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FAILURE BEHAVIORS AND STRENGTH ESTIMATION OF PCa•PC FRAMED SHEAR WALLS

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ABSTRACT

This paper describes the failure behaviors of precast-prestressed concrete framed shear walls and the simplified formula for evaluating their maximum strength. The experiment was executed on the specimens with perfect or imperfect constraint against sliding along the bottom joints of columns. The experimental results show that the specimens have sufficient strength and high ductility. The simplified formulae were derived only from equilibriums of the assumed stress condition at the maximum strength. The calculated maximum strengths agree well with the observed maximum strengths in the experiment.

KEYWORDS

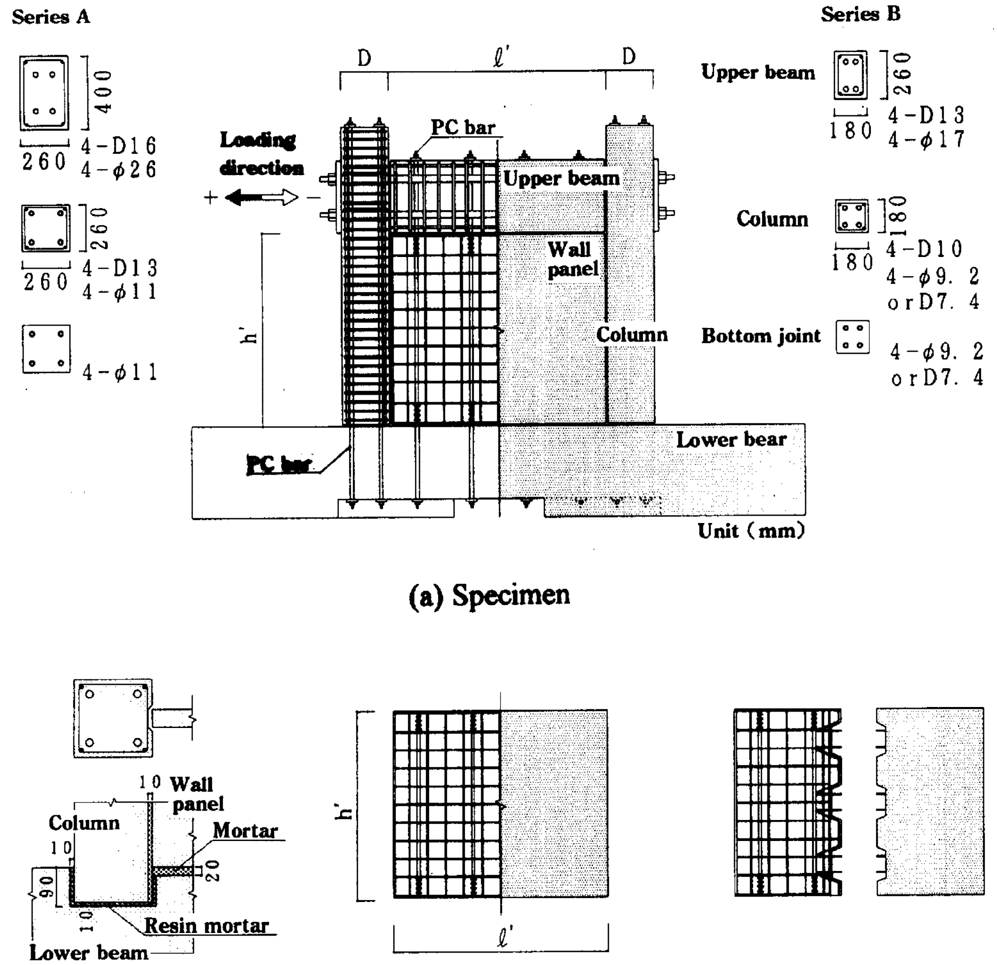
Precast Concrete, Prestressed Concrete, Framed Shear Wall, Maximum Strength, Macroscopic Model.

OBJECT

As an effective shear resisting element, a precast-prestressed concrete frame with an infilled precast wall panel (hereafter referred to PCa·PC framed shear wall) has been used, but its structural behaviors are not yet clarified. Especially, the simplified method for evaluating the maximum strength with sufficient accuracy is not given. The objectives of this paper are firstly, to clarify the failure behaviors of PCa·PC framed shear walls with perfect or imperfect constraint against sliding along the bottom joints of columns, secondly, to derive the simplified formulae for evaluating the maximum strength, and finally, to ascertain the validity of the simplified formulae.

EXPERIMENTAL PROGRAM

The experiment was executed on twelve specimens of PCa·PC framed shear walls of two Series A and B. All the specimens consist of upper and lower beams, two columns, and a wall panel. Fig.1(a) shows the configuration, bar reinforcement, and method of connection of precast elements of the specimens in Series A



(a) Specimen

(b) Detail of bottom end of column

(c) Wall panel I

(d) Wall panel II

Fig.1 Specimen

and B. The columns of the specimens in Series A are imperfectly constrained only by shear friction and dowel action of connecting PC bars, but the columns of the specimens in Series B are inserted sufficiently to assure perfect constraint against sliding into the lower beam as shown in Fig.1(b). The wall panel of 92PCWB-1,2 in Series A and all the specimens in Series B is shown in Fig.1(c), and is connected with the frame through PC bars manually prestressed. The vertical and horizontal joints of the wall panel are grouted, though they are not connected by reinforcing bars. The wall panel of 92PCWC-1,2 in Series A is composed of two panels shown in Fig.1(d), of which the central joint has three cotters and connecting bars, but other joints are the same as those of other specimens in the same series. The sheaths of the wall panel are not grouted in all the specimens in Series A and B. All the specimens were subjected to a lateral cyclic force acting on the upper beam of the specimen by an actuator. The loading cycles were two for every incremental amplitude of $R = 1.0 \times 10^{-3}$ rad. of story deflection angle until the maximum strength. After that, only one loading cycle was applied. In Table 1, the properties of all the specimens are shown.

Mochizuki

Table 1 Properties and experimental results

Series	Code of Specimen	$l' \times h'$	wall panel: te	wall panel: ps	wall panel: σ_y	wall panel: σ_B	Column: p_t column	Column: p_t bottom joint	Column: σ_y	Column: σ_B	Result: Q exp +	Result: Q exp −	Result: R b +	Result: R b −
A	92PCWB-1	120 × 105	6.0	0.53	12 167	300	1.31	0.56	12 167	585	52.7	60.2	12.0	10.7
	92PCWB-2	120 × 105	6.0	0.53	12 167	289	1.31	0.56	12 167	547	59.1	59.5	12.1	12.3
	92PCWC-1	120 × 105	6.0	0.53	12 167	348	1.31	0.56	12 167	444	53.2	62.2	14.2	9.2
	92PCWC-2	120 × 105	6.0	0.53	12 167	351	1.31	0.56	12 167	371	56.9	59.3	13.5	9.5
B	94PCWA-1	82 × 62	5.0	0.39	13 103	685	1.37	0.49	13 130	664	45.6	43.6	20 over	
	94PCWA-2	82 × 62	5.0	0.39	13 103	634	1.37	0.49	13 130	664	40.6	42.9	19.7	20 over
	94PCWA-3	82 × 62	4.0	0.49	13 103	685	1.70	0.89	13 750	664	49.2	49.4	16.9	16.4
	94PCWA-4	82 × 62	4.0	0.49	13 103	634	1.70	0.89	13 750	664	46.8	49.1	20 over	17.6
	94PCWB-1	82 × 112	5.0	0.39	13 103	694	1.37	0.49	13 130	675	29.8	32.3	10.3	14.3
	94PCWB-2	82 × 112	5.0	0.39	13 103	634	1.37	0.49	13 130	675	30.3	33.0	10.3	8.8
	94PCWB-3	82 × 112	4.0	0.49	13 103	694	1.70	0.89	13 750	675	37.3	32.2	18.6	20 over
	94PCWB-4	82 × 112	4.0	0.49	13 103	634	1.70	0.89	13 750	675	40.7	42.7	18.0	20.0

【Notation】 l' (cm) × h' (cm) : Inside mesurement of wall panel
te (cm) : Eeffective thickness
$te = t - \phi$ (t is the thickness of wall panel, and ϕ is the diameter of sheath.)
ps (%) : Ratio of reinforcement of wall panel
p_t (%) : Ratio of gross reinforcement of column
σ_y (kgf/cm^2) : Yeild strength of PC bar
σ_B (kgf/cm^2) : Compressive strength of concrete cylinder
Q exp (tf) : Observed maximum strength
Rb ($\times 10^{-3}$ rad.) : Observed maximum story deffection angle
+ , − : Positive and negative loading directions

【Common】

	Series A	Series B
Main reinforcing bar of column	D13, σ_y = 3 568(kgf/cm^2)	D10, σ_y = 3 608(kgf/cm^2)
Hoop bar of column	D10-@50 , σ_y = 3 792	D6-@40, σ_y = 3 021
Reinforced bar of wall panel	D6-@100 , σ_y = 3 568	ϕ 4-@50, σ_y = 5 552
Total prestressing force of upper beam	4- ϕ 26 , ΣPi =164.4 (tf)	4- ϕ 17 , ΣPi = 68.0 (tf)

EXPERIMENTAL RESULTS

Failure Behaviors

92PCWB-1,2 At $R = 1.0 \times 10^{-3}$ rad., the tensile and flexural cracks were observed at the bottom joints of the columns in tension and in compression, respectively. And at $R = 2.0 \times 10^{-3}$ rad., the cracks in the diagonal part of the wall panel and the flexural cracks at the upper and middle parts of both columns were observed. Until the maximum load, the mortar grouted at the vertical joints of wall panel crushed and slipped along the corner parts of the wall panel, and the sliding along the bottom joint of the wall panel and the partial crushing of its corner parts occurred. At $R = 8.0 \times 10^{-3}$ rad., the specimens reached the maximum load accompanied with sliding along the bottom joint of the column in compression. Both displacements due to sliding along the bottom joints of the wall panel and column in compression reached about 5mm and 2mm until $R = 8.0 \times 10^{-3}$ rad., respectively. Then, the sliding occurred continuously under cyclic loading, and the hysteretic behavior of force-displacement relationship was affected by sliding due to imperfect constraint at the bottom joints of the columns.

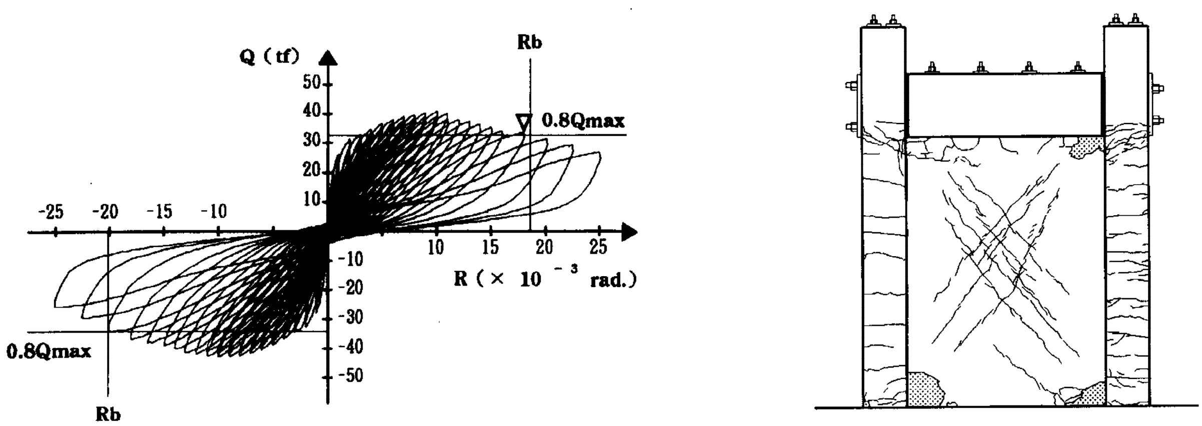
92PCWC-1,2 The failure behaviors of these specimens were similar to those of 92PCWB-1,2, and were not affected by composition of two panels.

94PCWA-1.2 At $R = 1.0 \sim 2.0 \times 10^{-3}$ rad., the flexural crack at the bottom end of the column in tension and the inclined cracks in the wall panel were observed. Until $R = 8.0 \times 10^{-3}$ rad., the inclined cracks and horizontal crack of the bottom joint of the wall panel extended, and the flexural cracks at the top ends and middle parts of the columns formed. The width of a gap between the wall panel and columns expanded along the corner parts of the wall panel. The specimens reached the maximum load at $R = 6.0 \sim 7.0 \times 10^{-3}$ rad., accompanied with crushing of the corner parts of the wall panel and with slipping of mortal grouted at the vertical joints of the wall panel. The maximum story deflection angles of both specimens were larger than 20.0×10^{-3} rad., and the failure behaviors were more ductile compared with those of specimens of Series A.

94PCWA-3,4 The failure behaviors of both specimens were similar to those of 94PCWA-1,2 until the maximum load. After that, the flexural cracks at the middle parts of the columns formed numerously.

94PCWB-1,2 At $R = 1.0 \times 10^{-3}$ rad., the flexural cracks at the bottom joint of the tensile column formed, and extended to the bottom joint of the wall panel. Until the maximum load, the tensile cracks between the top and bottom ends of the columns and inclined cracks of the wall panel formed numerously. Both specimens reached the maximum load at $R = 5.0 \times 10^{-3}$ rad., and the enveloped curve of the force-displacement relationship exhibited a straight line. However, the maximum story deflection angles were small because of the fracture of PC bars of the wall panel.

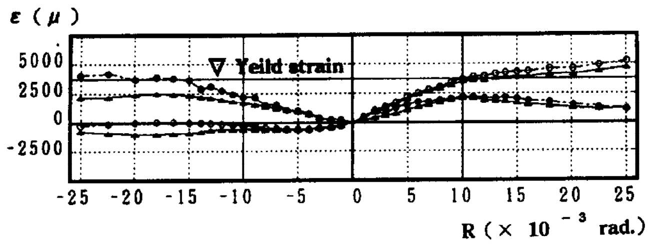
94PCWB-3.4 The failure behaviors of both specimens were similar to those of 94PCWB-1,2 until the maximum load. The specimens reached the maximum load at $R = 8.0 \sim 10.0 \times 10^{-3}$ rad., and lost gradually the resisting capacity. However, though the crushing of corner parts of the wall panel was intense, the slipping of mortar grouted at the vertical joint of the wall panel was not. Fig.2 shows the force-displacement relationship and crack pattern at the maximum load of 94PCWB-4.



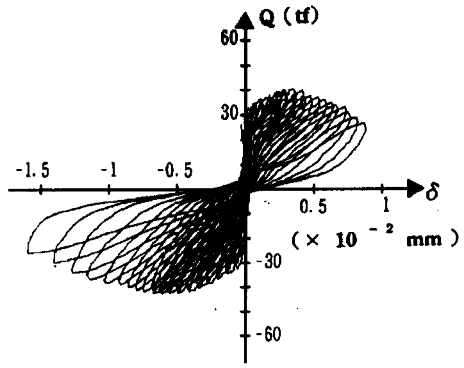
(a) Force-displacement relationship

(b) Crack pattern

Fig.2 Failure behavior of 94PCWB-4



(a) strains of PC bars



(b) Sliding displacement

Fig.3 Strain and displacement of 94PCWS-4

Resisting Mechanism

Figure 3(a) shows the relationship between the strains of PC bars at the bottom end of the right column and the story deflection angle of 94PCWB-4. The figure also shows that the right column is under tensile and flexural yielding at the maximum loads of positive and negative loading directions, respectively. These behaviors were also observed in other specimens of Series A and B. Fig.3(b) shows the relationship between the load and horizontal displacement at the bottom end of the right column of 94PCWB-4, and shows that the sliding of the column did not occur. These behaviors were common to other specimens of Series B. From the above mentioned failure behaviors, the resisting mechanism of the specimens at the maximum load is summarized as follows,

1) Wall panel behaves as compressive struts, and forms arch action with the column in tension.
2) Column in tension is under or near tensile yielding at the bottom end.
3) Column in compression is under flexural yielding at the bottom end.

Sliding along the bottom end of the column in compression does not occur in the specimens of Series B.

SIMPLIFIED FORMULAE

Macroscopic Model

Figure 4 shows the macroscopic model for evaluating the maximum strength of PCa·PC framed shear walls and its assumed stress condition. The macroscopic model consists of upper and lower beams, two side columns, compressive struts a and c with the same inclination angle of θ deg., and vertical PC bars. Each member of the macroscopic model is assumed to be under the following properties at the maximum strength.

1) Upper and lower beams are rigid, which do not fail.
2) Column in tension is under tensile at the bottom end. Column in compression is under flexural yielding at the bottom end. Its sliding strength $_sQ_c$ at the bottom joint is expressed by Eq.1.

$$_{s}Q_c = \mu \cdot \overline{N} + \lambda \cdot N_y/2, \tag{1}$$

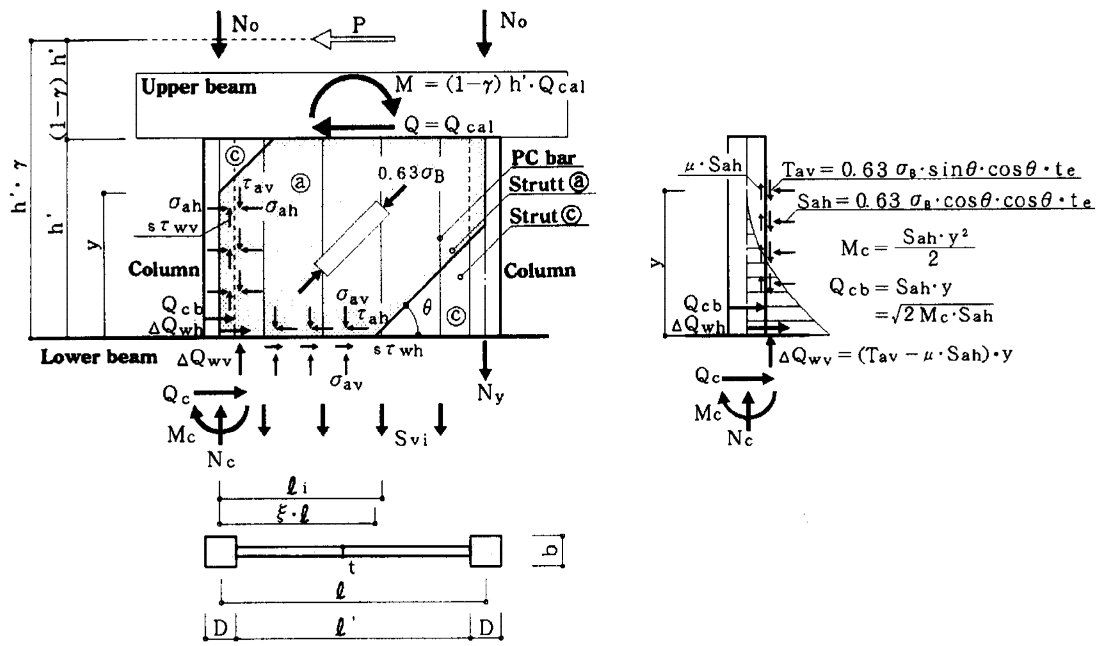
where, $\overline{N} = N_c + N_y/2$, Nc is the axial force of the column, Ny is the total yield strength of PC bars of the column, μ is the coefficient of friction, and λ is the coefficient on dowel action of PC bar. Column does not fail in shear. Column with perfect constraint does not slide.

3) Compressive struts a are under yielding at stress of $0.63\sigma_B$, which is based on the proposal by Mochizuki *et al.* (1990). Compressive struts c are removed because the part of the column in tension crossing the struts c is under flexural or tensile yielding. The sliding strengths at the horizontal and vertical joints of the compressive struts are expressed by Eqs.2(a) and 2(b), respectively.

$$_{s}\tau_{wh} = \mu \cdot \sigma_{av}, \quad _{s}\tau_{wv} = \mu \cdot \sigma_{ah}. \tag{2-a,b}$$

Where, $\sigma_{av} = 0.63\sigma_B \sin\theta \cdot \sin\theta$, and $\sigma_{ah} = 0.63\sigma_B \cdot \sin\theta \cdot \cos\theta$. In Eq.2, the term on the dowel action of PC bar is neglected because the sheath is not grouted. The differences $_{\Delta}\tau_{wh}$ or $_{\Delta}\tau_{wv}$ between the horizontal or vertical stress component of the compressive struts and $_{s}\tau_{wh}$ or $_{s}\tau_{wv}$ are assumed to be directly transmitted to the column in compression or the upper and lower beams, respectively.

4) Vertical PC bars of the wall panel are under tensile yielding.



(a) Macroscopic model.

(b) Assumed stress condition of column in compression.

Fig.4 Macroscopic Model

Simplified Formula

The maximum strength $Qcal$ of PCa·PC framed shear wall is evaluated as the summation of the total horizontal force Qw of the compressive struts and the shear force Qcb of the column in compression due to the horizontal force of the compressive struts along the column as follows, Morinaguchi

$$Q_{cal} = Q_w + Q_{cb} . \tag{3}$$

Where,
$$Q_w = T_{ah} \cdot \xi \cdot l = {}_sQ_{wh} + {}_\Delta Q_{wh}$$
$$Q_c = Q_{cb} + {}_\Delta Q_{wh} \quad \text{: for the case with perfect constraint}$$
$$= Q_{cb} + {}_\Delta Q_{wh} < {}_sQ_c \quad \text{: for the case with imperfect constraint}$$
$$T_{ah} = \tau_{ah} \cdot t_e = 0.63\sigma_B \cdot \sin\theta \cdot \cos\theta \cdot t_e \ , \ {}_sQ_{wh} = {}_s\tau_{wh} \cdot t_e \cdot \xi \cdot l \ , \ {}_\Delta Q_{wh} = \left(\tau_{ah} - {}_s\tau_{wh}\right) t_e \cdot \xi \cdot l .$$

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,

$$\xi = \eta \cdot T_{ah}/S_{av} + \sqrt{\left(\eta \cdot T_{ah}/S_{av}\right)^2 + 2\sum S_{vi} \cdot l_i / S_{av} \cdot l^2 + \chi} \ \leq 1 . \tag{4}$$

Where, $\chi = 2/S_{av} \cdot l \left[\left\{ \eta - \left(T_{av}/S_{ah} - \mu\right) D/2l \right\} \sqrt{N_y \cdot D \cdot S_{ah}} + N_0 + N_y \left(D/2l + 1\right) \right]$, $\eta = -h' \cdot \gamma / l$,

$T_{av} = 0.63\sigma_B \cdot \sin\theta \cdot \cos\theta \cdot t_e$, $S_{ah} = 0.63\sigma_B \cdot \cos\theta \cdot \cos\theta \cdot t_e$, and $S_{av} = 0.63\sigma_B \cdot \sin\theta \cdot \sin\theta \cdot t_e$.

In the derivation of Eq.3, the following first approximate values of Mc and Qc are used.

$$M_c = N_y \cdot D/2, \qquad Q_{cb} = \sqrt{2M_c \cdot S_{ah}} = \sqrt{N_y \cdot D \cdot S_{ah}} . \tag{5-a,b}$$

Equation5(b) is based on that the column in compression behaves as a cantilever column subjected to horizontal forces from the compressive struts, and its bottom end is under flexural yielding (Fig.4(b)). The second approximate value of Qcb is evaluated in consideration of the effect of axial force on flexural yield strength of column. The value of Nc at the bottom end is given from the equilibrium of moment at the bottom end of the column in tension using ξ from Eq.3 as follows.

$$N_c = \left\{ \eta' - \left(\frac{T_{av}}{S_{ah}} - \mu \right) \left(1 - D/2l\right) \right\} \sqrt{N_y \cdot D \cdot S_{ah}} + \left\{ \eta' - \frac{S_{av}}{T_{ah}} \left(1 - \xi/2\right) \right\} Q_w + \frac{\sum S_{vi} \left(l - l_i\right)}{l} + N_0 - N_y \cdot D/2l , \tag{6}$$

where, $\eta' = h' \cdot \gamma / l$. The flexural strength Mu at the bottom end of the column in compression is given by the following equation using the practical formula for flexural strength of column and Nc from Eq.6.

$$M_u = \left(0.5N_y + N_c\right)\left(0.5D - 0.42\chi_n\right) + 0.5^2 N_y \cdot e . \tag{7}$$

Where, $\chi_n = \left(0.5N_y + N_c\right) / 0.83b \cdot \sigma_B$, and e is distance between outer and inner PC bars of column. The second approximate value of Qcb is obtained using Mu from Eq.7 as follows.

$$Q_{cb} = \sqrt{2M_u(N_c)S_{ah}} \geq 2M_u/h' . \tag{8}$$

Finally, the maximum strength of PCa•PC framed shear wall is evaluated from Eq.3 as the summation of Qw using ξ from Eq.4 and Qcb using Mu from Eq.7.

ANALYTICAL RESULTS

The analyses using the simplified formulae are executed for all the specimens in Series A and B. In the analyses, μ and λ are taken as 0.6 and 0.4, respectively, based on the previous studies. The second term in $\overline{N}$ of Eq.1 is neglected based on the proposal by Kanamoto *et al.*(1992). The effective thickness *te* is used as the thickness of the compressive struts based on the proposal by Suzuki *et al.* (1992). And the inclination angle of the compressive struts is taken as the value shown in Fig.5, which is based on the experimental results of reinforced concrete framed shear walls executed by the authors. Table 2 shows the calculated maximum strengths and the ratios of the observed maximum strength to the calculated maximum strength. This table shows that the proposed simplified formulae are adequate for evaluating the maximum strength of PCa•PC framed shear walls.

Table 2 Analytical results

Series	Code of Specimen	Qexp (tf) +	Qexp (tf) −	Qcal (tf)	Qexp/Qcal +	Qexp/Qcal −
A	92PCWB-1	57.2	60.2	59.3	0.96	1.02
	92PCWB-2	59.1	59.5	59.4	0.99	1.00
	92PCWC-1	53.2	62.2	58.8	0.90	1.06
	92PCWC-1	56.9	59.3	58.8	0.97	1.01
B	94PCWA-1	45.6	43.6	42.8	1.07	1.02
	94PCWA-2	40.6	42.9	42.4	0.96	1.01
	94PCWA-3	49.2	49.4	57.0	0.86	0.87
	94PCWA-4	46.8	49.1	56.3	0.83	0.87
	94PCWB-1	29.8	32.3	30.9	0.96	1.05
	94PCWB-2	30.3	33.0	30.5	0.99	1.08
	94PCWB-3	37.3	32.2	40.5	0.92	0.80
	94PCWB-4	40.7	42.7	40.4	1.01	1.06

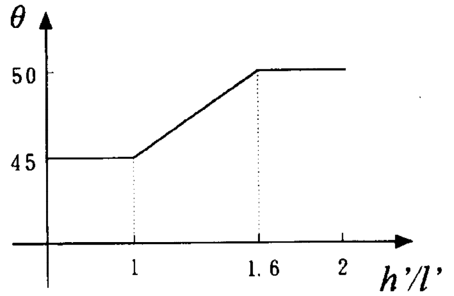


Fig.5 Inclination angle of compressive struts

CONCLUSION

From the experimental and analytical studies on the PCa•PC framed shear wall, the conclusions of this paper are summarized as follows.

1) PCa•PC framed shear walls have sufficient strength and high ductility.
The simplified formulae for evaluating the maximum strength are ascertained to be valid.

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