ELSEVIER

Cement & Concrete Composites 25 (2003) 459–471

Cement & Concrete Composites

www.elsevier.com/locate/cemconcomp

Reinforcement corrosion in concrete structures, its monitoring and service life prediction—a review

Shamsad Ahmad *

Department of Civil Engineering, King Fahd University of Petroleum and Minerals, 31261 Dhahran, Saudi Arabia

Abstract

Reinforcement corrosion has been widely reported in the literature over the last two to three decades. It is one of the major durability problems, mainly when the rebar in the concrete is exposed to the chlorides either contributed from the concrete ingredients or penetrated from the surrounding chloride-bearing environment. Carbonation of concrete or penetration of acidic gases into the concrete, are the other causes of reinforcement corrosion. Besides these there are few more factors, some related to the concrete quality, such as w/c ratio, cement content, impurities in the concrete ingredients, presence of surface cracks, etc. and others related to the external environment, such as moisture, oxygen, humidity, temperature, bacterial attack, stray currents, etc., which affect reinforcement corrosion. The assessment of the causes and extent of corrosion is carried out using various electrochemical techniques. Prediction of the remaining service life of a corroding RC structure is done with the help of empirical models and experimental methods. In this paper a review is presented on the mechanism of reinforcement corrosion, techniques utilized to monitor reinforcement corrosion and methodologies that are utilized for the prediction of remaining service life of structures.

Keywords: Mechanisms; Monitoring; Service life prediction; Reinforcement corrosion

1. Introduction

Concrete normally provides a high degree of protection to the reinforcing steel against corrosion, by virtue of the high alkalinity ($pH > 13.5$) of the pore solution. Under high alkalinity steel remains passivated. In addition, well-consolidated and properly cured concrete with a low w/c ratio has a low permeability, which minimizes penetration of corrosion inducing agents, such as chloride, carbon dioxide, moisture, etc. to the steel surface. Further, the high electrical resistivity of concrete restricts the rate of corrosion by reducing the flow of electrical current from the anodic to the cathodic sites. At the outset, it must be mentioned that, usually in a properly designed, constructed and maintained structure, there should be little problem of steel corrosion during its design life. Unfortunately, this highly desirable durability requirement is not always achieved in practice due to which corrosion of reinforcement in concrete has become a commonly encountered cause of deterioration in many RC structures in the recent years.

In general, there are two major factors, which cause corrosion of reinforcement in concrete to proceed to an unacceptable degree. They are: (i) carbonation, and (ii) presence of chloride ions, which may either have been present in the concrete constituents right from the beginning or are introduced into the concrete through ingress during the service life.

The quality of concrete, mainly the permeability, nature and intensity of cracks, and the cover thickness, have also a great bearing upon the initiation and sustenance of reinforcement corrosion.

Once reinforcement corrosion is initiated, it progresses almost at a steady rate and shortens the service life of the structure, by causing surface cracking and subsequently spalling of the cover concrete due to expansion of the corroding steel. The rate of corrosion directly affects the extent of the remaining service life of a corroding RC structure.

A wide range of techniques have been reported in the literature that may be suitably employed for the monitoring of corrosion of steel in concrete structures for the purpose of diagnosing the cause and extent of the reinforcement corrosion.

---

* Fax: +966-3-860-2879.
E-mail address: shamsad@kfupm.edu.sa (S. Ahmad).

0958-9465/03/$ - see front matter © 2002 Elsevier Science Ltd. All rights reserved.
doi:10.1016/S0958-9465(02)00086-0

The prediction of the remaining service life of a corroding structure may be carried out with the help of various available prediction models or experimental techniques, using the data obtained through monitoring.

In this paper a review of the following aspects of reinforcement corrosion are presented:

- mechanism of reinforcement corrosion,
- factors affecting reinforcement corrosion,
- initiation and progress of reinforcement corrosion,
- effects of reinforcement corrosion on performance of the structure,
- monitoring techniques,
- prediction of the remaining service life.

2. Mechanism of corrosion of steel in concrete

2.1. Corrosion cell

Corrosion of steel embedded in concrete is an electrochemical process. The corrosion process is stated to be similar to the action, which takes place in a flash battery [1]. The surface of the corroding steel functions as a mixed electrode that is a composite of anodes and cathodes electrically connected through the body of steel itself, upon which coupled anodic and cathodic reactions take place. Concrete pore water functions as an aqueous medium, i.e., a complex electrolyte. Therefore, a reinforcement corrosion cell is formed, as shown in Fig. 1 [2].

2.1.1. Anodic and cathodic reactions

Reactions at the anodes and cathodes are broadly referred to as ‘half-cell reactions’. The ‘anodic reaction’ is the oxidation process, which results in dissolution or loss of metal whilst the ‘cathodic reaction’ is the reduction process which results in reduction of dissolved oxygen forming hydroxyl ions.

For steel embedded in concrete, following are the possible anodic reactions depending on the pH of interstitial electrolyte, presence of aggressive anions, and the existence of an appropriate electrochemical potential at the steel surface:

$$3Fe + 4H_2O \rightarrow Fe_3O_4 + 8H^+ + 8e^- \tag{1}$$

$$2Fe + 3H_2O \rightarrow Fe_2O_3 + 6H^+ + 6e^- \tag{2}$$

$$Fe + 2H_2O \rightarrow HFeO_2^- + 3H^+ + 2e^- \tag{3}$$

$$Fe \rightarrow Fe^{2+} + 2e^- \tag{4}$$

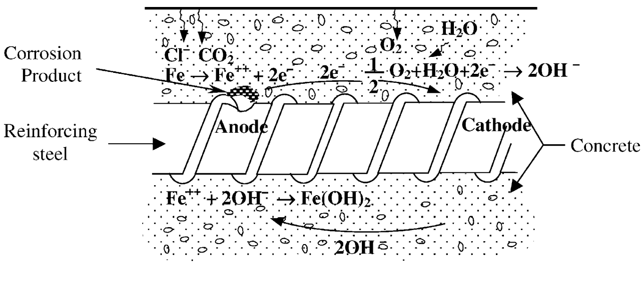


Fig. 1. Schematic illustration of the corrosion of reinforcement steel in concrete—as an electrochemical process.

The possible cathodic reactions depend on the availability of O_2 and on the pH in the vicinity of the steel surface. The most likely reactions are as follows:

$$2H_2O + O_2 + 4e^- \rightarrow 4OH^- \tag{5}$$

or

$$2H^+ + 2e^- \rightarrow H_2 \tag{6}$$

2.1.2. Anodic and cathodic potentials and emf of the corrosion cell

The electrode potentials, for dissolution (oxidation) reaction (Eq. (4)) at anode, E_A, and the reduction reaction (Eq. (5)) at cathode, E_C, can be derived as given below [3].

Referring to anodic reaction (Eq. (4)) the anodic electrode potential, E_A, can be expressed by the Nernst equation as:

$$E_A = E_A^0 + \frac{R_cT}{nF} \ln \frac{[Fe^{2+}]}{[Fe]} \tag{7}$$

where: E_A^0 is standard electrode potential of steel at anode, R_c is gas constant, F is Faraday’s constant (i.e., 96,500 Coulomb per mole of electrons or Ampere-second), T is absolute temperature, n is number of electrons taking part in the reaction.

Substituting, $E_A^0 = -0.44$ V, $R_c = 8.314$ J/°K, $F = 96{,}487$ C, $T = 25$ °C, $n = 2$, and $[Fe] = 1$ (the activity of an Fe molecule in the steel bar), with proper conversion of units, the above equation (7) can be simplified in terms of $[Fe^{2+}]$ as:

$$E_A = -0.44 + 0.0296 \log[Fe^{2+}] \text{ (in V)} \tag{8}$$

where $[Fe^{2+}]$ is the concentration of Fe^{2+} in moles per liter of electrolyte, which is related to the mass concentration of $Fe(OH)_2$ per unit volume of concrete and the amount of capillary water per unit volume of concrete.

Similarly, referring to the cathodic reaction (Eq. (5)) the cathodic electrode potential, E_C, can be expressed by the Nernst equation as:

$$E_C = E_C^0 + \frac{R_cT}{nF} \ln \frac{[O_2][H_2O]^2}{[OH^-]^4} \tag{9}$$

Substituting, $E_C^0 = -0.401$ V, $R_c = 8.314$ J/°K, $F = 96{,}487$ C, $T = 25$ °C, $n = 4$, and $\log[OH^-] = pH - 14$, Eq. (9) can be simplified in terms of $[O_2]$ and pH as:

$$E_C = 1.229 + 0.0148 \log[O_2] - 0.0591 \text{ pH (in V)} \tag{10}$$

where $[O_2]$ is the concentration of oxygen, expressed in terms of molarity, and the amount of capillary water per unit volume of concrete.

The electromotive force, emf (e), of the reinforcement corrosion cell involving a couple of anodic and cathodic reactions, as expressed by Eqs. (4) and (5), can be derived as a difference of E_C and E_A as:

$$\begin{aligned} e &= E_C - E_A \\ &= 1.229 + 0.0148 \log[O_2] - 0.0591 \text{ pH} \\ &\quad + 0.44 - 0.0296 \log[Fe^{2+}] \end{aligned}$$

or

$$\begin{aligned} e = {} & 1.669 + 0.0148 \log[O_2] - 0.0591 \text{ pH} + 0.44 \\ & - 0.0296 \log[Fe^{2+}] \text{ (in V)} \end{aligned} \tag{11}$$

The emf (e) of reinforcement corrosion cell, as given by Eq. (11), drives the corrosion current through the electrolyte from anode to cathode. It is evident from the Eq. (11) that the corrosion current, i.e., corrosion rate, is affected in general, by the following factors:

(i) the pH of electrolyte in concrete which is affected mainly by the carbonation,
(ii) the availability of oxygen and capillary water, and
(iii) the concentration of Fe^{2+} in concrete near the reinforcement.

3. Factors affecting corrosion of steel in concrete

The factors affecting corrosion of steel in concrete may be classified into two major categories, as follows:

(a) external factors,
(b) internal factors.

3.1. Internal factors affecting corrosion of steel in concrete

They include mostly the environmental parameters, as follows:

- *Availability of oxygen and moisture at rebar level:* Presence of moisture and oxygen supports the corrosion. Moisture fulfills the electrolytic requirement of the corrosion cell, and moisture and oxygen together help in the formation of more OH^- thereby producing more rust component, i.e., $Fe(OH)_2$. Oxygen also affects the progress of cathodic reactions. In the absence of enough oxygen, even in a situation of depassivation, corrosion will not progress due to cathodic polarization.
- *Relative humidity and temperature:* The relative humidity mainly affects the carbonation of concrete. Within 50–100% RH, the increase of environmental relative humidity decreases carbonation of concrete. Based on their calculation, Cahyadi and Uomoto [4] have found that, within 50–30% RH, a decrease in environmental RH may not cause a decrease in carbonation of concrete especially in normal concentration of CO_2 even after a long period of exposure.

A rise in temperature may result in a two fold effect: (i) the electrode reaction rates are generally increased, and (ii) the oxygen solubility is decreased resulting in a reduction in the rate of corrosion [5].

If the situation is conducive for corrosion to take place, the corrosion rate is increased by high temperature and high humidity [6].

- *Carbonation and entry of acidic gaseous pollutants to rebar level:* The effect of carbonation and other acidic gases, such as SO_2 and NO_2, is due to their tendency to reduce the pH of the concrete. The fall in the pH to certain levels may cause commencement of reinforcement corrosion, loss of passivity of concrete against reinforcement corrosion, and catastrophic reinforcement corrosion [7], as indicated in Table 1.
- *Aggressive anions, mostly chloride ions, reaching to the rebar level, either through the concrete ingredients or from the external environment:* Chloride in concrete may be present in either of the following forms:
  (i) 'Acid soluble chloride' which is equal to the total amount of chloride present in the concrete or that is soluble in nitric acid [8],
  (ii) 'Bound chloride' which is the sum of chemically bound chloride with hydration products of the cement, such as the C_3A (tricalcium aluminate) or C_4AF (tetracalcium aluminoferrite) phases, and loosely bound chloride with C–S–H gel, and
  (iii) 'Free or water-soluble chloride' which is the concentration of free chloride ions (Cl^-) within the pore solution of concrete, and is extractable in water under defined conditions.

It is generally recognized that only the "free chloride" ions influence the corrosion process [9]. It is reported [10] that the resistivity decreases and corrosion rate increases with an increase in the chloride content. However, the change in pH is found to be insignificant due to a change in the chloride content of concrete [10].

The risk of reinforcement corrosion associated with the levels of chloride content in both uncarbonated and carbonated concrete is presented in Table 2 [11].

Table 1
State of reinforcement corrosion at various pH levels [7]

pH of concrete	State of reinforcement corrosion
Below 9.5	Commencement of steel corrosion
At 8.0	Passive film on the steel surface disappears
Below 7	Catastrophic corrosion occurs

Table 2
Corrosion risk in concrete containing chlorides [11]

Total chloride (wt.% of cement)	Condition of concrete adjacent to reinforcement	Corrosion risk
Less than 0.4%	Carbonated	High
	Uncarbonated, made with cement containing less than 8% C_3A	Moderate
	Uncarbonated, made with cement containing 8% or more C_3A	Low
0.4%–1.0%	As above	High
	As above	High
	As above	Moderate
More than 1.0%	All cases	High

Table 3
Chloride content limits, as recommended by some codes of practices

Country	Recommended limits of chloride content (% by mass of cement)
USA	0.15% for chloride exposures and 0.30% for chloride free exposure
UK	0.30%
India	0.15%

The limits imposed on the total chloride ion content (by weight of cement) of the concrete, by codes of practice of some of the countries, are presented in Table 3.

- *Stray currents:* Stray currents from the various sources, e.g., building power supply systems, cathodic protection systems, locomotive power supply systems, etc. cause the electrolytic corrosion [12].
- *Bacterial action:* Bacterial action is found to be effective in three ways:
    - (i) The bacteria decrease the amount of cover by disintegration of the cementitious materials [13].
    - (ii) The anaerobic bacteria produce iron sulfides in the oxygen deficit condition, such as concrete sewers, which enables the corrosion reaction to proceed even in the absence of oxygen [7], and
    - (iii) aerobic bacteria may also aid in the formation of differential aeration cells, which can lead to corrosion.

3.2. Internal factors affecting reinforcement corrosion

They include concrete and steel quality parameters, as discussed below:

- *Cement composition:* The cement in the concrete provides protection to the reinforcing steel against corrosion in the following ways:
    - (i) By maintaining a high pH in the order of 12.5–13 owing to the presence of $Ca(OH)_2$ and other alkaline materials in the hydration product of cement, and
    - (ii) by binding a significant amount of total chlorides as a result of chemical reaction between C_3A and C_4AF content of cement in concrete. Thus the threshold chloride value shifts to higher side with an increase in the C_3A content of cement [14].

      The use of blended cement, such as microsilica-blended high-C_3A cement, is found to be concomitantly resistant to sulfate attack and chloride corrosion of reinforcement [15].
- *Impurities in aggregates:* Aggregates containing chloride salts cause serious corrosion problems, particularly those associated with sea-water and those whose natural sites are in ground water containing high concentration of chloride ions [16].
- *Impurities in mixing and curing water:* Mixing and curing water, either contaminated with sufficient quantity of chloride or being highly acidified due to any undesirable substance present in water, may prove to be detrimental as for as corrosion of rebar is concerned.
- *Admixtures:* Addition of calcium chloride in concrete, as a common admixture for accelerating the hydration of cement is perhaps a most significant reason for the presence of chloride in concrete in the RC structures exposed to normal weather conditions. Some water reducing admixtures also contain chlorides [16].
- *w/c ratio:* The w/c ratio, known to control principally strength, durability and impermeability of concrete, does not itself control the rate of corrosion of reinforcement. When RC structures are immersed in some aggressive solution, it is the 'permeability' of concrete, which is a function of w/c ratio, affects the corrosion of rebar. The depth of penetration of a particular chloride threshold value increases with an increase in the w/c ratio [17]. Carbonation depth has been found to be linearly increasing with an increase in the w/c ratio [18]. The oxygen diffusion coefficient is also found to be increasing with an increase in the w/c ratio [19].

Goto and Roy [20] have found a 100-fold increase in the permeability of hardened cement paste when the w/c ratio was increased from 0.35 to 0.45. Al-Saadoun [21] has observed that the time to initiation of reinforcement corrosion in a sample with w/c ratio of 0.4 is 2.15 to 1.77 times more than that in a sample with a w/c ratio of 0.55, under the accelerated corrosion testing. Al-Amoudi [22] has reported that permeability is significantly reduced for a w/c ratio below 0.45. From these observations it is to be noted that the w/c ratio should be less than 0.45 and preferably around 0.40 to obtain good durability of concrete. Suitable dosage of admixtures may be added to obtain the desired workability at low w/c ratio.

Low w/c ratio increases the concrete impermeability, which in turn reduces the chloride penetration, carbonation penetration, and oxygen diffusion in concrete. However, a low w/c ratio does not by itself assure concrete of low permeability. For example, 'no fines' concrete can have a low w/c ratio and yet be highly permeable. Thus, in addition to the low w/c ratio, the concrete must be properly proportioned and well consolidated to produce a concrete of low permeability.

- *Cement content:* The cement content in concrete does not only affect the strength but it also has a significant effect on durability. Due to inadequate amount of cement in mix the concrete is not consolidated properly leading to the formation of honeycombs and other surface defects. These honeycombs and surface defects help in the penetration and diffusion of corrosion causing agents, such as Cl^-, H_2O, O_2, CO_2, etc., in concrete. This results in the initiation of reinforcement corrosion due to the formation of differential cells. Further, concrete with low cement content has a lack of plastic consistency due to which it does not form a uniform passive layer on the surface of the steel bars.

Therefore, it is important to maintain minimum cement content from the durability point of view. Verbeck [23] has recommended minimum cement content of 350–375 Kg/m^3 for the concrete structures exposed to direct marine exposure.

- *Aggregate size and grading:* Since the size of aggregates has a bearing upon the consistency of concrete, it may have an effect upon reinforcement corrosion. Aggregate grading is another factor, which should be considered for high quality impermeable concrete. It has been observed that for a given w/c ratio, the coefficient of permeability of concrete increases considerably with increasing size of aggregates [24]. Keeping this in view, Cordon and Gillespie [25] have recommended maximum size of aggregate as: $1\frac{1}{2}$ in. for 5000 psi concrete, 3/4 in. for 6000 psi concrete and 3/8 to 1/2 in. for concrete with a compressive strength of more than 6000 psi.

The proportioning of coarse and fine aggregates is important for the production of a workable and durable concrete. The aggregate proportioning for this purpose consists of fixing the optimum volume fraction of sand in the total aggregate content. Studies conducted by Maslehuddin [26] reveal that both 'too much' and 'too less' sand reduces the workability of concrete. The optimum proportioning for a maximum workability has been reported to be corresponding to a specific surface area of the combined aggregates in the range of 70–75 cm^2/cm^3 for concrete with a cement content in the range of 300–390 kg/m^3 [26]. This enhanced workability on account of an optimum aggregate grading allows a reduction in the w/c ratio resulting in increased strength and durability of concrete.

- *Construction practices:* Serious corrosion problems may occur if enough care, such as listed below, is not taken at the construction stage:
  (i) aggregate washing for deleterious materials [27], if any;
  (ii) control of chloride in almost all ingredients of concrete, i.e., water, cement, aggregate, and admixtures;
  (iii) strict enforcement of designed and recommended levels of w/c ratio, cement content, cover thickness, etc.;
  (iv) proper consolidation of freshly placed concrete [27]; and
  (v) proper curing of concrete [27].
- *Cover over reinforcing steel:* Cover depth has a significant effect in case of corrosion due to penetration of either chloride or carbonation [28]. This effect of cover is limited within the time of casting to the time at which the rebar is depassivated and corrosion is initiated. The rate of corrosion, once it has started, is independent of the cover thickness [29].

Risk of reinforcement corrosion with low cover thicknesses, has been reported by various researchers [30–32]. Rasheeduzzafar et al. [33], based on their field and laboratory studies, have recommended the safe cover thicknesses for RC structures exposed to the various aggressive environments of the Arabian Gulf. Codes of practices on design of RC structures also specify cover thicknesses for various types of exposures.

Further, cover thickness is one of the factors, which affects the cracking and spalling of the concrete due to the reinforcement corrosion.

- *Chemical composition and structure of the reinforcing steel:* The differences in the chemical composition and structure of reinforcing steel and presence of stress in the reinforcement, either static or cyclic, create different potentials at different locations on the surface of reinforcement, causing the formation of differential corrosion cells, which leads to its corrosion [5].

4. Time-dependent states of reinforcement corrosion

The state of corrosion of steel in concrete may be expected to change as a function of time. Corrosion process has three distinct stages, namely; depassivation, propagation, and final state [34], as shown in Fig. 2.

Depassivation is the loss of oxide (passive) layer over the rebar, which is initially formed due to the high alkalinity of concrete. The process of depassivation takes an initiation period, t_p, which is the time from construction to the time of initiation of corrosion (depassivation).

The propagation phase starts from the time of depassivation, t_p, to the final state, is reached at a critical

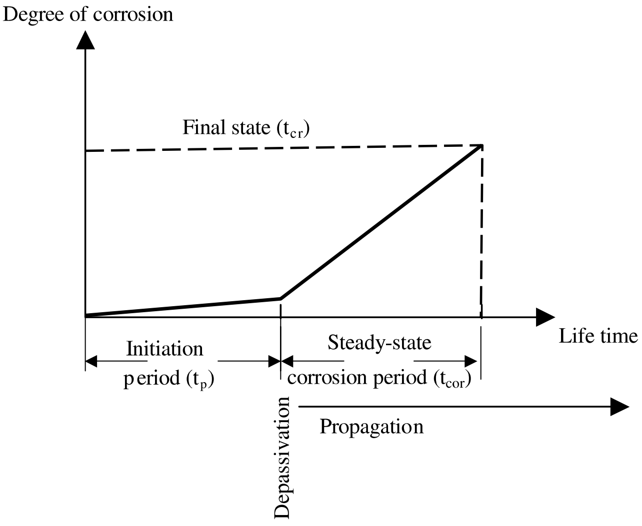


Fig. 2. The stages of rebar corrosion.

time, t_{cr}, at which corrosion would produce spalling of concrete cover or cracking through the whole of concrete cover. During the propagation period, i.e. corrosion period, t_{cor}, which begins at the moment of depassivation, the rebar corrosion is usually assumed to be in a steady state, as indicated by a straight line in Fig. 2.

The critical time, t_{cr}, as defined above can be expressed as:

$$t_{cr} = t_p + t_{cor} \tag{12}$$

For reinforced concrete, it has been assumed reasonable to equate the unacceptable corrosion damage to the onset of spalling of concrete cover [28]. Therefore, the *service life* can be equated to the critical time, as given by Eq. (12). The depassivation time, t_p, can be assumed to be zero when the quantity of free chloride ions, introduced in concrete at the time of construction itself by any means, is found to be more than the rebar corrosion threshold value.

5. Effect of reinforcement corrosion on structural behavior

The corrosion of reinforcing steel has the following two major detrimental effects on the durability of RC structures:

(i) Since the rust produced as a result of corrosion has a volume 2–4 times than that of steel [3], it causes volume expansion developing tensile stresses in concrete, which ultimately results in cracking and spalling of the cover concrete. Due to the loss of cover concrete there may be significant reduction in the load bearing capacity of the structure, and besides this, steel may be more accessible to the aggressive agents leading towards further corrosion at an accelerated rate, and

(ii) corrosion reduces the cross-sections of the steel and thereby the load carrying capacity of the structure. Pitting (i.e., localized) corrosion of the rebar is more dangerous than uniform corrosion because it progressively reduces the cross-sectional area of rebar to a point where the rebar can no longer withstand the applied load leading to a catastrophic failure of the structure.

6. Strategies for investigation of a corroding RC structure

A visual survey of a corroding structure provides valuable information as to, whether the corrosion of rebar is really a cause of distress or there is some other cause of distress. This survey consists of a careful investigation of the structure for any sign of distress, such as cracking, spalling, and rust staining.

If visual inspection of the structure suggests that the cause of distress is the corrosion of rebar only, the next step is to make a careful examination of the structure and carry out detailed tests which will positively identify the cause and extent of the distress, and allow prediction to be made about the remaining service life of the structure.

A widely accepted flowchart for investigation of a corroding RC structure, suggested by Pullar-Strecker [11], is reproduced in Fig. 3 with some improvements.

7. Reinforcement corrosion monitoring techniques

7.1. Corrosion measurement parameters

The information about the state of reinforcement corrosion is obtained usually in terms of three measurement parameters, namely (i) half-cell potential, E_{corr}, (ii) concrete resistivity, ρ, and, (iii) corrosion current density, I_{corr}. These parameters are summarized below:

(i) *Half-cell potential* (E_{corr}): The half-cell potential, also called the open-circuit potential or rest potential or corrosion potential (E_{corr}), is measured at several distinct points over a given area to be surveyed, and is used as a *qualitative index* for ascertaining whether or not reinforcement is likely to be corroding, and also for obtaining a contour plot to delineate anodic (corroding) and cathodic (non-corroding or passive) portions of the RC structure under investigation.

Interpretation of half-cell potential test results is carried out as per the ASTM C876 [36] guidelines, as presented in Table 4.

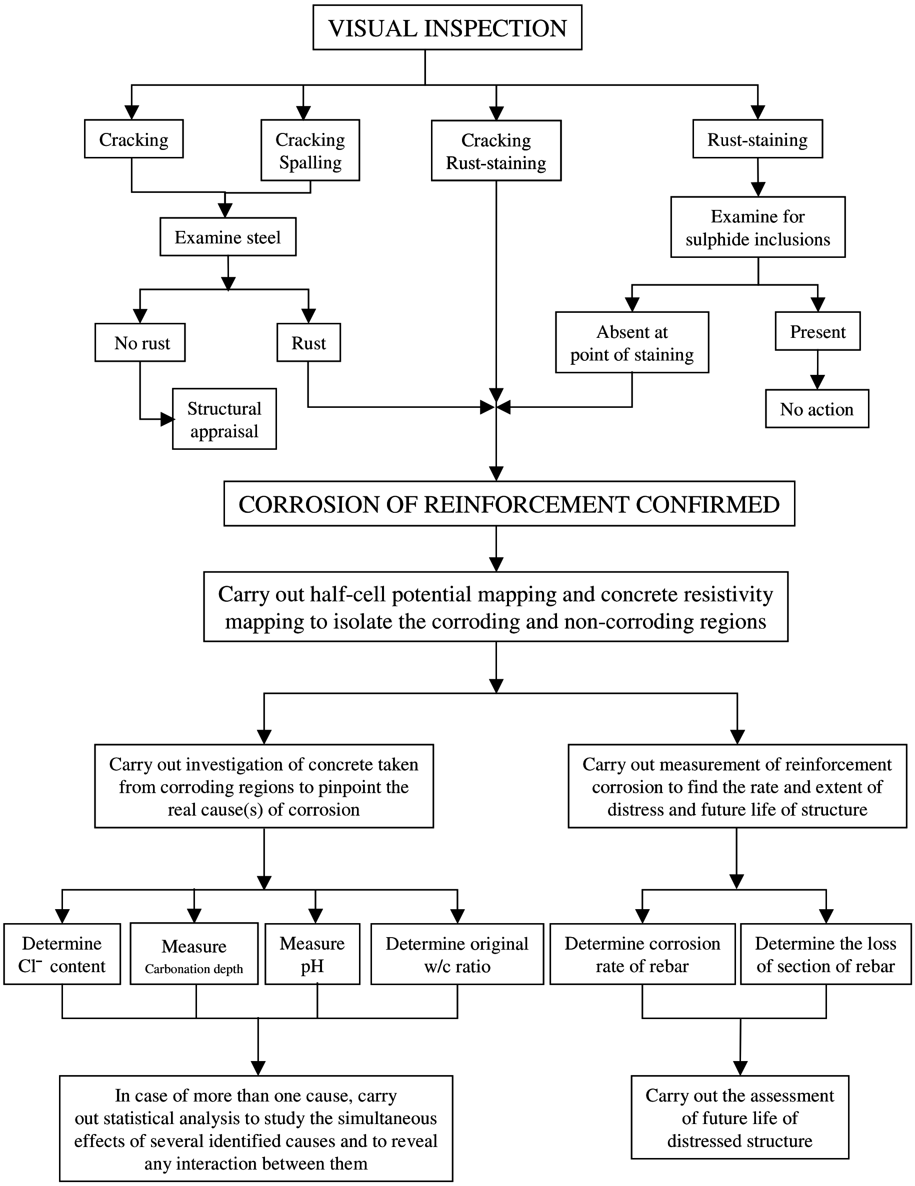


Fig. 3. Flowchart of investigation strategies of a corroding RC structure.

(ii) *Concrete resistivity* (ρ): The corrosion of a specific length of reinforcement is dependent on the algebraic summation of the electrical currents originating from the corroding sites on the steel and flowing through the surrounding concrete to the non-corroding sites. Hence, the electrical resistance of the concrete plays an important role in determining the magnitude of corrosion at any one specific location [7]. This factor is measured in terms of electrolytic resistivity of concrete and is usually expressed in ohm-centimeters.

Classification of the likelihood of corrosion, actually occurring, can be obtained on the basis of the values given in Table 5, and this is best suited when half-cell potential measurements indicates that corrosion is possible [37].

(iii) *Corrosion current density* (I_{corr}): The corrosion rate is measured in terms of the corrosion current density, I_{corr}, and is *a quantitative index*, which represents an overall estimate of the corrosion attack of reinforcement. The I_{corr} is measured electrochemically. The electrochemically measured value of I_{corr} can be converted [38] to the instantaneous

Table 4
Interpretation of half-cell potential values as per ASTM C876 [36]

Half-cell potential (mV) relative to $Cu/CuSO_4$ reference electrode	Percentage chance of active corrosion
<-350	90
-200 to -350	50
>-200	10

Table 5
Interpretation of concrete resistivity with regard to reinforcement corrosion [37]

Resistivity (ohm cm)	Likelyhood of significant corrosion (non-saturated concrete when steel is activated)
<5000	Very high
5000–10,000	High
10,000–20,000	Low/moderate
>20,000	Low

corrosion rate, J_r, and penetration rate, P_r, through Faraday's law, as follows:

$$J_r = \left(\frac{W}{F}\right) I_{corr} \tag{13}$$

$$P_r = \left(\frac{W}{F\rho_{st}}\right) I_{corr} \tag{14}$$

where:
W = equivalent weight of steel = 55.85/2 = 27.925 g, F = Faraday's constant = 96487 C (or Ampere-second), ρ_{st} = density of steel (7.85 gm/cm^3), I_{corr} = corrosion current density (Amp/cm^2), J_r = instantaneous corrosion rate (gm/cm^2/s), P_r = penetration rate (cm/s).

The corrosion rate measured may be either passive or active depending on the corrosion rate [39]. Passive state rate of corrosion of rebar is found to be relatively low in the order of 10^{-9} to 10^{-7} Amp/cm^2, whereas the active state corrosion rate is found to be relatively high, compared to that of passive state, in the order of 10^{-6} to 10^{-5} Amp/cm^2.

7.2. Methods of corrosion monitoring

Measurement of reinforcement corrosion in concrete structures needs a method which can determine simply, accurately, and non-destructively not only whether or not corrosion of reinforcement is taking place but also its intensity and the rate of damage [35]. A summary of the features of available monitoring techniques, as given by Rodriguez et al. [39], is presented in Table 6.

Of all the techniques used for monitoring reinforcement corrosion, half-cell potential mapping is the simplest. Although, it is the most widely used method, it does not provide any quantitative information on corrosion. Corrosion rate, I_{corr}, measurement provides the most useful quantitative parameter for studying corrosion in reinforced concrete. Among all the methods of I_{corr} measurement, as mentioned in Table 6, the fastest, simplest, non-destructive, relatively inexpensive and most widely used alternative method is the 'linear polarization method' [34,35,39]. However, application of this technique to the RC specimens and on-site corrosion survey poses a number of difficulties. Notable amongst such difficulties are the following:

(i) The high resistance of concrete between the rebar, i.e., working electrode, and the reference electrode, which offers a potential drop commonly referred to as Ohmic drop [40]. This must be either compensated externally or eliminated mathematically.
(ii) The sample being large compared to the counter electrode, the distribution of the electrical signal applied for polarization of the rebar is non-uniform throughout the cross-section of the beam [41] and this aspect must be duly considered.
(iii) The corrosion current density, I_{corr}, is mathematically related to the Tafel constants, which must be known accurately for error free estimation of I_{corr}.

Ahmad and Bhattacharjee [42] have suggested a simple arrangement, as shown in Fig. 4, and a calculation procedure, for in situ measurement of corrosion potential, E_{corr}, concrete resitivity, ρ, and corrosion rate, I_{corr}, of rebar embedded in concrete.

Using the observations taken from the above arrangement, the ohmic resistance and resistivity of concrete, the polarization resistance of rebar, Tafel slopes, and corrosion rate can be determined.

- *Determination of Ohmic resistance, R, and resistivity, ρ, of concrete:* The ohmic resistance, R, of concrete may be determined by plotting $1/R_L$ against $1/V$ data, generated by the arrangement, and using the following linear equation [42]:

$$\frac{1}{V} = \left(\frac{R}{E}\right)\frac{1}{R_L} + \frac{1}{E} \tag{15}$$

where: V is voltmeter reading taken by setting decade box resistor at a value of R_L and keeping key switch K_2 off (Fig. 4).

According to Eq. (15), slope and intercept of the straight line, obtained through the best fit of $1/R_L$ vs $1/V$ data, will be equal to R/E and $1/E$, respectively. Using this slope and intercept, R may be determined as:

$$(R) = \frac{\text{slope } (R/E)}{\text{intercept } (1/E)}$$

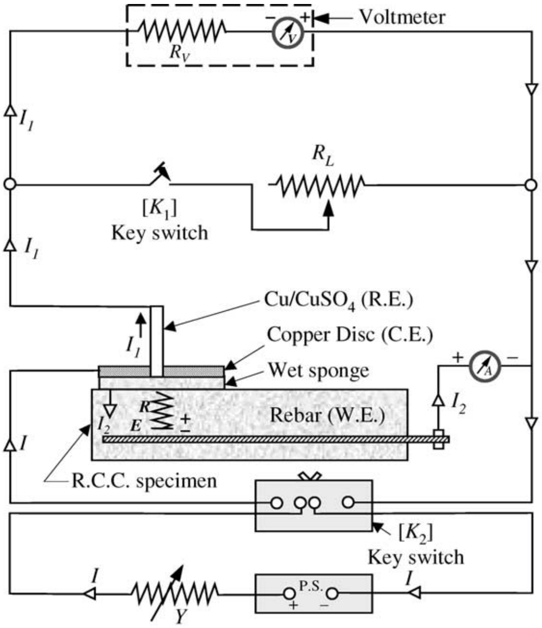
knowing R, the *concrete resistivity*, ρ, may be obtained as: $\rho = 2RD$ where, D is the diameter of the counter electrode.

- *Determination of polarisation resistance, R'_p:* Referring to Fig. 4, the polarisation resistance measurements may be carried out keeping switch K_1 off throughout. First, the corrosion potential (E_{corr}) is recorded at the K_2 off position, allowing a sufficient response time of 30–60 s. Once the stabilized value of E_{corr} is recorded the switch K_2 is put on and the cathodic polarising current, I_2, is applied in steps until the maximum value of the over-voltage, ε, is reached, which is usually 20 mV for the polarisation curve to be in linear range, with the help of

Table 6
Features of the most widely used methods of corrosion monitoring in RC structures [39]

Characteristic	Potential mapping	Concrete resistivity testing	Linear polarization method	Guard ring method	Coulostatic method	Electrochemical noise method	EIS or A.C. impedence method	Harmonics	Gravimetric test	Quantitative visual observation
Speed of individual measurements	A	A	A	A	A	B	B	B	C	C
Speed of response to change	A	A	A	A	A	A	A	A	C	C
Quantitative information	C	B	A	A	A	B	B	A	A	A
Non-destructive	A	A	A	A	A	A	A	A	C	C
Non-perturbing	A	C	C	C	B	A	B	C	C	C
Measurement parameter	Probability of corrosion	Probability of corrosion	I_{corr}	I_{corr}	I_{corr}	I_{corr}	I_{corr} Mechanism	I_{corr}	Average I_{corr}	Geometrical feature of attack

'A' instantaneous (optimum), 'B' fairly slow (satisfactory), 'C' very slow (un-satisfactory).



R_L = standard decade box resistor
R_V = voltmeter resistance
V = voltmeter capable of reading 0-1.0 V with a least count of 0.1 mV
R.E. = standard Cu/CuSO$_4$ reference electrode as per ASTM C-876
C.E. = disk-shaped counter electrode made of copper with a central hole of 25 mm diameter
W.E. = working electrode (i.e. rebar)
A = ammeter capable of reading 0-100 μA with a least count of 0.1 μA
R = ohmic resistance of concrete
E = e.m.f. of corrosion cell
I_2 = cathodic current applied to the W.E. (i.e. rebar) for polarization
P.S. = constant voltage power supplier
Y = variable resistance to keep circuit resistance high enough to maintain constant current

Fig. 4. Arrangement for determination of corrosion potential, ohmic resistance and polarization resistance.

a variable resistor, *Y*, which helps to keep the circuit resistance high enough to maintain a constant current. The initial cathodic polarising current may be usually taken as 2 μA, and then the second current step may be taken as 4 μA, the third 6 μA, etc. After allowing a response time of 2–3 min at each current step the steady state voltmeter reading may be recorded.

With the help of the polarisation data, i.e., V vs I_{2i} and the Ohmic resistance obtained, the polarised potential E with ohmic drop being eliminated may be obtained using the following equation:

$$E = V - I_2R \tag{16}$$

Then the apparent and true polarisation resistances R'_{p} and R_T, may be determined subsequently, using the following equations:

$$R'_{\mathrm{p}} = \frac{\sum_{i=1}^{n}(\varepsilon_i)^2}{\sum_{i=1}^{n}(I_{2i}\varepsilon_i)} \tag{17}$$

$$R_{\mathrm{T}} = \frac{4(R'_{\mathrm{p}})^2}{R_{\mathrm{e}}}(P) \tag{18}$$

where: $\varepsilon_i = E_i - E_{\mathrm{corr}}$, R_{e} = electrolytic resistance of concrete $= \rho/S$, S is area of the cross-section of the beam, P is perimeter of the rebar or the sum of the perimeters of the rebars, if there are several rebars.

- *Determination of the Tafel slopes, β_a and β_c, and I_{corr}:* Tafel slopes, β_{a} and β_{c}, may be determined by minimizing the error function, Z, which is the square of the difference of LHS and RHS of the following polarisation equation:

$$2.3R'_{\mathrm{p}}I_i = \frac{\beta_{\mathrm{a}}\beta_{\mathrm{c}}}{(\beta_{\mathrm{a}}+\beta_{\mathrm{c}})}\left[\exp\left(\frac{2.3\varepsilon_i}{\beta_{\mathrm{a}}}\right) - \exp\left(\frac{-2.3\varepsilon_i}{\beta_{\mathrm{c}}}\right)\right] \tag{19}$$

$$Z = \sum_{i=1}^{n}\left[A_i - \frac{1}{\left(\frac{1}{\beta_{\mathrm{a}}}+\frac{1}{\beta_{\mathrm{c}}}\right)}\left\{\exp\left(\frac{B_i}{\beta_{\mathrm{a}}}\right) - \exp\left(\frac{-B_i}{\beta_{\mathrm{c}}}\right)\right\}\right]^2 \tag{20}$$

where: $2.3R'_{\mathrm{p}}I_i = A_i$ and $B_i = 2.3\varepsilon_i$.

The minimization of Z may be carried out using a simple method of tabulation, whereby the values of Z may be tabulated for a large number of possible values of $1/\beta_{\mathrm{a}}$ and $1/\beta_{\mathrm{c}}$ at equal spacings within the feasible region, i.e., $0.02 < ((2.3/\beta_{\mathrm{a}}) + (2.3/\beta_{\mathrm{c}})) < 0.12$, and the minimum value of Z is then sought. The value $1/\beta_{\mathrm{a}}$ and $1/\beta_{\mathrm{c}}$ at Z_{minimum} will give the actual value of β_{a} and β_{c}. This minimization may be done using a small computer program; the time required for the computing procedure described above was found to be less than 1 min.

Once β_{a} and β_{c} are determined, the Stern–Geary constant, B, may be calculated using the following equation:

$$B = \frac{\beta_{\mathrm{a}}\beta_{\mathrm{c}}}{2.3(\beta_{\mathrm{a}}+\beta_{\mathrm{c}})} \tag{21}$$

Knowing the values of B and R_T, the I_{corr}, may be calculated using the following Stern–Geary [43] equation:

$$I_{\mathrm{corr}} = \frac{B}{R_{\mathrm{T}}} \tag{22}$$

8. Service life prediction of a corroding RC structure

For a corroding structure, the residual service life at any time may be taken as the time remaining for the crack to develop at concrete surface due to the rust expansion. There are various models [3,44–48] available in the literature for service life prediction. Besides these models, the experimental techniques are also available for predicting the service life. Some of the models and an experimental technique for service life prediction are summarized in the following subsections.

8.1. Service life prediction models

8.1.1. Bazant's model

Bazant's model [44] for the prediction of corrosion damage considers the volume expansion due to the formation of hydrated red rust, ($Fe(OH)_3$), over the residual rebar core. This red rust is expansive in nature and occupies four times the volume of parent steel. Thus, a uniform radial pressure is exerted onto the surrounding concrete resulting in outward radial deformation of concrete. This deformation is same as the increase in diameter ΔD of the rebar (due to deformation compatibility), and increases with an increase in the volume of rust till the cover concrete cracks and is rendered functionally unsatisfactory.

When corrosion is in a steady state with a constant rate, the unacceptable deformation of concrete at cracking, can be related to the duration of steady-sate corrosion, by the following equation:

$$t_{\mathrm{cor}} = \rho_{\mathrm{cor}}\frac{D\Delta D}{pJ_{\mathrm{r}}} \tag{23}$$

where: ρ_{cor} is combined density factor for steel and rust, D is diameter of rebar, p is perimeter of rebar, J_{r} is instantaneous corrosion rate, as given in Eq. (13), ΔD is increase in diameter of rebar due to rust formation, t_{cor} is steady-state corrosion period as defined in Fig. 2.

8.1.2. Morinaga's model

Morinaga [46] has developed an empirical model to compute the amount of corrosion, Q_{cr}, when concrete cover cracks due to expansion by means of rust formation on rebar surface.

The expression to estimate, Q_{cr}, is:

$$Q_{\mathrm{cr}} = 0.602\left(1+\frac{2C_v}{D}\right)^{0.85}D \tag{24}$$

where: Q_{cr} is amount of corrosion when concrete cracks ($\times 10^{-4}$ gm/cm^2).

The duration of steady state corrosion, t_{cor} corresponding to Q_{cr} can be obtained as:

$$t_{\mathrm{cor}} = \frac{Q_{\mathrm{cr}}}{J_{\mathrm{r}}} \tag{25}$$

8.1.3. Wang and Zhao's model

Wang and Zhao [47] have suggested a step method of using finite element analysis to determine the thickness of the corrosion product, Δ, corresponding to the time duration when the surface concrete cracks. Further, by analyzing a large number of rebar corrosion data collected from laboratory and comparing them with the results of finite element analysis, the authors [47] have established an empirical expression to determine the ratio of thickness of corrosion product, Δ, to the depth of rebar penetration, H, corresponding to the cracks in cover concrete. The ratio (Δ/H), as given below, is termed as expansion coefficient, γ and has been expressed as a function of cube strength of concrete, f_{cu}, as:

$$\frac{\Delta}{H} = \gamma = 0.33\left(\frac{D}{C_v}\right)^{0.565} f_{\mathrm{cu}}^{1.436} \tag{26}$$

Using the value of Δ obtained through the finite element model, the value of H corresponding to cracks in cover concrete can be obtained. Further, the value of H can be used to determine the time necessary for longitudinal cracking of concrete cover, t_{cor} as:

$$t_{\mathrm{cor}} = \frac{H}{P_{\mathrm{r}}} \tag{27}$$

where: P_{r} is penetration rate of rebar due to corrosion, as given by Eq. (14).

The model suggested by Wang and Zhao [47] can be used only in conjunction with the finite element model requiring the determination of Δ.

8.1.4. Dagher and Kulendran's model

Dagher and Kulendran [48] have also carried out a finite element modeling of corrosion damage in concrete structures. This numerical model is rather versatile in terms of estimating the radial bar expansion Δ, and includes: (a) a number of options for modeling crack formation and propagation, (b) the capacity to accept any shape of corrosion around the rebars, (c) the ability to incorporate dead and live load stress and initial shrinkage and temperature cracks in the analysis, and (d) pre- and post-processing modules which offer automatic mesh generation and visual representation of crack propagation.

In the context of service life prediction of RC structures subjected to rebar corrosion, this model can be used more reliably to determine the radial bar expansion Δ, at which the cracks in cover concrete would occur. However, their work [48] will require extension to make it capable of service life prediction.

8.2. Experimental method of service life prediction

An experimental methodology, suggested by Ahmad et al. [49], for service life prediction based on cumulative damage theory is described below:

If a RC specimen is allowed to corrode for a time duration, L_{c}, at its natural corrosion rate, I_{corr}, from the time of depassivation, t_{p}, onward and then the same is subjected to an impressed anodic current, I_{a}, up to the time of cracking, t_{cor}, of cover concrete, then according to the cumulative damage theory, one can write the following equation:

$$\frac{L_{\mathrm{c}}}{L_{\mathrm{t}}} + \frac{L_{\mathrm{a}}}{L_l} = 1 \tag{28}$$

where: $L_{\mathrm{c}}/L_{\mathrm{t}}$ is fraction of damage due to the natural corrosion, L_{a}/L_l is fraction of damage due to externally applied current, L_{a} is the time taken by the specimen for its complete damage under the effect of external current being applied after the natural corrosion that had already taken place; L_l is the time taken by the specimen for its complete damage under applied current alone excluding the effect of natural corrosion; L_{c} is duration of the natural corrosion, starting from the time of depassivation of rebar to the date of application of external current; and L_{t} is the total life of specimen against corrosion cracking of cover concrete.

The values of L_{a} and L_l may be obtained through an accelerated corrosion test by applying anodic current to rebar of a core-shaped corroding RC specimen, either cast in laboratory or obtained from a corroding structure. Before the application of anodic current to the RC specimen using the arrangement for the accelerated test, as shown in Fig. 5, the failure tensile stress of concrete, F_0, of the RC specimen under splitting, is determined.

After applying the anodic current, I_{a}, for a time period of T, the failure tensile stress of concrete, F_T, of the RC specimen under splitting, is to be determined.

Then the value of L_{a} may be determined using the following equation, derived by Ahmad et al. [49]:

$$L_{\mathrm{a}} = \frac{F_0 f\left(\frac{C_v}{D}\right)}{E I_{\mathrm{a}} K_1} \tag{29}$$

where: $f(C_v/D)$ is a function of the ratio of cover thickness, C_v, to the rebar diameter, D

The above ratio function is given as:

$$f(C_v/D) = \frac{4.72\left(\frac{C_v}{D}\right)^2 + 4.72\left(\frac{C_v}{D}\right) + 1}{\left(\frac{C_v}{D}\right) + 1} \tag{30}$$

E is modulus of elasticity of concrete, K_1 is constant, which may be determined through the best fit of the following equation:

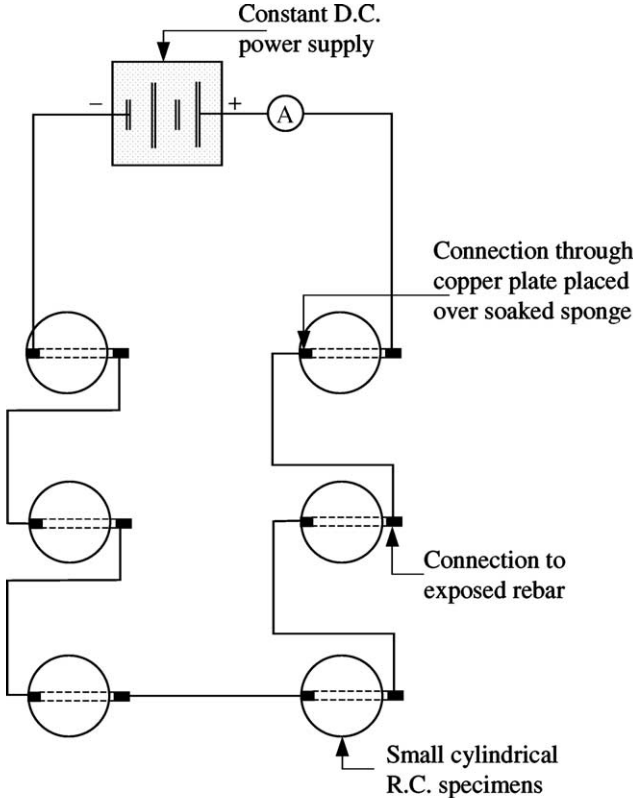


Fig. 5. Arrangement for accelerating the corrosion of rebar in the RC specimens by applying anodic current, I_a, to rebar.

$$\frac{F_T}{F_0} = 1 - K_1\left[\frac{ETI_a}{F_0 f\left(\frac{C_v}{D}\right)}\right] \tag{31}$$

Then the value of L_l may be determined using the following equation, derived by Ahmad et al. [49]:

$$L_l = \frac{1}{K_2} \tag{32}$$

K_2 is a constant, which may be determined through the best fit of the following equation;

$$\frac{1}{L_a} = K_2 + K_3 \cdot \mathrm{NCF} \tag{33}$$

where: K_3 is a constant, NCF is a natural corrosion factor, which is given by the following equation:

$$\mathrm{NCF} = \frac{EL_c I_{\mathrm{corr}}}{f_t' f\left(\frac{C_v}{D}\right)} \tag{34}$$

where: f_t' is cylindrical tensile strength of concrete, I_{corr} is corrosion rate of rebar of RC specimen before the application of I_a.

Substituting the above expressions of L_a and L_l in Eq. (28), the final expression for L_t may be obtained as:

$$L_t = L_c \Big/ \left[1 - \left(\frac{K_2}{K_1}\right)\left\{\frac{F_0 f(C_v/D)}{EI_a}\right\}\right] \tag{35}$$

9. Conclusions

A brief review of the various aspects of reinforcement corrosion, presented in this paper, may be useful for the following purposes:

(i) To understand the theory of reinforcement corrosion,
(ii) to appreciate the internal (pertaining to the materials) and external (pertaining to the surrounding environment) factors affecting reinforcement corrosion,
(iii) to study how reinforcement corrosion progresses with time and what are the effects of corrosion on the behavior of the structures,
(iv) to plan a strategy for investigating a corroding RC structure,
(v) to select suitable corrosion monitoring techniques for first confirming the corrosion and then assessing its extent,
(vi) to use an experimental set-up, based on linear polarization method, for in situ measurement of corrosion rate, and
(vii) to make use of various models and an experimental technique for predicting remaining service life of a corroding RC structure.

Acknowledgements

The author acknowledges the support provided by King Fahd University of Petroleum and Minerals in the preparation of this review paper.

References

[1] ACI Committee 222-corrosion of metals in concrete. ACI Manual of Concrete Practice. Part-1; 1992.
[2] Hansson CM. Comments on electrochemical measurements of the rate of corrosion of steel in concrete. Cement Concr Res 1984;14:574–84.
[3] Bazant ZP. Physical model for steel corrosion in concrete sea structures-theory. ASCE J Struct Div 1979;105:1137–53.
[4] Cahyadi JH, Uomoto T. Influence of environmental relative humidity on carbonation of concretre (mathematical modeling). In: Nagataki S et al., editors. Durability of building materials and components 6. E & FN Spon; 1993. p. 1142–51.
[5] Mozer JD et al. Corrosion of reinforcing bars in concrete. J Am Concr Inst 1965:909–31.
[6] Uhlig HH. Corrosion and corrosion control. New York: John Wiley and Sons; 1983.
[7] Berkely KGC, Pathmanaban S. Cathodic protection of reinforcement steel in concrete. London: Butterworths & Co. Ltd.; 1990.
[8] ASTM C: 1152- standard test method for acid-soluble chloride in mortar and concrete 1990:609–10.
[9] Arya C, Buenfeld NR, Newman JB. Assessment of simple methods of determining the free chloride ion content of cement pastes. Cement Concr Res 1987;17:907–18.

[10] Hope BB, Alan Ip KC. Chloride corrosion threshold in concrete. ACI Mater J 1987;84:306–14.

[11] Pullar-Strecker P. Corrosion damaged concrete: assessment and repair. London: Butterworths; 1987.

[12] Uhlig HH. Corrosion and corrosion control—an introduction to corrosion science and engineering. New York: John Wiley & Sons, Inc.; 1971.

[13] ASTM Special Technical Publication 818-corrosion of metals in association with concrete: 1983.

[14] Rasheeduzzafar. Influence of cement composition on concrete durability. ACI Mater J 1992;89(6):574–86.

[15] Rasheeduzzafar, Dakhil FH, Al-Gahtani AS, Al-Saadoun SS, Bader MA. Influence of cement composition on the corrosion of reinforcement and sulfate resistance of concrete. ACI Mater J 1990;87(2):114–22.

[16] ACI Committee 201-guide to durable concrete. ACI Mater J 1991;88(5).

[17] Jaegermann C. Effect of water–cement ratio and curing on chloride penetration into concrete exposed to Mediterranean sea climate. ACI Mater J 1990;87(4):333–9.

[18] Ho DWS, Lewis RK. Carbonation of concrete and its prediction. Cement Concr Res 1987;17:489–504.

[19] Kobayashi K, Suttoh K. Oxygen diffusivity of various cementitious materials. Cement Concr Res 1991;21:273–84.

[20] Goto S, Roy DM. The effect of w/c ratio and curing temperature on the permeability of hardened cement paste. Cement Concr Res 1981;575.

[21] Al-Saadoun SS, Rasheeduzzafar, Al-Gahtani AS. Mix design considerations for durable concrete in the Arabian Gulf environment. The Arab J Sci Eng 1992;17(1):17–33.

[22] Al-Amoudi OSB. Studies on the evaluation of permeability and corrosion resisting characteristics of portland pozzolan concretes. M.S. Thesis, King Fahd University of Petroleum and Minerals, Dhahran; 1985.

[23] Verbeck GJ. Field and laboratory studies of the sulphate resistance of concrete. In: Performance of concrete. University of Toronto Press; 1968. p. 113.

[24] US Bureau of Reclamation. Concrete Manual, Denver. 1975;37.

[25] Cordon WA, Gillespie HA. Variables in concrete aggregates and portland cement which influence the strength of concrete. ACI J Proc 1963;60:1029.

[26] Maslehuddin M. Optimization of Concrete Mix Design for Durability in the Eastern Province of Saudi Arabia. M.Sc. Thesis, Department of Civil Engineering, King Fahd University of Petroleum and Minerals, Dhahran; 1981.

[27] Rasheeduzzafar, Al-Gahtani AS, Al-Saadoun SS. Influence of construction practices on concrete durability. ACI Mater J 1989;86(6):566–75.

[28] Beeby AW. Corrosion of reinforcing steel in concrete and its relation to cracking. The Struct Engr 1978;56A(3):77–81.

[29] Schiessl P. Admissible crack width in reinforced concrete structures. Contribution II, 3-17. International Colloquium on the Behavior in Service of Concrete Structures; 1975.

[30] Rasheeduzzafar, Dakhil FH, Al-Gahtani AS. Corrosion of reinforcement in concrete structures in the Middle East. Concr Int: Design Construct 1985:48–55.

[31] Dakhil FH, Cady PD, Carrier RE. Proc J, Am Concr Inst 1975;72(8):421.

[32] Tyler IL. Long-time study of cement performance in concrete, chapter 12, concrete exposed to seawater and fresh water. J Am Concr Inst Proc 1960;57:825–36.

[33] Rasheeduzzafar, Dakhil FH, Bader MA. Toward solving the concrete deterioration problem in the Arabian Gulf region. The Arab J Sci Eng 1986;11(2):129–46.

[34] Schiessl P. Corrosion of steel in concrete. Report of the Technical Committee, 60-CSC, RILEM; 1988.

[35] Dhir RK et al. Measurement of reinforcement corrosion in concrete structures. Concrete 1991:15–9.

[36] ASTM C876—standard test method for half-cell potentials of uncoated reinforcing steel in concrete 1991;03(02):434–9.

[37] Bungey JH. The testing of concrete in structures. London: Surrey University Press; 1989.

[38] Ijsseling FP. Application of electrochemical methods of corrosion rate determination to system involving corrosion product layers. Br Corros J 1986;21(2):95–101.

[39] Rodriguez P, Ramirez E, Gonzalez JA. Methods for studying corrosion in reinforced concrete. Mag Concr Res 1994;46(167):81–90.

[40] Jones DA. Polarization in high resistivity media. Corros Sci 1968;8:19–27.

[41] Feliu S, Gonzalez JA, Andrade C, Feliu V. On-situ determination of the polarization resistance in a reinforced concrete beam. Corrosion 1988;44(10):761–5.

[42] Ahmad S, Bhattacharjee B. A simple arrangement and procedure for in-situ measurement of corrosion rate of rebar embedded in concrete. Corros Sci 1995;37(5):781–91.

[43] Stern M, Geary AL. Electrochemical polarization—a theoretical analysis of the shape of polarization curves. J Electrochem Soc 1957;104:56–63.

[44] Bazant ZP. Physical model for steel corrosion in concrete sea structures—application. ASCE J Struct Div 1979;105:1155–66.

[45] Ahmad S, Bhattacharjee B. Assessment of service lives of reinforced concrete structures subjected to chloride-induced rebar corrosion. J Struct Eng 1997;23(4):177–82.

[46] Morinaga S. Prediction of service lives of reinforced concrete buildings based on the corrosion rate of reinforcing steel. Proceedings of Building Materials and Components, Brighton, UK, 7–9 November 1990:5–16.

[47] Wang XM, Zhao HY. The residual service life prediction of R.C. structures. In: Nagataki S et al., editors. Durability of building materials and components 6. E & FN Spon; 1993. p. 1107–14.

[48] Dagher HJ, Kulendran S. Finite element model of corrosion damage in concrete structures. ACI Struct J 1992;89(6):699–708.

[49] Ahmad S, Bhattacharjee B, Wason R. Experimental service life prediction of rebar corroded reinforced concrete structure. ACI Mater J 1997;94(4):311–6.