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STEEL PANEL SHEAR WALLS, WITH AND WITHOUT CONCRETE COVERING

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ABSTRACT

This paper presents the experimental and analytical researches on the elasto-plastic deformation behaviors of steel panel shear walls with and without concrete covering under monotonous as well as repeated cyclic loadings. Tests are carried out on the specimens with various shear span ratios of surrounding frames and with various thicknesses of steel panel and concrete covering, and the test results show, for the specimens without concrete covering, that the initial stiffness is not so high but the ductility is very large by a formation of the diagonal tension field of steel panel. For the specimens with concrete covering, the test results show very high initial stiffness and very high maximum resistance by a diagonal compression field of concrete . And under the assumption of a simple diagonal steel tension field and concrete compression field with a certain equivalent effective width Be, the deformation processes from the initial loading to the ultimate state are computed. The computed results coincide fairly well with the test results under monotonous and cyclic loadings.

KEYWORDS

Steel panel shear wall ; concrete covering ; buckling wave ; shear crack ; diagonal tension field ; diagonal compression field ; effective width ; sufficient ductility

INTRODUCTION

As aseismic panel element there are two kind of panel, reinforced concrete shear wall and steel shear panel. Reinforced concrete shear wall is one of the best aseismic element for reinforced concrete and composite structures. It shows after the formation of initial shear crack of concrete wall at a story sway angle of 0.001-0.002, the stiffness gradually decrease and many cracks spread out and the maximum resistance is achieved by a diagonal compression resistance of concrete at a very small story sway angle R of 0.005-0.006. After that, very brittle compressive fracture of concrete occurs, and the resistance is falling down. On the contrary, steel panel shear wall buckles by a diagonal compressive force at a lower loading stage and shows very large plastic deformation by a diagonal tension field of steel panel formed along the buckled waves. Tests are carried out on the steel panel shear walls with and without concrete covering, with various shear span ratios of surrounding frame, various steel panel thicknesses and various concrete wall thicknesses under monotonous and cyclic shear loadings. The combination of both aseismic panel elements may have more effective characteristics of aseismic resistance.

TEST

Testing Method and Test Specimens

Loading method, measuring method and test specimen are illustrated in Fig.1, and the series of shear span ratio is illustrated in Fig.2, and the test specimens are classified into five test series such as shown in Table 1. Tests are carried out on 1/10 scale models of composite frames with various spans and a height of 30cm infilled with various shear walls under monotonous and cyclic loadings. Surrounding composite frames are composed of a cross section of 40 × 40 × 2.1mm encased in a reinforced concrete cross section of 60 × 60mm with longitudinal reinforcements of 2- ϕ 3mm × 2 and ϕ 2mm hoops and stirrups with 20mm pitches such as shown in Fig.1. Story sway displacements δ of both sides of the panel are measured by dial gauges of 1/100mm. The mechanical properties of materials are shown in Tables 2, 3.

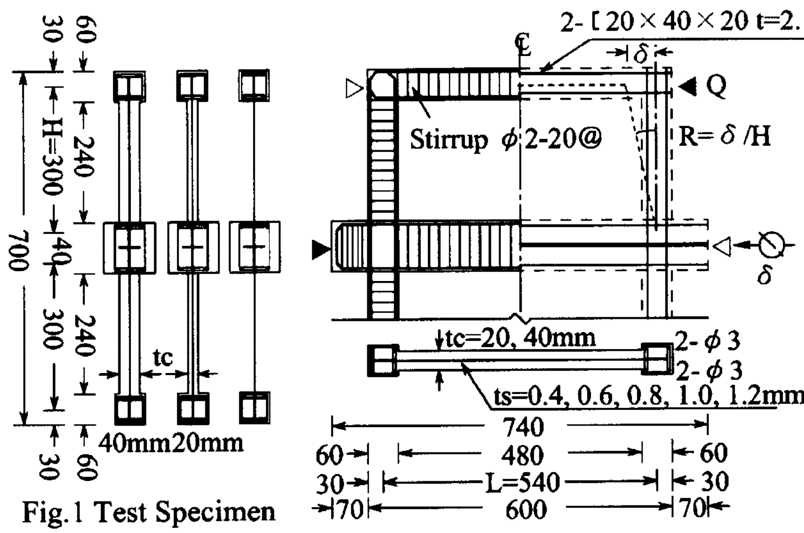

Fig.1 Test Specimen

Table 1 TEST Series

Test Series	Specimens	Shear Span Ratio H : L	Wall Thickness ts(mm)	tc(mm)	Loading	Covering
	SWR10	1:1.0	0.6	-		
SWR	SWR15	1:1.5	0.6	-		
	SWR18	1:1.8	0.6	-		
	SWR20	1:2.0	0.6	-	Monotonous	
	SWT04	1:1.8	0.4	-	Loading	
	SWT06	1:1.8	0.6	-		without
SWT	SWT08	1:1.8	0.8	-		Concrete
	SWT10	1:1.8	1.0	-		Covering
	SWT12	1:1.8	1.2	-		
	SWT04CYC	1:1.8	0.4	-		
	SWT06CYC	1:1.8	0.6	-	Cyclic	
SWTCYC	SWT08CYC	1:1.8	0.8	-	Loading	
	SWT10CYC	1:1.8	1.0	-		
	SWT12CYC	1:1.8	1.2	-		
	SCWR10	1:1.0	0.6	20		
	SCWR15	1:1.5	0.6	20		
SWRCW2	SCWR18	1:1.8	0.6	20		
	SCWR20	1:2.0	0.6	20		with
	SCWR30	1:3.0	0.6	20	Monotonous	Concrete
SWCW2	SW04CW2	1:1.8	0.4	20	Loading	Covering
	SW06CW2	1:1.8	0.6	20		
SWCW4	SW04CW4	1:1.8	0.4	40		
	SW06CW4	1:1.8	0.6	40		

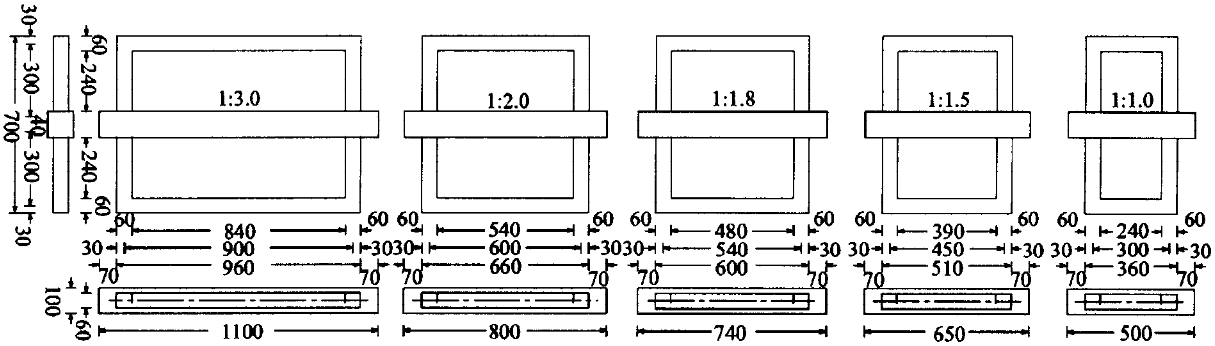

Fig.2 Shear Span Ratio Series (SWR, SWRCW2)

Table 2 Mechanical Properties of Concrete

Test Series	Fc(MPa)	Ft(MPa)
SWR	16.2	1.9
SWT	25.9	2.5
SWRCW2	34.0	3.1
SW04CW2,4	41.1	1.6
SW06CW2,4	42.0	1.1
SWT04CYC	40.5	2.4
SWT06CYC	42.0	2.7
SWT08CYC	36.0	2.5
SWT10CYC	38.3	3.4
SWT12CYC	34.3	2.2

Table 3 Mechanical Properties of Steel

Plate (mm)	σ y(MPa)	σ max(MPa)	ε max(%)
0.4	238	335	42
0.6	218	340	33
0.8	229	343	40
1.0	230	344	46
1.2	234	354	37
2.1	381	480	30
2.3	293	381	24
ϕ3	231	346	–

Test Results

Relationships between shear force (Q) and story sway angle (R) are shown in Fig.3-(a) for SWT series, in Fig.4-(a) for SWR series (Yamada, 1993a, 1993b), in Figs.5-(a), 6-(a) for SWTCYC series (Yamada and Sugii, 1995a), in Figs.7-(a), (a') for SWRCW2 series (Yamada and Sugii et al, 1994a, 1994b), in Figs.8-(a), 9-(a) for SWCW series (Yamada and Sugii, 1995b). The characteristic values of test results of the specimens with concrete covering are indicated in Table 4, where SC indicates the first diagonal shear crack, CC the formation of compressive crack of concrete, MAX the maximum resistance of the shear wall.

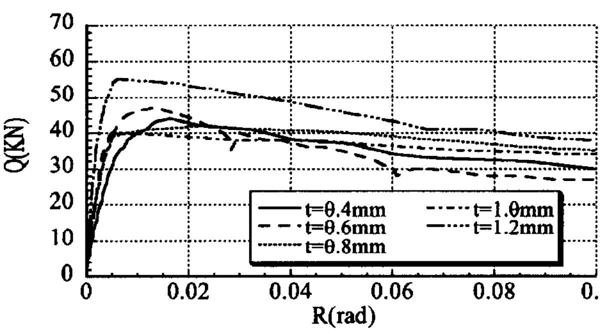

Fig.3-(a) Q-R Relationships of SWT Series (Tested)

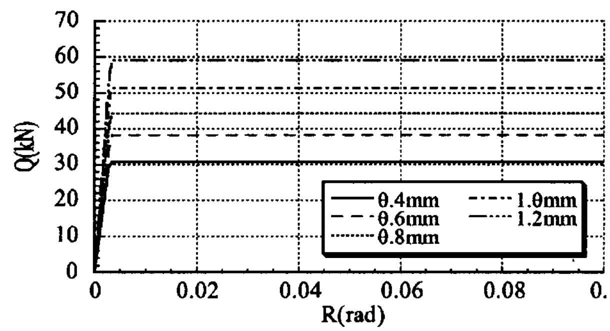

Fig.3-(b) Q-R Relationships of SWT Series(Computed)

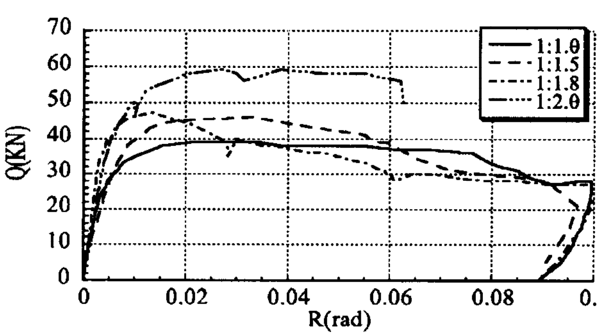

Fig.4-(a) Q-R Relationships of SWR Series (Tested)

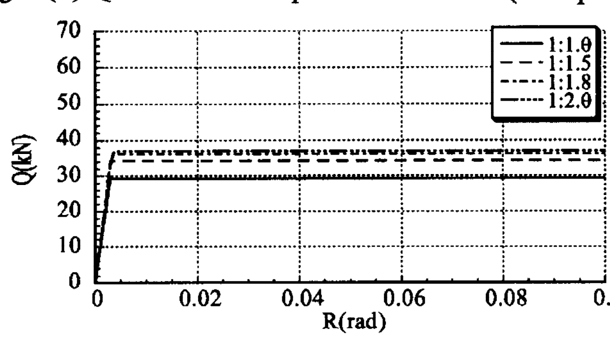

Fig.4-(b) Q-R Relationships of SWR Series(Computed)

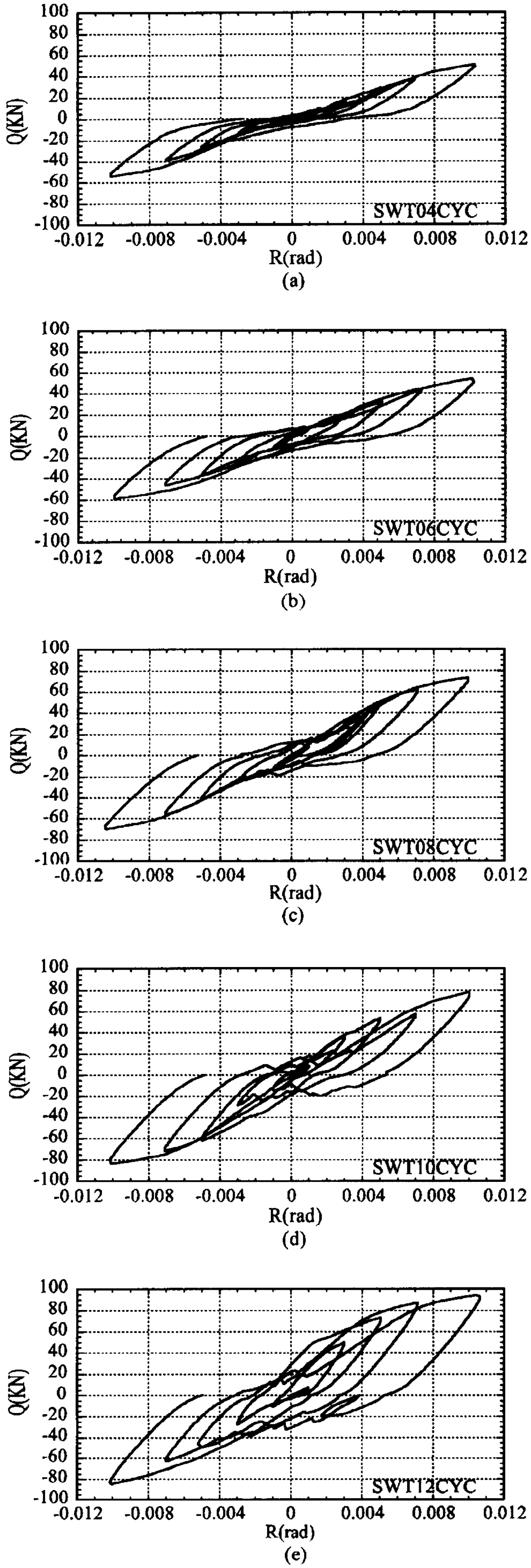

Fig.5-(a) Q-R Relationships (Tested)

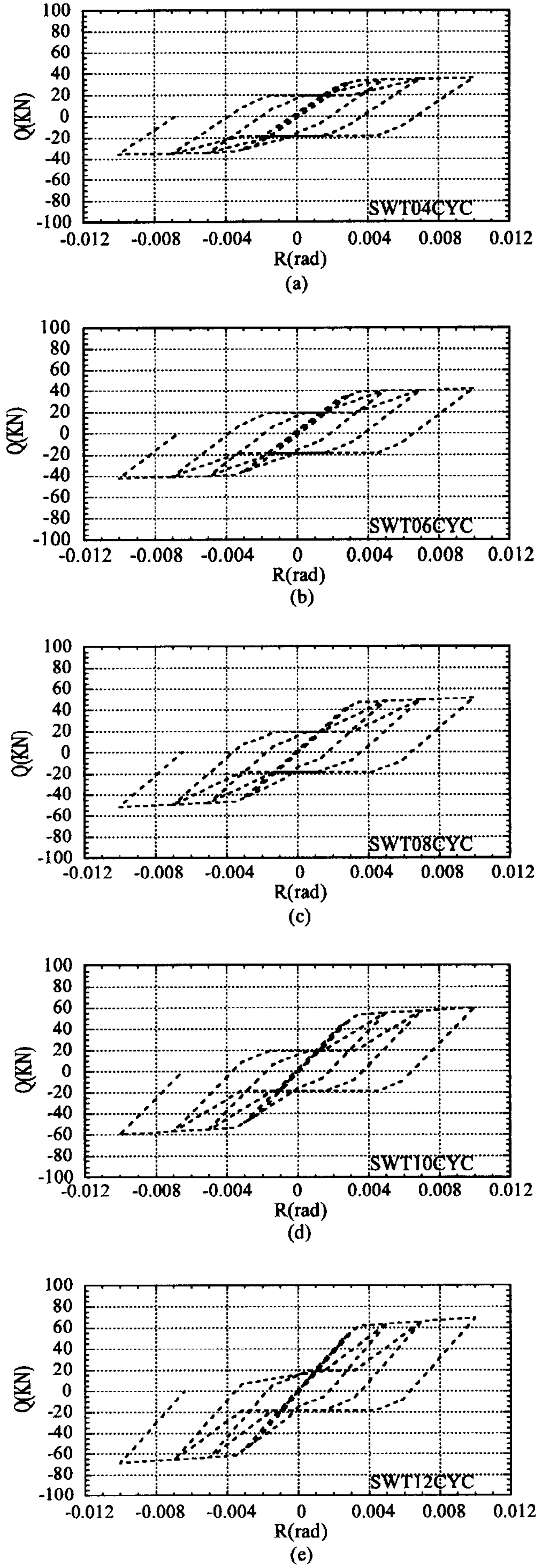

Fig.5-(b) Q-R Relationships (Computed)

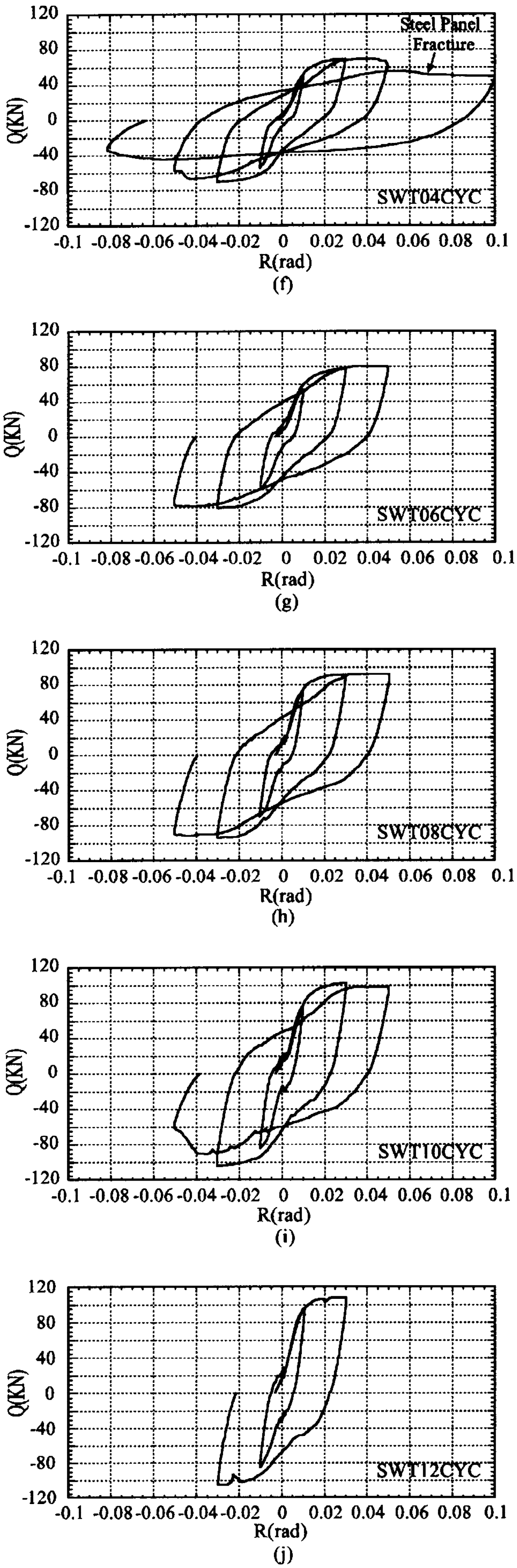

Fig.6-(a)Q-R Relationships (Tested)

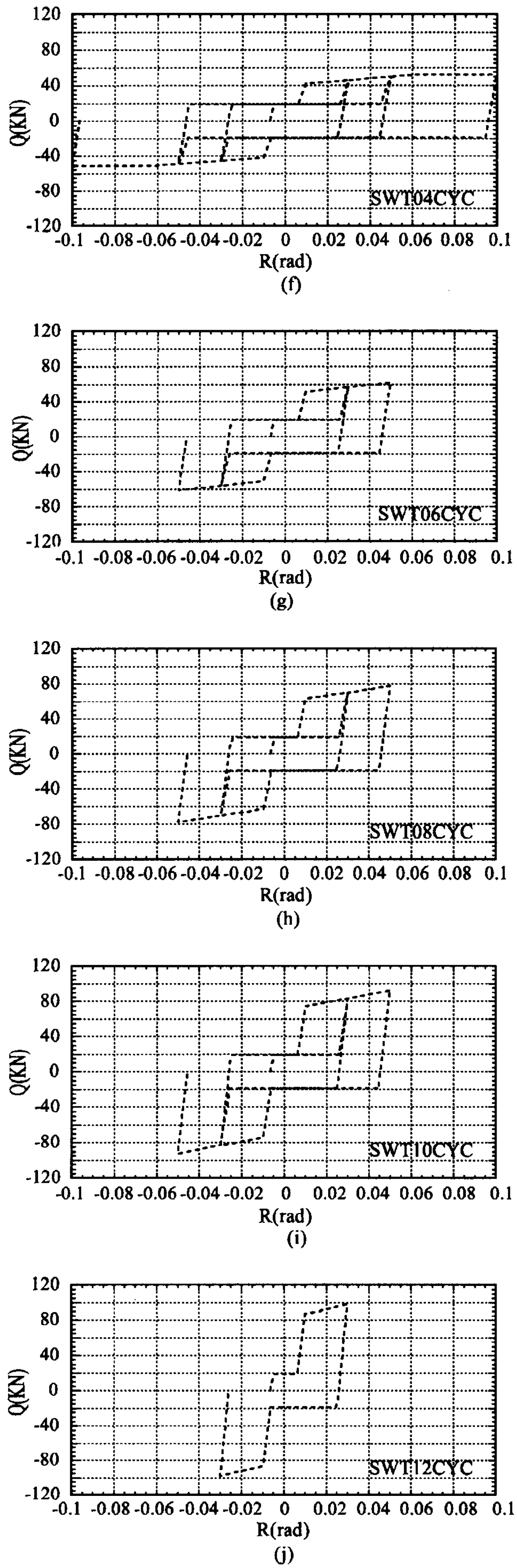

Fig.6-(b) Q-R Relationships (Computed)

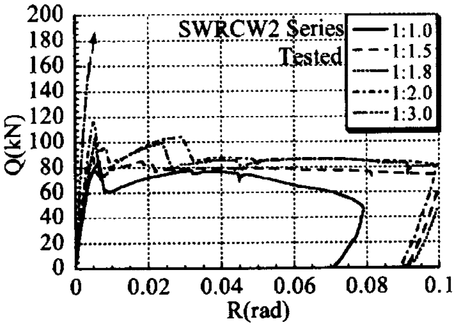

Fig.7-(a) Q-R Relationships

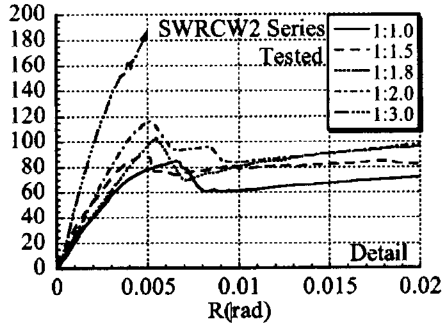

Fig.7-(a') Q-R Relationships

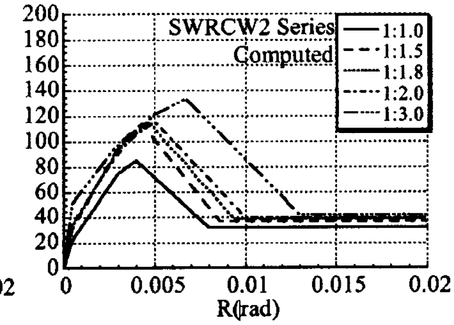

Fig.7-(b') Q-R Relationships

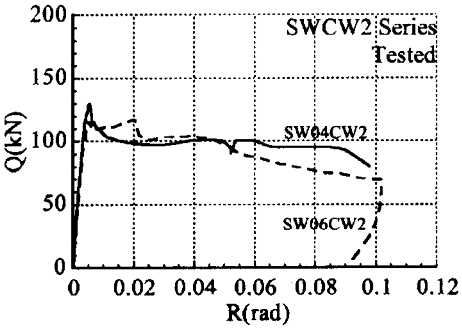

Fig.8-(a) Q-R Relationships

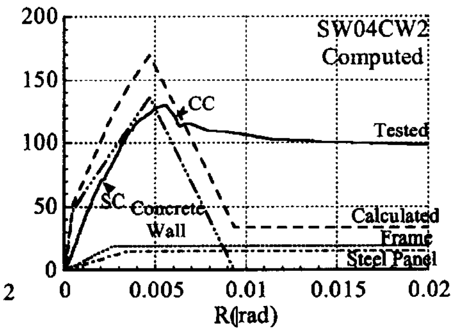

Fig.8-(b)Q-R Relationships

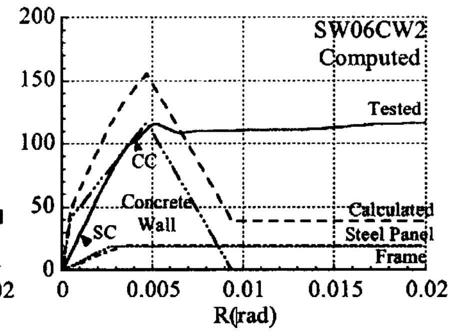

Fig.8-(c) Q-R Relationships

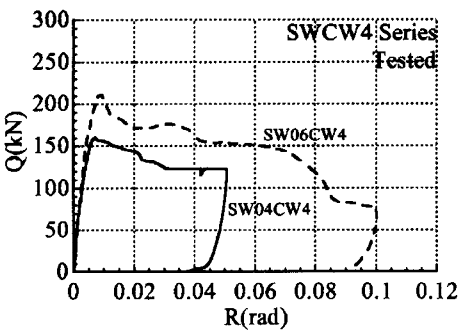

Fig.9-(a) Q-R Relationships

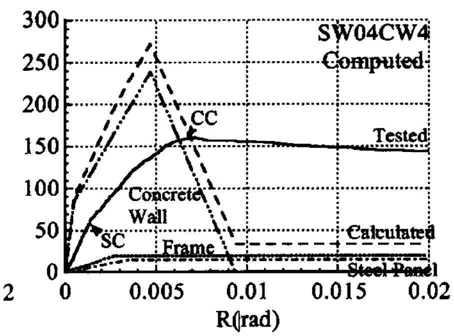

Fig.9-(b) Q-R Relationships

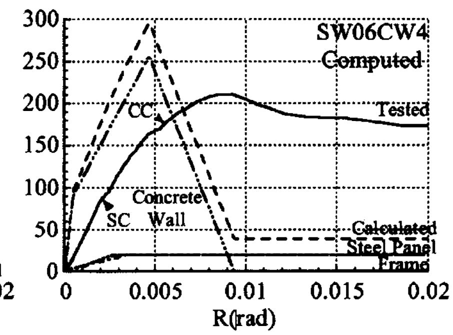

Fig.9-(c)Q-R Relationships

Table 4 Test Results

	SC		CC		MAX	
Specimens	Q(kN)	R(rad)	Q(kN)	R(rad)	Q(kN)	R(rad)
SWR10CW2	31.0	0.0015	82.0	0.0058	85.0	0.0066
SWR15CW2	36.0	0.0012	72.0	0.0031	89.3	0.0048
SWR18CW2	40.0	0.0019	101.0	0.0047	103.0	0.0059
SWR20CW2	47.5	0.0018	107.0	0.0044	116.3	0.0051
SWR30CW2	43.5	0.0010	120.0	0.0026	168.0	0.0045
SW04CW2	72.0	0.0021	120.0	0.0063	130.0	0.0057
SW06CW2	29.0	0.0010	102.5	0.0041	117.0	0.0204
SW04CW4	66.0	0.0015	159.0	0.0066	160.0	0.0072
SW06CW4	87.5	0.0021	172.5	0.0054	210.5	0.0092

ANALYSIS

In this paper, the resisting mechanisms of a shear walls are replaced with two resisting elements, i.e. the compression brace of concrete covering and the tension brace of thin steel panel. Stress-strain relationships of the concrete and steel are assumed such as shown in Figs.10, 11-(a), (b) and the moment-curvature relationship of surrounding frame is assumed in Fig12, Q-R relationship of steel panel under cyclic loading is assumed in Fig.13. The computed results are shown in Figs.3-(b), 4-(b), 5-(b), 6-(b), 7-(b'), 8-(b), (c), 9-(b), (c).

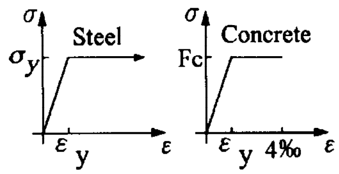

Fig.10 Surrounding Frame

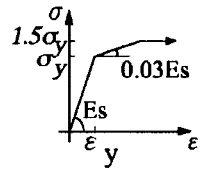

Fig.11-(a) Steel Brace

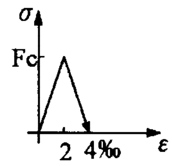

Fig.11-(b) Concrete Brace

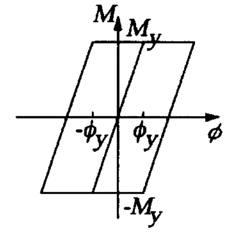

Fig.12 M- ϕ Relationship of Surrounding Frame

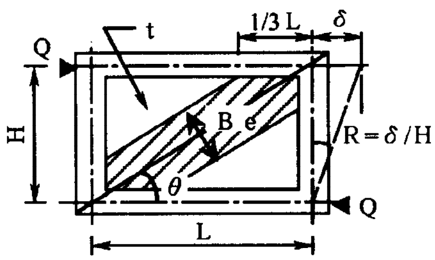

Fig.14 Steel Tension Brace

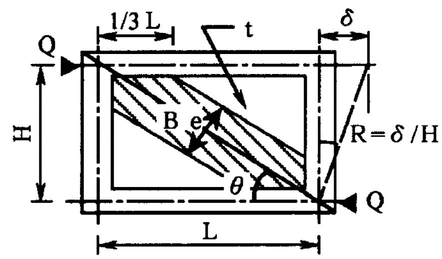

Fig.15 Concrete Compression Brace

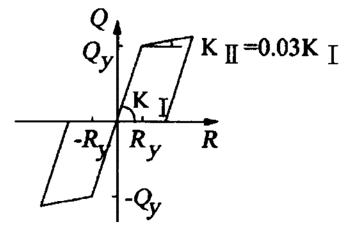

Fig.13 Q-R Relationship of Steel Panel

Assumption of Shear Walls

The resistance of steel panel is assumed to be a diagonal tension brace with a certain equivalent effective width Be (=2/3L sin θ) such as shown in Fig.14 (Yamada, 1992) :

$$s\,Q_y = s\,\sigma_y \cdot B_e \cdot t \cdot \cos\theta = \frac{2}{3}\, s\,\sigma_y \cdot L \cdot t \cdot \sin\theta \cos\theta \tag{1}$$

$$s\,R_y = \frac{s\,\varepsilon_y}{\sin\theta \cos\theta} \tag{2}$$

where $s\sigma_y$, $s\varepsilon_y$ is a yield stress and yield strain of a steel panel. For the range of R $\geqq$ 0.01, the value of Be=L sin θ is assumed.

The resistance of concrete wall is assumed to be a diagonal compression brace with a certain equivalent effective width Be such as shown in Fig.15 (Yamada, 1992). The concrete walls are assumed to resist by pure shear of panel until initial crack occurs :

$$Q_{cr} = \tau_{cr} \cdot L \cdot t = \frac{Fc}{10} \cdot L \cdot t \tag{3}$$

$$R_{cr} = \tau_{cr} \cdot \frac{1}{Gc} = \frac{Fc}{10} \cdot \frac{1}{Gc} \tag{4}$$

where Gc=Ec/ 2(1+ ν). And it is assumed that the maximum resistance is achieved when the compressive stress of the equivalent concrete brace reach the compressive strength of concrete:

$$Q_u = Fc \cdot Be \cdot t \cdot \cos\theta = \frac{2}{3} Fc \cdot L \cdot t \cdot \sin\theta \cos\theta \tag{5}$$

$$R_u = \frac{c\varepsilon_{nax}}{\sin\theta\cos\theta} \tag{6}$$

where $c\varepsilon_{nax}$ is the strain of concrete at the maximum resistance Fc (see Fig.11-(b)).

DISCUSSION

The maximum resistance of steel panel shear wall without concrete covering i.e. SWR, SWT series, is achieved at a story sway angle R of 0.02-0.03 ,and only a slight reduction in resistance after the maximum resistance is observed such as shown in Figs.3-(a), 4-(a). The steel panel shear wall with concrete covering , i.e. SWRCW2, SWCW2, SWCW4 series, show at first very high initial stiffness and very high maximum resistance at a very small story sway angle R of only 0.005-0.006 by the diagonal compression field of concrete. Under cyclic loading, the specimen of SWTCYC series with the thinner panel shows the hysteresis loop with slipping. The primary difference in hysteresis behavior is attributed to the thickness of the steel panel. The maximum resistance is achieved at a story sway angle R of nearly $\pm$ 0.03 for all thickness of steel panel. For the specimen with the steel panel thickness of 0.4mm, the steel panel fracture occurred at a story sway angle R of 0.07 near the bottom of column. The computed results coincide fairly well with the test results quantitatively.

CONCLUDING REMARKS

Experimental and analytical researches on the steel panel shear walls with and without concrete covering under monotonous and cyclic shear loadings are conducted. As the results, a diagonal tension field of steel panel is formed along the buckling wave after the first buckling, and the steel panel show very high stable ductility with no reduction of resistance (see Figs. 3-(a), 4-(a)). The specimens with concrete covering show very high initial stiffness and very high resistance by a diagonal compression field of concrete which is formed in the opposite direction to the tension field of steel panel (see Figs.7-(a), (a'), 8-(a), 9-(a)). Under cyclic loading, steel panel shear walls with the thinner panel show hysteresis loops with the more slipping, and the effect of the strain hardening of steel panel is very large. Under the assumptions of a simple diagonal tension field of steel panel and compression field of concrete with a certain equivalent effective width Be, the computed results coincide fairy well with the test results under monotonous and cyclic loadings (see Figs.3-(b), 4-(b), 5-(b), 6-(b), 7-(b'), 8-(b), (c), 9-(b), (c)). The optimum selection of the thicknesses of steel panel and concrete covering may be able to control the resisting behaviors of structures.

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