# INFLUENCE OF DESALINATION ON BOND BEHAVIOR BETWEEN CONCRETE AND REINFORCING STEEL

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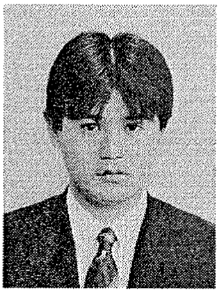

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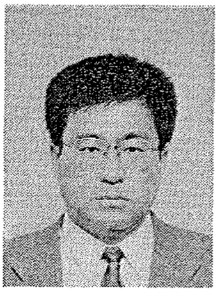

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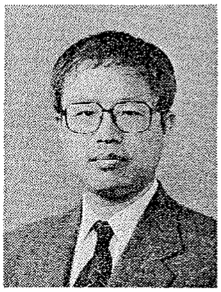

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Recently, some electrochemical approaches have been developed as repair methods for reinforced concrete structures deteriorated by carbonation of concrete, or chloride corrosion of steel bar reinforcement. Desalination is an electrochemical method aimed at removing chlorides from reinforced concrete structures. However, few reports have been published concerning its availability. This paper mainly describes the influence that the premixed chlorides amount and the passing of electric current have on the bond behavior between concrete and steel bar reinforcement.

***Keywords:*** *desalination, electrochemical technique, migration of ions, pull-out test, bond stress, Vickers hardness, flexural test of beam*

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## 1. INTRODUCTION

It has been considered that with adequate design and construction reinforced concrete structures were extremely durable. However, recently, early age deterioration of such structures caused by chloride attack has become a serious problem. Desalination is an electrochemical repair method aiming to remove chloride ions ($Cl^-$) contained in the concrete structures. Currents used for desalination are approximately two orders of magnitude higher than those used for cathodic protection, which is the most popular electrochemical method at present. However, desalination has an advantage over other methods in that it is only temporary and not permanent like cathodic protection.

The mechanism of desalination is shown in Fig. 1. The electrochemical removal of $Cl^-$ is accomplished by placing an anode and electrolyte on the concrete surface and passing direct current between this anode and the reinforcing steel, which acts as a cathode. Since anions migrate toward the anode, it is possible to remove $Cl^-$ from the concrete structure and, at the same time, cations ($Na^+$, $K^+$ etc.) migrate toward the reinforcing steel.

On the other hand, the problems of desalination are pointed out as the possible bond degradation between steel bars and concrete, acceleration of alkali aggregate reaction and hydrogen embrittlement of high strength steel bars [1]. However, few reports have been published concerning such problems.

In this research, effectiveness of removing $Cl^-$ and the influence of treatment on bond behavior between concrete and reinforcing steel bars are mainly investigated by applying desalination to reinforced concrete specimens contaminated with chlorides.

## 2. TEST PROGRAM

This project consists of chemical analysis, measurement of half-cell potential, measurement of micro-hardness distribution, pull-out test, (which all use specimens for pull-out test after applying desalination), and flexural test of RC beams (flexural and shear type). The test program outline is shown in Table 1. All values of current density and total current density are referenced to the surface of steel bars because, in this study, mainly the phenomena occurring at the interface between concrete and reinforcing steel is investigated.

### 2.1 Preparing Specimens and Passing Electric Current

a) Mix proportion of concrete
Mix proportion of concrete is shown in Table 2. Cement type was ordinary portland cement and refined salts (99% NaCl) were used as premixed chlorides. When chlorides were mixed, the equal weight fine aggregate was decreased.

b) Pull-out test specimen characteristics
Specimens for pull-out test were based on 'Method of Test for Bond Strength between Concrete and Steel Reinforcement by Pull-out Test (draft), JSCE'. They were 10×10×10 (cm) concrete cubes with deformed steel bars D13 SD295A (JIS number) at their center. Bond length was 5.2 cm which is 4 times larger than the steel bar's diameter. Spiral steel ($\phi$ 6 mm, 2 cm pitch) was used to prevent splitting failure when the pull-out test was carried out.

c) Flexural test RC beam characteristics
The test beams had a rectangular cross section of width × full depth = 10 × 20 cm and a total length of 160 cm. A deformed bar D13 SD295A (JIS number) was arranged with effective depth of 17.5 cm in the beam section. The bar's anchorage had a 180 degree hook to prevent bond failure at the anchorage and observe the crack behavior at the mid span. Moreover, part of the beams had a lap splice of 25.4 cm long at mid span, which is referred from 'Standard Specification for Design and Construction of Concrete, JSCE' [2]. This length of lap splice corresponds to 20

**Table 1** Outline of test program

| Amount of $Cl^-$(kg/m$^3$) | 0.0 | | 4.0 | | | | | | 6.0 | | | | 8.0 | | | | | | 10.0 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Current Density(A/m$^2$) | 0.0 | 5.0 | 0.0 | 5.0 | | | | | 0.0 | 5.0 | | | 0.0 | 2.5 | | 5.0 | | | 0.0 | 5.0 | | |
| Period of Treatment(weeks) | 8 | 8 | 8 | 4 | 5 | 6 | 7 | 8 | 8 | 4 | 8 | 16 | 8 | 2 | 8 | 4 | 8 | 12 | 8 | 4 | 8 | 12 |
| Chemical Analysis | ○ | ○ | ○ | ○ | ○ | ○ | ○ | | ○ | ○ | | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | | |
| Microhardness Measurement | | | | | | | | | | | | | | | ○ | | ○ | | | | | |
| Pull-out Test | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Flexural Test of RC Beam | | | | | | | | | | | | | ○ | | | ○ | ○ | | | | | |

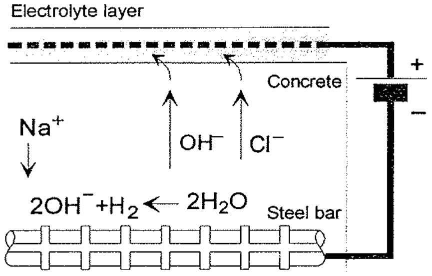
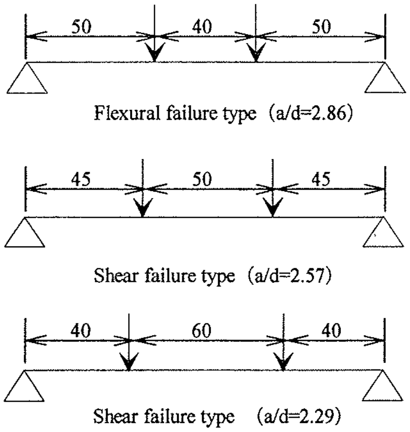


**Fig. 1** Mechanism of desalination



Flexural failure type (a/d=2.86)

Shear failure type (a/d=2.57)

Shear failure type (a/d=2.29)

Unit : cm

**Fig. 2** Loading conditions of RC beam

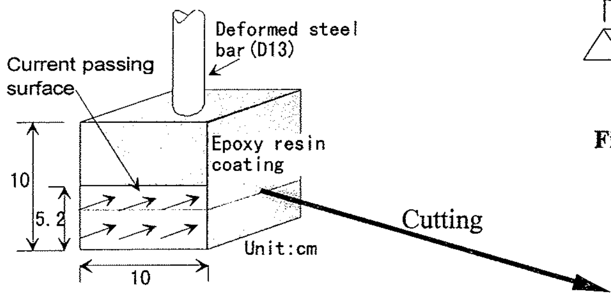


**Fig. 3** Specimen for pull-out test (Bottom surface was also coated)

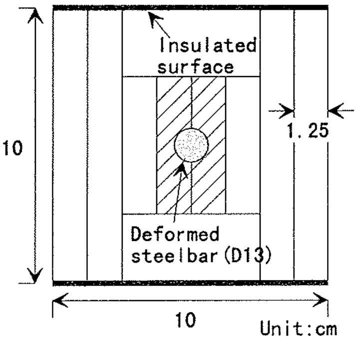


**Fig. 4** Cutting for chemical analysis

**Table 2** Mix proportion of concrete

| W/C | s/a | NMS | Unit mass(kg/m$^3$) | | | | | |
|---|---|---|---|---|---|---|---|---|
| (%) | (%) | (mm) | W | C | S | G | AEA | WRA |
| 54 | 45 | 13 | 177 | 328 | 801 | 988 | 0.82 | 1.8 |

times the nominal diameter of main reinforcement. The cover thickness at the lap splice was 1.87 cm which is larger than the 1.80 cm calculated by 'Standard Specification for Design and Construction of Concrete, JSCE' as the diameter of a steel bar which has the same section area as the total section area of lapped steel bars. Some of these beams were designed as flexural failure type with shear reinforcement, and others as shear failure type with no reinforcement to fail in shear mode. In case of flexural failure type, vertical stirrups were provided over the entire shear span at 10 cm intervals. Epoxy coated bars D10 SD295A were used as insulated stirrups and erection bars. The loading condition of test beams is shown in Fig. 2.

d) Procedure of passing electric current

Specimens for pull-out test were removed from forms at one day after concrete casting. Then, after moist air curing for 28 days in polyethylene bags, electrodes were placed around specimens which were immersed in electrolyte liquid and direct current was applied. RC beams were covered with a curing mat after concrete casting and sprinkled with water for 28 days. After curing, forms were removed and direct current was applied through the RC beams. A saturated solution of $Ca(OH)_2$ was used as the electrolyte and titanium mesh plated with platinum was used as the anode. The electric current was passed through two sides of the specimens for the required period while the other faces were insulated by epoxy resin coating as shown in Fig. 3. After treatment for the required period, the following tests were carried out.

## 2.2 Chemical Analysis

Cutting diagram of concrete cubes for chemical analysis is shown in Fig. 4.

a) Measurement of $Cl^-$
Specimens for pull-out test were cut as shown in Fig. 3 and all concrete was crushed into powder. After extraction of $Cl^-$ by acid extract solution, the total amount of $Cl^-$ was measured by calibration curve method with ionic electrodes.

b) Measurement of $Na^+$ and $K^+$
As in the case of $Cl^-$ measurement, cut concrete was crushed into powder until all passed a 0.15 mm sieve. Then, this powder was dried for 3 hours in a drying machine at 100℃ and after extraction of each ion by acid extract solution, the concentration of each ion was measured by means of an atomic absorption photometer.

## 2.3 Measurement of Half-cell Potential

Half-cell potentials of steel bars embedded in the specimens for pull-out test were measured. As a reference electrode, a saturated silver chloride electrode (Ag/AgCl) was used.

## 2.4 Measurement of Micro-hardness Distribution

Sample plates were cut from specimens for pull-out test and the plates surfaces were polished like a mirror with #800 and #1200 polishing paper. After polishing surfaces for measurement, the hardness of cement paste was measured by Vickers hardness scale. Pressing load of Vickers needle was 5 gf and measuring points were chosen in a random direction as far as 6 ~ 7 mm from the steel surface.

## 2.5 Pull-out Test

a) Measured points
Load and free end slip of steel bars were measured by means of load cell (capacity ; 10 tf) and displacement meter (capacity ; 20 mm, accuracy ; 0.005 mm) respectively.

b) Loading
Pull-out test was carried out based on 'Method of Test for Bond Strength between Concrete and Steel Reinforcement by Pull-out Test (draft), JSCE'. Continuous one direction loading was carried out until free end slip reached about 10 mm.

## 2.6 Flexural Test of RC Beams

a) Measured points
Load and mid span deflection were measured by means of load cell (capacity ; 10 tf) and displacement meter (capacity ; 10cm, accuracy ; 0.01 mm) respectively.

Furthermore, to measure crack width at the main reinforcement level (d=17.5 cm), 14 pieces of $\pi$ type gages (measure length ; 5 cm, capacity ; 2 mm) were inserted without spaces.

b) Loading
One step of loading was 0.25 tf until the yield point and from this point, loading was controlled by

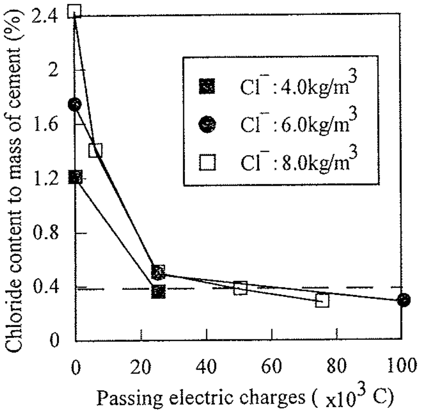


Fig. 5 Removal of chlorides around the steel bar

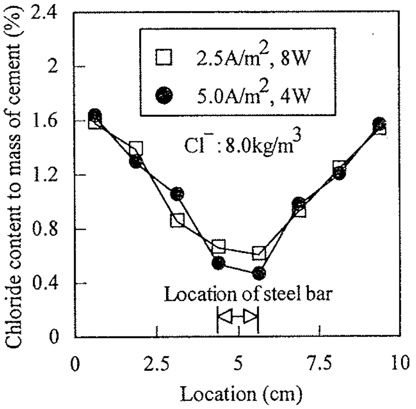


Fig. 7 Distribution of chlorides ($3360A\cdot h/m^2$)

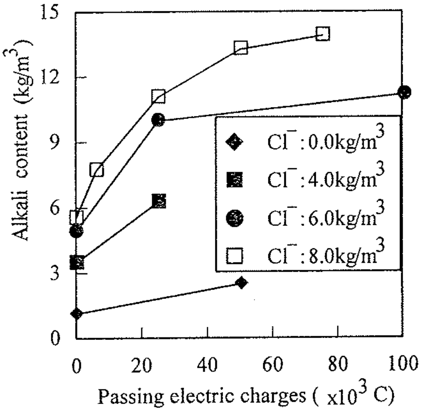


Fig. 6 Accumulation of alkali around the steel bar

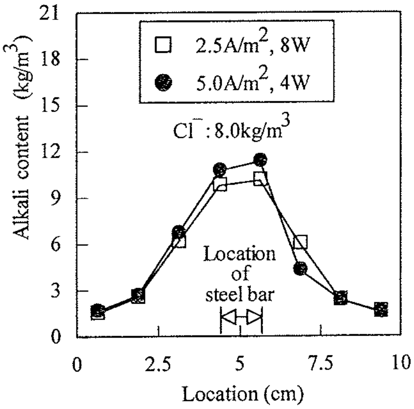


Fig. 8 Distribution of alkali ($3360A\cdot h/m^2$)

deflection at mid span, that is, one step was 0.3 mm when the deflection was less than 5 mm, and from 5 mm to 10 mm of deflection one step was 0.5 mm, and after that, one step was increased to 1 mm. When a flexural crack was generated, the beam was unloaded to 0.25 tf once and loaded again until the load reached less than 80 % of the maximum load.

## 3. CHEMICAL ANALYSIS

### 3.1 Migration of $Cl^-$

Relation between total chlorides near the steel bar (part of diagonal lines shown in Fig. 4) and passing electric charges is shown in Fig. 5. The greater passing electric charges become, the more total chlorides decrease near the steel bar. This is the effect of desalination.

It is known that fixed chlorides in concrete in the form of Friedel's salt is about 0.4% of the mass of cement in the concrete [3]. Considering this, when the amount of $Cl^-$ was 4.0 $kg/m^3$, free chlorides near the steel bar might have been extracted almost completely at 25160 C (Passing electric current for 4 weeks). However, when the amount of $Cl^-$ was more than 6.0 $kg/m^3$, total chloride content to mass of cement is more than 0.4 % at 25160 C. Generally, the longer treated periods become, the more $Cl^-$ can be extracted, but the rate of extraction decreases after the total chloride content to mass of cement becomes lower than 0.4 %.

### 3.2 Migration of Alkali Ions

Relation between equivalent alkali, ($R_2O = Na_2O + 0.658K_2O$) calculated by $Na^+$ and $K^+$ content near the steel bar (part of diagonal lines shown in Fig. 4), and passing electric charges is shown in Fig. 6.

As this accumulation of alkali is mainly due to the migration of $Na^+$ toward the cathode (steel bar), if chlorides were mixed in concrete as NaCl, it can be said that the larger the amount of chlorides found in specimens, the larger the amount of alkali concentrate that can be found around the steel bar due to applying desalination. As shown in Fig. 6, the peak value of alkali around the steel bar becomes larger when the amount of premixed $Cl^-$ becomes larger. Furthermore, similarly to the case of $Cl^-$, the longer treated periods become, the larger the accumulation of alkali, but after a certain period, the increase of the amount of accumulated alkali becomes smaller.

From this, if concrete structures to be repaired by desalination contain a large amount of chlorides, it should be noted that an excessive amount of alkali may accumulate around the steel bars.

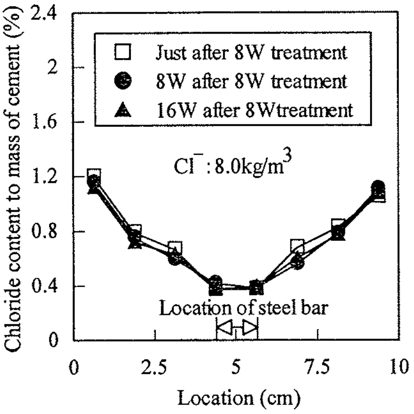


**Fig. 9** Redistribution of chloride after 8weeks treatment

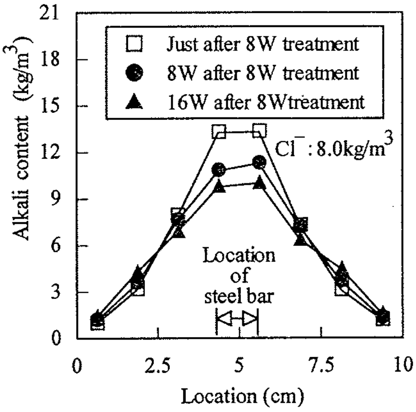


**Fig. 10** Redistribution of alkali after 8weeks treatment

### 3.3 Total Current Density and Migration of Ions

It is considered that the amount of $Cl^-$ extraction depends on the total current density, which is calculated by multiplying current density by treatment period [4]. In this study, 2.5 $A/m^2$ of current density over 8 weeks and 5.0 $A/m^2$ of current density over 4 weeks derived the same value of total current density of 3360 $A \cdot h/m^2$. Distribution of chlorides and alkali in these specimens are shown in Figs. 7 and 8 respectively. From these figures, the same total current density gives almost the same migration situation of $Cl^-$, $Na^+$ and $K^+$ even if current densities differ.

### 3.4 Redistribution of Ions

There are unbalanced distributions of chlorides and alkali in the specimens after finishing treatment and it is expected that each kind of ion will be diffused and make a new balanced situation with elapsed time. For this reason, if $Cl^-$ migrated from the concrete's surface back near the steel bar, renewed corrosion of the steel bar could occur. Moreover, alkali content concentrated near the steel bar could decrease over time.

Redistribution of total chlorides in specimens premixed with 8.0 $kg/m^3$ of $Cl^-$ after 8 weeks treatment with 5.0 $A/m^2$ of current density is shown in Fig. 9 and redistribution of alkali in these specimens is shown in Fig. 10. From these figures, it can be seen that there is almost no redistribution of total chlorides until 16 weeks after finishing desalination, but $Na^+$ and $K^+$ accumulated near the steel bars show relatively rapid diffusion.

Although it is generally considered that diffusion of $Cl^-$ is quicker than that of $Na^+$ or $K^+$, in this case, redistribution of $Na^+$ or $K^+$ was quicker than that of $Cl^-$. The reason for this may be the difference in content gradient. As $Cl^-$ ions are extracted from concrete by applying desalination, the content gradient made between concrete surface and steel bar does not continue increasing. Furthermore, taking fixed $Cl^-$ into consideration, the content gradient of free $Cl^-$ may be smaller than that of total chlorides. On the other hand, as the total amount of alkali in the specimens does not change by applying desalination, it is possible that the content gradient made between concrete surface and steel bar becomes extremely large. From the above observations, it can be considered that the accelerating force of diffusion after treatment due to the alkali content gradient is greater than that due to the chloride content gradient.

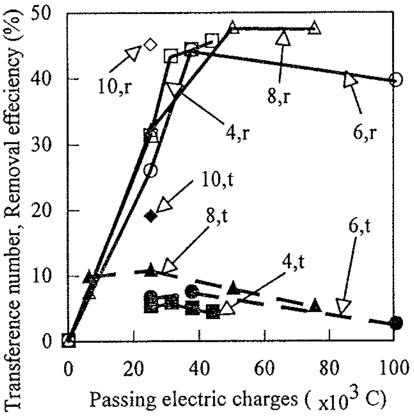


**Fig. 11** Transference number and removal efficiency of chlorides

### 3.5 Transference Number of $Cl^-$ and Chloride Removal Efficiency

Transference number of $Cl^-$, $t_{Cl^-}$ (%) is calculated by the following equation.

$$t_{Cl^-} = \frac{A}{T} \times 100, \quad T = \frac{M \cdot I \cdot t}{Z \cdot F}$$

where A : amount of extracted chlorides (g),
T : amount of extractable chlorides calculated by passing electric charges (g),
M : atomic weight (35.5 for Cl), I•t : passing of electric charges (C),
Z : valence (1 for Cl), F : Faraday constant (96500)

Moreover, in this study, chloride removal efficiency is defined as the weight percentage of extracted chlorides to premixed chlorides.

Relation between transference number of $Cl^-$, chloride removal efficiency and passing of electric charge is shown in Fig. 11. Each explanatory label shown in this figure consists of a number and "t" or "r". This number stands for the amount of premixed chlorides and the following "t" and "r" mean transference number of $Cl^-$ and chloride removal efficiency respectively.

Regardless of the amount of premixed chlorides, chloride removal efficiency tends to reach a ceiling of about 50 %. This means that after chloride removal efficiency reaches a certain level, the effectiveness of chloride removal decreases, even if longer term desalination is applied. If all $Cl^-$ ions contained in concrete could be extracted, the larger the amount of chlorides mixed, the greater the amount of extractable $Cl^-$ ions existing in concrete, when the rate of fixed $Cl^-$ is constantly about 0.4 % of mass of cement. Then, it is expected that longer term treatment results in a larger amount of chloride removal. Nevertheless, chloride removal efficiency can not pass 50 % as shown in Fig. 11. One of the main causes of this phenomenon may be the influence of $OH^-$. That is, $OH^-$ ions are increased because cathodic reaction around the steel bar generates $OH^-$ and, at the same time, $Cl^-$ ions are decreased by desalination. So, the rate of $Cl^-$ to total ions contained in concrete is decreased with the extension of treatment period and it is difficult to exceed a certain level for amount of extracted chlorides or chloride removal efficiency.

At the early stage of treatment, the larger the amount of chlorides mixed, the larger the transference number of $Cl^-$ becomes. The reason for this is considered to be that when the concrete contains a large number of extractable $Cl^-$ ions, the rate of $Cl^-$ to total ions contained in concrete is increased, and $Cl^-$ is capable of carrying a large number of electric charges. Furthermore, after passing electric charges reach a certain level, the transference number of $Cl^-$ decreases because the amount of extracted $Cl^-$ doesn't increase even if longer term treatment is applied.

## 4. MEASUREMENT OF HALF-CELL POTENTIAL

Changes in half-cell potentials of steel bars embedded in the specimens, after 8 weeks treatment with 5.0 A/m$^2$ of current density, is shown in Fig. 12. These specimens contained 8.0 kg/m$^3$ of $Cl^-$. The three lines in Fig. 12 correspond to the three specimens of the same type, and the four divided areas shown follow the ASTM criterion for adding cathodic protection area (from Pourbaix diagram). All half-cell potentials of non-treated specimens just after finishing desalination were classified in the corrosion area. On the other hand, those of treated specimens just after finishing treatment were classified in the protection area. If a new balanced electrochemical state were created in the specimens after finishing treatment over elapsed time and half-cell potentials were classified in the non-corrosion area, the repair effect of desalination could be confirmed with the recovery of passivation of steel bars.

From Fig. 12, the influence of electric current can be observed until 4 weeks after finishing treatment, but all half-cell potentials measured at 5 weeks after finishing treatment are classified in the uncertain area, and after 5 weeks, all four specimens approach the non-corrosion area over elapsed time.

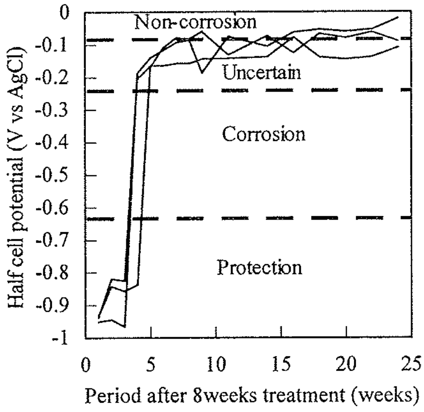


**Fig. 12** Change of half cell potential after 8weeks treatment

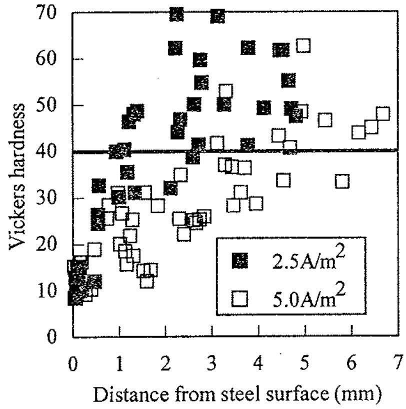


**Fig. 13** Distribution of Vickers hardness of cement paste

## 5. MEASUREMENT OF MICRO-HARDNESS DISTRIBUTION

Distribution of cement paste micro-hardness on a section perpendicular to the steel bar was measured just after treatment for 8 weeks with 2.5 A/m$^2$ of current density (total current density was 3360 A·h/m$^2$) and with 5.0 A/m$^2$ of current density (total current density was 6720 A·h/m$^2$). The measurement results for specimens premixed with 8.0 kg/m$^3$ of $Cl^-$ are shown in Fig. 13.

Although the results were scattered to some extent, it can generally be concluded that a soft layer was formed around the steel bar in both of the specimens. Furthermore, when comparing the specimens, the specimen with 5.0 A/m$^2$ of current density had a wider range of softened cement paste than the specimen with 2.5 A/m$^2$ of current density due to the larger total current density. In the sound region, sufficiently distant from the steel bar, the Vickers micro-hardness of cement paste was above 40 for both of the specimens. Therefore, in the case of 3360 A・h/m$^2$ of current density, a width of about 3 mm of cement paste surrounding the steel surface was softened, and in the case of 6720 A・h/m$^2$ of current density, this layer was about 6 mm in width.

It is considered that this softening of cement paste around the steel bar due to desalination is the cause of degradation in bond strength between concrete and steel bar. Moreover, the cause of cement paste softening is considered to be that silicates contained in C-S-H of cement paste change in quality into soluble silicates with the concentration of NaOH and KOH [5] due to the accumulation of the alkali produced from the passing of electric current, as mentioned in the results of chemical analysis.

## 6. PULL-OUT TEST

Bond stress was calculated by the following equation.

$$\tau = \frac{P}{4D \times \pi D} \cdot \alpha$$

where $\tau$ : bond stress (MPa),
P : load (N),
D : diameter of steel bar (mm), D = 13 mm
4D : bond length (mm), 4D = 52 mm,
$\pi$ D : nominal perimeter of steel bar, $\pi$ D= 40 mm
$\alpha$ : compensating rate, $\alpha$ = 29.4/f'c,
f'c : compressive strength of concrete at time of test (MPa)

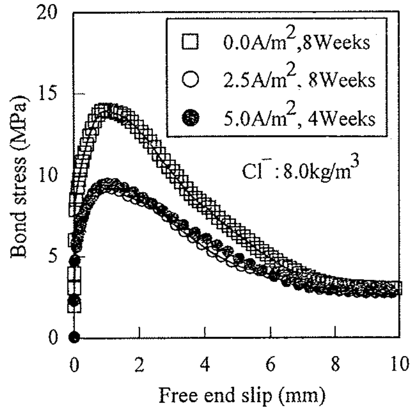


**Fig. 14** Bond stress - free end slip relation ($3360A \cdot h/m^2$)

Compared with non-treated specimens, the bond strength of treated specimens was decreased by applying treatment. It is thought that this reduction of bond strength was caused by the softening of cement paste around the steel bar and the degradation of bond interface between concrete and steel bar due to the accumulation of alkali ions around the steel bar produced from the passing of electric current, as shown in the results of chemical analysis and micro-hardness distribution. Furthermore, as shown in the results of chemical analysis, the amount of alkali accumulated around the steel bar by desalination increases when the amount of premixed chlorides is increased, or when the treatment period becomes longer up to a certain point. From these results, it is expected that a larger amount of premixed chlorides and a longer term of treatment result in more intensive degradation of bond interface between concrete and steel bar, so the decrease in maximum bond stress becomes greater.

Results of pull-out test are evaluated quantitatively as follows. In this study, bond stress ratio means the ratio of bond stress of treated specimens to that of non-treated ones.

### 6.1 Total Current Density

It has been confirmed from the results of chemical analysis that the migration of $Cl^-$ or alkali ions is dependant on total current density. In this study, 2.5 $A/m^2$ of current density over 8 weeks and 5.0 $A/m^2$ of current density over 4 weeks gave the same value of total current density of 3360 $A \cdot h/m^2$ (amount of premixed $Cl^-$ is 8.0 $kg/m^3$). The curves of bond stress - free end slip of these specimens are shown in Fig. 14. From this figure, it is observed that the two curves are almost the same. Assuming that the decrease in bond strength was caused by the accumulation of alkali ions, it can be concluded that the degradation of bond strength depends on the applied total current density, on the condition that the amount of chlorides, which controls the amount of $Na^+$, and the mix proportion of concrete are identical.

For above reasons, the total current density is used in this study, as the index represents the extent of passing electric current.

### 6.2 Amount of Premixed $Cl^-$

a) Maximum bond stress ratio

The relation between total current density and maximum bond stress ratio is shown in Fig. 15. As a whole, a larger amount of premixed chlorides and longer term treatment result in a greater decrease of maximum bond stress, until total current density reaches about 5000 $A \cdot h/m^2$, as expected. However, when the total current density increases to over 6000 $A \cdot h/m^2$, a great decrease in bond strength is observed even if the amount of premixed $Cl^-$ is 4.0 $kg/m^3$.

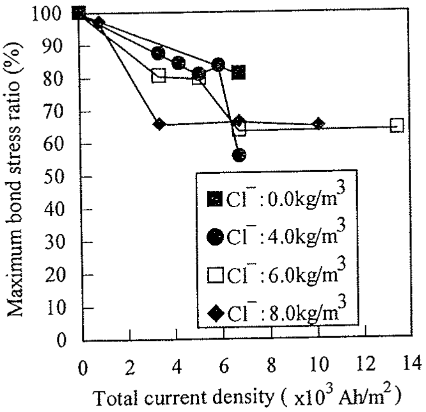


**Fig. 15** Maximum bond stress ratio

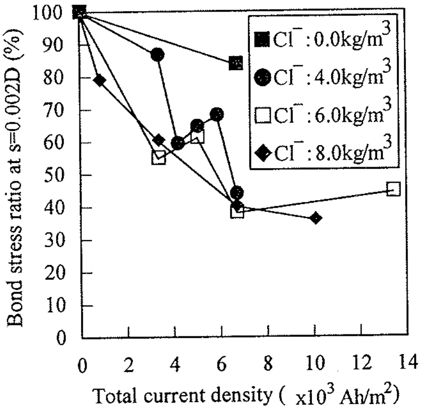


**Fig. 16** Bond stress ratio at s=0.002D

After the maximum bond stress ratio is decreased to about 60 %, this ratio maintains almost the same level even if more total current density is applied. It is considered that this phenomenon corresponds to the amount of accumulated alkali approaching a certain value as shown in Fig. 6. However, after the amount of accumulated alkali slows down its rate of increase, $OH^-$ ions are generated around the steel bar due to cathode reaction and so, it is possible that the degradation of bond interface at this point becomes more serious.

b) Bond stress ratio at s=0.002D
According to 'Method of Test for Bond Strength between Concrete and Steel Reinforcement by Pull-out Test (draft), JSCE', bond stress at free end slip s = 0.002D (D is diameter of steel bar) is given as an index to investigate the bond behavior at the small slip stage. In this study, this index (bond stress at s = 0.002D = 0.0254 mm) is used to consider the small bond slip stage as follows.

Relation between bond stress ratio at s = 0.002D and total current density is shown in Fig. 16. Compared with the case of maximum bond stress ratio, a larger influence of total current density can be observed in Fig. 16. That is, the influence of desalination on bond interface is strong when the free end slip is relatively small. Taking the results of micro-hardness measurement into consideration, Fig. 13 shows that the extremely soft part which has low micro-hardness less than 20 is only the part of treated specimens which is 1 ~ 2 mm in distance from the steel's surface. It is considered that degradation of this part affects the early stage bond behavior which is influenced slightly by the mechanical bond action of the steel bar's deformed section. On the other hand, at the ultimate stage corresponding to maximum bond stress, a relatively wide section of concrete around the steel bar receives shear force due to the mechanical bond action of the steel bar's deformed section and then, this process results in bond failure. For this reason, the influence of treatment on maximum bond stress ratio may be relieved.

## 7. FLEXURAL TEST OF RC BEAMS

### 7.1 Load-deflection Relation and Failure Mode

Load-deflection relation at mid span of flexural failure type specimens and shear failure type specimens is shown in Fig. 17. Relation between ultimate load and period of treatment is shown in Fig. 18.

a) Flexural failure type
In the ultimate state, all of the failure was caused by the crush of the upper concrete. And there was no diagonal crack at shear span. In Fig. 17, the load-deflection curve does not change significantly by 4 weeks of treatment, compared with that of non-treated specimens. On the other

hand, at 8 weeks of treatment, maximum deflection becomes about 10 mm greater than that of non-treated beams. Furthermore, Fig. 18 shows the length of treatment has little effect on the ultimate load.

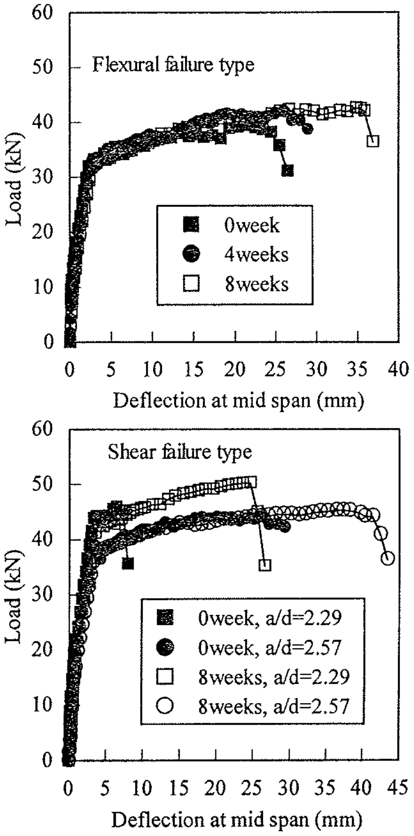


**Fig. 17** Load - deflection relation at mid span

The increase of deflection capacity may be caused by the degradation of bond strength between steel reinforcing bars and concrete, which is also confirmed by the pull-out test. Generally, after flexural cracks are generated, the deflection of beam increases gradually and the force to prevent cracks from opening is caused by this bond. Moreover, the reaction force against them, that is, compressive force acting to the upper side of the RC beam leads to ultimate concrete crush. On the other hand, when the bond strength is reduced, the amount of steel slip corresponding to a specific load is increased. So, in this case, it is considered that the force to restrain the crack width was reduced and the compressive force corresponding to a certain deflection was decreased.

On the other hand, the influence of desalination on the ultimate load was slight, because steel reinforcing bars were tensioned over the yield point due to hooks at their anchorages. In case of actual structures, if bond failure at the anchorage can be prevented, the same level of treatment as this experiment may also have little influence on the ultimate load.

b) Shear failure type
All of the shear failure type beams showed shear compressive failure in their ultimate state. From Fig. 17 and Fig. 18, regardless of a/d (shear span / effective depth of beam), treated specimens show significant increase of the maximum deflection at mid span and slight increase of ultimate load, compared with non-treated specimens.

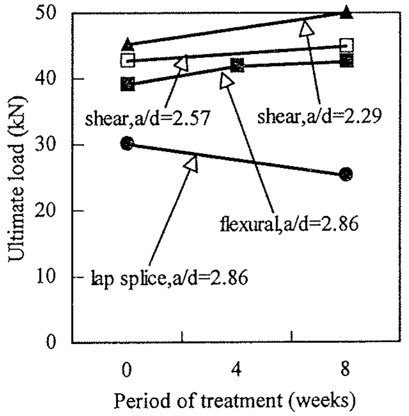


**Fig. 18** Influence of treatment on maximum load

It is reported that RC beams with unbonded longitudinal tension reinforcement show significant increase of load carrying and energy absorption capacity at their ultimate state, compared with beams reinforced with bonded deformed steel bars [6]. In case of beams with unbonded reinforcement, the upper concrete higher than tension reinforcement level forms a rigid stationary arch rib which is not damaged by the diagonal cracks and this structure can support the load in good condition. In this experiment, treated specimens would form the rigid tied arch structure after the diagonal cracks were generated, which was influenced by bond degradation between steel reinforcing bars and concrete.

c) Lap splice
All specimens which have lap splices showed very brittle bond splitting failure. That is, a large

crack was generated along the reinforcement in the bottom surface of the specimens and the load was decreased drastically without crushing of the upper side of the concrete. The reason for such a failure type would be a too thin concrete cover. The cover thickness used in this experiment was 1.87 cm which is greater than the value (1.80 cm) calculated by the 'Standard Specification for Design and Construction of Concrete, JSCE' but it is relatively small. So, it is considered that this concrete cover could not support the sever splitting load generated in the lap splice section.

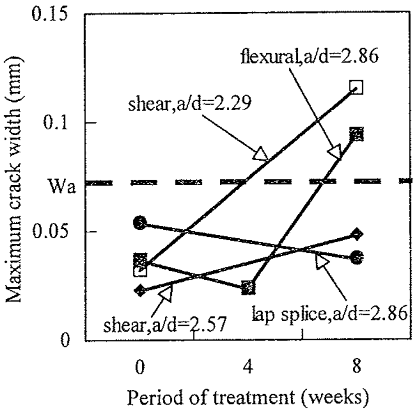


**Fig. 19** Influence of treatment on maximum crack width

Figure 18 shows the decrease of ultimate load with treatment. When a lap splice is made, continuity of reinforcement and unification of reinforcement with concrete depends on the bond strength at the lap splice section between reinforcing bar and concrete. For this reason, the bond degradation caused by desalination would affect the ultimate load of a beam which has a lap splice.

In this case, as non-treated specimens also showed very brittle failure, the difference in treated specimens was relatively slight. However, if desalination is applied to an actual structure containing sections with lap splices, it is possible to greatly influence its structural behavior. So, sufficient investigation and study before applying desalination is needed.

### 7.2 Cracking Behavior

a) Crack width

'Standard Specifications for Design and Construction of Concrete, JSCE' prescribes permissible crack width in corrosive environments as 0.004C (C : cover thickness). Permissible crack width $w_a$ in corrosive environments is calculated as wa=0.0746 mm by use of C=1.87 cm which is the cover thickness of beams used in this study. Furthermore, 'Standard Specification for Design and Construction of Concrete, JSCE' prescribes that crack width w calculated by the following equation should be less than permissible crack width $w_a$.

$$w = k\{4c + 0.7(c_s - \phi)\}\left[\frac{\sigma_{se}}{E_s} + \varepsilon'_{cs}\right]$$

where k : constant to take into account the influence of bond characteristics of steel, which may be set equal to 1.0 for deformed bars,
c : concrete cover (cm),
$c_s$ : center to center distance of reinforcing steel(cm),
$\phi$ : diameter of reinforcement (cm),
$\varepsilon'_{cs}$ : compressive strain for evaluation of increment of crack width due to drying shrinkage and creep in concrete,
$\sigma_{se}$ : increase of stress in reinforcing bars (kg/m$^2$)

Using this equation, the stress increase was calculated on the condition that $w=w_a=0.0746$ mm. Then, maximum crack width at the location of steel reinforcing bars on the condition that calculated stress acting on the reinforcement is shown in Fig. 19.

Figure 19 shows that, in the case of flexural failure type beams, maximum crack width of specimens receiving 4 weeks treatment is less than that of non-treated ones but by 8 weeks treatment, it becomes greater than that of non-treated specimens with permissible crack width $w_a$. Since load- mid span deflection curves also did not change significantly with 4 weeks treatment, it could be considered that the bond degradation caused by desalination at this level has little influence on the mechanical behavior of an RC beam. However, in the case of 8 weeks treatment, it is possible that crack width is increased at the loading condition corresponding to the service state.

b) Crack spacing
One of the influences of decrease in bond strength between steel reinforcing bars and concrete on mechanical property of RC members is deterioration of crack distribution.

As a result of crack spacing measurement after flexural testing of beams, in the case of flexural failure type specimens, the number of cracks was decreased from 9 to 6 and average crack spacing was increased from 6.7 cm to 8.8 cm by 8weeks treatment. Such a tendency could be observed in the case of shear failure type specimens. It can be considered that these phenomena were also caused by the decrease in bond strength.

However, when desalination is applied to actual structures, cracks formed in the service state are already dispersed and treatment would be started after finishing modification of a section by crack injection. For this reason, desalination can not make crack distribution of actual structures worse significantly and the influence of treatment on crack width may be relieved when it is applied to actual structures.

## 8. CONCLUSIONS

The results obtained in this project can be summarized as follows.

(1) $Cl^-$ ions contained in the specimens could be extracted by applying desalination. At the early stages of treatment, chloride removal efficiency was improved with the increase of passing electric charges but it did not reach over 50 %. At the early stages of treatment, the transference number of $Cl^-$ was increased with the increase in the amount of premixed chlorides but was decreased with the increase of passing electric charges.

(2) Alkali metal ions such as $Na^+$ or $K^+$ accumulated around the steel bar. With a larger amount of premixed chlorides and passing electric charges the amount of alkali accumulated also increased. However, when the amount of alkali reached a certain level, the increase of alkali accumulation ceased.

(3) When specimens premixed with 8.0 kg/m$^3$ of $Cl^-$ and treated for 8 weeks were kept for an additional 16 weeks after completion of treatment, redistribution of $Cl^-$ was not observed but the amount of alkali around the steel bars was reduced to 74 % of that measured just after completion of treatment.

(4) Half-cell potentials of steel bars embedded in specimens just after completion of treatment were classified in the cathodic protection area. However, these potentials changed to nobler potentials with elapsed time and they were classified in the uncertain area at 5 weeks after completion of treatment.

(5) As a result of micro-hardness measurements of cement paste around the steel bars embedded in specimens premixed with 8.0 kg/m$^3$ of $Cl^-$, a soft layer was formed around the steel bars due to passing of electric current. Furthermore, this layer became wider with an increase in total current density.

(6) As a result of the pull-out test, maximum bond stress of treated specimens was found to be smaller than that of non-treated specimens. The decrease due to desalination became larger with an increase in the amount of premixed chlorides or an increase in the total current density.

(7) According to the results of the flexural test of beams containing 8.0 kg/m$^3$ of $Cl^-$, the change in

the ultimate load was slight and the capacity of deflection increased at 8 weeks treatment.

(8) In the case of flexural failure type beams, maximum crack width at the reinforcing steel level of specimens receiving 4 weeks treatment was less than that of non-treated ones. However, in some specimens treated for 8 weeks, maximum crack width at the reinforcing steel level was greater than the permissible crack width. Moreover, by applying desalination, the number of cracks was decreased and crack distribution became deteriorated.

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