seismic performance of their proposed moment-resisting connections compared to conventional monolithic connections. In a similar research, Ertas et al. [16] presented the test results of four types of ductile moment-resisting precast concrete frame connections and one monolithic concrete connection, all designed for use in high seismic zones. Based on their results, the hysteresis behaviour of the precast specimens was similar to the monolithic connection, and most of the precast concrete connections reached their designed ultimate moment strength capacity.

Kulkarni et al. [17] proposed a new precast hybrid-steel concrete connection, which showed satisfactory flexural performance under inelastic cyclic loading. Based on the results of their study, the thickness of the connecting plate plays an important role in the energy dissipation and deflection capacity of the connection. Xue and Yang [18] reported the results of their experimental investigations on four full-scale precast concrete beam-column connections including an exterior connection, an interior connection, a T connection, and a knee connection. In a recent study, Vidjeapriya and Jaya [19] experimentally investigated the hysteresis behaviour, load carrying capacity, energy dissipation capacity and ductility of two types of precast beam-column connections using J-bolts and cleat angles. It was concluded that beam-column connection using J-Bolt can provide higher ductility (and energy dissipation) compared to similar monolithic specimens, whereas cleat angle connections are less ductile.

Different types of open and closed stirrups (such as multi-leg, U-shape and spliced) can be used as shear reinforcement in RC beams [20]. Varney et al. [21] investigated the effects of stirrup anchorage on the shear strength of RC beams. The results of their study indicate that the reinforcement anchorage does not have a significant effect on the shear capacity of RC beams. However, several studies showed that the shear behaviour and cracking strength of RC beams can be considerably improved by using closed-stirrups and/or decreasing the stirrup spacing [22, 23]. Most of the previous studies concluded that the precast beam-column connections can be detailed as strong as conventional monolithic connections. However, the need for welding, bolting or extensive cast-in-place concrete work for installation of most of the existing prefabricated concrete connections can considerably increase the construction costs and installation time. Moreover, moment-resisting connections in the seismic regions should have adequate ductility to dissipate large amounts of energy and prevent failure under strong earthquakes. In this study, a new ductile moment-resisting connection is developed for precast concrete frames in seismically active regions. The main advantages of the proposed connections compared to the conventional precast connections are speed of construction, lower cost, and reduced need for skilled labour, which are achieved by minimizing cast-in-place concrete volume and eliminating the need for formworks, welding, bolting and prestressing. The proposed detailing can also enhance the flexural strength, ductility and energy dissipation capacity of the connections and provide better structural integrity between beam, column and slab elements. The efficiency of the proposed system is investigated by comparison between the performance of eight full-scale precast and monolithic connections (interior and exterior) under inelastic cyclic loading.

2- DEVELOPED MOMENT-RESISTING CONNECTION

Fig. 1 illustrates details of the developed interior and exterior moment-resisting connections for precast concrete frames. In the proposed system, prefabricated concrete columns are cast continuously in the elevation with a free space in the connection zone to connect beam elements. Four diagonal bars are used in the empty zone of the precast columns (i.e. beam-column joint core) to provide adequate strength and stability during the installation process (see **Fig. 1**). The diagonal bars behave like truss elements and can considerably increase the axial strength of the columns under construction/transportation loads. The shear links used in the connection core (shown in **Fig. 2**) can prevent the buckling of the longitudinal and diagonal bars under the self-weight of the precast columns. However, for the safe transportation and handling of full scale 12m columns, temporary supports (e.g. by using steel profiles) may be required.

In the connection zone, the precast concrete beams have a hollow U shape cross section. A longitudinal bar is used in the precast concrete U section to support diagonal stirrup bars (see **Fig. 1**), which can also provide adequate tensile strength to resist installation loads. The surface of precast members at the U section zone was smooth, and no slippage was observed between the precast members and the grout during the experimental tests. The length of the connection zone (i.e. plastic hinge zone) of the precast beams was calculated to be 600 mm (hollow U shape section in **Fig. 1**).

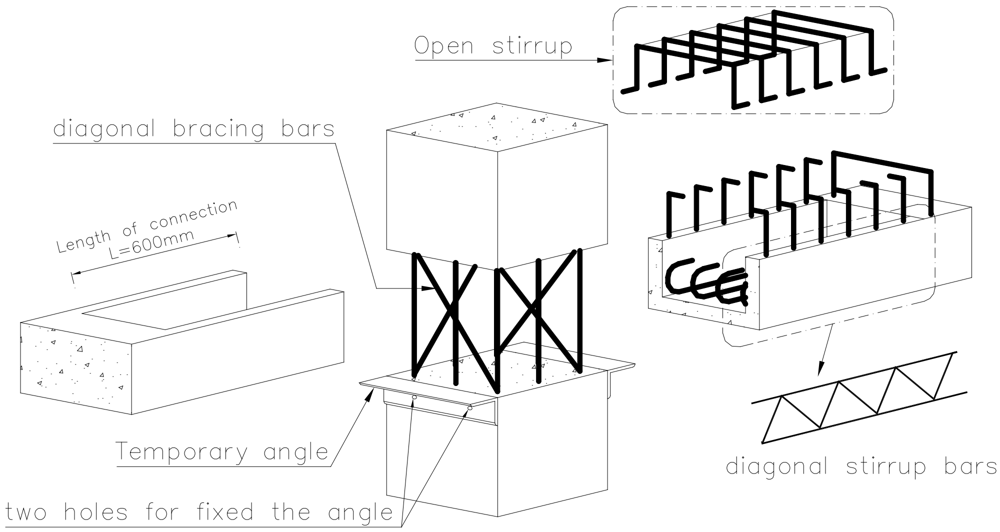

Fig.1 Detailing of reinforcement in the connection zone of the precast elements

After fixing the longitudinal reinforcement bars, the connection zone is filled with cast-in-place concrete to provide a good structural integrity between beam, column and slab elements. The bottom longitudinal reinforcement bars are spliced in the cast-in-place area of the beam**.** The top reinforcement bars are continuous through the beam-column joint (as shown in **Fig. 2)** and are fixed to the precast beams outside the connection zone with a layer of grout. Diagonal stirrup bars and U-shaped anchorages are used to provide enough shear strength before using cast-in-place concrete. The connection region is then grouted to form a moment-resisting beam-column connection. No diagonal bracing bar is used in the joint core of the monolithic specimens.

The proposed precast connection can be easily assembled as it eliminates the need for welding or using mechanical splices for beam longitudinal reinforcement at joints. Temporary supports for beams are provided by means of steel angles on each side of the columns (see **Fig. 1)**. The steel angles provide enough bearing area for sitting the RC beams and transferring the construction loads before in-situ concrete becomes structural. Therefore, in the proposed system, 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.

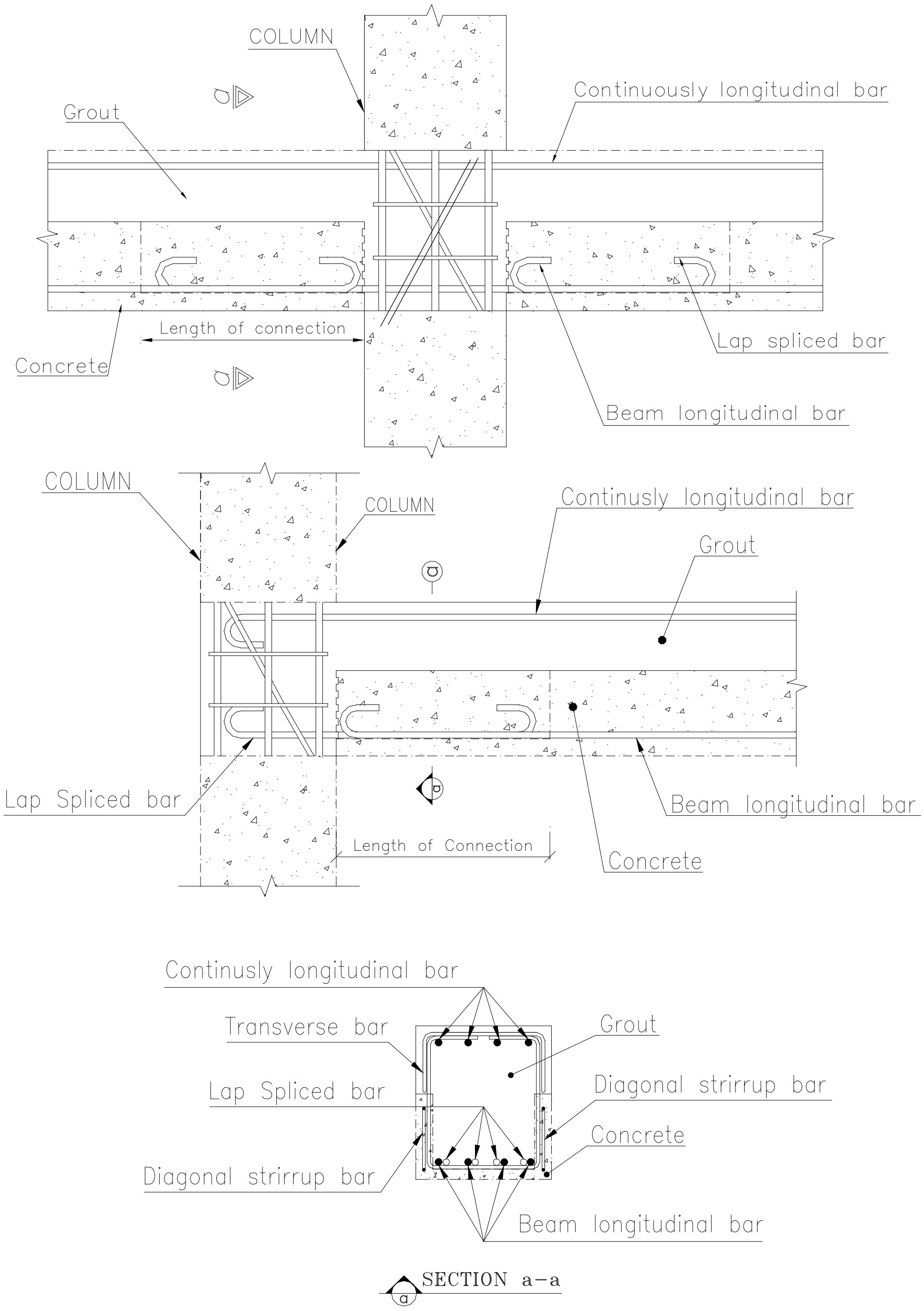

Fig.2 Detailing of interior and exterior precast connections

3- EXPERIMENTAL PROGRAM

3-1 Design Basis

All tested specimens were designed to accommodate loads of a four storey building shown in **Fig. 3**. Connections were designed for lateral loads in both transverse and longitudinal directions. The prototype building has plan dimensions of 37.2×18.6m, column spacing of 4.65m, and storey height of 3.2m as shown in **Fig. 3**.

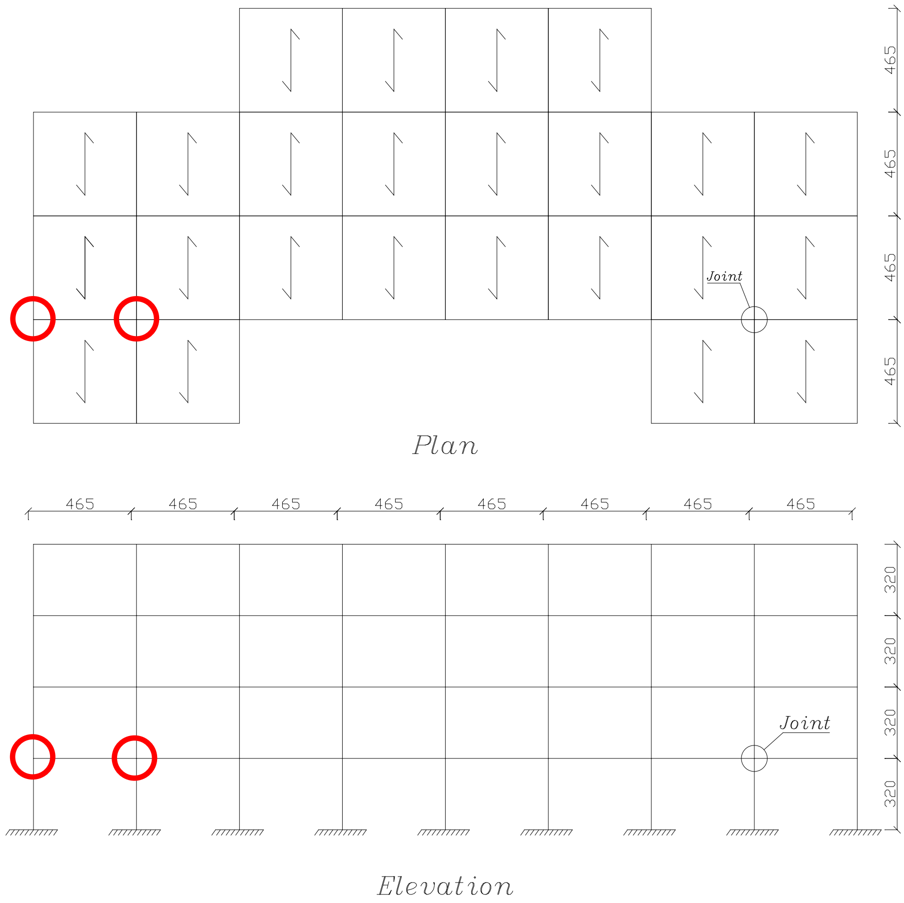

Fig.3 Prototype structure (dimensions are in cm)

The test specimens were representative of interior and exterior beam-column joints of the first storey (marked with circles on **Fig. 3**). The design forces were calculated according to ASCE 7-05 [24], and the building was assumed to have medium ductility and be located on soil type D of IBC-2009 [25]. The design spectral response accelerations at short period and 1-sec period considered to be 1.1g and 0.64g, respectively. **Table 1** outlines the design forces for the interior and exterior connections. P_u, M_u and V_u are ultimate axial load, bending moment and shear force, respectively.

Table 1- Design forces for exterior and interior joints

Specimen	$P_{u\ column}$ kN	$M_{u\ column}$ kN.m	$V_{u\ column}$ kN	$M_{u\ beam}$ kN.m	$V_{u\ beam}$ kN
Interior Joints	682	138	75	121	129
Exterior joints	384	145	79	112	122

Columns and beams in the prototype building were 400×400mm and 400×450mm, respectively, and were designed based on ACI 318-11 [26]. The design concrete compressive strength was f'_c=25 MPa. The yield strength of longitudinal and transverse reinforcement was f_y=400 MPa and 300 MPa, respectively. Eight 20mm diameter bars (Φ20) were used in the columns with reinforcement ratio of ρ=1.6%, and four 18mm diameter bars (Φ18) were used on top and bottom of the beams to provide adequate strength under lateral earthquake loads. The lap splice length for the bottom longitudinal reinforcement was calculated based on ACI 318-11 [26]. Configuration and reinforcement details of the precast and monolithic specimens are illustrated in **Fig. 4**.

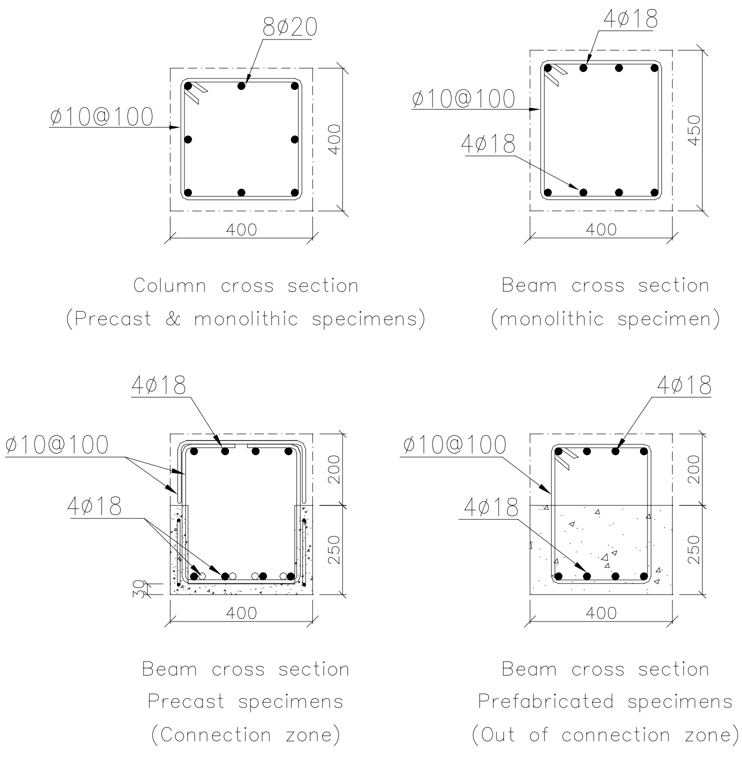

Fig.4 Reinforcement details of precast and monolithic specimens (dimensions are in mm)

3-2 Test Specimens

To investigate the efficiency of the proposed connection system, six full-scale precast and two full-scale monolithic specimens were prepared based on the prototype building. The interior and exterior specimens were designated as BC# and BCT#, respectively, where notation # represents the specimen number. Test variables were the type of stirrups (open and closed) and the stirrup spacing in the connection region. While closed stirrups can provide more confinement in the connection zone, using open stirrups can increase the speed of installation as there is no need to bend the stirrup bars.

- **Specimens BC1 and BCT1:** The monolithic specimens BC1 and BCT1 were used as reference specimens. The beam longitudinal reinforcement continuously passed through the connection region without splicing.
- **Specimens BC2 and BCT2:** Open stirrups with 100 mm spacing were used in the specimens BC2 and BCT2. In these test specimens, top longitudinal reinforcement bars were passed continuously through the connection core, while the bottom reinforcement bars were spliced within the connection zone (see **Fig. 2**).
- **Specimens BC3 and BCT3:** These test specimens are similar to the BC2 and BCT2 specimens, except that closed stirrup bars are used in the connection zone.
- **Specimens BC4 and BCT4:** To increase the confinement at the connection zone, the spacing of the closed stirrups in these specimens was reduced from 100 to 75 mm.

The reinforcement details and material properties of the interior and exterior connection specimens are summarized in **Table 2**.

Table 2- Specifications of the interior and exterior test specimens

Specimen	Type	Compressive strength of concrete f_c', MPa	Compressive strength of grout, MPa	Specification
BC1	Interior connection	30	-	Monolithic
BC2		25	24	Open stirrup-spacing 100 mm
BC3		27	25	Closed stirrup-spacing 100 mm
BC4		22	23	Closed stirrup-spacing 75 mm
BCT1	Exterior connection	30	-	Monolithic
BCT2		25	27	Open stirrup-spacing 100 mm
BCT3		27	25	Closed stirrup-spacing 100 mm
BCT4		22	28	Closed stirrup-spacing 75 mm

3-3 Experimental Test Setup

The interior and exterior test specimens were constructed with column height of 3200 mm and beam length of 2400 mm. For installation of the moment-resisting connections in the lab, the precast beams were fixed on the steel angles on each side of the precast column, and the connection region was grouted after putting the longitudinal reinforcement bars in place (see **Figs. 1 and 2)**. The specimens were then centred between two rigid steel frame columns that were fixed to the strong floor of the structural laboratory. **Fig. 5** shows the schematic of test-setup for the interior connections (BC1 to BC4 specimens). Roller supports were used at the end of the beam and top of the column elements and a hinge support was used at the column base as shown in **Figs. 5 and 6**. Two horizontal and one vertical 500 kN actuators were placed at the top of the precast column to apply lateral displacement and axial loads, respectively (see **Fig. 6-b**). To increase the bearing resistance of the tested elements at the supports, steel plates were placed at the free ends of the beam and column elements.

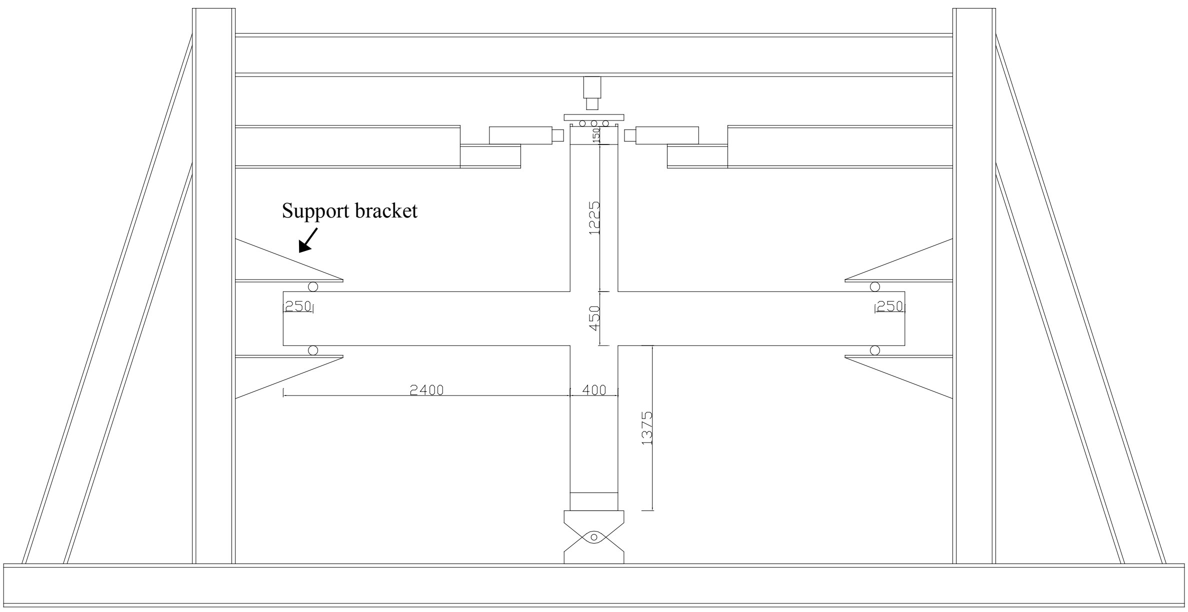

Fig.5 Schematic of test-setup for interior connections (dimensions are in mm)

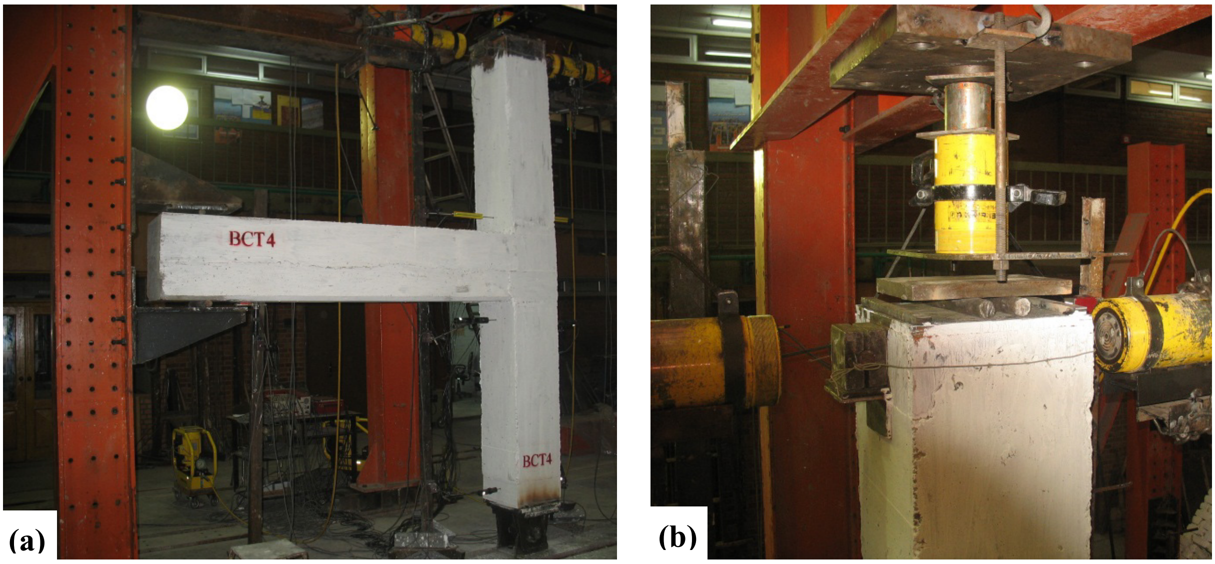

Fig.6 (a)**:** Testing frame and boundary conditions of the BCT4 specimen; (b): Roller support and actuators at the top of the precast column

Three load cells were utilized to monitor the applied lateral and vertical loads during the cyclic loading tests. Ten and nine LVDTs were used in the interior and exterior specimens, respectively, to measure the rotation of the elements and to ensure that the vertical deflections at the supports are nearly zero. **Fig.7** shows the location of the LVDTs in the interior and exterior connections. For practicality reasons, the LVDTs at the supports were installed after the support brackets (see **Fig. 5**). In addition, 32 and 28 strain gauges were installed on the longitudinal reinforcement bars of the pre-cast and monolithic specimens, respectively. The strain gauge results were used to measure uniaxial strains of steel reinforcement, control bond slip behaviour, and to find the yield lateral displacement of the connections. **Fig. 8** illustrates the strain gage arrangement in the precast and monolithic specimens.

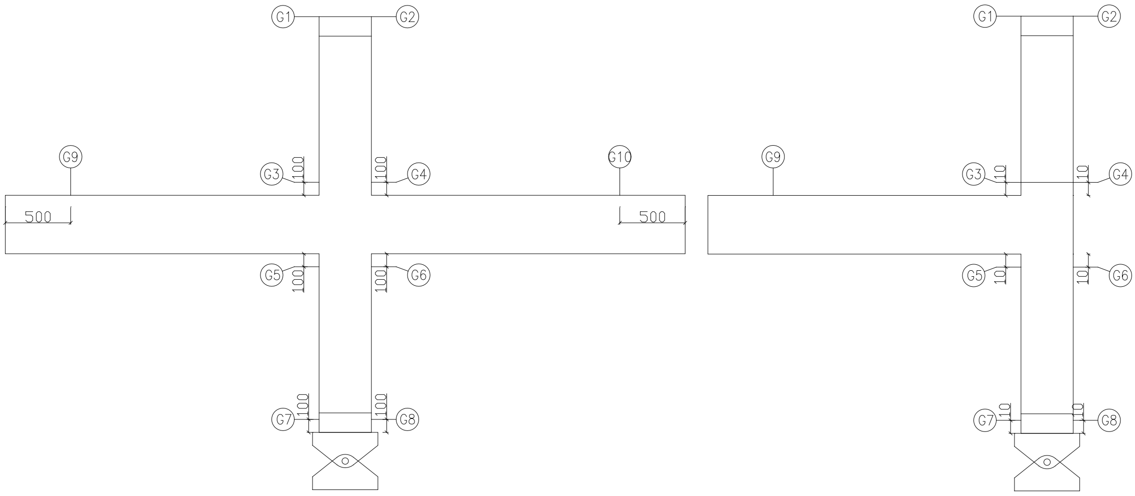

Fig.7 Location of LVDTs (a) exterior and (b) interior connections (dimensions are in mm)

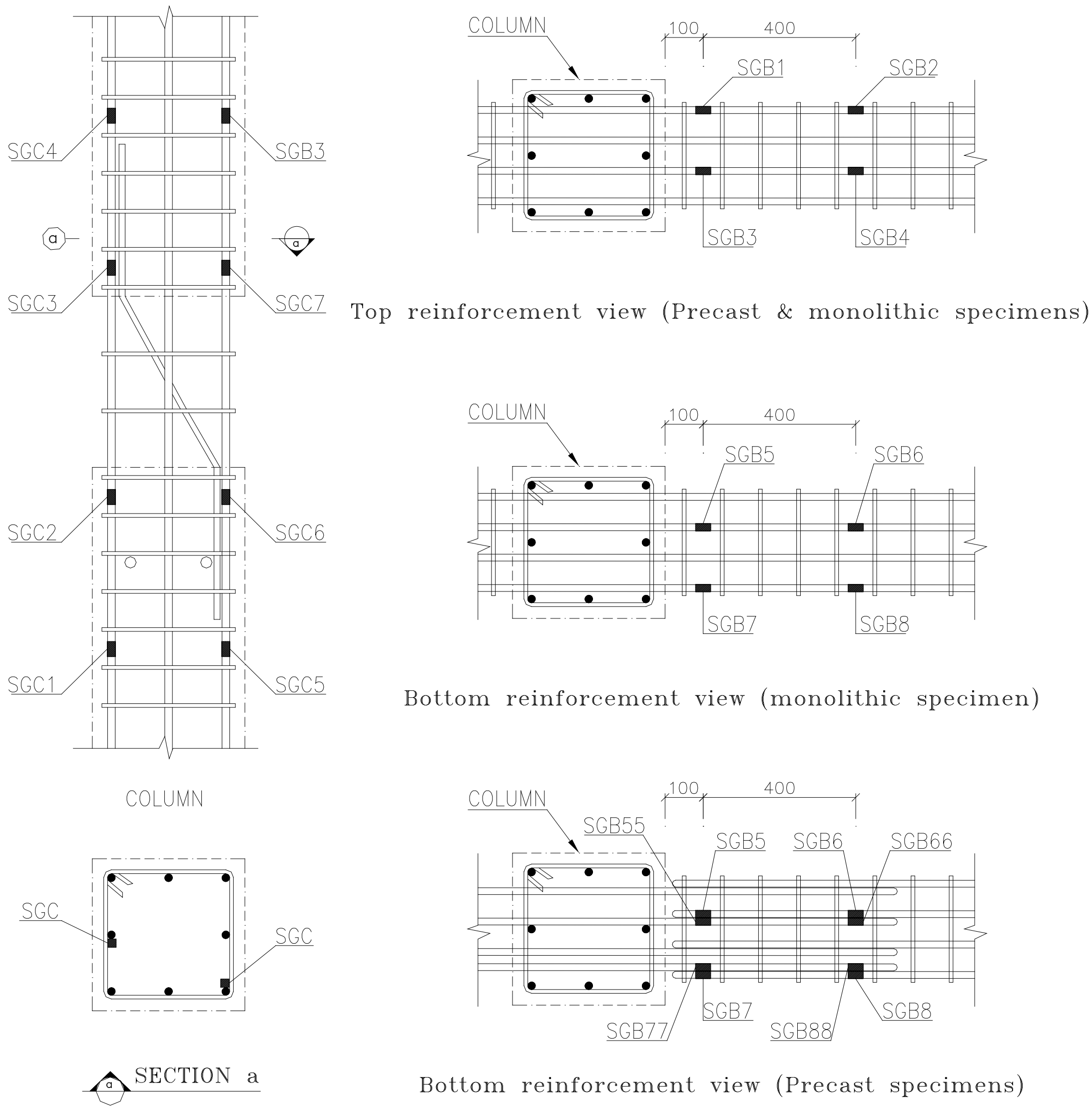

Fig.8 Arrangement of the strain gages on steel reinforcement (dimensions are in mm)

3-4 Testing Procedure

To take into account the dead load transferred from upper floors, an axial load of 400 kN was applied to the precast column at the beginning of each test and maintained throughout the test by using a vertical actuator (see **Fig. 6b**). This axial load is equal to 10% of the ultimate axial capacity of the precast column (0.1fc' Ag) as suggested by Cheok and Lew [27]. Experimental tests were conducted under displacement control using a predetermined cyclic displacement shown in **Fig. 9**. The loading procedure involved applying four levels of displacements before attaining the joint yielding displacement (δ_y) and then applying a set of 3 cycles at each displacement level. The displacement levels were determined by increasing the displacement by a pre-determined increment of δ_y (i.e. δ_y, $2\delta_y$, $3\delta_y$, …). The yield displacement of the connections was calculated based on the measured longitudinal strains in the beam reinforcement bars. The load was paused at the end of each half cycle to mark and measure the cracks and to set the axial load on the column to 400kN. Experimental tests were terminated at lateral displacement around 120mm (4% lateral drift) due to limitations of the test setup and to prevent damage to laboratory equipment. All data (i.e. loads, strains and deflections) were collected by a data acquisition system at a sampling frequency of 1Hz. In this study, the specimens were considered failed when the applied lateral load reduced to less than 80% of the maximum lateral load.

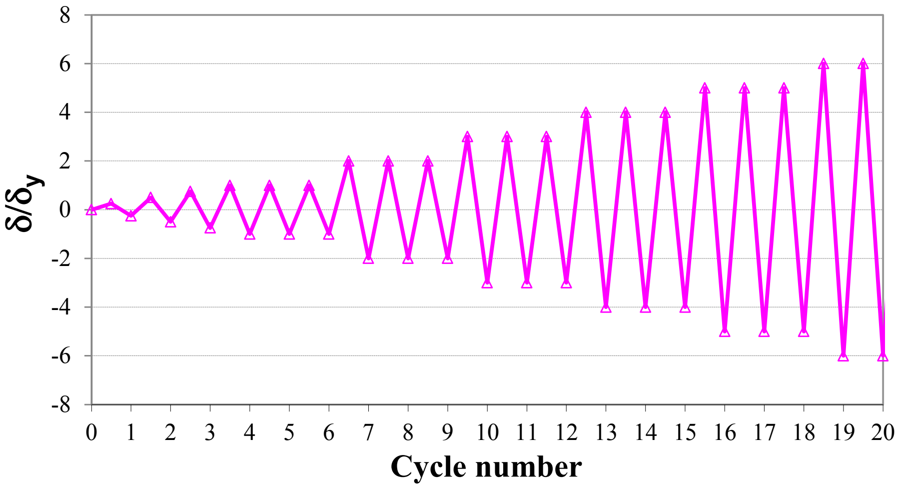

Fig.9 Cyclic loading procedure

4- EXPERIMENTAL RESULTS AND OBSERVATIONS

The test specimens were subjected to the cyclic loading shown in **Fig. 9** up to their failure point. **Figs. 10 and 11** show the relationships between lateral load and lateral displacement at the top of the column for different interior and exterior connections. Experimental data and observations are used to study the failure mode, drift capacity, flexural strength, strength degradation, ductility, and energy dissipation capacity of the monolithic and precast connections.

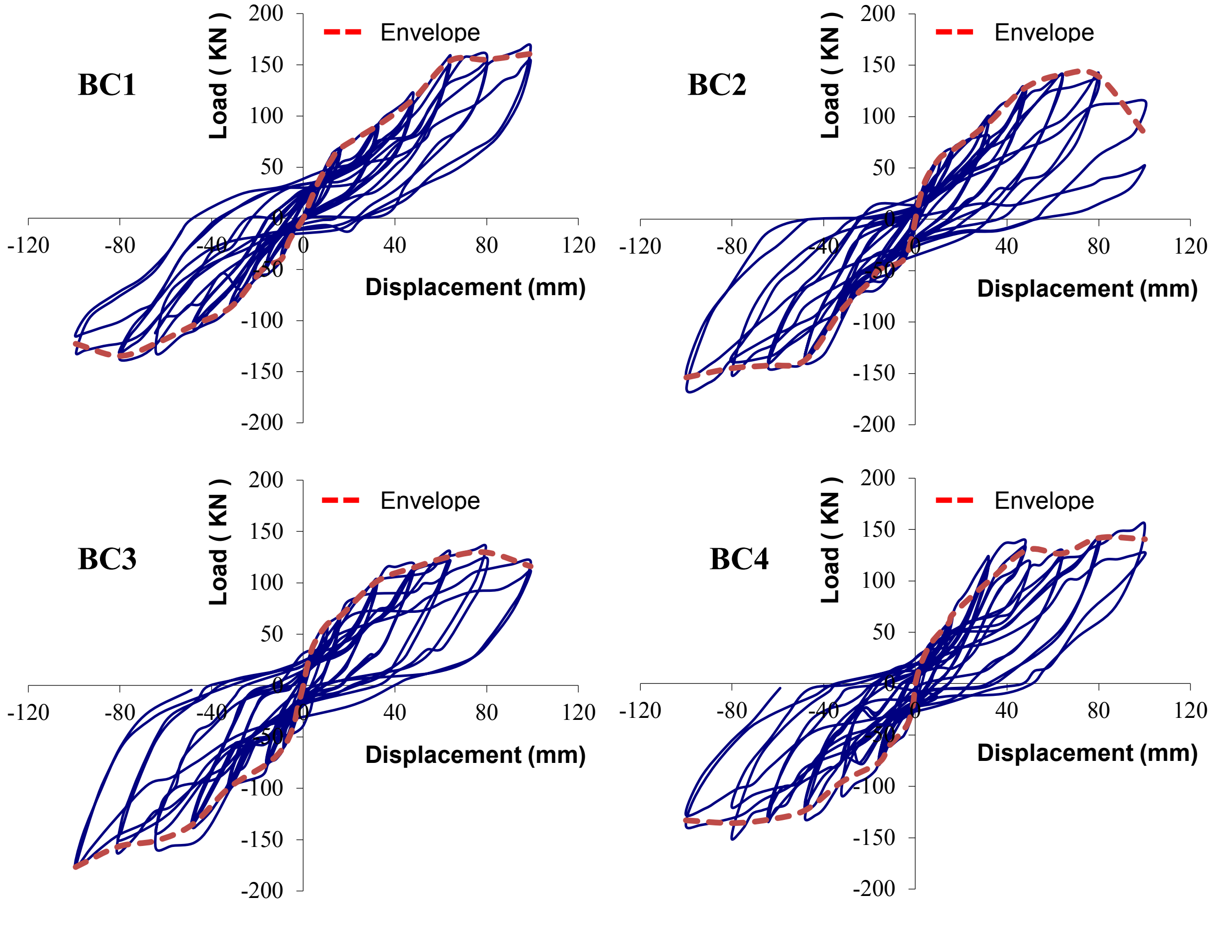

Fig.10 Hysteretic and envelope curves for interior connections BC1, BC2, BC3 and BC4

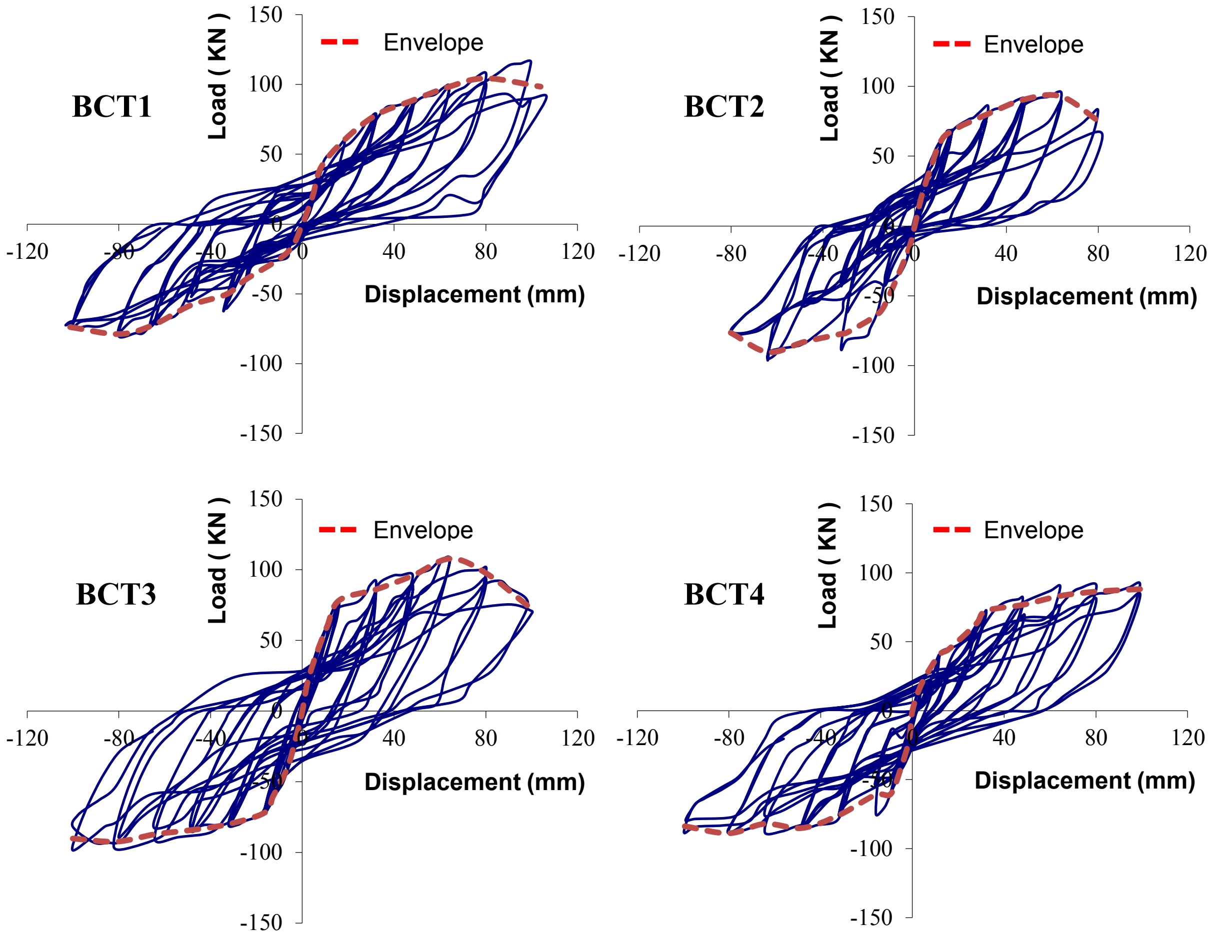

Fig.11 Hysteretic and envelope curves for exterior connections BCT1, BCT2, BCT3 and BCT4

4-1 Failure Modes

Fig. 12 compares the crack propagation pattern and failure modes of the exterior monolithic and precast connections. The flexural crack in these test specimens were initiated at 2nd cycle of the loading (crack width of about 1 mm). While the initial cracks on precast specimens were observed at the beam-column joint interface, the first cracks in the monolithic specimen (BCT1) initiated at a distance of 30 to 50 mm from the column face. This behaviour can be mainly attributed to discontinuity of concrete in beam-column interface in the precast specimens. By increasing the load, the flexural cracks were extended in the beam elements and concrete spalling was more notable. In the precast specimens, the flexural cracks penetrated to the grouted region of the connection, which indicates a good integrity between the beam and column elements in the precast connections (**Fig. 12**).

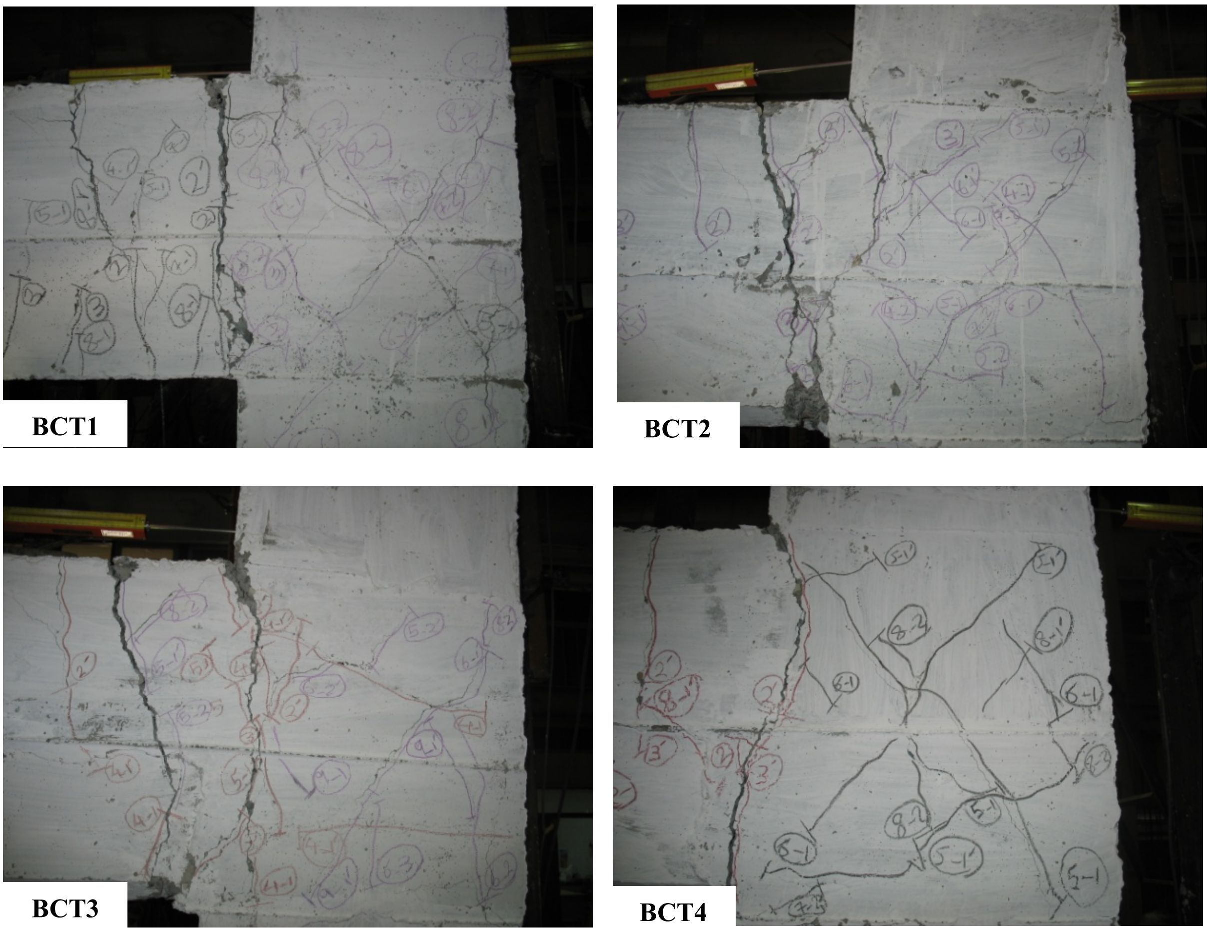

Fig.12 The crack formation and failure mode of exterior monolithic and precast connections

The flexural cracks in the precast specimens were mainly concentrated in the connection zone of the beams, which prevented the development of excessive flexural cracks along the beam length. Diagonal shear cracks at beam-column joint core were initially observed at 2.5% drift in monolithic specimen. The first shear cracks in precast specimens were appeared at 3.0% and 3.5% drift in the precast elements with stirrup-spacing of 100 mm and 75 mm, respectively. This indicates that the diagonal reinforcement bars in the joint core of the precast connections could delay the development of the diagonal cracks. The precast connections were designed to have adequate shear strength to avoid shear failure and yielding of the stirrups in the core area of the joints. This behaviour was confirmed by the experimental results and test observations. It should be mentioned that while the shear strength of the joints was on average around 400 kN, the maximum shear force demand was less than 130 kN.

Fig. 13 shows the typical failure mode of the monolithic and the precast interior connections. It is shown that the shear cracks in the precast connections are less concentrated in the joint core, which can prevent undesirable shear failure modes in the connections. Relatively small shear crack width in the core area of the joints indicate that the shear stirrups did not yield during the tests. The main reason for higher damage at the joint core of the monolithic connections is that no diagonal bracing bar was used in the joint core of these specimens (see **Fig. 1**).

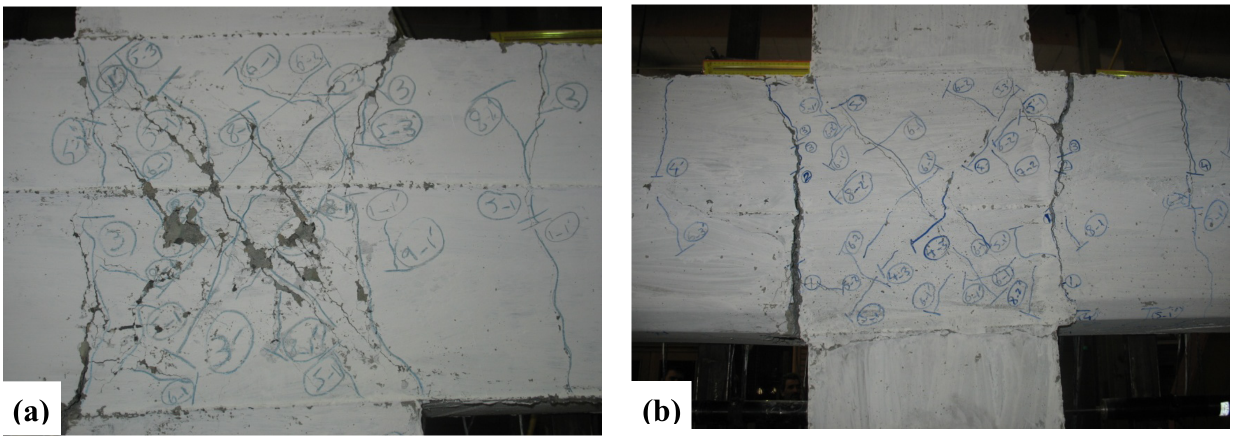

Fig.13 Typical failure mode and flexural cracks of (a) interior monolithic connection BC1, and (b) interior precast connection BC4

At failure point, the maximum flexural crack width in the precast and monolithic specimens was, on average, 12.5 and 8 mm, respectively. Cracking provides a means of energy dissipation at the material level. Therefore, higher crack width in the precast connections results in a higher energy dissipation capacity that will be discussed in more detail in section 4.6. The precast connections, in general, exhibited a strong column–weak beam failure mechanism and their failure was mainly due to yielding of the longitudinal reinforcing bars followed by crushing of concrete in the plastic hinge zone of the beams. Although precast connections exhibited more concentrated cracks compared to monolithic specimens, experimental results indicate that a plastic hinge zone was developed in the precast beams. **Fig. 14** shows the load-strain relationship for the top and the bottom longitudinal bars in the exterior connection BCT3 (see **Fig. 8** for the location of the strain gauges). Strain measurements show that the length of the plastic hinge in the precast beam was around 500 mm, which is in good agreement with the calculated length of 600 mm (see section 2).

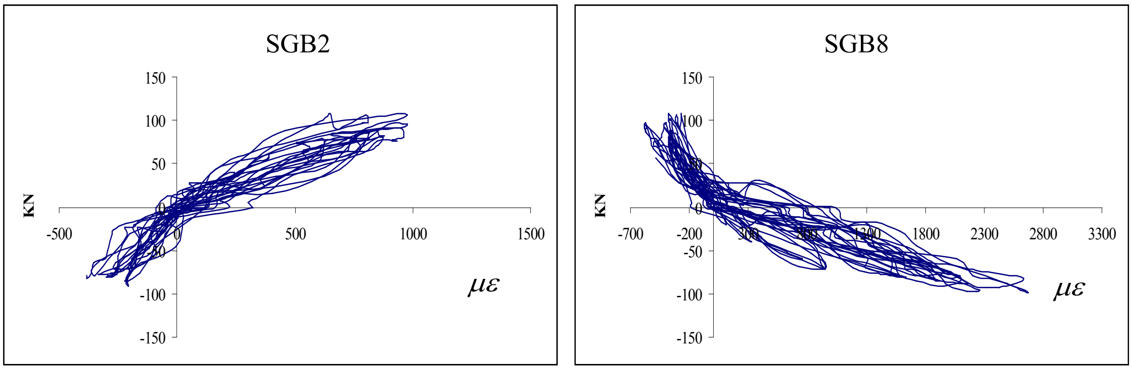

Fig.14 Typical experimental load vs. strain relationships for top and bottom longitudinal bars, BCT3

At the failure point, there were no sliding between grout and concrete in the precast beams, and no slippage was observed between the reinforcement bars and the grout. This indicates a good integrity between the prefabricated beam and the cast-in-place grout in the connection zone. These experimental observations are in agreement with the strain gauge measurements in longitudinal reinforcement bars of the beam elements. For example, **Fig. 15** shows typical load-strain relationships for two adjacent longitudinal bars in the interior precast connection BC4. The measured strains indicate that no significant bond-slip accrued in the reinforcement bars of the precast specimens up to the failure point.

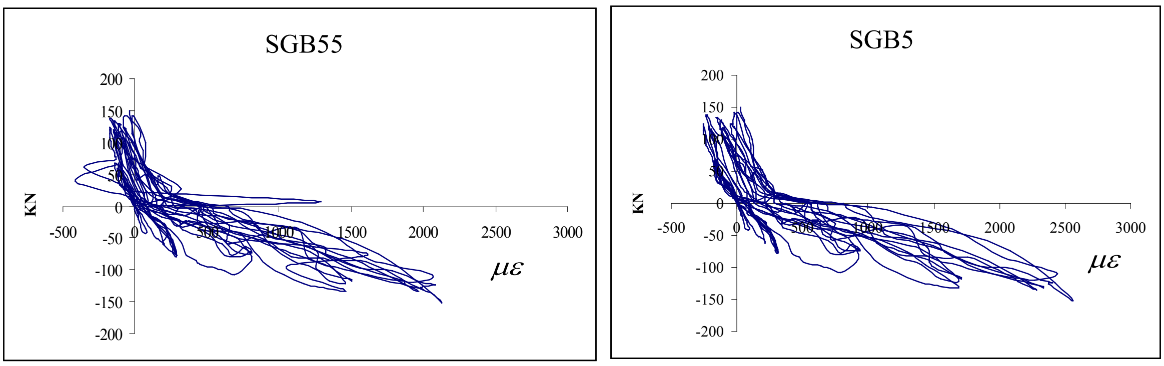

Fig.15 Typical experimental load vs. strain relationships for adjacent longitudinal bars, BC4

Table 3 summarises the drift ratio of the different test specimens at the failure point (i.e. drift capacity). The results are obtained by dividing the failure displacement to the effective column height. It is shown that the precast connections with closed stirrups had slightly higher ultimate drift ratios compared to the similar monolithic specimens. Based on ASCE 41-06 [29], RC concrete frames should be able to resist 2% and 4% inter-storey drift to satisfy Life Safety (LS) and Collapse Prevention (CP) performance levels, respectively. It is shown in Table 3 that the drift capacity of the precast specimens with open stirrups (i.e. BC2 and BCT2) is sufficient up to LS performance level. However, by using closed stirrups, the proposed precast connections could satisfy the CP performance level criteria, and therefore, can be used in high-seismic regions.

Table 3- Maximum bending moment, yield displacement, ultimate ductility, drift capacity and ASCE performance level

Specimen	Maximum bending moment (kN.m)	Yield displacement δ_y (mm)	Ultimate ductility μ_u	Story drift at failure (%)	Performance level
BC1	241	23	4.1	3.9	CP
BC2	240	21	4.7	3.5	LS
BC3	249	18	5.5	4.0	CP
BC4	237	17	6.0	4.0	CP
BCT1	138	22	4.5	3.9	CP
BCT2	139	18	4.9	3.0	LS
BCT3	147	17	5.7	3.9	CP
BCT4	140	16	6.2	4.0	CP

4-2 Flexural Strength

Based on the experimental setup shown in **Fig. 5**, bending moments at the connections can be easily calculated as the applied force times the lever arm. The maximum measured bending moment in the test specimens are compared in **Table 3**. The results in this table are the average of the maximum bending moments in positive and negative directions. The results indicate that all of the precast concrete connections reached their designed ultimate moment strength capacity. Although the compressive strength of concrete was higher (on average 20%) in monolithic specimens (**Table 2**), the precast connections exhibited similar or even slightly higher flexural strength. It should be noticed that the reinforcement volume ratio in the precast elements is duplicated in the connection zone due to the lap splices of the bottom longitudinal bars (see **Fig. 2**). This additional reinforcement is the main reason for higher flexural strength in the precast connections compared to the monolithic specimens.

4-3 Ductility

Ductility is defined as the ability of the structure to undergo plastic deformations without significant loss of strength. The concept of ductility is a key element in earthquake resistant design of structures. The ductility of the connections, μ, is defined as the ratio of the maximum displacement at any cycle to the yield displacement of the connection. The yield displacement was determined based on the ASCE/SEI Standard 41-06 recommendations [29]. In this method, the hysteresis envelope curve is represented by a bilinear curve with a post-yield slope (αK_e) as shown in **Fig. 16**. The yield displacement (δ_y) is determined on the condition that the secant slope intersects the actual envelope curve at 60% of the nominal yield force (F_y), while the area enclosed by the bilinear curve is equal to that enclosed by the original curve bounded by the peak displacement (δ_{peak}).

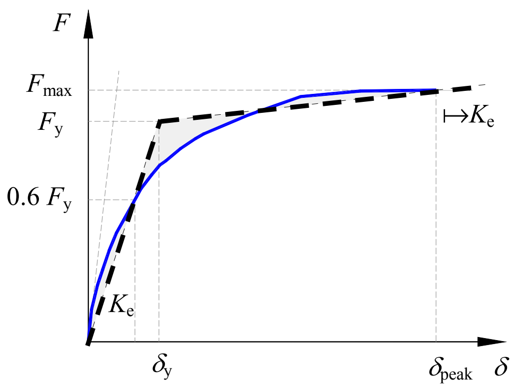

Fig.16 Definition of the yield point [29]

Table 3 shows the ductility of the different precast and monolithic connections determined at the failure point (i.e. ultimate ductility). The results are the average of the ductility ratios in positive and negative directions. It is shown that precast specimens exhibited considerably higher ductility (up to 46%) compared to monolithic connections. This implies that the proposed details for precast beam-column connections (**Figs.1 and 2**) could significantly enhance the ductility behaviour of the precast moment-resisting connections. This is especially important for high seismic regions, where structures are expected to undergo large nonlinear deformations under strong earthquakes.

Based on the results in **Table 3,** using closed stirrups in BC3 and BCT3 precast specimens could increase their ductility, on average, 17% compared to the similar specimens with open stirrups (BC2 and BCT2). This can be due to higher confinement level in the connection zone of the specimens with closed stirrups. Similarly, reducing the spacing of the closed stirrups from 100 mm in BC3 and BCT3 specimens to 70 mm in BC4 and BCT4 specimens resulted in around 10% increase in the ductility of the connections. This can also be attributed to increasing the confinement in the connection zone.

4-4 Strength Ratio

Strength ratio (or strength degradation) is an important parameter to evaluate the performance of connections under dynamic/cyclic loadings such as earthquake ground motions. In this study, strength deterioration is evaluated using the ratio of the moment at peak rotation to the initial yield moment calculated from hysteresis envelope curves. **Fig. 17** shows the variation of strength ratio in the interior and exterior connections at different drift levels. The results shown in this figure are calculated based on the average of the three positive excursions at each drift level.

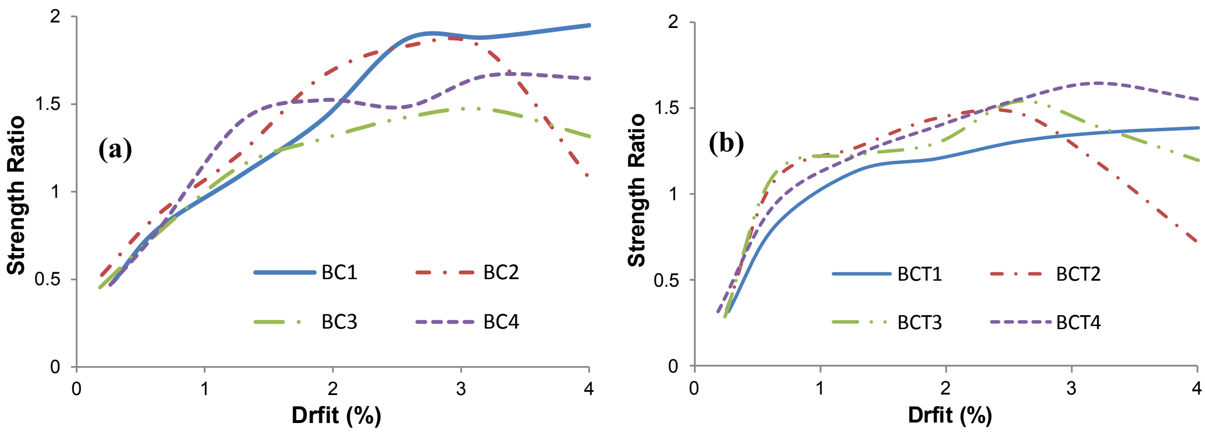

Fig.17 Strength ratio of (a) interior and (b) exterior connections for positive excursions

Based on the results shown in **Fig. 17**, there is no deterioration in the strength of the interior and exterior precast elements up to 3% drift (LS performance level). Test specimens BC2 and BCT2 (with open stirrups, spacing 100 mm) exhibited significant strength deterioration at higher drift levels. This is in agreement with the previous results that showed this type of detailing is not appropriate for CP performance level with target drift ratio of 4%.

Although there is a small deterioration in the strength of the precast connections with close stirrups (BC3, BCT3, BC4 and BCT4) beyond 3% drift, it is not considered to be significant up to 4% drift. In general, **Fig. 17** shows that the strength degradation of precast connections at higher drift ratios can be controlled by using low spacing closed stirrups. This indicates that the proposed precast moment-resisting connection can be designed efficiently for high seismic regions.

4-5 Moment-Rotation Relationship

Moment-rotation relationships are widely used for modelling and evaluating the behaviour of beam-column connections. In this study, the measured uniaxial strains in the top and the bottom longitudinal bars are used to determine the rotation of the beam and the column elements at different load levels. Subsequently, the joint rotations are calculated based on the difference between the beam and column rotations. **Fig. 8** shows the location of the strain gauges in the precast and monolithic specimens. Detailed calculations of moment-rotation relationships can be found in ref [28].

The moment-rotation hysteresis envelope curves of the interior and exterior joints are compared in **Fig. 18**. The initial rotational stiffness of the connections can be determined from the slope of a tangent to the moment-rotation curves. It is shown that the initial rotational stiffness of the interior and exterior precast specimens was slightly higher than those of the monolithic connections. This can be the result of an increase in the moment of inertia of the beams in precast specimens due to lap-splicing of longitudinal reinforcement in the connection zone (see **Fig. 2**). **Fig. 1**8 shows that the moment-rotation behaviour of the precast connections, in general, is very close to the monolithic connections. This implies that the proposed precast connection can be designed to be as strong as a monolithic connection with the same beam size.

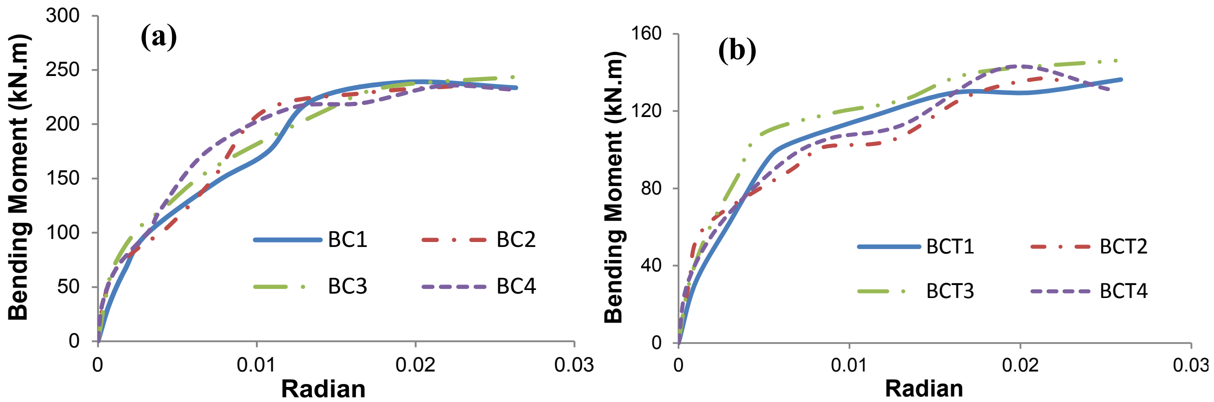

Fig.18 Moment-rotation envelope curves for positive excursions, (a) interior joints and (b) exterior joints

4-6 Energy Dissipation

The inelastic deformation of connections helps to dissipate some energy through hysteretic behaviour and thereby reduce the transmitted energy to other structural elements. This behaviour can improve the seismic performance of the whole structural system under strong earthquakes. The energy dissipation capacity of the connections can be identified as the summation of the areas enclosed by the hysteresis load-displacement loops. As mentioned before, compared to the monolithic specimens, the precast connections exhibited wider crack width, which is expected to help them dissipate more hysteretic energy at large displacements.

The hysteretic energy dissipation capacity of the test specimens was calculated for each load cycle based on the load-displacement curves presented before. To eliminate the effects of concrete and grout strength variation in different test specimens (see Table 2), calculated hysteretic energy dissipations were normalized to the area of elastic-perfectly plastic rectangular block at each load cycle by using the following equation:

$$\text{Normalized Energy Dissipation (NED)} = \frac{A}{4V_{\max}\delta_{\max}} \tag{1}$$

where V_{max} and $\delta_{\max}$ are the average of the maximum load and displacement for positive and negative excursions, respectively, at each load cycle; and A is the area enclosed by the hysteresis loops as shown in **Fig. 19**.

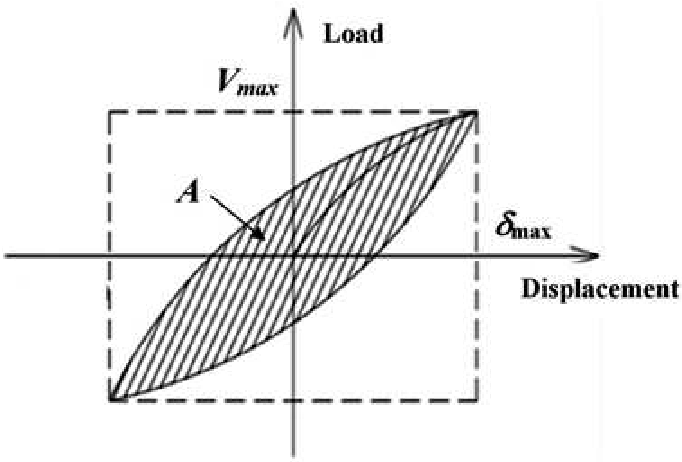

Fig.19 Normalizing hysteretic energy dissipation at each load cycle

Fig. 20 compares the normalized hysteretic energy capacity of the precast and monolithic interior and exterior connections at different storey drift ratios. It is shown that overall the precast specimens exhibited considerably more energy dissipation capacity compared to monolithic specimens, especially at higher drift ratios. For drift ratios between 1% and 4%, the hysteretic energy dissipated by the interior and exterior precast specimens was on average 26% higher than the energy dissipated by monolithic specimens. This can be mainly attributed to the wider crack widths at beam-column joints in the precast specimens, which leads to higher energy dissipation at the material level as discussed before. It is shown in **Fig. 20** that the energy dissipation capacity of the precast connections increased faster than monolithic specimens (higher initial slope). This behaviour is in agreement with the experimental observations, as initial cracks at beam-column joint interface developed earlier in precast connections.

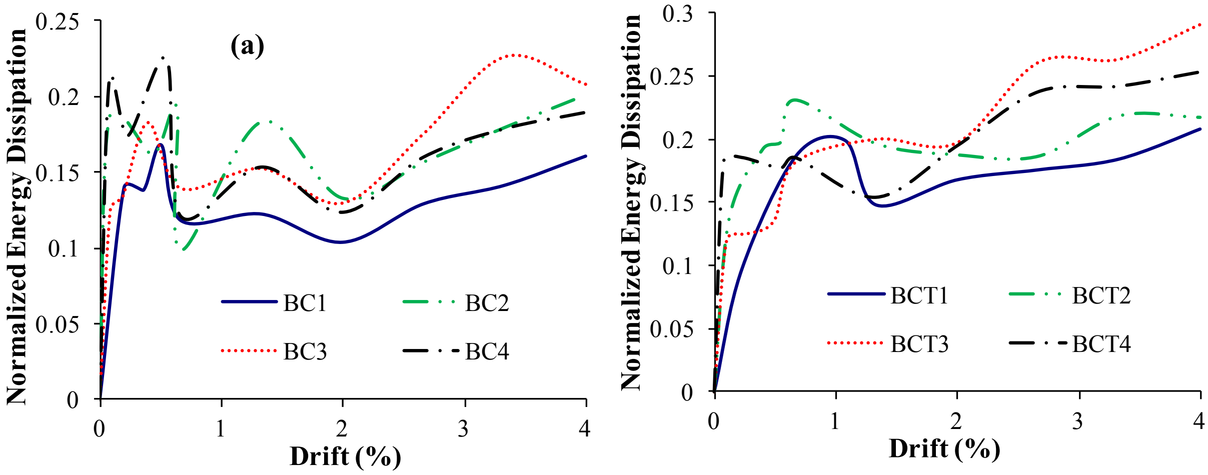

Fig.20 Normalized energy decapitation capacity (a) interior and (b) exterior connections

The results discussed in this paper, in general, demonstrate the efficiency of the proposed moment-resisting precast connections at enhancing the flexural strength, ductility and energy dissipation capacity, which are important parameters in seismic resistant design of structures. The new connection is currently being used in the construction of 700 residential apartment blocks, as a part of the Mehr project in Pardis, Iran. To provide design recommendations, detailed analytical studies have been conducted based on the results of this study, which will be presented in future publications.

5- SUMMARY AND CONCLUSIONS

This research study aimed at developing a new ductile moment-resisting precast connection suitable for RC frames located in high seismic zones. The proposed system enables easy construction work by minimizing cast-in-place concrete volume and eliminating the need for formworks, welding, bolting and prestressing. Based on the results of cyclic loading tests on six precast and two monolithic full-scale specimens, the following conclusions can be drawn:

1. The proposed precast connections exhibited higher flexural strength and initial stiffness compared to similar monolithic specimens, due to lap-splicing of the longitudinal reinforcement in the connection zone.
2. The strength degradation of the precast connections with closed stirrups was acceptable up to 4% drift. However, precast connections with open stirrups exhibited considerable strength degradation at drift ratios more than 3%.
3. Flexural cracks in the proposed precast beam-column connection were mainly concentrated in the plastic hinge zone of the beams, which is in line with the strong-column/weak-beam concept in seismic resistant design.
4. Using diagonal reinforcement bars in the joint core of the precast moment-resisting connections could delay the development of diagonal cracks in the precast connections compared to the monolithic specimens. The shear cracks in the precast connections were less concentrated in the beam-column joint core, which can help to avoid undesirable failure modes in the connections under strong earthquakes.
5. Both interior and exterior precast connections exhibited considerably higher ductility (up to 46%) compared to monolithic specimens. It was shown that the ductility of the precast connections can be further improved by using closed stirrups and smaller stirrup spacing.

6. For similar drift ratio, the hysteretic energy dissipated by the precast moment-resisting connections was up to 30% higher than that of monolithic specimens. This can be mainly attributed to wider crack widths at beam-column joints in the precast connections.

In view of these observations, the proposed moment-resisting precast connection can provide adequate strength, ductility and energy dissipation capacity with respect to monolithic connections, and therefore, can be efficiently used in precast concrete frames in seismic regions.

ACKNOWLEDGMENT

The authors gratefully acknowledge the support of Kesting Prefabricated factory and Building & Housing Research Center (BHRC) laboratory, particularly Mr. Alizadeh and Mr. Aghahosseini.

REFERENCES

1. Elliott K. Precast Concrete Structures, Butterworth-Heinemann; 1st edition, 2002.
2. Mitchell D, DeVall RH, Saatcioglu M, Simpson R, Tinawi R, and Tremblay R. Damage to concrete structures due to the 1994 Northridge earthquake. Canadian Journal of Civil Engineering 1995; 22:361-77.
3. METU, The Ceyhan–Misis Earthquake of 27 June 1998: A Preliminary Engineering Reconnaissance Report. Technical Report, Middle East Technical University Disaster Management, Implementation and Research Center, Ankara, Turkey, 1998.
4. Park R. Seismic Design and Construction of Precast Concrete Buildings in New Zealand. PCI Journal 2002; 47(5):60–75.
5. Korkmaz H, Tankut T. Performance of a Precast Concrete Beam-to-Beam Connection Subject to Reversed Cyclic Loading. Engineering Structures 2005; 27(9): 1392–1407.

6. Park R, Bull DK. Seismic Resistance of Frames Incorporating Precast Prestressed Concrete Beam Shells. PCI Journal 1986; 31(4): 54–93.
7. French CW, Amu O, Tarzikhan C. Connections between Precast Elements-Failure outside Connection Region. Journal of Structural Engineering 1989; 115(2):316–40.
8. French CW, Hafner M, Jayashankar V. Connections between Precast Elements-Failure within Connection Region. Journal of Structural Engineering 1989; 115(12):3171–92.
9. Cheok G, Lew HS. Performance of Pre-cast Concrete Beam to Column Connections Subject to Cyclic Loading. PCI Journal 1991; 36:56-67.
10. Castro JJ, Yamaguchi T, Imai H. Seismic performance of Precast Concrete Beam- Column Joints. Journal of Structural Construction Engineering 1994; 455:113-26.
11. Stone WC, Cheok GS, Stanton JF. Performance of hybrid moment-resisting precast beam-column concrete connections subjected to cyclic loading. ACI Structural Journal 1995; 91(2):229-49.
12. Priestley MJN, MacRae GA. 1996. Seismic Tests of Precast Beam-to-Column Joint Sub-assemblages with Unbonded Tendons. PCI Journal 1996; 41(1):64–80.
13. Cai JG, Zhu HJ, Feng J, Liu YF, Huang LF. Experimental study on seismic behavior of middle joints of SCOPE system, Journal of Central South University (Science and Technology) 2012, 43 (5):1894-1901.
14. Khaloo AR, Parastesh H. Cyclic Loading of Ductile Precast Concrete Beam-Column Connection. ACI Structural Journal 2003; 100(4):291-6.
15. Khaloo AR, Parastesh H. Cyclic Loading Response of simple Moment-Resisting Precast Concrete Beam-Column Connection. ACI Structural Journal 2003; 100(4):440-5.
16. Ertas O, Ozden S, Ozturan T. Ductile Connections in Precast Concrete Moment Resisting Frames. PCI Journal 2006; 51(2):2–12.

17. Kulkarni SA, Li B, Yip WK. Finite element analysis of precast hybrid-steel concrete connections under cyclic loading. Journal of Constructional Steel Research 2008; 64(2):190-201.
18. Xue W, Yang X. Seismic tests of precast concrete moment-resisting framers and connections. PCI Journal 2010; 55:102-21.
19. Vidjeapriya R, Jaya KP. Behaviour of precast beam-column mechanical connections under cyclic loading. Asian Journal of Civil Engineering 2012; 13(2):233-45.
20. Hassoun MN. Structural Concrete Theory and Design.3rd ed. New Jersey: John Wiley & Sons Inc, 2005
21. Varney JC, Brown MD, Bayrak O. Effect of Stirrup Anchorage on Shear Strength of Reinforced Concrete Beams. ACI Structural Journal 2011; 108 (4): 469-478
22. Zia P, Hsu T. Design for Torsion and Shear in Prestressed Concrete Flexural Members. PCI Journal 2004; 49(3): 34-42.
23. Murty DSR, Papa Rao G. Influence of Stirrup Spacing on Shear Resistance of Reinforced Concrete Beams, Magazine of Concrete Research 2013, 65(14): 829-836.
24. ASCE 7-05. Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers (ASCE), Reston, Virginia; 2006.
25. IBC-2009. International Building Code. International Code Council, Country Club Hills, USA; 2009.
26. American Concrete Institute. Building Code Requirements for Reinforced Concrete, ACI-318-11; 2011.
27. Cheok GS, Lew HS. Model precast concrete beam-to-column connections subjected to cyclic loading. PCI Journal 1994; 38(4):80-92.

28. Ramezani R. Experimental Study on Cyclic Behaviour of Precast Concrete Beam-Column Connections. MSc. Thesis, K.N.T. University of Technology, Faculty of Civil Engineering, Tehran; 2010.
29. ASCE 41-06. Seismic Rehabilitation of Existing Buildings. Edition: 1st. American Society of Civil Engineers; 2007.