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Scientific paper

Experimental and Analytical Study of Structural Performance of RC Share Walls with Corroded Reinforcement

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Abstract

In this study, we made examinations by means of experimental and analytical methods for the purpose of grasping the shear characteristics of RC walls with corroded reinforcement and verified the validity of analytical models and constitutive laws by making comparisons between the analysis results and the test results. First of all, we conducted two kinds of element tests on the bond strength of reinforcement greatly affected by corroded reinforcement and tension stiffening of concrete around the reinforcement. Regardless of the diameter of the reinforcement, we found that the bond strength could be assessed using past equations and, for tension stiffening, proposed an equation in consideration of the effect of corrosion weight loss by obtaining data of constitutive laws for FEM analyses. Then, we conducted in-plane shearing tests on the RC walls with corroded reinforcement and found that the effect of the corrosion weight loss of reinforcement on the shear strength of the walls was low. Finally, we found that the analyses almost follow the test results through FEM analyses of the RC share walls with corroded reinforcement using the constitutive laws obtained through element tests and past constitutive laws.

1. Introduction

Carbonation of concrete and chloride penetration are regarded as degradation factors considered in the evaluation of long-term soundness of structures in nuclear facilities (AIJ 2008). Since these factors cause corrosion of reinforcement in concrete, many studies have been made on to prevent reinforcement corrosion.

On the other hand, in recent years, a lot of studies have been made on the influence of corrosion of reinforcement on the structural performance of RC structures, and experimental and analytical studies have been made on allowable amount of corrosion for structural performance.

For example, some studies have been made on the

mechanical properties of corroded reinforcement, such as tensile strengths and yield points, by Lee *et al.* (1995), and Iwanami *et al.* (2002) showing their major variations due to the effect of asymmetry/non-uniformity of the cross sections of reinforcement and pitting corrosion. For the bond performance between the corroded reinforcement and concrete, the JCI (1998) is organizing past studies on pullout tests for the relation with the corrosion weight loss ratio and bond strength, reporting that there is a correlation between the corrosion weight loss and bond strength ratio, that in some cases corroded reinforcement is higher than sound reinforcement in bond strength in the range of a corrosion weight loss of approx. 1% to 3%, that the bond strength gradually decreases in the range over a corrosion weight loss of approx. 3%, and that in many cases the bond strength ratio becomes lower than 0.2 when above approx. 10%. There is also a decrease in tension-stiffening effect as an effect on RC members due to decreased bond strength. Experimental and analytical examinations have been made on the mean stress-strain relation of concrete in RC with corroded reinforcement by Shimomura *et al.* (2000), Matsuo *et al.* (2001) and Kato *et al.* (2003). They reported when bond performance between reinforcement and concrete were deteriorated due to corrosion of reinforcement, the tension stiffening effect and crack distribution decreased, and when compared sound reinforcement with about 2.5% cross sectional loss of reinforcement, tensile stress borne by concrete was decreased about 50%.

As mentioned above, many knowledge about the bond performance of corroded reinforcement and concrete has been accumulating, however data on large diameter reinforcement such as D25 or more which is frequently used in nuclear facilities is not enough at present.

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Especially, the impact, as well as the mechanical properties of corroded reinforcement and the relation of bond strength between corroded reinforcement and concrete, is the effect of corroded reinforcement on the structural performance of RC members. For past findings on the effect of corroded reinforcement on the structural properties of RC members, there are many experimental and analytical studies on beam members (e.g. Uomoto 1984) but few past studies on wall members which are frequently used in nuclear facilities. An examination of RC box culverts consisting of four faceplates (wall members) was reported by Matsuo *et al.* (2008). However, it examined the problem with the out-of-plane bending shear on the walls and, as a result of the examination using linear-element beam members, reported that the decrease in shear strength as a structure is low even under the condition where corrosion weight loss exceeded 10%. As an examination with emphasis on the in-plane shear force of walls in the same way as in this paper, the only experimental study in Japan is that conducted by Matsunaga *et al.* (1994). Although Matsunaga *et al.* conducted in-plane shear tests on RC walls with corrosion weight loss of 7% to 14% and reported that there is little effect of the presence of corrosion on the strength of the walls, that walls with no corrosion are higher in deformation performance, that corroded walls tend to be higher in initial rigidity, and that exfoliation of the cover concrete rapidly decreases the yield strength. However, the test was conducted under limited conditions and technical data on behaviors with an in-plane shear force applied onto the wall members is not enough at present. Also, since no analytical method to simulate the experiment was shown, the results of the experiment are only to explain the behavior under limited conditions.

There are some examples of analyses on RC members in consideration of corroded reinforcement (e.g. Coronelli *et al.* 2002) attempted to analyze concrete beams with corroded reinforcement using the finite element method, showing that the strength of bending members with corroded reinforcement could be calculated through numerical analyses. However, we found few examples of analyzing members and bearing walls that cause shear failures (e.g. Li *et al.* 2014) and they only consider the bond properties of the reinforcement and do not consider the concrete's tensile properties.

In this study, we make examinations by means of ex-

perimental and analytical methods for the purpose of grasping the shear characteristics of RC walls with corroded reinforcement and verify the validity of analytical models and constitutive laws by making comparisons between the analysis results and the test results. First of all, in chapter 2, we conduct two kinds of elemental experiments on bond strength of large-diameter reinforcement greatly affected by corroded reinforcement and tension stiffening of concrete around the reinforcement and examine assessment methods of the bond strength and the tension stiffening to obtain data of constitutive laws for FEM analyses. Then, in chapter 3, we conduct in-plane shearing tests on the RC walls with corroded reinforcement to understand the effect of the corrosion weight loss of reinforcement on the shear strength of the walls. Finally, in chapter 4, we verify the validity of the analytical methods in comparison with the test results through FEM analyses of the RC walls with corroded reinforcement using the constitutive laws obtained through element tests and the past constitutive laws.

2. Element test using corroded reinforcement

2.1 Both ends pulling test

(1) Overview of tests

Concrete specimens which were set SD345 reinforcement of D13, D25 or D38 in it were prepared. The overview of the specimen is shown in **Fig. 1**, the dimensions of the specimen in **Table 1**, the constitutive materials of concrete in **Table 2**, and the mix proportion of concrete in **Table 3**. For the section size of the specimen, D13 was 63×63 mm that matches the thickness of cover concrete used for the member tests shown in Chapter 3. The cross section area of the concrete was 30 times of the cross section of the reinforcement. Also for D25 and D38, we determined the cross section area of the concrete that matched 30:1, the ratio of cross section area mentioned above. Effective anchorage length of reinforcement was 1400mm.

Concrete was ready mixed concrete of $F_c = 24 \text{ N/mm}^2$, and high-early-strength Portland cement was used. The specimen underwent wet curing in a room without air conditioning for 14 days and then electric corrosion.

Fig. 1 Overview of specimen. (unit: mm)

Table 1 Dimensions of specimen.

Nominal name	Thickness of cover concrete (mm)	H (mm)	Effective anchorage length (mm)
D13	25 ¹⁾	63	1,400
D25	50 ²⁾	125	
D38	75 ²⁾	188	

- 1) Determined based on 25 mm, the thickness of cover concrete of the specimen we used for the member tests in Chapter 3. The cross section of the concrete at this point was 30 times larger than that of the reinforcement.
- 2) The cross sections of the concrete were 30 times larger than those of the reinforcement.

Table 2 Materials.

Materials of construction	Symbol	Type	Density (g/cm ³)	Remarks
Cement	C	High-early-strength Portland cement of Taiheiyo Cement Corp.	3.16	
Water	W	Underground water	-	
Fine aggregate	S1	Crushed sand from Okutama-machi, Nishitama-gun	2.66	S1:S2 = 0.7:0.3
	S2	Pit sand from Obitsudai, Kimitsu-shi	2.57	
Coarse aggregate	G	Crushed sand from Okutama-machi, Nishitama-gun	2.66	Percentage of absolute volume in 2005: 60%
Admixture	ad	Standard air entraining and water reducing agent of BASF	-	

Table 3 Mix proportion.

W/C (%)	s/a (%)	Slump (cm)	Air content (%)	Content (kg/m ³)				
				W	C	S	G	ad
57.1	50.4	18.0	4.5	184	323	886	883	3.23

Table 4 Rough calculation of energizing time.

Target weight loss (mg/cm ²)	10	50	100
Integrated amperage (A·h/cm ²)	0.02	0.10	0.20
Current density (mA/cm ²)	0.85		
Energizing time (h)	23.5	117.5	235.3

The reinforcement was corroded by electric corrosion which was applied until the target corrosion weight loss per unit surface area of reinforcement reached 0 mg/cm² (with no corrosion), 10 mg/cm², 50 mg/cm², and 100 mg/cm². The electric corrosion was provided by immersing the specimen in a 3% NaCl solution and applying a direct current to the reinforcement in the specimen. With a current density of 0.85 mA/cm², the application of direct current was shut off when the target weight loss was reached. The relation between the corrosion weight loss and the integrated current was $W = 0.49$ It based on preliminary tests, where W: Corrosion weight loss (g), It: Integrated amperage (A·hr). **Figure 2** shows the outline of electric corrosion method of specimen. **Table 4** shows the rough calculation of energizing time.

We then determined the load-mean strain relation of the concrete through monotonous loading both ends pulling test and, from the mean stress-mean strain relation of the concrete, assessed the change in the tension stiffening effect due to the reinforcement corrosion. The stress-strain relation of the concrete was calculated using Eq. (1) (Kato *et al.* 2003).

$$\sigma_c = \frac{P - A_s E_s \epsilon}{A_c} \quad (1)$$

where σ_c : Mean stress of concrete, P: Working load, A_s : Mean cross section after reinforcement corrosion, E_s : Young's modulus of reinforcement, ϵ : Mean strain, A_c : Cross section of concrete. Equation (1) is satisfied only before the reinforcement yield. In addition, the mean cross section after reinforcement corrosion was determined by subtracting the area depending on the corrosion weight loss from the nominal cross section of the reinforcement.

The electrically corroded specimens were set up in the pressing system with frames and jacks set up and then tested. The overview of the loading test is shown in **Fig. 3**. We measured the amount of elongation of the specimens using a displacement meter via a jig attached in place in the concrete (measuring length: 1,300 mm). The mean strain of the specimens was determined by dividing the amount of elongation by 1,300 mm, the measurement length. For the electrically corroded specimens, we took reinforcement out of the specimens after the test, immersing in a 10% citric acid hydrogen diammonium solution, removing corrosion products, and measuring the weight to determine the corrosion weight loss using weight difference before and after corrosion.

Fig. 2 Electric corrosion method of specimen.

Table 5 Mechanical properties of reinforcement.

Nominal name	Yield stress (N/mm ²)	Young's modulus ($\times 10^4$ N/mm ²)	Tensile strength (N/mm ²)
D13	395.8	19.1	553.8
D25	390.5	19.0	578.2
D38	388.3	19.1	583.5

(2) Test results and discussion

Figure 4 shows the result of the compressive strength test of concrete, **Fig. 5** the result of the splitting tensile test of concrete, and **Table 5** the mechanical properties of the reinforcement.

Figure 6 is an example of the cracks of D13 specimens after the test. The figures show four faces of each specimen, and the figure on left shows specimen with no corrosion and right shows with target corrosion weight loss of 100mg/cm². The red solid line shows the crack caused by the load and the red dotted line shows electric corrosion crack. The numbers above and below the dotted line show the electric corrosion crack width, and in the absence of a number it means the crack width was

Fig. 3 Overview of pressing system.

Fig. 4 Result of compressive strength test of concrete.

less than 0.05mm. In specimens using D13 reinforcement with target corrosion weight loss of 50mg/cm² and 100mg/cm², all specimens had corrosion crack. On the other hand, in specimens using D25 or D38 reinforcement with target corrosion weight loss of 100mg/cm², all specimens had corrosion crack. Since the cover of specimen using D13 was thin, the electrolyte tended to permeate around the reinforcement. Therefore, it is assumed that concrete resistance around the reinforcement decreased and corrosion was accelerated.

Besides, as Kato *et al.* (2003) supposed, crack distribution tended to decrease in corrosion cracked specimens.

Figure 7 shows comparison between the calculated values and the measured values of the corrosion weight loss. It shows that the dispersion of the measured values is greater than that of the calculated values of the corrosion weight loss. First, the cause of the major variation would be an effect of corrosion cracks. In this test, we controlled the corrosion weight loss by integrated amperage, and the timing of occurrence and the length, width or number of corrosion cracks are different by specimens. It is expected that if cracks occur, macrocell corrosion with anodes around the cracks and the cathodes in the other areas would progress and corrosion weight loss would increase. Different timing of occurrence and length, width or number of corrosion cracks would cause

Fig. 5 Result of splitting tensile test of concrete.

D13 – No corrosion

D13 – Target corrosion weight loss = 100mg/cm²

Fig. 6 Example of cracks.

Numbers are crack widths (unit: mm)

a difference in corrosion weight loss and increasing the variations.

The load-mean strain relation obtained using D13 as a sample is shown in Fig. 8, and the mean stress-mean strain relation of concrete obtained using Eq. (1) based on the load-mean strain relation in Fig. 8 is shown in Fig. 9.

Matsuo *et al.* (2001) and Kato *et al.* (2003) showed that the mean stress of concrete decreased with the increasing of corrosion weight loss. Nevertheless, in this experiment, many specimens with target weight loss at 50mg/cm² indicated maximum mean stress. The reason for this has not been elucidated and remains as a future subject.

Figure 10 shows the ratio of the maximum mean stress determined by Eq. (1) for each diameter of reinforcement and each corrosion weight loss to the average of the maximum mean stress with no corrosion reinforcement. The black-filled markers show the specimens with corrosion cracks, and the white-filled markers show the specimens without corrosion cracks.

Figure 10 shows that the ratios of the specimens with a corrosion weight loss of around 100 mg/cm² are around

0.8 regardless of the reinforcement diameter and the values of maximum mean stress tend to decrease with the increasing of corrosion weight loss.

On the other hand, we found some cases where the maximum stress ratio with a corrosion weight loss of less than 50mg/cm² was higher than 1.0. The reason is that, in this experiment, many specimens with target weight loss at 50mg/cm² indicated maximum mean stress, as we have mentioned above, and the reason remains as a future subject.

The approximation obtained from the test result is shown as Eq. (2). Equation (2) will be used in Chapter 4 as constitutive law. Equation (2) plotted as the red line in Fig. 10. In Fig. 10, maximum stress ratio of some specimens were higher than 1.0, but in the past research, it was shown that the maximum stress ratio decreases with corrosion weight loss, and in this experiment, the reason why the maximum stress ratio exceeds 1.0 was not clear, therefore, we set the maximum value of the maximum stress ratio to 1.0.

Fig. 7 Comparison of corrosion weight loss between calculation value and measurement value.

Fig. 8 Example of load-mean strain relation.

Fig. 9 Example of mean stress-strain relation of concrete.

Fig. 10 Relation between corrosion weight loss and maximum stress ratio.

Table 6 Dimensions of specimen.

Nominal name	Length of short side (mm)	Length of long side (mm)	Bond length (mm)	Non-bond length (mm)	Thickness of cover concrete (mm)
D13	76	127	51	25	25 ¹⁾
D25	153	255	102	51	40 ²⁾
D38	228	381	152	76	40 ²⁾

1) Determined based on the thickness of cover concrete for the pressing tests of RC walls.

2) The minimum value of the thicknesses of cover concrete of the outdoor columns, beams, and bearing walls in no contact with the soil as specified in AIJ (2013a).

$$\frac{f_t}{f_0} = 1.0 \quad (C' < 50)$$

$$\frac{f_t}{f_0} = e^{-0.04C'} + e^{-0.0027C'} - e^{-0.08C'} \quad (C' < 50) \quad (2)$$

where C' is a corrosion weight loss (mg/cm^2).

Figure 10 shows that there is no clear difference depending on the diameter of reinforcement and corrosion cracks, and Eq. (2) can be applied.

Figure 11 is a plot of comparisons of the area of the curve after the occurrence of the maximum stress in mean stress and mean strain of the concrete at the time when the reinforcement shown in Fig. 9 are corroded and the area with no corrosion to assess how the fracture energy after the maximum stress ratio is decreased due to corrosion. Note that the mean stress of the concrete is normalized so that all the maximum values become 1.0 (See the right figure in Fig. 11. This value is hereinafter referred to as a normalized fracture energy reduction ratio). The normalized fracture energy reduction ratio is equivalent to the ratio that the fracture energy changes because of corrosion after cracks occur. Figure 11 shows a downward-sloping tendency in which the area decreases with an increase in corrosion weight loss, and we used Eq. (3) as a constitutive law in Chapter 4.

$$\frac{A_t}{A_{t0}} = e^{-1.57 \times 10^{-4} C'^{1.55}} \quad (3)$$

where A_{t0} is an area of stress $\times$ strain after occurrence of cracks with no corrosion and A_t is an area with corrosion.

For the cause of the increase in variation of the nor-

malized fracture energy reduction ratio in **Fig. 11** despite the same degree of corrosion weight loss, it is expected that corrosion did not develop uniformly on the surface of reinforcement but non-uniformly, which caused stress concentration due to partial pitting corrosion, resulting in differences in mean stress and mean strain of concrete. This tendency is remarkable especially for small diameter rebar, and it is considered that assuming the pitting corrosion depth is the same, the smaller the diameter of the reinforcement is, the larger the ratio of the partial loss area becomes.

2.2 Pull out test

(1) Overview of test

We conducted pull out test to determine the maximum bond strength under the condition where reinforcement corrosion decreases the bond strength and we modified the existing CEB-FIP equations (CEB-FIP 1990). The overview of the specimen is shown in **Fig. 12** and the dimensions of the specimen in **Table 6**. SD345 reinforcement of D13, D25, or D38 were set and performed electric corrosion until the target corrosion weight loss reached 0 mg/cm^2 (with no corrosion), 10 mg/cm^2 , 50 mg/cm^2 , and 100 mg/cm^2 . We then determined the relation between the bond stress and the slippage through monotonous loading pull out test. We referred to the standards of the Japan Testing Center for Construction Materials (proposed by the JMC Committee) for the configuration and the dimensions of the specimen and the bond tests by Tepfers (1982) for the reinforcement positions. For the thickness of cover concrete of the specimen, D13 was 25 mm that matched the thickness of

Fig. 11 Relationship between area ratios related to stress and strain, and corrosion weight loss.

Table 7 Mechanical properties of reinforcement.

Nominal name	Yield stress (N/mm ²)	Young's modulus ($\times 10^4$ N/mm ²)	Tensile strength (N/mm ²)
D13	391.2	18.8	548.3
D25	399.6	19.2	578.1
D38	391.3	19.3	578.8

cover concrete used for the member tests described in Chapter 3. The thicknesses of cover concrete of D25 and D38 were 40 mm from "The minimum value of the thicknesses of cover concrete of the outdoor columns, beams, and bearing walls in no contact with the soil" specified in AIJ (2013a). The constitutive materials of concrete, mix proportion, and curing of concrete were the same as those for the both ends pulling test. The electric corrosion method is shown in Fig. 13. The electrolyte used, applied current density, and integrated amperage were also the same as those for the both ends pulling test. The force was applied using a 200-t universal tester. The overview of the loading test is shown in Fig. 14. The slippage of the reinforcement was measured using a displacement meter via a jig attached to the concrete. For the electrically corroded specimens, we took reinforcement out of the specimens after the test and cut only bond part out, immersing bond part in a 10% citric acid hydrogen diammonium solution, removing corrosion products, and measuring the weight to determine the corrosion weight loss using weight difference before and after corrosion. The weight of reinforcement before

corrosion was calculated by measuring the weight per unit of each reinforcement beforehand and multiplying by the length of the bond part after cutting.

(2) Test result and discussion

The result of the compressive strength test of concrete is shown in Fig. 15, the result of the splitting tensile test of concrete in Fig. 16, and the mechanical properties of the reinforcement in Table 7. In addition, Fig. 17 shows the graph of comparisons between the calculated values and the measured values of the corrosion weight loss. Although the measured values varied widely, many specimens were higher than target weight losses. It is likely that the reason why corrosion progressed is that in the specimens with corrosion crack, current flowed out to the environment from cracks, and in the specimens with no corrosion crack, since some specimens were corroded at the top of the bond part covered with insulating tape, current flowed out to the environment from there unintentionally.

Figure 18 is a plot of the bond strength ratios for each reinforcement diameter and each corrosion weight loss

Fig. 12 Overview of pull out test specimen.

Fig. 13 Electric corrosion method.

Fig. 14 Overview of loading test.

Table 8 Test specimen.

Test specimen No.	Target Corrosion quantity (mg/cm ²)	Web wall reinforcement ratio p_s (%)	Shear span to depth ratio M/QD	Shear strength [JEAC4601 equation] (kN)	Flexural strength (AIJ 2013b) equation (kN)	Shear strength to Flexural strength ratio
CSW-1	0	1.01 (2-D13 @ 125)	0.71	2113	3097	0.68
CSW-2	20		0.71	2110	3094	0.68
CSW-3	200		0.71	2082	3069	0.68

ratio with the average of the maximum load of the specimens with no corrosion defined as 1.0. The black-filled markers show specimens with corrosion cracks. Regression 1 and Regression 2 in Fig. 18 show Eqs. (4) and (5), respectively, which express the relation between the bond strength ratio and the corrosion weight loss ratio described in the literature (JSCE 2006).

$$\text{Regression 1: } \frac{\tau_b}{\tau_{b0}} = e^{-0.0607 \cdot Cr} \quad (4)$$

$$\text{Regression 2: } \frac{\tau_b}{\tau_{b0}} = e^{-1.2220 \cdot Cr} + e^{-0.1641 \cdot Cr} + e^{-2.8188 \cdot Cr} \quad (5)$$

where τ_b/τ_{b0} : Bond strength ratio, Cr: Corrosion weight loss ratio (%).

Regardless of diameter of reinforcement, the regression equation 1 is close to the test results, so we used Eq.

(4) as the constitutive law for the bond strength in Chapter 4. In addition, it also matches these test results that the bond strength is higher than that of a sound reinforcement in some cases (JSCE 2006) where the corrosion weight loss is within the range of approx. 1 to 3%.

3. Shearing tests on the RC walls

3.1 Test planning

(1) Overview of test specimen

The configuration of the test specimen and the reinforcement arrangements are shown in Fig. 19 and the test specimen is listed in Table 8. The test specimens with the shape of an I section consist of shear walls (I-section web walls), flexural walls (I-section flange walls) on both sides of them, a pressing stub at the wall top, and a footing stub at the wall footing.

We use the reinforcement corrosion quantity in the

Fig. 15 Result of compressive strength test of concrete.

Fig. 16 Result of splitting tensile test of concrete.

Fig. 17 Comparison of corrosion weight loss between calculation value and measurement value.

Fig. 18 Relationship between corrosion weight loss ratio and bond strength ratio.

shear walls as the test specimen parameter. We prepare three test specimens: CSW-1 with no corrosion; CSW-2 with a target corrosion quantity of 20 mg/cm^2 that is two times the criteria (at which point corrosion cracks occur), 10 mg/cm^2 , shown in the Guidelines (AIJ 2008) for maintenance and management; and CSW-3 with a target corrosion quantity of 200 mg/cm^2 that is much higher than the criteria. The specifications other than the corrosion quantity are common to all the test specimens.

The test specimens have dimensions and reinforcement arrangements so that the flexural strength of walls is higher than the shear strength. For dimensions, the shear walls have an inside length of 1,500 mm, an inside height of 1,000 mm, and a wall thickness of 200 mm, whereas flexural walls have a wall length of 1,000 mm

and a wall thickness of 250 mm. For concrete, the walls are $F_c = 24 \text{ N/mm}^2$ (an actual strength of approx. 30 N/mm^2), and pea gravel with a maximum grain size of 10 mm is used as coarse aggregate. On the other hand, the pressing stub and the footing stub are $F_c = 80 \text{ N/mm}^2$ to prevent cracks. For the reinforcements, the shear walls are 2-D13@125 (SD295) for both vertical and horizontal reinforcements and the wall reinforcement ratio is $\rho_s = 1.01\%$. For the flexural walls, the horizontal reinforcements are 2-D13@125 (SD295), the vertical reinforcements are 2-D16@200 (SD390). The mechanical properties of the concrete and the reinforcements are shown in **Table 9** and **Table 10**, respectively.

For the test specimens, we fabricated web walls first, and then, as shown in **Fig. 20**, made them corrode forci-

Fig. 19 Configuration of test specimen and reinforcement arrangement. (unit: mm)

Table 9 Mechanical properties of concrete.

Specimen	Part	Compressive Strength σ_B (N/mm ²)	Strain at compressive strength ϵ_m (μ)	Young's modulus E_c ($\times 10^4$ N/mm ²)	Poisson's ratio	Tensile strength σ_T (N/mm ²)
CSW-1	Web wall	40.6	2983	2.288	0.190	2.81
	Flange wall	36.6	2301	2.518	0.199	3.01
	Pressing stub	102.8	3765	3.521	0.203	5.67
	Footing stub	109.2	3543	3.822	0.217	6.08
CSW-2	Web wall	39.6	2916	2.369	0.180	2.60
	Flange wall	35.5	2311	2.466	0.188	2.88
	Pressing stub	100.0	3404	3.690	0.227	5.80
	Footing stub	104.0	3408	3.770	0.211	5.24
CSW-3	Web wall	40.2	2764	2.404	0.174	2.77
	Flange wall	35.7	2308	2.576	0.191	2.80
	Pressing stub	104.3	3488	3.802	0.223	5.79
	Footing stub	106.5	3392	3.854	0.219	5.47

Table 10 Mechanical properties of reinforcement.

Steel type	Material	Yield stress	Tensile strength	Yield ratio	Yield strain		Young's modulus	Breaking elongation	Application
		σ_y (N/mm ²)	σ_u (N/mm ²)	σ_y / σ_u	σ_y / E_s (μ)	$\epsilon_{y, up}^{(1)}$ (μ)	$E_s^{(2)}$ ($\times 10^5$ N/mm ²)	ϵ_u (%)	
D13	SD295A	342.8	488.1	0.70	1773	1832	1.934	27.37	Vertical/Horizontal reinforcement of web wall
D13	SD295A	350.0	513.4	0.68	1874	2045	1.868	28.79	Horizontal reinforcement of flange wall
D16	SD390	472.6	647.7	0.73	2344	2438	2.016	22.75	Vertical reinforcement of flange wall

(1) Strain at upper yield point (2) Demanded by the least square method using data between $0.1\sigma_y$ and $0.7\sigma_y$.

bly through electrolytic corrosion using the vertical and horizontal reinforcements protruding from the web walls. The corrosion cracks caused by the electric corrosion are shown in **Fig. 21**. The CSW-2 test specimens with a corrosion quantity of 20 mg/cm² that is two times the quality of the criteria in the Guidelines for maintenance

and management showed no corrosion cracks, whereas the CSW-3 test specimens caused corrosion cracks in the entire walls. After the corrosion of the web walls, we made reinforcement arrangements in the order of a footing stub, flange walls, and a pressing stub, and poured concrete.

Fig. 20 Overview of electric corrosion methods for web walls. (unit: mm)

Table 12 Test result list.

Test Case	Loading direction	Initial stiffness ¹⁾		Maximum shear force	
		eK_i [kN/10 ⁻³ rad]	Ratio to CSW-1	eQ_{max} [kN]	Ratio to CSW-1
CSW-1	+	3645	-	3264	-
	-	3446	-	-3093	-
CSW-2	+	3662	1.00	3266	1.00
	-	3503	1.02	-3029	0.98
CSW-3	+	3107	0.85	3088	0.95
	-	3388	0.98	-3041	0.98

1) Secant stiffness connecting between the points where shear cracks occurred in the web walls and the origin in the relationship between the shear force and the shear deformation angle

3.2. Test Results and Discussion

(1) Test Process and Damage

The test environment is shown in Fig. 25, the load-deformation diagram in Fig. 26, and the damage to the web walls in Fig. 27. In addition, a comparison of the initial stiffness and a comparison of the largest strengths in the relationship between loading and deformation are shown in Table 12. For each test specimen, flexural cracks occurred in flange walls first and then in web walls, decreasing the stiffness in the relationship between loading and deformation. Afterwards, the vertical reinforcements of the flange walls and the reinforcements of the web walls in the wall footing yielded, reaching the maximum strength. After the maximum strength was reached, shear cracks in the web walls were spread as deformation increased, and the horizontal load decreased.

In comparison of the CSW-1 with no corrosion and the CSW-2 with a corrosion quantity of 20 mg/cm², there was no clear difference between the relationship between loading and deformation, the occurrence of cracks, and

the final failure properties. On the other hand, for the CSW-3 with a corrosion quantity of 200 mg/cm², the initial stiffness decreased down to 0.85 times lower on the positive side and the maximum strength decreased down to 0.95 times lower on the positive side compared to the CSW-1 with no corrosion. This would be due to the

Fig. 24 Loading history.

Fig. 23 Pressing system.

corrosion cracks; however, we found no big difference between the final failure properties.

(2) Discussion on Maximum Strength

Table 13 shows the relationship between various evaluation equations for the in-plane shear strength of shear walls and the maximum load obtained through the test. In application of the evaluation equations, we made evaluations in consideration of the effect of corroded reinforcement as a reduction of cross-sections of reinforcement. The test values were 1.13 to 1.21 times the evaluation equations for the in-plane shear strength of the walls in JEAC4601 (2008) used in the nuclear power field, showing that the JEAC evaluation equations evaluate the test values conservatively. As evaluation equations for shear strength other than the JEAC results, we made evaluations using the Hirose (1975) equations common in the Japanese construction field and the ultimate strength guiding equations of the Architectural Institute. It showed that each evaluation equation evaluates the test values conservatively. In addition, for every specimen, the test value exceeded the calculated value of 2,724 kN in the case where corroded reinforcement was not considered, and as a result of the range of this test, it would be almost not necessary to be concerned about a decrease in shear strength for any corrosion with a corrosion weight loss of approx. 200 mg/cm².

(3) Discussion on Hysteresis Characteristics

Figure 28 shows the relationship between the envelope curve, which is in the relationship between the shear forces and the shear deformation angles, and the hysteresis characteristic values calculated using the equations provided by JEAC4601. For CSW-3 with corrosion cracks, the initial stiffness was lower than the other specimens as described in (1) above, but the deviation from the calculated value was not so large in **Fig. 28**. On the other hand, the shear crack strength, τ_1 , and the shear strength, τ_3 , of the calculated value were mostly underestimated compared to the ones of the test and, for the skeleton curve provided just after the first break point, the test value exceeded the calculated value. This result shows that the calculated hysteresis characteristic values are evaluated to be safe.

3.3 Conclusion on experimental study on shear wall

To understand the effect of reinforcement corrosion quantity on the structural performance of reinforced concrete nuclear facilities, we did structural performance tests on shear failure I-section type walls with their reinforcement corrosion quantities defined as a test parameter. As a result, we found that there was no structural effect as a bearing wall when the reinforcement corrosion quantity was about two times the critical corrosion quantity in a case where corrosion cracks occurred, 10 mg/cm², described in the maintenance guideline of the Architectural Institute of Japan (2008). We also found

Fig. 25 Test environment.

(a) CSW-1

(b) CSW-2

(c) CSW-3

Fig. 26 Shear force – deformation angle relationship.

Table 13 Comparison between test results of maximum strength and various strength equations.

Test Case	Experimental value		Calculated values					
	Maximum shear force		JEAC4601		Hirosawa eq.		AIJ eq.	
	Loading direction	eQmax (kN)	Qsu1 (kN)	$\frac{eQ_{\max}}{Q_{su1}}$	Qsu2 (kN)	$\frac{eQ_{\max}}{Q_{su2}}$	Qsu3 (kN)	$\frac{eQ_{\max}}{Q_{su3}}$
CSW-1	+	3264	2724	1.20	2738	1.19	2941	1.11
	-	-3093		1.14		1.13		1.05
CSW-2	+	3266	2704	1.21	2703	1.21	2910	1.12
	-	-3029		1.12		1.12		1.04
CSW-3	+	3088	2683	1.15	2711	1.14	2913	1.06
	-	-3041		1.13		1.12		1.04

that when the reinforcement corrosion quantity greatly exceeded the critical corrosion quantity and reached about 20 times the critical corrosion quantity, corrosion cracks occurred and, partly due to its effect, the initial stiffness decreased, but the shear strength of the walls had a strength higher than the preexisting equations, such as those in JEAC4601.

4. Analytical study of the shear strength of RC walls with corroded reinforcement

(1) General description of analysis

We simulated the RC wall tests through FEM analyses. For the analytical program, we used CARC-ASE

Fig. 28 Comparison of envelope curve.

Green lines show corrosion cracks; black lines show the state at positive loading(+); red lines show the state at negative loading(-)
Fig. 27 Damage to the web wall.

(Morikawa *et al.* 2006) developed by Kajima Corporation. This program uses the non-orthogonal four-direction crack models of Maekawa *et al.* (Fukuura 1999; Bernhard 1999) supporting repeated-loading. The stress-strain relation of concrete is shown in **Fig. 29**. The compression side of the stress-strain relation uses the Fafitis-Shah model (Fafitis *et al.* 1985) and the tension stiffening after the occurrence of cracks uses the model of Izumo *et al.* (1987) shown in Eq. (6).

$$\sigma = f_t (\varepsilon_t / \varepsilon)^{C_t} \quad (6)$$

where f_t is the tensile strength, and ε_t is the strain with the tensile strength applied. C_t is a coefficient showing the tension stiffening properties. The Collins model (Vecchio and Collins 1982) was used for reduction of compressive strength caused by cracks, and the Habasaki model (Habasaki *et al.* 2000) for shear transfer characteristics of the cracked surfaces. For the stress-strain relation of reinforcement, the Zulficar-Filippou model (Zulficar 1990) was used. The stress-strain relation of reinforcement is shown in **Fig. 30**. For the relationship between the shear force and the slip of bond elements connecting concrete and reinforcement, we used a model that was expanded from the Morita and Fujii's model (Morita *et al.* 1984) to consider degradation and repetition. **Figure 31** shows the relationship between the bond stress and the slip of concrete and reinforcement.

(2) Analytical model

The analytical model is shown in **Fig. 32**. Using a symmetric condition, we modeled the 1/2 part in the thickness direction of the web wall. A web wall was modeled with a four-node shell element, a flange wall with an eight-node solid element, and a reinforcement with a rod element. In addition, for the bonding between concrete and reinforcement, the web wall reinforcement and the flange wall vertical reinforcement were connected with bond elements, and the flange wall horizontal reinforcement were rigidly connected.

The loading conditions and the boundary conditions are shown in **Fig. 32**. For the force application, we applied a given axial force, and then conducted static cyclic-loading by the displacement control at both ends of the pressing stub.

(3) Material properties

For the material properties of concrete with no reinforcement corrosion, we used the results of the material tests in **Table 9**. For the compressive strength and the Young's modulus, we used the results of the material tests. The tensile strength was calculated using the equation of the Japan Society of Civil Engineers ($= 0.23f_c^{2/3}$) (JSCE 2007). Based on Okamura's method (Okamura 1991), we used 0.4 as the coefficient C_t showing the tension stiffening properties. In addition, modeling and material properties of CSW-2 and CSW-3 with corroded reinforcement are described in next section. The softening properties after the compressive strength were set so as to

be approximated to the fracture energy model of Nakamura *et al.* (2001).

For the material properties of the reinforcement, we used the material properties in **Table 10**. The second stiffness was 1/1,000 of the initial stiffness.

The bond characteristics of concrete and reinforcement were calculated using the CEB-FIP equation (CEB-FIP 1990).

In addition, we used upper and lower stubs with elasticity.

Fig. 29 Stress-strain relation of concrete.

Fig. 30 Stress-strain relation of reinforcement.

Fig. 31 Relationship between bond stress and slip of concrete and reinforcement.

(4) Modeling of the RC walls with corroded reinforcement

For the effect of wall reinforcement corrosion, we considered the reduction of the tensile strength and the tension stiffening characteristics of concrete, the reduction of the bond strength between wall reinforcement and concrete, and the reduction of the cross section of wall reinforcement based on element experiments. In addition, we reduced the cross sections of the wall reinforcement depending on the amount of corrosion. The reduction of the tensile strength and the tension stiffening characteristics of concrete caused by reinforcement corrosion were considered only for the outside of the wall reinforcement, and the inside of the wall reinforcement was modeled as sound concrete. The web walls were modeled with shell elements in which two shell elements were placed in the same node: the shell element in consideration of the effect of corrosion and the shell element of sound concrete. The image of the modeling of the RC walls with corroded reinforcement is shown in Fig. 33. Note that the model was made under the symmetric condition in the direction of the web wall thickness; therefore, the 1/2 part was modeled in the direction of web wall thickness.

For the concrete with reinforcement corrosion, the tensile strength was reduced by Eq. (2). In addition, the coefficient C_t , which showed tension stiffening properties in Eq. (6), was changed from the sound concrete ($C = 0.4$) so that the normalized fracture energy reduction ratio determined by Eq. (3) became A_f/A_{f0} . For the cor-

rosion weight loss used for reduction, we decided to use the mean value of the corrosion weight losses on the surfaces A and B, which were measured from the reinforcement taken out by chipping after the experiments described in Table 11, and the values of the web wall's horizontal reinforcement with a large quantity of corrosion accordingly. Each reduction ratio is shown in Table 14, and the comparison of the tensile characteristics of the sound concrete and the concrete with corroded reinforcement in Fig. 34. For the tensile characteristics of the concrete with corroded reinforcement based on the proposed equations obtained by the element experiments, CSW-2 was low even in reduction because of the normalized fracture energy reduction ratio with no reduction of the tensile strength, whereas CSW-3 showed a reduction of the tensile strength to approx. 1/2 and of the normalized fracture energy reduction ratio to approx. 40%.

For the reinforcement of the web walls, we reduced the cross sections of the reinforcement depending on the corrosion weight loss ratios of the horizontal reinforcement and the vertical reinforcement.

For the bond characteristics of the reinforcement and the concrete of the web walls, we reduced the bond strengths by Eq. (4) depending on the corrosion weight loss ratios of the horizontal reinforcement and the vertical reinforcement. Table 15 shows the bond strength list.

In the element tests, the maximum corrosion weight loss used in the both ends pulling test for assessment of tension stiffening characteristics was approx. 150

Fig. 32 Analytical model.

Fig. 33 Modeling of concrete in consideration of reinforcement

Table 14 Tensile characteristics of concrete in consideration of reinforcement corrosion.

Specimen	Condition	Corrosion weight loss (mg/cm ²)	Corrosion weight loss ratio (%)	Tensile strength reduction ratio	Tensile strength f_t (N/mm ²)	Normalized fracture energy reduction ratio	Tension stiffening factor C_t
CSW-2	Sound	—	—	—	2.7	—	0.40
	Corrosion considered	28	1.1	1.0	2.7	0.97	0.41
CSW-3	Sound	—	—	—	2.7	—	0.40
	Corrosion considered	272	10.9	0.48	1.3	0.39	0.55

Table 15 Bond strength list.

Specimen	Condition	Part	Corrosion weight loss (mg/cm ²)	Corrosion weight loss ratio (%)	Bond strength reduction ratio	Bond strength (N/mm ²)
CSW-2	Sound	Vertical/Horizontal reinforcement	—	—	—	15.7
	Corrosion considered	Vertical reinforcement	17	0.7	0.96	15.1
		Horizontal reinforcement	28	1.1	0.93	14.6
CSW-3	Sound	Vertical/Horizontal reinforcement	—	—	—	15.9
	Corrosion considered	Vertical reinforcement	223	8.9	0.58	9.2
		Horizontal reinforcement	272	10.9	0.52	8.2

mg/cm², and the maximum corrosion weight loss ratio used in the pull out test for assessment of bond characteristics was approx. 6%. Therefore, for CSW-3, it was necessary to extrapolate from the results of the element tests for all the assessment equations.

(5) Simulation analysis result

The comparison of the maximum strengths is shown in **Table 16**, the final failure mode in **Fig. 35**, and the comparisons of the relations between loads and shear deformation angles in **Fig. 36**. Note that the views of the cracks in the final fracture mode were enlarged with the crack at the first integral point as the element center. In the analysis result of CSW-1 with no corrosion, the differences in maximum strengths are within 10% on both positive and negative sides. Therefore, the analysis can

evaluate the maximum strengths. For the relation between the shear force and the shear deformation angle, the analysis result shows greater deformation in the secondary stiffness after a decrease in stiffness, including the cyclic property, but almost follows. The fracture mode also shows greater damage in the proximity of the center in the direction of the wall height, which is even lower than the experimental results, but follows.

Meanwhile, CSW-2 and CSW-3 with corrosion also almost follow the maximum strengths and the relation between the shear force and the shear deformation angle, which can be considered to be assessed with the accuracy similar to that of CSW-1 with no corrosion. Therefore, the modeling method and the material constitutive laws of the RC walls with corroded reinforcement we used in this study would allow us to analyze RC earthquake

Fig. 34 Comparison of tensile characteristics.

Table 16 Comparison of maximum strengths.

Specimen	Loading direction	Experimental (kN)	Analytical (kN)	Comparison (Analytical/Experimental)
CSW-1	Positive side	3264	3402	1.04
	Negative side	-3093	-3306	1.07
CSW-2	Positive side	3266	3330	1.02
	Negative side	-3028	-3282	1.08
CSW-3	Positive side	3088	3208	1.04
	Negative side	-3041	-2984	0.98

Table 17 List of reduction parameters and maximum strengths.

Parameter	Maximum Strength (kN)	Ratio to no corrosion case
All reduced (CSW-3)	3208	0.95
No corrosion (No reduction at all)	3374	1.00
Tensile strength of concrete	3246	0.96
Tensile strength and tension stiffening characteristics of concrete	3240	0.96
Wall reinforcement diameter	3292	0.98
Bond strength	3334	0.99

resisting walls with corroded reinforcement.

(6) Effect of Each Parameter for Corrosion

We examined the effect of each parameter for corrosion using the simulation analytical model of CSW-3. We used the five cases for the examination: with no corrosion, reduction of the concrete's tensile strength only, reduction of the tensile strength and the tension stiffening characteristics only, reduction of the wall reinforcement diameter only, and reduction of the bond strength only. **Table 17** shows the list of the analyzed cases and the reduction parameters. We applied the reduced parameter values used in the analyses of CSW-3. Note that as it was based on CSW-3, the material characteristics and the loading patterns of concrete differed from those of CSW-1 and the result of the case with no corrosion slightly differed from the analysis result of CSW-1. The comparisons with the maximum strengths are also shown in **Table 17**. The table also shows the ratio of the maximum strength to the case with no corrosion. In addition, **Fig. 37** shows the case with no corrosion and the case with only the tensile strength and the tension stiffening characteristics reduced as examples of load-shear

deformation angle relation. The strength of the case with no corrosion is approx. 5% higher than that of the analysis result of CSW-3 with all reduced. On the other hand, for the effect of each parameter for corrosion, the reduction of concrete's tensile strength showed the highest effect, resulting in a reduction of 4%. In contrast, the reduction of bond strength showed the lowest effect, resulting in a reduction of approx. 1%.

(7) Comparison with Past Corrosion Models

We conducted analyses using the concrete's tensile strength reduction equations and the tension stiffening reduction models described in JSCE (2006) as analytical methods for concrete members with corroded reinforcement, and compared the results with those of this analytical model based on the element tests. Note that the concrete's tensile strength reduction models and the tension stiffening models used in JSCE (2006) are not generalized but they are often used by the Japan Society of Civil Engineers, which tends to conservatively evaluate, that is, overestimate the reduction in strength due to corrosion. For this reason, JSCE (2006) limited the reduction in concrete's tensile strength due to corrosion up to 1.0 N/mm² based on the past test results. Eq. (7) shows the reduction equation of the concrete's tensile strength due to corroded reinforcement in JSCE (2006).

$$f_t = \exp(-2.77 \cdot Cr / 100) \cdot f_{t0} \quad (7)$$

where Cr is the corrosion weight loss ratio of reinforcement (%), f_{t0} is the tensile strength of sound concrete. As mentioned above, JSCE (2006) limited the reduction in concrete's tensile strength due to corrosion to 1.0 N/mm², but this study did not use limitation values for reduction. **Table 18** shows the list of the tensile strengths obtained by Eq. (7) and those obtained by the approximate equation in the element tests. The tensile

Fig. 35 Final failure mode.

Fig. 36 Comparison of relations between loads and shear deformation angles.

Fig. 37 Relationship between load and shear deformation angle.

strength of CSW-2 was not reduced for the approximate equation resulting in 2.7 N/mm^2 , whereas it was reduced

to 2.0 N/mm^2 for Eq. (7). The tensile strength of CSW-3 was 1.3 N/mm^2 for the approximate equation, whereas it was 0.1 N/mm^2 for Eq. (7), resulting in a nearly-zero tensile strength. For the tension stiffening characteristics of concrete with corroded reinforcement, JSCE (2006) showed the values in **Table 19** based on Matsuo *et al.* (2002). According to **Table 19**, $C_t = 0.4$ is used for the corrosion weight loss ratio of CSW-2, which is the same as that for sound concrete, and $C_t = 2.0$ for that of CSW-3.

We conducted analyses using these values of the tensile strength and the tension stiffening coefficient C_t . The other parameters were the same as those used in the simulation analyses using the approximate equation. **Table 20** shows the comparison of the calculated tensile strengths between the approximate equation and the past equation. The list of the maximum strengths obtained

Table 18 Tensile strength of concrete in consideration of corroded reinforcement.

Specimen	Parameter	Corrosion Weight Loss (mg/cm ²)	Corrosion Weight Loss Ratio (%)	Tensile Strength Reduction Ratio	Tensile Strength (N/mm ²)
CSW-2	No corrosion				2.7
	With corrosion	28	1.1	1.0	2.7
				0.74	2.0
CSW-3	No corrosion				2.7
	With corrosion	272	10.9	0.48	1.3
				0.05	0.1

Table 19 Tension stiffening characteristics in JSCE (2006).

Mass Reduction Ratio C_r (%)	Tension Stiffening Coefficient C_t
$C_r \leq 2$	0.4
$2 < C_r \leq 5$	0.6
$5 < C_r \leq 8$	0.8
$8 < C_r \leq 10$	1.0
$10 < C_r$	2.0

Table 20 Tensile characteristics used.

Specimen	Parameter	Tensile Strength (N/mm ²)	Tension Stiffening Coefficient C_t
CSW-2	No corrosion	2.7	0.40
	Approximate equation	2.7	0.41
	Past equation	2.0	0.40
CSW-3	No corrosion	2.7	0.40
	Approximate equation	1.3	0.55
	Past equation	0.1	2.0

from the analysis results is shown in **Table 21**, and the relationship between the shear force and the shear deformation angle obtained from the analytical models using the past equation is shown in **Fig. 38**. **Table 21** also shows the simulation analysis results using the approximate equation in the element tests and the ratios to them. In CSW-2, the maximum strength obtained from the analytical models using the past equation was 3,262 kN, resulting in 2% lower than the maximum strength of 3,330 kN in the case of using the approximate equation in the element tests. On the other hand, in CSW-3, the maximum strength obtained from the analytical models using the past equation was 3,182 kN, resulting in 1% lower than the maximum strength of 3,208 kN in the case

of using the approximate equation in the element tests. In both cases, the maximum strengths were slightly reduced in the case of using the past equation, showing more conservative evaluation results. There was no significant difference in the relationship between the load and the shear deformation angle compared to the simulation analysis results.

(8) Discussion

For the specimens with corroded reinforcement, we considered reduction of tensile strength and reduction of tension stiffening characteristics of concrete, and reduction of bond strength between the wall reinforcement and concrete based on the element tests. In addition, we re-

Fig. 38 Relationship between load and shear deformation angle (Past equation).

Table 21 Comparison of maximum strengths.

Specimen	Parameter	Maximum Strength (kN)	Ratio to Approximate equation case
CSW-2	Approximate equation	3330	1.00
	Past equation	3262	0.98
CSW-3	Approximate equation	3208	1.00
	Past equation	3182	0.99

duced the cross-sections of the wall reinforcement depending on the amount of corrosion. We applied the reduction of the tensile strength and the tension stiffening characteristics of concrete to the outer concrete of the wall reinforcement and applied the proposed equations obtained through the element tests. As a result, we found that the test results could almost be simulated regardless of the presence of corrosion.

We examined the degree of the effect of each factor for the effect of corrosion and the result showed that the concrete's tensile strength had the greatest effect on shear walls that caused shear failures. In contrast, the bond strength showed the lowest effect. The reason why the bond strength had a low effect would be the wall reinforcement completely anchored with the right and left flange walls and the upper and lower stubs. The analyses of the concrete's tensile characteristics of concrete for concrete members with corroded reinforcement using the past equations showed a slightly greater reduction in strength compared to the proposed equations, which gave conservative evaluation results. Both were reduced by several percent only compared to the specimens with no corrosion, and the corrosion did not lead to a major reduction in strength.

Using the analytical models and the constitutive laws used in this study would allow us to analyze even RC shear walls of different corrosion weight loss and different size of reinforcement.

4. Summary

In this study, we make examinations by means of experimental and analytical methods for the purpose of grasping the shear characteristics of RC walls with corroded reinforcement and verify the validity of analytical models and constitutive laws by making comparisons between the analysis results and the test results.

By both ends pulling test, the ratios of the specimens with a corrosion weight loss of around 100 mg/cm^2 were around 0.8 regardless of the reinforcement diameter and the values of maximum mean stress tended to decrease with the increasing of corrosion weight loss. In addition, the normalized fracture energy reduction ratio showed a downward-sloping tendency with an increase in corrosion weight loss. By pull out test, regardless of diameter of reinforcement, the result of the relationship between bond strength ratio and corrosion weight loss ratio was almost close to previous studies.

By shearing tests on the RC walls, we found that there was no structural effect as a shear wall when the reinforcement corrosion quantity was about two times the

critical corrosion quantity in a case where corrosion cracks occurred, 10 mg/cm^2 , described in the maintenance guideline of the Architectural Institute of Japan. Also, we found that when the reinforcement corrosion quantity greatly exceeded the critical corrosion quantity and reached about 20 times the critical corrosion quantity, corrosion cracks occurred and, partly due to its effect, the initial stiffness decreased, but the shear strength of the walls had a strength higher than the preexisting equations. Based on the result mentioned above, it was revealed that there was no major change in structural performance when the corrosion weight loss ratio was about 10% or less, which was studied in this experiment.

We simulated the RC wall tests through FEM analyses with corroded reinforcement using the constitutive laws obtained through element tests and past constitutive laws, and found that the test results could almost be simulated regardless of the presence of corrosion. We examined the degree of the effect of corrosion and the result showed that the concrete's tensile strength had the greatest effect on shear walls that caused shear failures. In contrast, the bond strength showed the lowest effect. The shear strength with corroded reinforcement was reduced by several percent only compared to the specimens with no corrosion, and the corrosion did not lead to a major reduction in strength.

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