

# Potential use of zinc in the repair of corroded reinforced concrete

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## Abstract

A detailed research program was undertaken to explore the possibility of using zinc to enhance the repair of corroded concrete. The variables studied were the water–cement ratio, chloride content, and amount of zinc coverage in the repair area. The results confirmed that using dense repair mortar alone can move the corrosion damage to adjacent unrepaired zones and that the presence of zinc can mitigate this problem. However, the engineering solution is complex since the effectiveness of zinc in the prevention of further corrosion may be influenced by various factors, such as leaking joint, zinc–steel contact area.

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**Keywords:** Corrosion; Reinforced concrete; Repair; Zinc

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## 1. Introduction

Several different approaches can be utilized for corrosion repair and rehabilitation of reinforced concrete structures. The major existing repair and rehabilitation methods can be classified into: (1) typical mortar/concrete patch repair, (2) use of corrosion inhibitors, (3) surface coating, and (4) electrochemical methods.

The typical repair procedure is to remove the loose concrete detected by “sounding” (hammer or chain drag) or other non-destructive evaluation techniques. The area is then cleaned by blasting with high pressure water jets or by sand blasting. The excavated cavity is filled with a dense, chloride-free concrete/mortar placed by hand application or shotcrete. In order to keep aggressive agents away from reinforcing steel and to prevent further corrosion, mineral admixtures, water reducing agents, polymer, etc., are introduced in the repair mortar/concrete to obtain a

low water–cement ratio ( $W/C$ ), (or impermeable) mortar/concrete in the repair area. The good quality mortar/concrete may provide a high alkaline environment and make the reinforcing steel repassivated.

Since the current practice for conventional patch repair is to replace the deteriorated concrete with a highly impermeable chloride-free concrete mortar, this leads to an increase in the driving voltage of the corrosion cell because of differences in chloride ion concentration, moisture, and electrical conductivity between the repair patch and the old concrete. This increase in driving potential causes accelerated corrosion of the reinforcing bar (rebar) adjacent to the patch. As a result, for concrete of medium quality exposed to an aggressive medium with high levels of contamination from the environment, such localized repairs may cause the onset of corrosion in other locations, increasing the problem instead of solving it.

One current philosophy to combat this problem is to use a patching material that matches the properties of the old concrete. However, it will create a condition which is conducive to corrosion of the rebar within the patch. As a result, a repair strategy is needed to address the condition of the remaining structure and the rehabilitation of the whole structure.

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Recently, cathodic protection (using sacrificial anode) technique is increasingly used in corrosion repair. Protection of reinforcing steel from corrosion can be accomplished by coating or connecting the steel with a sacrificial metal. Zinc is the metal commonly used for this purpose.

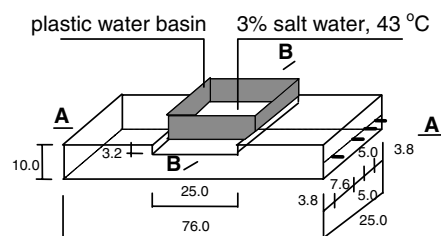
The objective of the present work is to investigate the mechanisms and causes of further corrosion in a locally repaired concrete structure as well as to investigate the effectiveness of the zinc anode. An experimental program, with and without repair systems, was conducted. The repair systems included different water/cement ratio, chloride content, and amount of zinc coverage in the repair area. Groups of the specimens without repair were designed to be representative for different parts of the repaired specimens so as to separate different factors affecting further corrosion. All the specimens were exposed to an accelerated laboratory environment, with a high temperature (43 °C) and 3% salt solution on the top of the specimens. Cu/CuSO<sub>4</sub> half-cell potential measurements in different locations of each beam were taken every week, and the total time for the measurement was one year. The microstructure of concrete and corrosion products were also studied.

## 2. Experimental work

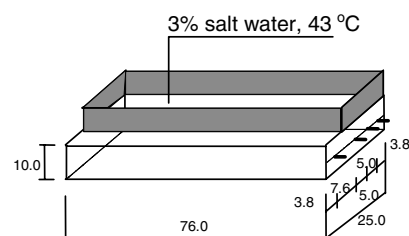
A total number of 48 concrete specimens were prepared in the experimental program. Twenty of these concrete specimens, with a size of 76 cm × 25 cm × 10 cm (30 in. × 10 in. × 4 in.) (shown as Fig. 1), were repaired with five different repair systems. As described in Table 1, repair system I was designed to simulate a structure element repaired with a high quality (or impermeable) mortar ( $W/C = 0.35$ ). Repair systems II and III were deliberated to study the effect of the zinc coverage on further corrosion. Compared with repair system I, repair system IV was intended to investigate the effect of chloride content. In contrast to repair system I, repair system V used the mortar having the same  $W/C$  as that in concrete ( $W/C = 0.65$ ), reflecting a repair material that matches the properties of the old concrete. These concrete beams were cured in the fog room for 7 days, and then moved into a 43 °C room. A small water pond was placed on the top of each beam, where the reinforcement had only 1.3 cm (0.5 in.) concrete cover. After 10 weeks, two of the beams were broken to ensure that corrosion occurred, and then five different repair systems were used to repair the corroded area. After repair, the beams were cured in a fog room for another 7 days, and then moved into the 43 °C room again, with a big water pond placed on the top of each beam with water covering the whole area of the top of the beams.

The other 28 specimens, with a size of 30.5 cm × 10 cm × 10 cm (12 in. × 4 in. × 4 in.) (shown as Fig. 2), were without repair. As listed in Table 2, these specimens were divided into seven groups to study the factors affecting further corrosion. The parameters studied included thickness

### Before Repair



### After Repair



### Section

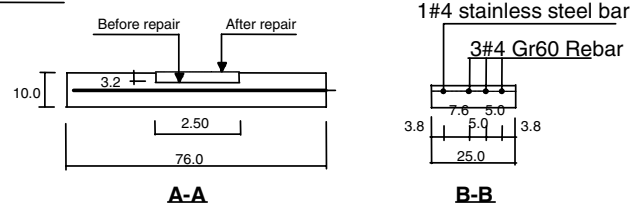


Fig. 1a. Configuration of concrete specimens (CS groups, unit: cm).

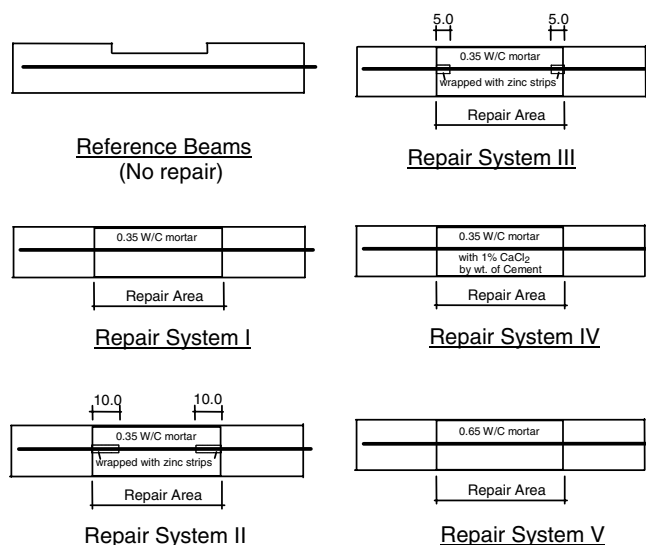


Fig. 1b. Repair systems of concrete specimens (CS groups, unit: cm).

of the concrete cover, the water/cement ratio of the repair mortar, the chloride content in the repair mortar, and the effectiveness of the zinc strips. Specimens were also exposed to an accelerated laboratory environment, with a high temperature (43 °C) and 3% salt solution in the pond and on the top of the specimens. All specimens were subjected to 6 days of wetting and 1 day of drying per week. Half-cell

Table 1  
List of concrete specimens

Group	Repair system	Description of repair systems (see Fig. 4.1b)
CS1	Reference	Reference beam—no repair
CS2	I	Reference mortar—mortar $W/C = 0.35$ without $\text{CaCl}_2$
CS3	II	Effect of zinc anode—mortar $W/C = 0.35$ with 10 cm (4 in.) zinc coverage at the ends
CS4	III	Effect of zinc anode—mortar $W/C = 0.35$ with 5 cm (2 in.) zinc coverage at the ends
CS5	IV	Effect of $[\text{Cl}]$ —mortar $W/C = 0.35$ with $\text{CaCl}_2$ (1% weight of cement)
CS6	V	Effect of $[\text{O}]$ —mortar $W/C = 0.65$ without $\text{CaCl}_2$
Note		Two specimens were broken at the age of repair

Three specimens in each group.

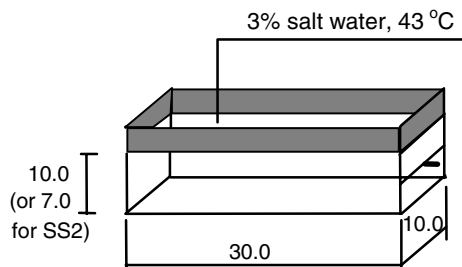


Fig. 2. Configuration of small specimens (SS groups, unit: cm).

Table 2  
List of small specimens

Group	No. of specimen	Description of specimens
SS1	SB1–SB4	Concrete $W/C = 0.65$ cover of rebar = 4.5 cm (1.75 in.), representative of the ending parts of concrete specimens
SS2	SB5–SB8	Concrete $W/C = 0.65$ cover of rebar = 1.3 cm (0.5 in.) (beam size = 7 cm $\times$ 10 cm $\times$ 30.5 cm (2.75 in. $\times$ 4 in. $\times$ 12 in.)), representative of the middle parts of concrete specimens
SS3	SB9–SB12	Repair mortar $W/C = 0.65$ without $\text{CaCl}_2$
SS4	SB13–SB16	Repair mortar $W/C = 0.35$ without $\text{CaCl}_2$
SS5	SB17–SB20	Repair mortar $W/C = 0.35$ with $\text{CaCl}_2$ (1% weight of cement)
SS6	SB21–SB24	Repair mortar $W/C = 0.35$ without $\text{CaCl}_2$ + Zinc strip instead of rebar
SS7	SB25–SB28	Repair mortar $W/C = 0.35$ without $\text{CaCl}_2$ + Rebar 100% wrapped with zinc strips

Four specimens in each group.

potential measurements in different locations of each beam were taken every week, on the days when the specimens were in drying, and the total time for the measurement was one year. The potentials were measured against Cu/ $\text{CuSO}_4$  electrode.

After one year, all specimens were broken to inspect the location of corrosion. Some concrete cores were drilled, and microscopic studies of these core samples were performed.

The compressive strength of the concrete and repair mortar was tested at 7 days, 28 days, repair age (three months), and final age (one year). Chloride permeability of the concrete and mortar was measured at 28 days according to AASHTO T277. Phenolphthalein (Phenolphthalein:alcohol:de-ion water = 1:50:49 by weight) was used for carbonation measurement. The Cu/ $\text{CuSO}_4$  half-cell potentials of reinforcing steel in concrete were measured according to ASTM C876. Tables 3–7 give the mix proportion and material properties of the concrete and repair mortar used.

The microstructure of concrete was also studied under scanning electron microscope (SEM). A polished sample, from a reinforced concrete with 1.3 cm (0.5 in.) cover, was used for electron microprobe analysis. The ARL SEMQ wavelength dispersive electron beam microprobe was equipped with eight crystal spectrometers for wavelength dispersive X-ray (WDX) analysis. The spectrometers were set up to analyze simultaneously for Si, Fe, Ca, Al, O,

Table 3  
Material properties

Cement (Type I/II)	Coarse aggregate (natural gravel)
Composition	%
$\text{C}_3\text{S}$	57.5
$\text{C}_2\text{S}$	19.8
$\text{C}_3\text{A}$	4.5
$\text{C}_4\text{AF}$	11.2
Specific gravity	3.15
Zinc strip (diameter = 2.38 mm (3/32 in.))	
Composition, %	Zn = 99.017 Cu = 0.5–1.5 Ti = 0.12–0.15
Reinforcing steel: Grade 60 #4	

Table 4  
Mix proportion of concrete and repair mortars

Materials	Concrete, $\text{kg/m}^3$ (lb/cy)	Mortar 1, $\text{kg/m}^3$ (lb/cy)	Mortar 2, $\text{kg/m}^3$ (lb/cy)
Cement	305 (515)	692 (1166)	546 (920)
Fine aggregate	917 (1545)	1385 (2334)	1228 (2070)
Coarse aggregate	914 (1540)	—	—
Water	199 (335)	243 (410)	355 (598)
$W/C$	0.65	0.35	0.65
Dry superplasticizer	—	0.4% weight of cement	—
Calcium chloride	5.15 (1% wt. of cement)	—	—

Table 5  
Fresh concrete and repair mortar properties

Property	Concrete	Mortar 1 ( $W/C = 0.35$ )	Mortar 2 ( $W/C = 0.65$ )
Slump, cm (in.)	18.0 (7)	5.0 (2)	Fluid
Unit weight, $\text{kg/m}^3$ (lb/cf)	2355 (147)	2307 (144)	2114 (132)
Air content, %	2	1.5	2

Table 6  
Hardened concrete properties

	7 Days (fog room)	28 Days (fog room)	3 Months (fog room)	3 Months (hot room)	1 Year (fog room)	1 Year (hot room)
Strength, MPa (psi)	18.7 (2710)	30.5 (4420)	34.0 (4930)	25.0 (3620)	38.2 (5540)	27.6 (4000)
28-Day rapid chloride permeability, Coulomb	9960					

Table 7  
Compression strength of the repair mortar

Strength, MPa (psi)	7 Days (fog room)	28 Days (fog room)	9 Months (fog room)	9 Months (hot room)
Mortar 1 ( $W/C = 0.35$ )	51.6 (7480)	55.5 (8050)	78.4 (11,370)	61.3 (8890)
Mortar 2 ( $W/C = 0.65$ )	21.4 (3100)	29.0 (4210)	40.0 (5800)	31.8 (4610)

K, Na and Cl. An area ( $375 \mu\text{m} \times 264 \mu\text{m}$ ) was scanned with analyses being taken at 2- $\mu\text{m}$  intervals for 10 s.

### 3. Repair process

1. Cut the concrete with a diamond saw at the edge of the repair area.
2. Chop the concrete off with a jack hammer.
3. Rough the surface of the cut area with a hand chopper.
4. Clean the corroded reinforcing bars with a steel brush and air pressure.
5. Wrap the zinc strip on the rebars for the specimens in repair systems II and III.
6. Wet the concrete surface with water.
7. Coat the surface of the concrete in the repaired area with the cement paste.
8. Cast the repair batch in the area.
9. Cure all CS specimens in a fog room for 7 days.
10. Put the water pond on the specimens and move them into the 43 °C room.

### 4. Results and discussion

#### 4.1. Results from concrete specimens (CS groups)

About 50 days after the concrete specimens (CS groups) were cast, rust was seen on the areas where the cross-section

of the specimens changed. At the age of 84 days, two concrete specimens were broken, and the location of corrosion products is shown in Fig. 3. No corrosion product was observed at the ends of rebars, where the concrete cover is 4.5 cm (1.75 in.). Corrosion occurred non-uniformly in the areas where the concrete cover was 1.3 cm (0.5 in.), and severe corrosion occurred where the cross-section changed.

However, for the small specimen (group SS2) which is representative of the middle part of concrete specimens (CS groups), rust was seen on the surface of the concrete cover at the age of 64 days, uniformly distributed along the rebar. This indicates that not only the concrete cover thickness but also the variety of cross-section of concrete cover have a great influence on the corrosion of reinforcing steel. At the cross-section transition area, there is a larger difference in oxygen and chloride concentration than at the zone of the concrete cover with a uniform thickness. Therefore, it is the difference in oxygen and chloride concentration that creates the driving force for the onset of corrosion. Since there was a water pond on every specimen, no carbonation was measured at the final age.

It was noted that at the age of one year, the corrosion of the rebar in the reference concrete beam (Fig. 4) stayed non-uniformly. Severe corrosion kept in the rebar area where the beam cross-section changed. However, in the middle of the 1.3 cm (0.5 in.) concrete cover area, only medium corrosion was detected. Based on this observation, one could infer that if a water tank covered the whole beam, medium or slight corrosion might occur at the ends of the beam since they were far from the area of the beam cross-section change and covered with thicker (3.8 cm or 1.5 in.) concrete.

As observed in Fig. 4, the specimens with repair system I (0.35  $W/C$  mortar) had severe corrosion sites at the ends of rebars. It is evident that the localized repair did cause the onset of corrosion in non-repair locations. Before repair,

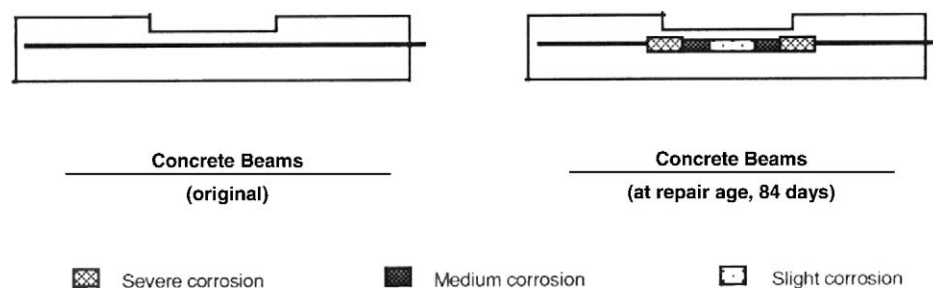


Fig. 3. Location of corrosion at repair age (84 days).

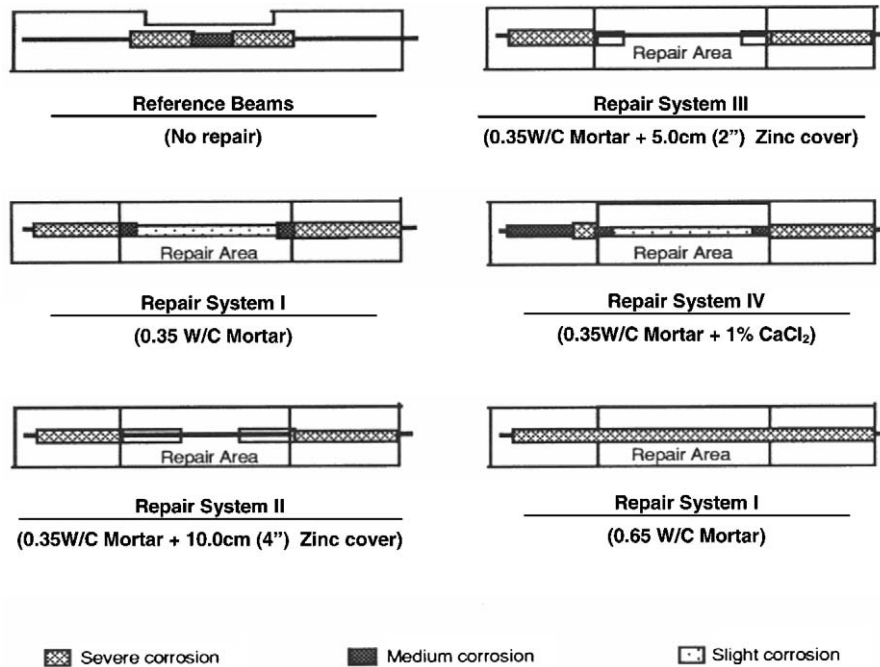


Fig. 4. Location of corrosion in different repair systems (at final age, 1 year).

the  $\text{Cu}/\text{CuSO}_4$  half-cell potential at the middle point of the specimens was higher than those at the ends; i.e., the middle part of the reinforcement served as anodes and the ending parts served as cathodes. However, after repair, the potential at the middle point of the specimens was much lower than those at the ends because of the dense repair material, alternating positions between anodes and cathodes.

As discussed before, theoretically, when zinc touches steel, the zinc should behave as a sacrificial anode and protect the steel from corrosion. However, the results from present experimental work (Fig. 4, repair systems II and III) showed that a zinc strip wrapped around reinforcing steel only prevented the steel from further corrosion at the region it covered, but it did not seem to prevent the alternative of positions between anodes and cathodes after a localized repair. The onset of corrosion in unrepair locations occurred even zinc strips were used.

There are several factors affecting the effectiveness of zinc:

(1) *Leaking joints*, which commonly exist between old concrete and new repair material. Although the specimens were carefully repaired, a space existing between the old concrete and repair mortar was still observed under a scanning electron microscope (see Fig. 5). The space, or leak joint, provided a path for water and aggressive ions, such as chloride. Under unilateral water pressure, the corrosion of the reinforcing steel near the joints increased sharply. For the specimens with zinc strips in the repaired areas, the zinc next to the leaking joints corroded, and it protected the reinforcing steel in the area it covered. White zinc corrosion products were observed on the zinc strips

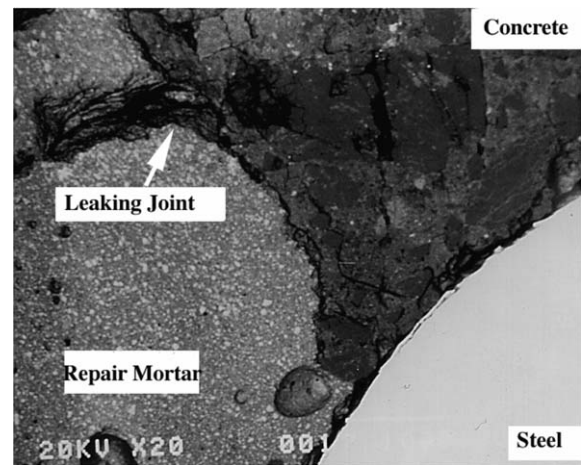


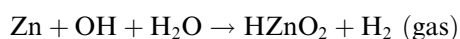
Fig. 5. Leak joint between old concrete and repair mortar.

after the specimens were broken. If the compounds formed from the zinc diffused and deposited onto the surface of reinforcing steel, the rate of corrosion of the steel would have been further reduced. However, the leaking joints might cut off the path of the zinc ion and its compound diffusion. Inside the repaired area, since the repair mortar has a much lower permeability than the old concrete, the reinforcing steel might repassivate and serve as a cathode; while the unrepaired area had a higher permeability and served as an anode. Because of a large difference in ion concentration between the old concrete and repair mortar, the corrosion rate of the reinforcing steel in the unrepaired area was even faster than that specimen without repair.



(2) *Contact area of the zinc strips with the rebar.* It is clear that zinc can serve as a sacrificial anode only when it contacts steel. Unlike zinc coating, zinc strips wrapped on rebar have a limited area in contact with the rebar. Therefore, zinc strips cannot function as an insulating barrier for reinforcing steel bars with the same efficiency as zinc coating does. Also, after zinc corrodes, the corrosion products tend to diffuse away, thereby further reducing the contact area of zinc with the rebar. If the zinc corrosion products diffuse and deposit on the surface of reinforcing steel, they could cover the rebar and also prevent the rebar from further corrosion. However, it may be difficult for the zinc corrosion products to diffuse over a leak joint and to prevent the steel in unrepaired areas from corrosion. Fig. 6 shows the zinc corrosion products diffusing into a hole at the interface between old concrete and repair mortar.

(3) *Alkalinity of the pore solutions.* The corrosion rate of zinc can be high in either an acidic solution,  $\text{pH} < 5.5$ , or in a very alkaline solution,  $\text{pH} > 12$ , [1,2]. At a pH value of about 12.5, zinc reacts rapidly to form soluble zincates and hydrogen gas:



In the present experiment, zinc strips placed in the repair mortar had a low  $W/C$  ratio, and the pH value surrounding the zinc probably reached 12.5, thus causing a rapid corrosion of zinc.

Compared with repair system I, the corrosion of reinforcing steel in repair system IV was not aggravated by the addition of 1%  $\text{CaCl}_2$ , by weight of cement, into the mortar mixture. In fact, the degree of corrosion seemed to be reduced a little. This might be because unhydrated cement reacted with the chloride so that the chloride ions had a greater chance of being chemically bound than freely penetrating in the hardened cement paste. The increase in chemical binding is not reflected in a higher critical chloride concentration for corrosion, and actually, the tendency is the opposite [3]. The amount of soluble chloride may be also influenced by the density and permeability of con-

crete/mortar. In the present work, a 0.35  $W/C$  ratio mortar was used as the reference repair material. Since chloride might hardly dissolve and diffuse in the dense, well-compacted repair mortar, the addition of chloride in the mortar mixture will have little effect on the degree of corrosion of reinforcing steel. Also, the effect of chloride on the corrosion rate of reinforcing steel is influenced by the availability of oxygen for the cathodic reaction. In the present experiment, the wetting–drying cycle was 6 days in wetting and 1 day in drying, and the specimens were in a water-saturated state for most of the time. Investigation by Hansson and Sorenson [3] indicated that, although chlorides are present, the rate of steel corrosion will be very slow if the concrete is continuously water-saturated. In wet concrete, dissolved oxygen primarily diffuses through the solution, and the diffusion rate is much slower than it is in partly dry concrete. The slow oxygen diffusion and low oxygen availability in wet concrete limit the cathodic reaction in the corrosion process and reduce the overall corrosion rate. As a result, in some cases, adding a certain amount of calcium chloride in concrete mixtures might not be the cause of corrosion. However, the chloride ions can penetrate the mortar very rapidly together with the water as it is drawn in by capillary suction when the sample is subsequently immersed in NaCl solution. In porous or badly compacted concrete/mortar, calcium chloride will increase the amount of corrosion over that which might be expected to take place in the absence of calcium chloride.

In practice, high  $W/C$  mortar is generally not used in repair. Although no alternation of cathode–anode position occurs, the results from repair system V show that when repair material was the mortar with a high  $W/C$  ratio of 0.65, relatively uniform corrosion occurred after repair; that is, corrosion occurred along the entire surface of the reinforcing steel rather than concentrating in one area. About 3 months after repair, the potential value at all points measured showed at the same level,  $-0.5$  to  $-0.6$  V. A major effect of increasing the  $W/C$  ratio was an increase in the porosity, which would increase diffusion of chloride and oxygen ions into the steel surface and decrease the electrical resistivity of concrete. Increasing the  $W/C$  ratio could also result in both lower pH, and lower chloride concentration in the pore solution. Low pH requires a lower critical chloride concentration for corrosion, but, the net effect of these factors is negative. Another important effect of increasing the  $W/C$  ratio is an increase in both plastic and drying shrinkage, resulting in increasing microcracks in concrete/mortar.

The half-cell potential ( $\text{Cu}/\text{CuSO}_4$ ) measurements from repair system V indicated that high corrosion activity occurred throughout the beam, in both repaired and non-repaired areas. However, the time for the half-cell potential value measured in the non-repaired area of repair system V to reach a rapid drop, becoming more negative, was much longer than that measured in the non-repaired area of repair system I. Specifically, about 1.5 months after repair, the half-cell potential reading in the non-repaired areas of

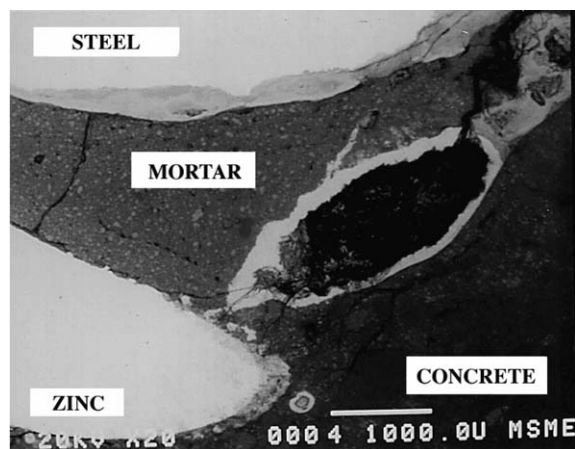


Fig. 6. Zinc corrosion products in a hole.

repair system I dropped from  $-0.25$  V to  $-0.52$  V and kept approximate  $-0.52$  V until the end of one year testing. Differently, it took approximate 3 months after repair, the half-cell potential reading in the most non-repaired areas of system V dropped approximately from  $-0.25$  V to  $-0.48$  V and kept approximate  $-0.48$  V until the end of one year testing. This comparison between systems I and V supported the inference that the localized repair with dense material did cause the onset of corrosion in non-repair locations.

#### 4.2. Results from small specimens without repair (SS groups)

Fig. 7 shows the effect of different factors on corrosion potential. The straight line in the figure indicates the Cu/CuSO<sub>4</sub> half-cell potential reading of  $-0.35$  V. Based on ASTM C876, if the reading is more negative than  $-0.35$  V, the probability of corrosion activity in the tested samples 90%. Fig. 8 presents the corrosion rate of each group of the small specimens. No corrosion products of zinc strips were observed in the specimens with zinc strips only, instead of reinforcing steel. Little corrosion was found in specimens with zinc as representative of rebar (SS6) and the entire rebar wrapped by zinc strips (SS7).

Fig. 7a shows the effect of the thickness of the concrete cover on corrosion potential. It is observed that the con-

crete specimens with a 1.3 cm (0.5 in.) concrete cover (SS2) might corrode at a very early age. After 7 weeks, the potential of the specimens reached as high as  $-0.55$  V, and it kept such high potential for the rest of the measurement. The average corrosion rate of the specimens was about  $87\text{ }\mu\text{m}/\text{year}$ . For the corresponding specimens with 4.5 cm (1.75 in.) concrete cover (SS1), the potential varied between  $-0.25$  and  $-0.3$  V in the first 3 months. At this moment, corrosion might be in the incubation stage. During the age of 3–6 months, the potential varied between  $-0.3$  to  $-0.35$  V, and it might be in the development stage of corrosion. After 6 months, the potential increased rapidly, and it was in the accelerated stage of corrosion. The average corrosion rate of the specimens was about  $75\text{ }\mu\text{m}/\text{year}$ .

Fig. 7b shows the effect of mortar  $W/C$  ratio on the corrosion potential. It seems that the mortar specimens with 0.65  $W/C$  ratio (SS3) reached the accelerate stage at the age of 8 weeks. Due to its high porosity and low strength, cracks were observed on the surface of the specimens as early as 10 weeks. The cracks were parallel to rebar and mainly caused by corrosion (see Fig. 9). Based on the ASTM G1-90 test method, the average corrosion rate of SS3 specimens was about  $113\text{ }\mu\text{m}/\text{year}$ , about 25% higher than that of SS2 specimens. This relative high corrosion rate might be due to the early cracking of specimens, resulting in the direct contact of chloride (from salt water) with

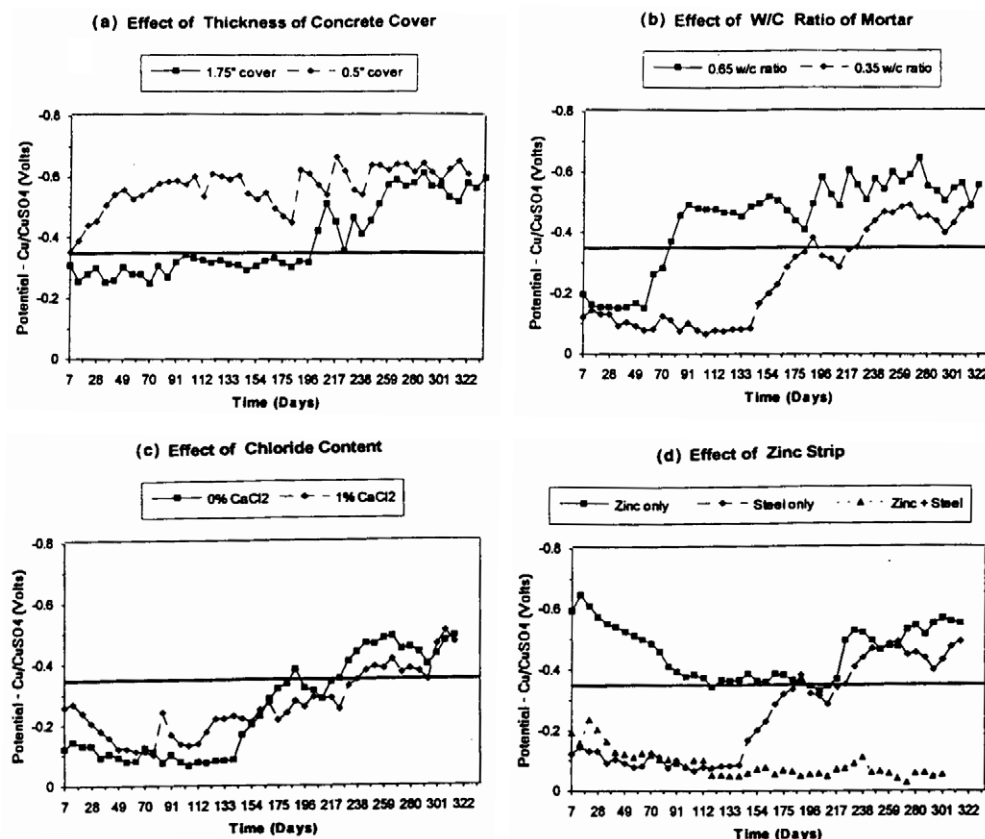


Fig. 7. Effects of different factors on corrosion potential (SS group).

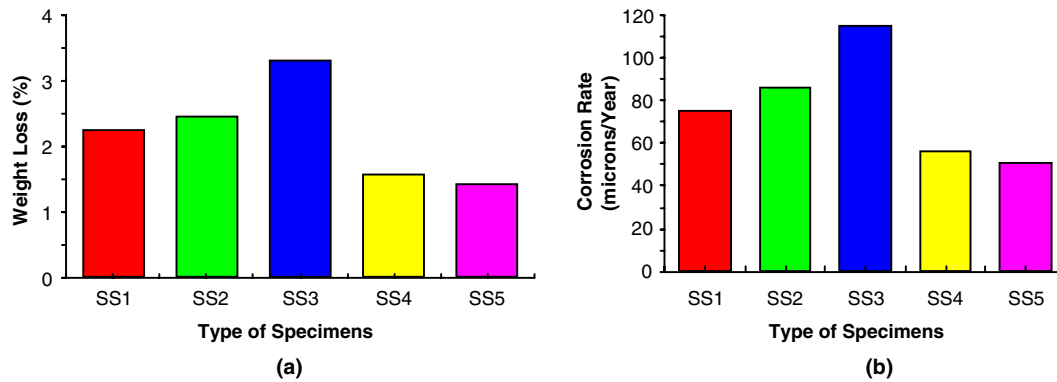


Fig. 8. Corrosion rate of reinforcing steel (ASTM Designation G1-90). Group SS1: Concrete beams with 4.5 cm (1.75 in.) cover of rebars. Group SS2: concrete beams with 1.3 cm (0.5 in.) cover of rebars. Group SS3: mortar beams with 0.65  $W/C$  ratio. Group SS4: mortar beams with 0.35  $W/C$  ratio. Group SS5: mortar beams with 0.35  $W/C$  ratio + 1%  $\text{CaCl}_2$  (by weight of cement).

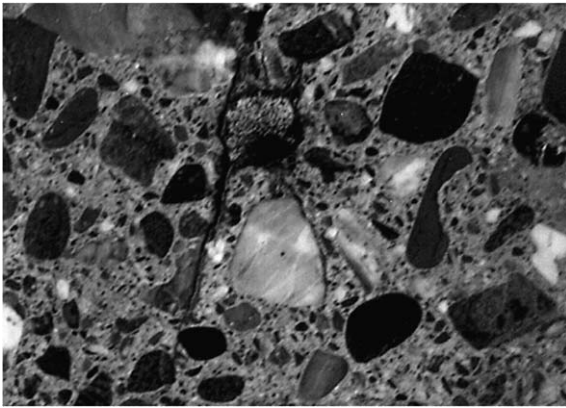


Fig. 9. Specimen SB6 (1.3 cm or 0.5 in. cover).

reinforcing steel bars. However, according to Andrde and Gouzalez [4], the corrosion penetration rate of 113  $\mu\text{m}/\text{year}$  is almost equal to a current density of 0.01  $\text{mA}/\text{cm}^2$ , which still indicates a very slow corrosion rate [5]. For the mortar specimens with 0.35  $W/C$  ratio (SS4), an accelerated stage of corrosion started at 5 months. Before 5 months, the negative potential of these specimens was very small ( $-0.2$  V), and after 5 months, the potential increased rapidly. The potential was about  $-0.5$  V at the final age of testing. The average corrosion rate of the specimens is about 60  $\mu\text{m}/\text{year}$ .

Fig. 7c shows the effect of chloride content on corrosion potential. It again indicates that the mortar specimens with 1%  $\text{CaCl}_2$  in the mixtures, Group SS5, have little difference from the specimens without calcium chloride, Group SS4. The potential of SS5 specimens was a little higher than SS4 specimens before 5 months, and it was a little lower after 5 months. The average corrosion rate of the specimens was about 55  $\mu\text{m}/\text{year}$ , a little less than the specimen without calcium chloride. As indicated before, most chloride from the addition of calcium chloride might have combined chemically with the hydration products of cement, and thus not be available to induce corrosion. Severe localized attack was observed in specimens with calcium chlo-

ride in mortar mixture, where the anodic corrosion site was about 5 cm (2 in.) over 28 cm (11 in.) in the length of the reinforcing steel bar. Corrosion products came out from the anodic site and formed a cap on the surface of the inclusion site. As indicated before, this is the type of pitting corrosion. Jones [6] found that the insoluble cap may be the collection of  $\text{Fe}(\text{OH})_3$  at the pit mouth when  $\text{Fe}^{2+}$  diffuses out of the pit and oxidizes to  $\text{Fe}^{3+}$ . This cap impedes the escape of  $\text{Fe}^{2+}$ , but might be porous enough to permit the ingress of chlorides, thereby sustaining a highly acidic, chloride saturated solution in the pit.

Fig. 7d presents the effect of zinc strips on corrosion potential. It shows that the specimens with zinc strips instead of a steel bar (SS6) had the highest negative potential over the whole year of study. This probably resulted from the reaction between zinc and cement. The figure shows that the specimens with the steel bar with zinc strips wrapping along the whole bar (SS7) had very low negative potential over the whole year of testing, while the steel in the corresponding specimens without zinc strip wrapping were corroded. This indicates that the zinc strips do reduce the corrosion rate of reinforcing steel wrapped or covered.

At the final age of one year, all SS specimens were broken to measure the weight loss and corrosion rate of the reinforcing steel. It was observed that the rust in 0.65  $W/C$  mortar specimens covered the entire rebars, or relatively uniform corrosion occurred. However, the rust in 0.35  $W/C$  mortar specimens was only concentrated on one end of the rebar; i.e., the end of the rebar served as anode and the other served as cathode. No zinc corrosion products were found in specimen SS6, in which a zinc strip was used instead of reinforcing steel so as to study the corrosion of zinc itself in concrete, without coupling with steel. The specimens in Group SS7 had the reinforcing steel entirely wrapped by zinc strips. Three of the four specimens in this group had a negative potential lower than 0.2 V for the entire period of the potential measurement. Only one specimen in this group had a negative potential higher than 0.4 V after the age of 189 days. After the specimen was broken, some white zinc corrosion products were observed;



however, only one tiny spot of iron rust was discovered on the reinforcing steel. This is evidence that the zinc strips wrapped on the reinforcing steel did behave as sacrificial anodes and reduced the corrosion rate of reinforcing steel, although it may not effectively prevent the alternation of anode–cathode position after a localized repair has been performed.

## 5. Conclusions

1. It is evidenced that for corrosion of reinforcing steel in medium quality concrete, localized repairs with a highly impermeable, chloride-free mortar will cause accelerated corrosion in adjacent unrepaired locations.
2. The effectiveness of zinc in the prevention of further corrosion may be influenced by various factors, such as leaking joint, zinc–steel contact area, and alkalinity of concrete pore solution. Special measures shall be taken to ensure the zinc effectiveness in concrete practice.
3. Not only the concrete cover thickness but also the variety of cross-section of concrete cover have a great influence on the corrosion of reinforcing steel.

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