# BENDING SHEAR EXPERIMENT OF REINFORCED CONCRETE BEAMS WITH CORRODED REINFORCING BARS

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## Abstract

The direction towards extending the life of buildings has already become a common concept. Today we need to assume that concrete structures can stand one or two centuries, far longer than the traditional average life of 40 years. As a result, "aged" structures can more and more inevitably encounter earthquakes, and in the future, it will be more and more important to estimate the degree of aging deterioration of structures. In general, the structural life of a concrete structure is influenced by internal corrosion due to rainwater or moisture in the air intruding into the concrete through cracks. A decrease in the cross-section of reinforcing bars (rebars) has the most direct and significant effect on the life of a concrete structure. The level of corrosion of reinforcing bars and of reduction in their cross-sectional area that generates a problem is called a "significantly degraded state" in the deterioration stage of a concrete structure. The structural performance can be degraded in earlier deterioration stages. At present, however, research in the corrosion of rebars mainly focuses on the aspect of materials, while research in its influence on the structural performances is not enough. Moreover, studies of the initial stage of deterioration as described above are significantly lacking. In addition, studies on the relationship between the corrosion and the mechanical behavior of reinforced concrete members are mainly for longitudinal bars related to the bending strength of concrete members, and there are few studies on the corrosion of shear reinforcement bars. Since shear reinforcing bars are located outside bending reinforcing bars in the member, the corrosion conditions are more severe for the shear rebars. Therefore, it is considered that a decrease in the shear strength decreases due to corrosion of the shear reinforcing bars as corrosion degrades the bending strength of bending reinforcing bars.

The purpose of this experiment is to examine the difference in structural performance depending on the corrosion degree of the shear reinforcing bars. Four specimens with the same figure were prepared. The shear reinforcing bars were corroded by the electrolytic corrosion. The degrees of the corrosion were made varied by changing the period current flows. Specifically, the corrosion rates (the percentage value obtained by dividing the mass of corroded reinforcing bars by the mass of the reinforcing bars before corroded) of the shear reinforcing bars were set to 0%, 10%, 15%, and 25%. Such specimens with the rates were named No.1, No.2, No.3, and No.4, respectively. A bending shear was then applied to the four specimens to examine how their mechanical properties and cracking properties change.

According to the experimental results, the specimens with corroded reinforcing bars exhibited higher rigidity and strength in the range of the member deformation angle less than 1/250 than the standard specimen without corroded reinforcing bars regardless of the corrosion degree of rebars because corrosion increases the bond stiffness of rebars. In the range where the deformation of specimens is larger (the member angle more than 1/250), the specimens No.3 and No.4 with a high degree of the corrosion showed smaller tolerance than the standard specimen with normal ones. Furthermore, the cracks did not expand in specimens of smaller deformations, while they expanded rapidly as the deformation increased.

*Keywords: reinforced concrete beam, electrolytic corrosion, bending shear experiment, corroded reinforcing bar*

## 1. Introduction

It is said that the main cause why the structural performance of the reinforced concrete structures is degraded is corrosion of the reinforcing bars due to the aging deterioration. However, in the field of structures, there has been no studies on structural performance deterioration due to corrosion of the reinforcing bars. Therefore, the authors considered that the deterioration of the bonds between the shear reinforcements and the concrete would occur if the shear reinforcements were corroded and then conducted the bending shear experiments on the beams without bonds between the shear reinforcements and the concrete [1, 2]. From these experiments, the authors clarified that the bond deterioration of the shear reinforcements affects the mechanical performance of the reinforced concrete members. In this paper, the bending shear experiment was conducted on the reinforce concrete beams that corroded reinforcing bars and then changes in their mechanical behavior were examined.

## 2. Outline of experiment

The purpose of the experiment is to examine changes in bending shear properties of the beams with the corroded shear reinforcing and the main reinforcing bars. Four beam specimens (No.1 to No.4) having a rectangular cross section as shown in Fig.1 were prepared. All specimens have the same conditions except for the amount of corrosion of the reinforcing bars (see Table 1). For convenience of the following explanation, the four sides of the specimen will be called East, West, South, and North.

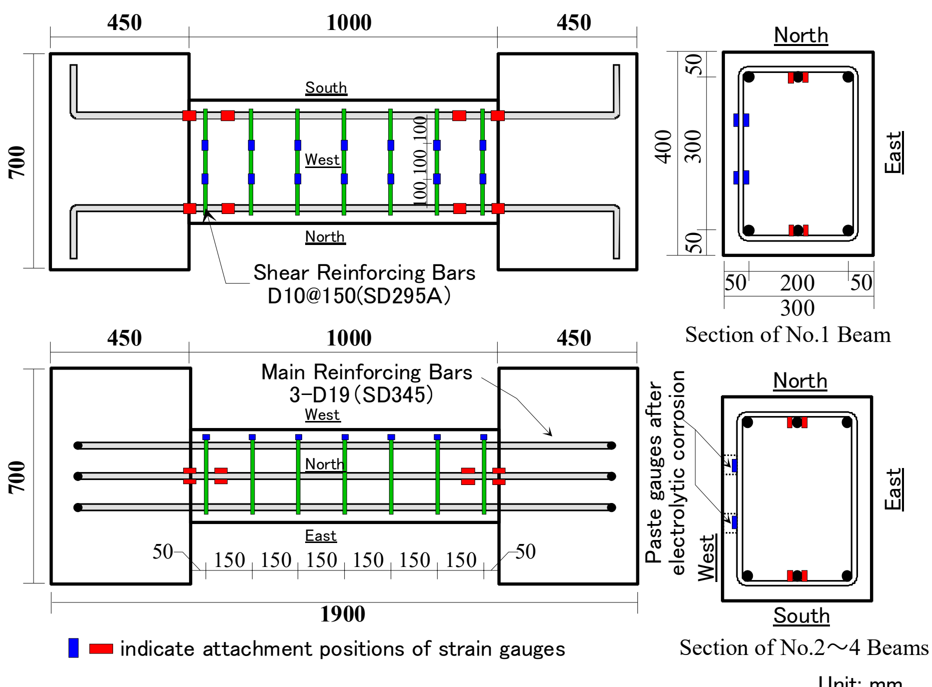



Fig.1 Shapes of Specimens

The specimens were designed with a shear margin of approximately 1.0. Specifically, the ultimate bending strength obtained using “Mu = 0.9・d・at・sσt (d: effective depth, at: cross-sectional area of tensile reinforcement, sσt: yield strength of tensile reinforcement)”, the shear strength obtained from the Arakawa Mean Formula [3], and the shear strength of calculated by the shear strength equation described in “Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept” (hereinafter referred to “Inelastic Displacement Design Guidelines”) [4] were designed to be almost the same shear strength (the shear margin is nearly equal 1.0.). However, as will be described in detail later, because the concrete strength of the specimens and the yield strength of the shear reinforcements were high and the corrosion of the main reinforcement progressed more than expected, the shear margins of the prepared specimens were 1.17 to 1.31.

Table 1 Specimen specifications

| No. | Target corrosion rate for shear bar | Planned Strength | | | | Material Strength | | | Actual Specimen | | | | | | Experimental value | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $Q_{Mu}$ | Shear Strength | | $Qu2/Q_{Mu}$ | Con. | Reinforcement | | Corrosion Rate | | $Q_{Mu}$ | Shear Strength | | $Qu2/Q_{Mu}$ | Max Strength (Qmax) *2 | $Qmax/Q_{Mu}$ |
| | | | Qu1 | Qu2 | | | Main bar *1 | Shear bar | Main bar | Shear bar | | Qu1 | Qu2 | | | |
| | (%) | (kN) | (kN) | (kN) | | (N/mm2) | | | (%) | (%) | (kN) | (kN) | (kN) | | (kN) | (%) |
| No.1 | --- | 187 | 215 | 194 | 1.04 | 26.8 | 396 | 385 | --- | --- | 215 | 260 | 263 | 1.22 | 250 | 116 |
| No.2 | 5 | | 213 | 190 | 1.02 | 29.3 | | | 4.4 | 4.9 | 206 | 266 | 269 | 1.31 | 253 | 123 |
| No.3 | 8 | | 212 | 188 | 1.01 | 29.8 | | | 5.3 | 8.4 | 204 | 266 | 238 | 1.17 | 236 | 116 |
| No.4 | 13 | | 210 | 184 | 0.99 | 30.0 | | | 6.4 | 17.1 | 201 | 262 | 260 | 1.29 | 229 | 114 |

**Common specifications**
Section : 300 x 400 (mm)
Length : 1,000 mm
Shear spanratio : 1.42
Ming reinforcements : 3-D19, SD345 (Yield strain = 2144 $\mu$ )
Shear reinforcements : 2-D10@150, SD295A (Yield strain = 2005 $\mu$ )
Shear reinforcement ratio : 0.32% (No.1)

*1 Each main bar (12 places in total) was cut out from the stub in a range of 150 mm, and their corrosion rates were averaged.
*2 The average value of the maximum strength of the positive loading and the negative loading.

The corrosion of the reinforcing bars was electrolytic corrosion generated by flowing a constant direct current of $5A/m^2$ to them. To generate electrolytic corrosion, the positive electrode was directly connected to the shear reinforcements, and the steel materials (the copper plates) with a low ionization tendency were used for the negative electrode. Usually, in order to energize between the two electrodes, the specimen and the negative electrode are immersed in a bath filled with an electrolytic solution (salt water). However, in this experiment, since the specimens were relatively large and all specimens need to generate electrolytic corrosion only to the beam at the same time, it was difficult to perform a method of directly immersing the specimens in the electrolytic solution. Therefore, as a simpler method, the copper plates were brought into close contact with each of the four surfaces of the specimen, these were wrapped with cloth and fastened with PB bands from the outside. In addition, salt water was circulated from the top of the specimen, and the specimen was energized within salt water always present between the copper plates and the specimen (see Fig.2 and Photo.1). The energization time to the specimens was obtained by conducting a preliminary experiment with the Tamori equation [5] and correcting its coefficient, $W = 0.383 \cdot It$ (W: amount of corrosion, It: total amount of current).

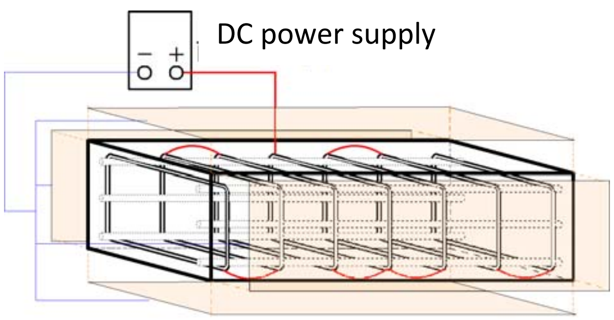



Fig.2 Electrolytic corrosion circuit diagram

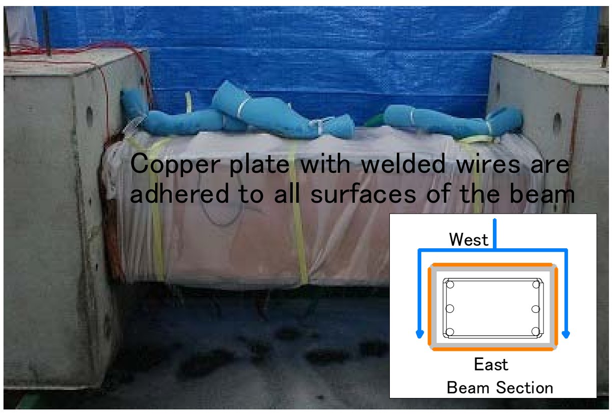



Photo.1 State of the electrolytic corrosion



Fig.3 and 4 show the distribution and amounts of the corrosion. Amounts of corrosion were determined by measuring the mass of the reinforcing bars removed corrosion and comparing them with non-corroded reinforcing bars. At this time, the reinforcing bars were taken out from the concrete after loading and immersed in the diammonium hydrogen citrate having a mass concentration of 10% to remove corrosion. In the preliminary experiment, it was confirmed that the corrosion amounts of the shear reinforcements can be expected to be almost the same at all four faces. However, in this experiment, amounts of the corrosion were large on the upper side (West) directly pouring water during generating electrolytic corrosion, and amounts of the corrosion were small on the lower side (East). This is thought to be because the copper plate was applied from the lower side on the East face and it was separated from the concrete surface by its own weight. Here, for the evaluation of the shear strength after corrosion the shear reinforcement ratio (pw) which is obtained from the corrosion ratio (average value of East and West) of all the shear reinforcements in the cross section of the depth direction was used. As shown in Table 1, Fig.3 and Fig.4, the corrosion ratios of the shear reinforcements for the specimens from No.2 to No.4 were 4.9%, 8.4% and 17.1% respectively. And those of the main reinforcements were 4.4%, 5,3% and 6.4% respectively for three specimens.

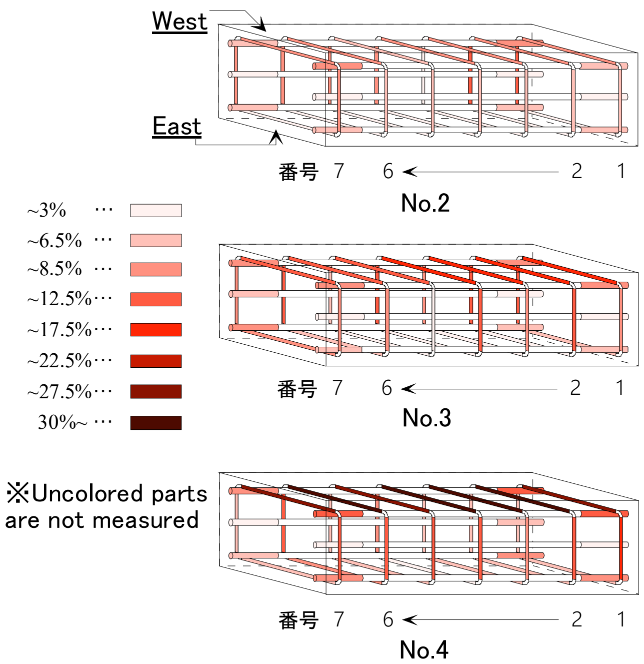



Fig.3 Electric corrosion results

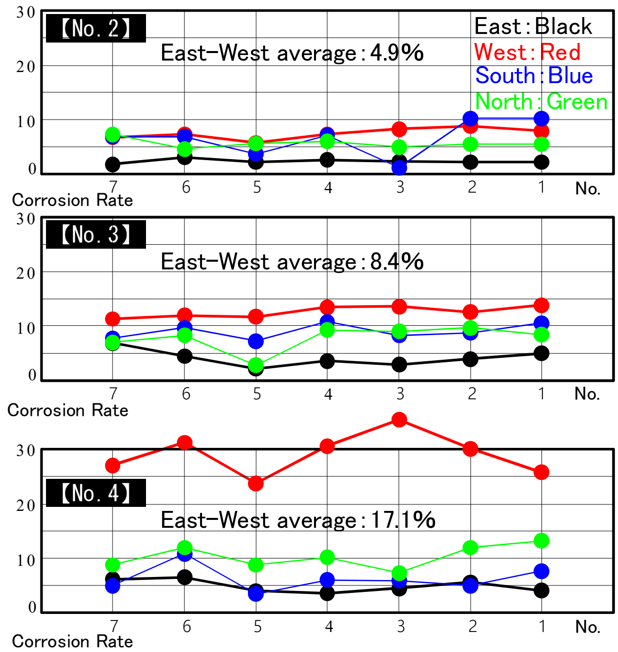



Fig.4 Electric corrosion results graphs

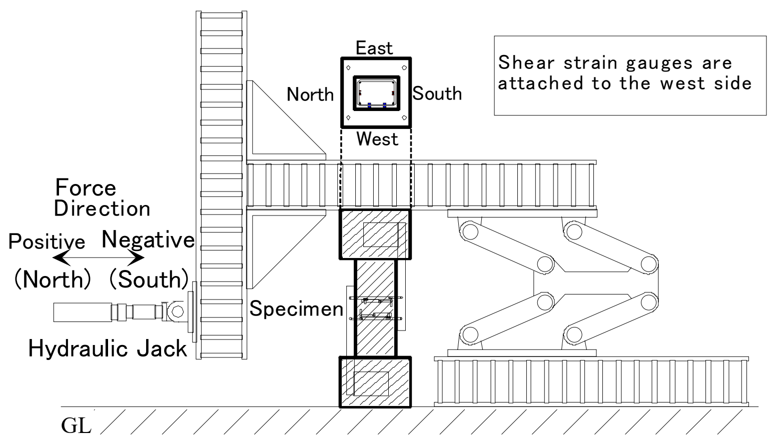



Fig.5 Sketch of loading device



Fig.5 shows the loading situation in the bending shear experiment. The loading histories were the positive/negative alternating loadings with each member deformation angle of R = 1/2000, 1/1000, 1/500, 1/250, 1/130, 1/100, 1/67 and 1/50. The direction in which the specimen is loaded by pulling is defined as the positive loading. Here, the member deformation angle (R) is a value obtained by dividing the relative displacement of the upper and lower stubs by the beam length. The crack width was measured by using a digital microscope. When a crack occurred, a mark was placed at the position where the crack width was the largest and it at the same point was measured thereafter. At the same time, the pictures were taken with a digital camera to observe the progress of the cracks. The strain of the reinforcing bars was measured by using a foil gauge. For all the specimens, the strain of the main reinforcing bars was measured at the point marked ■ shown in Fig.1. The foil gauges for all the specimens were affixed before placing concrete. Similarly, the strain of the shear reinforcing bars was measured at the point indicated by ■ in Fig.1. The foil gauges in the No.1 specimen were affixed before placing concrete, and those of the No.2, No.3 and No.4 specimens were after placing concrete.

Table 1 shows the material strengths and the calculated shear strengths etc. of the specimens. The shear margin (Qu/Mu) of the specimens at the time of planning was approximately 1.0. The strengths of the concrete (Fc18) were approximately 30 N/mm$^2$ and the yield strength of the shear reinforcement (SD295) was 385 N/mm$^2$, so that the shear margins were larger values of about 1.2 to 1.3. In addition, the ratio of the shear reinforcement strength to the ultimate shear strength calculated by the Arakawa Mean Formula was as small as about 40%, so that the effect of deterioration of the shear reinforcement was hard to be reflected in the mechanical properties of the specimens.

## 3. Results of experiment

The results of the experiment are described separately in the range where the member deformation is small (member deformation angle R ≤ 1/250) and the large range (R> 1/250). From the obtained experimental data, the strength and the rigidity of the member, and the crack width (cracks at the peak of each loading floor and residual cracks) were considered. The behavior of the strain values of the corroded shear reinforcements was also investigated.

### 3.1 Range where deformation angle of member is small (R ≤ 1/250)

Fig.6 shows the load-deformation relationship with a member deformation angle (R) of 1/250 or less for all specimens. In order to avoid complication of the figure, the figure shows it only at the loading floors of R = 1/500 and 1/250. From the figure, at the same deformation the load for all specimens (No.2 to 4) with corroded reinforcing bars exceeds the value of the standard specimen (No.1). When the corrosion has occurred in the main reinforcing bars, it can be said that the rigidity of the member is increased in a range where the deformation is small. In order to examine this reason, as an example Fig.7 shows a comparison of the strains on the upper main reinforcing bars at the South side of the specimens. The loading is in the negative direction (the pushing direction) and the reinforcing bars received tension. From the figure the strains of the main reinforcements are larger in all specimens from the No.2 to No.4 than the No.1 specimen. This is caused by the change in the bond between the reinforcing bar and the concrete due to the corrosion. As shown in Fig.8, “the length of the main reinforcing bar related to the crack” in the vicinity of the bending crack at the edge of the member varies depending on the bond properties. Compared to the case where corrosion does not occur in the main reinforcing bars, the maximum bond strength of the corroded reinforcing bars increases [6] and “the length of the main reinforcing bar related to the crack” becomes shorter. Even if the crack width is the same size, the strain increases. So that the stress of the main reinforcing bar increases and then the load increases.

Fig.9 shows the cracks on the West surface at R = 1/250. For No.4 specimen with the highest degree of corrosion, the East surface is also shown. As a situation of the No.4 specimen, it can be confirmed that there is no difference in the occurrence of cracks on the East and West surfaces. The other specimens were also similar. Specimens No.3 and 4 also show cracks (initial cracks) that occurred during the corrosion promotion process (blue line in the figure). The initial cracks occurred mainly along the main reinforcing bars in the axis direction of the member.

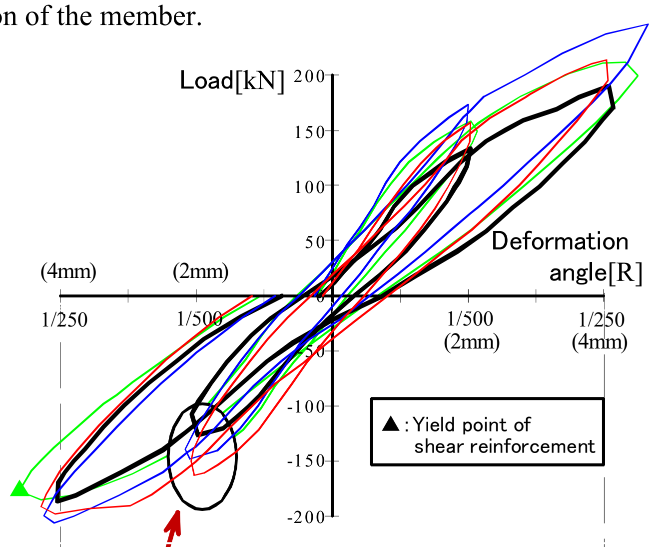



Fig.6 Load – deformation relationship (deformation angle of member is small)

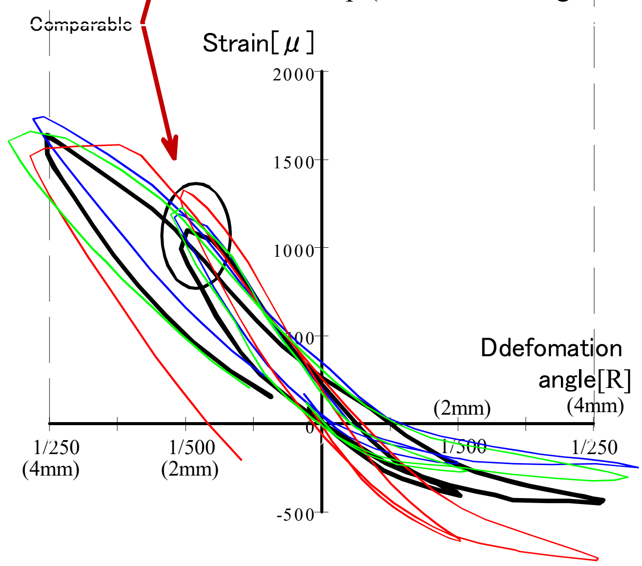



Fig.7 Strain of main reinforcing bar at south upper part

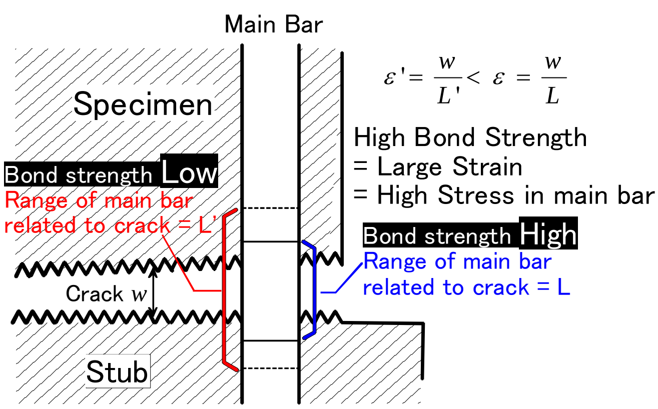



Fig.8 Conceptual figure of bond strength and reinforcement stress



As shown in Fig.9, in all specimens bending cracks and shear cracks following them occurred at the end of each member. There is no significant difference in the situation of their occurrence. Fig.10 shows the crack widths at the peak load and the zero load in each displacement stage. Cracks occurred by loading (indicated by black lines) increase in their widths with deformation of the member, though the initial cracks (indicated by blue lines) have little change. As mentioned above, the initial stiffness of the specimens with corroded reinforcements are higher than that of the No.1 specimen where no corrosion occurs. If the corrosion is about 6% or less (No.4 Specimen), it is considered that the initial cracks by corrosion along the main reinforcing bars have no significant effect on the mechanical performance deterioration of the member.

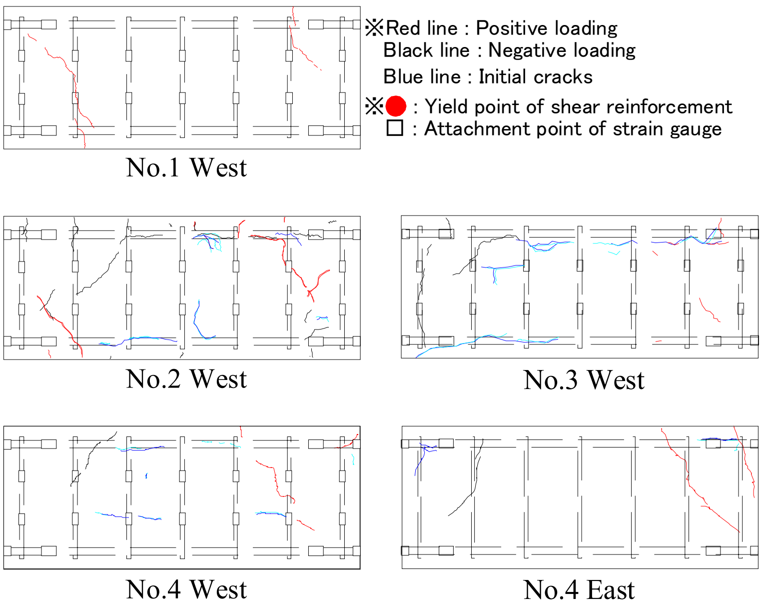



Fig.9 Crack state at R = 1/250

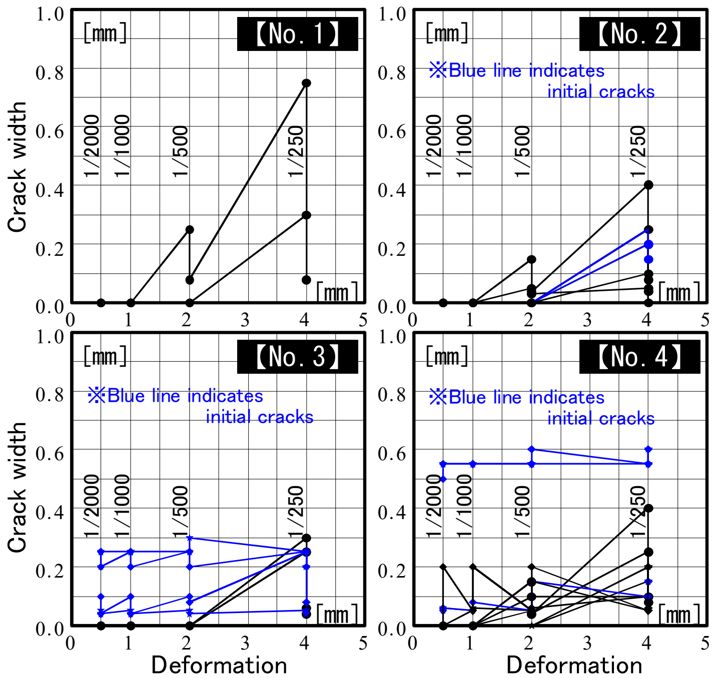


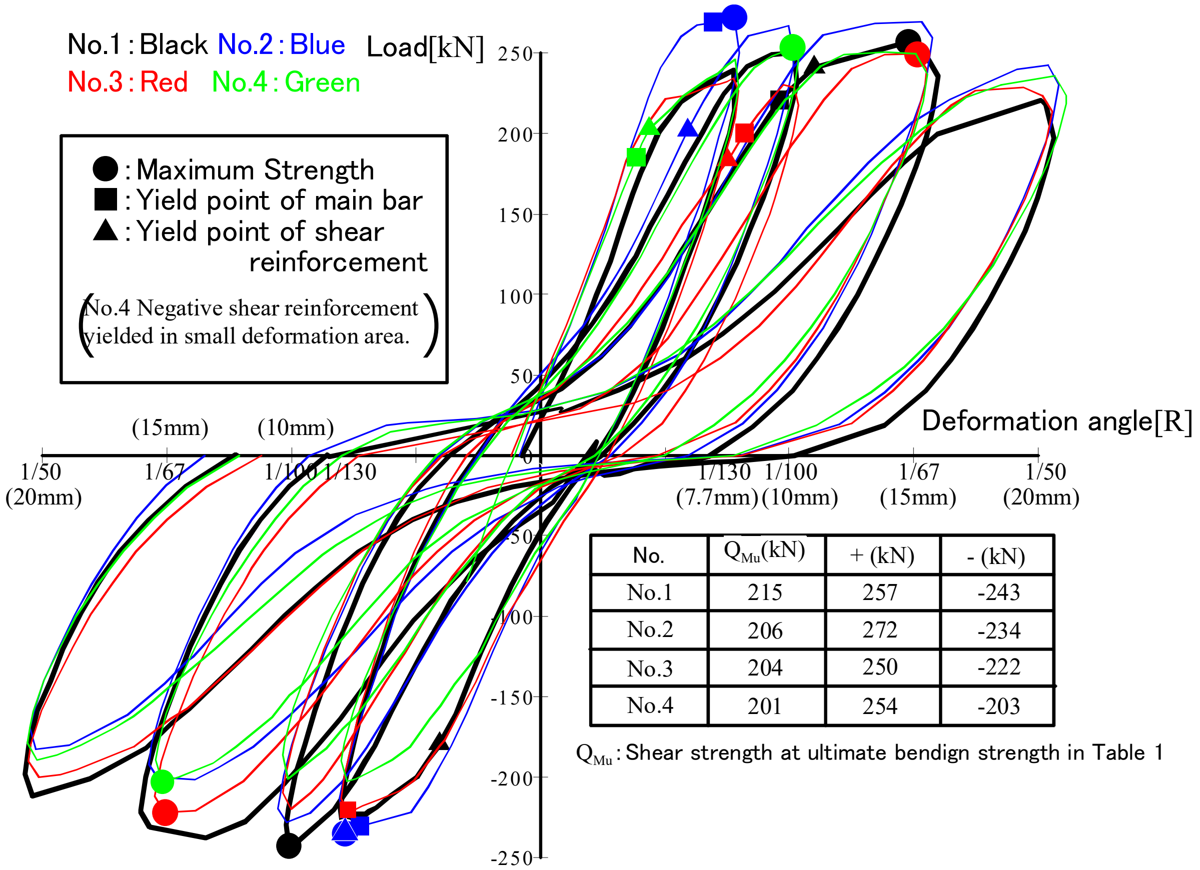

Fig.10 Crack widths at the peak load and the zero load



### 3.2 Range where deformation angle of member is large (1/250 < R ≤ 1/50)

Fig.11 shows the load-deformation relationship with the member deformation angle (R) exceeds 1/250 for all specimens. From the figure, all specimens have a history shape of reverse S-shaped slip due to the bond deterioration of the main reinforcing bars. And even for No.4 with the largest corrosion degree of the shear reinforcements, the strength of the member is not extremely reduced. As shown in Chapter 2 (refer to Table 1), it is because of that the shear margins of the specimens are as large as about 1.2 to 1.3 and the main reinforcing bars in all specimens yielded. In addition, the cause is that the concrete share of the ultimate shear strength is relatively high at approximately 60% according to the Arakawa Mean Formula.




| No. | $Q_{Mu}$(kN) | + (kN) | - (kN) |
|---|---|---|---|
| No.1 | 215 | 257 | -243 |
| No.2 | 206 | 272 | -234 |
| No.3 | 204 | 250 | -222 |
| No.4 | 201 | 254 | -203 |

$Q_{Mu}$ : Shear strength at ultimate bendign strength in Table 1

Fig.11 Load – deformation relationship (deformation angle of member is large)

In order to investigate the transition of load in more detail, Fig.12 shows the envelope curves from average values of positive and negative at the peak of each loading floor. As described in Section 3.1, the peak load on the standard specimen (No.1) is the smallest in the region where the deformation of the member is small, and the specimens with the corroded reinforcements have almost the same value regardless of the corrosion degree. When the member deformation angle (R) is greater than 1/250, a difference in the peak load appears between the corroded specimens. The values of No.3 and No.4 specimens are smaller than it of the No.2 specimen with the smallest degree of corrosion. After R = 1/130, the effect of corrosion for the shear reinforcements is observed, and those values are smaller than No.1 specimen. At R = 1/50, the peak loads of all specimens are almost the same value and there is no difference in the peak load. Fig.13 shows the crack state at R = 1/50. From the figures, cracks along the main reinforcing bars occurred in all the specimens. From this point, it is considered that the strength could not increase due to the breakage of the bond between the main reinforcements and the concrete. Therefore, it can be said that the influence of the corrosion for the shear reinforcements does not appear greatly in the value of the load.

Fig.14 shows the crack width at the peak of each loading cycle. In all the specimens, the crack widths increase as the deformation increases. Compared to the specimen No.1, the specimens with the corroded reinforcing bars have a larger increasing rate for the crack width in the range where the deformation angle exceeds R = 1/100. However, the difference in the degree of the corrosion is not clear. And as described above, though the initial cracks (blue lines in the figure) hardly change their widths in the region where the deformation is small, they very significantly increase in the deformation region exceeding R = 1/100.

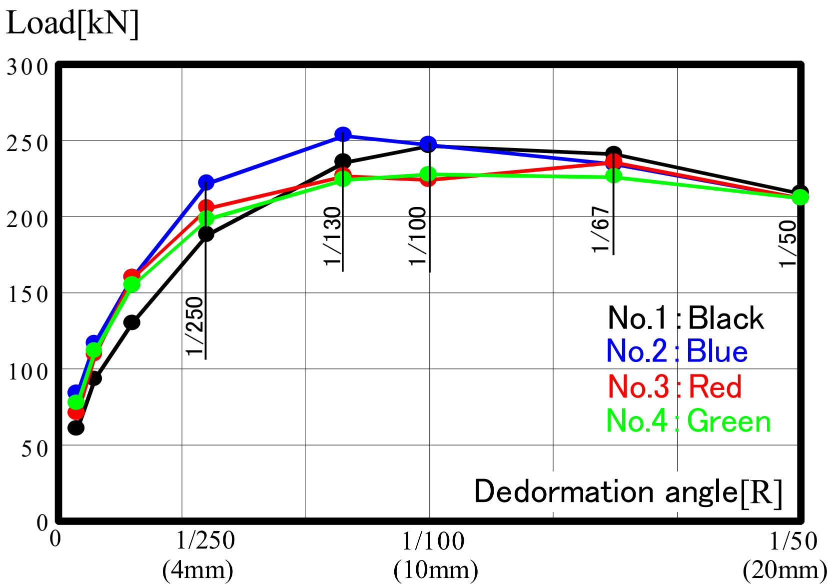



Fig.12 envelope curves from average values of positive and negative at the peak of each loading floor

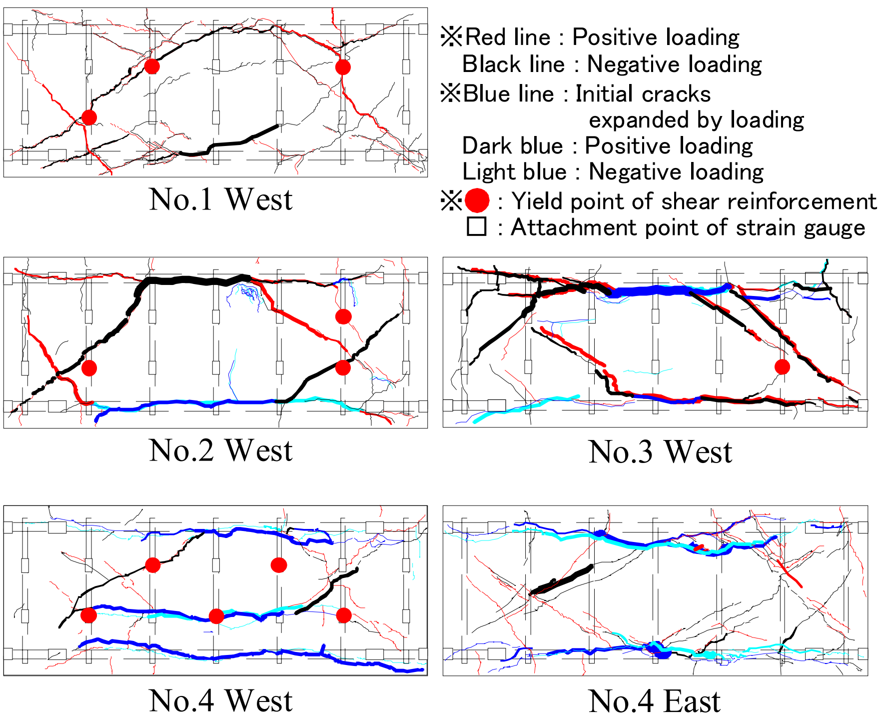



Fig.13 Crack state at R = 1/50

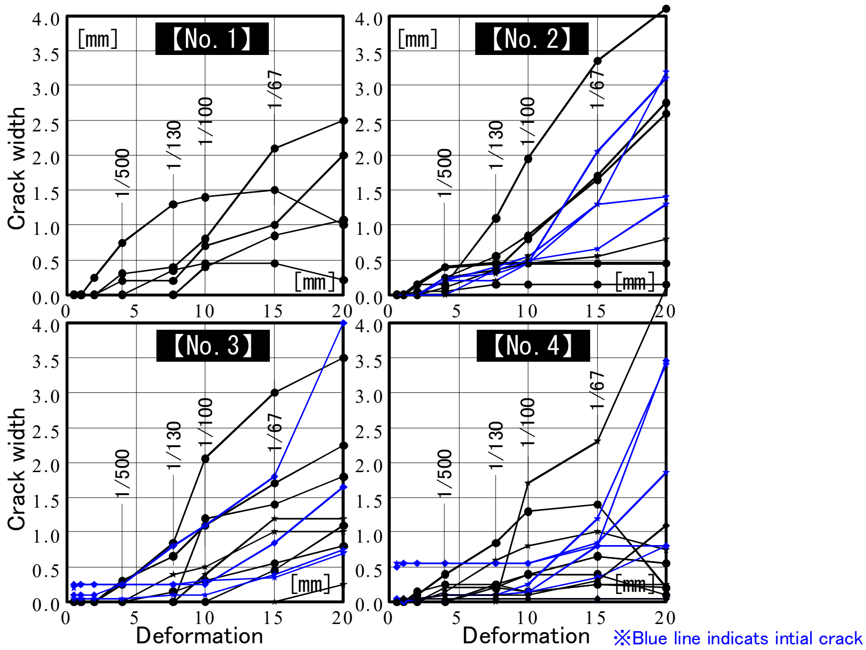



Fig.14 Crack width at the peak of each loading cycle

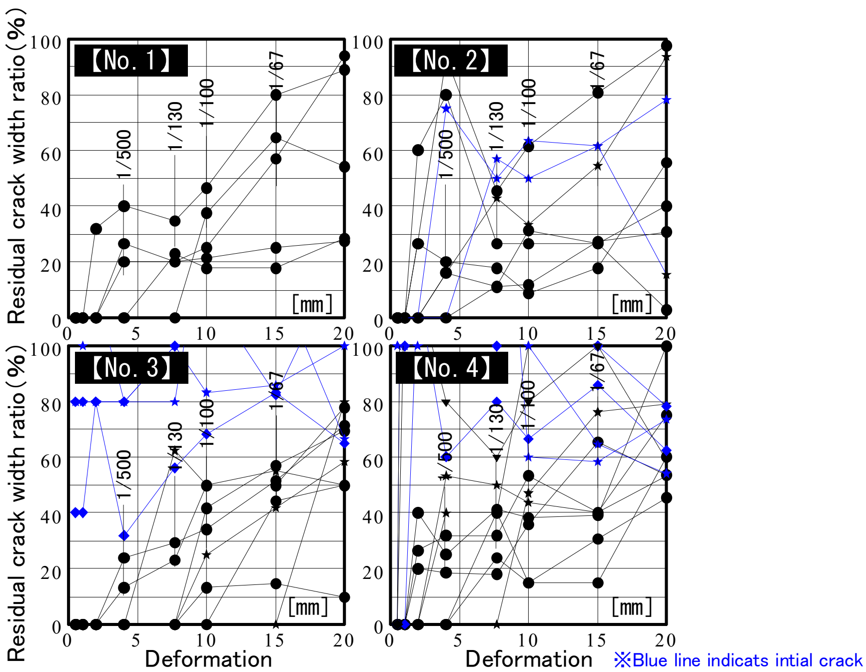



Fig.15 Residual crack width ratio

Fig.15 shows the transition of the residual crack width ratio (crack width at peak / crack width at zero load) at each loading cycle. From the figure, in the standard specimen, No.1 the residual rate increases as the deformation increases, while in the specimens with the corroded reinforcing bara the residual rate is already high in the area where the deformation is small. The initial cracks generally have larger residual rates than the cracks occurred after loading, and the residual ratio tends to be high on any loading cycle. As shown in Fig.14, since the residual ratio remains high even in the region where the deformation is large, the residual crack width increases as the crack width at the peak increases.



3.3 Difference in behavior of shear reinforcement

This section describes the behavior of the shear reinforcements of each specimen. Fig.16 shows the strain transition of the shear reinforcement at the position shown in the figure. In the figure, the horizontal axis indicates the deformation of the member, and the vertical axis indicates the strain of the shear reinforcement. This part is the place that seems to be most suitable for comparison because the shear crack crosses the shear reinforcement in all specimens. The following characteristics are observed in the behavior of the shear reinforcement.

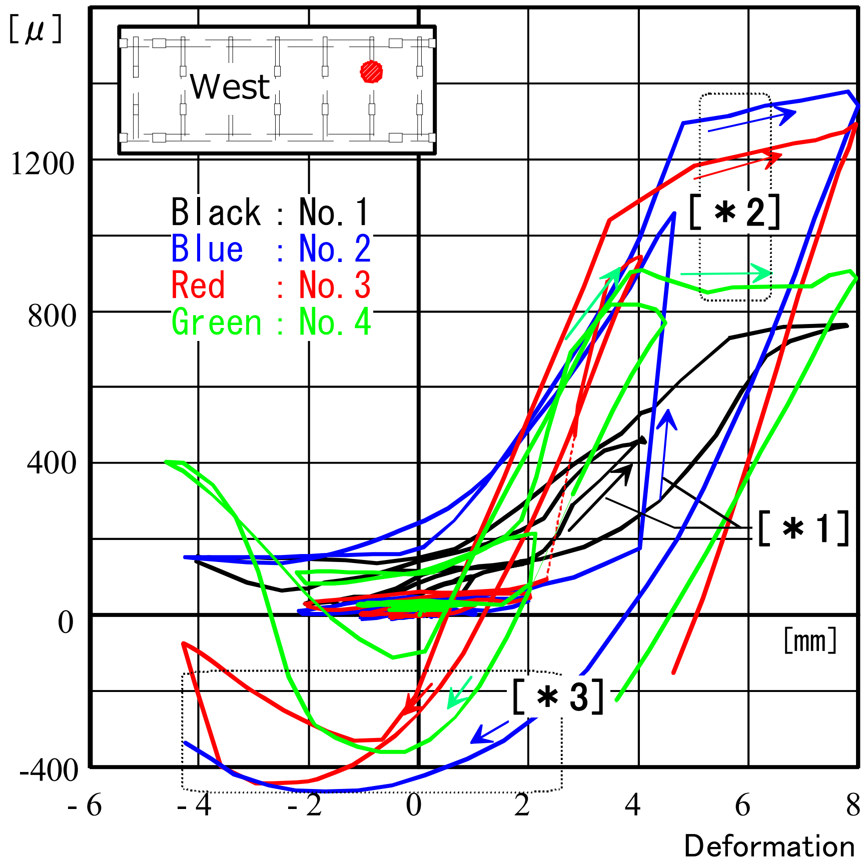



Fig.16 Strain of shear reinforcing bars

On the strains indicated by [*1] in the figure, the increase in strain immediately after the occurrence of shear cracks in No.1 and No.2 specimens is compared. With the occurrence of shear cracks, the strain of the shear reinforcement increases in both specimens. As mentioned in Fig.8 because the specimen with the corroded reinforcements has higher bond stiffness [6], the increasing rate in strain of No.2 specimen is greater than No.1 specimen. This inclination is the same for No.3 and No.4 specimens.

The strain indicated by [*2] shows the transition where the strain increasing rate decreases when the deformation is loaded from the negative side to the positive side. This reduction depends on the width of the cracks across the shear reinforcement and the state of the bond between the shear reinforcement and the concrete. No.1, No.2, and No.3 specimens have the plus inclination, but the inclination of No.4 specimen which has the highest degree of corrosion, is unstable and very small. It is considered that this is because as the member deformation increases, the slippage between the reinforcing bar and the concrete increases, and the greater the degree of corrosion the lower the bond performance.



The strain indicated by [*3] shows that in the specimens with corroded reinforcements the strain is compressed when unloaded. Usually, as shown by the No.1 specimen in the figure, the strain of the shear reinforcement is close to 0 when the shearing force acting on the member is near 0 and the strain becomes tensile again when the member is loaded to the opposite side. This is considered that because the diameter of the reinforcing bar increases when it returns from the tensile state to the original state, and the spike effect by the corrosion product acts more strongly on the corroded reinforcing bar. Since the shear reinforcement is in a compressed state, cracks across the reinforcing bar are prevented from closing. In other words, the residual crack width increases.

## 4. Conclusion

The bending shear experiment was conducted on the reinforce concrete beams that corroded reinforcing bars and then changes in their mechanical behavior were examined. The obtained findings are described below.

1) In the range where the deformation of the member is small (the member deformation angle $R \leq 1/250$ or less), the specimens with the corroded reinforcements have higher rigidity and higher strength than the standard specimen without corrosion because the corrosion of the reinforcements caused increasing of the bond stiffness.
2) In the range where the deformation of the member is large (the member deformation angle $R > 1/250$), the strength of No.3 and No.4 specimens with high degree of corrosion were lower than the specimen without corrosion.
3) The initial cracks do not expand in the range where the deformation of the member is small, but there are some that expand rapidly when the deformation becomes large.
4) After occurring crack, the behavior of the corroded shear reinforcement is higher in the rate of strain increasing than without corrosion and is compressed state when unloading. In the case of high corrosion degree, as increasing in crack width, the increasing rate of the shear reinforcement strain becomes small and unstable. It is considered that this is because the bond performance of the reinforcement is degraded.

## 5. References

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