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Seismic Resistance of Cast-in-place Concrete-filled Hollow PC Columns with Various Tie Configurations

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ABSTRACT

Two types of cast-in-place concrete-filled hollow PC (HPC) columns were developed to reduce lifting load of heavy-weight PC columns and to improve the structural integrity of joints. Special cross-tie configurations were used for economical production and satisfying the reinforcement details specified in current design code provisions. To evaluate the seismic resistance, full scale specimens of two HPC columns and a conventional RC column were tested under combined axial compression and lateral cyclic loading. The test results showed that the seismic performance of the proposed HPC columns was comparable to that of the conventional RC column.

1. INTRODUCTION

Precast concrete (PC) columns have been widely used to reduce construction time, but for heavy-weight columns such as mega-sized and/or long columns, the use of the PC columns is limited due to an excessive lifting load (lifting weight and/or numbers). Hence, a conventional RC or segmental PC method is usually used for heavy-weight columns, in spite of its long construction time or high cost for segmental joints.

As an alternative for the RC and segmental PC methods, a half precast concrete (HPC) or cast-in-place concrete-filled hollow PC column method can be applicable to heavy-weight columns. In the HPC method, a reinforced hollow PC column is prefabricated and concrete core is poured in sites after installing the hollow PC column. Thus, compared to a conventional RC column, construction time is reduced because

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there are no reinforcement and formworks in sites, and compared to a segmental PC column, lifting load is significantly reduced because of a void section (or empty core). Additionally, the structural integrity of joints can be improved by the cast-in-place concrete core, and the lap splices of longitudinal rebars, which are more economical than splice sleeves, are possible through the void section.

The HPC column method has been studied in the name of outer-shell or semi PC columns (Kono et al. 1995, Iso et al. 1999, Hagiwara et al. 2001, and Xiao et al. 2012), and the key issue is the interference between internal molds and cross-ties when manufacturing the hollow PC column. In existing methods, the void section is formed by centrifugal or casting method, but cost increases or productivity decreases.

In the study, two types of HPC columns were developed to reduce lifting load of heavy-weight PC columns and to improve the structural integrity of joints. Special cross-tie configurations were used for economical production and satisfying the reinforcement details specified in current design code provisions. To evaluate the seismic resistance, full scale specimens of two HPC columns and a conventional RC column were tested under combined axial compression and lateral cyclic loading.

2. PROPOSED METHODS FOR HPC COLUMNS

Figs. 1 and 2 show the proposed methods for HPC columns. In HPC1 type, a couple of reinforced PC panels (the first and second PC faces) are prefabricated (Fig. 1(a)). After rotating the PC panels and placing longitudinal rebars for the third and fourth PC faces, concrete for the third PC face is poured between the PC panels (Fig. 1(b)). After a 180-degeree rotation, a hollow PC column is completed by pouring concrete for the fourth PC face (Fig. 1(c)). Although 2 times of rotation and 3 times of concrete casting are required, HPC1 is applicable to all types of rectangular sections, the thickness of PC parts can be minimized and lifting weight reduction is maximized around 70% (substitutable for a 3-segment PC column), and mold consumption is minimized. In HPC2 type, a reinforcing cage is prefabricated using saddle-shaped ties and dual hoops (Fig. 2(a)), and a hollow PC column is completed by pouring concrete between external and internal molds (Fig. 2(b), (c)). The concept of HPC2 is similar to that of existing walls and hollow bridge columns, and the saddle-shaped ties, which are anchored by inner hoops, and outer hoops prevent the buckling of longitudinal rebars and laterally confine concrete. Although a certain thickness of PC parts is required due to reinforcements and lateral pressure, HPC2 is applicable to any type of rectangular and circular sections, and the lifting weight reduction is about 50% (substitutable for a 2-segment PC column).

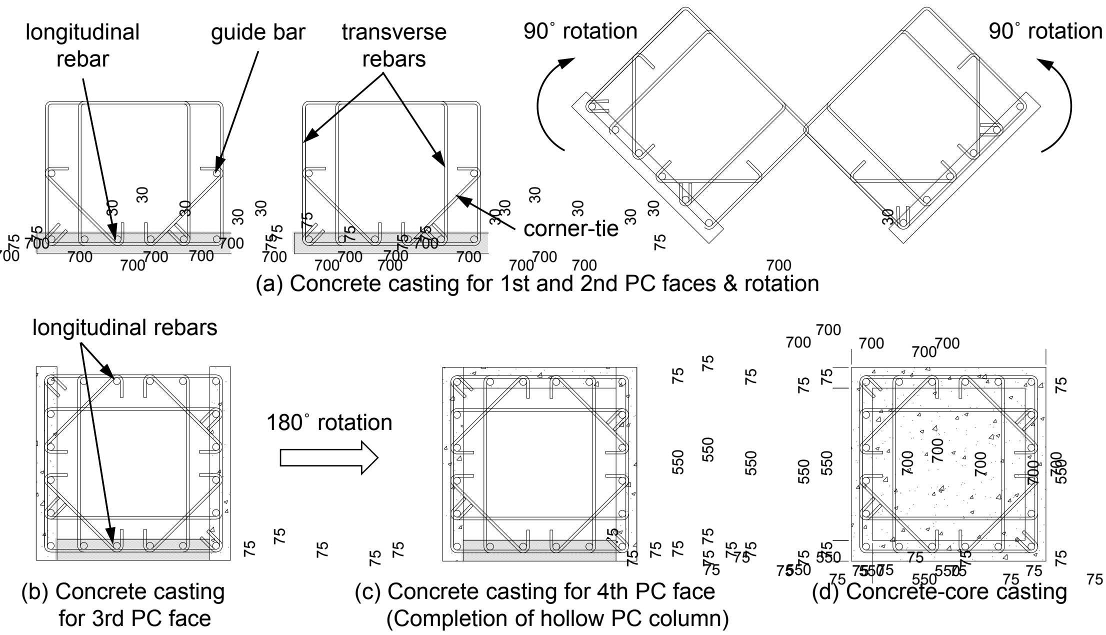


Fig. 1 HPC1 using prefabricated PC panels

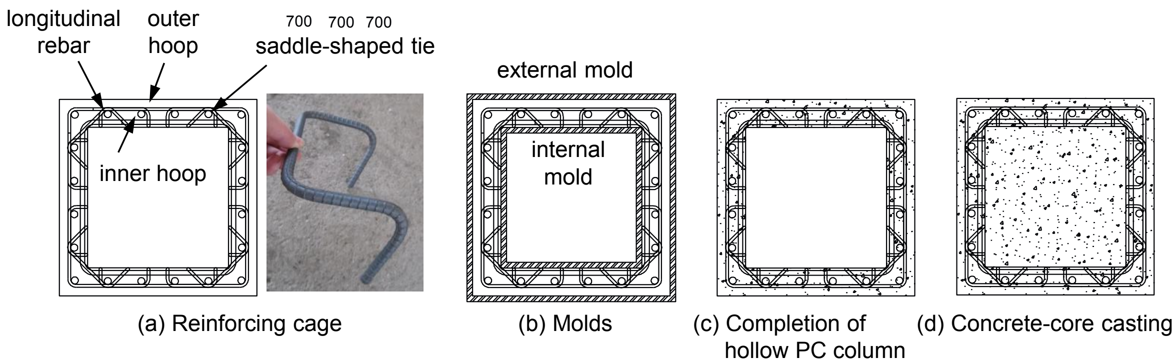


Fig. 2 HPC2 using saddle-shaped ties and dual hoops

3. TEST PROGRAM

3.1 Test Specimens

To evaluate the seismic resistance, full scale specimens of HPC1 and HPC2 columns were tested under combined axial compression and lateral cyclic loading. For comparison, a conventional RC column was also tested (Fig. 3).

Fig. 3 shows the dimensions and configuration of the test specimens. RC specimen has identical configuration and details with HPC1 specimen. The cross-section was 700×700mm, and column length was 4000mm. The design compressive strength of concrete was 40MPa, and 20-D25 (SD400) rebars were used for longitudinal reinforcements. For testing, foundations were installed at the bottom of the columns, and the longitudinal rebars of the columns were welded to 20mm-thick base plates. D10 (SD400) rebars were used for transverse reinforcements except the inner hoops (D16, SD400) of HPC2 specimen. The vertical spacing of the transverse rebars was one-half of the cross-sectional depth (D/2 = 350mm), and the spacing was decreased to 175mm at potential plastic hinge (bottom of the columns) and loading regions. The spacing conforms to the requirement for the special moment frame specified in ACI 318-08 (2008).The thickness of PC parts was 75mm for HPC1 and 100mm for HPC2, and hollow ratio was about 62% for HPC1 and 51% for HPC2, respectively.

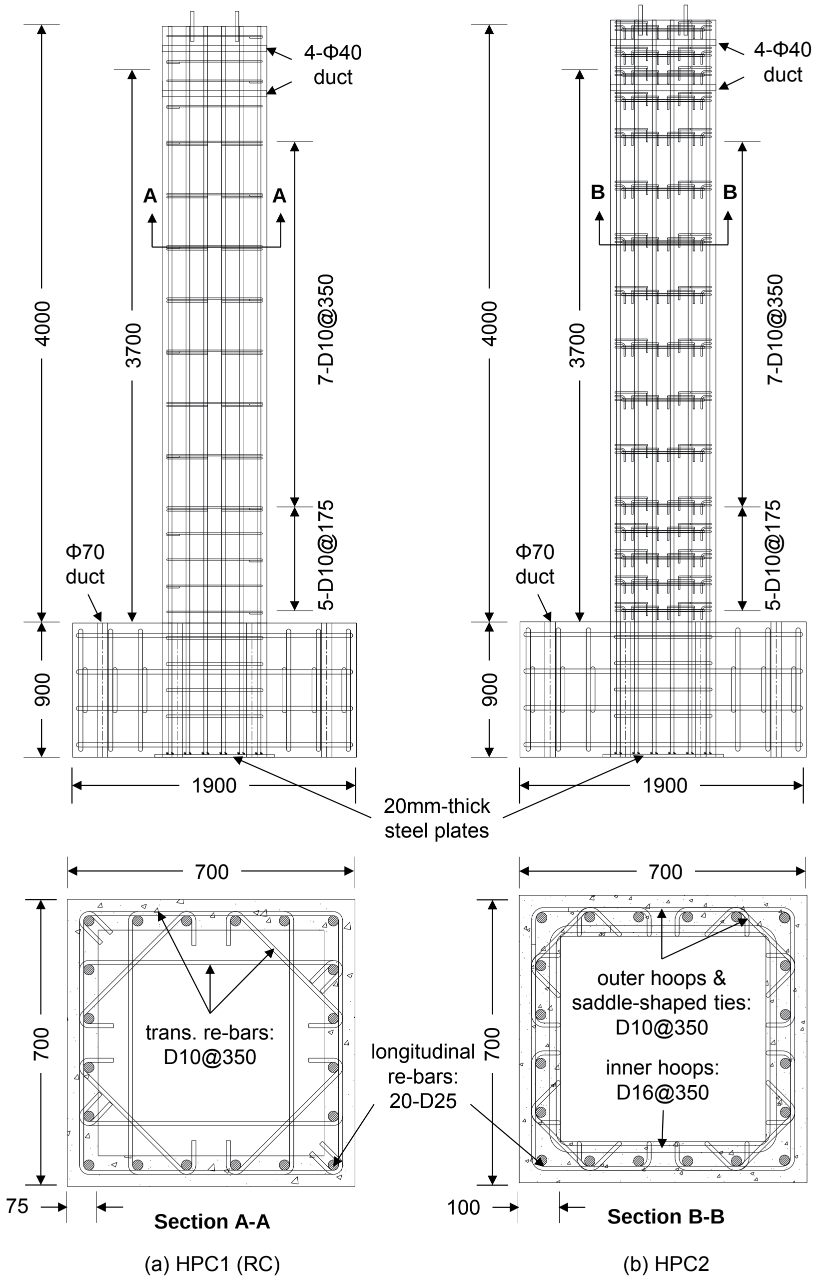


Fig. 3 Dimensions and configuration of test specimens (units: mm)

Table 1 Properties of test specimens

Speci -men	Cross -section, $B \times D$ (mm)	Column length (mm)	Hollow ratio (%)	Longi -tudinal rebars	Transverse rebars	$M_{max,p}$ [1)] (kN-m)	$V_{max,p}$ [2)] (kN)
RC	700×700	4000	0	20-D25 (ρ = 2.16%)	Ends: D10@175mm (ρ_h = 2.33%) Middle: D10@350mm (ρ_h = 1.16%)	1654.2	447.1
HPC1			62			1648.9	445.6
HPC2			51		Ends: D10@175mm (ρ_h = 2.18%) Middle: D10[3)]@350mm (ρ_h = 1.09%)	1604.3	433.6

[1)]Predicted maximum moment capacity: calculated from $P-M$ interaction (cross-sectional analysis)
[2)]Predicted maximum lateral load: $V_{max,p} = M_{max,p} / h'$, where h' = net column length (= 3700mm)
[3)]D16 rebars were used for inner hoops

3.2 Materials

According to ASTM C39 / C39M, compression tests for concrete were carried out, and Table 2 shows the 28-day cylinder compressive strength.

Table 2 Strength of concrete

Specimen		Strength, f'_c (MPa)
HPC1	1st and 2nd PC faces	45.3
	3rd PC face	43.3
	4th PC face	39.6
	Concrete core	40.1
HPC2	PC	37.7
	Concrete core	40.1
RC		40.1

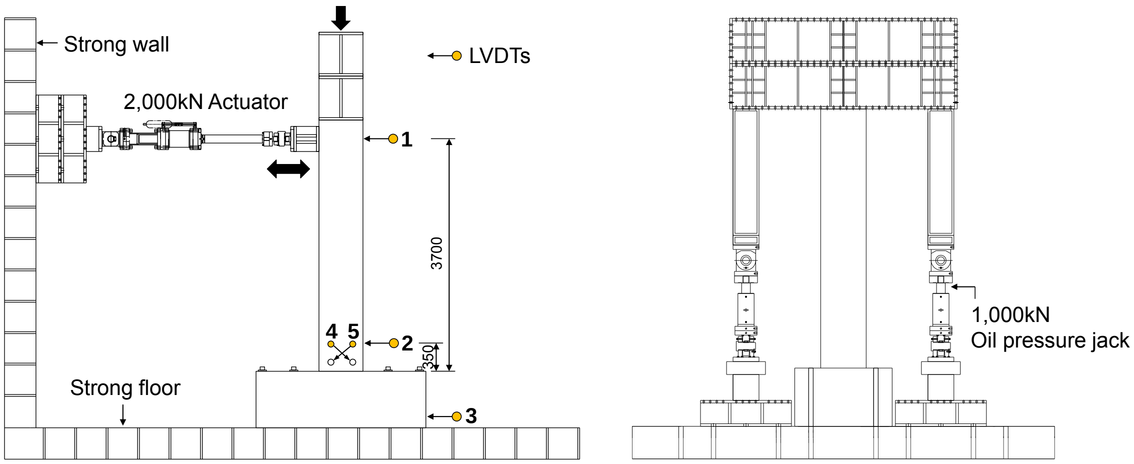
According to ASTM E8 / E8M-09, direct tension tests for rebars were performed, and Table 3 shows yield and tensile strengths.

Table 3 Strength of rebars

Type	Yield strength, f_y (MPa)	Tensile strength, f_u (MPa)
D10 for transverse rebars	434.0	685.4
D16 for inner hoops of HPC2	533.5	662.7
D25 for longitudinal rebars	459.7	610.3

3.3 Test Setup and Instrumentation

Fig. 4 shows the test setup and instrumentation for the test specimens. The constant axial load of 0.1 $f_c'A_g$ was applied by using two 1000kN oil pressure jacks. The lateral cyclic load was applied by a 2000kN actuator (max. stroke $\pm$250mm) under displacement control, using the cyclic loading history proposed in the ACI 374 (2005). To monitor displacements (No.1, No.2, No.3) and shear deformations (No.4, No.5), five linear variable displacement transducers (LVDTs) were used for each specimen. To measure compressive and tensile strains, strain-gauges were placed on rebar surfaces (12 for HPC1 and RC, 20 for HPC2).



(a) Side view (b) Front view

Fig. 4 Test setup and instrumentation

4. TEST RESULTS

4.1 Failure Modes

Fig. 5 shows the damage patterns and failure modes of the test specimens at the drift ratios of 2.86% and 4.83%. The drift ratio is the ratio of the net column length (3,700mm) to the lateral displacement at loading point. In RC specimen (Fig. 5(a)), concrete crushing began at 1.69%. After concrete cover spalling (2.20%), load-carrying capacity decreased, and the longitudinal rebars buckled at -3.71%. At the end of the test, plastic hinge length was about 350mm from the bottom of the column. In HPC1 specimen (Fig. 5(b)), concrete crushing began at -1.69%, and load-carrying capacity reached its peak at 2.20%. After buckling of the longitudinal rebars (3.71%), vertical cracks were observed between the PC faces, and the compressive PC face was delaminated in whole. After the delamination, crushing of concrete core followed, and fracture of the longitudinal rebars occurred at -4.83%. Plastic hinge length was about 400mm. In HPC2 specimen (Fig. 5(c)), concrete cover spalling began at 2.20%, and peak load occurred at 3.71%. Buckling of the longitudinal rebars (-3.71%) was followed by delamination of the compressive PC part, and rebar fracture was occurred at 4.83%. Plastic hinge length was about 450mm.

draft ratio = 2.86% draft ratio = 2.86% draft ratio = 2.86%

draft ratio = 4.83% draft ratio = 4.83% draft ratio = 4.83%

(a) RC (b) HPC1 (c) HPC2

Fig. 5 Damage patterns and failure modes of test specimens

4.2 Lateral Load – Displacement Relationship

Fig. 6(a), (b), and (c) show the lateral load – displacement relationships of the test specimens. In the figures, Δ_y, $V_{\max}$, and Δ_{ult} represent the yield displacement defined as the suggestion of Park (1988), maximum load, and ultimate displacement corresponding to 80% of $V_{\max}$ (El-Tawil and Deierlein 1999), and ①, ②, and ③ indicate the main events in the behavior of the test specimens such as concrete crushing, and buckling and fracture of the longitudinal rebars, respectively. For comparison, the predicted maximum load $V_{\max,p}$ (Table 1) is also shown in the same figures, and the envelope curves of the hysteresis curves are shown in Fig. 6(d).

Table 3 summarizes the test results. The maximum loads of RC specimen were $V_{\max}^{+} =$ 462.9kN at 2.20% drift ratio (positive loading) and $V_{\max}^{-} =$ -480.4kN at -2.86% (negative loading). In case of HPC1, $V_{\max}^{+} =$ 454.5kN at 2.20% and $V_{\max}^{-} =$ -488.3kN at -3.71%, and in case of HPC2, $V_{\max}^{+} =$ 438.5kN at 3.71% and $V_{\max}^{-} =$ -418.6kN at -2.86%. The ratio of the measured maximum load to the predicted maximum load was 104% for RC, 102% for HPC1, and 101% for HPC2, respectively. The load-carrying capacity of the test specimens was more than 75% of the maximum load at the third cycle of 3.5% drift ratio, which is specified in ACI 374. The displacement ductility $\mu (= \Delta_y / \Delta_{ult})$ was 4.89 (positive loading) and 4.32 (negative loading) for RC, 3.85 and 4.49 for HPC1, and 3.89 and 4.19 for HPC2, respectively. The initial stiffness defined at the yield displacement was 12.7kN/mm (positive loading) and 11.9kN/mm (negative loading) for RC, 12.6kN/mm and 12.6kN/mm for HPC1, and 12.7kN/mm and 12.7kN/mm for HPC2, respectively.

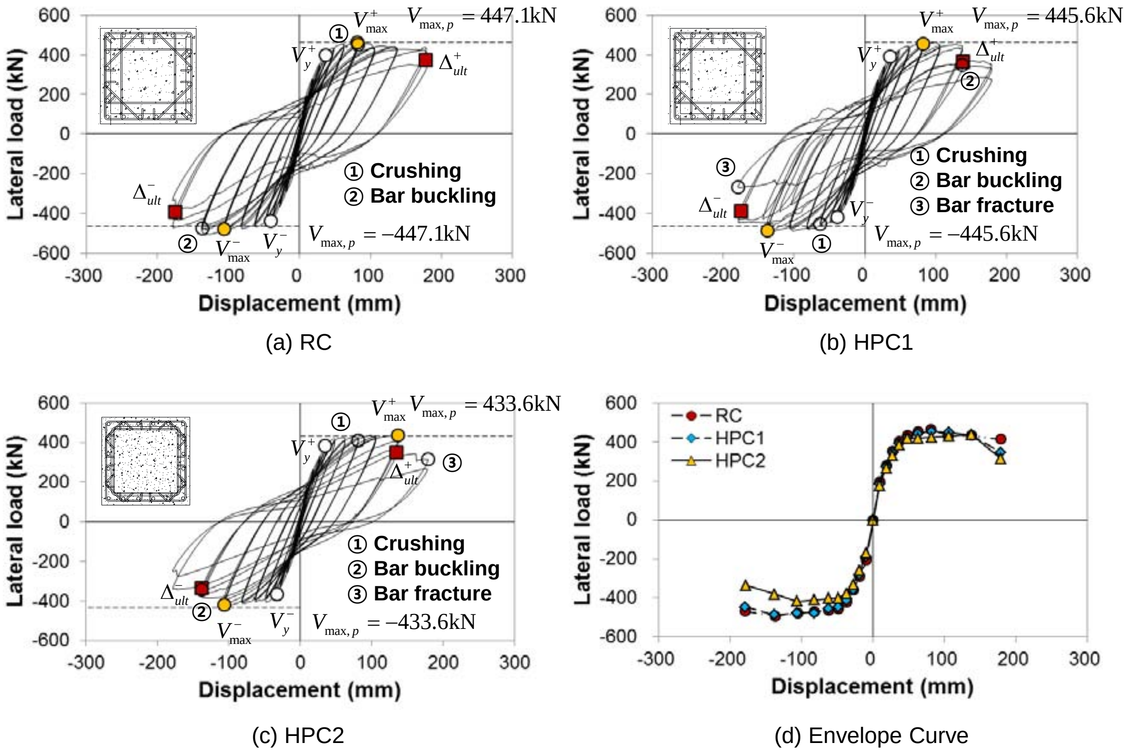


Fig. 6 Lateral load – displacement relationships

Table 3 Test results

Speci-men	Maximum load						Ultimate displacement					
	Positive (+)			Negative (-)			Positive (+)			Negative (-)		
	drift ratio (%)	V_{max} (kN)	Δ_{max} (mm)	drift ratio (%)	V_{max} (kN)	Δ_{max} (mm)	drift ratio (%)	V_{ult} (kN)	Δ_{ult} (mm)	drift ratio (%)	V_{ult} (kN)	Δ_{ult} (mm)
RC	2.20	462.9	81.4	-2.86	-480.4	-105.8	4.81	370.3	177.9	-4.73	-394.3	-175.1
HPC1	2.20	454.5	81.8	-3.71	-488.3	-137.3	3.74	363.6	138.5	-4.71	-390.6	-174.4
HPC2	3.71	438.5	137.2	-2.86	-418.6	-106.0	3.64	350.8	134.7	-3.72	-334.8	-137.7

Speci-men	Yield displacement								$\frac{V_{max}}{V_y}$		$\mu=\frac{\Delta_y}{\Delta_{ult}}$	
	Positive (+)				Negative (-)							
	drift ratio (%)	V_y (kN)	Δ_y (mm)	k_y (kN/mm)	drift ratio (%)	V_y (kN)	Δ_y (mm)	k_y (kN/mm)	(+)	(-)	(+)	(-)
RC	0.98	399.8	36.4	12.7	-1.09	-438.6	-40.5	11.9	1.16	1.10	4.89	4.32
HPC1	0.97	389.5	36.0	12.6	-1.05	-420.0	-38.8	12.6	1.17	1.16	3.85	4.49
HPC2	0.94	381.9	34.6	12.7	-0.89	-366.9	-32.9	12.7	1.15	1.14	3.89	4.19

As shown in Fig. 6 and Table 3, the behavior of HPC1 and HPC2 specimens were similar to that of RC specimen. Especially, the structural performance of HPC1 was considerably close to that of RC.

4.3 Energy Dissipation

Fig. 7 shows the cumulative energy dissipation and hysteretic energy dissipation ratio for drift ratios. The energy dissipation and energy dissipation ratio were defined as the enclosed area of the hysteresis curve and the ratio of the idealized elastic-plastic energy dissipation capacity E_{eq} to the energy dissipation E_D per cycle, respectively.

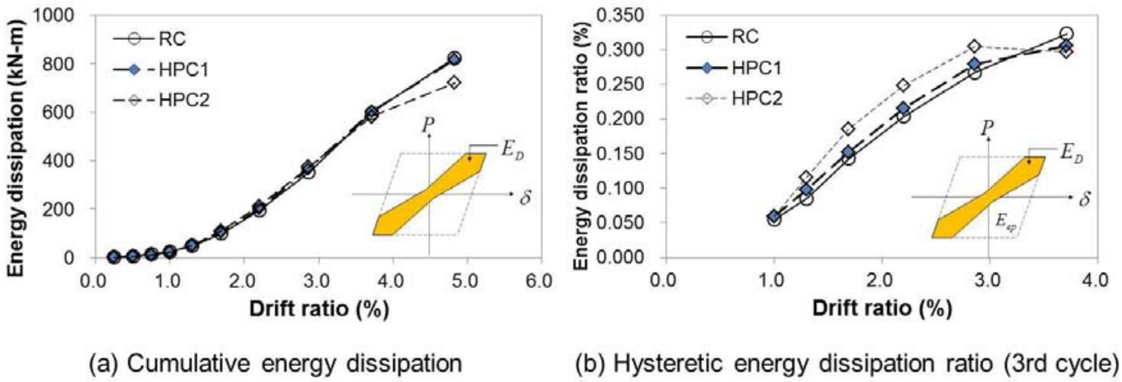


Fig. 7 Energy dissipations

As shown in Fig. 7(a), the energy dissipations of the test specimens were very similar to each other before rebar buckling (3.71% drift ratio), but the cumulative energy dissipation of HPC2 slightly decreased at 4.83%. As shown in Fig. 7(b), the energy dissipation ratio of the test specimens was more than 0.125 at the third cycle of 3.5% drift ratio, which is specified in ACI 374.

5. CONCLUSIONS

To reduce lifting load of heavy-weight PC columns and to improve the structural integrity of joints, two types of half precast concrete (HPC) or cast-in-place concrete-filled hollow PC column methods were developed. For economical production and satisfying the rebar details specified in current design code provisions, special cross-tie configurations were used. To evaluate the seismic resistance, full scale specimens of two HPC columns and a conventional RC column were tested under combined axial compression and lateral cyclic loading. The test results showed that the seismic performance of the proposed HPC columns was comparable to that of the conventional RC column.

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