Cyclic behavior of precast concrete beam-column connection using steel fiber reinforced cast-in-place concrete

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Three equivalent exterior precast concrete beam-column (PCBC) connections have been investigated in this study in order to analyze the effect of steel fiber reinforced concrete (SFRC) as cast-in-place (CIP) on the seismic performance of the PCBC connection. The connection was designed as a ductile connection for a moment-resisting frame and consists of a precast U-beam, precast column with corbel, interlocking bars, and CIP-concrete to connect the precast beam to precast column. The volume fractions of steel fiber incorporated within the CIP-concrete were 0%, 0.5% and 1%. A quasi-static load was applied vertically to the beam tip of the PCBC specimen. The results showed that the steel fibers contained within the CIP-concrete provided 2% increase of the maximum load, 17.7% increase of the energy dissipation, and increase in the joint stiffness of the PCBC connection. The steel fibers delayed the onset of cracking and slowed down the crack propagation, resulting in shorter cracks in the joint core of PCBC specimen, which correlates well with the deflection-hardening characteristic found from the modulus of rupture test.

Keywords: *precast concrete, beam-column, connection, steel fiber, cast-in-place, energy dissipation*

1. Introduction

Fiber reinforced concrete (FRC) is a cement based material reinforced with short randomly distributed fibers. FRC is an alternative to improve the structural performance under seismic loading. Currently, fibers are produced from several materials such as steel, glass, carbon, and steel combined with synthetic fibers (nylon, acrylic, polyester, etc.). Research has shown that FRC has better mechanical properties when compared to plain concrete [1, 2]. The fibers, which are dispersed homogeneously in the concrete, can bridge cracks and distribute tensile stresses so that crack sizes become smaller and are spread evenly [3]. The fibers can also control crack formation, delay crack propagation, and improve ductility of the concrete. In addition, the presence of fibers in concrete can enhance the bond between concrete and deformed steel bars [4, 5].

The ability of fibers to enhance the mechanical properties depends on several factors, such

as fiber type, fiber modulus, fiber aspect ratio, fiber strength, surface bonding characteristics, fiber content, fiber orientation, and aggregate size and strength of the concrete matrix itself. Steel fibers are more widely used because they have a relatively high strength and modulus of elasticity, they are protected from corrosion by the alkaline environment of the cementitious matrix, and the bond strength between concrete and fiber can be enhanced by mechanical anchorage or surface roughness [6]. Hooked-end fibers enhance end anchorages significantly when compared to straight fibers; they increase the energy absorption capacity and concrete toughness [7].

The addition of fiber has been effective in improving the performance of column-beam joints due to gravity and cyclic loads, both in monolithic and precast joints [8–10]. Steel fiber reinforcement in concrete can also reduce the density of the transverse reinforcement in the beam-column joint.

Precast concrete systems continue to develop and have become more popular in the last few decades. These systems are believed to be an alternative solution to many of the problems in the con-

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struction industry, because they offer advantages in terms of quality, time, and cost, in comparison with cast-in-situ construction. Furthermore, the manufacture of precast units in a controlled factory environment improves the quality of the material selection, mixing, molding, and the curing process, such that the quality and strength can be better guaranteed. These advantages make precast systems potentially more appealing to the construction industry.

Many studies have been carried out to improve our understanding of the joint behavior between precast concrete elements. Several studies on precast concrete beam-column (PCBC) joints, with different designs and detailing, which were subjected to cyclic loading, have shown that the precast connections provide sufficient strength and energy dissipation compared to monolithic specimens [10–12].

In general, the types of beam-column connections that have been developed in terms of precast concrete moment-resisting frames are bolted, welded, prestressed, cast-in-place (CIP), or a combination consisting of any two or more of these. Welded connections could satisfy strength and stiffness requirements; however, they can introduce excessive heat, which can damage or cause cracks in the adjacent precast concrete [13]. In addition, the implementation of this type of connection needs skilled site staff who can guarantee the quality of the welding in the connection [8, 14].

The easiest method to connect precast elements on site is using bolted connections. Unfortunately, this method requires a high degree of precision in placing the channels or steel plates before casting the precast elements. Problems also exist due to the sliding risk [14]. Another study negated the sliding risk by providing additional tolerances in the bolt holes. However, this caused an initial loss of stiffness in the connection [15].

The CIP-concrete method benefits from the monolithic advantages of reinforced concrete systems as it will help to achieve a more continuous/composite section. This type of hybrid system is recommended for seismic designed buildings [11]. It also allows more tolerance in the precast connections. Unfortunately, it takes longer to

install as the concrete has to gain strength and it needs additional formwork and scaffolding on site. All of these factors will lead to an increase in cost and construction time, although this can be off-set to some degree by the improved monolithic structural system.

The main objective of this research is to study the effect of steel fiber reinforced concrete (SFRC) as a material in CIP-connections to the seismic performance of PCBC joints, in terms of the maximum load, the energy dissipation, and the crack pattern. The type of precast beam-column connection discussed in this study has been developed in a previous study. The connection offers several advantages; this connection will negate the need for high precision engineering, hence increasing practicality; reduce the use of formwork due to the use of a precast partial U-beam (acting as permanent formwork); and lower the volume of CIP-concrete. The use of corbels, which support the precast beam, will minimize the need for scaffolding. This new connection also avoids the use of welding, and (potentially) bolts and pre-stressing, which therefore leads to a reduced need for skilled labor and a reduction in the construction time. It may be noted that bolts for any additional corbels required in this connection design are installed on the site before the installation of the precast beam element and are easy to install precisely. So, there should not be a problem regarding precision, sliding risk, and initial loss of stiffness. Overall, this new connection can be expected to offer a more economical and practical solution [16].

2. Research significance

This study illustrates that the steel SFRC used as CIP has the potential to increase the seismic performance of a PCBC connection. Steel fibers can slow down the spread of cracks and increase the energy dissipation of the joint.

3. Experimental program

A beam-column specimen used in this investigation consisted of one precast concrete beam and one precast concrete column joined using CIP-

concrete, and represented a half-scale of an exterior beam-column joint of a moment-resisting frame. The length of the column and the beam were determined based on the location of the contra-flexure points determined via a software analysis of a representative planar frame. The prototype building was rectangular (three bays – three stories high).

Figure 1 shows the reinforcement details of the PCBC specimens, whereas Figure 1B presents the PCBC specimen as an isometric. The construction process of the joint in the test rig is as follows: (1) Precast columns were vertically set up in the test rig. Column ends were restrained by steel plates, which were bolted to the test rig; (2) One end of the precast U-beam was horizontally placed on the corbel, while the free end was supported by scaffolding (temporary supports, which on site would not be required); (3) All reinforcements (interlocking bars and stirrups) in the joint core and the beam core were installed; (4) The joint core was covered by the form and sealed to avoid the concrete mix leaking from the form; (5) The connection region was cast using CIP-concrete; (6) One day after casting, the joint core was grouted using a non-shrinking grout material to fill any gap between the hardened joint core concrete and the upper part of the precast column; (7) The scaffolding was removed after 2 weeks and the PCBC specimens were ready to test after the CIP-connection had aged a minimum of 28 days. It is noteworthy to mention that in a real application, the precast U-beam can be placed directly on the corbels of the precast columns at both sides (left and right precast beam), so that the scaffolding could be removed after 7 days.

The joint incorporated two concrete types. One is a plain concrete with a design compressive strength of 30 MPa; this was used for the precast concrete beam and column. The other was a SFRC with a design compressive strength of 45 MPa, which was used for the CIP-connection. The CIP-concrete has a higher compressive strength than the precast elements because the connection region was expected to be the more vulnerable, and hence experiences greater stresses from the applied loads. Three PCBC specimens were fabricated and all had the same dimension and reinforcement detail. The

fiber content in the CIP-concrete was varied, i.e., 0%, 0.5%, and 1% for specimens P2, P3, and P4, respectively. A 1% fiber content was the maximum fiber content considered; greater volumes caused practical problems with the placing of the CIP-concrete. The hooked-end steel fiber is used in this study and its properties are presented in Table 1. The mix proportion of CIP-concrete for specimens P2, P3, and P4 is given in Table 2. Four different diameters of steel bar were used for the PCBC specimens, i.e. 8 mm (for the beam stirrups), 10 mm (for the longitudinal bars of the U-beam), and 12 mm and 16 mm (for the flexural bars of the beam core and the column). The yield strength of the steel reinforcing bars was 500 MPa and the yield strain was 0.0030.

Table 1. Properties of the steel fibers

Brand	Dramix RC 65 35 BN
Length (l_f)	35 mm
Diameter (d_f)	0.55 mm
Aspect ratio (l_f/d_f)	65
Tensile strength	1345 N/mm ²
Young's Modulus (E_{mod})	210000 N/mm ²

For these tests, both the column ends were restrained by steel plates which were bolted to the test rig, while the beam end was free. The load was applied vertically to the tip of the beam. No vertical axial load was applied to the top of the column since this tends to enhance the joint shear strength; hence, this load setup represented a worst-case loading scenario [17]. Figure 2A shows the setup of the test.

A quasi-static load was applied to the specimen using displacement control; displacements of 3 mm, 8 mm, 12 mm, 18 mm, 24 mm, 36 mm, 48 mm, and 60 mm were used, as shown in Figure 2B. For each displacement two cycles of quasi-static loading were applied. After the second cycle at 60 mm displacement, the test specimen still appeared to be in good condition. At this stage, there was no significant peak load degradation; all peak loads at all cycles after displacement level of 12 mm were higher than 75% of the maximum load in both loading directions. To finish the test, the

Table 2. Material composition of precast elements and CIP-connection of specimens P2, P3, and P4

Materials	Precast elements	CIP-connection of P2	CIP-connection of P3	CIP-connection of P4
Coarse aggregate (kg/m ³)	1000.55	1028.4	1028.4	1028.4
Fine aggregate (kg/m ³)	818.63	685.6	685.6	685.6
Cement (kg/m ³)	335.82	441	441	441
Water (kg/m ³)	208	210	213	213
Steel fiber (kg/m ³)	—	—	39.25	78.5
w/c	0.62	0.47	0.52	0.52
Slump (mm)	150	140	125	85

CIP, cast-in-place.

Fig. 2. (A) Test setup of PCBC specimens; (B) Loading history for reversed cyclic load test. PCBC, precast concrete beam-column.

Table 3. Average of compressive strength, modulus of elasticity, and modulus of rupture

Specimen	Average compressive strength (MPa)	Average modulus of elasticity (MOE) (GPa)	Average modulus of Rupture (MOE) (MPa)
P2 Precast beam	40.95	30.697	4.03
Precast column	55.87	28.940	5.42
CIP-concrete	50.86	31.518	5.99
P3 Precast beam	36.82	30.461	5.36
Precast column	42.83	28.009	5.21
CIP-concrete	47.36	33.782	6.06
P4 Precast beam	50.58	31.304	5.88
Precast column	51.35	29.841	5.65
CIP-concrete	60.26	42.679	7.76

CIP, cast-in-place.

Table 4. Comparison of the mechanical properties of the CIP-concrete of the PCBC specimens

	($V_f = 0\%$)	($V_f = 0.5\%$)	($V_f = 1.0\%$)	Increase compared to ($V_f = 0\%$)	
				($V_f = 0.5\%$)	($V_f = 1.0\%$)
Average of compressive strength (MPa)	50.86	47.36	60.26	-6.88	18.48
Average modulus of elasticity (GPa)	31.518	33.782	42.679	7.18	35.41
Average modulus of rupture (MPa)	5.99	6.06	7.76	1.17	29.55

CIP, cast-in-place; PCBC, precast concrete beam-column.

load was applied in the negative direction (going down) until failure.

The crack development at every level of displacement was recorded on the test specimen using different colors to indicate the different loading directions.

4. Results and discussion

4.1. Mechanical properties of concrete

Table 3 provides the average concrete cube strength, modulus of elasticity, and modulus of rupture of the precast units and the CIP-concrete for specimens P2, P3, and P4, each of which was determined on three samples.

Table 4 presents a comparison of the mechanical properties of the CIP-concrete. Adding 0.5% of steel fiber by volume to the concrete only caused

a slight increase in values compared to that of the plain concrete in terms of the compressive strength, modulus of rupture, and modulus of elasticity (i.e., -6.88%, 1.17%, and 7.18%, respectively). However, adding 1.0% steel fiber by volume of concrete increased the compressive strength, modulus of rupture, and modulus of elasticity significantly (i.e., 18.48%, 29.55%, and 35.41%, respectively).

The slight decrease in terms of the compressive strength (i.e., -6.88%) of the CIP-P3 concrete was caused by the amount of water added to the concrete mix. As can be seen in Table 2, the CIP-P3 mix contained more water (i.e., 213 kg/m³) compared with the CIP-P2 mix (i.e., 210 kg/m³). This was done in order to maintain the workability of the fresh concrete due to the inclusion of the steel fibers.

The effect of steel fibers on the compressive strength is not too significant. This is in line

with the results of Oh [18]; this study found that the compressive strength increase was about 17% when steel fibers ($V_f = 2\%$) were added to the concrete, and less than 10% at $V_f = 1\%$. In addition, Altun *et al.* [3] that found that the addition of 30 kg/m^3 of steel fiber decreased the compressive strength of a C30 concrete by approximately 11.5%.

Figure 3 shows the failure mode of the MOR prism concrete. The plain concrete prisms ($V_f = 0\%$) failed suddenly once the first crack occurred and fractured into two parts. On the other hand, the FRC prism exhibited cracks but did not fully fracture. This is because the random steel fibers bridge the cracks and resist the cracks from developing and widening through the de-bonding and pulling-out mechanism and so no sudden fracture occurred.

Fig. 3. Failure mode of the flexural test of the prism, (A) plain concrete, (B) steel fiber reinforced concrete.

The effect of steel fibers in terms of the load-deflection relationship of the prism tests is presented in Figure 4. It can be seen that the plain concrete ($V_f = 0\%$) is a brittle material, with sudden failure as the first crack occurred. This means that the first-crack load is the peak load. A bet-

ter performance was observed for the SFRC used in the CIP-connection of the PCBC specimens P3 and P4. The curves for the CIP of P4 ($V_f = 1\%$) showed that after cracking, the load continued to increase. Once the peak load was achieved, the load decreased gradually with the prism exhibiting significant deflection. However, the CIP-connection curve of specimen P3 ($V_f = 0.5\%$) showed that after cracking, the load decreased gradually accompanied by large deflection. Both SFRC material with $V_f = 0.5\%$ and 1% showed better ductility in comparison to the plain concrete material.

Fig. 4. Load-deflection comparison for the prisms made of the concrete used in the CIP-connections of (A) Specimen P2 ($V_f = 0\%$); (B) Specimen P3 ($V_f = 0.5\%$) and P4 ($V_f = 1\%$). CIP, cast-in-place.

The curves of the SFRC with $V_f = 1\%$ could be categorized as deflection-hardening, as the peak load was higher than the first-crack load. The curves of the SFRC with $V_f = 0.5\%$ could be cat-

egorized as deflection-softening, due to the first-crack load and the peak load being similar.

There is a slight difference in the curves for the CIP-concrete containing 1% steel fibers. This highlights the potential variability when incorporating steel fibers in a mix and the difficulty in obtaining an even distribution of the fibers.

The energy which could be absorbed by the specimen during loading can be calculated as it is equal to the area under the load-deflection curves [19]. It is observed that the area under the load-deflection curves of the SFRC with $V_f = 1\%$ is about 40% larger than that of the SFRC specimen with $V_f = 0.5\%$.

4.2. The effect of the steel fiber in the CIP-connection on the behavior of the beam-column connections

The load-deflection hysteretic loops of the PCBC connections of specimens P2, P3, and P4 are presented in Figure 5. The load was applied at the beam tip, and the deflection was measured at the beam tip. This data is used to investigate the energy dissipation, which is presented in the next section.

It appears that the presence of steel fibers in the CIP-concrete has enhanced the characteristics of the hysteretic load-deflection curves of the PCBC specimens, slightly increasing the maximum load and producing wider loops at each displacement level; this indicates a greater dissipation of energy. The steel fibers within the CIP-concrete also delay crack formation and effect crack propagation. For instance, the diagonal cracks in the joint core on PCBC specimen P2 (with plain concrete as CIP-connection) appeared immediately and occurred at one load level (at the deflection level of 12 mm), as can be seen in Figure 6A, whereas the diagonal cracks (diagonal red cracks in specimen P4, see Figure 6B) were shorter and occurred during several load levels (at the deflection levels of 12 mm, 18 mm, and 36 mm) and were spread-out over the surface of the concrete. This behavior is in agreement with the data of the MOR test, which showed that the first crack in the plain concrete ($V_f = 0\%$) is followed by a sudden drop in the load-deflection curve (see Figure 4A). However, when the first

Fig. 5. Load-deflection hysteresis loops of PCBC specimens: (A) P2 with $V_f = 0\%$, (B) P3 with $V_f = 0.5\%$, (C) P4 with $V_f = 1\%$. PCBC, precast concrete beam-column.

crack occurred in the SFRC MOR test ($V_f = 1\%$), there was subsequent deflection (see Figure 4B). This means that, once cracked, the steel fibers still bridge the cracks and, therefore, restrict the crack propagation (until the steel fibers were pulled-out from the concrete matrix) and delay internal cracks reaching the surface until higher displacements are achieved.

Fig. 6. Crack pattern in the joint core (CIP-concrete): (A) P2 with $V_f = 0\%$, (B) P4 with $V_f = 1\%$. CIP, cast-in-place.

The energy dissipation of a structure indicates the ability of a structure to resist the loading from an earthquake through inelastic deformation. Greater energy dissipation will improve the seismic performance of a structure. Energy dissipation was calculated as the area enclosed by the hysteretic loops in the corresponding beam tip load vs. deflection graphs [12, 20]. Cumulative energy dissipation (CED) during the reverse cyclic load test was calculated by summing the energy dissipated

in consecutive load-displacement loops throughout the test, as presented in Figure 7. In order to eliminate the effects of the concrete strength variation in the different beam-column specimens, the calculated energy dissipations were normalized with respect to the area of elastic-perfectly plastic rectangular stress block at each cycle using Eq. (1) [12, 21]. Figures 8 and 9 present the CED and the normalized energy dissipation (NED) of PCBC specimens P2, P3, and P4, respectively.

Fig. 7. Definition of normalized dissipated energy normalizing hysteretic energy dissipation at each load cycle [12].

Fig. 8. CED of PCBC specimens. CED, cumulative energy dissipation; PCBC, precast concrete beam-column.

From Figure 8, it is clear that the PCBC specimens with steel fibers had a higher CED than the PCBC specimen without fibers, i.e., about 17.7%. From Figure 9, we observe that specimen P4 has the highest result of NED, particularly at the beginning of the test, where more cracks were developed at this stage. The presence of steel fibers in the CIP-concrete appears to improve the energy dissipation capacity of the PCBC joint.

Fig. 9. NED of PCBC specimens P2, P3, and P4. PCBC, precast concrete beam-column; NED, normalized energy dissipation.

Theoretically, specimen P4 should have performed better than specimens P2 and P3 due to the greater fiber content. In trying to unearth an explanation for the energy dissipation results, an examination of the experimental procedure during the construction of the specimens was performed. It was discovered that tape had been stuck to the polystyrene void formerly used to manufacture the precast U-beam for P4; this had inadvertently caused a smooth surface to the inner walls of the U-beam, lowering the bond at the interface between the U-beam and the beam CIP-concrete. The lower energy dissipated by P4 can be explained by the lack of bond/reduced composite behavior between the two components; the effect of this can also be seen in Figure 8 where the curve decreases (the curve for specimen P3 increases).

$$\text{Normalized Energy Dissipation (NED)} \quad (1)$$

$$= \frac{A}{4V_{max}\delta_{max}}$$

where V_{max} is the average of the maximum load; δ_{max} is the average of displacement for positive and negative loading directions; and A is the area enclosed by the hysteretic loops.

Stiffness degradation, in this study, was measured using the secant stiffness (K_{sec}) principle (peak-to-peak stiffness); this was calculated at every displacement level (3 mm, 8 mm, 12 mm, 18 mm, 24 mm, 36 mm, 48 mm, and 60 mm). Secant stiffness is defined as the slope of the straight

line between the maximum load of the positive and negative direction, at the last cycle of each displacement level or drift ratio level [14, 20]. The stiffness-deflection relationship is presented in Figure 10, and used to compare the stiffness degradation of the beam-column specimens from one cycle to the following cycle.

Fig. 10. Secant stiffness degradation of the PCBC specimens. PCBC, precast concrete beam-column.

Figure 10 shows that for all specimens, as the deflection increases, the stiffness decreases. At the beginning of the test (the deflection level of 3 mm), P4 had the highest secant stiffness.

A high energy dissipation does not mean a high secant stiffness. Energy dissipation is obtained from the area enclosed by the load-displacement loop. Steel fibers within the CIP-concrete cause wider loops, meaning it has higher energy dissipation. On the other hand, the secant stiffness is the slope of the straight line between the maximum load of the positive and negative direction. At displacement level of 36 mm, 48 mm, and 60 mm the maximum load in both directions for all PCBC specimens are about same, hence the similar secant stiffness.

The effect of steel fiber on the stiffness degradation was only apparent at the beginning of the test (from deflection level of 3 mm to 24 mm). This finding is also in agreement with Ganesan et al. [5] and Marthong and Marthong [8]. From the deflection level of 36 mm to 60 mm, the curves appear similar. However, the connections with steel fibers

in them absorb more energy (the loops were fatter in comparison with the connection without steel fiber) and more energy absorption is also indicated by the extended flatter part of the curve representing the plastic behavior.

The steel fibers that are spread evenly within the concrete will increase the stiffness of the uncracked concrete matrix, which also therefore increases the joint stiffness. The greater the quantity of steel fibers, the larger the increase in joint stiffness. However, this only appeared to apply at the beginning of loading. After the concrete has cracked it appears that the influence of steel fibers on the joint stiffness decreases.

5. Conclusions

Based on the test results, the mechanical properties of SFRC tests and its application on the connection of precast beam-column under the reversed cyclic test, the following conclusions are drawn:

1. The addition of 0.5% and 1% of steel fibers by volume appear to change the properties of concrete from a brittle material to a more ductile material. The concrete with steel fiber $V_f = 0.5\%$ has deflection-softening characteristic, whereas the concrete with steel fiber $V_f = 1\%$ has deflection-hardening characteristics (as shown in MOR test). This means that once the applied tensile stress exceeds the tensile strength capacity of the concrete and the first crack occurs, the stress in the beam element can still increase. During this hardening stage, the steel fibers, when spread uniformly within the concrete matrix, prohibit the crack extension until they are pulled out of the concrete matrix.
2. The use of SFRC, for the material connection of a typical exterior precast concrete beam-to-column joint in this study, can alter the pinch load-deflection hysteretic loops, making them wider, which means an increase in the energy dissipation (about 17%) and the ductility of the joint in comparison with the PCBC joint using plain concrete as the material connection (CIP-concrete). The

addition of fibers in concrete is not to improve strength (even though there is a little improvement), but primarily to control cracking (through friction forces between steel fiber and the concrete), and improve the toughness or energy absorption capacity. It is clear from the tests that the steel fibers delayed the onset of cracking and slowed down the crack propagation, resulting in shorter cracks in the joint core and the top surface of the beam core (as shown in specimen P4: $V_f = 1\%$). This behavior correlates well with the deflection-hardening characteristic found from the MOR test.

3. The steel fibers will also increase the joint stiffness until the first crack occurred. After the concrete has cracked, the influence of steel fibers on the joint stiffness decreases.
4. In general, the application of the Dramix RC 65 35 BN steel fiber to the concrete used in the CIP-connection of a typical PCBC joint developed by Noorhidana & Forth [12], can improve the performance of the joints against cyclic loads.

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Conflict of interest

V. A. Noorhidana and J. P. Forth state that there are no conflicts of interest.

Ethical approval

This article does not contain any studies with human participants or animals performed by any of the authors.

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