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# Optimised cathodic protection design for maximum bond performance in reinforced concrete

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## Abstract

The influence of the long term effects of impressed current cathodic protection (ICCP) on the bond strength is investigated. A total of 16 pull-out specimens were divided into four series and the steel bars pre-corroded to target degrees of 0 % (control), 1 %, 2 % and 5 % weight loss. Four levels of ICCP current density ranging from approximately 0 (control) to over 1000  $\mu\text{A}/\text{cm}^2$  were applied to each series. These are considerably higher than the levels typically used in practice, which rarely exceed 2  $\mu\text{A}/\text{cm}^2$ , but were used to replicate long term application within a reasonable timescale. Following pull-out tests, the values of bond load were determined. The analysis accounts for the accelerated ICCP by relating the total charge to the steel reinforcement in coulombs (current x time) to the bond values and migration of chloride ions. The results show that although some loss in bond is evident from the ICCP, an optimised current density can be applied to a real structure without affecting the as-designed

bond strength in the long term e.g.  $\leq 0.65 \mu\text{A}/\text{cm}^2$  for an ICCP duration of 70 years. Application of the same current density reduces the chloride concentration from around the steel irrespective of the degree of corrosion.

## **1. Introduction**

Cathodic protection is an effective technique for the corrosion protection of steel in concrete [1-4]. It is important that these systems are correctly designed and installed as the strength and durability of concrete could potentially be reduced by over-protection through the application of an excessive cathodic current, leading to softening of the C-S-H gel [5, 6]. The Concrete Society's Technical Report No. 73 (TR 73) [7] recommend that a density of 0.02 to  $2 \mu\text{A}/\text{cm}^2$  ( $0.2\text{-}20 \text{ mA}/\text{m}^2$ ) can be applied to protect the steel for effective protection. For existing structures that are subject to heavy chloride contamination (in particular marine structures such as jetties, sea defences, offshore structures, coastal bridges, etc.), design currents in the range  $1\text{-}2 \mu\text{A}/\text{cm}^2$  ( $10\text{-}20 \text{ mA}/\text{m}^2$ ) will be required. For structures with only moderate or light chloride contamination, design currents as low as  $0.5 \mu\text{A}/\text{cm}^2$  ( $5 \text{ mA}/\text{m}^2$ ) can be sufficient to give protection. Generally, structures in higher ambient temperature locations will require higher current densities.

Where cathodic prevention is applied before corrosion of the steel has initiated, even lower current densities of the order of  $0.02$  to  $0.2 \mu\text{A}/\text{cm}^2$  ( $0.2$  to  $2 \text{ mA}/\text{m}^2$ ) are sufficient to ensure that the steel remains passive and that corrosion is prevented from occurring. As only very small current densities are required in such cases, much simpler anode systems are required. This type of application is commonly referred to as 'cathodic prevention'.

Alkali-silica reaction could also be accelerated due to the additional alkalinity generated by the application of the cathodic current [5], or the cathodic protection current may distribute non-uniformly due to the inherent physical and chemical inhomogeneity of concrete. This,

therefore, suggests that a current at the lower end of the scale is preferable to reduce the risk of localised over-protection.

Chloride ions may be expected to migrate away from the negatively charged steel surface when impressed current cathodic protection (ICCP) is applied [8]. The electrochemical extraction of chlorides in concrete largely relies upon the electro-migration of charged ions due to the applied DC field and direct absorption and electro-osmotic permeation of the electrolyte. Ionic electro-migration involves the transportation of both anions and cations caused by the repulsive electrical potential of the embedded steel reinforcement and external anode respectively. Anions including chlorides ( $\text{Cl}^-$ ), hydroxyls ( $\text{OH}^-$ ), carbonates ( $\text{CO}_3^{2-}$ ) and sulphates ( $\text{SO}_4^{2-}$ ) migrate away from the cathode (steel reinforcement) in the direction of the anode, typically toward the concrete surface where the anode is located. Cations, including calcium ( $\text{Ca}^{2+}$ ), potassium ( $\text{K}^+$ ) and sodium ( $\text{Na}^+$ ) ions, migrate to cathodic regions (in this case, the steel) in relative proportion to their respective transport numbers.

What is not in doubt is the ability of CP to increase the service life of a structure compared to the 'do nothing' approach that is commonly employed. An increased understanding of the influence of different levels of current density will help to optimise the design to ensure side effects are minimised or eliminated altogether. In this research, the impact of ICCP on bond between the steel and concrete and the migration of chlorides from around the steel was investigated and recommendations are made on optimising the current density.

## **2. Research significance**

An excessive level of ICCP current has been identified as a possible cause of reduction of the bond strength at the reinforcing steel and concrete interface, potentially leading to a reduction in the strength of structures [9, 10]. This can happen if ICCP systems are incorrectly designed or installed, but when functioning as-designed, it offers excellent protection to the steel reinforcement. Designers typically apply long term current densities in the range 0.02 to 2

$\mu\text{A}/\text{cm}^2$ . This paper aims to establish an optimum current density within this range for different levels of in-service use which will provide the long term performance as required by the designer. In particular, it will enable ICCP to be designed which will minimise any significant impact on bond in addition to confirming its influence on chloride ion migration from around the steel surface.

### **3. Experimental programme**

Corrosion of steel in concrete typically occurs over a long period. There is an initiation period where the concrete environment becomes conducive to the onset of corrosion, for example through chloride ingress. This is followed by a propagation stage where corrosion of the steel reinforcement occurs. These periods can take many years, if not decades, and will continue unless an intervention is applied which interrupts the rate of corrosion, such as the installation of an ICCP system, thereby ensuring the concrete structure reaches its intended design life. These three stages (initiation, propagation, intervention) can only be replicated in the laboratory by accelerating the time taken to reproduce deterioration and prevention to a reasonable level as follows:

- Initiation: accelerated by adding sodium chloride (3.5 % by weight of cement) to the concrete at the mixing stage.
- Propagation: accelerated by using an anodic impressed current technique at a level ( $1 \text{ mA}/\text{cm}^2$ ) between one and two orders of magnitude greater than the typical corrosion rates encountered on an actual structure. This current density has been used extensively in the past by the authors [e.g. 11-13] and is based on achieving the desired degree of corrosion within an acceptable timescale. The steel reinforcement was pre-corroded to four different degrees of corrosion (0 % (control), 1 %, 2 %, 5 %) which would be typically found in concrete structures before an intervention is applied. Lower degrees of corrosion yield a general corrosion (accumulation of very

small pits on the surface of the steel giving the appearance of general corrosion)

whereas deeper and more pronounced pits are evident at higher degrees of corrosion.

The influence of the different degrees of corrosion on the microstructure of the steel

[e.g. 14] is not considered as it is the bond characteristics between the steel and

concrete that is of primary interest. The accelerated technique meant that corrosion to

the specimens could be replicated in the laboratory between one and a half and eight

days.

- Intervention: accelerated by using current densities higher than would typically be used in the field. Four different levels of current densities ranging from 0 (control) to over  $1000 \mu\text{A}/\text{cm}^2$  were applied. These are considerably higher than the levels typically used in practice, which rarely exceed  $2 \mu\text{A}/\text{cm}^2$ . The different levels of current densities were analysed as an equivalent total charge (amps x time) to the steel within the test period ((0, 9, 32, 80) $10^3$  coulombs). Acceleration meant that 58 days ICCP in the laboratory at a current density of, say,  $114.6 \mu\text{A}/\text{cm}^2$  represented a total charge to the steel of  $9 \times 10^3$  coulombs. An in-service current density of, say,  $2 \mu\text{A}/\text{cm}^2$  would yield a total charge of only 157 coulombs over the same time period which was considered insufficient to yield any meaningful output. A similar approach has been applied elsewhere [15] and so forms the basis for this analysis.

### **3.1 Test specimens**

The pull-out test samples used in this investigation were based on BS EN 10080:2005 [16],

"Steel for the reinforcement of concrete - Weldable reinforcing steel - General, Annex D,

Bond test for ribbed and indented reinforcing steel – Pull-out test". The specimens were 100

mm sided cubes with a 10 mm diameter steel bar located centrally. Although the test standard

followed was based on ribbed and indented bars [16], plain bars were used in this test as a

worst case scenario. A large number of existing reinforced concrete structures were

constructed with plain reinforcing bars before the 1970s [17]. Therefore, older reinforced concrete benefitting from cathodic protection would have plain bars as reinforcement so test results would better suit this type of structure. The bonded length was 5 times the diameter. This embedment length was selected to avoid yielding of the steel bar under pull-out loading. A plastic sleeve was used to provide an un-bonded length of 50 mm; it was fitted onto the steel bar before placing the bar into the mould. The internal diameter of the sleeves gave a 1.5 mm tolerance around the bar. The steel bar protruded beyond the two ends of the concrete cube and the pull-out force was applied to the longer threaded end. In order to ensure that only the bonded length of the steel bar was corroded, a proprietary polyester resin mortar was applied to the protruding end of the steel bar, thereby, insulating it from corrosion. In addition, the same polyester resin was used to seal the end of the sleeve to prevent grout entering and possibly influencing the bond. A schematic diagram of the bond sample is shown in Fig. 1. The test sample series are detailed in Table 1.

Referring to Table 1, the main variable across series S1, S2 and S3 is the target degree of corrosion, namely 1 %, 2 % and 5 % and the ICCP current densities which were  $114.6 \mu\text{A}/\text{cm}^2$ ,  $407.6 \mu\text{A}/\text{cm}^2$  and  $1019.1 \mu\text{A}/\text{cm}^2$  respectively. With regards to the corrosion of the specimens, the current density remained constant at  $1 \text{ mA}/\text{cm}^2$  but the corrosion time varied between 2250, 4500 and 11250 minutes for the 1 %, 2 % and 5 % samples respectively. Each series also included a control specimen without ICCP application. The non-ICCP sub-group S\*1 includes control specimens (S1.1, S2.1, S3.1). The ICCP specimens were divided into 3 sub-groups. Group S\*2 consisted of the samples cathodically protected with an applied current density of  $114.6 \mu\text{A}/\text{cm}^2$  (S1.2, S2.2 and S3.2) over 1390.5 hours. Group S\*3 (S1.3, S2.3, S3.3) and Group S\*4 (S1.4, S2.4, S3.4) were cathodically protected with applied current densities of  $407.6 \mu\text{A}/\text{cm}^2$  and  $1019.1 \mu\text{A}/\text{cm}^2$  respectively over 1390.5 hours (Table 1).

## **3.2 Material properties**

### *3.2.1 Concrete*

The concrete mix was cast in the laboratory using a concrete with target cube strength of 40 N/mm<sup>2</sup>. Mix proportions were 1:2:3:0.5 of ordinary Portland cement: fine aggregate:coarse aggregate:water. Fine and coarse aggregates were oven dried at 100°C for 24 hours. All specimens were produced carefully to ensure the same quality and strength throughout. The concrete was cast in steel moulds in three layers, each layer being carefully compacted on a vibrating table. The specimens were then placed under polyethylene sheets. The samples were demoulded after 1 day and cured in water at 20 °C for a further 27 days (28 days in total). Shrinkage, as a result, was therefore eliminated so would not influence the bond. Since the concrete remained unloaded, creep would also be absent, again providing a worst case scenario.

### *3.2.2 Sodium chloride*

Sodium chloride (GPR grade) was added to stimulate chloride induced corrosion of the reinforcement. Sodium chloride was added to the concrete mix at 3.5 % by weight of cement.

### *3.2.3 Steel reinforcement*

The bars used in these tests were plain reinforcing steel of Grade 250, 10 mm diameter, 255 mm length with a yield strength of 250 MPa. Plain steel bars were used to promote uniform bond distribution. One end of each steel bar was threaded for connecting to the loading grip of the testing machine. All steel bars were cleaned in a 5 % solution of di-ammonium hydrogen citrate (C<sub>6</sub>H<sub>14</sub>N<sub>2</sub>O<sub>7</sub>) for 4 hours and rinsed with clean water containing a corrosion inhibitor (V<sub>p</sub>CI-337) to prevent further reaction of di-ammonium hydrogen citrate with the steel. Subsequently, all steel bars were dried and their weights recorded before casting the bond test samples.



### 3.3 Accelerated corrosion

An anodic impressed current technique was used to accelerate the corrosion of the steel bars to their pre-determined degree of corrosion as given in Table 1. After 28 days, each bond specimen was partly immersed in 3.5 % NaCl solution (Fig. 2). The un-bonded length of the steel was protected by the polymer sleeve and the level of electrolyte was kept below the sleeve to prevent corrosion of the protruding bar. The steel bars in the bond specimens were corroded by connecting to the positive terminal of a direct current (D.C) power supply, acting as the anode while an austenitic stainless steel plate (Type 316/1.4401) was employed as the non-corroding cathode.

The duration required to achieve the pre-determined level of corrosion was calculated using Faraday's Law for each specimen. The percentage reduction in reinforcing bar diameter in  $T$  years,  $\frac{2RT}{D} \times 100(\%)$ , was defined as the degree of reinforcement corrosion in which  $R$  (cm/year) is the metal section loss per year,  $D$  (cm) is the diameter of the steel bar [11, 18, 19]. A current density,  $i$ , of  $1 \text{ mA/cm}^2$  was adopted in this investigation and three different percentages of corrosion were selected following trials:  $m = 1 \%$ ,  $2 \%$  and  $5 \%$ . A  $5 \%$  limit was imposed to ensure cracking did not occur in the concrete which would adversely affect the bond strength.

The applied current and its duration required for accelerated corrosion of steel in each bond specimen is given in Table 1. Equation 1 was used to determine the time taken to achieve these degrees of corrosion:

$$\frac{2 \times R \times T}{D} = \frac{m}{100}$$

Equation 1

where

$R$  = material loss per year due to corrosion ( $1165i$ )

$T$  = Time (years)

$D$  = diameter of the tendon (cm)

Rearranging Equation 1 and substituting  $R = 1165i$  gives

$$T = \frac{m \times D}{200(1165 \times i)}$$

Equation 2

Substituting,  $m = 1$  %,  $i = 1$  mA/cm<sup>2</sup> and  $D = 1$  cm into Equation 2 gives:

$$T = \frac{1 \times 1}{200(1.165)} = 0.00429 \text{ yrs} = 2250 \text{ min s}$$

Equation 3

The length of steel reinforcement,  $L$ , for corrosion is 5 cm and the total surface area,  $a$ , is shown in Equation 4:

$$a = \pi \times D \times L = 5\pi \text{ cm}^2$$

Equation 4

The current required for 1 % degree of corrosion per steel reinforcement is obtained from:

$$I = i \times a = 1 \times 5\pi = 15.7 \text{ mA} / \text{ steel reinforcement}$$

Equation 5

Applying the same procedure above for 2 % and 5 % degree of corrosion, the time required is increases proportionally for 2 % (4500 mins) and 5 % (11250 mins). The current remained constant at 15.7 mA.

A total of 12 pre-corroded ICCP specimens (S\*2, S\*3, S\*4, Table 1) were cured in water after inducing corrosion for further periods of 14 to 56 days before application of ICCP. This was to ensure that all corroded steel bars achieved stable potentials before the ICCP application stage of the testing.

### **3.4. ICCP application**

After accelerated pre-corrosion of steel bars followed by 14 to 56 days of further curing in water, ICCP was applied to all ICCP specimens (S\*2, S\*3 and S\*4). The ICCP current densities applied to three sub-group specimens (S\*2, S\*3 and S\*4) are described in Table 1 and Section 3.1. The duration of ICCP operation for the three groups was 1390.5 hours (approximately 2 months).

The ICCP was applied to each group with the pre-corroded steel bars connected to the negative terminal of a direct current (DC) power supply working as the cathode while a mixed metal oxide (MMO) coated titanium ribbon was employed as the anode and connected to the positive terminal. The MMO titanium ribbon was shaped around the specimens to ensure an even current distribution (Fig. 3). Tap water was used as the electrolyte. Prior to ICCP application, the rest potentials of the steel bars were measured using a Cu/CuSO<sub>4</sub> reference electrode. These were later converted to Ag/AgCl/0.5M KCl reference electrode equivalent values based on TR 73 [7]. During the ICCP operation, the ‘on’ and ‘instant-off’ potentials of steel bars of the three sub-groups of specimens (S\*2, S\*3 and S\*4) were recorded. All remaining control specimens S\*1 (S0.1, S1.1, S2.1 and S3.1, Table 1) were stored in a water tank until the pull-out tests commenced.

### **3.5 Pull-out test**

All specimens were continuously stored in water for 7 days after the completion of the ICCP application to ensure that the corrosion products at the steel/concrete interface were stabilized. This was also aimed at eliminating the effect of shrinkage on bond strength. The pull-out test was then conducted using a specially designed loading frame which was attached to a tensile testing machine (see Fig. 4). The load from the testing machine was recorded using a video camera. The loading rate was 50N/s based on the EN 10080:2005(E) standard [20].

This type of concentric pull-out test is widely used due to its relative simplicity. It is employed to compare the bond strength of steel in different concretes and is, therefore, suitable for comparing the bond strength of samples in this test. The concrete surrounding the tensile reinforcement in reinforced concrete flexural members is in tension. In this test, the concrete is in compression which eliminates tension cracks in the concrete and tends to increase the measured bond strength [21]. However, it is relative bond strength which is of interest in this test. After the pull-out test, the specimens were broken apart and the steel bars were cleaned to removed corrosion product with 10 % di-ammonium hydrogen citrate solution for calculation of the actual degree of corrosion.

### **3.6. Chloride analysis**

After conducting the pull-out test, the steel bars were removed and concrete around the steel bars was chemically analysed to determine the chloride concentration. Concrete powder samples from near the steel bar surface (within 10 mm) was extracted using a masonry drill [9]. The chloride concentration of the powder was determined according to BS 1881-124: 2015 [22]. Sub-samples of the concrete powder (2 g) were extracted in hot, 20 % nitric acid. The extract was cooled and neutralised with calcium carbonate. The mixture was then diluted with water and potentiometric titration was performed with a standardised titration unit using silver nitrate as the titrant.

## **4. Results and Discussion**

### **4.1 Actual degree of corrosion**

The gravimetric mass loss method was used to calculate the degree of corrosion of steel bars. A satisfactory correlation between target and actual corrosion is evident (Table 2). For example, for the 1 % target degree of corrosion, the four actual degrees of corrosion ranged between 1.1 % and 1.4 %. The 2 % target degree of corrosion had actuals between 2.1 % and 2.3 % and the 5 % target degree of corrosion ranged from 5.0 % to 5.6 %. The appearance of

the bars were similar to those found in a naturally corroding environment, hence the accelerated technique was successfully applied.

#### **4.2 Effectiveness of ICCP**

The rest potentials of the steel bars of groups S\*2, S\*3, S\*4 prior to application of ICCP were -503 mV, -468 mV and -495 mV (vs Ag/AgCl/0.5M KCl) respectively. These recorded half-cell potentials were more negative than -250 mV (Ag/AgCl/0.5KCl), representing a high (>90 %) risk of corrosion according to ASTM C876 [23]. The potentials and potential decays of the steel bars recorded during the operation of the ICCP are plotted in Fig. 5. Referring to Fig. 5, it shows that the initial 'on' potential of sub group samples S\*2, S\*3, S\*4 were 1397 mV, 2446 mV, 5100 mV (vs Ag/AgCl/0.5M KCl) at applied ICCP current densities of 114.6  $\mu\text{A}/\text{cm}^2$ , 407.6  $\mu\text{A}/\text{cm}^2$  and 1019.1  $\mu\text{A}/\text{cm}^2$  respectively. After 100 hours from the start, the 'on' potential of sub group samples S\*2 and S\*3 stabilised at approximately -1500 mV (vs Ag/AgCl/0.5M KCl) and -1900 mV (vs Ag/AgCl/0.5M KCl) respectively but due to the higher ICCP current application of sub group samples S\*4 (1019.1  $\mu\text{A}/\text{cm}^2$ ) on potentials were recorded either side of -3000 mV (vs Ag/AgCl/0.5M KCl). The ICCP application was terminated at 1390.5 hours and the potential decays after 4 hours were recorded. The potential decays of all steel bars from instant-off were 542 mV, 864 mV, 2279 mV for sub-groups samples S\*2, S\*3 and S\*4 respectively. These potential decays were more than 100 mV after 4 hours which, according to BS EN ISO 12696-2016 [24], confirms the achievement of effective cathodic protection.

#### **4.3 Influence of current density on bond load of pre-corroded steel**

The ultimate bond loads of the different sample groups are given in Table 3. The relationship between degree of corrosion and bond load for the various current densities is plotted in Fig. 6. Referring to Fig. 6, a line of best fit in the form of a polynomial is given for each of the four cases. Despite bond test results being notably difficult to analyse due to scatter, the

results presented in Fig. 6 exhibit a very good trend. The Control samples (S\*1) which were not exposed to CP show that the datum bond load at 0 % corrosion is 6049 N and this is highlighted in Fig. 6. The line of best fit for these samples (S\*1) indicate a peak bond load at about 5 % degree of corrosion. This is consistent with the influence of the expansive corrosion products providing a mechanical interlock as a result of the roughened surface of the steel enhancing bond up to about 5 % for steel in concrete (Almusallam et al attained a maximum bond strength at 4 % degree of corrosion [21]). Beyond 5 %, the bond load drops off as the degree of corrosion increases giving a loss in bond strength. At levels of corrosion higher than those used in this study, there is a risk of the cover cracking and the bond being compromised. Specimens S\*2 exhibit a shallower increase in bond load with increasing degree of corrosion. The bond load at 0 % corrosion is marginally lower (5746 N) compared to S\*1. Based on the line of best fit, the peak bond load occurs at 5-6 % loss of cross sectional area in corrosion, similar to specimens in S\*1.

The lowest increase in bond occurs in Specimens S\*3 and S\*4 (Fig. 6). The bond load for these groups at 0 % corrosion is 3160 N and 1580 N respectively, much lower than the datum value of 6049 N for S\*1. Their peak bond load is again around 5-6 % degree of corrosion. Based upon the data given in Fig. 6, the application of the protection current leads to loss of bond, especially at very high current density levels. However, the current densities applied ( $114-1019 \mu\text{A}/\text{cm}^2$ ) are much higher than those normally used in properly controlled ICCP systems (e.g.  $0.02$  to  $2 \mu\text{A}/\text{cm}^2$ ). This is taken into account in the following sections by adopting an approach used elsewhere [15] by relating the total charge applied to the steel reinforcement to long term in-service performance.

#### **4.3.1 Optimising current density for cathodic protection**

An appreciation of the impact of long term ICCP on bond can be developed by investigating the magnitude of the total charge passed within the period of the tests and comparing this to

the total charge throughout the lifetime of a properly designed and controlled ICCP system.

Equation 6 can be used to determine the charge passed as follows:

$$C = (I_{cp})(t)$$

Equation 6

where  $C$  is the charge in coulombs,  $I_{cp}$  is the cathodic protection current in amps and  $t$  is the time in seconds. The total charge passed over the specimens is given in Table 4, cols. 1-6. The assumption is made that ICCP will be applied to the reinforced concrete structure when corrosion levels reach say 5 % (hence cathodic protection as opposed to cathodic prevention), corresponding to the peak bond loads in Fig. 6. Referring to Fig. 6, the bond load at these four charges is given in col. 7 (rounded to the nearest hundred Newtons for simplicity). The data presented in Table 4, cols. 6 and 7 is shown graphically in Fig. 7. It shows a reduction in bond load as the total charge increases considerably. By inspection of Fig. 7, a zero reduction in bond load i.e. the bond which would have been present before any corrosion or application of ICCP, corresponds to approximately  $22.5 \times 10^3$  coulombs, meaning a total charge,  $C$ , in excess of this magnitude would need to be applied before the bond becomes worse than the as-constructed stage (zero corrosion and zero ICCP).

The in-service current density is normally between  $0.02$  to  $2 \mu\text{A}/\text{cm}^2$  which can be converted to current (amps) for the steel used in this research. Substituting  $C = 22.5 \times 10^3$  coulombs and a range of current values ( $I_{cp}$ ) based on a current density between  $0.02$  to  $2 \mu\text{A}/\text{cm}^2$  for  $C$  in Equation 6 yields the relationship as shown in Fig. 8. A line of best fit gives the relationship:

$$i_{cp} = 45.444(T)^{-1}$$

Equation 7

where  $i_{cp}$  is the cathodic protection current density and  $T$  is the time in years.

According to Table 2.1 in BS EN 1990 [25], indicative working lives are given for design purposes for various types of structures. Bridges and other civil engineering structures are classed as Category 5 and have an indicative design working life of 100 years. The total design life of a bridge consists of the corrosion initiation phase and corrosion propagation phase and there are numerous publications on predicting the long term performance of concrete structures [e.g. 26, 27], with many variables responsible for the variances in predicted lifespan. Therefore, if it is assumed that the corrosion initiation phase is 20 years and propagation phase to reach a degree of corrosion of 5 % is a further 10 years (see Fig. 9 [28]), then an intervention, such as the installation of an ICCP system, will be required to keep the structure in service for a further 70 years.

Therefore, the current density to give an in-service life of, say, 70 years before a reduction in bond below the datum is achieved can be found by substituting  $T=70$  into Equation 7. This gives a current density,  $i_{cp}$ , of  $0.65 \mu\text{A}/\text{cm}^2$ . If the time of initiation and propagation change, simply subtract this total from the design life of 100 years to give a new value for  $T$ . For example, if the time of initiation and propagation is 60 years then  $T=40$  years. Therefore, a current density,  $i_{cp}$ , of up to  $1.14 \mu\text{A}/\text{cm}^2$  from Equation 7 could be applied without a net loss in bond strength.

The recommended current densities as described earlier from TR 73 [7] highlighted that existing structures that are subject to heavy chloride contamination should have ICCP design currents in the range  $1\text{-}2 \mu\text{A}/\text{cm}^2$ . This research shows that reducing this to  $\leq 0.65 \mu\text{A}/\text{cm}^2$  would be beneficial to ensure an ICCP working life of 70 years would be achieved without loss of as-constructed bond strength, or  $1.14 \mu\text{A}/\text{cm}^2$  if ICCP is applied for only 40 years. It was also recommended that for structures with only moderate or light chloride contamination, design currents as low as  $0.5 \mu\text{A}/\text{cm}^2$  can be sufficient to give protection. This current density, if substituted in Equation 7, would give  $T=91$  years and when added to the initiation and



propagation duration (say 30 years) would give an in-service life of 121 years, greater than the design life of the bridge.

Therefore, the analysis has shown that the benefit of an increased bond due to corrosion ( $m \sim 5\%$ ), coupled with knowledge of the combined initiation and propagation period to reach this level of corrosion, can be used to design ICCP systems for optimised performance.

#### **4.3.2 Optimising current density for cathodic prevention**

Lower current densities of the order of  $0.02\text{-}0.2\ \mu\text{A}/\text{cm}^2$  are recommended for installation at the construction stage for cathodic prevention (steel remains passive and corrosion free i.e.  $0\%$ ) [7]. This is different to the application of an ICCP system for cathodic protection as shown in Section 4.3.1 where a permissible level of corrosion was assumed before a current density was applied. To get an indication of the impact of ICCP on bond for steel in the passive state, the bond load against the total charge applied for the specimens without pre-corrosion ( $0\%$  degree of corrosion, samples S0.1-S0.4, Table 3) is shown in Fig. 10.

Referring to Fig. 10, there is a slight decrease in bond load between S0.1 and S0.2 ( $\sim 5\%$ ), the latter having a total charge of 9007 coulombs applied. Reduction in bond loads for S0.3 and S0.4 exhibit a much steeper decline but the total charge applied to these is well outside what would normally be applied in-service. For example, assuming ICCP is applied for the 100 years design life duration, a current density of the maximum allowed in this category ( $0.2\ \mu\text{A}/\text{cm}^2$ ) would yield a charge of 9902 coulombs whereas the total charge for the lower current density ( $0.02\ \mu\text{A}/\text{cm}^2$ ) would be 977 coulombs (total charge for S0.3 and S0.4 was  $32 \times 10^3$  and  $80 \times 10^3$  respectively). If it is assumed that the bond over the 100 year lifespan should not decrease by more than  $5\%$  (which corresponds to S0.2 in Fig. 10), then reducing the total charge from 9902 coulombs ( $i_{cp} = 0.2\ \mu\text{A}/\text{cm}^2$ ) to 9007 coulombs would require a reduction in the upper limit current density to  $0.18\ \mu\text{A}/\text{cm}^2$ . A bond load reduction of this magnitude is acceptable compared to the implications of the 'do nothing' approach as

identified in the next section. Therefore, current densities in the range 0.02-0.18  $\mu\text{A}/\text{cm}^2$  are recommended for cathodic prevention in new build structures.

#### 4.3.3 Comparison between ICCP application and 'do nothing'

The data presented in this paper focuses on the design of ICCP and provides recommendations for optimising the current density for efficient long term performance. However, a common approach is to 'do nothing' meaning reinforced concrete structures will deteriorate in the propagation phase until a level of damage ensues making the structure potentially unsafe. There is also the high risk that the design life (typically 100 years) will not be met (Fig. 9 [28]). An indication of the impact of the 'do nothing' approach can be obtained with reference to Fig. 6. The line of best fit for the control specimens S\*1 (no ICCP) gives a relationship:

$$y = 373.61x^2 + 3726.7x + 5751.1$$

Equation 8

Substituting  $y = 6000$  (i.e. datum bond load) into Equation 8 gives a degree of corrosion of 9.9 %, meaning this level of corrosion is required before the bond reduces to as-designed levels. Substituting  $y = 0$  into Equation 8 and solving the quadratic equation yields a positive root of 11.3 %. This is a theoretical value meaning the bond load would disappear completely at 11.3 % degree of corrosion due to the sharp decline of the curve. It has to be acknowledged, however, that there are only four points on the graph but nevertheless follows structural convention where low corrosion can increase bond but higher levels lead to bond breakdown [21]. In a previous publication by O'Flaherty *et al* [12], it was stated that '*beams exhibiting main steel corrosion greater than 10 % generally failed in flexure before reaching the service load. Therefore, beams in practice with main steel corrosion approaching 10% should be considered as reaching their serviceability limit state and repair and maintenance is required to extend their service life*'. If this 10 % limit is the 'significant level of damage' (or maximum

permissible corrosion) as shown in Fig. 9 [28], the optimum time to apply CP should be when the corrosion has advanced by no greater than 5 %. The increased bond as a result will be beneficial but despite the long term perceived reduction in bond due to the application of CP, the benefits far outweigh a 'do nothing' approach. If it is assumed that the time to achieve a 5 % degree of corrosion is 30 years (initiation plus propagation), then the ~10 % degree of corrosion would be reached long before the design life of the structure has been realised. Hence, an intervention is required and ICCP, designed on the recommendations given in this paper, will enable the 100 year lifespan to be achieved.

#### **4.4 Influence of ICCP on chloride migration at optimised current density**

Chloride concentrations obtained from concrete bond samples adjacent to the steel bars from the different degree of corrosion are shown in Table 3. The relationships between chloride concentration, current densities and degree of pre-corrosion are shown in Fig 11.

Referring to Fig. 11, it is apparent that each degree of corrosion exhibits a similar profile. Chloride concentrations (by mass of cement based on 18 % cement content) averaged 2.73 % for the control specimens (no ICCP applied) but these concentrations decreased as the total charge increased. BRE Digest 444 Part 2 [29] suggests that a chloride ion content of 1-2 % by weight of cement would be in the extremely high risk category so the average chloride concentration at zero charge (2.73 %) is very high. At a charge of  $80 \times 10^3$  coulombs, the chloride concentration reduced to an average of 0.23 %.

Therefore, the chloride concentration reduction due to ICCP application does not depend on the degree of corrosion, but rather on the magnitude of the ICCP current densities. A charge of  $80 \times 10^3$  coulombs is very high. Referring to Section 4.3.1, it was suggested that the application of an ICCP current density ( $i_{cp}$ ) of  $0.65 \mu\text{A}/\text{cm}^2$  would enable a bridge to reach its design life of 100 years with ICCP applied for 70 years without any loss in bond (compared to the datum). This was the equivalent of a total charge of  $22.5 \times 10^3$  coulombs at  $m = 5$  %. By

inspection of Fig. 11, the average chloride concentration at this charge is approximately 0.7 % meaning the chloride concentration expressed by mass of cement has reduced, on average, by 2 %. Referring to BRE Digest 444 [29], this equates to a low to moderate risk of steel reinforcement corrosion. This shows that the application of ICCP is beneficial in migrating chloride ions away from the steel surface and in this instance, a higher charge leads to a higher migration of chlorides.

## 5. Conclusions

The main conclusions from the results reported in this paper are as follows:

1. Bond load increases as the degree of corrosion increases from 0 % to about 5 %. If an intervention in the form of ICCP is not made, then the as-designed service life of 100 years for bridges is unlikely to be met due to the significant level of damage encountered due to steel reinforcement corrosion
2. The reduction in bond due to the application of ICCP is offset by the mechanical interlock as a result of the roughened surface of the steel. Based on the findings in this paper for steel corroded to about 5 % loss of cross sectional area, designing ICCP systems based on the time remaining to reach the design life of 100 years, an optimised current density can be obtained. As examples, limiting the current density to  $0.65 \mu\text{A}/\text{cm}^2$  for an ICCP application of 70 years enables an in-service life of 100 years to be reached with no loss in the as-designed bond characteristics. The current density can increase to  $1.14 \mu\text{A}/\text{cm}^2$  if the ICCP duration is reduced 40 years, again with no net effect on bond loss
3. Lower current densities of the order of  $0.02\text{-}0.18 \mu\text{A}/\text{cm}^2$  recommended for installation at the construction stage for cathodic prevention are satisfactory. There is only a 5 % loss of bond predicted for a 100 year design life which highlights the

benefits of adopting ICCP for new construction compared to the 'do nothing' approach (no installation of ICCP)

4. The chloride concentration in the concrete near the steel bar surface reduces with the application of ICCP. At an in-service current density of say  $0.65 \mu\text{A}/\text{cm}^2$ , the chloride concentration level reduced to 0.73 % by mass of cement from a high starting level of 2.73 %.

The present study is based on the accelerated corrosion and cathodic protection methods which may differ from the natural corrosion process and CP technique. In addition, the effect of concrete mix of proportions and different diameters of steel, confinement etc. on bond behaviour is beyond the scope of this study and needs further investigation.

#### **Compliance with ethical standards**

**Conflict of interest:** The authors declare that they have no conflict of interest.

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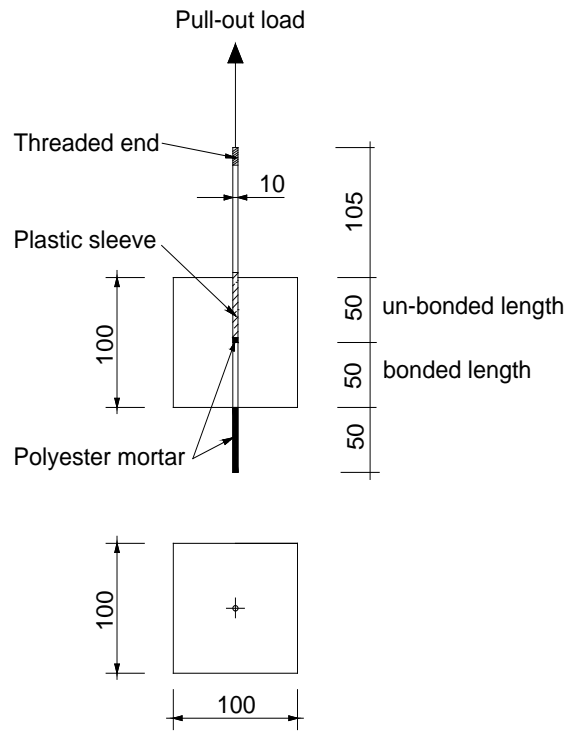
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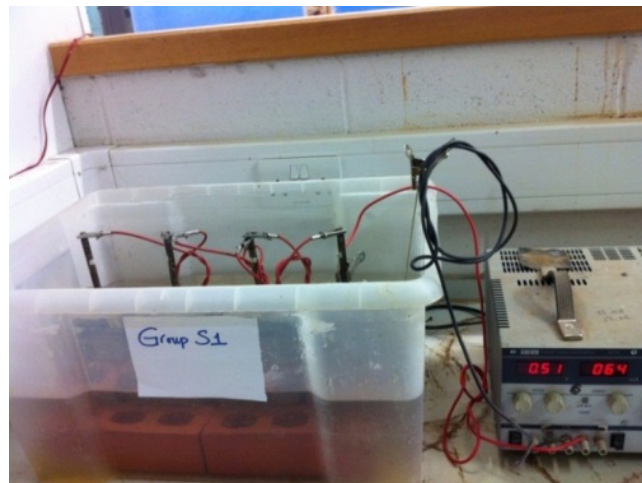
#### **References**

- [1] P. Lambert, Anti-Corros. Methods Mater. 1999, 42, 4.
- [2] C.H. Haldemann, A. Schreyer, presented at EUROCORR'97, Trondheim, Norway, 22-25 September, 1997, pp. 18.
- [3] US Federal Highway Administration, Memorandum on FHWA position on cathodic protection systems, 1982.
- [4] P. Pedferri, Constr. Build. Mater. 1996, 10, 391.
- [5] M.G. Ali, Rasheeduzzafar, ACI Mater. J. 1993, 90, 247.

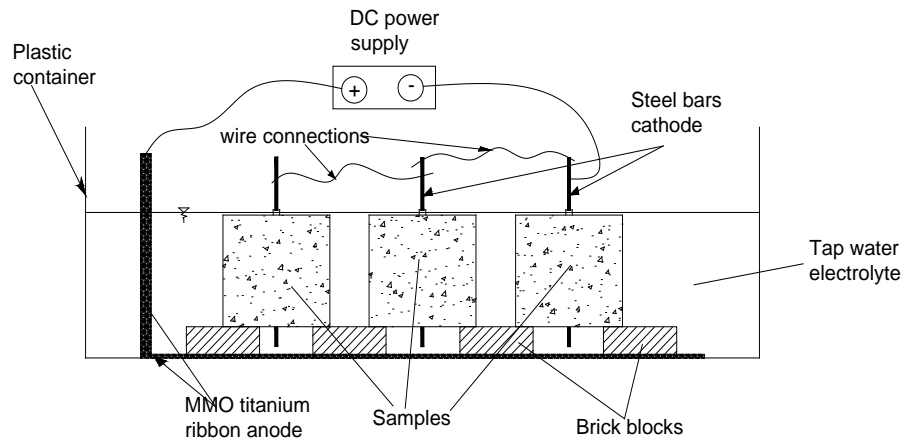
- [6] C.E. Locke, C. Dehghanian, L. Gibbs, presented at NACE Corrosion 83, Anaheim, U.S. 1983, 178/1.
- [7] The Concrete Society, Cathodic protection of reinforced concrete. Technical Report No 73, 2011.
- [8] N.M. Ihekweba, B.B. Hope, C.M. Hanssont., Cem. Concr. Res. 1996, 26, 267.
- [9] J.J. Chang, Cem. Concr. Res. 2006, 32, 657.
- [10] Rasheeduzzafar, M.G. Ali, G.J. Al-Sulaimani, ACI Mater. J. 1993, 90, 8.
- [11] P. Lambert, C. Van Nguyen, P.S. Mangat, F. J. O'Flaherty, G. Jones, Mater. Struct. 2015, 48, 2157.
- [12] F.J. O'Flaherty, P.S. Mangat, P. Lambert, E. Browne, presented at IABMAS '08, 2009, p. 232.
- [13] F.J. O'Flaherty, P.S. Mangat, P. Lambert, E.H. Browne, Mater. Struct. 2008, 41, 311.
- [14] U. Angst, B. Elsener, C. Larsen, Ø. Vennesland, Electrochim. Acta, 2011, 56, 5877.
- [15] J.J. Chang, Y. Weichung, R. Huang, J. Mar Sci Tech, 1999, 7, 89.
- [16] British Standards Institution, BS EN 10080: 2005(E) Annex D, Bond Test for Ribbed and Indented Reinforcing Steel- Pull out Test.
- [17] E. Hertanto, Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand, 2005.
- [18] C.V. Nguyen, P. Lambert, P.S. Mangat, F. J. O'Flaherty, G. Jones, Struct. Infrast. Eng. 2015, 12, 356.
- [19] P. Lambert, C.V. Nguyen, P.S. Mangat, F.J. O'Flaherty, G. Jones, Mater. Struct. J., 2015, 48, 2157.
- [20] British Standards Institution. BS EN 10080, 2005, Steel for the reinforcement of concrete. Weldable reinforcing steel.
- [21] A.A. Almusallam, A.S. Al-Gahtani, A.R. Aziz, Rasheeduzzafar, Constr. Build. Mater. 1996, 10, 123.
- [22] British Standards Institution, BS 1881-124: 2015, Testing concrete- Part 124: Method for analysis of hardened concrete.
- [23] American Society for Testing and Materials, Standard Test Method for Half-cell Potentials of Uncoated Reinforcing Steel in Concrete, 2015, ASTM C876-15
- [24] British Standards Institution, BS EN ISO 12696-2016, Cathodic Protection of Steel in Concrete.
- [25] British Standard Institution, Eurocode — Basis of structural design. BS EN 1990:2002 +A1:2005. Incorporating corrigenda December 2008 and April 2010
- [26] S. Dhawan, S. Bhalla, B. Bhattacharjee presented at 9th International Symposium on Advanced Science and Technology in Experimental Mechanics, 2014, New Delhi, India, 1-6 November.
- [27] L. Bertolini, Struct. Infra. Eng. 2008, 4, 123.
- [28] K. Tuutti, Report 4-82, 1982, Swedish Cement and Concrete Research Institute.
- [29] BRE Centre for Concrete Construction. Corrosion of steel in concrete, 2000, Investigation and assessment. Digest 444 Part 2. CI/SfB q4.



**Fig.1.** Details of specimens (all dimensions in mm)



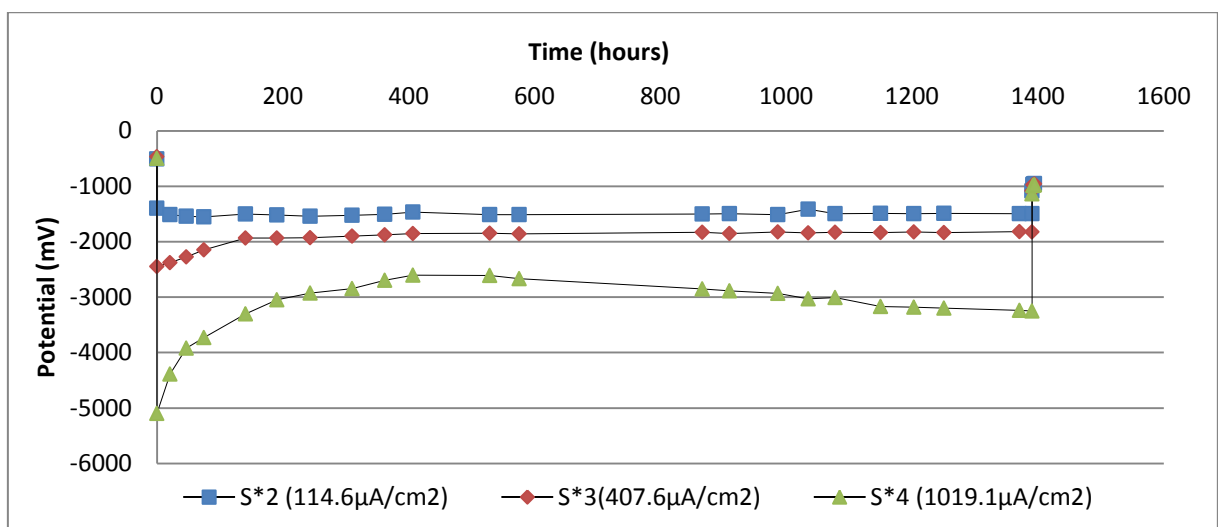
**Fig.2.** Accelerated corrosion of steel bars



**Fig.3.** Schematic ICCP application to group of specimens

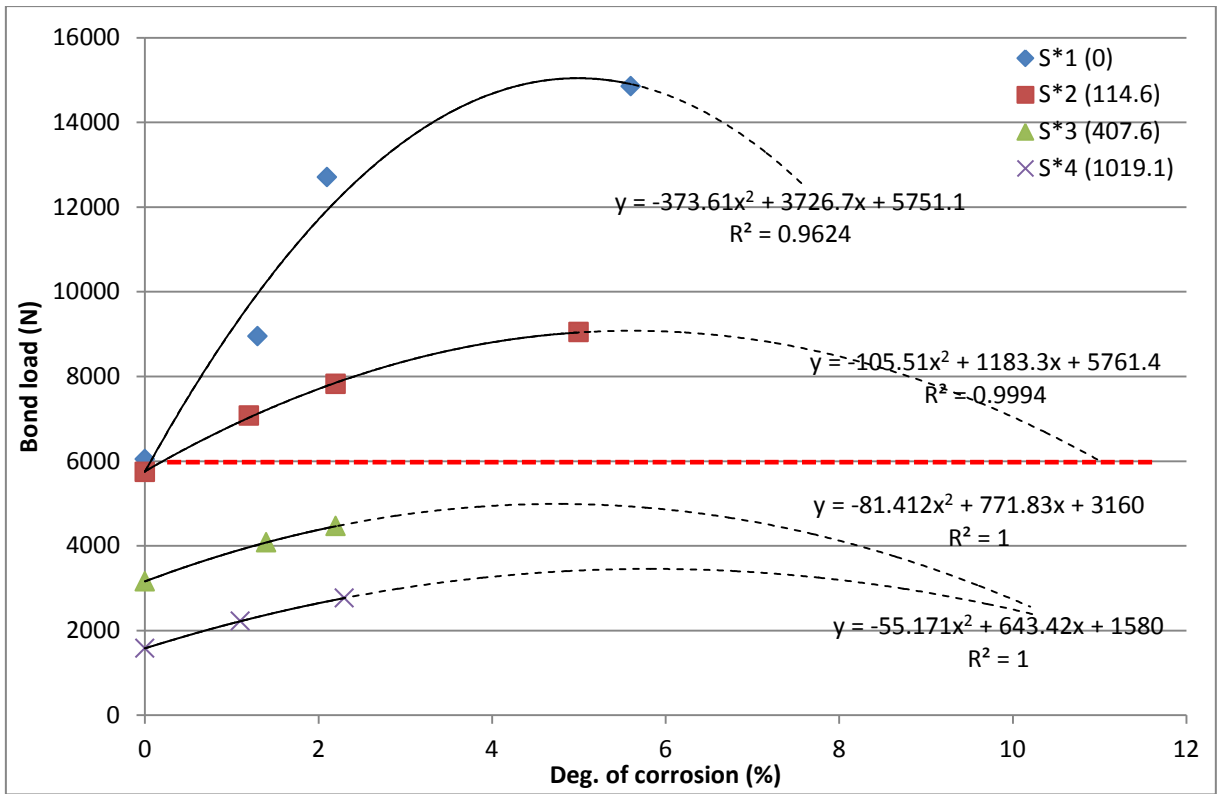


**Fig.4.** Pull-out test arrangement

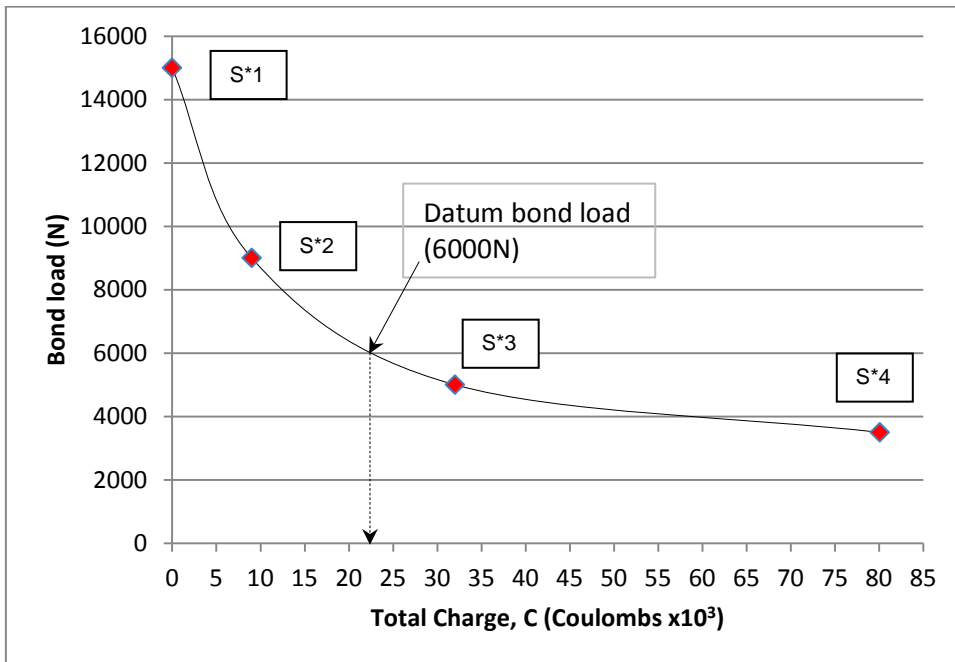


**Fig.5.** On- potentials of steel bars during ICCP operation (vs. Ag/AgCl/0.5M KCl)

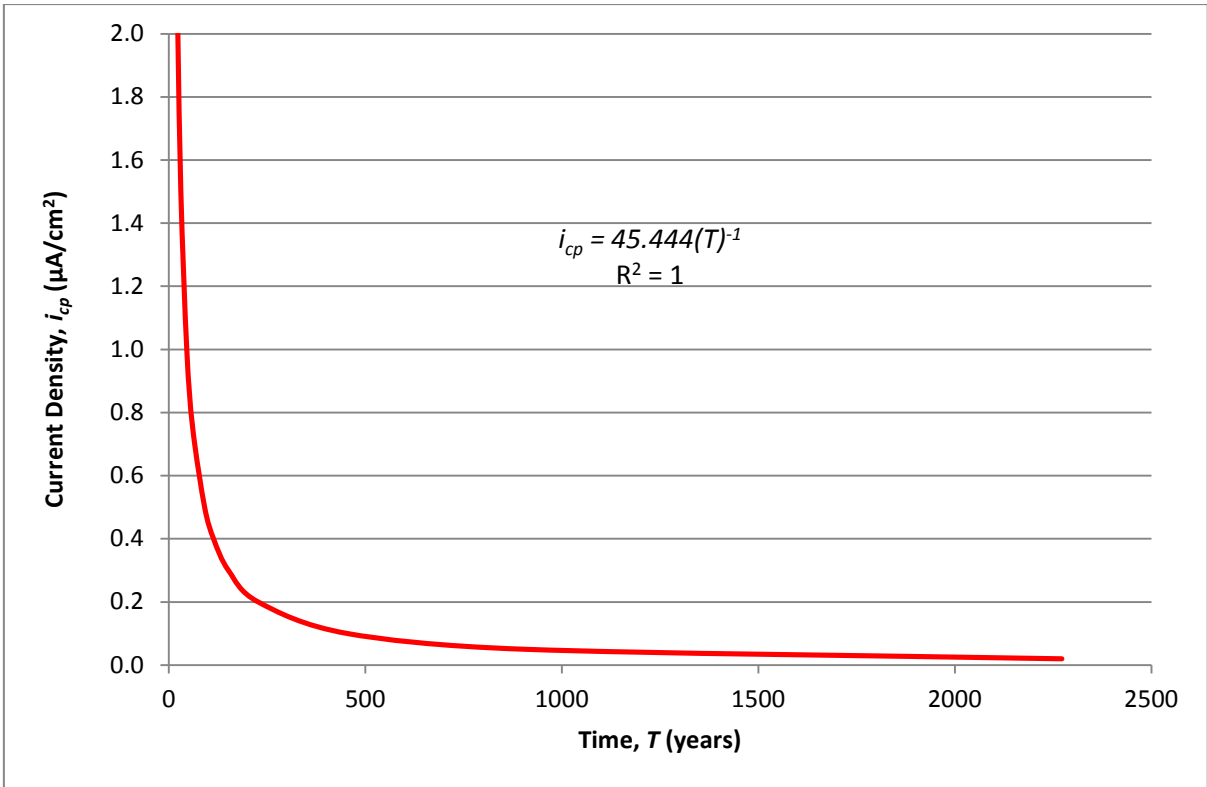




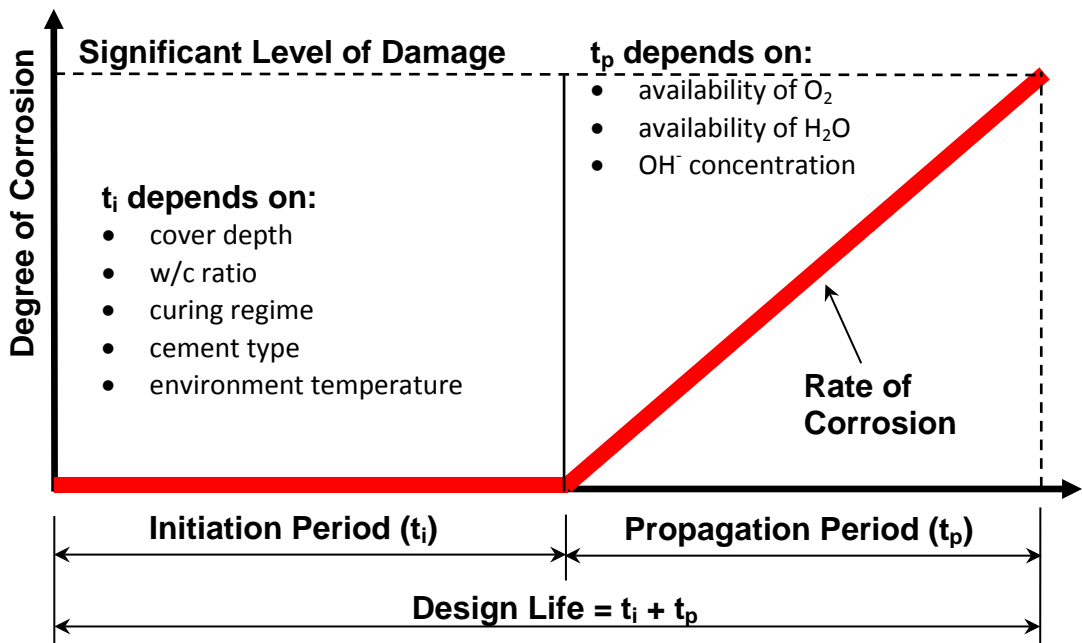
**Fig.6.** Bond load versus degree of corrosion



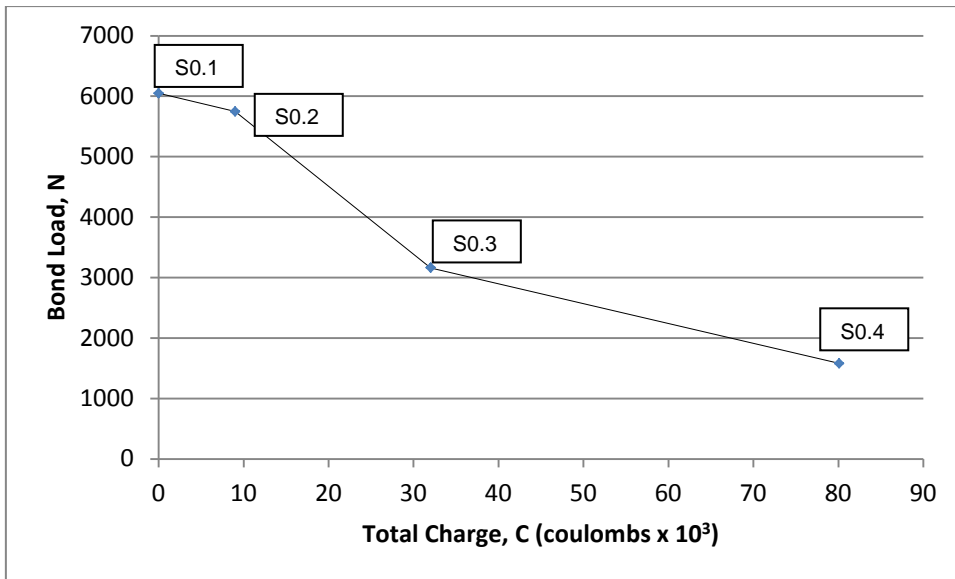
**Fig. 7** Bond load versus total charge at  $m = 5\%$



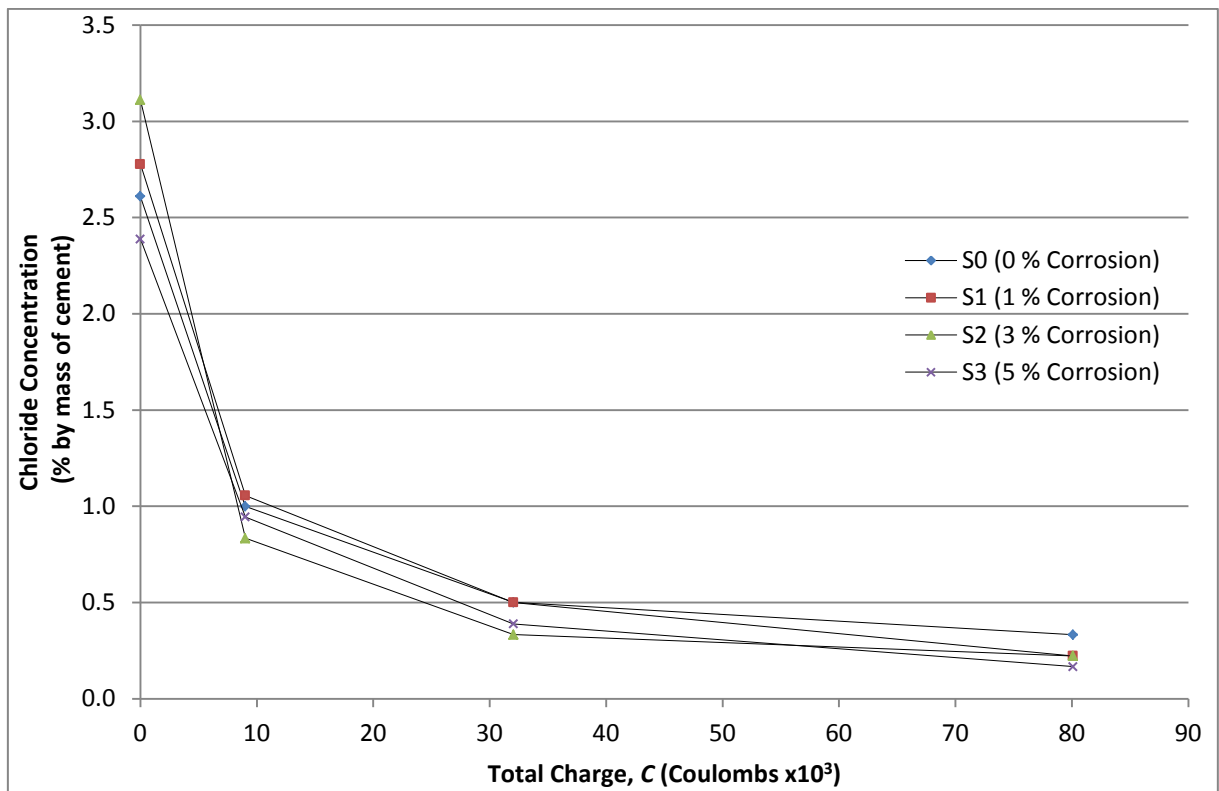
**Fig. 8** In-service current density versus time relationship at  $C = 22.5 \times 10^3$  coulombs



**Fig. 9** Service life model of corroded structures [28].



**Fig. 10** Bond load versus total charge for ICCP samples with 0 % corrosion



**Fig.11.** Relationships between chloride concentrations, total charge and degrees of corrosion

**Table 1** Details of test programme

Series	ID Sample	Accelerated Corrosion				Current Density ( $\mu\text{A}/\text{cm}^2$ )	Group Samples	ICCP	Comments
		Target Degree (%)	Current Density ( $\text{mA}/\text{cm}^2$ )	Duration (minutes)	Duration (hours)				
S0	S0.1	0	0	-	0	S*1	0	Control	
	S0.2	0	0	-	114.6	S*2	1390.5	ICCP	
	S0.3	0	0	-	407.6	S*3	1390.5	ICCP	
	S0.4	0	0	-	1019.1	S*4	1390.5	ICCP	
S1	S1.1	1	1	2250	0	S*1	0	Control	
	S1.2	1	1	2250	114.6	S*2	1390.5	ICCP	
	S1.3	1	1	2250	407.6	S*3	1390.5	ICCP	
	S1.4	1	1	2250	1019.1	S*4	1390.5	ICCP	
S2	S2.1	2	1	4500	0	S*1	0	Control	
	S2.2	2	1	4500	114.6	S*2	1390.5	ICCP	
	S2.3	2	1	4500	407.6	S*3	1390.5	ICCP	
	S2.4	2	1	4500	1019.1	S*4	1390.5	ICCP	
S3	S3.1	5	1	11250	0	S*1	0	Control	
	S3.2	5	1	11250	114.6	S*2	1390.5	ICCP	
	S3.3	5	1	11250	407.6	S*3	1390.5	ICCP	
	S3.4	5	1	11250	1019.1	S*4	1390.5	ICCP	

**Table 2** Measured weight loss of steel bars

Series	ID	Target Degree of Corrosion (%)	Weight of Steel		Weight Loss (g)	Actual Degree of Corrosion %
			Before Corrosion (g)	After Corrosion (g)		
S0	S0.1	0	-	-	-	0
	S0.2	0	-	-	-	0
	S0.3	0	-	-	-	0
	S0.4	0	-	-	-	0
S1	S1.1	1.0	138.91	138.10	0.81	1.3
	S1.2	1.0	139.87	139.13	0.74	1.2
	S1.3	1.0	138.90	138.02	0.88	1.4
	S1.4	1.0	138.76	138.06	0.70	1.1
S2	S1.2	2.0	139.81	138.18	1.33	2.1
	S2.2	2.0	138.76	137.28	1.38	2.2
	S2.3	2.0	139.25	137.90	1.35	2.2
	S2.4	2.0	139.49	138.30	1.39	2.3
S3	S1.3	5.0	139.02	135.52	3.50	5.6
	S3.2	5.0	139.92	136.85	3.07	5.0
	S3.3	5.0	139.70	136.61	3.09	5.0
	S3.4	5.0	138.63	134.53	3.30	5.4

**Table 3** Bond loads and chloride concentration of test samples

Series	Bond Test ID	Measured Degree of Corrosion (%)	Current Density ( $\mu\text{A}/\text{cm}^2$ )	Ultimate Load (N)	Reduction in Bond Load (%)	Chloride Concentration (% by mass of dry sample)	Chloride Concentration (% by mass of cement)
S0	S0.1	0	0	6049		0.47	2.6
	S0.2	0	114.6	5746	5	0.18	1.0
	S0.3	0	407.6	3160	48	0.09	0.5
	S0.4	0	1019.1	1580	74	0.06	0.3
S1	S1.1	1.3	0	8946		0.50	2.8
	S1.2	1.2	114.6	7074	21	0.19	1.1
	S1.3	1.4	407.6	4081	54	0.09	0.5
	S1.4	1.1	1019.1	2221	75	0.04	0.2
S2	S2.1	2.1	0	12704		0.56	3.1
	S2.2	2.2	114.6	7821	38	0.15	0.8
	S2.3	2.2	407.6	4464	65	0.06	0.3
	S2.4	2.3	1019.1	2768	78	0.04	0.2
S3	S3.1	5.6	0	14850		0.43	2.4
	S3.2	5.0	114.6	9044	39	0.17	0.9
	S3.3	5.0	407.6	7002	53	0.07	0.4
	S3.4	5.4	1019.1	4570	69	0.03	0.2

**Table 4** Total charge applied and bond characteristics

1 Group	2 Current Density ( $\mu\text{A}/\text{cm}^2$ )	3 Surface area ( $\text{cm}^2$ )	4 Current (Amps)	5 Time (secs)	6 Charge (Coulombs)	7 Bond load at 5 %, Fig. 6 (N)
S*1	0	15.7	0.00000	5005800	0	15000
S*2	114.6	15.7	0.00180	5005800	9007	9000
S*3	407.6	15.7	0.00640	5005800	32034	5000
S*4	1019.1	15.7	0.01600	5005800	80092	3500