Studies on Chloride Induced Corrosion of Reinforcement Steel in Cracked Concrete

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Abstract: The durability of reinforced concrete (RC) exposed to severe environments depends largely on its ability to resist the penetration of aggressive compounds. This paper deals with the corrosion of reinforcement bars (rebars) in cracked concrete exposed to chlorides. A simple U-shaped specimen was used in the study, with a high yield strength deformed bar of 12 mm diameter placed at with 20 mm clear cover. Concrete specimens with three water to cement ratios (w/c) and pre-cracks exposed to chlorides were studied for quality assessment and quantifying the corrosion damage in terms of gravimetric weight loss. Water absorption and rapid chloride permeability tests indicated that as the w/c decreases, the water absorption and the charge passed were decreased. Accelerated corrosion test results indicate that flexural cracks cause macrocell corrosion of rebars in reinforced concrete. Concrete with higher w/c suffers higher rebar weight loss, even under cracked conditions. It appears that the corrosion induced crack width decreases as the pre-crack width increases and increases as the w/c decreases. The simple U-shaped specimen is found to be useful for the comparative evaluation of corrosion in different concretes under cracked conditions.

Keywords: Corrosion; Chloride attack; Cracking; Crack width; Water-cement ratio.

1 Introduction

Concrete is a durable material and the encasement of steel in it provides the latter with a protective environment and allows it to function effectively as a reinforcement. Ideally, such a combination should be highly durable [Bentur, Diamond and

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Berke (1998)]. However, the main limitation of concrete, even of good quality, is the presence of microcracks, voids and capillary pores through which aggressive corrosive agents like chlorides, CO₂, etc., penetrate easily and deteriorate the structure. It is important to note that concrete always contains cracks and that cracks formed prior to initiation of reinforcement corrosion have an immense impact on the corrosion of rebar. The degradation of reinforced concrete (RC) in an aggressive environment can be considered in terms of the initiation and propagation periods. The initiation period is when ions move through the cover until they reach, in sufficient concentration (to break down the passivity), the rebar to initiate active corrosion. Propagation commences once corrosion is activated and proceeds up to the damage threshold level [Tutty (1980)]. The variables affecting the corrosion rate of reinforcement steel in concrete include the cement type, cement content, water/cement ratio (w/c), quality of construction, cover thickness, (pre-) cracking, environment, etc. [Bentur, Diamond and Berke (1998); Tutty (1980)]. This paper presents the results of laboratory studies on accelerated chloride-induced corrosion of reinforcement steel in concrete with pre-cracks of 0.2 mm and 0.4 mm width. The basic corrosion mechanisms involved and the influence of cracks on corrosion are also reviewed.

2 Corrosion Mechanisms

The corrosion of steel in concrete is an electrochemical process in which both chemical reactions and flow of electrical current are involved [Bentur, Diamond and Berke (1998); Broomfield (1997)]. It involves two separate, though coupled, chemical reactions that take place simultaneously on the steel surface. They are known as 'anodic' and 'cathodic' reactions:

Anodic reaction

$2 \text{ Fe}^{0} - 4 \text{ electrons}$ (metallic atoms at the steel surface)	 2 Fe ²⁺ (1) (ions dissolved in pore solution)
Cathodic reaction	
$O_2 + 2 H_2O + 4$ electrons (dissolved oxygen molecules)	 4 (OH ⁻) (2) (ions dissolved in pore solution)

The actual loss of metal involved in the corrosion processes takes place near the anodic sites, and is indicated by Eq. 1 and is shown in Fig. 1 (a) [Bentur, Diamond and Berke (1998)]. The actual metal removal process described in Eq. 1 will continue only if there is a cathodic reaction, which will act as a sink for the electrons produced at the anodic site. Therefore, if oxygen and water are not available at the cathodic sites, the corrosion process will be terminated.

The electrochemical character of the corrosion process is demonstrated by the flow of current in the closed loop of Figs. 1 (a) and (b). An external current flow through the solution through the pores of the concrete surrounding the steel, which consists of hydroxide ions moving from the cathode to the anode, and also ferrous ions moving from the anode to the cathode. The water in the concrete pores (basically a dilute solution of alkali and calcium hydroxides) serves as a vehicle for ionic flow. If the pores dry out or if the structure of the concrete is so dense that the pores are not well interconnected, the electrical resistance in the concrete is high and the flow of ions through the pores is impeded (Fig. 2). Under such circumstances, the corrosion process will slow down or stop.

The influence of cracking on the durability of concrete structures is a complex function of the composition, concrete quality, exposure conditions, crack width, type, length, depth, frequency and orientation with respect to reinforcement, crack healing, etc. [Bentur, Diamond, and Berke (1998); Broomfield (1997); ACI 222R (1996); Scott and Alexander (2007); Otieno (2008); Otieno, Alexander and Beushausen (2010)]. Cracks in the cover produce at least three specific effects: (i) they tend to facilitate the onset of corrosion by providing easy access for the penetration of dissolved Cl⁻ ions and of CO₂ so as to induce depassivation; (ii) they accelerate the rate of corrosion once begun, by reducing the barrier to diffusion of oxygen, at least near the cracks themselves; and (iii) they produce substantial non-uniformity in the physical and chemical environment around the steel, thus providing favourable conditions for corrosion to occur.

Two different corrosion mechanisms are theoretically possible for steel corrosion in concrete. In Mechanism 1, the anodic and cathodic reactions take place side by side (i.e., microcell corrosion). In Mechanism 2, the anodic and cathodic reactions occur some distance apart (i.e., macrocell corrosion). In an aggressive environment, depassivation of the steel near the crack will take place before it occurs in uncracked areas [Bentur, Diamond and Berke (1998); Francois and Maso (1988)]. Corrosion then begins, with the depassivated steel in the cracked zone being the anode and the remaining steel being cathodic, as shown in Fig. 3 [Bentur, Diamond and Berke (1998)]. Subsequently the rate of corrosion will be controlled either by the diffusion of oxygen through the uncracked cover to the cathodic areas or by the electrical resistance of concrete around the steel. Thus, the subsequent corrosion rates are







(b)

Figure 1: (a) Corrosion processes on the surface of steel, (b) Reaction at anodic and cathodic sites

limited by the same factors as for uncracked concrete.

Considerable research has been carried out describing the influence of cracking in concrete on reinforcement corrosion. Beeby (1978) concluded that corrosion was likely to start first where the rebar intersects a crack, and that in the short term (say, 1 to 2 years), there would be considerable influence of the crack width on the amount of corrosion and in the longer term, the influence of the width would be very small or negligible. This implies that once corrosion initiates, the influence of the crack on the corrosion rate is very small, which was supported by other studies [Arya and Ofori-Darko (1996); Schießl and Rapauch (1997)]. Nevertheless, several researchers did suggest a relationship between crack width and corrosion rates



Figure 2: Schematic description of factors that control the corrosion rate



Figure 3: Depassivation of the steel near the crack

[Francois and Maso (1988); Okada and Miyagawa (1980); Ohno, Praparntanatorn and Suzuki (1996)] on the basis that cracks permit air, salt and other impurities to penetrate to rebar and also make reinforced concrete so heterogeneous as to cause macrocell corrosion of reinforcing steel [Arya and Ofori-Darko (1996)]. Suzuki, Ohno, Praparntanatorn and Tamura (1990) showed that for specimens with a single crack, the steel at the crack intersection was depassivated earlier than that at uncracked sections, and in specimens with multiple cracks, corrosion initiated earlier at the major (wider) cracks, which was attributed to macrocell corrosion. Neverthe-

less, it was observed that the macrocell action was not strong enough to prevent subsequent corrosion at the minor cracks completely. In another work, Arya and Ofori-Darko (1996) found that the crack frequency was a governing factor in the rate of corrosion and suggested that the cover be increased to reduce the crack frequency. Mohammed, Otsuki, Hisada and Shibata (2001) observed that the crack frequency depends on the type of rebars, i.e. plain or deformed. A cracked beam with plain bars has fewer surface cracks as well as internal cracks compared to a cracked beam with deformed bars. Pettersson, Jorgensen and Fidjestol (1996) conducted both laboratory and field experiments (in marine atmosphere) with crack widths of 0.4 mm and 0.8 mm. They found that the corrosion rate decreased as a function of exposure time, which was attributed to the healing effect in the cracks and increase of concrete resistivity. Crack healing is the deposition of a solid material containing ettringite, magnesium hydroxide (brucite) and calcium carbonate [Mohammed, Otsuki and Hamada (2003)] which is beneficial for smaller crack widths. Water to cement (w/c) ratio appears to be the principal factor affecting macrocell corrosion in cracked concrete [Ohno, Praparntanatorn and Suzuki (1996)].

Most codes of practice specify maximum permissible crack widths from the durability point of view. BS 8110-2 (1985) states that for appearance and corrosion, the maximum surface width of crack should not exceed 0.3 mm. Similarly, in IS 456 (2000), the surface width of the crack, in general, should not exceed 0.3 mm in members where cracking is harmful and has serious adverse effects upon the preservation of reinforcing steel nor upon the durability of the structures. ACI 224 (2009) and ACI 318 (2009) limit crack widths to 0.4 mm, though it is recognised that corrosion is not clearly correlated with surface crack widths. CEB, the European design code (1989) also cautions that the limitation of crack widths is not the means to avoid corrosion in the case of severe chloride attack on horizontal concrete surfaces.

From the above studies, it is understood that reinforcement corrosion can be solved only by a combination of good concrete quality, adequate cover and crack width limitation. Though considerable research has been carried out on the influence of cracking on reinforcement corrosion, aspects regarding the type of binder and admixtures used, and width of pre-crack need more detailed study. An attempt to understand the effects of such factors on reinforcement corrosion so that they can be quantified and used in service life prediction is one of the main focuses of this work. In the present study, RC specimens with three water to cement (w/c) ratios 0.57, 0.47 and 0.37, representing commonly used concrete grades, are considered, having induced pre-cracks with surface widths of 0.2 mm and 0.4 mm (i.e., below and above, respectively, of the value of 0.3 mm specified by BS 8110-2 (1985) and IS 456 (2000).

3 Experimental Program

3.1 Materials Used

Commercially available ordinary portland cement (OPC), ordinary potable water and crushed granite aggregates, as per IS:383 (1997) with maximum size aggregate (msa) of 20 mm and 12 mm in the ratio of 1.5:1, were used. The chemical composition of the cement, as per IS 4032 (2004), is presented in Table 1, and the physical properties for the materials are presented in Table 2. Fe 415 grade high yield strength deformed bars (thermo mechanically treated bars - TMT bars) of 12 mm diameter were used as reinforcement.

Compound	OPC
CaO (%)	64.0
SiO ₂ (%)	18.8
Al ₂ O ₃ (%)	4.6
$Fe_2O_3(\%)$	3.7
MgO (%)	1.0
Loss on Ignition (LOI) (%)	3.6

Table 1: Chemical composition of cement

 Table 2: Physical and engineering properties

Physical Properties	OPC	Sand	Aggregate
Specific gravity	3.15	2.67	2.68
Bulk density (kg/m ³)	-	-	1600
Fineness modulus	-	2.78	6.81
Initial setting time	140 minutes	-	-
Final setting time	220 minutes	-	-
Compressive strength of mortar	48.0	-	-
cubes @ 28 days (N/mm ²)			

3.2 Preparation of Specimens

ACI 211 (2007) guidelines were followed in the concrete mix proportioning (see Table 3). Standard specimens such as cubes of $150 \times 150 \times 150$ mm for compressive strength, cylinders of 150 mm diameter and 300 mm height for split tension tests, and prisms of $100 \times 100 \times 500$ mm for flexural strength tests were cast. 75×150 mm

w/c	Cement	Sand	CA	Water
0.57	299	870	1056	170
0.47	362	815	1056	170
0.37	460	732	1056	170

Table 3: Quantities per cubic meter (kg/m^3)

and 100×200 mm cylinders were also cast for water absorption and rapid chloride penetration tests (RCPT).

Different methodologies have been adopted for initiating flexural cracks in test specimens used for corrosion studies [Francois and Maso (1988); Arya and Ofori-Darko (1996); Okada and Miyagawa (1980); Suzuki, Ohno, Praparntanatorn and Tamura (1990); Pettersson, Jorgensen and Fidjestol (1996); Mohammed, Otsuki and Hamada (2003); Raupach (1996); Rajamane, Chellappan, Neelamegam, Peter, Dattatreya, Prabakar, Srinivasan, Bhaskar, Sabita, Ambily and Harish (2008)]. A simple U-shaped specimen recently developed at the CSIR-Structural Engineering Research Centre (SERC) in Chennai, India, as shown in Fig. 4 is used for initiating flexural cracks of a required width. The specimen has a horizontal beam of $100 \times 100 \times 600$ mm and two integral vertical stubs of size 100×150 mm cross section and 200 mm height at the ends. High vield strength deformed steel bar of 12 mm diameter as shown in Fig. 4 is placed with a 20 mm clear cover at bottom. Holes were provided at the top of the stub to insert two tie rods for inducing a compressive force producing bending in the beam. After curing in water for 28 days and in air for 30 days, cracks of required width were created by stressing the tie rods with nuts. This led to multiple cracks on the surface of the beam. However, the maximum surface crack width was limited to either 0.2 mm or 0.4 mm. Both, 0.2 mm and 0.4 mm width cracks were observed on the surface to reach the reinforcement bars in all the specimens.

3.3 Accelerated Corrosion Test (Impressed Voltage Test)

The corrosion process was accelerated by impressing an anodic potential [Saraswathy and Song (2008); Güneyisi, Özturan and Gesolu (2006); Song, Saraswathy, Muralidharan, Lee and Thangavel (2009)] between the rebar anode and steel plate cathode, and recording the variation of current with time. Initially, the specimens were put in a 3.5% NaCl solution for 24 hours to ensure full saturation of the test specimen. The NaCl solution level in the plastic tray was maintained well above the rebar portion and is shown in Fig. 5 (a). Later, a constant impressed voltage of 10 V was applied for the specified durations to attain a weight loss of about



Mould and rebar



20% in the rebars of the uncracked specimens with w/c = 0.57. Fig. 5 (b) shows a photograph of the accelerated corrosion test specimens.

4 Discussions of Results

4.1 Standard tests

The 28-day compressive, split tensile and flexural strength results are presented in Table 4. These are the most commonly performed tests to evaluate the mechanical properties of concrete for structural purposes. Compressive and flexural strengths were evaluated as per IS 516 (2002). In each case, three specimens were tested and the average values were reported. The increase in compressive, split tensile and flexural strength is in the range of 34% to 62%, 58% to 62% and 36% to 56%, respectively, as the w/c ratio decreases from 0.57 to 0.37. Durability related properties such as water absorption on cylinders of 75×150 mm and rapid chloride ion







(b)

Figure 5: (a) Schematic of the accelerated corrosion test setup, (b) Photograph of the accelerated corrosion test specimens

penetration on discs of 100 mm diameter and 50 mm thick slices were determined as per ASTM C 642 (2003) and ASTM C 1202 (2003), respectively. The results of water absorption and chloride ion penetration are also presented in Table 4. For the selected concretes, the water absorption is in the range of 4% to 5% and therefore, the selected concretes can be graded as 'average permeability' concretes [Rendell, Jauberthie and Grantham, (2002)]. Based on ASTM C 1202, the chloride penetrability can be classified as 'moderate' for the selected concretes. It can be seen that as the w/c decreases, the mechanical properties are improved, as expected. Also, the water absorption and the charge passed decreased due to the reduction in permeability and electrical conductivity.

Description	0.57	0.47	0.37
Cube strength, MPa	33.2	44.7	53.7
Split tensile strength, MPa	2.4	3.8	4.0
Flexural strength, MPa	4.1	5.6	6.4
Water absorption, %	5.1	4.6	4.1
RCPT, Coulombs	2600	2100	1900

Table 4: 28 days mechanical and durability test results

4.2 Impressed voltage test

As described earlier, the U-shaped specimens were subjected to impressed voltage for a specified time duration to achieve an estimated weight loss of about 20% in the uncracked specimen with w/c = 0.57. The tentative time duration was estimated based on Faraday's law, by which the loss of iron over time can be obtained from the area under the curve of corrosion current versus time [Liu and Weyers (1996)]. In other words, the amount of corrosion is related to the electrical energy consumed and is a function of voltage, amperage and time interval [Cabrera (1996); Mangat and Elgarf (1999); Congqi, Lundgren, Plos and Gylltoft (2006)]. The theoretical weight loss was estimated based on the relationship between corrosion current and the weight of metal lost due to corrosion:

$$\Delta \omega = \frac{\text{AIt}}{\text{ZF}} \tag{3}$$

where:

$\Delta \omega$	metal weight loss due to corrosion
А	atomic weight of iron, (g)
Ι	corrosion current, (amp)
t	time elapsed (sec)
Ζ	valency of the reacting electrode (iron)
F	Faraday's constant

For this, the corrosion current was monitored and recorded every hour using an Agilent data logger. In order to attain a weight loss of about 20% in the uncracked

specimen of 0.57 w/c, a duration of 22 days was estimated using Eq. (3) and the same was adopted for all other series. It should be noted that the Eq. (3) was used only to fix the tentative time duration for accelerated impressed voltage test. The charges passed in all the series for 22 days (528 hours) are presented in Fig. 6. In each series, three specimens were used and the average values were reported. Each series is designated by the nomenclature 'cement type-w/c-pre-crack width'. It can be seen from Fig. 6 that a reduction in w/c from 0.57 to 0.37 reduced the total charge passed in both cracked and uncracked specimens. The charge passed in concrete specimens with w/c = 0.57 is higher than that of specimens with w/c = 0.47 and the charge passed for w/c = 0.47 is more than that of concrete with w/c = 0.37. Also, the charge passed is more in the cracked specimens compared to their corresponding uncracked specimens, which can be attributed to macrocell corrosion.

The corrosion current vs. time (change in charge passed) seen in Fig. 6 can be divided into three stages. The first stage is characterized by an initial decrease in current values with time, which may be attributed to the formation of iron oxide layer that protects the steel temporarily. The second stage is characterized by an increase in current due to the formation of microcracks that result from steel corrosion and facilitate the penetration of ions into the steel. The third stage is characterized by relatively small current variation. Similar observations were also made by Sakr (2005). It can be seen that concrete with a lower water to cement ratio prolongs the corrosion initiation or depassivation period (completion of first stage).

Since current is defined as the amount of charge that passes through a given conductor per unit time, the areas under the curves in Fig. 6 are proportional to the amount of corrosion products that are developed in the specimens [Holm (1987)]. The areas under the curve or the total charge passed are presented in Fig. 7. It can be seen that the total charge passed decreases with a decrease in the w/c and is higher for the cracked specimens due to macrocell corrosion, as reported by Okada and Mayagawa (1980). Moreover, as the crack width increases, the total charge passed also increases. The decrease in charge passed for lower w/c is due to increased impermeability and lower electrical conductivity of concrete as indicated by the lower water absorption and chloride ion penetration. The higher charge passed in cracked specimens is due to the fact that the time to depassivation is lowered by the cracking.

4.3 Corrosion induced cracks

Visual observations, in the initial stages of accelerated corrosion, indicated that the corrosion follows Mechanism 1 in uncracked specimens and follows Mechanism 2 in pre-cracked specimens. In the latter, rust stains were observed at the pre-



Figure 6: Plot of corrosion current versus test duration



Figure 7: Total charge passed for different concretes (with w/c = 0.57, 0.47 & 0.37)

cracks and later corrosion induced longitudinal cracks developed. In the uncracked specimens, more uniform corrosion was observed and the corrosion induced cracks developed along the rebar at later periods. After some time, more uniform corrosion was observed in both uncracked and pre-cracked specimens because the corrosion cracks were interconnected and a single longitudinal corrosion crack developed along the reinforcement. After the test duration, the specimens were taken out from NaCl solution to measure the corrosion induced crack widths after flushing the surface with water. Subsequently, the corrosion induced crack length (crack trajectory) and average crack widths in the middle 300 mm portion covered by the cathode plate were noted.

The crack width was measured at every 30 mm intervals using a crack comparator with an accuracy of 0.02 mm. The results are presented in Figs. 8 (a) and 8 (b). It can be observed that, for pre-cracked specimens, the average corrosion crack width was lower than that of uncracked specimens. Similarly, it was observed that the corrosion crack width decreases as the pre-crack width increases. Nevertheless, the corrosion induced crack length increases as the pre-crack width increases. The decrease in corrosion crack width with an increase in pre-crack width may due to the free outflow of corrosion products through crack openings that reduces the expansive force of the corroding rebar and consequently the tensile stress induced in the cover concrete. The increase in crack width with a decrease in w/c ratio may be due to the higher tensile strength resulting in increased build-up of corrosion products before corrosion crack develops.

After measuring the crack width and length along the crack trajectory, the middle 300 mm length horizontal portion of each specimen was split open and the rebar was visually examined for corrosion. It was observed that more uniform/generalized corrosion occurred in the rebars of the uncracked specimens, and both uniform and macrocell corrosion was observed in rebars of pre-cracked specimens.

4.4 Weight loss measurements

The actual amount of corrosion of the rebar was estimated by determining the gravimetric weight loss in the reinforcement bar, which was measured as the loss in weight over the middle 300 mm length portion of the rebar, and represents the average weight loss over the length considered:

$$CR = \frac{W_0 - W}{W_0} \times 100 \tag{4}$$

where, W_0 is the initial weight of the 300 mm length of rebar and W is the weight of the rebar after removal of corrosion products. After breaking open the specimen, the rebar was cleaned as per ASTM G1 (2003) to remove the rust; Fig. 9 shows the corroded rebars after cleaning. The initial weight was taken from the measured weight of rebar per running length.



(a)



(b)

Figure 8: (a) Corrosion crack widths in uncracked and pre-cracked specimens, (b) Corrosion crack lengths in uncracked and pre-cracked specimens

The gravimetric weight loss results are shown in Fig.10 and Table 5. The increase in weight loss is observed to be higher between the uncracked and 0.2 mm crack width specimens than between the 0.2 mm and 0.4 mm crack width specimens. That implies that the presence of the crack influences corrosion more than the crack width itself. This may be due to the effect of macrocell corrosion and can also indi-



Uncracked specimens



0.2 mm pre-cracked specimens



0.4 mm pre-cracked specimens

Figure 9: Corroded specimens and rebars after split open and cleaning, OPC concrete, 0.37 w/c ratio

cate a nonlinear relationship between the pre-crack width and degree of corrosion. The reductions in weight loss as w/c reduces from 0.57 to 0.47 and 0.57 to 0.37 are shown in Table 6. The improvement in corrosion inhibition capacity is in the range of about 23% to 33% whereas the increase in compressive strength is in the range of 34% to 62%.

Table 5: Gravimetric weight loss for in the different cases (averages from 3 specimens)

w/c ratio	0.57	0.47	0.37
Uncracked	18.51	13.95	11.90
0.2 mm-crack	30.20	23.12	18.20
0.4 mm-crack	33.40	25.21	22.43

5 Concluding Remarks

Based on the experimental results, the following conclusions can be drawn:



Figure 10: Gravimetric weight loss measurements

Table 6: Reduction in percent weight loss as the w/c ratio reduces from 0.57 to 0.37

w/c	uncracked	0.2 mm pre-crack	0.4 mm pre-crack
0.57 to 0.47	24.6%	23.4%	24.5%
0.57 to 0.37	35.7%	39.7%	32.8%

- 1. Basic durability test results show that as the w/c decreases from 0.57 to 0.37, the water absorption and chloride penetrability reduces, as can be expected.
- 2. Flexural/bending cracks can cause macrocell corrosion of rebars within the concrete, resulting in higher charge passed as the pre-crack width increases.
- 3. In the pre-cracked specimens, initially rust stains were observed at the precracks due to macrocell corrosion whereas, in the case of uncracked specimens, more uniform corrosion was observed. This shows that rebar corrosion initiates more rapidly in cracked concrete than in uncracked concrete, confirming that cracking influences the corrosion process.
- 4. The increase in weight loss is observed to be higher between the uncracked to 0.2 mm crack width specimens than between the 0.2 mm and 0.4 mm crack width specimens. This implies that the presence of the crack influences corrosion more than the crack width itself. This may be due to the effect of

macrocell corrosion and can also indicate a nonlinear relationship between the pre-crack width and degree of corrosion.

- 5. The rebar corrosion is lower in concrete with lower water to cement ratios, as indicated by a reduction in the weight loss of rebars. This is due to increased impermeability and lower electrical conductivity.
- 6. The results show that the corrosion induced crack width decreases as the pre-crack width increases. This is attributed due to more egress of corrosion products through crack openings. It is also seen that the corrosion induced crack width increases as the w/c decreases. This may be due to higher tensile strength resulting in increased build-up of corrosion products before corrosion crack develops.
- 7. The simple U-shaped specimen adopted in the present study for the accelerated corrosion test is found to be useful for the comparative evaluation of different concretes under cracked conditions in aggressive chloride induced corrosion.

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