(22) STRENGTH AND DEFORMABILITY OF FRAME COMPOSED OF STEEL TUBE JACKETING RC SHORT COLUMN AND SPANDREL WALL

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This paper proposes a new constructional frame which is supposed to be use as perimeter frame in Double Tubes Hybrid System (DTHS) where the RC core walls are designed as the Energy Dissipation Structural Walls (EDSW). The EDSW is composed of RC coupled shear walls linked by short steel H-shaped beams as the energy dissipation devices. The hysteretic behavior of EDSW as the core tube has been experimentally investigated and reported elsewhere. It is clarified that EDSW behave in a very ductile manner under cyclic lateral loading, and have an ample energy dissipation capacity.

The frame for perimeter frame is designed as a sub-assemblage of the two stories and single bay of the perimeter frame of DTHS is composed of two rigid spandrel wall and two RC short columns. With respect to the RC short columns, we introduce steel Tube jacketing Reinforced Concrete (TRC) columns instead of the ordinary RC columns. We call such this new constructional frame as the Spandrel Wall Frame (SWF). In order to establish a reliable performance-based seismic design method for DTHS, the strength and deformability of two new constructional frames were investigated under cyclic lateral loading test.

The study was programmed to investigate the strength and deformability of SWF specimens under horizontal cyclic loads. The loading programs of cyclic lateral loading test were used as parameter study. The cyclic loading program for SWF-1 increases the amplitudes of *R* in a step wise until 2.0/100 radian in fourteen cycles whereas the SWF-2 was cyclically loaded with a constant large amplitude of R = 3.0/100 radian in eight cycles. The elasto-plastic behavior of the SWF showed sufficient deformability and horizontal force carrying capacity as the exterior tube of DTHS buildings.

Key Words : RC spandrel wall, RC short column, steel tube jacketing, elasto-plastic behavior, column yield mechanism

1. INTRODUCTION

A new constructional frame was proposed as a part of research on Seismic Design Method for the Double Tubes Hybrid System (DTHS) for buildings. Fig.1 shows the isometric of the floor plan of the DTHS. As shown in the figure this structural system consists of RC core walls as the interior tube and the perimeter frames as the exterior tube. The interaction mechanism between the interior and exterior tubes improves an overall lateral force resistance of the building system. The interior tube is the Energy Dissipation Structural Walls (EDSW). The investigations on the EDSW already have been done experimentally and analytically¹⁾⁻⁵⁾. Fig.1 shows the plastic collapse mechanism of a building with the EDSW which prevents the perimeter frames from the story mechanism at any single story under earthquake excitations.

Under the condition that the EDSW is installed in



Fig.1 Isometric of the floor plan of DTHS.

the building, we introduced the Spandrel Wall Frame (SWF) system as the exterior frame. The SWF is composed of RC spandrel walls and RC short columns as shown in Fig.3. The columns are the steel Tube jacketing Reinforced Concrete (TRC) columns which are experimentally proved to have the extraordinary deformability⁶. In the plastic collapse mechanism of the SWF, the TRCs yield at the both ends as shown in Fig.3(b). As mentioned before, the overall deformations in the whole building are guaranteed by the interior tube made of the EDSW.

As the TRC has the high shearing strength, a brittle shear failure hardly occurs. The shear span ratio of the TRC becomes smaller, however, very short columns may fail in shear, especially after the flexural rotation increases in the plastic hinge area. Therefore, it should be important to investigate whether the TRC short column can avoid such a brittle failure. The purpose of this paper is to demonstrate the stable elasto-plastic behavior of the SWFs composed of TRC short columns and spandrel walls which are supposed to be used in the DTHS building.

2. EXPERIMENTAL PROGRAMS

2.1 Test Specimens

The experiment was programmed to investigate the elasto-plastic behaviors of the Spandrel Wall Frame (SWF). The test specimen was designed as a sub-assemblage of the two stories and single bay of the SWF, whose proportion was based on the DTHS prototype building in reference 7). Fig.4 shows the feature of SWF test specimens which are composed of two massive spandrel walls of 250x810x1850 mm



Fig.2 EDSW as interior tube (a) and its collapse mechanism (b)



Fig.3 SWF as exterior tube (a) and its collapse mechanism (b)

and two RC short columns encased in square steel tubes of 250x250x6 mm. Therefore, the thickness of spandrel walls is same as the width of TRC columns. The frame was connected to the foundation beam through very short of RC columns of 10mm in length. The concrete of foundation beam was casted at first with remaining stacks of steel bars for columns. After that, the steel bars for the frame were arranged



Fig.4 SWF test specimen



Fig.5 Detail of SWF test specimen

by welded to the stacks, and then concrete for the frame was casted. Deformed bars with a nominal diameter of 13mm (D13) were used as longitudinal reinforcements for TRC column and its longitudinal reinforcement ratio p_g was 0.89% (4-D13). Moreover, 5-D16 (p_i =0.49%) was utilized for main reinforcement of spandrel wall and D10@100 with p_w of 0.57% was utilized for the stirrups of the spandrel wall.

Fig.5 shows the detail of the SWF test specimen. At the interface of a steel tube of TRC to lower spandrel

Table 1 Mechanical properties of concrete material

Test specimens	F_c (N/mm ²)	σ_B (N/mm ²)	E_c (kN/mm ²)	<i>Slump</i> (cm)	Air content (%)
SWF-1	27	30.0	28.9	20.3	5.9
SWF-2	27	28.8	27.9	20.7	4.7

Note : F_c = Nominal strength, σ_B = Compressive strength,

 E_c = Young's modulus of concrete

Table 2 Mechanical properties of steel material

Steel	Specification	σ_y (N/mm ²)	$\boldsymbol{\varepsilon}_{y}$ (×10 ⁻²)	σ_u (N/mm ²)	E _f (%)	$\sigma_{y'}\sigma_u$
D-6 Steel bar	SD295	364.93	0.178	485.10	21.77	0.75
D-10 Steel bar	SD295	342.51	0.167	482.74	20.52	0.71
D-13 Steel bar	SD345	380.84	0.186	558.45	17.09	0.68
D-16 Steel bar	SD295	336.62	0.164	478.21	20.81	0.70
D-19 Steel bar	SD345	394.13	0.192	576.27	14.84	0.68
\Box -250 × 250×6	STKR400	381.06	0.186	471.84	31.84	0.81
Note : $\sigma_{}$ =vield strength of steel. $\varepsilon_{}$ =vield strain of steel. $\sigma_{}$ =ultimate strength of steel.						

 ε_f =maximum elongation of steel

wall are shown in the figure, in which a horizontal clearance of 10mm is provided so that longitudinal stresses are not introduced in steel tube.

The other detail shows a horizontal clearance of 20mm between a lower spandrel wall and a foundation beam. The mechanical properties of concrete and steel are



shown in Tables 1 and 2 respectively.

The experimental parameter is the loading programs of the horizontal cyclic force. The specimen SWF-1 is subjected to the loads with gradually increasing amplitudes of horizontal displacements in subsequent loading steps. The SWF-2 is subjected to the horizontal loads with constant and large amplitude of horizontal displacements.

2.2 Loading Apparatus and Measurement System

Fig.6 shows the loading apparatus, which was designed to apply the cyclic horizontal loads H simulating earthquake excitations. The vertical load P corresponding to the gravity load was applied to the specimen concentrically by the 5MN capacity universal testing machine. The vertical load was kept constant during the cyclic horizontal loading. The magnitude of the vertical load was 506 kN. When the horizontal load was equal to 0, the axial force of each TRC column was 253 kN, which corresponds to the axial force ratio of 0.12. The axial force ratio was adopted based on a condition of the columns of the first story of a 12 story building model ⁷¹.

The horizontal loads H were imposed to the test specimen in a manner of pushing in both directions repeatedly. The pull action of a hydraulic jack generated the tensile force in a pair of PC rods of 19mm in diameter, which push the specimen from the opposite side of hydraulic jack position. The horizontal loads were measured by a load cell installed at the head of hydraulic jack.

Fig.7 shows the loading programs. The horizontal deformations were controlled by the drift angle R monitored in loading procedure. The value of R was obtained by dividing the lateral deformation at loading point by the distance between the central axes of cross sections of the upper and lower spandrel walls as



Fig.8 Story drift angle R

shown in Fig.8. The cyclic loading program for SWF-1 increases the amplitudes of R in a step wise by 0.25/100 radian after three successive cycles up to 1.0/100 radian. The large amplitudes of R = 1.5/100 radian and 2.0/100 radian were also tested in one cycle each at the end of loading. SWF-2 was cyclically loaded with a constant large amplitude of R = 3.0/100 radian in eight cycles.

As shown in Fig.8, the joint translation angle of a TRC short column R_c are greater than R. The ratio of R_c / R is 2.4. Therefore the TRC short column is expected to yield in the early loading stage, so that the columns may behave as the hysteretic dampers.

Fig.9 shows the measuring system. As shown in the figure, the horizontal and vertical displacements at the center of the beam to column joint were measured by displacement transducers. The diagonal deformations of spandrel walls were also measured,



(a) Position of displacement transducers



(b) Position of strain gaugesFig.9 Measurement system

which were used to obtain the shear distortion angles of the walls. Furthermore, the out-of-plane displacements at upper wall were also measured.

The strains in steel bars, steel tube, and concrete surface were measured by strain gauges as shown in Fig.9 (b). The strain gauges for steel bars were mounted in plastic hinge zones of upper and lower walls, TRC short columns, and very short RC columns. Strain gauges for steel tubes were mounted at tops and bottoms of them in the horizontal direction. Strain gauges of three axes were also mounted at the centers of concrete surfaces of lower and upper walls to measure the shear strains at the measuring points.

2.3 Collapse Mechanism of Specimens

Two SWF specimens were designed to develop a collapse mechanism shown in Fig.10, where four plastic hinges were assumed to be formed at the top and bottom interfaces between TRC short columns and spandrel walls. It is reported that the ultimate moment of TRC column ($_{C}Q_{f}$) can be estimated as the full plastic moment in which the confinement effect of steel jacket is taken into consideration in the concrete strength $_{\sigma_{CB}}^{8}$.

The equation to calculate full plastic moment of TRC column is shown as follows:

$${}_{c}Q_{f} = 2 \cdot \frac{{}_{c}M_{u} + {}_{s}M_{u}}{h} \tag{1}$$

Where ${}_{c}M_{u}$ is the ultimate bending moment of concrete under the applied axial force of concrete portion, ${}_{s}M_{u}$ is also the full plastic moment of reinforcing bars under the applied axial force of their portion, and *h* is height of the column.

The ${}_{c}M_{u}$ and ${}_{s}M_{u}$ are calculated under the assumption that the neutral axis is same for both portion, in other words, the general superpose strength formulae are used. Moreover, the equation to calculate ${}_{c}\sigma_{cB}$ ⁸⁾ is shown as follows:

$$_{c}\sigma_{cB} = K \cdot _{c}\sigma_{B} \tag{2}$$

in which $K = 1.0 + 11.5 \cdot \frac{st \sigma_y}{c \sigma_B} \cdot \frac{4t^2 (D-t)}{(D-2t)^3}$

Where, *D*, *t*, ${}_{st}\sigma_y$ are the depth, wall thickness and the yield stress of a steel tube, and ${}_c\sigma_B$ is the compressive strength of plane concrete, respectively.

A collapse mechanism is possible due to four plastic hinges formed in TRCs fail in flexure. This mechanism is considered to be avoided in case of the actual design, because formation of plastic hinges in deep spandrel walls or girders results in large amount of axial elongation of spandrel walls. It is very easy to



Fig.10 Plastic collapse mechanism of SWF

design spandrel walls whose ultimate moments are much larger than those of TRC columns.

There are other possible collapse mechanisms as followings:

1) Shear failure of TRC columns

2) Punching shear failure at the top and bottom TRC columns where steel jacket is not provided as shown in Fig.5, and the very short RC columns which connect lower spandrel wall to foundation beam also shown in Fig.5.

3) Shear failure of spandrel walls

These possible collapse mechanisms should be avoided in SWF because of brittle nature of these collapse mechanisms. Occurrence of shear cracks in spandrel walls is also considered to be avoided from the view point of damage control. It is necessary to estimate shear strengths of columns and spandrel walls which is considered as beams with large depth, and also punching shear strength of columns and shear crack strength of spandrel walls.

The shear strength $Q_{su}^{(9)}$ of the TRC short column is

$$Q_{SU} = \left\{ \frac{0.053 p_t 0.23 (18 + F_c)}{M / (Q.d) + 0.12} + 0.85 \sqrt{p_{W} \cdot s \sigma_{Wy}} + 0.1 \sigma_0 \right\} b.j \quad (3)$$

calculated using the method as follows:

Where p_i , p_w , $s\sigma_{wy}$ are the tensile chord bars ratio, lateral reinforcement ratio, and yield stress of lateral reinforcement respectively. Furthermore, σ_0 , *j*, *b*, *d*, are the average of axial stress in concrete of the column under vertical load, the distance between compressive and tensile resultants in a section, the width, and the depth of concrete section, respectively.

The Eq.(4) is the punching shear strength Q_p , which is based on the reference 9), and was modified by the method described in the reference 5).

$$Q_p = 1.25 \times K_{av} \times \tau_0 \times b \times D \tag{4}$$
$$K_{av} = \frac{0.58}{2}$$

in which
$$K_{av} = \frac{0.38}{0.76 + \frac{a}{D}}$$

Table 3 Calculated lateral load capacities of specimens

	Colu	Column		Spandrel Wall	
Collapse Mechanism	SWF-1	SWF-2	SWF-1	SWF-2	
Flexure failure (kN)	300	292	734	734	
Shear failure (kN)	525	521	1159	1141	
Punching shear failure (kN)	960	980		\sim	
Shear crack (kN)	/		562	550	

$$\begin{aligned} \tau_0 &= 0.98 + 0.1 F_{c1} + 0.85\sigma \quad (0 \le \sigma \le 0.33 F_{c1} - 2.75) \\ &= 0.22 F_{c1} + 0.49\sigma \quad (0.33 F_{c1} - 2.75 < \sigma \le 0.66 F_{c1}) \\ &= 0.66 F_{c1} \quad (0.66 F_{c1} < \sigma) \\ \sigma &= P_g \cdot \sigma_y + \sigma_0 \end{aligned}$$

Where, p_g , σ_y , and σ_0 are the longitudinal reinforcement ratio, the yield stress of vertical reinforcement bars, and the average of axial stress in concrete of the column under vertical load, respectively. The *a*, *b*, and *D* are the shear span, the width, and the depth of concrete section, respectively.

The bending moment strength of spandrel walls are based on the full plastic moment without the confining effect of concrete, and shear strength of wall calculated based on Eq.(2) without considering the average of axial stress σ_0 .

The shear crack strength $V_c^{(10)}$ is calculated by the following formula:

$$Vc = \phi(\sqrt{\sigma_T^2 + \sigma_T \cdot \sigma_0}) \cdot b \cdot D / \kappa$$
(5)

in which $\sigma_T = 0.33 \sqrt{\sigma_B}$

Where, $\phi(=1)$, $\kappa(=1.5)$, are the coefficient of cross-sectional shape and the coefficient of concrete strength, respectively.

Lateral load capacities of two SWF specimens failed in various collapse mechanisms including shear crack capacities are compared in Table 3. The TRC column collapse mechanism (Fig.10) should be formed in SWF specimens because lateral strength calculated by

 Table 4 Comparisons of the horizontal ultimate strengths between the experiment and calculation results

Test Specimens	H_{exp} (kN)	H_{cal} (kN)	H_{exp}/H_{cal}	Error (%)	
SWF-1	296.3	300.4	0.99	1.0	
SWF-2	282.8	292.3	0.97	3.0	
Note: H_{em} = horizontal ultimate strengths by experiment, H_{em} = horizontal ultimate strengths by					

calculation the flexure strength is the least.

The ultimate capacities of TRC columns are estimated ignoring additional axial force due to overturning moments at collapse mechanism.

3. EXPERIMENTAL RESULTS

3.1 H-R relationships and failure modes

Figs.11 (a) and (b) show the relationships between the horizontal loads H and the drift angle R of the specimens SWF-1 and SWF-2, respectively. Dotted lines indicate the calculated horizontal strengths being based on the full plastic moments of TRC short columns as shown in the Eq.(1), and dash-dotted lines indicate crack strengths of the spandrel walls. The solid circle indicates the initial yielding point of longitudinal reinforcements of TRC short column determined by strain measurements. Furthermore, the solid triangle indicate the ultimate strength of SWF specimens which are obtained by the experiment results whereas their values are shown in the Table 4.

Fig.12 and Fig. 13 show the specimens SWF-1 and SWF-2, respectively, after the tests.

The comparisons of the horizontal ultimate strengths between those obtained by experiment and calculation are shown in Table 4. The experimental result and the calculation show a good agreement. In other words, calculation strength by full plastic moment of TRC short column can estimate the strength of the test specimens accurately.



Fig.11 Relationship between Lateral force H and Drift angle R

(a) Test specimen SWF-1

As shown in Fig.11 (a), the type of the hysteresis loops of the SWF-1 is the cyclic hardening type, which is the maximum strength at each loading cycle increases with increasing of the amplitude of the R. The pinching is also observed in the loops, so that the self-centering effect can be expected for the SWF system.

The cyclic horizontal loading was started from a small amplitude of R = 0.25/100 radian. At this stage, the yielding of longitudinal steel bars in the columns was already observed. At the amplitude of the *R* of 0.5/100 radian, the flexural cracks were observed by naked eye at the top and the bottom of TRC columns.



(a) Whole aspect



(b) The upper spandrel wall



(d) The lower edge of TRC columnFig.12 The SWF-1 specimen after test



(a) Whole aspect



(b) The upper spandrel wall



(d) The lower edge of TRC columnFig.13 The SWF-2 specimen after test

The remarkable cracks at the spandrel walls appeared at the amplitude of R = 1.5/100 radian. Although the cracks propagated gradually in the subsequent load steps, the spall off of concrete was not observed until the loading program was completed as shown in Fig.12. The diagonal cracks in the walls were not observed.

With respect to steel tubes, the damage such as the local buckling was not observed.

(b) Test specimen SWF-2

The hysteresis loops of SWF-2 are shown in Fig.11 (b). When the R reached 0.38/100 radian in the first loading cycle, the yielding of longitudinal reinforcements in TRC short columns were



(c) The lower spandrel wall



(e) The upper edge of TRC column



(c) The lower spandrel wall



(e) The upper edge of TRC column



Fig.14 Relationships between axial elongation of column δ and drift angle *R*

observed. At the R of 1.7/100 radian in the first loading cycle, the flexural cracks were observed by naked eye at the ends of TRC short columns. As the horizontal load increased, cracks propagated.

The remarkable wide cracks were observed in the walls near the columns at R = 2.0/100 radian as shown in Fig.13.

As the loading cycles increased, the horizontal loads deteriorated gradually. The strength reduction was caused by the spall off of cover concrete and the buckling of longitudinal reinforcements in the walls near the columns as shown in Fig.13 (d).

Some of the longitudinal reinforcements buckled in the walls around the column joints at the fifth loading cycle. The diagonal fine cracks were also observed in the lower wall. It is noteworthy that the loading program of SWF-2 is considered to be a very severe loading condition which would hardly occur in practical situations.

3.2 Behaviors of TRC short columns

Figs.14 (a) and (b) show the axial elongation of columns δ and the drift angle *R* relationships of the specimens SWF-1 and SWF-2, respectively. The δ was the averaged displacements of the story height, which were measured by vertically located displacement transducers as shown in Fig.9 (a).

In each specimen, the δ increased in accordance with the increase of the *R*. However, the δ restored to almost zero, when the *R* restored to zero. It is the typical behaviors of the TRC columns under the low axial force ratio, such a load condition is expected in



(b) The crack openings were closed when horizontal load return to zero

Fig.15 TRC column's behavior of SWF-1

(b) The crack openings were closed when horizontal load return to zero

Fig.16 TRC column's behavior of SWF-2



Fig.17 Relationships between Strain ε of longitudinal bars of TRC column and Drift Angle R

the typical SWF as the exterior frames of a prototype of (DTHS). This phenomenon is caused by cracks at the interfaces of the TRC short columns to the spandrel walls as shown in Fig.15 and 16. On the other hand, the crush in compression was scarcely observed in concrete portions, except for the final stage of the SWF-2 subjected to very large amplitude of the R under cyclic horizontal loads.

Fig.17 shows the relationships between the axial strains ε of longitudinal reinforcements of TRC short column and the drift angle R of SWF. The strains were by strain gauges as shown in the figure. The dotted horizontal lines shown in Fig. 17 indicate the yield strain defined by the coupon tensile test. From the figure, the longitudinal reinforcements of TRC short columns experienced the large tensile plastic elongations, but the small compressive plastic shortenings. In such a manner, it may be said that the longitudinal reinforcements in TRC short columns dominated the plastic behaviors of the SWF specimens. Due to some gauges become to faile when loading test in the progress, only the first cycle of the tensile yielding can be observed as shown in Fig 17(b). The longitudinal reinforcements of TRC short columns yielded in the early loading stage of R less

than 0.5/100 radian. Therefore, as expected in the design concept for SWF system, TRC short columns are supposed to behave as the hysteretic dampers

3.3 Behaviors of spandrel walls

Figs.18 (a) and (b) show the relationships between strains of horizontal reinforcements in spandrel walls and the drift angle R. The horizontal reinforcements function as the longitudinal reinforcements in girders. The strain shown in each figure is that in the bottom horizontal reinforcement of the lower spandrel wall which measured by strain gauges as shown in Fig.9 (b). The dotted lines on the figure are the yield strains of the reinforcements. As shown in the figure, the reinforcements in the walls remained in elastic until the loading program was completed.

Figs.19 (a) and (b) show the relationships between shear strains in spandrel walls γ and the *R* of the SWF-1 and SWF-2. The shear strains were measured by strain gauges mounted on concrete surfaces at the centers of the upper and the lower spandrel walls as shown in Fig. 9 (b). As shown in Figs.19 (a) and (b), the shear strains γ were limited up to around 0.01/100 radian. The limit is caused by the TRC short column



Fig.18 Relationships between Strain ε of wall's main reinforcement and Drift Angle R



Fig.19 Relationships between Shear Strain γ of wall and Drift Angle *R*

yielding. Therefore, a validity of the strength prediction in Table 3 is proved by the experiments. It may be said that no shear cracks in the walls can be realized in the SWF system.

5. CONCLUSIONS

Two specimens of a new constructional system frame called as the Spandrel Wall Frame (SWF) were investigated experimentally under cyclic horizontal loading. The SWF was composed of spandrel walls and steel Tube jacketing Reinforced Concrete (TRC) short columns. The following conclusive remarks were obtained:

(1) Both of the SWF-1 and SWF-2 specimens indicated the stable elasto-plastic horizontal behavior. The hysteresis loops are the cyclic hardening type with pinching. The severe damage was scarcely observed in the spandrel walls and the jacketing steel tubes, except for the final stage of the SWF-2 which was subjected to very large amplitude of the R under cyclic horizontal loads.

(2) TRC short columns yielded at the tops and the bottoms of the columns. The yield mode was the crack opening of concrete in bending and the tensile yielding in longitudinal reinforcements.

(3) The strength and the failure mode of the SWF can be dominated by the TRC short column yielding in bending, which can be easily designed and constructed in the prototype SWF system. The horizontal strength of TRC short column can be estimated by the full plastic moment of the column.

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