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AN EXPERIMENTAL STUDY ON PRECAST CONCRETE BEAM-TO-COLUMN CONNECTION USING INTERLOCKING BARS

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Key words: beam-to-column connection, precast concrete, interlocking bars, development length, static loading, quasi-static loading

Abstract. A new type of precast concrete exterior beam-to-column connection is introduced in this paper. The connection consists of a precast U-beam, precast column with corbel, and interlocking bars to connect the precast column and precast beam; these bars, act as the flexural reinforcement to withstand the hogging and sagging moment subjected during testing. The beam-column joint was designed according to the strong column-weak beam principle. The static-monotonic loading was applied to the first specimen (P1), while a quasi-static loading based on displacement control was applied on the second specimen (P2). Tip deflection and specimen cracking were monitored during the tests.

Plastic hinges were formed in the beam of both specimens. P1 has no significant cracks in the joint, while P2 has 'X' cracks in the joint but it was still deemed acceptable. The precast columns of both P1 and P2 were free from cracks. The connection met the acceptance criteria presented in ACI 374.1-05. The development length of the interlocking bars, i.e. 800 mm from the column face, was sufficient to transfer the load from the beam to the joints and columns.

1 INTRODUCTION

Precast concrete systems have become more popular in the last few decades. These systems have some advantages in comparison with the conventional in-situ construction methods, i.e. quality, time and cost [1-3]. Nurjaman et al. [4] concluded that the reduction in the construction time is around 30%. In addition, the cost efficiency of using a precast concrete system is around 20% compared to that of conventional systems [5]. Precast concrete systems are also one of the construction systems which satisfy the 'green construction' rules. Green construction is a part of the trend for sustainable construction that has been developing to face the issue of global warming [4].

On the other hand, precast concrete systems still face some technical problems, e.g. the behaviour of the connection and the ease of implementation of these joints. Sometimes, the connection is satisfactory, but is too complicated a detail or requires a high degree of precision, which causes difficulties in implementation. Furthermore, it can extend the construction time and increase construction costs.

Many types of connection have been developed, particularly beam to column connections in moment resisting frames, however, they are often partially bolted/welded/pre-stressed/cast-in-place (CIP) connection. Generally, though, they are categorized as either dry connection or wet connection, or strong connection and ductile connection [6].

Each type of connection has its own advantages and disadvantages. Welded connections, even though they satisfy strength and stiffness, can result in excessive heat, which can damage or cause cracks in adjacent precast concrete [7]. Furthermore, the implementation of this connection needs skilled labourers who can guarantee the quality of welding in the connection [8].

Bolted connections seem to be the easiest method to connect the precast elements on site. Unfortunately, these require a high degree of precision in placing the channels or steel plates before casting the precast elements. This is not easy to be done due to the sliding risk [8]. Another study anticipated this sliding risk by providing sufficient tolerance in the bolt holes. However, the hole tolerances caused a loss of initial stiffness in the connection [9].

The cast-in-place (CIP) concrete systems are more monolithic connections and are recommended for seismic resistant buildings [1]. It also provides more tolerance in precast connections. Unfortunately, it takes longer to construct as the concrete has to gain its strength; and it needs additional formwork and scaffoldings on site. All of these will lead to an increase in cost and time for construction.

The new connection developed in this paper was designed as a ductile connection. It combines a precast beam, corbel, interlocking bars and cast-in-place concrete. The precast beam is a combination of a solid beam and a U-beam. The interlocking bars were designed as flexural reinforcement which can withstand both hogging and sagging moments in the beam. The precast column is categorized as a one-piece multi-story column construction [10] which intends to accelerate the erection work for building the precast concrete column [11]. When constructing, the precast beams are put on the corbel, and then the interlocking bars are placed through the joint in the precast column and inside the U- beam. The concrete is then cast monolithically in the joint core and connection region.

The new connection offers some advantages. Particularly, the problem regarding the high precision which is needed for assembling the existing precast elements – this is minimized in this case. The use of the partial U-beam negates the use of formwork for casting the connection region. In addition, it lowers the volume of cast-in-place concrete. The number of scaffoldings can be minimized due to the existence of the corbel to support the precast beam. This new connection also avoids the use of welding, bolts and the pre-stressing process which therefore leads to a reduced need skilled labour and a reduction in the construction time. Overall, this new connection can be expected to offer a more economical and practical method.

The aim of this study is to develop a new ductile connection for a precast concrete moment-resisting frame. An experimental study of beam-column connections tested under static-monotonic and quasi-static loading was performed. The behaviour of the connection was evaluated based on the acceptance criteria in ACI 374.1-05 [12].

2 EXPERIMENTAL WORK

2.1 Detail of the Test Unit

The test specimen represents the exterior beam-to-column joint of a moment resisting frame. The length of the members was determined by the contra-flexure points resulting from a computer analysis. The column was designed to be stronger than the beam to meet the requirement of the strong column-weak beam principle, which is used imperative for seismic resistance structure. The reinforcement detail is presented in **Figure 1**.

The dimension of the precast column is 300 mm x 300 mm; the height is 2000 mm. The 12 reinforcing bars of 16 mm-diameter are used as longitudinal reinforcement; the reinforcement ratio is 2.67%. This meets ACI provisions [15] which states that the reinforcement ratio of the longitudinal steel bar in a column should be between 1% to 6%. The 8 mm-diameter steel shear link was spaced at 100 mm centres along the height of the column. There is a gap at mid-height of the precast column which is used for placing the interlocking bars. The height of the gap is 300 mm and is equal to the beam depth. The precast column has a corbel which is used for seating the precast U-beam.

The precast beam consists of two parts, i.e. the precast U-partial beam and the cast-in-place (CIP) reinforced concrete beam core. The outer dimension of the precast U-beam is 250 mm x 300 mm, and 1250 mm in length. The beam core is 150 mm x 250 mm in dimension, with a length of 800 mm. The beam core was cast monolithically with the joint core. The longitudinal bars of the beam core consist of 4 interlocking bars of 12 mm diameter, which act as both negative and positive moment reinforcement.

2.2 Test Setup and Instrumentation

The test setup used in this study is explained as following; both the column ends were restrained by steel plates which were bolted on to the test rig, while the beam end is free. The load was applied vertically to the tip of beam. No vertical axial load was applied to the top of the column since this tends to enhance the joint shear strength; hence, this is a worst loading

case scenario [13]. The static-monotonic loading was applied to the specimen P1 until failure. While a quasi-static loading was subjected to specimen P2 using the displacement control at 3, 8, 12, 18, 24, 36, 48, and 60 mm, as shown in **Figure 2**.

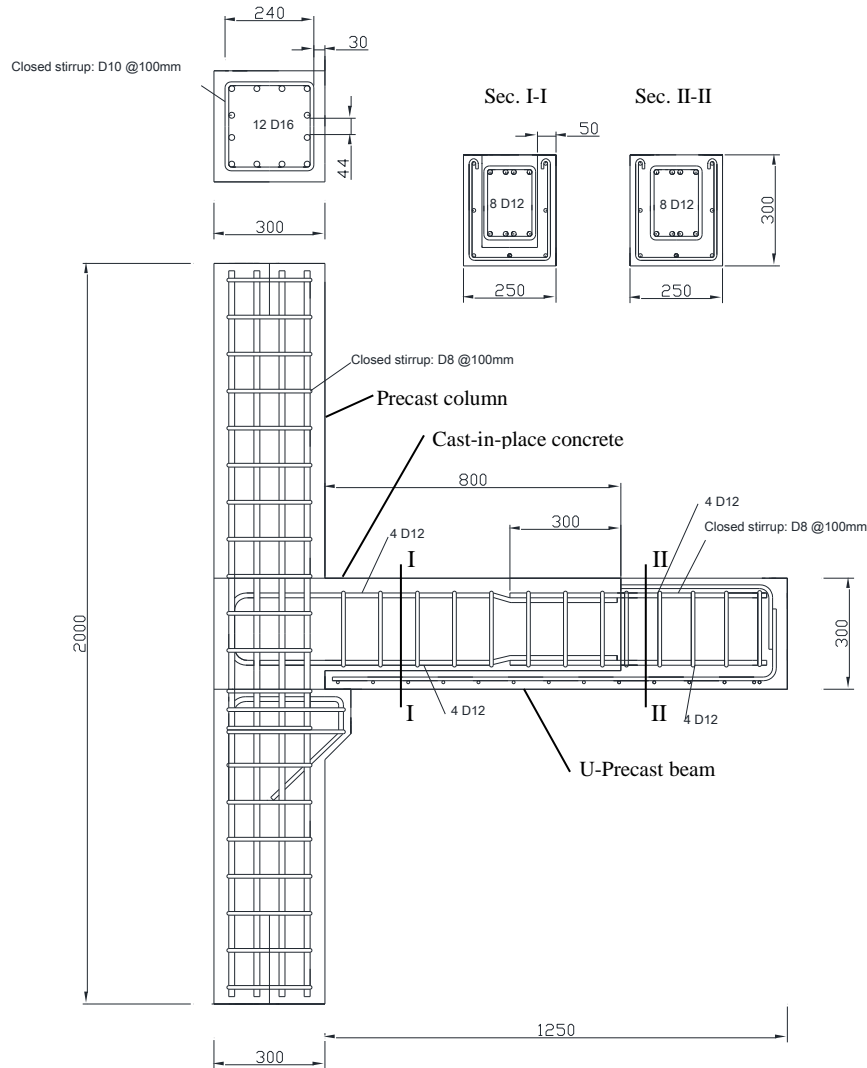


Figure 1: Details of P1 and P2

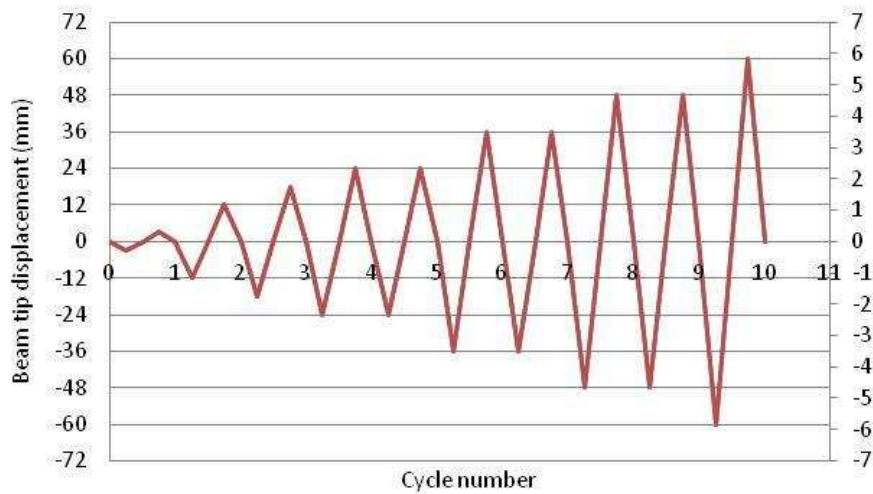


Figure 2: Load history for the reversed cyclic load test used for specimen P2

2.3 Material Properties

The precast elements were cast using normal concrete with a compressive strength of 30 MPa. The cast-in-place (CIP) were cast with higher compressive strength concrete. A maximum size of 10mm coarse aggregate was used to ensure that the fresh concrete filled the congested section. The concrete slump was between 75 and 125 mm.

Table 1 shows the compressive strengths of the concrete. The compressive strength was obtained from 100 mm x 100 mm cubes, where the concrete cylinder strength f_c' was taken as 80% of the cube strength f_{cu} [14]. Yield strength of the steel reinforcing bars was 500 MPa; Standard Deviation was 30 MPa.

Table 1: Compressive strength of concrete

Specimen	Average cube strength f_{cu} (f_c')* N/mm ²	
	Precast concrete	CIP concrete
P1	38.07 (30.46)	59.12 (47.30)
P2	40.95 (32.76)	50.8640.69)

*) Concrete cylinder strength was taken from 80% of the cube strength [14].

Table 2: Experiment results of P1

Items	Load (kN)	Deflection (mm)
First-crack	15.00	2.0
Yield	51.88	16.0
Maximum	59.80	23.5
Ultimate		46.5

3 TEST RESULT OF P1

3.1 Failure Mode

Cracking was observed throughout the test. **Figure 3** presents the crack-pattern of the precast concrete beam-column joint specimen P1 under negative bending moment (load applied to beam vertically downward).

The first crack appeared on the base of the beam adjacent to the column, in line with the tip of the corbel, where the maximum moment occurs. The first crack appeared on the surface on the beam core at the load level of 15 kN; the crack-width was 0.05 mm. This was followed by a delamination crack on the top surface of the beam core and the wall of the precast U-beam at the same load level. The next cracks again occurred on the beam core at a distance of 150 mm and 300 mm away from the column. In general, all cracks could be categorised as typical flexural cracks which were distributed along the connection region (cast-in-place) of the beam. The flexural cracks that occurred on the top surface of the beam were distributed at a distance of $2 \times$ the depth of beam (h) from the column face, which means $2 \times 300 \text{ mm} = 600 \text{ mm}$, which is the plastic hinge length stated in ACI Code.

The wall of the precast U-beam developed cracks at a load level of 20 kN. The cracks propagated vertically, perpendicular to the beam axis, and were an extension of the crack seen earlier in the beam core.

The longitudinal bars of the U-beam started to slip at a fairly high load level, i.e. 55 kN. This was indicated by a horizontal crack in the bottom of the U-beam which extended from the support until 200mm from the corbel. That means that the beam core and U-beam acted as a composite beam and that there was therefore good bond between the walls of the U-beam and beam cores very well.

No significant cracks occurred in the joint core. A small delamination crack at the gap between the joint core and the upper precast concrete appeared at a load level of 25 kN. The crack stopped at the middle of the column width.

The final gap-width between the precast beam and the precast column at the end of the test was 8 mm. While, no crack occurred on the corbel.

3.2 Load-Displacement

Figure 5 shows the load-displacement relationship of the beam tip. From the curve, an attempt to estimate the first crack, yield, and ultimate load of the joint can be made. All values are presented in **Table 2**.

From the load-deflection graph, when slope of the curve changes slightly, the first-crack appears to be at 15kN. This corresponds well with the load at which the first crack was visible during the test.

Although the yield condition is not clear either physically or from the graph several theories have been proposed by researchers to determine the yielding point from a load-deflection relationship [19-21]. By using the theory that the yield deflection is determined by taking the secant stiffness at 75% of the ultimate lateral load (H_u) of the real system [15], then the yield deflection can be determined and is presented in **Table 2**. The ultimate condition,

however, is very clear and the peak load, the deflection at the peak load and the maximum deflection can be determined easily.

4 TEST RESULT OF SPECIMEN P2

4.1 Failure Mode

Figure 4 shows the crack-pattern of specimen P2. The first crack occurred on the top surface of the beam core at a load of 15 kN, during the first cycle of negative loading (at the displacement level of 3 mm). The new cracks on the top surface beam core continued to develop at displacement levels of 8 mm, 18 mm, 24 mm, and 36 mm in negative loading along the connection region. The cracks extended to the wall of the U-beam at displacement level of 8 mm until 36 mm in both negative and positive loading directions.

Cracks in the joint-core initially appeared during the first cycle of displacement level of 8 mm. Then, the cracks extended forming an “X” crack in the joint core; the propagation of cracks stopped at the displacement level of 36 mm (DR = 3.5%). A thin delamination crack in the gap between the joint core (CIP-concrete) and the precast concrete (top and bottom part) occurred at the displacement level of 18 mm.

After the displacement level of 36 mm (DR = 3.5%), no more cracks developed in the joint core; the cracks were concentrated in the beam adjacent to the column. At the displacement level of 60 mm, the opening between the U-beam and the column was 15mm and the part of the U-beam supported by the corbel was crushed.

Overall, P2 experienced a ductile failure with the plastic hinge forming in the beam.

4.2 Load – Displacement

Figure 6 illustrates the beam tip load-deflection for specimen P2 (the quasi-static load history is given in **Figure 2**). Two full cycles were applied at every level of displacement.

In order to understand the performance of this new connection when subjected to quasi-static loading, the results of P2 were evaluated based on ACI 374.1-05 (Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary), as presented in **Table 3**.

The hysteresis loop appeared stable from the beginning of test until the last cycle at the deflection level of 60 mm. There was no significant strength degradation after the peak load. All the maximum loads at each deflection increment were more than 75% of the peak load. The connection fulfilled the requirements stated in ACI 374.1-05, that the peak load of the last cycle of the drift ratio of 3.5% should be not less than 0.75 of the peak load.

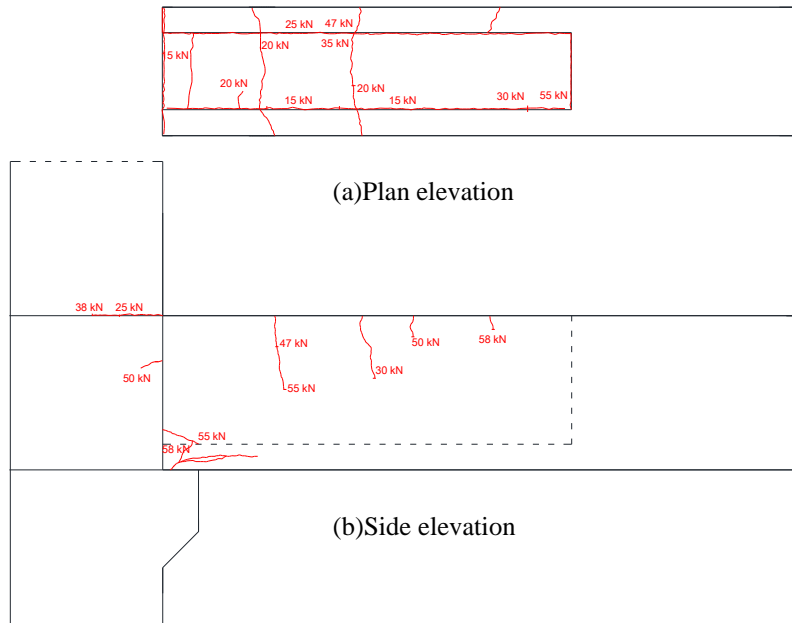


Figure 3: Crack-pattern of specimen P1

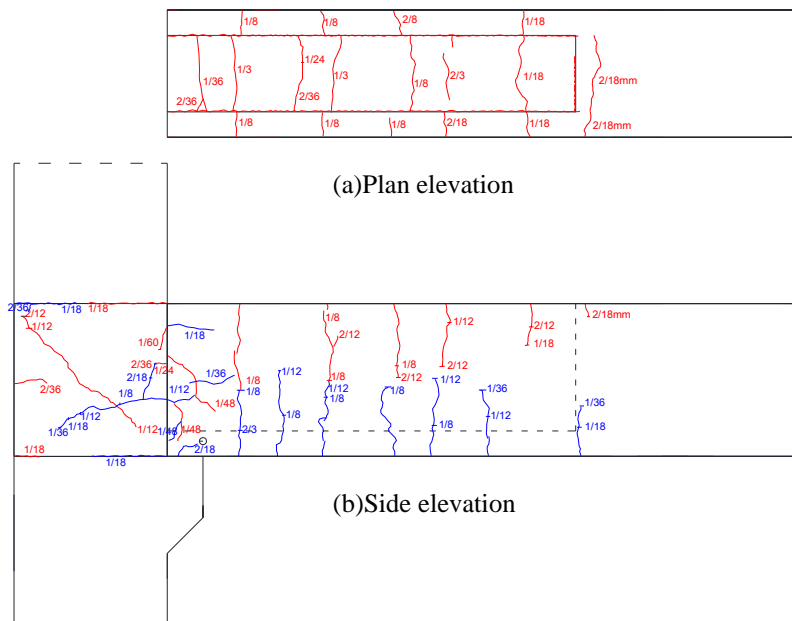


Figure 4: Crack-pattern of specimen P2

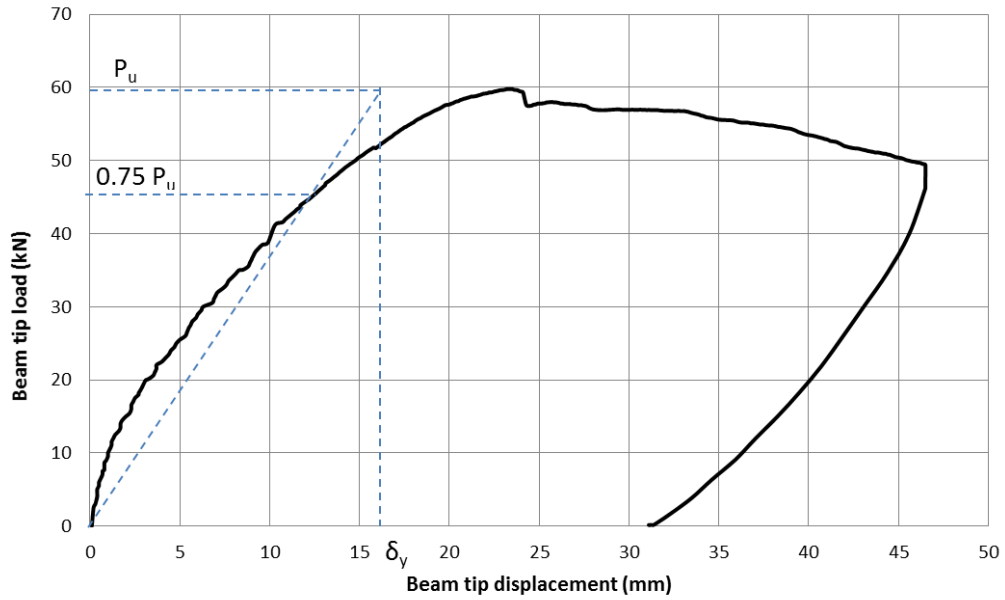


Figure 5: Load – displacement relationship for specimen P1

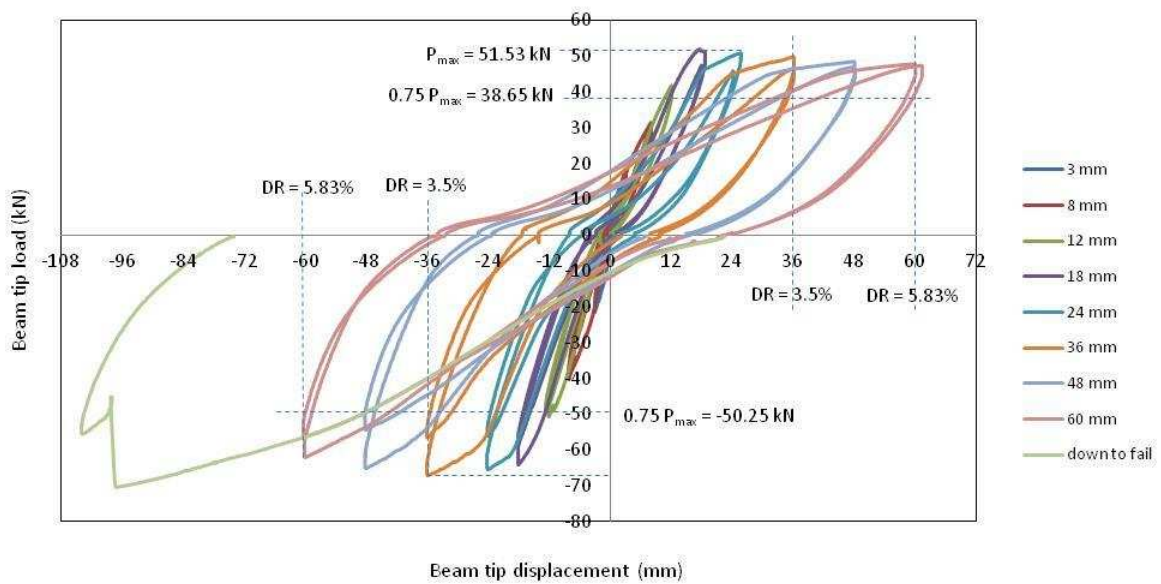


Figure 6: Load – displacement relationship for specimen P2

The ACI 374.1-05 requires that the relative energy dissipation ratio shall not be less than 0.125 at the last cycle at the drift ratio of 3.5%. The definition of relative energy dissipation ratio (β) according this code is the ratio of the area of the hysteresis loop to the area of a circumscribed parallelogram defined by the initial stiffness during the first cycle and the peak resistance, as presented in **Figure 7**. As shown in **Table 3** specimen P2 satisfied this requirement.

The secant stiffness is evaluated at the peak-to-peak stiffness of the load-displacement curves. The increasing the displacement level caused the decreasing of the stiffness of P2 due to the cracks formation and plastification of material. ACI 374.1-05 requires the secant stiffness at drift ratio of 3.5% (from positive to negative loading) should not be less than 0.05 times of the initial stiffness. As shown in **Table 3**, P2 satisfied this requirement. Even at the displacement level of 60 mm (DR = 5.83%) the stiffness was still over 0.05 times the initial stiffness.

Table 3: Comparison between test results of P2 and Acceptance Criteria ACI 374.1-05

Items	Specimen P2	Acceptance Criteria*
P_{2nd}/P_{max} : Negative loading	0.84	≥ 0.75
Positive loading	0.89	
β	0.23785	≥ 0.125
$K_{0.035}/K$	0.2863	≥ 0.05
$K_{0.035}/K'$	0.167	≥ 0.05

*) ACI 374.1-05

β = relative energy dissipation ratio

K and K' = initial stiffness for positive and negative loading for first cycle.

$K_{0.035}$ = secant stiffness at drift ratio of 3.5% .

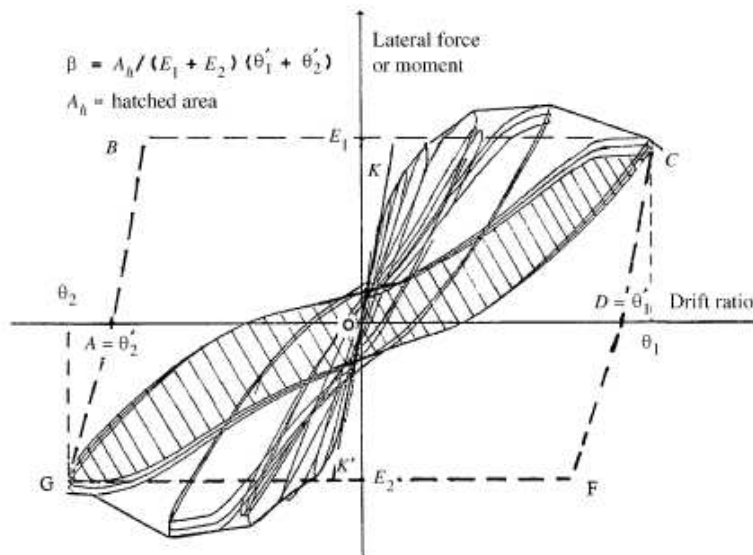


Figure 7: Relative energy dissipation ratio [12]

5 CONCLUSIONS

- 1) This type of beam-column connection performed well under both static-monotonic and quasi-static loading. Both specimens exhibited a flexural failure mode and behaved monolithically until failure.
- 2) With reference to the structural behaviour under static-monotonic loading (P1),
 - The first crack occurred on the top surface of the beam core at a load of 15 kN.
 - The cracks were typically flexural cracks, which occurred within the plastic hinge length of the beam.
 - There was no significant crack in the joint core.
 - The maximum deflection was 46.5mm. With a yield-deflection of 16 mm, the displacement ductility was $46.5/16 = 2.9$.
- 3) With reference to the structural behaviour under quasi-static loading (P2)
 - The first crack occurred on the top surface of the beam core at a load of 15 kN during the first cycle of loading.
 - The cracks were typically flexural cracks, which occurred within the connection region and in the wall of the U-precast beam.
 - An 'X' crack formed in the joint core.
 - The plastic hinge was formed in the beam adjacent to the column face.
 - The connection met the acceptance criteria in ACI 374.1-05.
- 4) The interlocking bars connecting the joint core and the beam core can act as flexural reinforcement for the beam core, under both static and cyclic loading. The development length of the interlocking bars, i.e. 800 mm from the face of the column, is considered necessary to distribute the internal forces without bond slip occurring at the beginning of loading; this allows the interlocking bars to develop their true tensile strength through to yield.

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