INFLUENCE OF CONCRETE TYPE AND STRENGTH TO SHEAR BEHAVIOR OF RC PANEL

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ABSTRACT

Collins et al. reported a method to grasp the shear resistance of RC panels, considering the following three points; compression characteristics of panels under tensile forces of concrete, softening field of panels in a relationship between tensile stress and strain, and transmitting shear stresses across crack by aggregate interlock. Based on this report, an experiment using more different cast materials and material strengths was performed and the evaluation method examined. From the results, equations of tensile strain versus compressive strength were proposed when 50 MPa grade mortar was utilized.
KEYWORDS: RC panels, bi-axial resistance, pure shear, stress versus strain, softening field

1. INTRODUCTION

In the past few decades, studies on shear resistance of reinforced concrete (RC) panels combining reinforcing materials (rebars) and concrete have been reported. One of the most typical reports on these studies is the Modified Compression-Field Theory (MCFT) by Collins et al.[1] Collins et al. proposed a method in which the shear resistance of RC panels was understood, considering the following three points; compression characteristics of panels under tensile forces of concrete, softening field of panels in a relationship between tensile stress and strain, and transmitting shear stresses across cracks by aggregate interlock. In their study, however, only one kind of cast material and strength was used. The effect of shear resistance of RC panels by varying concrete strength and class was not clarified. In addition to a cast material that Collins et al. used, higher strengths of concrete and mortar were prepared in this study. The same experiment was performed, and the results were analyzed in the same way as Collins et al. did. The purpose of this study is to examine the appropriateness of the evaluation method proposed by Collins et al.

2. OUTLINE OF EXPERIMENT

2. 1 SPECIMENS

Figure 1 shows a layout of a typical specimen. The specimen size is 300×300×25mm. 24 holes were prepared for a loading apparatus along the edges of the panel. Four

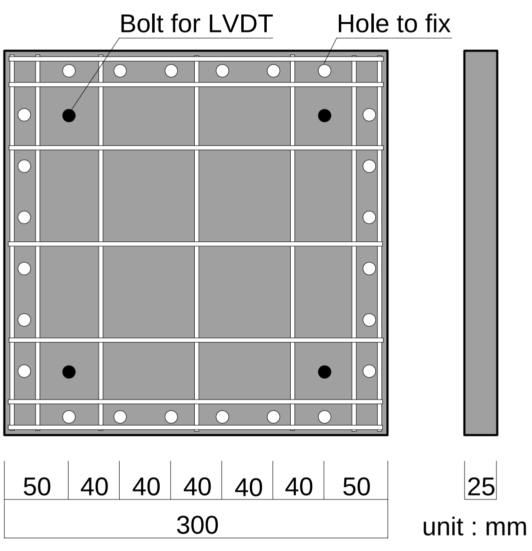

Fig. 1 Detail of specimen

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screw bolts for displacement transducers were located on each corner of the panel. The reinforcement bars were placed at intervals of 75mm parallel in both lateral and vertical directions. In addition, D4 deformed bars were prepared at 5mm and 25mm from the edge of the panel in order to prevent loading locations from failure. As the parameters of the specimens, two different kinds of concrete and one kind of mortar were used. Three different sizes of lateral and vertical reinforcements were utilized, which were D3, D4, and D6. Table 1 summarizes the details of the specimens.

Table 1 List of specimens

Identification	Concrete type	Reinforcements			
		Lateral	ρ_x (10^{-3})	Vertical	ρ_y (10^{-3})
C2-D33	20MPa class concrete (C2)	D3	2.83	D3	2.83
C2-D34		D4	5.02	D3	2.83
C2-D36		D6	11.3	D3	2.83
C2-D44		D4	5.02	D4	5.02
C2-D46		D6	11.3	D4	5.02
C2-D66		D6	11.3	D6	11.3
M5-D33	50MPa class mortar (M5)	D3	2.83	D3	2.83
M5-D34		D4	5.02	D3	2.83
M5-D36		D6	11.3	D3	2.83
M5-D44		D4	5.02	D4	5.02
M5-D46		D6	11.3	D4	5.02
M5-D66		D6	11.3	D6	11.3
C5-D33	50MPa class concrete (C5)	D3	2.83	D3	2.83
C5-D34		D4	5.02	D3	2.83
C5-D36		D6	11.3	D3	2.83
C5-D44		D4	5.02	D4	5.02
C5-D46		D6	11.3	D4	5.02
C5-D66		D6	11.3	D6	11.3

ρ = (total sectional area of reinforcements) / (panel width × panel thickness)

2. 2 MATERIALS

Table 2 shows the properties of concrete and mortar. The concrete strengths were designed as 20MPa and 50MPa. Maximum diameter 8mm of coarse aggregates, maximum diameter 2.5 of fine aggregates, and standard Portland cement were utilized for both kinds of concrete. The designed strength of the mortar was 50MPa. Pre-mixed mortar was used. The properties of reinforcements are shown in Table 3. Three different deformed bars, D3, D4, and D6 were prepared as reinforcement bars.

Table 2 Concrete properties

Identification	Compressive strength (MPa)	Elastic modulus (GPa)	Splitting strength (MPa)
C2	23.0	21.4	2.60
M5	48.5	20.5	3.09
C5	48.1	35.4	3.59

Table 3 Reinforcement properties

Identification	Sectional area (mm^2)	Yield strength (MPa)	Elastic modulus (GPa)
D3	7.07	222	225
D4	12.6	290	200
D6	32.0	412	186

2. 3 LOADING AND MEASUREMENT SYSTEM

Photo 1 shows the loading system. As shown in Photo 1, 24 oil jacks were used. The pure shear force was created by loading tensile forces and compressive forces diagonal to the specimen. The 24 oil jacks were divided in two ways, which were tensile and compressive. The loading jacks were controlled so as to create both tensile forces and compressive forces uniformly. The measurement items were the load per an oil jack, deformations at each edge and a tensile diagonal, and strains of rebars. Figure 2 and 3 illustrate the locations of displacement transducers and the placement of strain gauges, respectively. Arrows in both Figure 3 and 4 indicate loading directions of the oil jacks. The displacement transducers were placed at both sides of the panel. Four of the transducers were parallel to each edge (at 50mm from each edge) and one was on a tensile diagonal. A gauge length of strain gauges was 1mm.

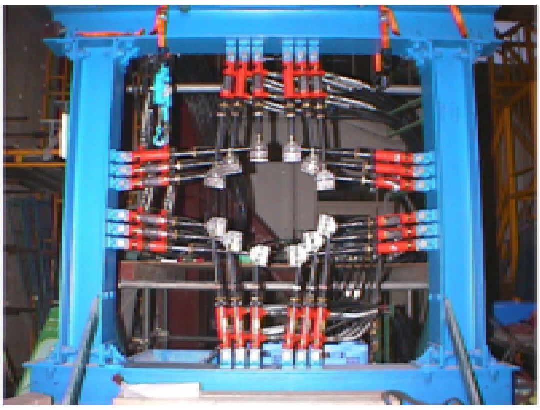

Photo 1 Loading system

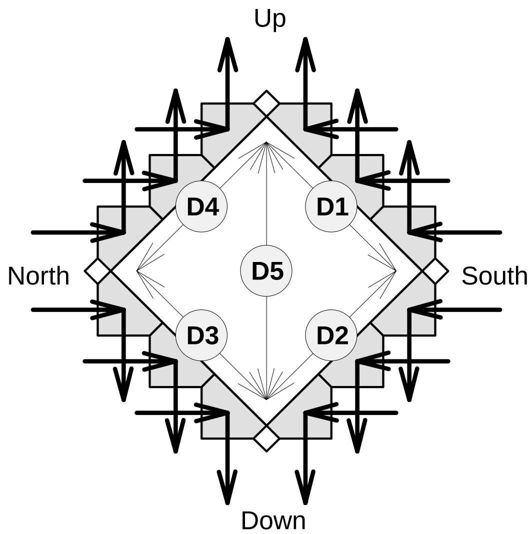

Fig. 2 Set up of LVDTs

3. TEST RESULTS

3. 1 FAILURE PROCESS

Shear stresses and shear strains were calculated using the following equations with data of the experiment.

$$\tau_{xy} = 3\sqrt{2} \cdot P / (t \cdot b) \quad (1)$$

$$\gamma_{xy} = \left\{ \sqrt{2} \cdot \delta_5 - (\delta_1 + \delta_3)/2 - (\delta_2 + \delta_4)/2 \right\} / l_0 \quad (2)$$

where,

τ_{xy} : shear stress of panel
γ_{xy} : shear strain of panel
P : load per oil jack
t : panel thickness
b : panel width
δ_i : deformation by LVDTs (corresponding to Fig. 2)
l_0 : marked interval (=200mm)

Figure 4 illustrates a typical result of the shear stress versus shear strain curves. The three different lines in Figure 4 represent specimens that used deformed bars D3 for both lateral and vertical reinforcements. Some specimens were eventually failed at the edge. However, shear stress versus shear strain curves showed tri-linear type diagrams for most of the specimens. The diagrams indicated that cracks of concrete, yield of rebars, and compressive failure of concrete occurred in the failure process in this order. Table 4 summarizes the results of the experiment. The results include shear stress and strain when the first crack occurred, shear stress at the ultimate strength, the ultimate shear strain, and the final failure type. The failure types were classified into three types as rebar rupture, edge failure, and concrete compressive failure. All strain gauges of specimens D33, D34, and D44 indicated that the reinforcements of these specimens yielded before the specimens failed.

3. 2 MAXIMUM SHEAR STRESS COMPARISONS

Figure 5 illustrates the concrete compressive strength versus the maximum shear stress for each specimen. Arrows in Figure 5 indicate that these specimens failed at the edge and the maximum stresses are possibly higher than the ones plotted.

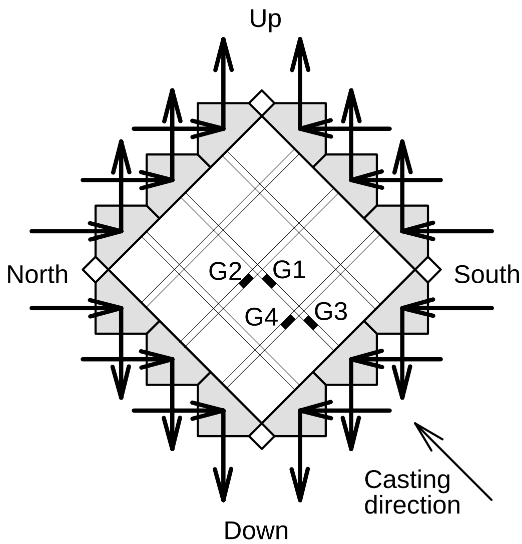

Fig. 3 Position of strain gauges

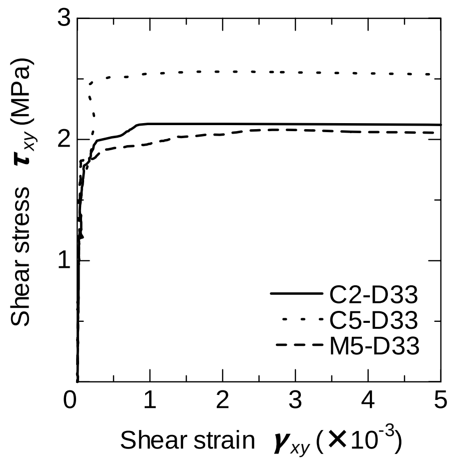

Fig. 4 Shear stress versus shear strain curves

Table 4 List of Test results

Identification	Cracking stress (MPa)	Cracking strain ($\times 10^{-3}$)	Max. stress (MPa)	Ultimate strain ($\times 10^{-3}$)	Failure mode
C2-D33	1.81	0.03	2.16	24.9	Rupture
C2-D34	1.66	0.37	2.34	78.6	Comp
C2-D36	0.76	0.02	2.75	0.68	Edge
C2-D44	1.37	0.06	2.71	0.91	Comp
C2-D46	1.86	0.13	2.72	1.92	Edge
C2-D66	1.42	0.26	3.04	2.69	Rupture
M5-D33	1.83	0.04	2.08	6.56	Comp
M5-D34	1.03	0.30	2.58	9.54	Edge
M5-D36	1.07	0.04	2.68	10.8	Edge
M5-D44	1.26	0.05	2.73	50.6	Comp
M5-D46	1.39	0.14	3.21	2.67	Edge
M5-D66	1.12	0.11	4.91	3.17	Edge
C5-D33	1.67	0.12	2.56	4.02	Edge
C5-D34	2.03	1.33	2.57	37.8	Edge
C5-D36	1.89	0.07	2.83	2.14	Edge
C5-D44	1.57	0.36	2.93	46.1	Comp
C5-D46	1.41	0.16	2.93	15.6	Edge
C5-D66	1.66	0.16	4.03	1.53	Edge

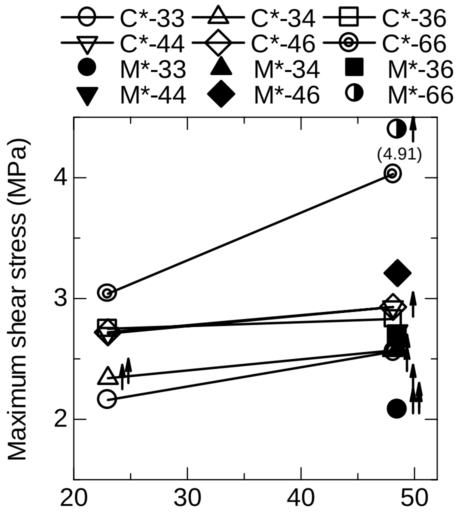

Fig. 5 Comparison of maximum shear stress

4. VERIFICATION OF PRINCIPAL STRESS AND STRAIN

The concrete principal tensile/compressive stress versus principal tensile/compressive strain and the principal tensile stress versus compressive concrete strength of cracked concrete were derived using the Mohr's strain circle principle. The data used in this principle were obtained from the strain calculated from displacement transducers. The results were compared and verified to the equation proposed by MCFT.

4. 1 CALCULATION METHOD

The concrete principal stress versus principal strain was derived as follows. Figure 6 and Figure 7 show stress and strain states of panels and the Mohr's strain circle, respectively. Strain of x direction(ε_x) and one of y direction (ε_y) are calculated from the displacement transducers. The average principal tensile strain (ε_1) and the average principal compressive strain (ε_2) are obtained from the Mohr's strain circle. It is assumed that the average strains of concrete and reinforcements are the same. Based on this assumption, the tensile and compressive stresses of the reinforcements are calculated from the perfectly elastic – plastic stress versus strain relationship. Since the concrete stress is the difference between the total stress of the specimen and the stress of reinforcements, the principal tensile and compressive stresses of the concrete are calculated.

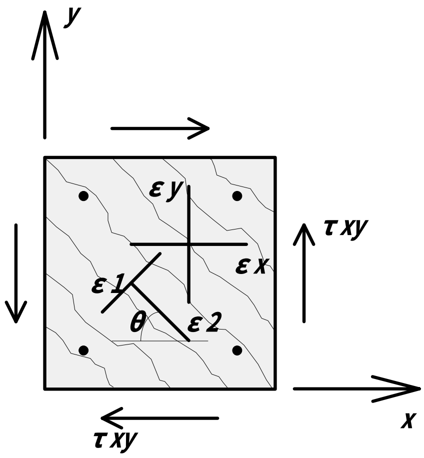

Fig. 6 Stress and strain on panel

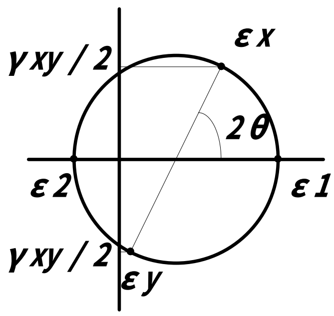

Fig. 7 Mohr's strain circle

4.2 PRINCIPAL TENSILE STRESS VERSUS PRINCIPAL TENSILE STRAIN

The MCFT proposed the following equations for the relationship between the principal tensile stress and the principal tensile strain.

$$\begin{cases} f_{c1} = E_c \cdot \varepsilon_1 \quad (\varepsilon_1 \le \varepsilon_{cr}) \\ f_{c1} = \dfrac{f_{cr}}{1+\sqrt{200 \cdot \varepsilon_1}} \quad (\varepsilon_1 > \varepsilon_{cr}) \end{cases} \tag{3}$$

where,

f_{cl} : principal tensile stress of concrete

E_c : elastic modulus of concrete

ε_{cr} : concrete strain at cracking

The equations above mean that the tensile stress of the concrete is proportional to the tensile strain of the one until before concrete cracks and that after concrete cracks, the tensile stress of the concrete gradually decreases corresponding to the increase of tensile strain. The f_{cr} represents cracking strength of concrete. The equation, f_{cr}, is as follows.

$$f_{cr} = 0.33\sqrt{-f_c} \tag{4}$$

where

f_c : compressive strength of concrete (negative) (MPa)

The calculated values from Equations (3) and (4), and experiment results for specimens C2 are compared in Figure 8. Some specimens using 50MPa of concrete compressive strength and using mortar indicated that the experiment results were lower than the calculated results. However, the experiment results generally correspond to the calculation analysis.

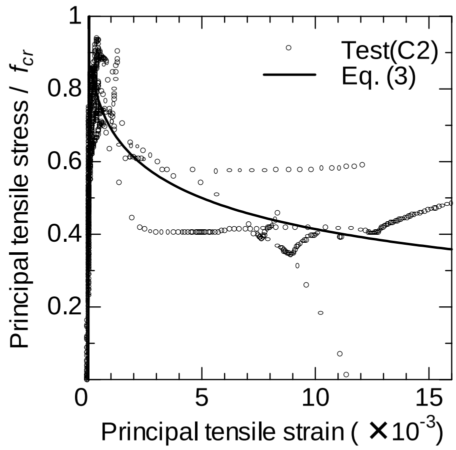

Fig. 8 Principal tensile stress – strain curves

4.3 PRINCIPAL COMPRESSIVE STRESS AND PRINCIPAL COMPRESSIVE STRAIN

The MCFT proposed the following equations for the relationship between the principal compressive stress and the principal compressive strain of concrete.

$$f_{c2} = f_{c2\max} \cdot \left\{ 2 \cdot \left(\frac{\varepsilon_2}{\varepsilon_c} \right) - \left(\frac{\varepsilon_2}{\varepsilon_c} \right)^2 \right\} \tag{5}$$

$$\frac{f_{c2\max}}{f_c} = \frac{1}{0.8 - 0.34 \cdot \varepsilon_1 / \varepsilon_c} \le 1.0 \tag{6}$$

where,

f_{c2} : principal compressive stress of concrete (negative)

f_{c2max} : compressive strength of cracked concrete (negative)

ε_c : strain at compressive strength (negative)

Equation (5) represents the principal compressive stress of concrete versus principal compressive strain of concrete at the pure compressive state, $f_{c2max} = f_c$. Equation (6) represents the decline of concrete compressive strength because of cracked concrete where the tensile force works. Figure 9 compares the calculated results from Equations (5) and (6) to the experiment data for specimen C2-D33. The experiment results generally correspond to the calculation analysis.

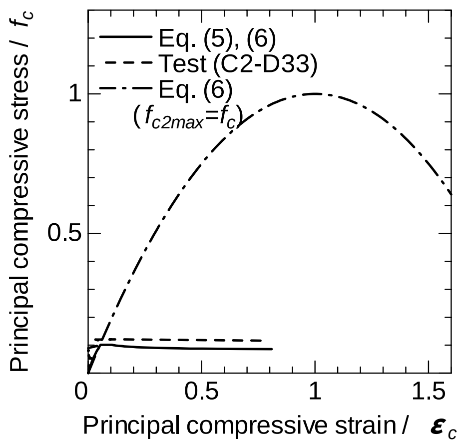

Fig. 9 Principal compressive stress – strain curves

4.4 PRINCIPAL TENSILE STRAIN VERSUS COMPRESSIVE STRENGTH

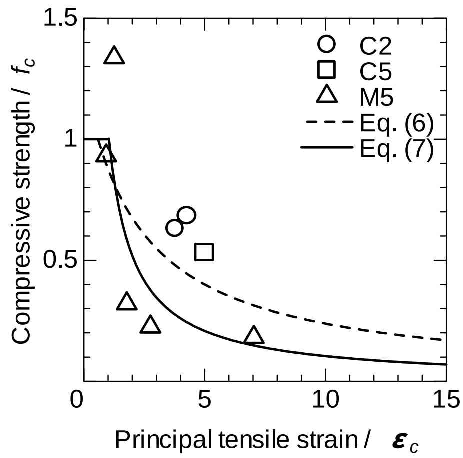

Fig. 10 Compressive strength of cracked concrete

Figure 10 illustrates f_{c2}/f_c versus $\varepsilon_1/\varepsilon_c$ using the experiment data. Equation (6) is represented as a dot line. In Figure 10, specimens that the f_{c2max} could not be calculated because the Mohr's strain circle did not cross to the y-axis in Figure 7 were excluded. Specimens C2 and C5 showed the experiment data were generally close to the calculation analysis by MCFT, regardless of the concrete strength. However, in terms of specimens using mortar, the experiment data tended to be lower than the calculated values from the proposed equations. This is because the f_{c2max} of specimens using mortar decreased faster than the one of specimens using concrete. It is assumed that there was no aggregate in mortar which supposed to make compressive stress stronger.

Equation (7) was obtained from the regression analysis by the least square method (square deviation 0.28). The range of x-axis value from this experiment was from 0.926 to 7.033.

$$\frac{f_{c2\max}}{f_c} = -\frac{1}{0.95 \cdot \varepsilon_1 / \varepsilon_c} \leq 1.0 \qquad (7)$$

Figure 11 shows the difference of the calculation analysis from Equation (6) and (7). The data of specimen M5-D44 were adopted as a typical example of the shear stress versus shear strain. The diagrams of Equations (6) and (7) were the same. However, it indicated that the f_{c2max} of Equation (7) declined as the shear strain increased. In other words, it represented that a compression failure tends to occur in the early phase.

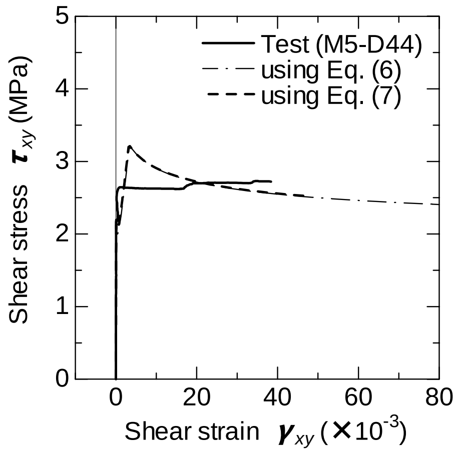

Fig. 11 Comparison of shear stress – strain curves

5. CONCLUSIONS

The calculation analysis by MCFT was compared to the experiment results in terms of the concrete principal tensile/compressive stress versus principal tensile/compressive strain. The principal tensile strain versus compressive strength of specimens using mortar represented that the calculation analysis slightly differentiated from the experiment results. In order to improve this difference between the calculation analysis and the experiment results, new equations were proposed for this relationship for specimens using mortar. Since the calculation results corresponded to the experiment data for specimens using compressive strength grade 20MPa and 50MPa of concrete, the proposed equations by MCFT were generally appropriate for both concrete.

REFERENCE

[1] Vecchio, F.J. and Collins, M.P. : "The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to Shear", ACI Journal, pp.219-231, March-April 1986.