

Corrosion of Reinforcing Steel in Concrete Structures Submerged in Seawater

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Synopsis: For an under-sea tunnel project, laboratory and field tests have been carried out to study the influence of permanent submersion in re-circulating seawater on the corrosion of steel in reinforced concrete. For each concrete mixture, cube specimens of plain concrete and beams of reinforced concrete were made. Sound as well as pre-cracked beams with a crack width of about 0.03, 0.3 and 1.0 mm were produced. The following parameters were measured as a function of time: chloride penetration as well as electrical resistivity in plain concrete specimens, and the corrosion rate of steel in reinforced concrete beams by visual and microscopic examinations.

The results show that chloride diffusion rate in concrete was reduced by decreasing the water-cement ratio (w/c) and by using fly ash or silica fume. In polymer impregnated concrete (PIC) specimens chloride penetration was negligible until about three months of permanent submersion in seawater; after this period, the penetration rate surprisingly increased according to a capillary suction mechanism. The electrical resistivity results were in agreement with those on chloride diffusion.

The corrosion rate, in terms of corroded surface area and pit depth, was not detectable on sound uncracked beams, within a 2 year period of permanent submersion in seawater, regardless of the w/c (0.65-0.35). It is expected that, even at longer ages, the corrosion rate will be negligible due to lack of oxygen which is needed for the corrosion process. In pre-cracked concrete beams, with crack widths greater than 0.2 mm, a pitting corrosion process was observed on the steel reinforcement close to the crack tip. No technical advantage in reducing the corrosion of steel was recorded when PIC, instead of regular concrete, was used.

Keywords: Chlorides; corrosion tests; cracking (fracturing); electrical resistance; reinforced concrete; seawater; underwater structures

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INTRODUCTION

Reinforced concrete structures are widely used in the marine environment. Main areas of concern, particularly for the corrosion of reinforcing bars, are in the splash and spray zones. On the other hand, submerged reinforced concrete structures have no durability problems, provided that quality concrete is used with a water-cement ratio (w/c) of 0.45 or less (1).

However, under certain circumstances severe corrosion of the embedded steel can occur within few years even for submerged reinforced concrete (2). These circumstances may include the presence of cracks caused by drying shrinkage and flexural stress of precast elements during transportation and placing under seawater or by dynamic loads in service on the submerged structures.

For concrete continuously submerged in seawater it is just a question of time before detrimental amounts of chlorides penetrate the concrete cover of cracked or un-cracked structures. The electrical resistivity of wet concrete is relatively low, and therefore the corrosion rate should be primarily controlled by the diffusion of dissolved oxygen through the wet concrete cover. Since the amount of the available oxygen in submerged structures is much smaller than in the splash or spray zones, the corrosion rate determined by the oxygen diffusion should be theoretically much lower in submerged reinforced concrete regardless of the crack pres-

ence.

The main purpose of the present work is to study whether or not the presence of cracks affects the steel corrosion in reinforced concrete permanently submerged in seawater. A further aim of the present work is to study the influence of the concrete composition on the chloride penetration and the steel corrosion, if any, of uncracked and cracked specimens.

SCOPE

This investigation is a part of a larger research project undertaken by the Ancona University and Engineering Concrete (ENCO) for the determination of the durability in marine environment of reinforced concrete structures to become a part of an under-sea tunnel (3-7). The project includes measurements of electrochemical potential, polarization resistance, cathodic protection, etc. of numerous reinforced concretes with different cover thickness (10-50 mm). These data will not be examined in the present paper. This paper will be limited to the following three aspects of the subject:

- i) chloride penetration in concrete
- ii) electrical resistivity of concrete
- iii) steel corrosion in sound or pre-cracked reinforced concrete specimens.

CONCRETE MIXTURES

A type CE I 42.5 R cement according to EN 197/Part I (European standard) was used which corresponds to the ASTM Type I portland cement. Fly ash and silica fume were obtained from plants in Italy and in France respectively. The physical properties and chemical analysis of these materials, which have been already used in a previous work (8), are shown on Table 1.

Natural sand (fineness modulus of 2.9) and gravel (nominal maximum size of 19.1 mm) were used. The gradings of the fine and coarse aggregates were within

the ASTM limits. The high-range water-reducing admixture (HRWRA) was a naphthalene-based product of Italian origin. It was used to produce a slump level of 220-250 mm in all concrete mixtures independently of the *w/c* (0.35-0.50-0.65). The mixture proportions and the properties of concretes are given in Table 2A and 2B respectively.

CASTING AND CURING OF TEST SPECIMENS

Cube specimens (100 mm) for chloride-penetration and electrical-resistivity tests were cast, vibrated with an internal vibrator, covered with a plastic sheet for 7 days and then air cured at 20°C for 45 days.

Notched beam specimens (400 x 150 x 100 mm) for the corrosion tests were cast in two layers - 70 and 30 mm thick - in order to place a steel plate (320 x 130 x 1 mm) between the two layers of concrete (Fig. 1). The thickness of the concrete layer near the notch was 30 mm except for the notched area (20 mm). Each layer was vibrated with an internal vibrator and then the beams were cured in their forms for 3 days. After this curing time, the beams were loaded in flexure using a single concentrated load to produce a crack of desired maximum width (Fig. 2). Crack widths were measured at the extreme tension fiber of the beam with an optical comparator. After removing the flexural load, the maximum crack size (Fig. 3) was in one of the following ranges:

0.02 - 0.04 mm	(mean value 0.03 mm)
0.20 - 0.40 mm	(mean value 0.30 mm)
0.90 - 1.10 mm	(mean value 1.00 mm)

The cracked surface area of the beams was completely sealed by epoxy resin except that corresponding to the notched surface area as well as the opposite one, so that the seawater penetration could freely occur only through the crack at the extreme tension fiber of the beams. Cracked and un-cracked reinforced beams were then cured as the above cube specimens for 45 days.

From the same batch as that of concrete mixture No. 2 ($w/c = 0.50$) cube and beam specimens of polymer-impregnated concrete (PIC) were produced. The 1-day cured specimens were dried at 105°C and then submerged in a methyl-methacrylate liquid monomer with 1% of azo-isobutyronitril as catalyst. After 24 hr, the monomer-impregnated specimens were washed down with water to remove the excess of monomer on the surface and then heated at 70°C for 16 hr to polymerize the monomer. At the end of the treatment the amount of polymer was about 5% by concrete mass, the thickness of the impregnated layer was about 20 mm, and the compressive strength was 89 MPa.

After an initial curing period of 45 days, the cube specimens as well as the cracked and un-cracked beams were submerged in natural recirculating seawater (chloride content of 21 g/L) onto each specimen and through the crack, if any (Fig. 4). In these field tests seawater flow was automatically blocked for 2 hr after each recirculating period of 1 hr.

TESTING OF HARDENED CONCRETE SPECIMENS

Chloride penetration

The chloride penetration depth of cube specimens was determined by a colorimetric method by spraying fluorescein and silver nitrate on the split surface area of the cube specimens (9). Pink and dark colored surfaces correspond to concrete areas penetrated or not respectively by chloride ions.

Electrical resistivity

The electrical resistivity of the concrete was determined by measuring the current after inducing known voltages between two opposite faces of the cube specimen covered by aluminum plates. Then the concrete electrical resistivity was calculated through the known geometrical data of the specimen.

Steel corrosion

The steel plate of cracked and un-cracked beams was visually examined every 6 months by removing the concrete portion (Fig. 5). The initial color of the corroded area was black due to the presence of Fe_3O_4 indicating a corrosion mechanism occurring in the absence of large amount of oxygen in seawater. Then, after about half an hour, the color changed from dark to red indicating a quick transformation of Fe_3O_4 to Fe_2O_3 due to the exposure of the steel plate to the air. After removing the whole concrete portion and polishing the steel plate, the surface area of the corroded pit (Fig. 6) was determined.

Then, after acid pickling the corroded area to remove the corrosion products (Fig. 7), the pit depth was determined through microscopic measurement of the cross section reduction (Fig. 8). A rating system was then established for the corrosion of the embedded reinforcement on the basis of the surface area and pit depth of the corroded steel plate.

RESULTS AND DISCUSSION

Portland cement concrete

This section deals with the performances of portland-cement concrete mixtures (Mix No. 1, 2 and 3 of Table 2A) made with three different water-cement ratios (0.35 - 0.50 - 0.65).

Figure 9 shows the chloride penetration as a function of the submersion time in seawater. As expected, the lower the w/c , the lower the penetration rate. This confirms other results on the penetration of chloride ions from NaCl or CaCl_2 aqueous solution (2, 10).

Figure 10 shows the concrete electrical resistivity as a function of the submer-

sion time in seawater. There is a dramatic and quick reduction in the electrical resistivity just after submerging the concrete specimens in seawater. This quick reduction is produced by the immediate water saturation of concrete specimens since the electrical resistivity of an air-void system is much higher than that of the corresponding water-saturated system (2). During the initial period of submersion, the electrical resistivity increases and then it is stabilized in less than 1 month at a level which is higher, the lower is the w/c . The increase in the electrical resistivity in the initial 1 month period of submersion in seawater can be related with the precipitation of brucite crystals - $Mg(OH)_2$ - which clogs the capillary pores and causes a densification of the cement matrix (2). In this study brucite in seawater-submerged specimens was detected, by XRD and SEM examination.

The reduction in the w/c , at a given submersion time, increases the concrete electrical resistivity, since it causes a densification of the cement matrix which, again, is responsible for a higher concrete electrical resistivity.

Table 3 shows the influence of the maximum crack width of the concrete on the surface area (S.A.) and pit depth (P.D.) of the steel reinforcement as a function of the water-cement ratio and the submersion time in years.

Uncracked beams are not shown in Table 3, since in no specimen there were traces of steel corrosion, regardless of the w/c and submersion time, although the concrete cover (20 mm) was in some cases completely penetrated by chloride ions. For instance, the penetration depth of the concrete mixture No. 1 - with a w/c of 0.65 - was 25 mm in less than 6 months (Fig. 9). However, no corrosion has been recorded in the absence of crack on the corresponding reinforced beam at 6 months as well as at longer ages up to 2 years. This confirms the general opinion that the reinforced concrete, regardless of the w/c , has a good corrosion behavior in submerged structures with respect to the splash or spray zones in a marine environment.

Turning to the cracked beams shown in Table 3, it seems that the crack width on the concrete surface plays a much more important role, in controlling the corrosion rate, than the quality of the concrete mixture as determined by the w/c . For

instance, with a maximum crack width of about 1 mm on the concrete surface, the surface of the corroded area on the steel plate is about 400 - 500 mm² independent of the w/c ratio of the concrete mixture. The mean pit depth of the corroded area is about 0.02 mm at 6 months, 0.05 mm at 1 year, 0.15 mm at 2 years and, again, the w/c ratio of the concrete does not affect significantly this trend. In some cases, at 2 years, the steel plate was completely perforated on few mm². This means that the maximum pit depth may become even more than 1 mm (the original thickness of the steel plate), although the mean pit depth at 2 years was only about 0.15 mm.

The critical crack width on the concrete surface, for which corrosion will not eventually occur, appears to be close to 0.3 mm and probably 0.2 mm. The pit depth values in Table 3 indicate that, with a mean concrete crack width of 0.3 mm, the corrosion rate is less than 10 $\mu\text{m}/\text{year}$ when the concrete cover is 20 mm, i.e. the value corresponding to the thickness of the concrete layer in the notched area.

The results would indicate the following corrosion mechanism in reinforced-concrete structures continuously submerged in seawater. In sound uncracked-concrete structures no corrosion occurs, although the chloride diffusion through the pore system of the cement matrix can reach as far as the reinforcement, particularly in concrete mixtures with higher w/c and thinner cover: this is due to the general shortage of available oxygen in seawater, particularly in the aqueous phase filling the pores of cement matrix close to the reinforcement.

Moreover, the oxygen diffusion through water saturated concrete is very low - about $10 \cdot 10^{-13} \text{ mol} \cdot \text{cm}^{-2} \cdot \text{s}^{-1}$ according to Gjorv and Vennesland (2) - and then it cannot feed the steel corrosion process to a significant extent, although chloride-contaminated steel surfaces are depassivated and then potentially prone to the corrosion process.

In slightly cracked concrete structures (with a crack width of 0.2 - 0.3 mm) the narrow cracks can be clogged with solid material, probably brucite precipitated by reaction of Mg^{2+} in seawater with OH^- of the cement paste. Therefore there is a critical crack width for which the corrosion is as negligible as that of the uncracked concrete, provided that a minimum concrete cover (at least 20 mm) is available.

In seriously cracked concrete structures, with crack widths over 0.3 mm, chloride can have immediate and easy access to the reinforcement, regardless of the w/c of the concrete mixture. On the other hand, due to the wave action of the seawater, soluble ions of the cement paste and in particular OH^- can be easily leached out from the concrete area near the cracked surface. Therefore, the ratio between corrosive ions (essentially Cl^-) and inhibitive ions (essentially OH^-) exceeds a critical limit value close to the crack tip, and then the passivity of the steel surface is locally destroyed (11).

Consequently, an electrochemical macro-cell is formed with a very small anodic area near the crack tip - where the Cl^-/OH^- ratio is relatively high - and a very large surface cathodic area not close to the crack apex, where the Cl^-/OH^- ratio is much lower. Because of the very high ratio between cathodic and anodic surface areas, a significant pitting corrosion process can be fed even in the presence of negligible amount of oxygen in seawater in direct contact with the anode. The surface of the anodic corroded area near the crack is relatively small (Fig. 5) and does not significantly increase with the time (Table 3). However, the pit depth can increase at a *mean* corrosion rate of about $70 \mu\text{m}/\text{year}$ (Table 3), but in some local areas it can be as high as 1 mm in less than 2 years.

Moreover, a pH-value as low as 3 has been measured in the concrete-steel area near the crack apex (Fig. 8). This value is significantly lower than the pH (about 7.5) of the seawater. The decrease in the pH is considered to be produced by the hydrolysis of the primary developing iron chlorides in the pit (11) and can further accelerate the corrosion process into the pit. On the other hand, the low pH is responsible for the stabilization of the pit since it inhibits the possible re-passivation of the anodic area in the presence of oxygen.

Fly-ash or silica-fume concrete

This section deals with the performances of portland cement concretes both with and without the addition of fly ash or silica fume. All these concrete mixtures, were manufactured at the same w/c of 0.35 (Mixtures No. 3, 4 and 5 of Table 2A).

Figure 11 shows the influence of the inclusion of fly-ash or silica-fume in the concrete on the chloride-penetration rate in concrete specimens continuously submerged in seawater. In the presence of fly ash the chloride diffusion rate is reduced with respect to the reference concrete without fly ash. The influence of silica-fume in reducing the chloride diffusion is even greater than that of fly-ash. Similar results were obtained by studying the influence of fly ash or silica fume on chloride diffusion in concrete specimens exposed to NaCl or CaCl_2 aqueous solutions (10).

Figure 12 shows that use of silica-fume addition increases the electrical resistivity of the concrete by more than 2 times, whereas fly ash modifies it to a negligible extent. Since the increase in electrical resistivity, caused by a reduction in the w/c , has been associated with a densification of the cement matrix (Fig. 10), the negligible effect of the fly ash on the electrical resistivity can be explained by assuming that it either does not change the porosity of the cement matrix or causes an increase in the ionic strength of the aqueous phase in the capillary pores.

Table 4 shows the influence of fly-ash or silica-fume on the corrosion behavior with respect of the reference control concrete at the same w/c . In uncracked beams, not shown in Table 4, no corrosion at all has been recorded within 2 years of permanent submersion in seawater. In cracked beams, the results of the corrosion behavior in terms of corroded surface area (SA) and pit depth (PD) are substantially the same as those described for portland-cement concrete without pozzolans (Table 3). Again, maximum crack widths larger than a given critical value (0.2-0.3 mm) as measured on the concrete surface are required to detect a corrosion process regardless of the presence of fly ash or silica fume.

Polymer impregnated concrete

This section deals with the performances of polymer-impregnated concrete with respect to plain concrete with and without silica fume at the same water-cement ratio (0.50). Compositions and properties of these concretes are shown in Tables 2A and 2B (Mixtures No. 2 and 6).

Theoretically, PIC appears as the best candidate for a durable material to be used in a special marine submerged structures where maintenance and repair work must be avoided because of difficulties and costs. Diffusion of chloride and oxygen should be completely blocked by the polymer-impregnated outside layer. Moreover, the expected higher electrical resistivity of PIC would reduce to a negligible value the electron movement caused by electrochemical elements, if any. Therefore, any corrosion process of the metallic reinforcement should be absolutely avoided at least for sound un-cracked concrete structures. On the other hand, the high flexural strength (12) of PIC should protect the reinforced structures by any crack caused by transportation movements or loads in service.

Figure 13 indicates that there is no penetration at all of chloride in PIC during the initial 3 months period, except for 2 mm of the outside layer which is immediately penetrated at the time of submersion in seawater. This quick penetration occurs by capillary suction and not by diffusion, and should be related with the partial drying of the outside monomer during the thermal treatment at 70°C. However, except for this thin and quick chloride penetration, PIC performed much better than the corresponding concrete at the same *w/c*, with or without silica fume. Surprisingly, after the initial 3 months period of submersion in seawater, the chloride penetration rate in PIC suddenly increased, so that in few months there was a full penetration of the specimens. This disappointing behavior could be explained by a sort of deterioration of the acrylic polymer caused by seawater. Since the impregnation involved only the outside layer of the concrete for a 20 mm thickness, the inside portion of the specimen - which was dried at 105°C to remove the existing saturated water and to favor the monomer intrusion - was able to absorb quickly seawater when the polymer in the outside layer was destroyed.

Figure 14 shows the electrical resistivity results and confirms the above behavior for the chloride penetration. The initial electrical resistivity of PIC was about $37500 \cdot 10^3$ ohm·cm and quickly dropped to 20000 ohm·cm after the submersion in seawater. This drop is due to the seawater absorption by the 2 mm unimpregnated outside layer. Then, the level of electrical resistivity of PIC for about 3 months is about 4 times that of control concrete and 2 times that of the silica-fume concrete. However, due to the degradation of the impregnating polymer after a 3-month period of submersion in seawater, the electrical resistivity of PIC went down to about the

same level as that of the control concrete.

Table 5 indicates the influence of the polymer impregnation on the corrosion behavior with respect to the control concrete and that with silica fume with the same *w/c*. Sound and cracked reinforced concrete specimens were tested. The flexural load required to crack the PIC beams was higher than that required for the reference concrete. There is no significant difference between the corrosion process of cracked PIC reinforced beams and that of the corresponding specimens of portland cement concrete or silica fume concrete. Again, the crack width determines the corrosion rate in terms of corroded surface area and pit depth, and a critical crack width of 0.2-0.3 mm was also established for PIC reinforced beams.

All the above results indicate that, for submerged structures in seawater, there is no technical advantage in using PIC instead of ordinary reinforced concrete as far as durability and in particular steel corrosion are concerned.

CONCLUSIONS

The corrosion of reinforced concrete did not occur, within a 2 year test period of continuous submersion in recirculating seawater, in uncracked structures, with a cover thickness of at least 20 mm and a *w/c* of 0.65 or less. Although chloride ions in this period of time can reach the reinforcement, the available amount of oxygen diffusing through the aqueous phase of the pores in the cement matrix is negligible and cannot feed any corrosion process to a detectable extent. This means that the corrosion rate will be zero or negligible (less than 1 $\mu\text{m}/\text{year}$), even at longer ages, unless cracks will be formed in service.

In cracked concrete structures with maximum crack widths greater than 0.2 mm, seawater has an immediate access to the reinforcement independently of the *w/c* of the concrete mixture and, just close to the crack tip, a pitting corrosion process can be developed. The corrosion rate corresponds to a mean pit depth rate of about 70 $\mu\text{m}/\text{year}$, but in some local areas it was found as high as 500 $\mu\text{m}/\text{year}$.

Reductions in the w/c from 0.65 to 0.35, as well as additions of fly ash or silica fume, decrease chloride diffusion and increase the concrete electrical resistivity, both potentially inhibiting the corrosion process of the steel reinforcement. However, from a practical point of view they have a negligible effect on the steel corrosion of cracked concrete surface in seawater submerged structures. Cracks can be produced before submerging concrete precast elements in seawater (drying shrinkage or local flexural stress during placing) and in submerged structures due to dynamic loads in service.

Surprisingly, PIC (which should be crack-free because of its high flexural strength) did not perform well since, after about 3 months of permanent submersion in recirculating seawater, the impregnating polymer appeared to be destroyed even in uncracked beams. Therefore it seems that there is no technical advantage in using PIC for structures submerged in seawater. Polymer-based materials are heavily promoted for repair of concrete structures. The results of the present study indicate that preliminary tests are needed for these repair materials before use in the case of sea structures.

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TABLE 1—COMPOSITION OF CONCRETE MIXES

CONCRETE MIXTURE No.	BATCH QUANTITIES (kg/m ³)						
	WATER	CEMENT	FLY ASH	S.F.**	SAND	GRAVEL	HRWRA*
1	194	299	-	-	734	1197	0.6
2*	154	308	-	-	754	1230	4.0
3	140	400	-	-	732	1194	9.0
4	138	394	60	-	721	1175	12.0
5	142	406	-	61	743	1211	12.2
6	149	298	-	45	732	1194	7.1

* This mixture was also used to manufacture polymer-impregnated concrete (PIC)

**S.F. = Silica Fume

* HRWRA = High-range water-reducing admixture

TABLE 2—PROPERTIES OF CONCRETE

MIXTURE No.	W/C	W/(C+FA)	W/(C+SF)	SLUMP (mm)	COMPRESSIVE STRENGTH (MPa) AT:		
					3 days	7 days	28 days
					1	0.65	-
2*	0.50	-	-	220	24.5	36.9	42.9
3	0.35	-	-	250	43.8	52.5	62.4
4	0.35	0.30	-	250	46.2	56.3	71.4
5	0.35	-	0.30	250	51.9	60.8	80.2
6	0.50	-	-0.43	220	22.0	31.5	53.5

* This mixture was used also to manufacture PIC specimens with a compressive strength of 89 MPa at the end of the polymerization treatment (3 days).

TABLE 3—INFLUENCE OF MAXIMUM CRACK WIDTH ON CORROSION OF REINFORCED PORTLAND CEMENT CONCRETES

CONCRETE MIXTURE (No. - W/C)	CORROSION RATINGS AT CRACK WIDTH (mm) OF:				
	0.03		0.3		1.0
	SA* (mm ²)	PD** (mm)	SA (mm ²)	D (mm)	SA (mm ²) PD (mm)
(1-0.65)					
0.5 yr.	NO CORROSION		NO CORROSION		460 0.02
1 yr.	NO CORROSION		NO CORROSION		430 0.05
2 yr.	NO CORROSION		380 0.02		480 0.15
(2-0.50)					
0.5 yr.	NO CORROSION		NO CORROSION		450 0.01
1 yr.	NO CORROSION		350 0.01		480 0.06
2 yr.	NO CORROSION		400 0.02		510 0.15
(3-0.35)					
0.5 yr.	NO CORROSION		NO CORROSION		500 0.03
1 yr.	NO CORROSION		NO CORROSION		440 0.05
2 yr.	NO CORROSION		360 0.01		480 0.14

* SA = Surface Area of the corroded steel reinforcement

** PD = Pit Depth of the corroded steel reinforcement (mean value)

TABLE 4—INFLUENCE OF MAXIMUM CRACK WIDTH ON CORROSION OF REINFORCED CONCRETE ($w/c = 0.35$) WITH AND WITHOUT FLY ASH (fa) OR SILICA FUME (sf)

CONCRETE MIXTURE (No.-min. add)	CORROSION RATINGS AT CRACK WIDTH (mm) OF:					
	0.03		0.3		1.0	
	SA* (mm ²)	PD** (mm)	SA (mm ²)	PD (mm)	SA (mm ²)	PD (mm)
(3-no fa or sf)						
0.5 yr.	NO CORROSION		NO CORROSION		500	0.03
1 yr.	NO CORROSION		NO CORROSION		440	0.05
2 yr.	NO CORROSION		300	0.01	480	0.14
(4-fa)						
0.5 yr.	NO CORROSION		NO CORROSION		450	0.02
1 yr.	NO CORROSION		340	0.01	480	0.06
2 yr.	NO CORROSION		320	0.01	440	0.16
(5-sf)						
0.5 yr.	NO CORROSION		NO CORROSION		500	0.01
1 yr.	NO CORROSION		NO CORROSION		450	0.04
2 yr.	NO CORROSION		360	0.01	500	0.14

* SA = Surface Area of the corroded steel reinforcement

** PD = Pit Depth of the corroded steel reinforcement (mean value)

TABLE 5—INFLUENCE OF POLYMER IMPREGNATION ON CORROSION OF REINFORCED CONCRETE

CONCRETE MIXTURE (No.-type)	CORROSION RATINGS AT CRACK WIDTH (mm) OF:					
	0.03		0.3		1.0	
	SA* (mm ²)	PD** (mm)	SA (mm ²)	PD (mm)	SA (mm ²)	PD (mm)
(2-Control)						
0.5 yr.	NO CORROSION		NO CORROSION		450	0.01
1 yr.	NO CORROSION		350	0.01	480	0.06
2 yr.	NO CORROSION		400	0.02	510	0.15
(6-sf)						
0.5 yr.	NO CORROSION		NO CORROSION		390	0.01
1 yr.	NO CORROSION		NO CORROSION		480	0.05
2 yr.	NO CORROSION		360	0.01	440	0.14
(2-PIC)						
0.5 yr.	NO CORROSION		NO CORROSION		480	0.02
1 yr.	NO CORROSION		380	0.01	490	0.06
2 yr.	NO CORROSION		410	0.02	500	0.13

* SA = Surface Area of the corroded steel reinforcement

** PD = Pit Depth of the corroded steel reinforcement (mean value)

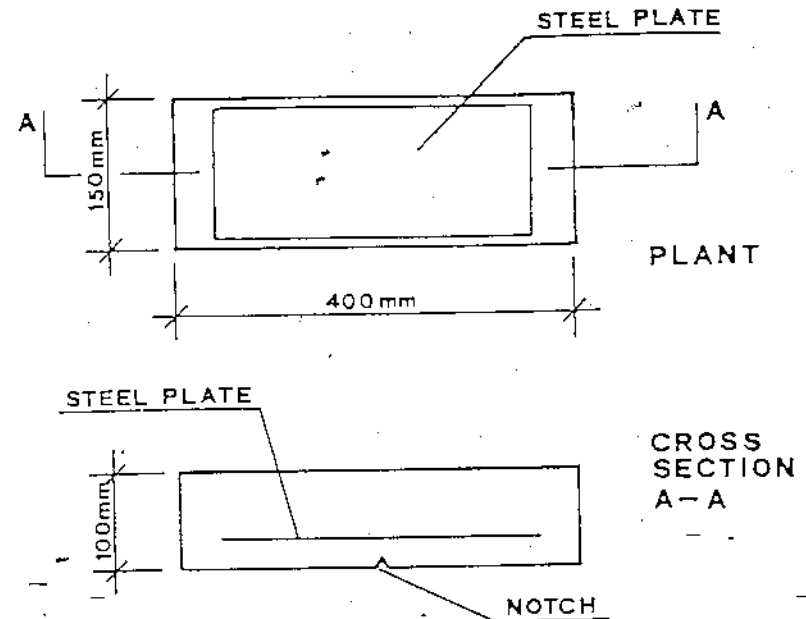


Fig. 1—Reinforced beam specimens

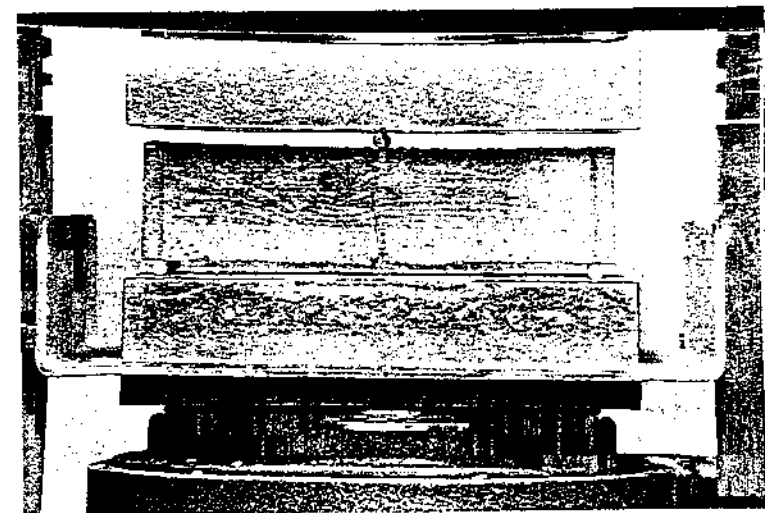


Fig. 2—Flexure loading to produce cracked reinforced beams

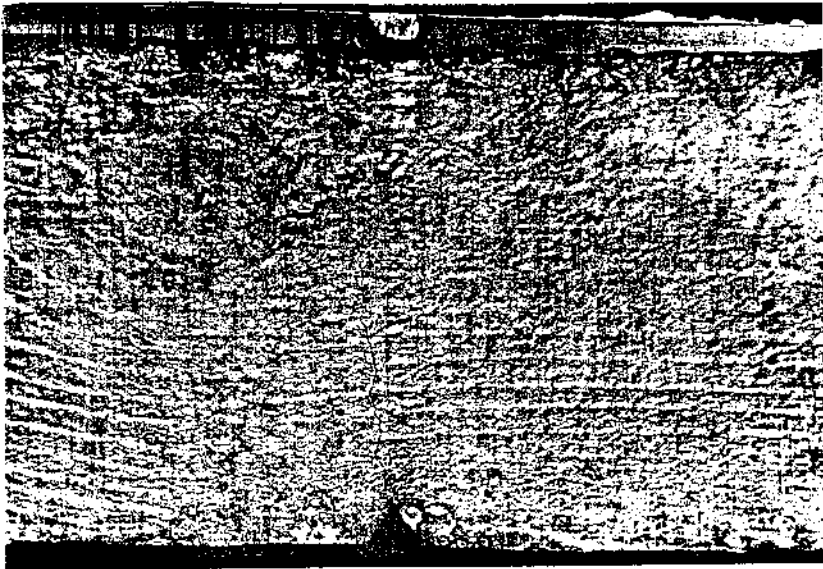


Fig. 3—Detail of crack width in notched beam

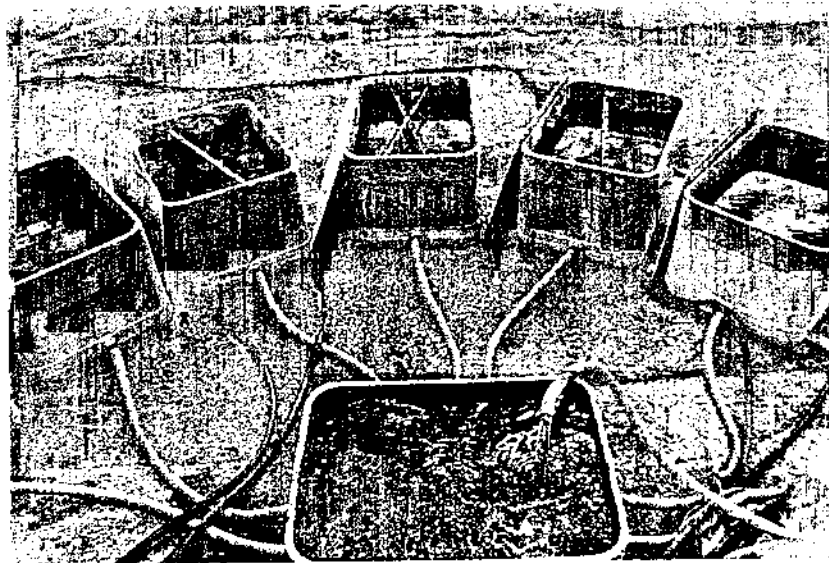


Fig. 4—View of field exposure of specimens to recirculating natural seawater

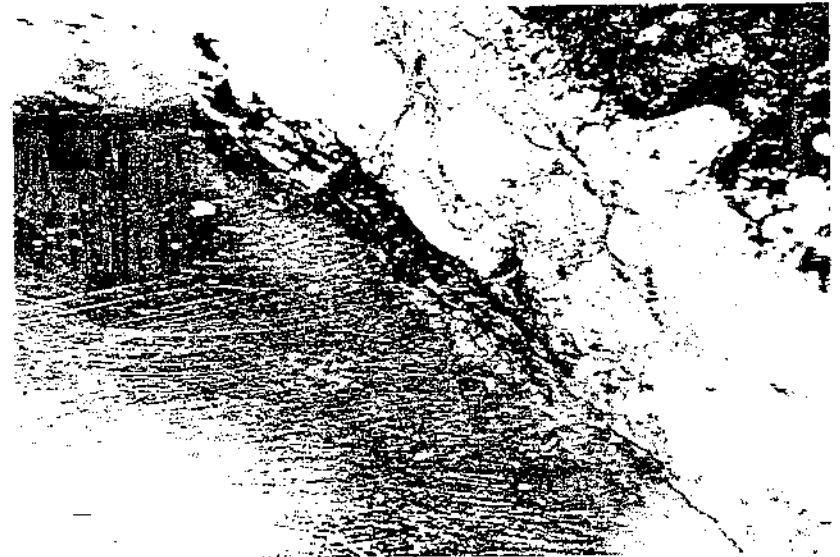


Fig. 5—View of steel plate corrosion products after removing the one-half concrete portion

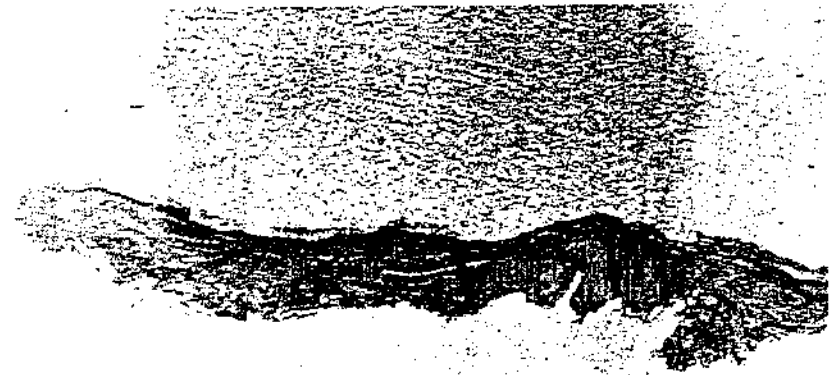


Fig. 6—View of polished steel plate after removing whole concrete



Fig. 7—Micrograph of steel plate after acid pickling to remove iron oxides and to measure corroded surface area

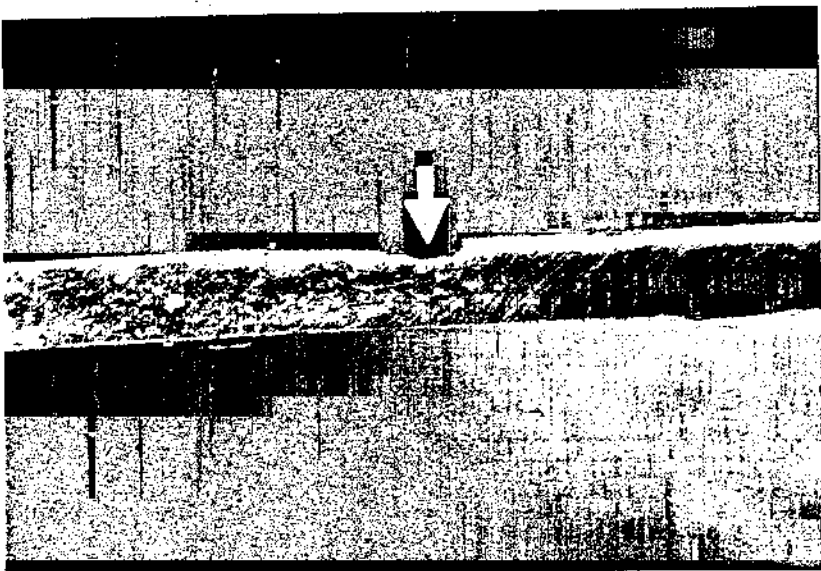


Fig. 8—Micrograph of steel plate (cross section) after acid pickling to measure depth of corrosion pit (below arrow)

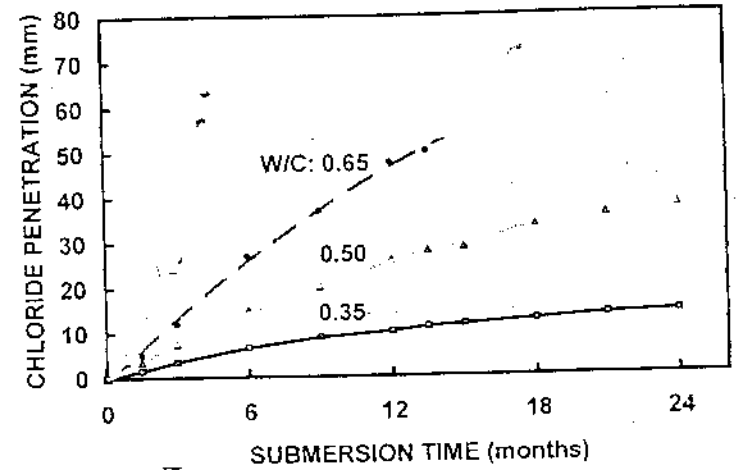


Fig. 9—Influence of *w/c* on chloride penetration of portland cement concrete

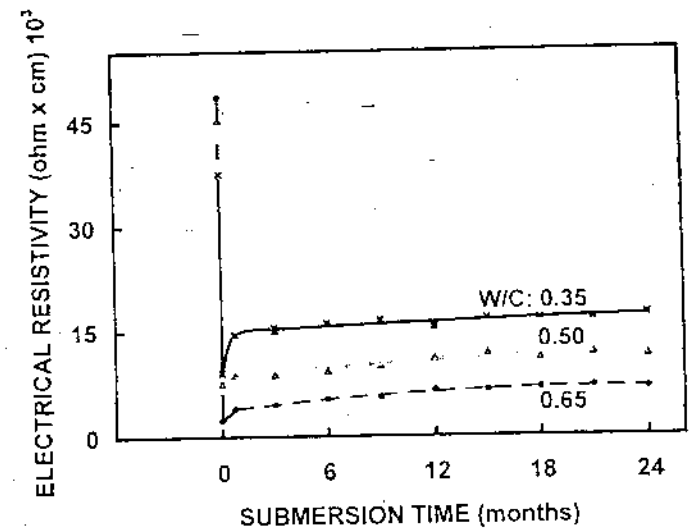


Fig. 10—Influence of *w/c* on concrete electrical resistivity

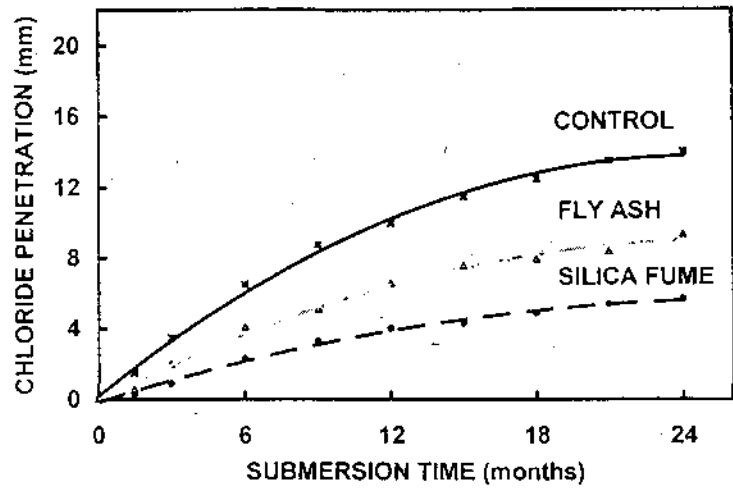


Fig. 11—Influence of fly ash or silica fume addition on chloride penetration in concrete ($w/c = 0.35$)

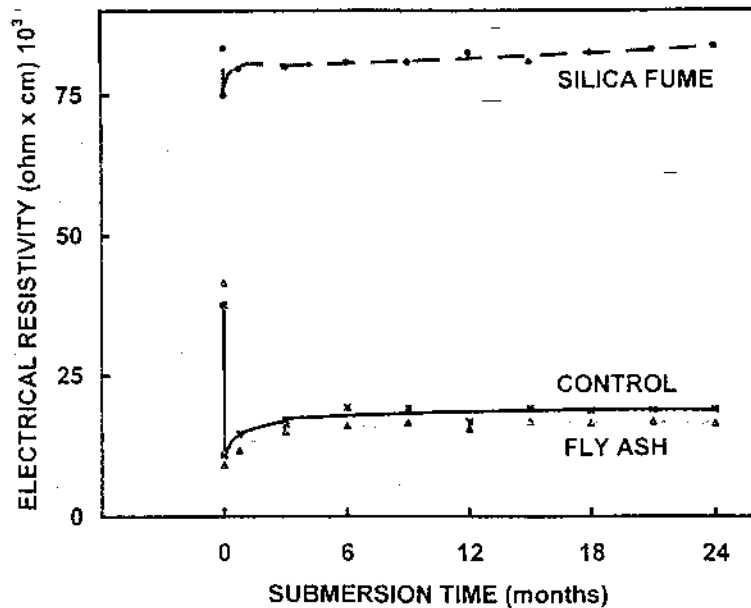


Fig. 12—Influence of fly ash or silica fume addition on concrete electrical resistivity ($w/c = 0.35$)

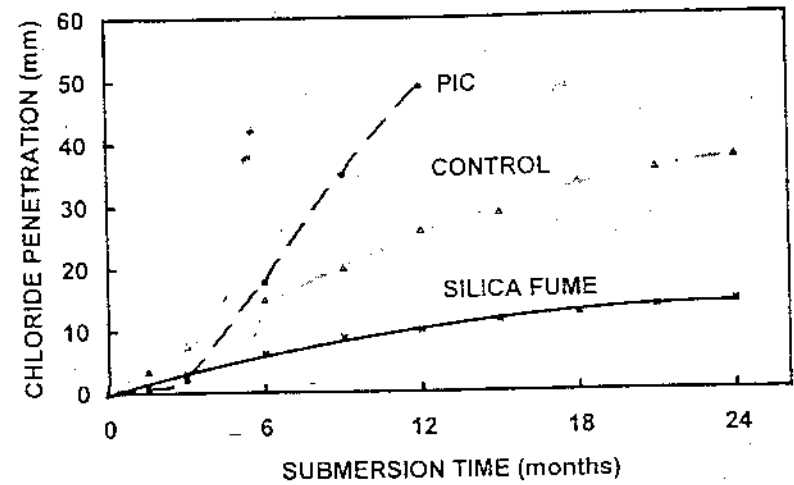


Fig. 13—Chloride penetration in PIC, portland cement and silica fume concretes ($w/c = 0.50$)

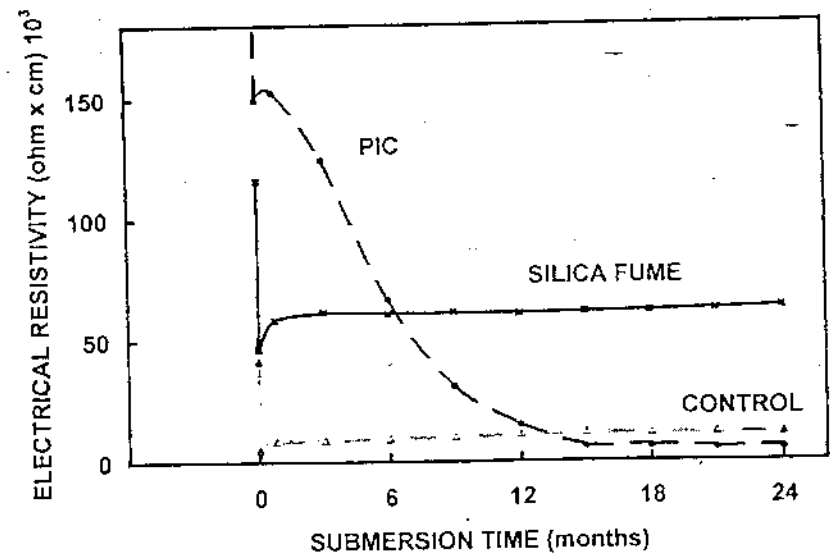


Fig. 14—Electrical resistivity of PIC, portland cement and silica fume concretes ($w/c = 0.50$)