Structual Characteristics of Shear Walls Unframed by Columns and Beams

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1. Introduction

The concrete shear walls of nuclear reactor building are usually designed to resist the seismic horizontal force. These walls, even though their thickness exceeds one meter, are framed by columns and beams in accordance with the conventional design practice.

It is hard to consider that there is much effect in increasing strengths of such thick shear walls by stiffening with columns and beams around them. While using shear walls unframed by columns and beams enables a rational designing and an efficient construction.

However, the effectiveness of these columns and beams on the elasto-plastic behavior of structure and the structural characteristics of unframed shear walls, such as the load-deflection relationship, the ultimate strength and the failure mode etc., have not been clarified yet in and out of Japan.

This paper reports on two kinds of tests (in-plane loading test and out-of-plane loading test) carried out to grasp the difference in the structural characteristics of shear walls framed and unframed by columns and beams.

2. In-plane Loading Test

The in-plane loading test has an objective of grasping what kind of difference exist in the structural characteristics such as the strength and the ductility between the walls framed and unframed by columns and beams subjected to an in-plane loading.

2.1 Outline of Test

The test models employed are the T-shaped cross sectional walls which simulate the stress condition of the main shear walls at the lower part of nuclear reactor building. Two models are one unframed by columns and beams (A1), and the other framed by columns and beams (A2).

According to the results of structural analysis of the building, the modelled part is a critical portion where shearing compression failure will be caused and is supposed to dominate the strength of shear walls.

The wall thickness of test models is 10cm. Model A2 is a wall latticed in 3-story and 3-span with columns and beams. The section of columns is 15cm × 15cm, and beams inside of the wall is 10cm × 14cm. The details of models are shown in Fig.1 and Table 1, and the material properties of the reinforcement and the concrete are given in Table 2.

A loading was done by applying a horizontal force repeatedly to the loading slab under the condition of a constant axial stress of 30 kg/cm^2. The loading setup is shown in Fig.2 and the loading cycles in Fig.3 respectively.

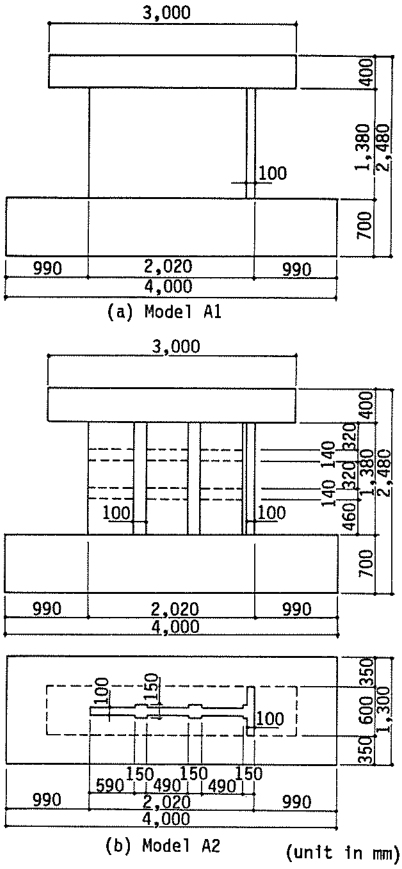


Fig. 1 Detail of model

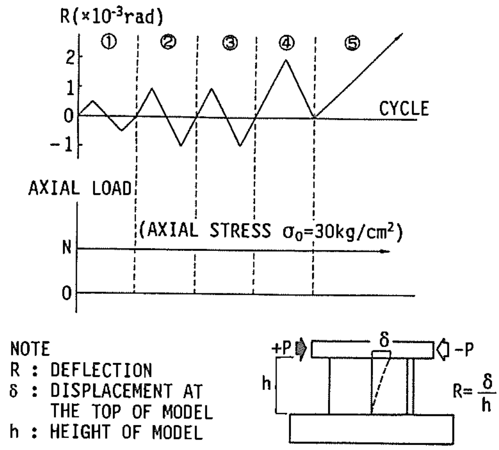


Fig. 3 Loading cycles

Table 1 Reinforcement of models

MODEL	WALL	COLUMN IN WALL		BEAM IN WALL	
	BARS	BARS	HOOPS	BARS	STIRRUPS
A1	2-D6@47 (Pg=1.36%)	–	–	–	–
A2	2-D6@47 (Pg=1.36%)	8-D6 (Pg=1.14%)	2-D3@31 (Pw=0.30%)	8-D6 (Pt=0.91%)	2-D3@70 (Pw=0.20%)

NOTE
Pg : GROSS REINFORCEMENT RATIO
Pt : TENSILE REINFORCEMENT RATIO
Pw: WEB REINFORCEMENT RATIO

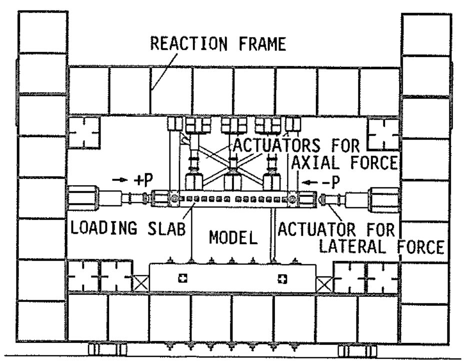


Fig. 2 Test setup

Table 2 Material properties

CONCRETE PROPERTIES	A1	A2
Fc (kg/cm^2)	299	292
Ft (kg/cm^2)	27.5	22.5
Ec (kg/cm^2)	2.92×10^5	2.80×10^5

NOTE
Fc : COMPRESSIVE STRENGTH
Ft : SPLITTING TENSILE STRENGTH
Ec : SECANT MODULUS

REBAR PROPERTIES	D3	D6
σ_y (kg/cm^2)	3230	4240
σ_t (kg/cm^2)	4740	5430
Es (kg/cm^2)	1.80×10^6	1.74×10^6

NOTE
σ_y : YIELDING STRENGTH
σ_t : TENSILE STRENGTH
Es : YOUNG'S MODULUS

2.2 Test Results

(1) Load-Defletion Relationship

The load-defletion relationships are shown in Fig.4. There was not a remarkable difference between the hysteresis loops of two models up to maximum load, except that the maximum load of Model A2 was a little higher than that of Model A1. Further more, it may be observed that the load reduction after reaching to the maximum load was greater for Model A1 compared with Model A2.

The deflections at the maximum load were 6.0×10^{-3} rad. for Model A2 and 5.4×10^{-3} rad. for Model A1 respectively.

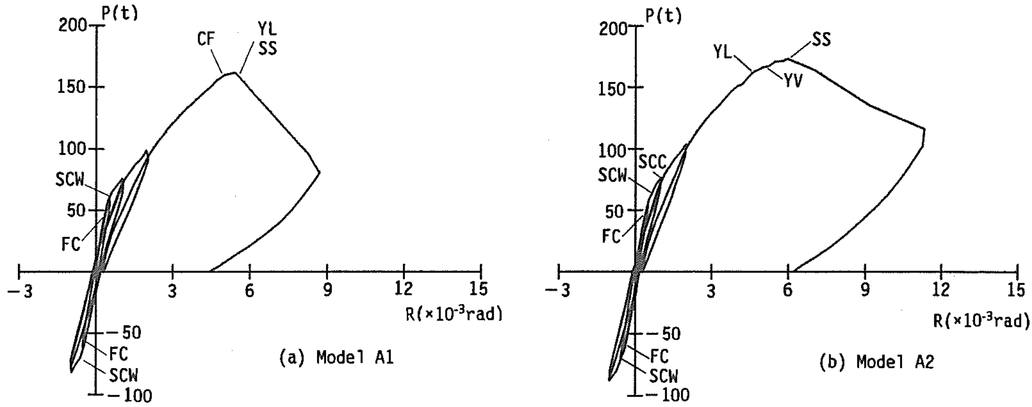


Fig. 4 Load-deflection relationship

NOTE
FC : FLEXURAL CRACK OF WALL
SCW : SHEAR CRACK AT SHEAR WALL
SCC : SHEAR CRACK AT COLUMN IN WALL
YL : YIELDING OF LONGITUDINAL BAR
YV : YIELDING OF VERTICAL BAR AT SHEAR WALL
CF : CRUSH FAILURE AT SHEAR WALL
SS : SHEAR SLIDING FAILURE AT SHEAR WALL

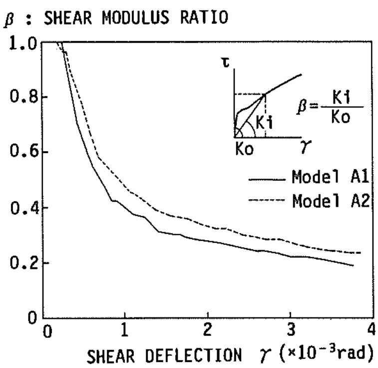
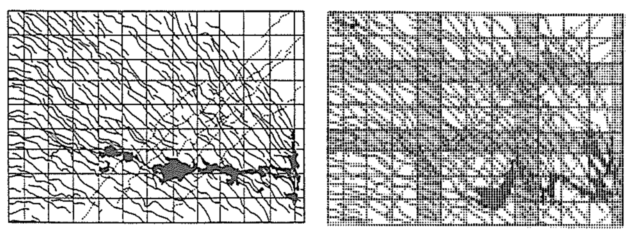


Fig. 6 Deterioration of stiffness



(a) Model A1 (b) Model A2

Fig. 5 Crack pattern at final stage

(2) Crack Pattern and Failure Mode

The final crack patterns of the two models are shown in Fig.5. There was no difference in the failure processes of both models, and the both ultimate failure modes were sliding shear failure triggered by shearing compression failure.

Some spalling of concrete surface occurred after reaching to the maximum load were slightly more prominent for Model A1 compared with Model A2. And the progress of crack tended to be weakened by columns.

Model A2 was cut to observe the detailed conditions inside the walls and columns. As a result, shear cracks could be seen not only in walls but also in columns. And still more local shear crushing had occurred in columns.

(3) Stiffness and Strength

The deterioration of shear stiffness is shown in Fig.6. The shear stiffness was reduced significantly in both models with a progress of shear cracks. However, the ratio of deterioration was lower for Model A2 than for Model A1, and it follows that the columns and beams acted to restrain the deterioration of shear stiffness.

The maximum strengths were 161.5 ton for Model A1 and 173.1 ton for Model A2 respectively, and the strength ratio of two models (A1/A2) was 0.93. This is roughly the same value as the ratio of horizontal cross-sectional area between Model A1 ($2020cm^2$) and Model A2 ($2245cm^2$).

The comparison of maximum shear strength between tested and calculated values using equation 1) (Hirosawa, 1975) and equation 2) (Sotomura et al, 1985) given below is shown in Table 3. The tested values of both models coincide well with those calculated by equation 2), provided that the wall cross-sectional area of Model A2 includes the column cross-sectional area.

The ratios of the calculated strength of Model A1 to that of Model A2 are 0.94 by equation 1) and 0.93 by equation 2) respectively. They are approximately equal to the tested result.

Table 3 Comparison of maximum shear strength

MODEL	TEST	CALCULATION			
	Pu (t)	cQ1 (t)	Pu / cQ1	cQ2 (t)	Pu / cQ2
A1	161.5	103	1.57	150	1.08
A2	173.1	109	1.59	162	1.07
A1 / A2	0.93	0.94	–	0.93	–

Equation 1)

$$cQ_1 = \left\{ \frac{0.0679\, p_{te}^{\,0.23} (F_c + 180)}{\sqrt{M/(QD) + 0.115}} + 2.7\sqrt{\sigma_{wh} \cdot p_{wh}} + 0.1\,\sigma_0 \right\} b_e \cdot j$$

Equation 2)

$$cQ_2 = \tau_u \cdot A_w$$

$$\tau_u = (1 - \tau_s / 4.5\sqrt{F_c})\,\tau_o + \tau_s$$

$$\begin{cases} \tau_o = (3 - 1.8\,M/QD)\sqrt{F_c} \\ \tau_s = (p_v + p_h)\,\sigma_y/2 + (\sigma_v + \sigma_h)/2 \quad (\tau_s < 4.5\sqrt{F_c}) \end{cases}$$

2.3 Evaluation

It is found that there are effects that the deterioration of shear stiffness is slightly restrained and the strength increases corresponding to the column cross-sectional area increment when shear walls are framed by columns and beams. However, even if columns and beams are eliminated, the structural characteristics of shear walls are not affected to the extent that the consideration should be necessary in the design. The shear strength of walls unframed by columns and beams can be estimated well by using conventionally-used calculation formulae.

3. Out-of-plane Loading Test

The out-of-plane loading test has the the objective of confirm that the joint portion of unframed shear wall has an adequate deformation capacity when it is subjected to the shear force and the bending moment perpendicular to the wall plane.

3.1 Outline of Test

Three models prepared for the test are Model (BJ1) representing an interior joint portion of beams and slabs connecting to the inner wall, and two models (BT1 & BT2) representing an exterior joint portions of a beam and slab connecting to the outer wall. The difference between Model BT1 and Model BT2 is whether or not there are the reinforcement at the joint portion to resist against an out-of-plane shear stress.

The detail of Model BJ1 is shown in Fig. 7, and the material properties of the reinforcement and the concrete are given in Table 4.

A loading was done alternately in the vertical direction at the both ends of beams, preventing the wall from heaving up and being the top and the bottom ends of wall pin or roller supported. The loading setup is shown in Fig. 8 and the loading cycles in fig. 9 respectively.

3.2 Test Results

The load-deflection relationships are shown in Fig. 10. Although the yielding of slab reinforcement occurred in all the models by the deflection of 6×10^{-3} rad., no remarkable change in each load-deflection relationship could be seen. The strains of wall reinforcement were still in the elastic range.

Subsequently, the load reduction according to the crack progress in the wall occurred at the deflection of 7×10^{-3} rad. in Model BT1, whereas it was not recognized in the other models up to the maximum loading. The deflection at the maximum load was $18\sim20\times10^{-3}$ rad. for all models.

The ultimate failure modes were punching failure of the wall for Model BT1, and in contrast, bending failure of beam and slab for Model BT2, and shearing failure of joint portion after yielding of beam reinforcement for Model BJ1.

All three models were cut to observe detailed conditions inside the wall and joint portion. As a result, there were less cracks inside the wall of Model BT2 compared with Model BT1, and it could be appraised that the wall of Model BT2 was almost sound up to the ultimate failure. It is clarified that the reinforcement for out-of-plane shear stress are fairly effective.

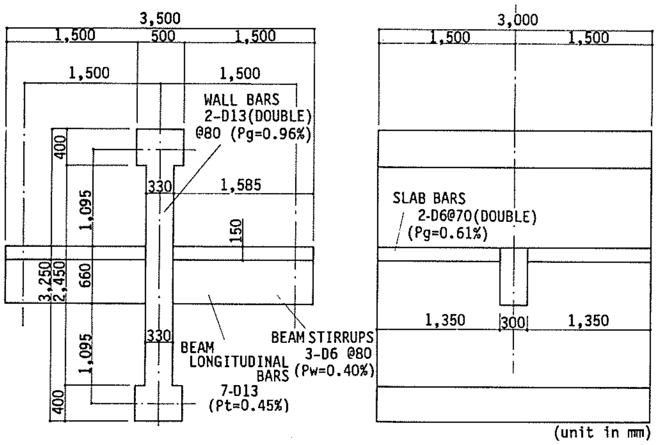


Fig. 7 Detail of model

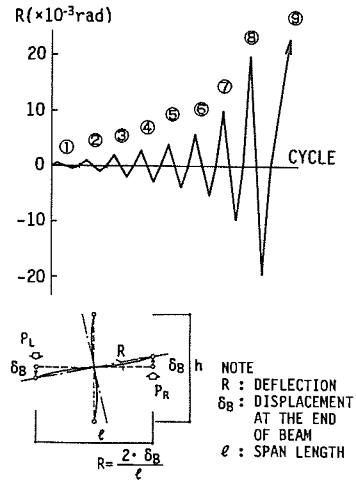


Fig. 9 Loading cycles

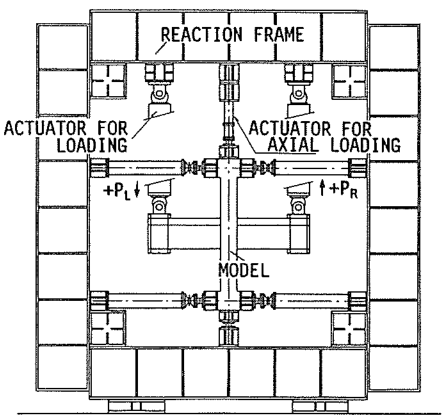


Fig. 8 Test setup

Table 4 Material properties

CONCRETE PROPERTIES	BJ1	BT1	BT2
Fc (kg/cm^2)	319	328	334
Ft (kg/cm^2)	26.1	22.9	26.1
Ec (kg/cm^2)	3.03×10^5	3.04×10^5	2.93×10^5

REBAR PROPERTIES	BJ1 BT2		BT1	
	D6	D13	D6	D13
σ_y (kg/cm^2)	4080	4010	4270	4200
σ_t (kg/cm^2)	5360	5560	5360	5800
Es (kg/cm^2)	1.93×10^6	1.90×10^6	2.07×10^6	2.09×10^6

3.3 Evaluation

In a nuclear reactor building, the shear walls are the main aseismatic elements, so that the frame members should be able to follow the deflection of shear walls. At the deflection of 6×10^{-3} rad. which was measured at the maximum load in in-plane loading test, it was ascertained that the damage to all the models in out-of-plane loading test occurred slightly, and walls and wall-beam joint portions were sound in all the models.

Consequently, the shear wall unframed by beams and columns possesses an adequate deformation capacity against an out-of-plane loading. Further, concerning the exterior joints, a deformation capacity can be improved by providing the reinforcement for the shear stress at the wall-beam joint portion.

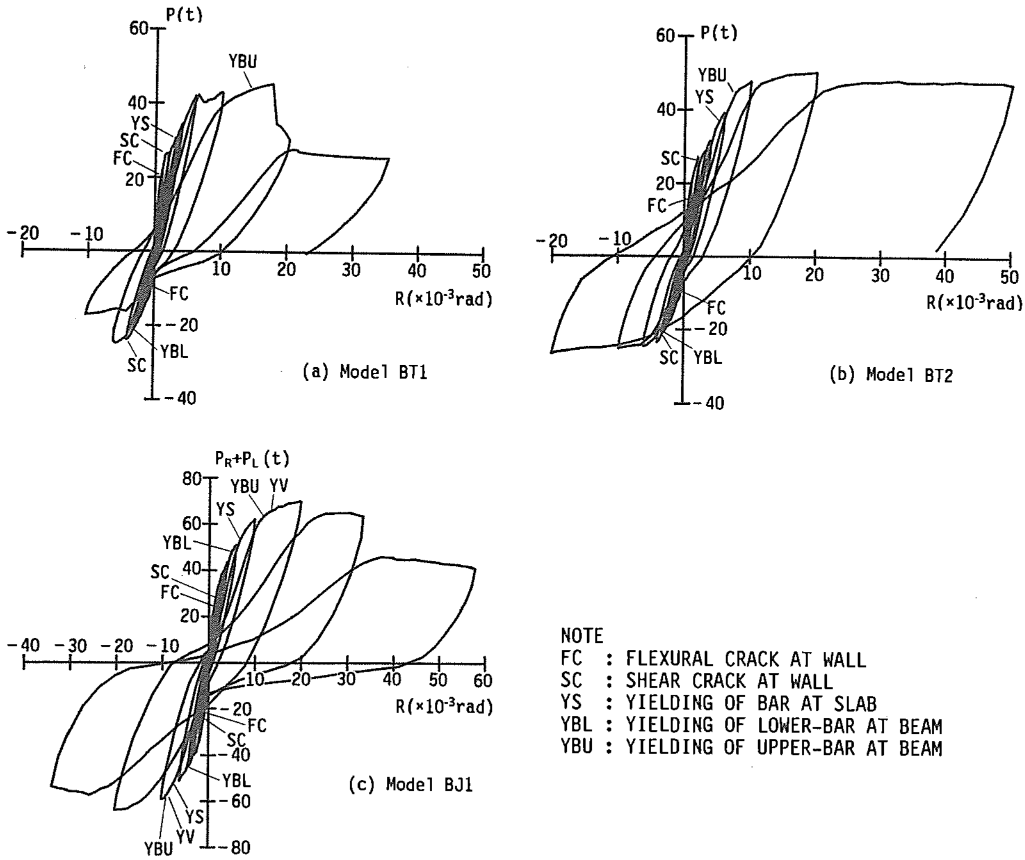


Fig. 10 Load-deflection relationship

4. Conclusions

As a result of these tests, the followings are confirmed. Even if the columns and beams, the ratios of whose dimensions to the wall thickness are not larger than those of these tests, are eliminated, the structural characteristics of shear walls are not greatly different from those of the conventional framed shear walls, and there are an adequate shear strength and a deformation capacity.

5. Acknowledgment

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6. Reference

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