Experimental Study on RCCV of ABWR Plant
Part 3: In-Plane Shear Test of RC Panels
Cylindrical Model

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1. INTRODUCTION

The aim of this study is to determine the in-plane shear behavior of RCCV for an ABWR plant (Saito et al., 1989) during earthquake load. The experimental study consists of two series of tests: a shear loading test on reinforced concrete (RC) panels and a torsional loading test on a RC cylindrical model. This paper reports the test results and discusses the agreement of the ultimate shear strengths obtained from the tests with those predicted by the design formula for RCCV.

2. TEST PROGRAMS

2.1 Panel Tests

Eight RC panel specimens as listed in Table 1 were used in the panel tests. Figure 1 shows the dimensions of a typical specimen. The specimens, 1/12 scale models of the cylindrical wall of the RCCV, were 2500mm square and 167mm thick. The parameters considered in the panel tests were loading condition, reinforcement ratio (0.85, 1.70 and 2.55%) and deformed rebar diameter (10mm and 16mm). Equal quantities of reinforcement were arranged in two layers in two orthogonal directions. The design compressive strength of concrete was settled at 300kg/cm² and the maximum aggregate size was 13mm. The results of the concrete and rebar material tests are shown in Table 2.

Two types of loading condition were employed: a) pure shear loading (PS specimens) and b) combined shear and biaxial tension loading (TS specimens). In the latter case, the shear stress was applied to the specimens under constant biaxial tensile stresses of $\sigma_x=20$kgf/cm² and $\sigma_y=10$kgf/cm², by the combined loading test equipment shown in Figure 2 (Ohmori et al., 1987). The shear stress was applied to the panels with reversed cyclic loading.

Using Mohr's strain analysis method, the strain state of the specimens was evaluated from the relative displacements of the panels measured by the six LVDTs as shown in Figure 1. Rebar strains, crack widths and intervals were also measured.

2.2 Cylindrical Model Test

One cylindrical RC specimen was employed. This specimen(CM1), a 1/12 scale model of the cylindrical wall of the RCCV, was 2400mm inside diameter, 3400mm height, and 167mm thick, as shown in Figure 3. The reinforcement ratio was 1.35%, and 10mm-diameter deformed rebars were arranged in two layers in two orthogonal directions, at 63mm spacing. The same type of concrete was used as in the panel tests.
A reversed cyclic torsional moment was applied to the specimen as shown in Figure 3, by pulling the wire strands attached to the top slab of the specimen. The shear deformation angle was evaluated from the difference in measured displacements between the top and bottom portions of the cylinder.

3. TEST RESULTS

3.1 Shear Stress - Shear Strain Relationships for Panel Test

Figure 4 shows the measured shear stress (τ) - shear strain (γ) relationships of the panels. The observed initial shear stiffness agreed well with the calculated elastic stiffness when only concrete was taken into account. The shear cracking stress ranged from 12.0kGF/cm² to 18.5kGF/cm². This was below $V_c = 0.33 \cdot f_{c'}^2$ (Mpa), the value given in the ACI code. The shear strain of specimen PS1, which had a small reinforcement ratio, increased very rapidly once cracks were initiated. The strain then increased gradually, ending in failure with a large shear strain accompanied by rebar yielding. This specimen showed a notable slip phenomenon in the low-shear-stress range, due to the reversed loading. On the other hand, the specimens with larger reinforcement ratio showed the higher stiffness after crack initiation, and their failure modes were crushing of concrete prior to rebar yielding. The shear strains for such specimens at failure were relatively small, and slip due to reversed loading was only slight. For specimens PS2 and PS3, which had the same reinforcement ratio but different rebar diameter, there was no significant difference observed in the τ - γ relationship.

The initial shear stiffness of the specimens subjected to biaxial tensile stress was lower due to the occurrence of orthogonal cracks induced by the biaxial tensile stress. A slip phenomenon was clearly observed; even in the specimens with a larger reinforcement ratio, and their failure modes were shear concrete failure accompanied by steel yielding similar to those observed in the specimens with the smaller reinforcement ratio. This behavior is considered to result from the decrease in apparent reinforcement ratio caused by the biaxial tensile stress.

3.2 Rebar Strain

Figure 5 shows some examples of the relationship between shear stress and rebar strain. Strain was not produced in the rebars of the specimens subjected to pure shear stress until cracks occurred. After crack initiation, strain first increased very rapidly, and then increased gradually, ending in rebar yielding. This agreed well with the calculated steel stiffness. For the combined stress condition, the initial rebars strain was caused by biaxial tensile stress. The measured rebar strain caused by the shear loading also agreed with the calculated steel stiffness, in which the effect of the biaxial tensile stress was taken into account.

3.3 Crack Widths and Intervals

The relationship between the average crack interval and the applied shear stress is shown in Figure 6, and that between crack width and applied shear stress is shown in Figure 7. As shown in these figures, crack intervals and cracks widths varied depending on the spacing and diameter of the rebars. This variation can be clearly evaluated by comparing the results of specimens PS2 and PS3. These two specimens have the same reinforcement ratio, but different spacing and rebar diameter. The observed crack intervals and crack widths in specimen PS3, which has larger rebar diameter and wider spacing, are larger than those of specimen PS2. Thus, the crack intervals and crack widths are closely related to the spacing and rebar diameter, but the magnitudes of shear strain are almost the same for both specimens. The width of orthogonal cracks caused by the biaxial tensile stress changed little even when the shear
stress was applied, and only the width of the diagonal shear cracks tended to increase.

3.4 Comparison of Cylindrical Model and Panel

Figure 8 shows the $\tau - \gamma$ relationship for the cylindrical model. The shear strain increased very rapidly after crack initiation. Cracks then developed quickly, covering almost the entire cylinder surface. Shear stiffness then increased, approximating the calculated steel stiffness. Afterward, the strain developed gradually, leading to rebar yielding. A maximum shear strength was reached, and then the load declined gradually, ending in shear failure at a strain of $\gamma=10\times10^{-3}$ rad.

The $\tau - \gamma$ envelope curve of the panel specimen PS2 is also shown in Figure 8. Although, there is a slight difference in reinforcement ratio, concrete strength, etc., between the cylindrical model and panel specimen, the general property of the $\tau - \gamma$ relationship indicates a similar tendency. In addition, their crack initiation and failure mode properties were also similar, proving that the shear properties of a cylindrical model can be estimated by modeling a panel element as part of a cylindrical wall.

3.5 Ultimate Shear Strengths

Figure 9 compares the ultimate shear strengths ($\tau_u$) obtained from the tests with those from Eqs.(1) and (2), below, which are used in RCCV design. This comparison confirms that $\tau_u$ for specimens with a small reinforcement ratio can be evaluated by Eq.(1), which corresponds to the failure mode of rebar yielding, while $\tau_u$ for specimens with a large reinforcement ratio can be evaluated by Eq.(2), which corresponds to the shear failure of concrete.

$$\tau_u = \rho \cdot f_y \quad \ldots \ldots \text{(1)}$$
$$\tau_u = 3.5\sqrt{f_c} \quad \ldots \ldots \text{(2)}$$

where,
\begin{align*}
\rho & : \text{reinforcement ratio} \\
f_y & : \text{yield strength of rebar} \\
f_c & : \text{concrete strength}
\end{align*}

4. CONCLUSION

As a result of tests on panels and a cylindrical model, it was clarified that shear behavior is dependent on the reinforcement ratio, rebar diameter, and the existence of biaxial tensile stress, and that changes in this behavior can be quantitatively described.

In addition, the measured ultimate shear strengths agreed well with those calculated from the design formula for RCCV.

5. ACKNOWLEDGMENT

This study has been carried out as a part of a joint research study on the "Evaluation of the RCCV Configuration and Confirmatory Test to Establish a Code ".

REFERENCES


Table 1. List of test specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>PS1</th>
<th>PS2</th>
<th>PS3</th>
<th>PS4</th>
<th>TS1</th>
<th>TS2</th>
<th>TS3</th>
<th>TS4</th>
<th>CMI</th>
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<tr>
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<td>2.55</td>
<td>0.85</td>
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<td>D16</td>
<td>D16</td>
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<td>D16</td>
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<td>Spacing (mm)</td>
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Note, P.S: Pure Shear, T.S: Shear Under Biaxial Tension, T: Torsional

Table 2. Material properties of concrete and reinforcement.

<table>
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<tr>
<th>Specimen</th>
<th>PS1</th>
<th>PS2</th>
<th>PS3</th>
<th>PS4</th>
<th>TS1</th>
<th>TS2</th>
<th>TS3</th>
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<td>Yield Strain $\varepsilon_y$ ($10^{-6}$)</td>
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Note; *1 : Panels, *2 : Cylindrical model

Figure 1. Test specimen of RC panel.

Figure 2. Outline of testing facility for RC panel.
Figure 3. Test specimen of RC cylindrical model and loading method.

Figure 4. Shear stress – shear strain relationships for RC panel.
Figure 5. Shear stress – rebar strain relationships for RC panel.

Figure 6. Shear stress – average crack interval for RC panel.

Figure 7. Shear stress – crack width for RC panel.

Figure 8. Shear stress – shear strain relationship for RC cylindrical model.

Figure 9. Comparison of measured ultimate shear strengths and evaluating formula.