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THE PROPOSAL OF AN ASEISMIC REINFORCEMENT METHOD OF RC FRAME STRUCTURES BY PRECAST-PRESTRESSED CONCRETE FRAMED SHEAR WALLS AND ITS EXPERIMENTAL VERIFICATION.

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SUMMARY

This paper has two objectives, first, to propose an aseismic reinforcement method of existing RC frame structures by connecting $\mathrm{PCa} \cdot \mathrm{PC}$ shear walls, and second, to verify the validity of the proposed aseismic reinforcement method by an experiment. In the proposed aseismic reinforcement method, the upper and lower beams of $\mathrm{PCa} \cdot \mathrm{PC}$ shear walls are only connected with those of existing RC frame structures by prestressing PC bars, but the side columns are not connected. This connecting method aims to avoid damage and shear failure of existing RC frame structures. The experiment for verification of the validity of the proposed aseismic reinforcement method was executed for six specimens. The main parameters of the specimens are the failure mode of RC frame structures and ones which have an opening of wall panels or not. All the specimens failed in the same failure mode as that of RC frame structures, and showed large resisting strength and sufficient ductility in any failure mode. The ratios of the observed maximum strength to the calculated maximum strength of the specimens are 0.81-0.96. From this study it is verified that the proposed aseismic reinforcement method is effective to increase the strength, rigidity, and ductility of existing RC frame structures

INTRODUCTION

Since the Southern Hyogo Prefecture Earthquake 1995, the aseismic reinforcement works of existing RC frame structures of school buildings have been actively executed in Japan. Existing RC frame structures in the longitudinal direction of school buildings are not sufficient in strength and rigidity for lack of shear walls. In order to reinforce such existing RC frame structures, cast in place concrete wall panels are widely adopted as shear walls. However, This aseismic reinforcement method has the following weak points.

1) Existing RC frame structures suffer, more or less, damage accompanied with aseismic reinforcement works.

2) In a severe earthquake the infilled wall panel acts as compressive struts against the side columns, and then the chance of shear failure of the side columns increases.

3) The term of aseismic reinforcement work is long, and then the availability of buildings is injured.

On the other hand, in the previous paper we ascertained experimentally that precast-prestressed framed shear walls (hereafter, referred to $\mathrm{PCa} \cdot \mathrm{PC}$ shear walls) are sufficient in strength, rigidity, and ductility [Mochizuki, M., *et al*., 1996]. This paper proposes an aseismic reinforcement method using $\mathrm{PCa} \cdot \mathrm{PC}$ shear walls to avoid the above mentioned weak points. In the proposed aseismic reinforcement method the upper and lower beams of $\mathrm{PCa} \cdot \mathrm{PC}$ shear walls are connected with those of existing RC frame structures by prestressing PC bars, but both columns of $\mathrm{PCa} \cdot \mathrm{PC}$ shear walls are not connected as shown in Figure 1. The objectives of this paper are first, to describe the proposed aseismic reinforcement method, second, in order to verify the validity of the proposed

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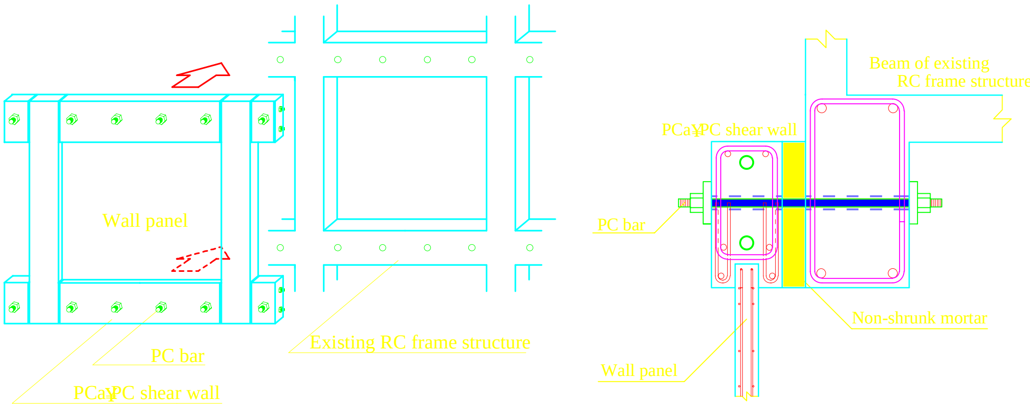


(a) Connecting method **(b) Detail of joint**

Figure 1: Aseismic reinforcement method

reinforcement method, to clarify the failure behaviors of the specimens of RC frame structures reinforced by PCa·PC shear walls, and finally to establish the estimation method of the maximum strength of the specimens by the authors' macroscopic model.

PROCEDURE OF THE PROPOSED ASEISMIC REINFORCEMENT METHOD

The procedure of the proposed aseismic reinforcement method takes the following steps.

Step 1. Making PCa·PC shear wall having necessary strength, rigidity, and ductility.

Step 2. Coring at the middle height of the upper and lower beams of existing RC frame structure in order to insert PC bars.

Step 3. Setting PCa·PC shear wall along existing RC frame structure, and then grouting non-shrunk mortar with σ_B =50 N/mm^2 grade into the gap between two beams of both structures.

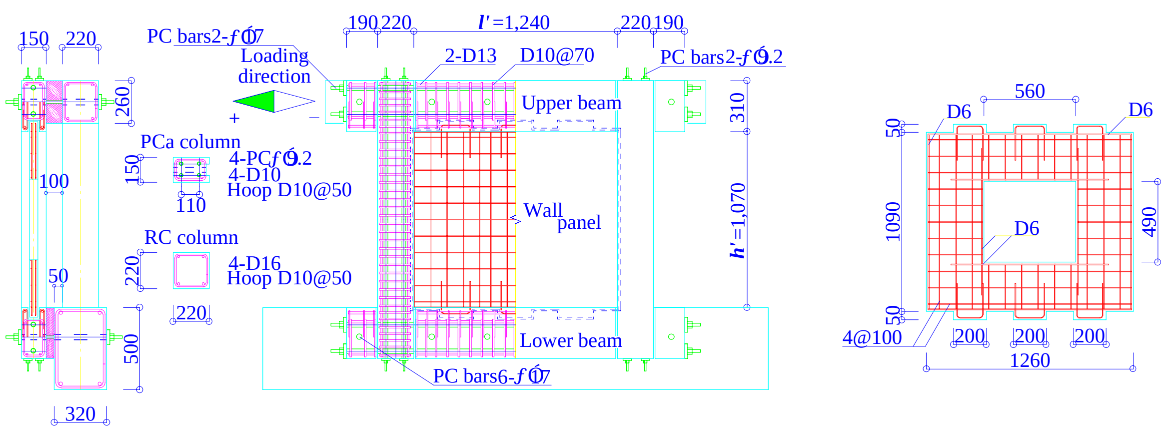
Step 4. Connecting both structures by prestressing PC bars after hardening of non-shrunk mortar.

This method can avoid the above mentioned weak points in the use of cast in place concrete wall panels, but the use of this method is limited to the outer sides of existing RC frame structures.

EXPERIMENTAL PROGRAM

Specimens

The experiment was executed on six specimens of RC frame structures reinforced by PCa·PC shear walls. The RC frame structures have one span and one story. The failure mode of RC frame structures is flexural for



(a) 98PCWB-1,2 **(b) Wall panel with an opening**

Figure 2: Specimen (Unit : mm)

98PCWB-1,2 and 98PCWO-1,2, and flexural-shear for 98PCWS-1,2. The PCa·PC shear walls are common to all the specimens except that the wall panels of 98PCWO-1,2 have an opening. The PCa·PC shear wall consists of upper and lower beams having large sectional area and sufficient reinforcement, two side columns, which were sufficiently reinforced to assure not to fail in shear, and a wall panel. The upper and lower horizontal sides of the wall panel are connected with the beams by cotter joints. And the vertical sides of the wall panel are not connected, but the wall panel is inserted into the side columns by 10mm in depth to avoid slippage of grouted mortar. Figure 2 shows the configuration, reinforcement of 98PCWB-1,2 and the reinforcement of the wall panel with an opening. This figure is common to other specimens except the reinforcement of the side columns of RC frame structure. The longitudinal bars of the side columns are 4-D16 for 98PCWB-1,2, 98PCWO-1,2 and 6-D19 for 98PCWS-1,2. The properties and experimental results of the specimens are shown in Table 1.

Loading and measuring method

All the specimens were subjected to a lateral cyclic force acting on the upper beam of the RC frame structure by an actuator. The loading cycles were two for every incremental amplitude of story deflection angle: ƒ¢$R = 1.0\times10^{-3}$ rad. until the maximum strength or story deflection angle: $R = 10.0\times10^{-3}$ rad.. After that, only one loading cycle was for every incremental amplitude of ƒ¢$R = 2.0\times10^{-3}$ rad. until $R = 20.0\times10^{-3}$ rad.. The relative horizontal displacement of the upper beam to the lower beam of the PCa·PC shear wall was measured with a displacement gauge, and the strains of PC bars and main reinforcing bars of the RC frame structures at the top and bottom ends of the columns were measured with strain gauges. The lateral displacement of the specimen perpendicular to loading direction was constrained, because the resultant line of rigidity of the specimen did not agree with the loading direction.

Table 1 (a): Properties of PCa·PC shear walls and experimental results of specimens

Code of Specimen	Wall panel			Column				Results			
								Qexp		Rb	
	$\ell'\times h'$	t	σ_B	Pg	σ_y	σ_B	Np	+	-	+	-
98PCWB-1			14	1.67		36		402	463	18.0	13.5
98PCWB-2			22		1340	23	216	452	471	Over 20	
98PCWS-1	124×107	5.0	14	4-φ 9.2		36		579	542	17.6	17.5
98PCWS-2			22		(348)	23	(4-φ 9.2)	564	567	11.5	11.2
98PCWO-1			22	(4-D10)		36		349	382	Over 20	16.1
98PCWO-2			22			30		341	385	17.2	13.9

[Notation] ℓ' (cm)×h′ (cm) : Inside measurement of wall panel, t (cm): Thickness of wall panel

σ_B (N/mm^2): Compressive strength of concrete cylinder

Pg(%): Gross main reinforcement ratio of column,

σ_y (N/mm^2): Yield strength of PC bar, ( ):yield strength of D10

Np (kN): Total prestressing force of column,

Qexp (kN): Observed maximum strength of specimen

Rb (× 10^{-3} rad.): Observed maximum story deflection angle of specimen at 0.8Qexp

b (cm)× D (cm) : Sectional dimension of column or beam

[Common] b×D =15(cm)× 22 (cm): column, b×D =15 (cm)× 31 (cm): upper beam

Reinforced bar of wall panel: φ 4-@100, σ_y =509 (N/mm^2)

Reinforcement ratio of wall panel: Ps=0.5 %

Hoop bar of column: D10-@50, σ_y =348 (N/mm^2)

Total prestressing force of upper beam: 2- φ 17,Np=333 (kN)

Table 1(b): Properties of RC frame structures

Code of specimen	$\ell\times h'$	Column					σ_B
		b×D	Pg	σ_y	Pw	σ_y	
98PCWB-1			1.64	355	1.29	348	28
98PCWB-2							
98PCWS-1	146×107	22×22	2.37	354	0.58	280	24
98PCWS-2							
98PCWO-1			1.64	355	1.29	348	28
98PCWO-2							

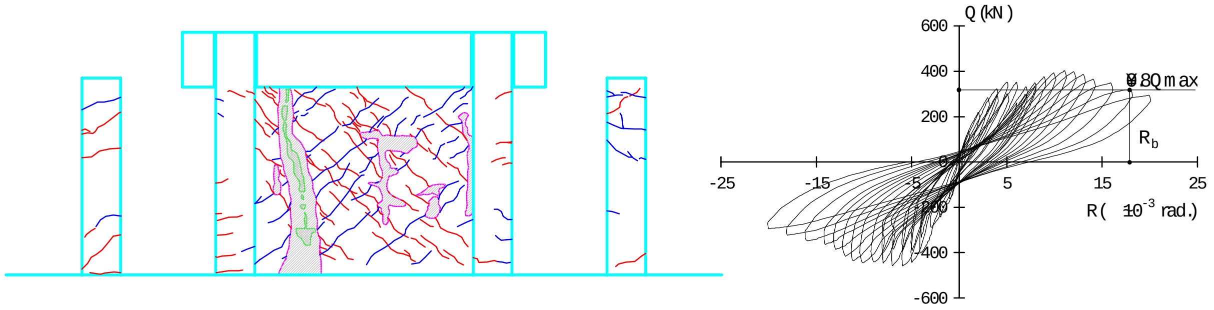
[Notation] ℓ (cm): Spacing of two columns, Pw(%): Hoop reinforcement ratio

[Common] Upper beam: b×D =22 (cm)× 24 (cm), 4-D22 (main bar), D10-@100 (stirrup bar)

EXPERIMENTAL RESULTS

Failure Behaviors

98PCWB-1: At $R=1.0\text{-}3.0\times10^{-3}$ rad., the flexural and diagonal cracks were observed at the bottom ends of the columns of the RC frame structure and on the wall panel, respectively. At $R=4.0\text{-}5.0\times10^{-3}$ rad., the flexural cracks were also observed at the bottom ends of the columns of the PCa · PC shear wall. The specimen reached the maximum load at $R=7.0\times10^{-3}$ rad., and then the load slowly decreased with crushing of the wall panel. The failure behavior of 98PCWB-2 was the same as 98PCWB-1. Figure 3 shows the crack pattern at the final state and the force-displacement relationship of 98PCWB-1.

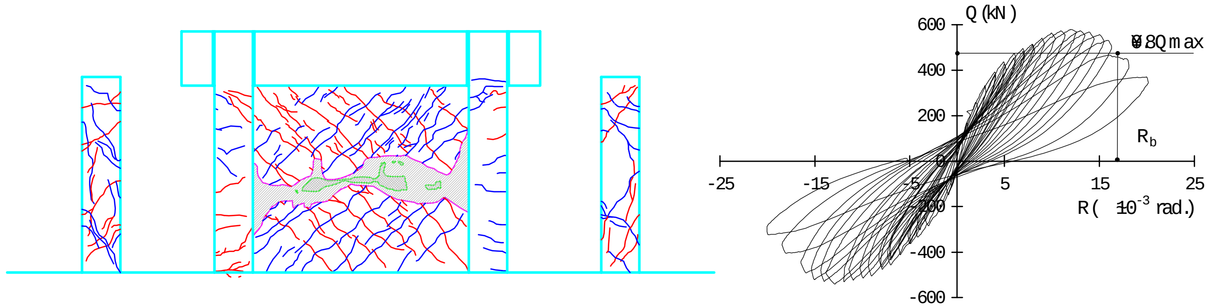


Back (Left RC column) | Front (**PCa** · **PC** shear wall) | Back (Right RC column)

(a) Crack pattern **(b) Force-displacement relationship**

Figure 3: 98PCWB-1

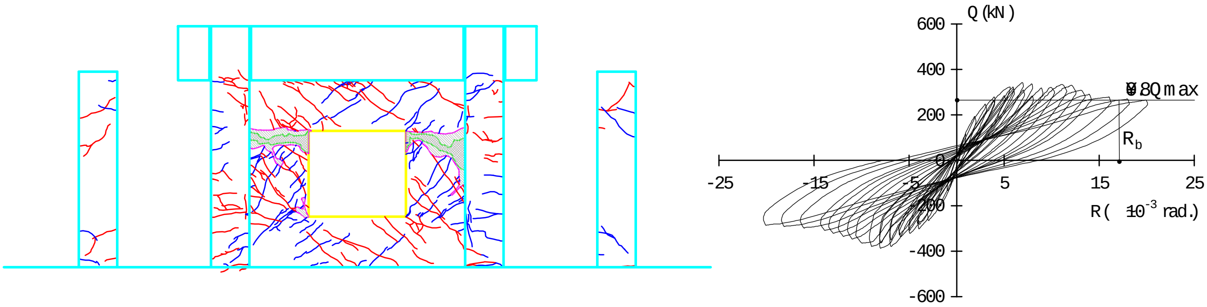
98PCWS-2: At $R=1.0\times10^{-3}$ rad., the inclined cracks on the diagonal part of the wall panel and the flexural cracks at the bottom ends of the columns of the RC frame structure were observed. At $R=5.0\times10^{-3}$ rad., the flexural cracks were also observed along the columns of the PCa · PC shear wall. The specimen reached the maximum load at $R=12.0\times10^{-3}$ rad.. After that, the crush and collapse of the wall panel were observed accompanied with increasing displacement, and the load slowly decreased. The failure behavior of 98PCWS-1 was the same as 98PCWS-2. These specimens were strong in comparison with 98PCWB-1,2 due to sufficient reinforcement of the columns of the RC frame structures, but the shear cracks of the side columns of the RC frame structures were numerous. Figure 4 shows the crack pattern at the final state and the force-displacement relationship of 98PCWS-2.



Back (Left RC column) | Front (**PCa** · **PC** shear wall) | Back (Right RC column)

(a) Crack pattern **(b) Force-displacement relationship**

Figure 4: 98PCWS-2



Back (Left RC column) Front (**PCa · PC** shear wall) Back (Right RC column)

(a) Crack pattern **(b) Force-displacement relationship**

Figure 5: 98PCWO-2

98PCWO-1: At $R=1.0\times10^{-3}$ rad., the inclined cracks from the corners of an opening of the wall panel and the flexural cracks at the bottom ends of the columns of the RC frame structure were observed. At $R=3.0\times10^{-3}$ rad., the flexural cracks of the columns of the PCa · PC shear wall were observed. The specimen reached the maximum load at $R=8.0\times10^{-3}$ rad.. After that, the load slowly decreased with crushing of the wall panel. Crack width at the bottom end of the columns in tension increased after $R=8.0\times10^{-3}$ rad. and reached about 5mm at the final state. The failure behavior of 98PCWO-2 was the same as 98PCWO-1. Figure 5 shows the crack pattern at the final state and the force-displacement relationship of 98PCWO-2.

The common failure behaviors of all the specimens are summarized as follows,

1) The column in tension and the column in compression at the bottom ends of the PCa · PC shear walls were under tensile yielding and under or near flexural yielding at the maximum strength, respectively. On the other side, the both columns of the RC frame structures were under flexural yielding at the top and bottom ends.

2) The slippage of grouted mortar along the vertical and horizontal sides of the wall panel was not observed.

3) The discrepancy between the RC frame structure and the PCa · PC shear wall was not observed.

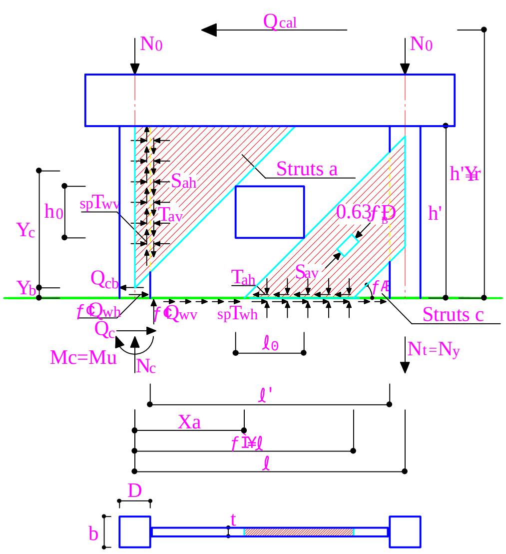


Figure 6: Macroscopic model of PCa · PC shear walls with an opening

MAXIMUM STRENGTH OF PCA · PC SHEAR WALL

The maximum strengths of RC framed shear walls and PCa · PC shear walls may be estimated from the assumed stress conditions of macroscopic models, which indicate the yield mechanisms at the maximum strength. This is based on the lower bound theorem of the limit analysis theory, and the maximum strength is derived only from the equilibrium conditions. Figure 5 show the authors' macroscopic model of the PCa · PC shear walls with an opening. In the case of PCa · PC shear wall without an opening, it is applied by treating the dimensions of the opening as zero. The macroscopic model consists of upper and lower beam, two side columns, and compressive strut ***a*** and ***c*** with the same inclination angle of θ deg., and is subjected to vertical constant loads *No* acting on the side columns and a horizontal load *Qcal* at the height of $h'\cdot r$ which is the height of inflections point of bending moment distribution. Each member of the macroscopic model is assumed to be under the following conditions at the maximum strength.

1) Upper and lower beams are rigid, and they do not fail.

2) Column in tension is under tensile yielding at the bottom end. And Column in compression is under flexural yielding at the bottom end.
3) Compressive strut ***a*** are under yielding at stress of $0.63\sigma_B$, which is the effective compressive strength of concrete proposed by the authors [Mochizuki, M., *et al.*, 1990]. Compressive strut ***c*** are removed, because the part of the column in tension crossing the strut ***c*** is under tensile yielding.
4) The difference ΔQwh between the summation of horizontal forces of the compressive struts ***a*** acting on the lower beam and the summation of sliding strength *spTah* of the lower horizontal joint of the wall panel is assumed to be directly transmitted to the column in compression, and similarly the difference ΔQwv between the summation of vertical forces and the summation of sliding strength *spTav* of the vertical joint of the wall panel is assumed to be transmitted to the lower beam.

Due to the assumption 4) the wall panel does not slide, and then the macroscopic model is almost the same as that of RC framed shear wall proposed by the authors [Mochizuki M., *et al*., 1992].

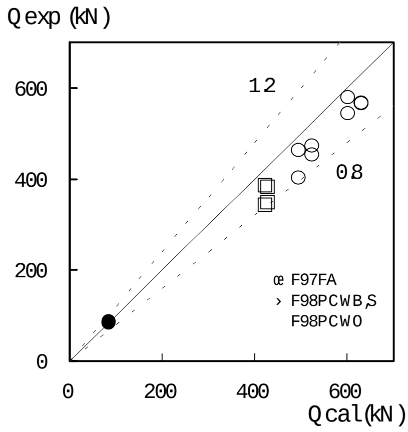


Figure 7: Comparison between observed and calculated maximum strengths

The maximum strength *Qcal* of $\mathrm{PCa}\cdot\mathrm{PC}$ shear wall with an opening is evaluated as the summation of total horizontal force *Qw* of the compressive struts on the top force of lower beam and the shear force *Qcb* of the column in compression as follows,

$$Qcal = Qw + Qcb \tag{1}$$

$$Qw = Tah(\xi \cdot \ell - \mathrm{X}a) \tag{2}$$

$$Qcb = Sah\left(\sqrt{Yb^2 + 2\frac{Mu}{Sah}} - Yb\right) \tag{3}$$

where, Tah=$0.63\sigma_B \cdot \cos\theta \cdot \sin\theta$, Sah=$0.63\sigma_B \cdot \cos\theta \cdot \cos\theta$.

Qcb in Equation (3) is derived by treating the side column in compression as a cantilever column which is subjected to horizontal forces along the column and a flexural yield moment *Mu* at the bottom end of the column.
The unknown coefficient ξ of effective horizontal width of the compressive struts is given from the equilibrium of moment at the bottom end of the column in compression as follows,

$$\xi = \eta \cdot \left(\frac{Tah}{Sav}\right) + \sqrt{\eta^2\left(\frac{Tah}{Sav}\right)^2 + \frac{1}{\ell}\left\{2\ell \cdot (-Xa) \cdot \eta \cdot \left(\frac{Tah}{Sav}\right) + Xa^2\right\} + \chi} \qquad \leq 1.0 \tag{4}$$

where,

$$\eta = -h' \cdot r / \ell\,,\ \chi = \frac{2}{Sav \cdot \ell}\left[\left\{\eta \cdot Sah - \frac{D \cdot (Tav - spTwv)}{2\ell}\right\}\left(\sqrt{Yb^2 + \frac{Ny \cdot D}{Sah}} - Yb\right) + No + Ny \cdot \left(\frac{D}{2\ell} + 1\right)\right],$$

Tav=$0.63\sigma_B \cdot \sin\theta \cdot \cos\theta$, Sav=$0.63\sigma_B \cdot \sin\theta \cdot \sin\theta$, $spTwv = \mu \cdot Sah$, μ :coefficient of friction.

The value of axial force *Nc* at the bottom end of the column in compression is given from the equilibrium of moment at the bottom end of the column in tension using ξ from Equation (6) as follows,

$$\begin{aligned} Nc = &\left\{\eta' \cdot Sah - (Tav - spTwv)\left(1 - \frac{D}{2\ell}\right)\right\}\left(\sqrt{Yb^2 + \frac{Ny \cdot D}{Sah}} - Yb\right) + No - \frac{Ny \cdot D}{2\ell} \\ &+ (\xi \cdot \ell - Xa)(\eta' \cdot Tah - Sav) + \frac{Sav}{2\ell}\left(\xi^2 \cdot \ell^2 - Xa^2\right) \end{aligned} \tag{5}$$

where, $\eta' = h' \cdot r / \ell$.
The second approximate value of *Qcb* is evaluated in consideration of the effect of the axial force *Nc* on flexural yield strength of column as follows,

$$Qcb = Sah\left(\sqrt{Yb^2 + 2\frac{Mu(Nc)}{Sah}} - Yb\right) \tag{6}$$

$$Mu = (0.5Ny + Nc)(0.5D - 0.42Xn) + 0.5^2 Ny \cdot e\ ,\ \ Xn = \frac{(0.5Ny + Nc)}{0.83b \cdot \sigma_B} \tag{7}$$

Equation (7) is the flexural yield strength of PC column given in Standard for Structural Design and Construction of Prestressed Concrete Structure of AIJ. Finally, the maximum strength *Qcal* of $\mathrm{PCa} \cdot \mathrm{PC}$ shear wall with an opening is evaluated as the summation of *Qw* from Equation (2) and *Qcb* from Equation (3).

ANALYTICAL RESULTS

The analyses were executed for all the specimens using the maximum strength formulae shown in the preceding chapter 5. For the inclination angle of the compressive struts of the $\mathrm{PCa} \cdot \mathrm{PC}$ shear wall, the following equation was used.

$$\left.\begin{array}{ll} \ell'/h' \le 0.8\ \ \theta = 25^\circ \cdot \ell'/h' + 25^\circ, & 1.2 \le \ell'/h' \le 1.8\ \ \theta = 25^\circ \cdot \ell'/h' + 15^\circ \\ 0.8 \le \ell'/h' \le 1.2\ \ \theta = 45^\circ, & \ell'/h' \ge 1.8\ \ \theta = 60^\circ \end{array}\right\} \tag{8}$$

This equation is derived by the authors from the experiments and analyses of RC framed shear walls. The maximum strength *Qcal* of the specimen was estimated as the summation of the shear force *Qcal1* of the RC frame structure, which is assumed to be under flexural yielding at the top and bottom ends of the columns and the maximum strength *Qcal2* of the $\mathrm{PCa} \cdot \mathrm{PC}$ shear wall. **Table 2** shows the analytical results, and **Figure 7** shows the comparison between the observed and calculated maximum strengths. In the figure the black point show the sample point of a RC frame structure which is similar to the RC frame structures of 98PCWB-1,2 and

Table 2:Analytical results

Specimen	Qexp		Qcal 1	Qcal 2	Qcal	Qexp/Qcal	
	+	-				+	-
98PCWB-1	402	463	84	410	494	0.81	0.94
98PCWB-2	452	471	84	439	523	0.86	0.90
98PCWS-1	579	542	192	410	602	0.96	0.90
98PCWS-2	564	567	192	439	631	0.89	0.90
98PCWO-1	349	382	84	344	428	0.81	0.89
98PCWO-2	341	385	84	338	422	0.81	0.91

[Notation] Qexp (kN) : Observed maximum strength of specimens
Qcal 1 (kN) : Calculated maximum strength of RC frame structures
Qcal 2 (kN) : Calculated maximum strength of $\mathrm{PCa} \cdot \mathrm{PC}$ framed shear walls
Qcal (kN) : Calculated maximum strength of specimens

98PCWO-1,2 and was conducted in the preceding experiment. These table and figure show that the analytical method using the authors' macroscopic model is effective to estimate the maximum strength of the specimens. However, the calculated values are estimated a little larger than the observed values. This is based on the discrepancy between the assumed and real stress distributions of the side columns of the $\mathrm{PCa} \cdot \mathrm{PC}$ shear walls.

CONCLUSIONS

The conclusions of this study are summarized as follows,

1) The failure behaviors of the specimens depend on the failure mode of the RC frame structure.
2) The maximum strength of the specimens is well estimated as the summation of the shear force RC frame structure under flexural yielding at the top and bottom ends of the side columns and the maximum strength of the $\mathrm{PCa} \cdot \mathrm{PC}$ shear wall.

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