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Structural design equations and recommendations for multi-storey shear walls

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ABSTRACT
Seismic tests of twenty six multi-story shear walls were conducted to develop structural design recommendations. The specimens were designed in order to investigate four aspects: the effect of boundary columns on deformation capacity after flexural yielding, the effect of boundary and inner columns on shear strength, the effect of inner beams on shear strength, and the structural performance of shear walls with openings. Based on these test results, equations used in practice for structural design were examined, and new design recommendations such as the limitation of axial stress level on shear walls and the required ductility of coupling beams were proposed.

1 INTRODUCTION

In order to develop reinforced concrete high-rise (from 6 to 15 stories) frame structures with wall columns shown in Fig. 1 (hereafter referred to as HFW), Ministry of Construction of Japanese Government proposed the cooperative research program to private construction companies and universities, and organized the research committee, chaired by Prof. H. Aoyama, University of Tokyo, in Building Center of Japan. The committee promoted the project by organizing three sub-committees and some working groups.

The shear wall working group chaired by Mochizuki was one of those and researched structural performance of multi-story shear walls in the span direction of HFW. The working group focused experimental works on the following four aspects (see Figs. 2 and 3).

1. Test on the effect of boundary columns on deformation capacity (five specimens)[1].
2. Test on the effect of inner beams on shear strength[2].
3. Test on the effect of boundary and inner columns on shear strength[3].
4. Test on The structural performance of shear walls with openings (G-type and F-type shear walls)[4][5].

2 SPECIMENS

There were twenty one shear wall specimens without openings (five specimens aiming at flexural yielding, nine specimens and three specimens regarding to effects of inner beams and columns on shear strength, respectively), and five

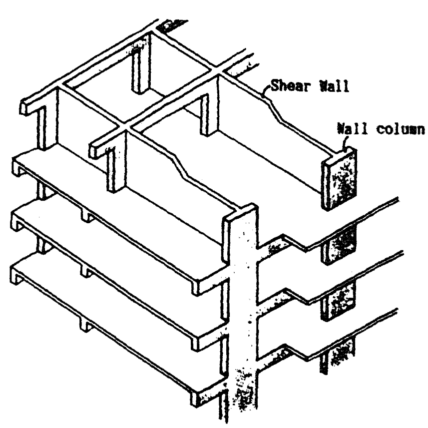


Fig. 1. A high-rise frame structure with wall columns (HFW).

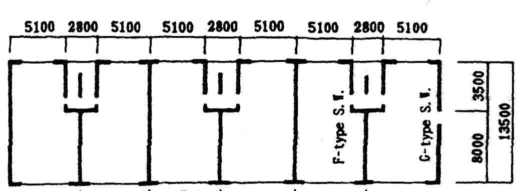
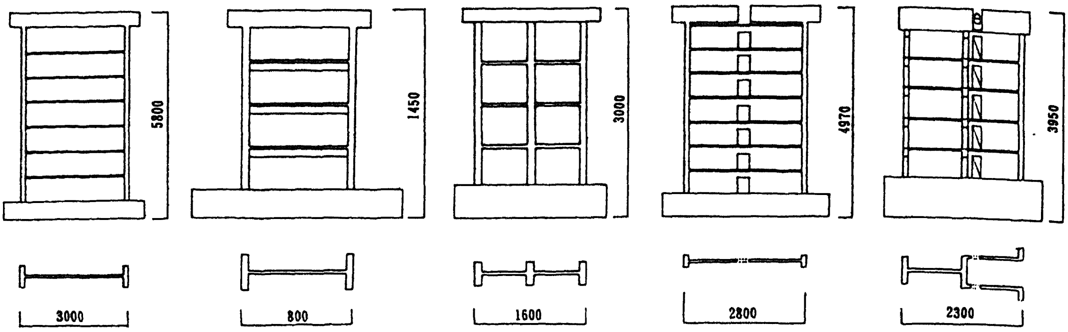


Fig. 2. An example of the plan with shear walls with openings.



S.W. failing in flexure (Shear walls without opening) S.W. failing in shear G-type S.W. F-type S.W. (Shear walls with openings)

Fig. 3. Elevations and plans of five types of shear wall specimens.

Table 1. Types of S. W. specimens and agencies in charge of each test.

Type of S.W. spec.		Agency	Members in W.G.	Number of spec.	Scale
S.W. failing in flexure		Building Research Insti.	H. Hiraishi, H. Shiohara	5	1/4
S.W. failing in shear (Effect of beams)		Meiji Univ.	Y. Kanoh, H. Takagi	9	1/10
S.W. failing in shear (Effect of columns)		Musashi Insti. of Technology	S. Mochizuki	7	1/10, 1/5
		Tokyu Construction co., LTD	T. Iwakura		
S.W. with openings	G-type	JDC Cooperation	N. Yanagisawa	2	1/5
	F-type	Tokyo Metropolitan Univ.	T. Endo, K. Iso	3	1/5

shear wall specimens with openings (two specimens of G-type and three specimens of F-type).

The specimens were designed so possibly as to represent the lower part of the fifteen-story shear walls although the specimens aiming at examinating the shear strength were different mainly in axial stress level and tensile strength in boundary columns. Table 1 shows the types of the specimens and the agencies in charge of each tests. The compression test results of cylinder of concrete were from 20 to 30 MPa.

3 CRACKING STRENGTHS

The accuracy of the equations commonly used to evaluate the cracking strengths in structural design was examined by using the forty nine isolated shear wall test results including twenty eight past ones whose shear span-depth ratios were greater than 1.

Equation examined here for flexural cracking moment is shown in Eq.(1).

$$M_{BC} = 0.56 \cdot \sqrt{\sigma_B} \cdot Z_e + N \cdot Z_e / A_e \qquad (1)$$

where:

M_{BC} = moment at flexural cracking (N·mm)

σ_B = compressive strength of concrete (MPa)

Z_e = effective section modulus (mm^3)

A_e = effective cross sectional area (mm^2)

N = axial force (N)

The following two equations were examined for shear cracking strength.

$${}_cQ_{sc1} = t \cdot I_e \cdot \sqrt{{}_c\sigma_t{}^2 + \sigma_{0e} \cdot {}_c\sigma_t} / S_{max} \qquad (2)$$

where:

${}_cQ_{sc1}$ = shear force at shear cracking by Eq. (2) (N)

I_e = effective geometrical moment of inertia (mm^4)

S_{max} = effective geometrical moment of area (mm^3)

t = thickness of wall panel (mm)

σ_{0e} = axial stress given by N/Ae (MPa)

${}_c\sigma_t$ = $0.56 \cdot \sqrt{\sigma_B}$ (MPa)

$${}_cQ_{sc2} = (0.043 \cdot p_g + 0.514) \cdot \sigma_B \cdot t \cdot \ell_w \qquad (3)$$

where:

${}_cQ_{sc2}$ = shear force at shear cracking by Eq. (3) (N)

p_g = longitudinal reinforcement ratio of a column

ℓ_w = span length between the centers of columns (mm)

The ${}_cQ_{sc1}$ is based on principal stress and the ${}_cQ_{sc2}$ is the empirical equation proposed by Sugano[6].

The averages and standard deviations of the ratios of the test results to the calculated ones given by Eqs.(1)-(3) were listed in Table 2. It is easily understood that the accuracy of the every equations is poor. Therefore safety margin against Equations(1)-(3) is recommendable in structural design.

Table 2. The ratios of the test results to the calculated results for cracking strengths.

	Eq.	Average	Standared Deviation	Range
flexure	1	1.136	0.351	0.6~2.0
shear	2	0.635	0.222	0.3~1.1
	3	1.129	0.624	0.5~2.5

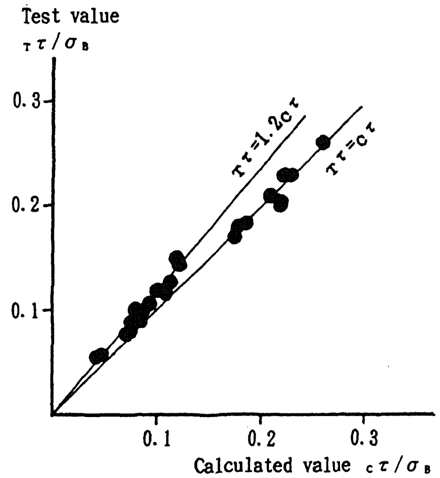


Fig. 4. Correlation of calculated values to test results for ultimate flexural strength.

4 ULTIMATE STRENGTHS

4-1 Flexural strength

Twenty seven shear walls failed in flexure (five shear wall specimens shown in Fig. 3, and twenty two shear walls tested in past) were used in order to examine the accuracy of the following equation which evaluate the ultimate flexural strength.

$$M_u=\Sigma(a_t\cdot\sigma_y)\cdot\ell_w+0.5\cdot\Sigma(a_w\cdot\sigma_{wy})\cdot\ell_w+0.5\cdot N\cdot\ell_w \qquad (4)$$

where:

M_u =ultimate flexural strength (N·mm)

a_t =amount of cross sectinal area of longitudinal steel bars in a column under tension (mm^2)

σ_y =yield strength of longitudinal steel bars (MPa)

a_w =amount of vertical steel bars in a panel wall (mm^2)

σ_{wy}=yield strength of vertical steel bars in a panel wall (MPa)

The correlation of test results to calculated results given by Eq.(4) is shown in Fig. 4. The almost test results range from 1 to 1.2 times the calculated value. It is concluded that Eq.(4) holds enough accuracy.

4-2 Shear strength

Sixteen shear walls failed in shear shown in Fig. 3 are used in order to examine the accuracy of the following two equations which evaluate shear strength.

$$_cQ_{sc1}=\left\{\frac{0.053\cdot p_{te}^{0.23}\cdot(18+\sigma_B)}{M/Qd+0.12}+0.85\cdot\sqrt{p_{wh}\cdot\sigma_{wy}}+0.1\cdot\sigma_0\right\}\cdot b_e\cdot j \quad (5)$$

$$_cQ_{sc2}=\left\{\frac{0.0679\cdot p_{te}^{0.23}\cdot(18+\sigma_B)}{\sqrt{M/Qd+0.12}}+0.85\cdot\sqrt{p_{whe}\cdot\sigma_{wy}}+0.1\cdot\sigma_0\right\}\cdot b_e\cdot j \quad (6)$$

where:

$_cQ_{sc1}$=shear strength by Eq.(5) (N)

$_cQ_{sc2}$=shear strength by Eq.(6) (N)

M/Qd =shear-span to depth ratio [$1\leq M/Qd\leq 3$ in Eq.(5)]

p_{te} =$100\cdot a_t/(b_e\cdot d)$

b_e =equivalent width of shear walls (mm) (see Fig. 5)

σ_0 =$N/\Sigma A_w$ average axial stress (MPa)

ΣA_w =effective cross sectional area for shear strength (see Fig. 5) (mm^2)

p_{wh} =$p_w\cdot(t/b_e)$

p_w =horizontal reinforcement ratio in panel wall

p_{whe}=equivalent horizontal reinforcement ratio in panel wall, which is given by regarding longitudinal steel bars in beams and slabs as a part of horizontal reinforcement in panel wall

σ_{wy} =yield strength of horizontal steel bars in panel wall (MPa)

j =$(7/8)\cdot d$ (mm)

d =$L-b/2$ (mm)

In Eq.(6), all horizontal reinforcement in wall panel, beams and slabs are considered to be equally effective on shear strength, based on the finding in Section 6. The ratios of test results to calculated value by Eq.(5) ranged from 1.16 to 1.56, and their average and standard deviation were 1.38 and 0.129, respectively. The same results regarding to Eq.(6) were from 0.87 - 1.19 for range, 1.04 for average and 0.105 for standard deviation. Figure 6 shows the correlation of calculated values to test results for shear strength. It is concluded that Equation (5) underestimates shear strength somehow and Equation (6) gives its average.

4-3 Lateral load carrying capacity of shear walls with openings

The lateral load carrying capacities of test result were from 0.98 to 1.11 times those predicted by applying the methods of virtual work in which Eqs. (4) and (6) were used to the frame model having rigid members.

5 DEFORMATION CAPACITY

Figure 7 shows the relationships between the mean shear stress and tip drift angle of the test results of the shear walls without opening. Most shear walls failed in shear prior to flexural yielding reached their maximum lateral load carrying capacity at $R=5\text{-}10\times10^{-3}$rad., and then decreased in strength rapidly.

Shear walls failed in flexure behaved in stable manner until R reached $15\text{-}20\times10^{-3}$rad. Figure 8 shows the relationship between the approximate axial compression stress level of the boundary column (η_c) and the deformation capacity. Here the Approximate axial compression stress level was given by Eq.(7). It is obviously found that the greater η_c is, the smaller the limitation of deformation capacity is. Therefore, safety margins against not only shear failure but also this axial stress level are highly recommended in the structural design of ductile shear walls.

$$\eta_c = \frac{a_t \cdot \sigma_y + a_{wy} \cdot \sigma_{wy} + N}{b \cdot D \cdot \sigma_B + a_c \cdot \sigma_y} \qquad (7)$$

where:

a_t、a_c = amount of cross sectional area of longitudinal steel bars, in the column under tension and that under compression, respectively (mm^2)

a_{wy} = amount of cross sectional area of vertical steel bars in the panel wall (mm^2)

b、D = width and depth of the column, respectively (mm)

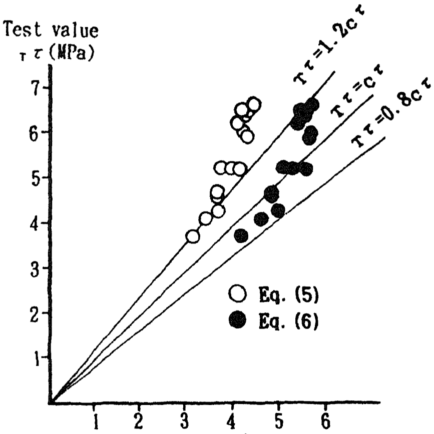


Fig. 6. Correlation of calculated values to test results for shear strength.

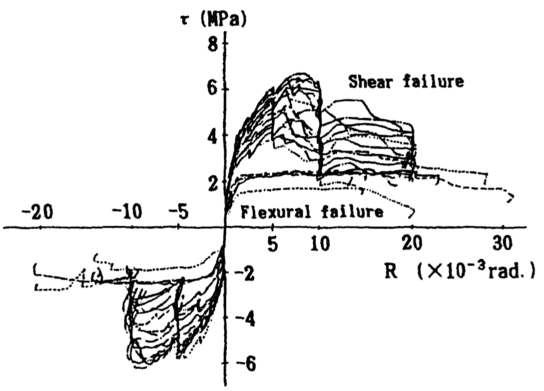
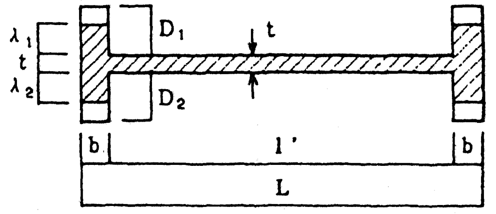


Fig. 7. Relationships between mean shear stress and tip drift angle of shear walls without opening.



$\lambda_1=D_1(\leq b)$, $\lambda_2=D_2(\leq b)$

$\Sigma A_w=l' t+(t+\lambda_1+\lambda_2)b$

$b_e=\Sigma A_w/L(\leq 1.5t)$

Fig. 5. Effective cross sectional area of boundary columns and effective thickness of shear wall on shear strength.

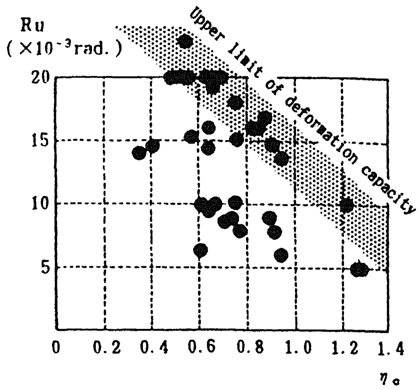


Fig. 8. Relationship between approximate axial compressive stress level of the boundary column and deformation capacity.

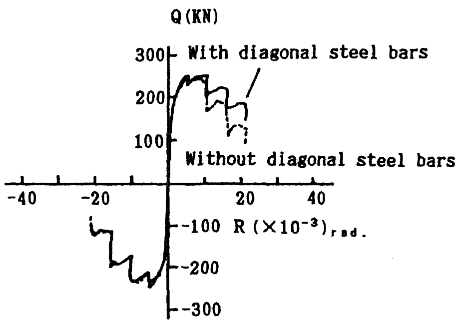


Fig. 9. Lateral load-tip drift angle relationships of G-type shear walls.

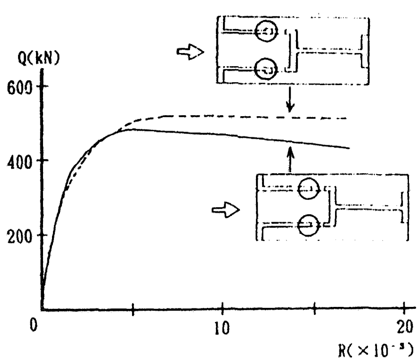


Fig. 10. Lateral load-tip drift angle relationships of F-type shear walls.

Figures 9 and 10 shows lateral load-tip drift angle relationships of shear walls with openings. Followings were concluded from these test results.

1. The diagonal steel bars in coupling beams of G-type shear wall specimen enhanced restoring force characteristics of the specimen.
2. The columns beside the opening of F-type shear walls was effective on delay of concrete crushing due to compression.

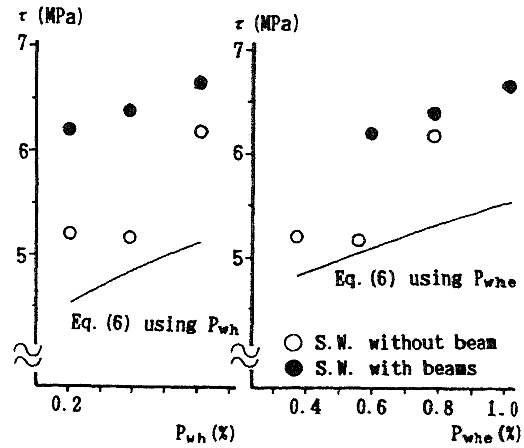


Fig. 11. Effect of longitudinal reinforcement in beams and slabs on shear strength of shear wall.

6 EFFECT OF INNER BEAMS ON STRUCTURAL PERFORMANCE

The followings were concluded from the test of shear walls without opening.

1. Inner beams enhance stiffness before yielding and shear strength of shear walls. However they do not influence other structural performance such as deformation capacity if shear walls fail in flexure.
2. Slab reinforcement parallel to the beam has same effect as longitudinal steel bars in the beam.

Figure 11 shows the effect of longitudinal reinforcement in beams and slabs on shear strength.

7 DEFORMATION OF COUPLING BEAMS OF SHEAR WALLS WITH OPENINGS

The relationships between the drift angle (Rw) of the shear wall and the deformation angle of coupling beam (R_B) of shear walls with openings were shown in Fig. 12.

In the figure, the dashed line are the values calculated by Eq.(8) which is based on the configuration of the specimen shown in Fig. 13.

It was concluded that Equation(8) gives excellent correlation to deformation angles of upper stories of the test result. This finding was confirmed by the other test results.

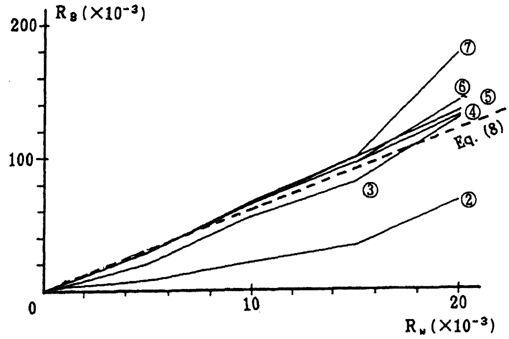


Fig. 12. Relationship between drift angle of shear wall and deformation angle of coupling beam (R_B) of shear walls with openings (G-type S.W.).
(Number in circle denotes story.)

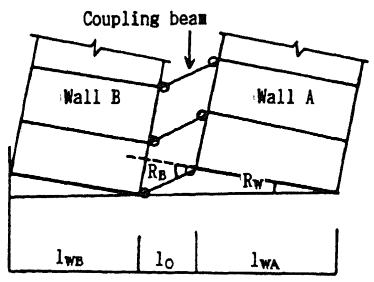


Fig. 13. Schematic relation for deformation compatibility between shear walls and coupling beam.

$$R_B = R_W \times (1 + \ell_{WA}/\ell_0) \qquad (8)$$

where:

ℓ_{WA} =wall length of compression-side shear wall (mm)

ℓ_0 =clear length of the opening (mm)

CONCLUSIONS

The following results were derived from this study.

1. The cracking strength is not sufficiently predicted by current simple equations.
2. The effect of longitudinal reinforcement in inner beams and slabs on shear strength is evaluated in the same way as of shear reinforcement in the wall.
3. The experimental flexural strength ranges from 1 to 1.2 times the analytically predicted values.
4. The shear strength is well estimated by considering the effective area of boundary columns.
5. The deformation capacity of the shear walls is significantly affected by the axial stress level of the boundary columns.
6. The upper limit of deformation angle of coupling beams is simply estimated by considering the span ratio of the beam to the wall and the drift of the wall.

ACKNOWLEDGMENT

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