SEISMIC BEHAVIOR OF R/C COLUMN MEMBERS USING PRECAST CONCRETE SHELL UNDER HIGH AXIAL LOAD

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SUMMARY

Previously, there were only a few data regarding the structural performance of the column members constructed using the precast shell-form method, especially when subjected to high axial loading. To investigate the matter mentioned above, eight column specimens, which were approximately one-third scale models of actual columns in a 30 story RC structure, were prepared. Five column specimens were constructed using the precast concrete shell and were designed for flexural, shear and bond splitting failure types. The other three column specimens were constructed using the conventional monolithic method and were compared with the PCa column specimens. The horizontal cyclic loading tests on these specimens were carried out under high axial load with ratios (N/N_u) of 0.3, or from -0.7 to 0.6, where N is axial load and N_u is ultimate axial strength of column considering the effect of main reinforcement. An investigation on the structural performance of PCa column members as compared to that of the conventional RC column members was performed. Moreover, the calculation methods used in the determination of the elastic rigidity and the ultimate strength of PCa column members under high axial load were also investigated.

INTRODUCTION

To facilitate construction of high-rise buildings, the precast shell-form method applied to a column member has been developed by the authors. In this method, the precast concrete shell is constructed with a high concrete strength of 70 N/mm^2, while the inner concrete is constructed using ordinary cast-in-place concrete with a strength from 27 to 48 N/mm^2. The precast concrete shell has lateral reinforcement (hoops and sub-hoops), and has shear cotters that are used to unite it with the inner concrete at each corner along the inner surface. After construction, the outer and the inner concrete of the column work together in resisting external forces. In addition, the precast concrete shell acts as the form for the inner concrete so the construction of formworks as well as the assembly of the lateral reinforcement in the field are not necessary. Thus, this precast shell-form method is considered more efficient than the conventional method. In this paper, the seismic behavior and performance of RC column member using this precast concrete shell are investigated by comparison with that of the conventional RC column member. Moreover, the calculation methods used in the determination of the elastic rigidity and the ultimate strength of PCa column members under high axial load are also investigated.

OUTLINE OF EXPERIMENT

Specimens

Specifications of column specimens are shown in Table 1. Dimensions and details are shown in Fig. 1. Eight column specimens, which were approximately one-third scale models of actual columns in the lower story of a 30 story RC structure, were prepared. Five column specimens (PC-1A, PC-2, PC-3, PC-1B, PC-4) were constructed using the precast concrete shell (hereafter referred to as PCa column specimens). The other three column

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specimens (RC-1A, RC-2, RC-3) were constructed using the conventional monolithic method (hereafter referred to as RC column specimens) and these specimens were compared with the PCa column specimens. PC-1A and RC-1A, PC-2 and RC-2 were designed for flexural failure type, PC-3 and RC-3 were designed for shear failure type, PC-1B was designed for flexural-shear failure type, PC-4 was designed for bond splitting failure type.

Every specimen except PC-2 and RC-2 had a 32 cm $\times$ 32 cm square cross section, and PC-2 and RC-2 specimens had a 28 cm $\times$ 28 cm square cross section. The shear span of PC-1A, RC-1A, PC-2 and RC-2 was 1.6, and that of PC-3, RC-3, PC-1B and PC-4 was 1.14. The ratio of main reinforcement (hereafter referred to as main bar) of PC-1A, RC-1A, PC-1B was 2.33%, and that of the other specimens was 3.88%. The ratio of lateral reinforcement (total of hoop and sub-hoop) was 1.13%, and only that of PC-4 was 0.81%. The precast concrete shell held hoops and sub-hoops, individual hoop and sub-hoop had a welding connection in the middle of the longer span. The precast concrete shell also held shear cotters, which were used to unite it with the inner concrete, in the corners along the inner surface. The width and length of the shear cotters were 35 mm $\times$ 57 mm, and the depth was 3 mm. The shear cotter spacing along the axial direction of column was 60 mm.

Table 1 Specification of column specimen

Specimen	PC-1A	RC-1A	PC-2	RC-2	PC-3	RC-3	PC-1B	PC-4
Classification	PCa	RC	PCa	RC	PCa	RC	PCa	PCa
Cross section (B×D)	32cm×32cm		28cm×28cm		32cm×32cm			
Height (H)	102cm		90cm		73cm			
H/D	3.19		3.21		2.28			
Main bar	12-D16		24-D13		20-D16		12-D16	20-D16
Ratio of main bar (p_g)	2.33%		3.88%		3.88%		2.33%	3.88%
Hoop	4-D6 @35		4-D6 @40		4-D6 @35		4-D6 @35	4-UHD6 @49
Ratio of hoop (p_w)	1.13%							0.81%
Ratio of axial stress (η)	0.3 [Constant load]		-0.7 ~ 0.6 [Fluctuating load]		0.3 [Constant load]			0.1
Pretest hypothetical failure type	Flexural failure				Shear failure		Flexural - shear failure	Bond splitting failure

Where η: [Compression] $\eta=N/\{F_c\cdot(B\cdot D-A_s)+1.12\sigma_{sy}\cdot A_s\}$, [Tension] $\eta=N/(1.12\sigma_{sy}\cdot A_s)$

N: Axial load, F_c: Concrete compressive strength, B: Width of column, D: Depth of column,

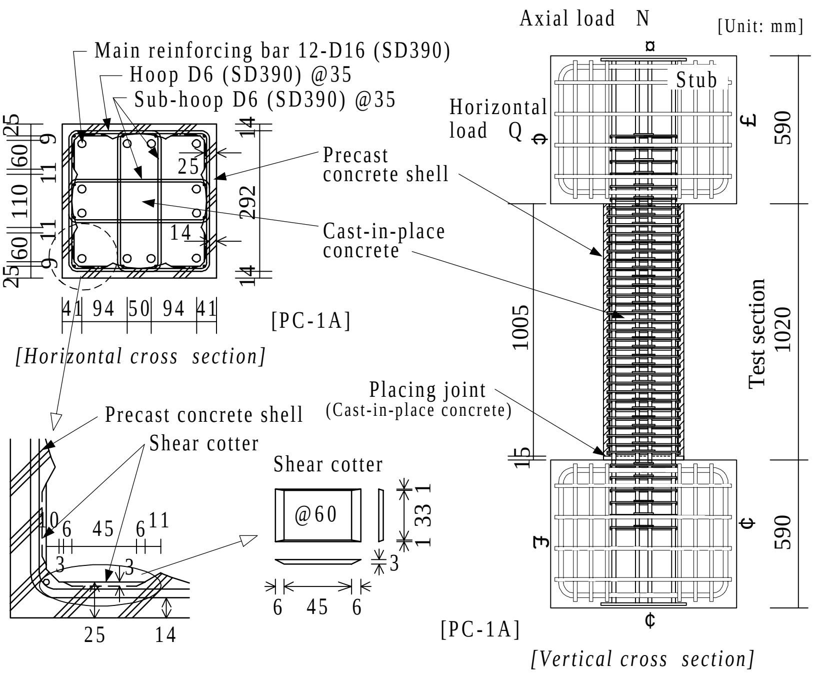


Fig. 1 Example of PCa column specimen

Material Properties

Compressive cylinder strengths of cast-in-place concrete for the column specimens were from 53.3 to 54.9 N/mm^2, and those for precast concrete were from 64.7 to 75.1 N/mm^2. Moduli of elasticity of cast-in-place concrete were from 32.3 to 33.4 kN/mm^2, and those of precast concrete were from 35.1 to 37.9 kN/mm^2. This concrete was normal weight concrete of which maximum aggregate size was 13 mm. Yield strengths of D13 and D16 using main bars were 451 and 469 N/mm^2 (SD390 grade). Yield strengths of D6 and UHD6 using lateral reinforcement were 449 and 724 N/mm^2 (SD390 and SD685 grade).

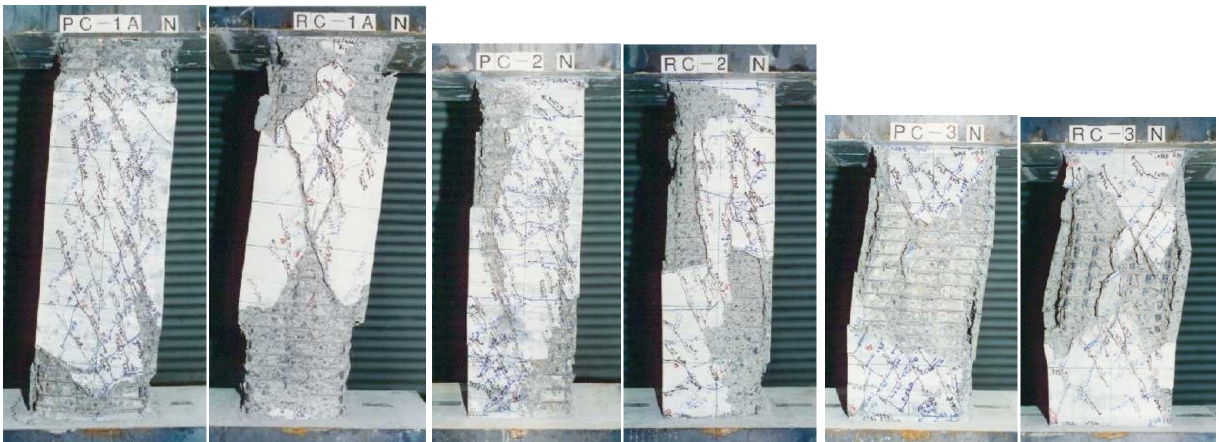
Loading procedure

Reversed cyclic horizontal loading, that produced an anti-symmetric bending moment of equal magnitude at both column ends, was applied to each specimen. Horizontal loading was controlled by drift angle and the amplitude was increased gradually. Drift angle was defined as the ratio of the relative displacement between the top stub and bottom stub to the column clear height. PC-1A and RC-1A specimens were subjected to two cycles of horizontal loading at a drift angle R equal to $\pm(2.5, 5, 10, 15, 20, 30, 40, 50, 60)\times10^{-3}$ rad. Other specimens were subjected to two cycles of loading at a drift angle R equal to $\pm(2.5, 5, 10, 15, 20, 30, 40)\times10^{-3}$ rad. After this, horizontal loading was applied to specimen in positive direction until column specimen strength decreased sufficiently. On PC-1A, RC-1A, PC-3, RC-3 and PC-1B, axial load was held constant and the ratio of axial stress (η: refer to the footnote of Table 1) was 0.3 which is similar to the axial stress of an inside column of a lower story of a high-rise building during an earthquake. On PC-2 and RC-2, the axial stress was fluctuated and the ratio of axial stress was from -0.7 to 0.6 which is similar to the axial stress of an outside column of a lower story of a high-rise building during an earthquake. On PC-4, axial load was held constant and the ratio of axial stress was 0.1, which was decided by preliminary analysis to achieve bond splitting failure. Horizontal loading and axial loading were applied by hydraulic jacks.

TEST RESULTS

Cracking pattern and failure mode

Examples of final cracking patterns and failure modes of column specimens, which were manufactured as pairs of PCa and RC columns, are shown in Fig. 2. PC-1A and RC-1A, PC-2 and RC-2 failed in flexure, PC-3 and RC-3 failed in shear, and PC-1B failed in shear after yielding of main bars. Although PC-4 was designed for bond splitting failure type in accordance with Structural Design Guideline for New RC Structure (1992) and Design Guidelines for Earthquake Resistant Reinforced Concrete Building based on Ultimate Strength Concept of AIJ (1990), PC-4 failed in shear after yielding of main bars. During the experiment, decrease of the bond strength was observed by measurement of the strain of main bars and few bond splitting cracks occurred on the surface of PCa column specimens. Cracking patterns were similar between PCa and RC column specimens that were manufactured as a pair. Although chipping off of covering concrete of RC column specimens increased in the large drift angle stage due to the fact that covering concrete of RC columns was thicker than that of PCa column specimens, no substantial difference in core concrete damage was observed between PCa and RC column specimens. A slippage failure in the horizontal joint at the bottom between precast concrete and cast-in-



PC-1A	RC-1A	PC-2	RC-2	PC-3	RC-3

Fig. 2 Final crack patterns

place concrete was not observed in the PCa column specimen until the large drift angle stage. In fact, the placing joint performance was perfect until the large drift angle stage. It was confirmed that under high axial load (η=0.3 and from -0.7 to 0.6), the failure modes of PCa column members were similar to those of RC column members.

Relationship between shear force and story drift angle

Shear force - story drift angle curves (Q-R curves) are shown in Fig. 3. Q-R curves in the pairs of PCa and RC columns were compared and though a difference of the maximum strength of each column specimen was observed, the form of Q-R curves were similar to each other. On PC-1A and RC-1A that were of flexural failure type, a substantial difference of drift angle at the yielding of main bars and the maximum strength was not

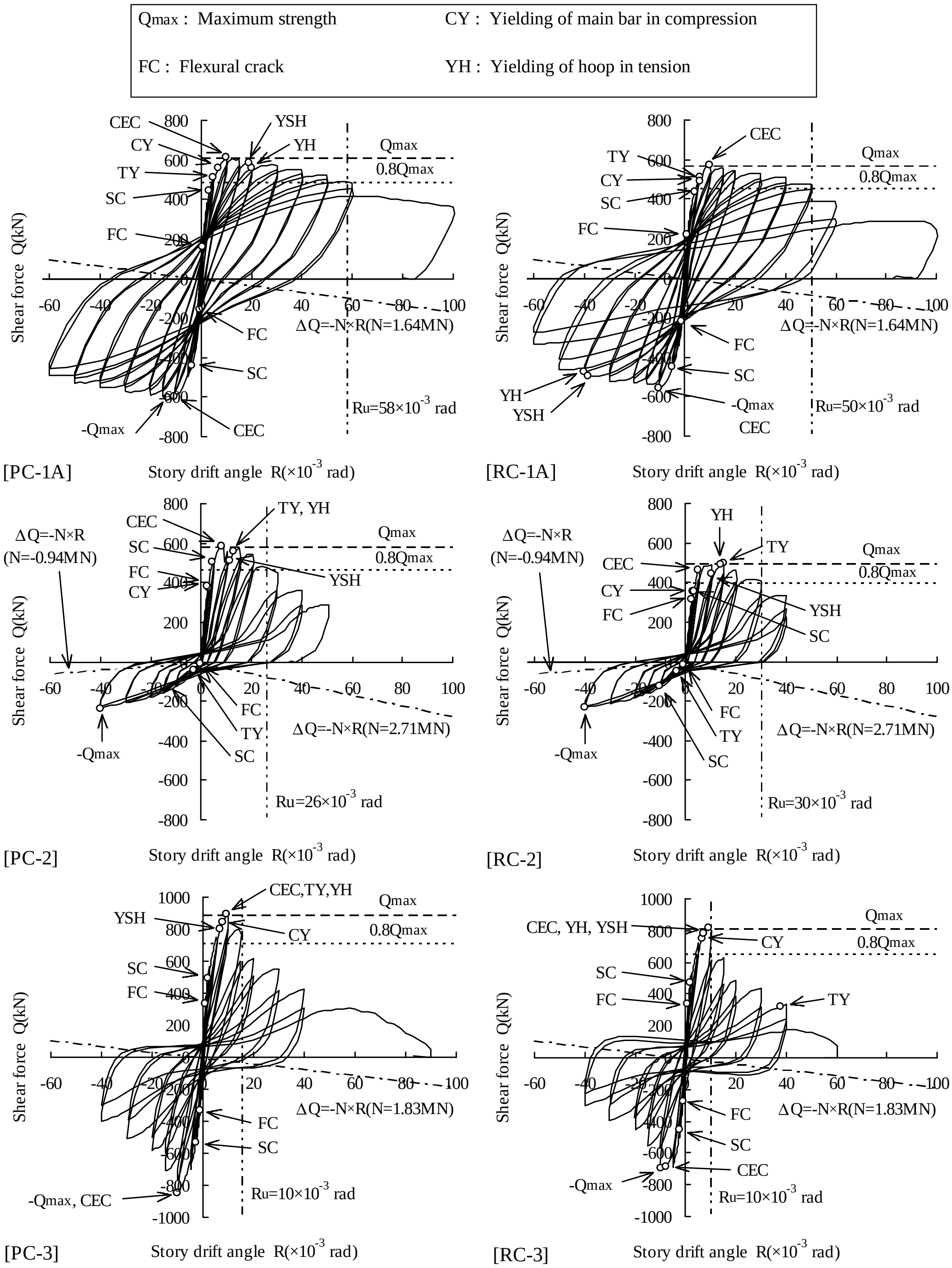


Fig. 3 Relationship between shear force and story drift angle

observed. And on PC-3 and RC-3 that were of shear failure type, a substantial difference of drift angle at the yielding of lateral reinforcement and the maximum strength was not observed. On PC-2 and RC-2 applied fluctuating load, no difference of the drift angle of yielding of main bars was observed in compression, but a difference of the drift angle of the maximum strength was observed. On the other hand, in tension the drift angle of yielding of main bars and the maximum strength of PCa column specimens corresponded with those of RC column specimens. In the case of ultimate drift angle defined as the drift angle at 80% of maximum strength, ultimate drift angle of PCa column member was equivalent or superior to that of the RC column member.

Average axial strain of column and rotation angle of column ends

Comparison of the average axial strain of column between PCa and RC column specimens that were prepared as a pair is shown in Fig. 4, and comparison of rotation angle of column ends is shown in Fig. 5. The average axial strain of the column was calculated by the measured axial displacement divided by initial column height. The rotation angles of the column ends were measured at intervals of 15 cm (approximately half the depth) from the stub. On PC-1A and RC-1A, PC-2 and RC-2, PC-3 and RC-3, in every case, an increase in concrete crushing of RC column specimen over PCa column specimens was noted from $R=15\times10^{-3}$ rad stage on, which caused the PCa column specimens to have a lower axial strain. The reason was that the rigidity and strength of precast concrete were superior to that of cast-in-place concrete. The rotation angles of the column ends of PC-1A and RC-1A, PC-2 and RC-2, which failed in flexure, were compared individually in each pair. Substantial difference on rotation angles hysteresis was not observed, though a slight difference was noted with regards to an increase of crushing of concrete from $R=40\times10^{-3}$ rad on. Under high axial load, it was confirmed that the performance of average axial strain of column and rotation angle of column ends of PCa column members were similar to that of RC column members.

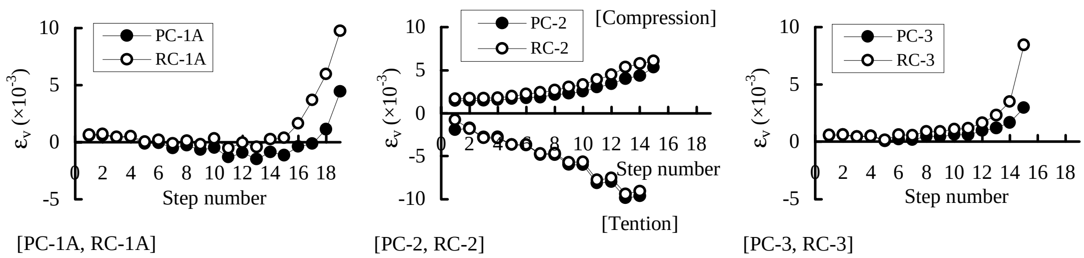


Fig. 4 Average axial strain of column

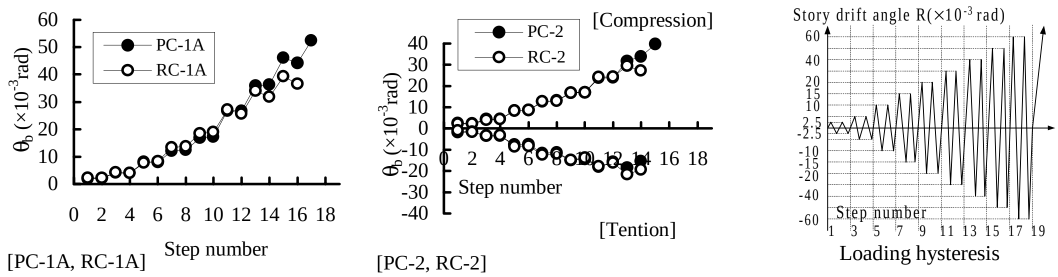


Fig. 5 Rotation angle of column ends

Bond strength of main bars

The distribution of bond stress of main bars of PC-4, which was designed for bond splitting failure type, is shown in Fig. 6. The bond stress was calculated using the measured strains of the main bars. The bond stress and strength that were calculated by Equation of Design Guideline for New RC (1992), and Equation of Design Guideline of AIJ (1990), are also shown in Fig. 6. The calculated maximum bond stress (τ_j) was 7.63 N/mm^2, and the maximum bond strength (τ_u) was 5.74 N/mm^2. In PC-4, at the drift angle of $R=30\times10^{-3}$ rad stage after the maximum strength, the experimental bond stress of main bars that were directly bound with lateral reinforcement was 7.22 N/mm^2, and the bond stress of main bars that were indirectly bound was 4.90 N/mm^2. At this time, in the relationship between bond stress and shear force of column, a decrease of bond stress was observed in spite of an increased shear force, and the maximum bond strength was recorded. The maximum bond strength of the main bars that were directly bounded with lateral reinforcement was 7.26 N/mm^2, and that indirectly bounded with lateral reinforcement was 5.88 N/mm^2. The calculated value of bond strength by Equation of Design Guideline for New RC was 5.74 N/mm^2. The experimental bond strength was greater than the calculated value. The experimental result of PC-3 was almost the same. It was confirmed that the bond strength of main bars of PCa column member could be conservatively estimated by Equation of Design Guideline for New RC.

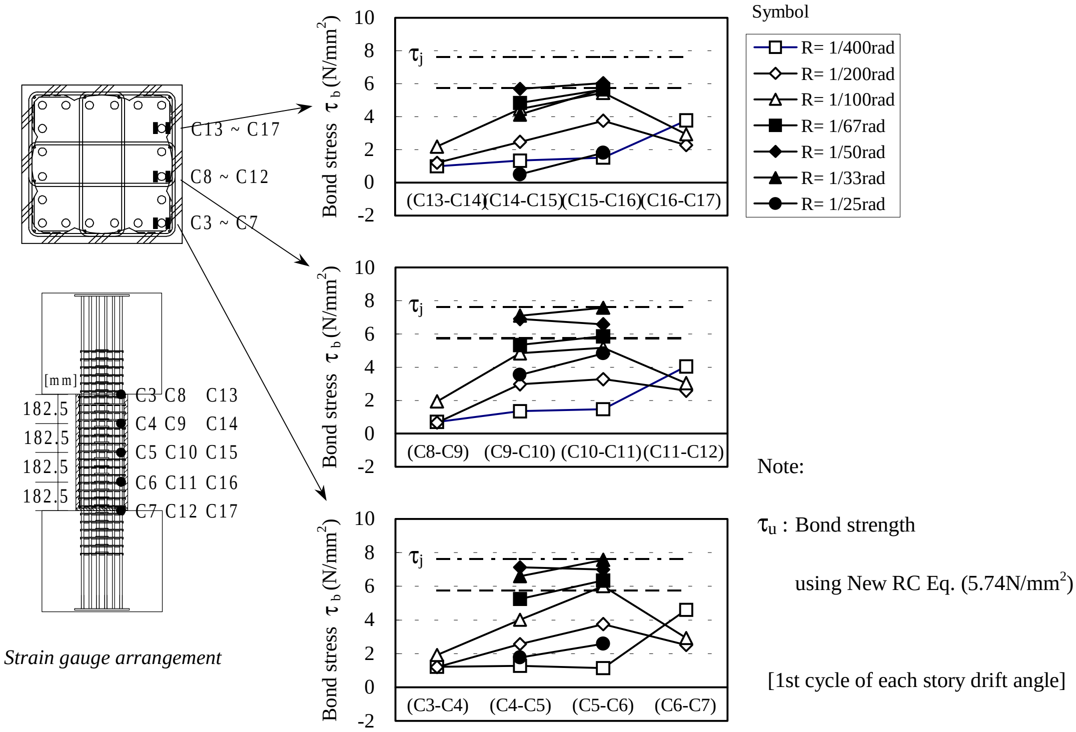


Fig. 6 Distribution of bond stress of main bars (PC-4)

Experimental values and relationship between experimental and calculated value of maximum strength

The experimental and calculated value of elastic rigidity and maximum strength are shown in Table 2 and Fig. 7. Here, elastic rigidity was calculated by Equation (1) in Table 3. Flexural strength was calculated by fiber-element model analysis, which considered the precast concrete strength and cast-in-place concrete strength individually. On the other hand, the shear strengths were calculated by the following two methods. [1st] The shear strengths were calculated by modified Kuramoto's Equation (2) in Table 3. Kuramoto's Equation (1990) could consider the effect of axial stress to shear strength. The angle of inclination of concrete struts in truss mechanism was assumed constant regardless of the concrete strength grade. At this time, precast concrete strength and cast-in-place concrete strength were considered individually in tensile main bar coefficient (ϕ) and lateral reinforcement coefficient (φ). [2nd] To compare the above-mentioned calculated values of shear strength, according to usual means of structural design, the shear strengths were calculated by Kuramoto’s Equation, Equation of Design Guideline for New RC (1992), Equation in Method A of AIJ Design Guideline (1990) and modified Arakawa’s Equation (1991) using the equivalent concrete strength of precast concrete and cast-in-place concrete (refer to the footnote of Table 2). In Table 2, the values of previous experiment specimens [Nakae and Hosoya et al., 1995] were also estimated. The experimental values of elastic rigidity, maximum strength and ultimate drift angle of PC-1A and RC-1A, PC-2 and RC-2, PC-3 and RC-3, which were manufactured as a pair, were compared individually. In every case, the value of the PCa column specimen was superior to or almost the same that of the RC column specimen. If the performance to unite precast concrete and cast-in-place concrete is as in this experimental case, it is expected that elastic rigidity, maximum strength and ultimate drift angle of PCa column members are equivalent to those of RC column members under high axial load. Elastic rigidities were calculated by the Equation (1) shown in Table 3. At this time, the values were calculated by considering the characteristics of precast concrete and cast-in-place concrete individually, and calculated values were compared to experimental values. The ratio of experimental values to calculated values was 0.98, and standard deviation was 0.05. The ratio of experimental values to calculated values of the flexural strength using fiber-element model analysis was 1.14, and standard deviation was 0.07. On the other hand, the ratio of the experimental values to the calculated values of shear strength using modified Kuramoto’s Equation (2) was 1.0, and standard deviation was 0.04. The calculated elastic rigidity, flexural strength, shear strength were similar to experimental values. Considering the specification of precast concrete and cast-in-place concrete individually, the elastic rigidity and ultimate strength can be accurately estimated by the above-mentioned methods.

Table 2 Comparison between experimental and calculated values

Specimen	Failure type	Experimental value			Calculated value						
		Elastic rigidity	Maximum strength	Ultimate drift angle **	Elastic rigidity (A)	Flexural strength (B)	Shear strength (B)	Shear strength (C) Equivalent concrete strength			
						Fiber model	Kuramoto	Kuramoto	New RC	AIJ	Arakawa
		$_{exp}K_e$ (MN/rad)	$_{exp}Q_{max}$ (kN)	$_{exp}R_u$ (rad)	$_{cal}K_e$ (MN/rad)	$_fQ_{fu}$ (kN)	$_{sk}Q_{su}$ (kN)	$_kQ_{su}$ (kN)	$_nQ_{su}$ (kN)	$_aQ_{su}$ (kN)	$_{am}Q_{su}$ (kN)
PC-1A	B	221	609	0.058	208	558	724	745	671	736	498
RC-1A	B	201	573	0.050	200	534	727	727	650	728	488
PC-2	B	147	584	0.026	165	471	570	592	572	570	519
RC-2	B	147	498	0.030	158	431	565	565	565	562	507
PC-3	S	321	892	0.015	311	978	858	908	746	737	627
RC-3	S	297	815	0.010	299	916	849	849	709	723	602
PC-1B	BS	265	807	0.015	292	761	837	862	737	732	572
PC-4	BS	311	776	0.030	317	792	796	831	810	773	542
PCS-4*	BS	275	688	0.015	284	724	705	697	587	574	508
PCS-5*	BS	288	746	0.020	278	724	706	697	619	574	508
Av.*** (Experimental value/Calculated value)					0.98	1.14	1.00	0.97	1.12	1.15	1.40
S. D.***					0.05	0.07	0.04	0.06	0.09	0.10	0.06

Where

B: Flexural failure type

BS: Shear failure type

$_fQ_{fu}$: Flexural strength using fiber-element model analysis, $_{sk}Q_{su}$: Shear strength using modified Kuramoto's Eq.(2), $_kQ_{su}$: Shear strength using Kuramoto's Eq. (1990), $_nQ_{su}$: Shear strength using New RC Eq. (1992), $_aQ_{su}$: Shear strength using Eq. in Method A of AIJ (1990), $_{am}Q_{su}$: Shear strength using modified Arakawa's Eq. (1991)

(A) Characteristics of reinforcement, precast concrete and cast-in-place concrete are considered separately

Table 3 Elastic rigidity ($_{cal}K_e$) Eq. and modified Kuramoto's shear strength ($_{sk}Q_{su}$) Eq.

1. Elastic rigidity

$$_{cal}K_e = 1/(1/K_f + 1/K_s) \qquad (1)$$

Where $K_f = 12 \cdot E_c \cdot I_e / H'^3$, $K_s = (G_{ic} \cdot A_{ic} + G_{oc} \cdot A_{oc})/(\kappa \cdot H')$

I_e: Equivalent geometrical moment of inertia considering cast-in-place concrete, precast concrete and main bars of column, G_{ic}, A_{ic}: Modulus of rigidity and area of cast-in-place concrete, G_{oc}, A_{oc}: Modulus of rigidity and area of precast concrete, κ=1.2, H'=H+2D/4, H: Height of column, D: Depth of column

2. Modified Kuramoto's shear strength Equation

$$_{sk}Q_{su} = b \cdot j_t \cdot p_w \cdot \sigma_{wy} + (\gamma - 2\alpha \cdot \varphi) b \cdot D \cdot \sigma_{iB} \qquad (2)$$

$$_{sk}Q_{su} \leq (b \cdot j_t \cdot \sigma_{iB}/2)\gamma/\alpha$$

where

$n \leq 0.5 - 2\phi$

$\gamma = \{\sqrt{4(n+2\phi)(1-n-2\phi)+\eta^2} - \eta\}/2$

$n > 0.5 - 2\phi$

$\gamma = \alpha = (\sqrt{1+\eta^2} - \eta)/2$

$\varphi = \varphi_i + \varphi_o$

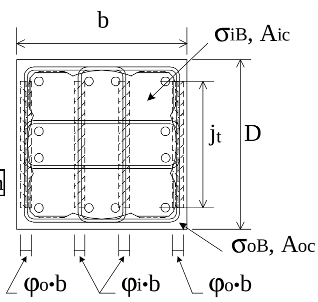
$\varphi_i = (2-k) \cdot (p_w/2) \cdot \sigma_{wy}/\sigma_{iB}$

$\varphi_o = \{k \cdot (p_w/2) \cdot \sigma_{wy}\}/\{(\sigma_{iB} + \sigma_{oB})/2\}$

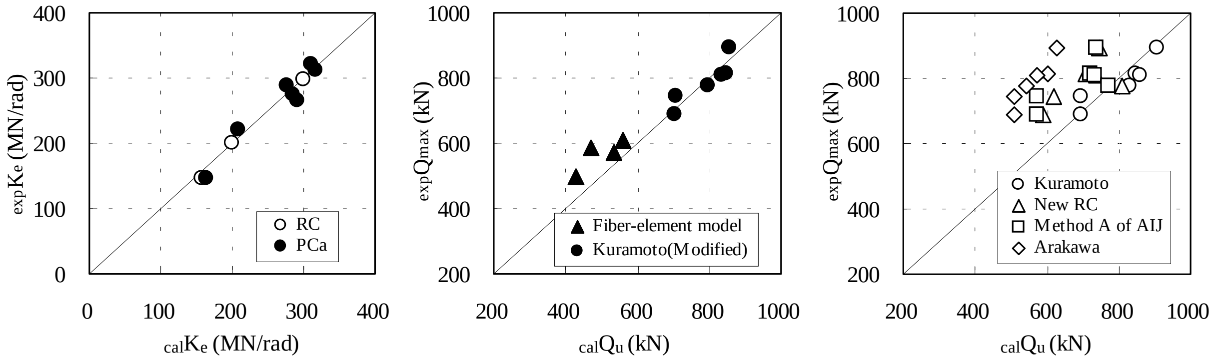
with sub-hoop: k=1
without sub-hoop: k=2

$\phi = p_t \cdot \sigma_{sy}/\{(A_{ic} \cdot \sigma_{iB} + A_{oc} \cdot \sigma_{oB})/(A_{ic} + A_{oc})\}$

Horizontal cross section



b: Width of column, j_t: Distance between the compressive and tensile main bars, p_w: Ratio of lateral reinforcement, σ_{wy}: Yield strength of lateral reinforcement, ϕ: Tensile main bar coefficient, φ: lateral reinforcement coefficient, $n = N/(A_{ic} \cdot \sigma_{ic} + A_{oc} \cdot \sigma_{oc})$, η=H/D, σ_{iB}, A_{ic}: Compressive cylinder strength and area of cast-in-place concrete, σ_{oB}, A_{oc}: Compressive cylinder strength and area of precast concrete, p_t: Ratio of tensile main bars, σ_{sy}: Yield strength of main bars, N: Axial load



Elastic rigidity Flexural strength and shear strength Shear strength

Fig. 7 Comparison between experimental and calculated values of elastic rigidity and maximum strength

The shear strengths were calculated using the equivalent concrete strength. The ratio of experimental shear strength to calculated shear strength using Kuramoto's Equation (1990), Equation of Design Guideline for New RC (1992), Equation in Method A of AIJ Design Guideline (1990), modified Arakawa's Equation (1991), was 0.97, 1.12, 1.15, 1.40, respectively. The calculated ultimate values were estimated conservatively except for Kuramoto's Equation, and these variation ratios of calculated values were higher than that of the modified Kuramoto's Equation (2). Although equivalent concrete strength was used to make the calculations on structural design due to convenience, this did not take into account differences in structural characteristics and shear resistant mechanisms between PCa and RC column members. It would be more rational to consider them.

CONCLUSION

The results can be summarized as follows:

1) No substantial difference was noted either in the crack pattern or the failure mode between PCa and RC column members under high axial load.
2) The elastic rigidity, ultimate strength, and ultimate rotation angle of the PCa column member were found to be equivalent to or superior to those of the RC column member.
3) The performance of average axial strain of column and rotation angle of column ends of PCa column member was similar to that of RC column member under high axial load.
4) The bond strength of the main reinforcement of PCa column members could be estimated using the bond strength equation as proposed by the Structural Design Guideline for New RC Structure.
5) The elastic rigidity, flexural strength and shear strength of PCa column members under high axial load could be correctly calculated by modified calculation methods, where the strength and characteristic of the precast concrete and the cast-in-place concrete are considered separately.

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