AN EXPERIMENTAL STUDY ON BEAM-COLUMN CONNECTIONS WITH PRECAST CONCRETE U-SHAPED BEAM SHELLS

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ABSTRACT :

The beam-column connections in a precast concrete moment resisting frame were tested to investigate their earthquake resistance. In the precast concrete moment resisting frame, the U-shaped beam shell and the column are constructed with precast concrete. The beam core and the beam-column connection are monolithically constructed with cast-in-place concrete. The test specimens were a cruciform sub-assemblage of the beam, column, and their connection. Five precast concrete specimens and one conventional monolithic specimen were tested for cyclic loading. The parameters for this test were the reinforcement detail for the precast beam shell, stirrup spacing in the beams, and length of the beam shell seated on the column. Based on the test results, the strength, stiffness, energy dissipation capacity, and deformation capacity were evaluated. The test results showed that regardless of the test parameters, the beam-column connections showed good strength and deformation capacity. However, due to bond-slip of the reinforcing bars located inside the precast concrete U shell, the energy dissipation capacity decreased. Because of the shear area of the connection, which was decreased by the beam shell seating on the column, the shear crack and shear deformation of the beam-column connection were increased.

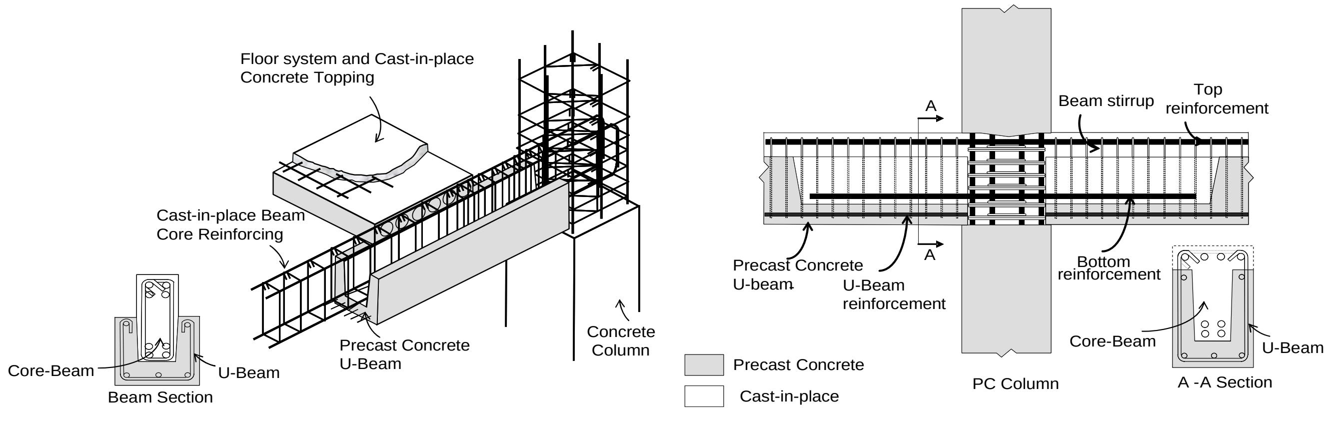
KEYWORDS: Precast Concrete, Beam-Column Connection, Energy Dissipation, Ductility, Stiffness

1. INTRODUCTION

The precast concrete moment resisting frame is constructed by assembling the precast concrete beams and columns on construction site. Therefore, the beam-column connection of the precast concrete moment resisting frame is vulnerable to severe loading including earthquake. Generally, the beam-column connection of the precast concrete moment resisting frame is classified as the emulated monolithic connection and the non-emulated connection. The goal for an emulated monolithic connection is to make the connection as strong as the connection in the cast-in-place concrete structure, so that the structural performance of the precast concrete structure is comparable to that of the cast-in-place concrete structure.

Figure 1 shows a type of precast concrete moment resisting frame. In this structure, the precast concrete beam has U-shaped cross section (U-shell). In the construction site, the U-shell beams are seated on the column. Longitudinal rebars are placed inside the U-shell and concrete is cast monolithically in the beam core and the beam-column connection. As shown in Figure 1(a), in the early development by Park and Bull (1986), the precast concrete shell was used as the formwork for core concrete, and only the core concrete enclosed by the stirrups was used for structural purpose.

In the later development of the U-shaped shell construction (Lee et al. 2004), the stirrups placed in the precast concrete shell was connected to the cast-in-place core concrete so that the core concrete and the precast concrete shell behave monolithically under loading. Further, to enhance the speed of the erection work for precast concrete columns, one-piece multi-story column construction was developed. In this one-piece multi-story column, the upper story- and lower story- columns are connected by using only longitudinal rebars without using concrete.



(a) Park & Bull (1986) (b) LEE (2004)

Figure 1. Beam-column connections with precast concrete beam with U-shaped cross-section

The beam-column joint was filled with cast-in-place concrete (Figure 1(b)). For the erection of this one-piece multi-story column, longitudinal rebars of the beams cannot be extruded from the beam ends. For this reason, continuous longitudinal bottom bars are placed inside the U-shell after the two continuous beams are seated on the column at the beam-column connection (Lee et al. 2004). Though this precast concrete construction method is useful for fast construction, the results of the test performed by Lee et al. (2004) showed that the load-displacement relationship of this beam-column connection exhibited very poor energy dissipation capacity, showing severe pinching. Further, severe diagonal cracks and concrete spalling occurred at the beam-column connection.

In the present study, an experimental study was performed for full-scale specimens of the beam-column connection assemblage with the precast concrete U-shell beam. The structural performance of the specimens including strength, stiffness, energy dissipation, and deformation capacity were evaluated. We investigated the causes of such poor structural performance of the beam-column connections reported in the previous study. Based on the results, design considerations for improving the structural performance were proposed.

2. SPECIMENS AND TEST SETUP

The test specimens were full scale models of interior beam-column connections with precast concrete U-shell beam. The configurations of the test specimens are shown in Figure 2. The cross-section of the beam was 400×700mm, and the beam net span between the supports was 4762mm. The height of the column was 2700mm, and its cross-section was 600×750mm. The specimens were designed according to the strong column - weak beam design concept so that the inelastic damage of the column did not occur.

Figures 2 (a) ~ (e) show the details of SP1 ~ SP5 specimens having precast concrete U-shell beam, respectively. In SP1, four 32mm-diameter re-bars were arranged at the top of the cross-section of the beam (Figure 2(a)). Four bottom bars with the same diameter were placed inside the precast concrete U-shell.

Table 1. Details of test specimens

Specimens	Reinforcement in Beams			Seated length of U-beam shell	Steel Angle-Strengthening for U-beam end
	Top reinforcement (ratio,%)	Bottom reinforcement (ratio,%)	Stirrups		
CP	4-D32(1.23)	2-D25, 2-D29(0.89)	D13@160 (0.22)	-	
SP1	4-D32(1.23)	4-D32(1.66)	D13@120 (0.30)	50mm	
SP2	4-D32(1.23)	4-D32(1.66)	D13@120 (0.30)	50mm	○
SP3	4-D32(1.23)	4-D32(1.66)	D13@160 (0.22)	50mm	○
SP4	4-D32(1.23)	4-D32(1.66)	D13@120 (0.30)	65mm	
SP5	4-D35(1.49)	4-D35(1.99)	D13@120 (0.30)	50mm	○

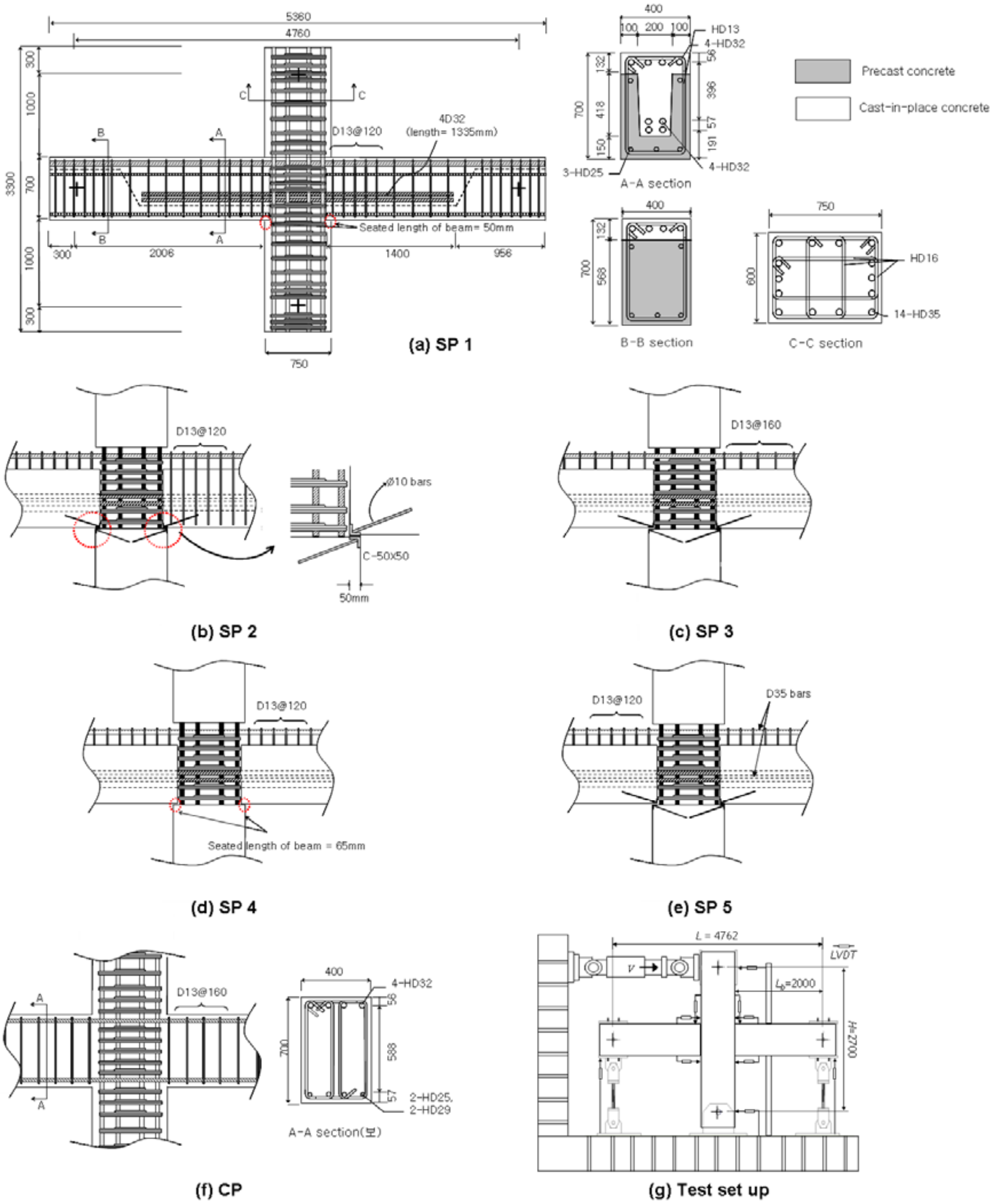


Figure 2. Dimensions and details of test specimens (mm) and test set up

Spacing of the stirrups in beams was 120mm. The length of the precast concrete U-shell beam seated on the column was 50mm. The details of SP2 specimen was identical to those of SP1, except for the strengthening for the end of the U-shell beam. (Figure 2(b)) In SP2, steel angles were installed to strengthen the contact surface between the U-shell beam and the column so that spalling of the column concrete cover did not occur at early loading. The details of SP3 specimen (Figure 2(c)) were identical to those of SP2, except for its stirrup spacing in the plastic hinge region. In SP3, the stirrup spacing was increased from 120mm to 160mm. The details of SP4 were the same as those of SP1. In SP4 specimen (Figure 2(d)), the seated length of the U-shell beam was increased from 50mm to 65mm without using steel angles for strengthening. In SP5 specimen (Figure 2(e)), four 35mm-diameter rebars were used each at the top and the bottom of the beam to increase the flexural capacity of the beam. Other details of SP5 were the same as those of SP2.

Figure 2(f) shows the monolithic reinforced concrete specimen CP, as a reference specimen. The details of the column were identical to those of SP specimens. In the beams, four 32mm-diameter re-bars were placed at the top of the beam, as was in SP specimens. However, unlike SP specimens, at the bottom of the beam, two 25mm- and two 29mm-diameter rebars were used. This is because in CP specimen the effective depth for the negative moment (tension in the bottom rebars) was greater than those of the SP specimens.

In all specimens, the shear capacity of the beam-column joints, and the development length of the longitudinal rebars in the beam-column joints were designed to satisfy the earthquake design requirements specified in ACI 318-05. The compressive strength of concrete for the precast concrete U-shell beam and the column were 35.1 and 47.5 MPa, respectively. The compressive strength of cast-in-place concrete for the beam-column connection and the beam core was 34.9MPa. The yield strengths and ultimate strengths of re-bars D13, D25, D29, D32, D35 were 503, 484, 514, 411.5, 493 MPa and 583, 626, 651, 599, 605 MPa, respectively.

Figure 2(g) shows the test setup. The column was pin-supported at its bottom, and the beam was roller-supported at its both ends. Cyclic lateral load was applied at the top of the column. The loading was displacement-controlled. The target displacement at each loading step was increased by 0.2% story drift at early loading. After 1% story drift, it was increased by 0.5% story drift, up to 5% story drift. Cyclic loading were repeated three times at each target displacement.

As shown in Figure 2(g), the displacements at each position of the specimens were measured by linear variable displacement transducers (LVDTs). The net displacement of the column was calculated by subtracting the lateral displacement at the column base and the contribution of the vertical displacements of the beam supports, from the total lateral displacement measured at the top of the column.

3. TEST RESULTS

Figures 3(a) to (f) show the lateral load-top displacement relationships of the test specimens. Table 2 shows maximum strength V_u, maximum displacement Δ_u, yielding displacement Δ_y, yielding stiffness k_y, and ductility μ of the specimens. The maximum displacement was defined as the post-peak displacement corresponding to 80% of the maximum strength. As shown in Figure 3, all SP specimens having the U-shell beam showed similar lateral load-top displacement relationships, irrespective of the variations in the test parameters. Yielding of the specimens occurred at 1% story drift, and the maximum displacement were 3.5~5.0 % story drift, which were greater than that of the monolithic CP specimen. All SP specimens satisfied the nominal strength predicted by ACI 318-05. However, the SP specimens showed poor energy dissipation capacity, compared to that of the CP specimen. This result indicates that shear deformation and rebar slip, which did not significantly contribute to the energy dissipation during cyclic loading, occurred in the SP specimens.

Figure 4 shows the crack development in SP1 specimen. At 0.2% story drift level, the first flexural cracks were observed in the beam (Figure 4(a)). At 0.5% story drift level, the first diagonal cracks were observed at the beam-column joint (Figure 4(b)). In all SP specimens, flexural yielding occurred at around 1.0% drift level. As shown in Figure 4(c), although the strength of the SP1 specimen reached its yield load, flexural cracks did not significantly propagate in the plastic hinge region of the beam. This result indicates that the yielding of the longitudinal re-bars in the beam propagates to the beam-column joint and bond-slip of the rebars occurs in the joint. This bond-slip behavior of the rebars in the beam-column joint was confirmed by the gap opening between the column face and the U-shell beam, which gradually increased with the lateral displacement from approximately 8mm at 1.0% drift (Figure 4(c)) to 15mm at 2.0% story drift (Figure 4(e)). At 2.5% story drift, the diagonal crack width increased to 3mm in the beam-column connection (Figure 4(f)). After this drift level, diagonal crack width increased in the beam-column connection, and concrete spalling became severe at the top and bottom of the connection. However, further flexural cracks did not develop in the plastic hinge region of the beam. This result indicates that the plastic deformation was concentrated more on the beam-column connection than at the plastic hinge region of beam (Figures 4(g), (h), and (i))). Despite such severe cracking and concrete spalling, the beam-column connection core enclosed by the closed stirrups remained intact (Figure 4(h)).

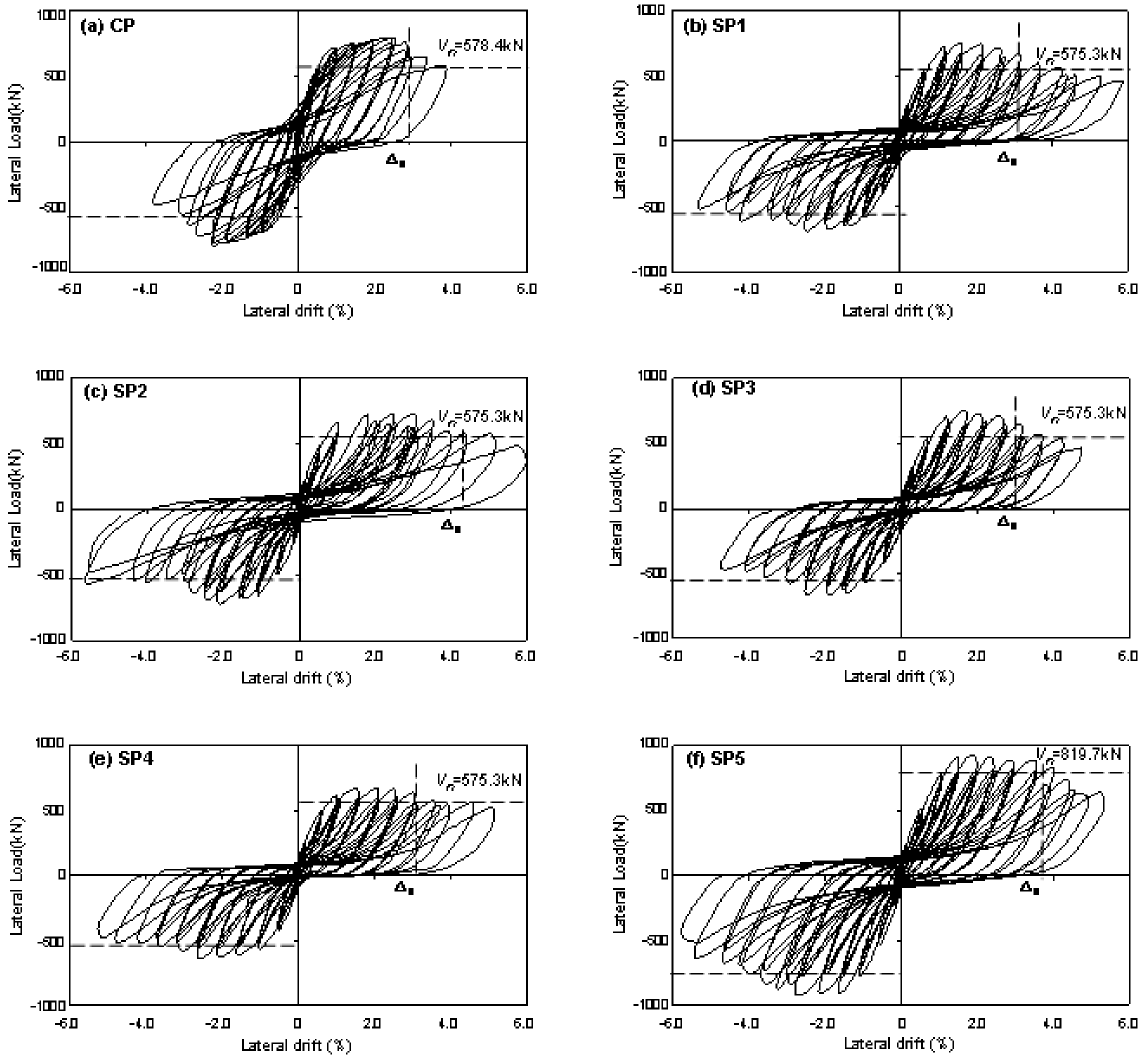


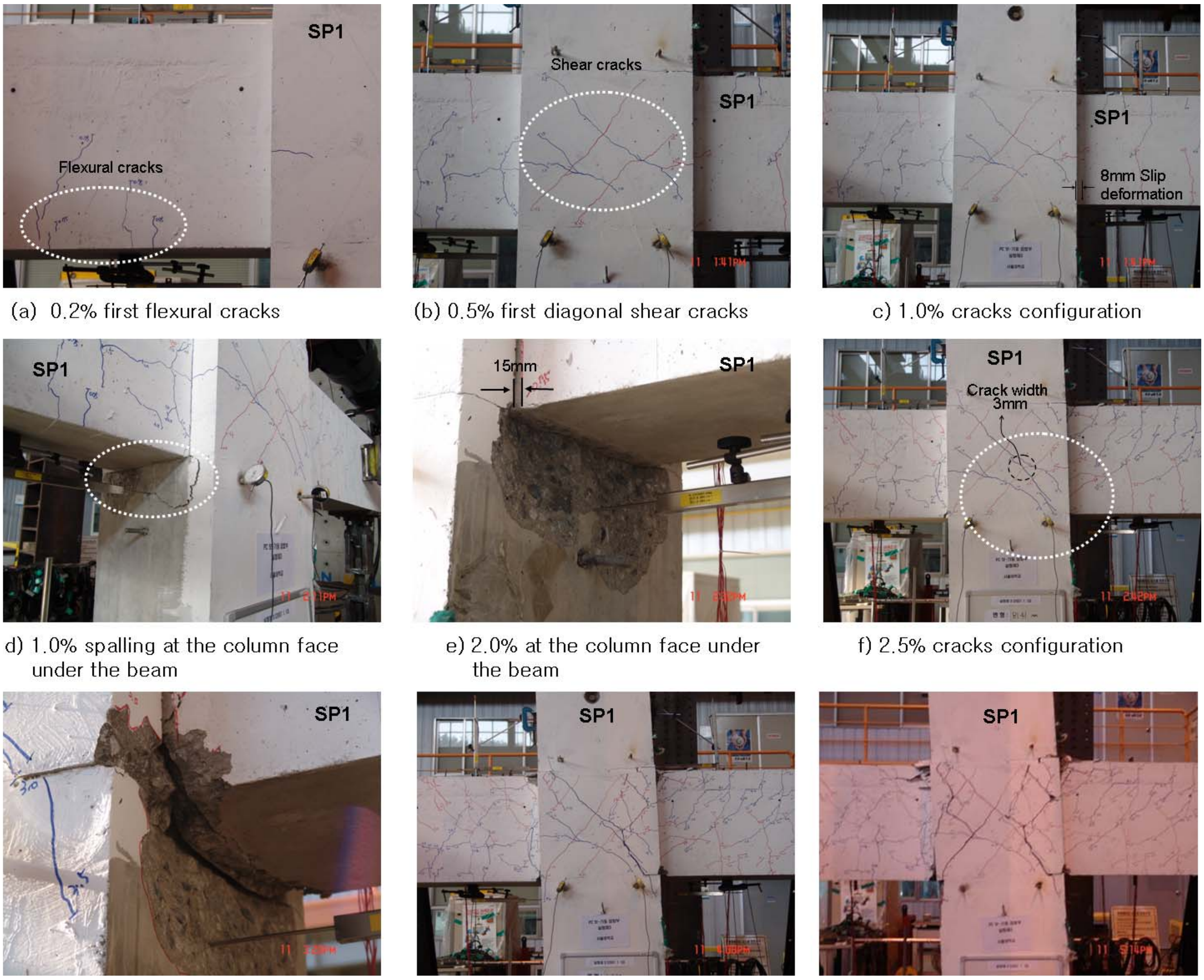
Figure 3. Load-top displacement relationships of test specimens

Table 2. Test results for test specimens

Specimens	Load Carrying Capacity			Deformation Capacity					
	V_u (kN)	Drift(%)	V_n (kN)	Δ_u (mm)	Drift (%)	Δ_y (mm)	Drift (%)	$\mu=\Delta_u/\Delta_y$	k_y (=V_u/Δ_y)
CP	794.6	2.53	578.4	78.84	2.92	13.39	0.50	5.89	38.84
SP1	735.6	2.17	575.3	98.28	3.64	19.42	0.72	5.06	28.76
SP2	722.9	2.51	575.3	136.9	5.07	18.63	0.69	7.35	29.10
SP3	752.1	1.68	575.3	96.39	3.57	20.77	0.77	4.64	27.13
SP4	667.8	2.09	575.3	96.12	3.56	16.74	0.62	5.74	29.92
SP5	926.8	1.90	819.7	118.3	4.38	23.75	0.88	4.98	29.26

As shown in Figure 3, even at large inelastic deformations, there was no significant strength degradation in the SP specimens.

Figures 4(d) and (e) show concrete damage of the column in the seated region of the U-shell beam at 1.0 and 2.0 % story drift levels. Concrete spalling of the column occurred in SP1 and SP4 having no steel angle strengthening. However, despite the concrete spalling in the column, the load-carrying capacity and stiffness of SP1 and SP4 specimens were comparable to those of other SP specimens strengthened with the steel angle (Figures 3(b) and (e)). In SP2, SP3 and SP5 strengthened with the steel angle, significant spalling of concrete did not occurred in the column. Figures 5(a) ~ (c) show the crack patterns at 1.5%, 2.5% story drift and the failure



(a) 0.2% first flexural cracks (b) 0.5% first diagonal shear cracks c) 1.0% cracks configuration

d) 1.0% spalling at the column face under the beam e) 2.0% at the column face under the beam f) 2.5% cracks configuration

g) 3.0% spalling at at the column face under the beam h) 3.5% crack configuration i) crack configuration at the end of testing

Figure 4. Development of Cracks in test

shape of the monolithic CP specimen, respectively. As shown in Figure 5(b), unlike the SP specimens, the diagonal shear cracks were minimized in the beam-column joint, and flexural cracks were significantly developed in the plastic hinge region of the beam. Finally, the CP specimen failed due to severe crack propagation and concrete crushing in the plastic hinge region of the beam.

Based on the test results, the effects of the test parameters on the structural performance of the SP specimens were investigated. 1) The steel angle for strengthening the contact region between the column and the U-shell beam did not significantly affect the structural performance of the SP specimens. Rather, the steel angle had adverse effect on the structural performance by decreasing the friction resistance of the contact surface and accelerating the slip-deformation of the re-bars in the beam. 2) Increasing the length of the U-shell beam seated on the column decreased the effective shear area of the beam-column joint (SP4), and therefore, the seated length should be reduced as much as possible. 3) The spacing of stirrups in the beam did not affect the structural performance of the specimen. This is because the majority of the plastic deformation occurred not in the plastic hinge region of the beam, but in the beam-column joint. 4) Increasing flexural capacity of the beam resulted in severe shear damage in the beam-column connection. However, the beam-column connection core enclosed by ties remained intact, and therefore, the load-carrying capacity was maintained at large inelastic deformations.

(a) 1.5% cracks configuration (b) 2.5% cracks configuration (c) crack configuration at the end of testing

Figure 5. Failure shapes of test specimens at the end of testing

4. CONCLUSION

An experimental study was performed to investigate the structural capacity of the beam-column connection as a part of a precast concrete moment resisting frame which had precast concrete U-shaped shell in beams. The U-shell beam and column were constructed as precast concrete members. The beam-column connection and the beam core were monolithically constructed by using cast-in-place concrete.

The specimens (SP) having the U-shell beam showed good load-carrying capacity and deformation capacity, which were comparable to those of conventional monolithic reinforced concrete specimen (CP). However, the energy dissipation capacity and stiffness of the SP specimens were significantly less than those of the CP specimen. Further, diagonal shear crack damage was significant in the beam-column connections. The causes of this poor performance of the SP specimens can be summarized as follows. 1) By seating the U-shell beam on the column, the effective shear area of the beam-column connection was significantly reduced. 2) For the SP specimens, larger rebars were used because of decrease in the effective depth for the negative moment due to the U shell construction. Therefore, the shear force applied to the beam-column joint increased. 3) Because of greater diameter rebars used in the SP specimens, the greater embedment length is required in the beam-column connection. For these reasons 1), 2) and 3), the diagonal shear cracks and the bond-slip of rebars occurred at the beam-column connection, which decreased the energy dissipation capacity.

Based on the results of investigation, the following methods for improving the structural performance of the beam-column connection are proposed.

1) To increase the effective depth for the beam moment, the bottom bars should be placed closer to the bottom of the cross-section. For this, the thickness of the U-shell should be decreased. When the flexural capacity of the connection needs to be increased, increasing the top bars could be better than increasing the bottom bars, which do not significantly contribute the energy dissipation capacity.

2) Using flexural rebars with smaller diameter is better because they require less embedment length in the beam-column connection, and bond-slip of rebars can be decreased.

3) The effective shear area of the beam-column connection should be increased. For this, the length of the U- shell beam seated on the column should be decreased as much as possible.

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