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Modelling corrosion propagation in reinforced concrete structures – A critical review

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ABSTRACT

The adoption of corrosion initiation as a limit state to define service life of RC structures has been challenged by researchers and engineers alike in light of the advancements in the concrete construction industry: improved reliability and safety, reduction in costs, and conservation of both materials and energy, which contribute towards sustainable concrete construction. The corrosion propagation phase is now appreciated as a significant component in the service life of RC structures and a good understanding of the propagation process is paramount. Various models have been developed to simulate and/or predict the propagation phase. This paper presents a critical review of some of the available models for corrosion propagation, and proposes ways forward to overcome some of these problems. Salient issues including the modelling techniques, input parameters and limit states are covered. Emphasis is also placed on the usefulness of the propagation models as tools to aid in the repair and maintenance of corrosion-damaged RC structures.

1. Introduction

Modelling has become a powerful tool used by researchers and engineers alike to understand the response of RC structures to service loads and to predict their performance, especially with respect to deterioration and residual load-carrying capacity under different service conditions. This trend has now been extensively embraced in the study of corrosion-affected RC structures, where focus has shifted to the propagation phase, but without neglect of the initiation phase. Several reasons may be cited for the increased use of modelling in the field of corrosion-affected RC structures but the main reasons are: (i) laboratory and field experiments (even with accelerated tests) are relatively expensive and time consuming and (ii) difficulty in replicating different test scenarios, i.e. isolating different variables in the test environment to replicate different real exposure conditions for RC structures.

However, even though numerical simulation of the corrosion process has been used to develop prediction models for the corrosion propagation phase in RC structures, results so far have shown that it cannot be used independently of laboratory and/or field tests to obtain accurate results. This is because in the majority of cases, the simulations do not replicate the real corrosion process and/or exposure conditions [1]. To overcome this limitation, parallel laboratory and/or field experiments, and modelling may be carried out, with the laboratory/field test results being used to validate the modelling process [2] and hence the model developed. The two approaches can be said to be complementary and should be treated as such. Only a few studies where such a process has been carried out can be cited in the literature [3–5].

Nevertheless, even models that have been validated may not be infinitely applicable with respect to their accuracy over a given period of time. It is therefore recommended that calibration of such models [6] using data from long-term field tests is done. Furthermore, improved understanding of both the corrosion mechanisms (chemical and kinetic processes) and material (concrete, steel and concrete–steel composite) properties warrants the refinement of previous models to account for such improvements.

This paper presents a critical review of the modelling of the corrosion propagation phase in RC structures. First, a brief overview of the different approaches that can be used to model corrosion propagation are presented. These will then be critiqued, and conclusions drawn.

2. Prediction models for corrosion propagation

The service life ($t_{service}$) of RC structures, with respect to reinforcement corrosion, is usually modelled as comprising of distinct phases following pre-defined (serviceability and ultimate) limit states with distinct corrosion-induced damage indicators. This approach was first used by Tuutti [7] who proposed a conceptual model dividing the service life of a RC structure into two distinct

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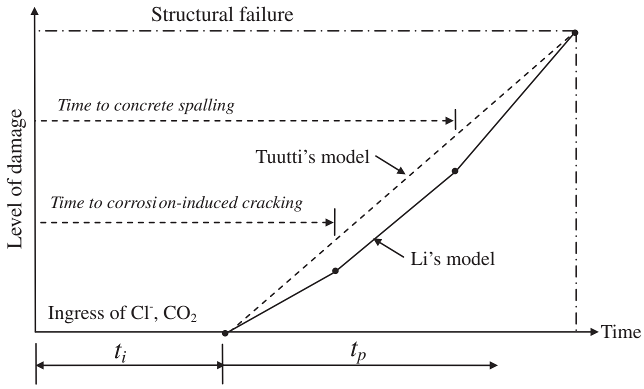

Fig. 1. Phases and sub-phases in the service life of corrosion-affected RC structures [7,8].

phases viz the corrosion initiation phase (t_i) and corrosion propagation phase (t_p), i.e. $t_{service} = t_i + t_p$ (Fig. 1). However, Tuutti's model was generalised with respect to t_p; it does not depict the different sub-phases of corrosion-induced damage in the propagation phase. To account for this (i.e. differentiate structural response to corrosion-induced damage), t_p, which is the focus of this study, can further be sub-divided into sub-phases as shown in Fig. 1, for example [8].

The duration of the propagation period depends principally on the corrosion rate, which is affected by several factors [9–14]. The associated deterioration leads to a variety of negative effects with respect to both structural and durability performance of the RC structure. The prediction of corrosion propagation is therefore a complex process mainly due to the difficulty in incorporating all the relevant factors affecting the process and the associated damage in a prediction model. Usually, one of the negative corrosion-induced damage effects is adopted as a limit state in the prediction model. The pre-defined acceptable level of damage (i.e. the *limit state* or *damage indicator*) can be said to denote the *end of corrosion propagation period*. Corrosion-induced damage in RC structures can range from loss of steel cross-section [15], loss in stiffness [16], loss of steel–concrete interface bond [17], cracking of concrete cover [3,4], to local or global failure of the structure or its members respectively. However, for repair purposes, global failure (collapse of the structure) cannot be adopted as a limit state mainly due to human safety reasons. A detailed coverage of these limit states can be found in the literature; but in summary, some of the basic requirements of a limit state indicator adopted should include the following: (i) it should be easy to assess and quantify, (ii) the level of damage should not compromise structural integrity such as its stability and hence safety of the users/occupants, and (iii) the damage should be relatively easy to repair in terms of restoring both structural integrity and durability performance requirements.

A corrosion propagation prediction model can be developed based on any, or a combination of, the already mentioned corrosion damage indicators. However, it is important to note that, to date, the available prediction models adopt only one damage indicator and are therefore only valid for the given damage indicator. The possibility of using more than one limit state criterion still remains to be explored objectively.

Regardless of the damage indicator adopted, prediction models for corrosion propagation can be grouped as either analytical, numerical or empirical depending on the criterion used in their development [18]. The following section will give a brief overview of prediction models for corrosion propagation, but without specific mention of specific available models, as this will be done in the next section.

2.1. Empirical models

These are models based on assumed direct relationships between corrosion rate and basic concrete parameters, e.g. w/b ratio, binder type and environmental parameters [19]. They are usually developed using data from laboratory factorial experiments that, by design, isolate other corrosion-influencing parameters. Empirical models are sub-divided into three types viz [20]:

(i) *Expert Delphic oracle models*: Corrosion rate is estimated based on past years' experience. However, it has not been used for chloride-induced corrosion due to its complexity.
(ii) *Fuzzy logic models*: In these models, sets of assumed relationships are defined hence allowing the calculation of corrosion rate using fuzzy set logic theory [21,22]. It has been used for the assessment of corrosion-induced deterioration and to estimate the reduction in steel cross-sectional area [19]. Fuzzy set theory has been criticised in the past for its inability to reflect different kinds of fuzzy phenomena in the natural world (e.g. corrosion process) correctly but this has been modified [23].
(iii) *Models based on electrical resistivity and/or oxygen diffusion resistance of concrete*: These assume that concrete electrical and oxygen diffusion resistance are the main controlling factors for the corrosion process. They indirectly takes into account other influencing factors including exposure conditions, w/b ratio and binder type [24,25].

One of the main disadvantages of empirical models is that the selected variables under consideration (for both concrete (*material*) and corrosion (*process*)) are investigated in isolation from other influencing parameters and/or the interaction thereof. Consequently such models may be limited to the set of conditions under which they are developed. However, the end-users are usually either not aware of the limitations associated with the models or choose to neglect them. A common procedure, especially among practising engineers, is to select the most convenient model (based on the available or easily quantifiable input parameters) and use it depending on the available input parameters. This can lead to either under- or over-estimation of the service life of the RC structure, of which the latter may be catastrophic with respect to structural failure and hence occupants' safety.

2.2. Numerical models

A numerical (mathematical/analytical) model is a set of mathematical (analytical) equations which when solved, gives approximate solutions of the subject parameter(s) over time [26]. Numerical simulations can be used to estimate corrosion rates, the effects of changes in electrochemical conditions, and structural response to corrosion-induced damage. Three different approaches can be used to develop numerical models viz [27]: (i) finite element method (FEM), (ii) boundary element method (BEM), and (iii) resistor networks and transmission line method. These are covered in the following sections.

2.2.1. Finite element method approach

The finite element method (FEM) is a process of approximation to continuum problems such that: (i) the continuum is sub-divided into a finite number of individual parts (elements), the behaviour of which is specified by a finite number of parameters whose behaviour can be readily understood and (ii) the solution/understanding of the complete system is an assembly of its individual elements, i.e. the sum of sub-models [28].

With respect to RC structures, the steel, concrete, concrete–steel interface and the bulk phase of the concrete can all be modelled using FEM. Therefore, in FEM models, all the three aspects of interfacial properties, changes in transport properties of the concrete with time, and the geometrical properties can be taken into account. FEM models also provide the ability to easily vary the bulk concrete properties. However, the main drawback of these models is that for practical situations, the numerical size of the model can be impractically large and hence expensive and time consuming [29].

In FEM models, the objective is to satisfy the boundary conditions set for the problem (in this case steel corrosion in concrete). The boundary conditions may include temperature, relative humidity, chloride content (in the case of chloride-induced corrosion), concrete resistivity and the electrochemical behaviour of both active and passive steel [5]. The active or passive state of the steel can be described, for example, using polarisation curves expressed as Butler–Volmer relations (Eqs. (1) and (2)) between current density and potential for the active and passive steel areas [29]:

$$g_A = i_{corr,A}\left\{\exp\left(\frac{V - V_{corr,A}}{b_{a,A}}\right) - \exp\left(\frac{-V - V_{corr,A}}{b_{c,A}}\right)\right\} \quad \text{for active steel area} \tag{1}$$

and;

$$g_P = i_{corr,P}\left\{\exp\left(\frac{V - V_{corr,P}}{b_{a,P}}\right) - \exp\left(\frac{-V - V_{corr,P}}{b_{c,p}}\right)\right\} \quad \text{for passive steel area} \tag{2}$$

where g: normal component of the current density (i_{corr}), V: potential, V_{corr}: free corrosion potential, b_a, b_c: slopes of the anodic and cathodic polarisation curves respectively, and subscripts *A, P*: active and passive states of corrosion respectively.

2.2.2. Boundary element method approach

In the boundary element method (BEM), only the concrete–steel interfaces are modelled as opposed to FEM models where the corrosion process is comprehensively described [29]. In BEM, the objective is to satisfy the differential equations used to describe the (corrosion) system. Distinct boundary conditions that can be applied to the surface elements of a model include: (i) constant corrosion potential, (ii) constant current density, and (iii) linear or non-linear relation between current density and potential.

In comparison with FEM, BEM has the advantage that much fewer number of elements is required and that two dimensional elements can be used to simulate a three dimensional problem. A further advantage of BEM is the reduction of the problem dimension and hence the cost of pre-processing and mesh generation is also greatly reduced. The main disadvantages of BEM are the requirements that the electrolyte conductivity has to be constant and that within the electrolyte, results can only be calculated at discrete points [26].

2.2.3. Resistor networks and transmission line approach

In this approach, the relationship between the driving voltage (corrosion potential), the resistances of the corroding system, and the electrical macrocell current, which is proportional to the corrosion rate, can be calculated based on simplified electrical circuits. Models developed using this approach consist of the driving voltage (U_e), the resistances of steel, anode, cathode and electrolyte, and the electrical current (I_e) flowing between the anode and the cathode (Fig. 2). The galvanic current can be calculated using the following equation [30]:

$$I_e = \frac{U_{R,c} - U_{R,a}}{\frac{r_a}{A_a} + \frac{r_c}{A_c} + \frac{\rho_{el}}{k}} \tag{3}$$

where I_e: corrosion rate, i.e. electrical current between the anode and cathode, $U_{R,c}$, $U_{R,a}$: equilibrium potential at the cathode and anode respectively, r_a, r_c: specific anodic and cathodic polarisation resistances respectively, A_a, A_c: Anodic and cathodic steel surface areas respectively, ρ_{el}: specific resistance of the electrolyte (concrete) and k: cell constant geometry. The resistance of the steel, R_{st}, with respect to the transport of the electrons, is usually ignored because it is negligibly small compared to other active resistances.

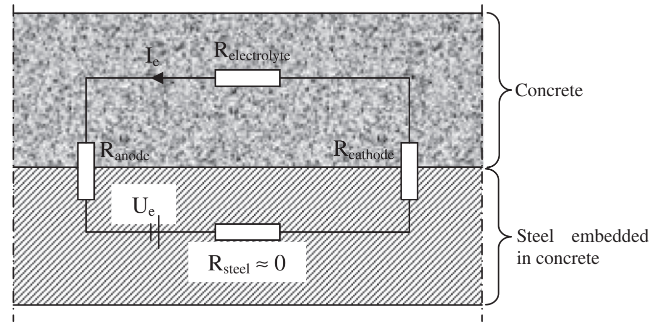

Fig. 2. Simplified schematic electrical circuit model for the corrosion of steel in concrete [28].

The shortcoming of this approach is that the geometry of the corrosion cells, i.e. the anode/cathode surface area ratio, insulation distance between the anode and cathode, electrolyte film depth and the shapes of the anode and cathode [31], which also have an influence on the corrosion rate, are not taken into account. The galvanic cell geometry determines the galvanic current and thus significantly influences the galvanic corrosion behaviour.

2.3. Analytical models

These are models based on closed-form solutions to mathematical equations, i.e. the solutions to the (known) theoretical equations used to describe the system and/or the changes in the system can be expressed as a mathematical analytical function [26]. In this approach, RC is usually modelled using a thick-walled cylinder approach as shown in Fig. 3 (where *D* is the diameter of steel bar, d_0 is the thickness of the annular layer of concrete pores (i.e. a pore band) at the concrete–steel interface, and *C* is the concrete cover) [32,33].

The schematic shown in Fig. 3 has been used in the past to model both corrosion-induced cracking of the concrete cover and corrosion-induced bond degradation. Examples of corrosion propagation prediction models developed using this technique include those by Li et al. [32], Bhargava et al. [33] and Coronelli [34].

However, the approach has been mostly used to model corrosion-induced cracking [32–35]; where, to simulate the process, the internal circular boundary at the concrete–steel interface is assumed to be displaced so as to accommodate the expansive corrosion products resulting in the evolution of the (uniform) expansive radial pressure at the boundary [33].

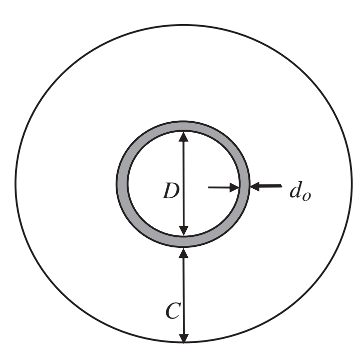

Fig. 3. Schematic representation of embedded steel in analytical RC models [30,31].

A limitation of the thick-walled uniform cylinder model described above is its inability to account for non-linear behaviour of concrete, which takes place when radial cracks start to form near the inner surface of the cylinder. This can be overcome by partition of the cylinder into two parts (a cracked inner cylinder and an uncracked outer one) [36] and taking into account tension softening. Another limitation of most corrosion-induced cracking models developed using this approach, as noted by Chernin et al. [37], is the assumption of plane stress formation; which is incorrect considering that the concrete cylinder around a reinforcing bar is actually within the bulk concrete of a RC element, which prevents free deformation in the direction of the cylinder axis. Under such conditions, plane strain formation is more appropriate. Plane stress would be applicable only if the ends of the pressurised cylinder were free. Similar to numerical models, these models should be calibrated against experimental (laboratory and/or field) test results before they can be reliably applied.

3. Critique of existing corrosion propagation models

A number of prediction models [3,4,24–26,29,33,35,38–45] have been developed in the past based on the approaches discussed in the previous section. However, it is necessary to note that although a variety of limit states are available within the corrosion propagation phase, most of the available models have adopted corrosion-induced cracking of the concrete cover as a limit state [3,4,40,45]. This can be attributed to the complexity in quantifying some of the limit states either in the laboratory or in real structures, or both. It is the aim of this section to critically review some of these models, and where possible, suggest ways forward with respect to modelling the corrosion propagation phase in RC structures. This will be done under different sub-sections.

3.1. Model input parameters (variables)

One of the major drawbacks of existing propagation models is the difficulty in obtaining accurate and easily quantifiable input parameters. Most of the existing prediction models, e.g. El Maaddawy and Soudki [35] and Liu and Weyers [40] the corrosion-induced cracking models, have input parameters, which in most cases due to the inability to easily obtain representative or actual values, are usually arbitrarily set, e.g. tensile strength, modulus of elasticity, Poisson's ratio, creep coefficient of concrete, type of corrosion products formed (and their chemical composition), and size of the concrete–steel interface.

3.2. Model validation

Model validation is the process of substantiating that the model, within its pre-defined domain (or context) of applicability, gives results with satisfactory accuracy, consistent with the modelling objectives [2]. However, inasmuch as model validation ensures that the model meets its intended requirements in terms of the theories employed and the results obtained it should not be taken as an absolute proof of its validity but as an indication of its validity [46]. Some of the existing prediction models for corrosion propagation have never been (successfully) validated [4,38,40] and hence cannot be reliably applied to real structures. In some cases, the validation is usually done using data from accelerated tests, the disadvantages of which will be discussed later.

3.3. Model assumptions

In an attempt to simplify the numerical modelling process, and due to lack of a good understanding regarding the material (concrete, steel) and its behaviour, several assumptions are usually made *a priori*. Some of the assumptions made in previous models include: (i) constant/steady corrosion rate [38], (ii) types of corrosion products formed [38], (iii) uniform loss of steel cross-section along the perimeter of the bar [4], (iv) critical amount of corrosion products required to induce cracking of the cover concrete [40,45], (v) thickness of the porous zone around the concrete–steel interface [40], and (vi) concrete as a homogeneous, isotropic and linear elastic material [33], among others. Inasmuch as it may be valid to make assumptions based on sound engineering knowledge and judgement, such assumptions should be checked at the model validation stage to ensure they still hold. Furthermore with improved knowledge and/or experience, such assumptions should be refined.

3.4. Accelerated laboratory tests

The use of accelerated galvanostatic or potentiostatic corrosion tests in the laboratory has become a common technique to simulate corrosion-induced damage in RC structures, mainly because results can be obtained within a short period of time. However, this technique has been criticised for not being representative of the natural corrosion process and hence the results obtained from such tests may not be reliably extended to real structures [3,47]. Furthermore, previous studies [47] have shown that the use of Faraday's law to obtain corrosion rates from such tests may not be valid and leads to over- or under-estimation of mass losses. In most studies, 100% current efficiency is assumed, i.e. all the applied/resulting current is assumed to be consumed in dissolution of the steel [47]. However, this may not be the case because (i) acidification (up to a pH of approx. 3) developed by the progressive corrosion may induce a simultaneous additional corrosion [47], (ii) there are parts of the metal that may not dissolve electrolytically but that spall out from the metal surface when the surrounding material is oxidised [47] and, (iii) heat generation may cause losses in current [3]. It is therefore important that results from accelerated tests are carefully used to validate numerical models.

3.5. Size of test specimens and sample size

It is common that accelerated laboratory tests, and even field tests, are carried out using small-sized as opposed to real-sized RC specimens mainly due to cost and space constraints [3,25]. Accelerated corrosion damage tests carried out using such specimens may not be representative of the real RC structure with respect to its response to applied loads (if any) and the corrosion-induced damage. Most existing models are usually developed based on isolated RC members [3,4], which in most cases are beams. It is important to appreciate that the response of an isolated RC member may not be the same as that of the same member when considered in conjunction with the RC structure as a whole.

The sample size used to obtain data, especially for the purposes of model validation, is also an important aspect to consider. The need for a statistically sufficient number of tests, even though costly, is paramount if the data collected are to be considered reliable [18].

3.6. Influence of cracking

The influence of service load-induced cracking on corrosion initiation and propagation has received much attention in the recent past and a general consensus, based on a number of studies [13,14,48–51], is that its influence should be taken into account when dealing with corrosion in RC structures. Only one empirical model by Scott [25], developed using cracked RC beam specimens, can be cited in the literature; and even in this model, cracking is not explicitly incorporated in the model. None of the available numerical models take the influence of load-induced cracking on corrosion into account. In most cases, interest is always on corrosion-induced cracking; and even in such instances, the influence of such type of cracking on subsequent corrosion rates is not considered. It is important to appreciate the influence of load-induced cracking on corrosion propagation when developing prediction models so that they can be representative of real structures.

3.7. Accounting for variability

Variability of model input parameters is important and should be modelled in such a way that realistic results, and hence decisions, can be derived from the model. Probability-based methods should be applied in the development of corrosion propagation prediction models to take into account this variability. A few examples can be cited in the literature where attempts have been made to carry out such an analysis viz Marita [41], Duracrete [44] and Christensen [39]. Aspects of variability which are seldom taken into account are spatial and temporal variability, although they greatly affect the behaviour of concrete structures [52,53]. The fact that many parameters also show spatial and temporal variability is usually not explicitly taken into account. Spatial variability may be linked to dependencies on temperature, w/b ratio, micro-climate, humidity and workmanship [54] while temporal variability takes into account the variation of concrete parameters that vary with time. The available models neglect spatial and temporal variation within a structure or an element and take the whole structure or element as fully homogeneous, the result(s) at one point being applicable for the entire structure or element. The consequences of such models on decision-making, for example, would be to suggest a total repair or replacement of the structure or element; and would rule out local repairs as a reasonable option.

Furthermore, a model may comprise several *sub-models*, and errors associated with its development can be cumulative. Although accurate representation of the RC concrete's applied loads, response to applied loads and the corrosion process is a difficult task in the modelling process, effort should be made to minimise, if not eliminate, errors in the sub-models. The use of appropriate statistical analyses should be used to resolve this [18].

4. Discussion

The damage caused by corrosion of reinforcing steel, coupled by the complex chemical and kinetic nature of corrosion, is not as straightforward as the available prediction models may imply. Even though some researchers have appreciated that several factors affect the response of a RC structure with respect to corrosion-induced damage, most of these factors are yet to be quantified, e.g. types of corrosion products formed and their chemical composition, and porosity of the concrete–steel interface. The common approach, especially in numerical models, is to arbitrarily assign values for these parameters, a process that compromises the accuracy of the prediction results, but which cannot be ascertained.

As a consequence, use of the available prediction models is usually limited to the exposure conditions under which they were developed in the case of experimental empirical models, or to the values set (some arbitrarily) for the different input parameters used and theories and/or assumptions adopted in the model development in the case of numerical ones. In cases where numerical models are validated using experimental and/or field tests, a few successes (with respect to convergence of both the numerical and experimental/field results) have been reported [3,4]. The discrepancies between the numerical and experimental results may be caused by a number of factors including:

(i) Use of inaccurate models for corrosion rate.
(ii) Lack of knowledge of the chemical composition and properties of the corrosion products.
(iii) Lack of knowledge of the residual strength of concrete after cracking.
(iv) Lack of models to appropriately describe the response of the RC structure to corrosion-induced damage.
(v) Improper selection of material properties, e.g. for the cover concrete.

Further, discrepancies have also been reported between similar prediction models of various researchers, with some over- or under-estimating either the time required to attain the adopted limit state (e.g. corrosion-induced cracking of the concrete cover) or the severity of the damage. These inter-model discrepancies may be ascribed to one or a combination of the following factors:

(i) Differences in test methods used to obtain the input parameters or to validate numerical models, e.g. tensile strength of concrete, accelerated versus natural corrosion processes, etc.
(ii) Differences in assumptions made at the model development stage, e.g. modelling corrosion as 2-dimensional as opposed to 3-dimensional, assuming concrete to be a homogeneous, isotropic and linear elastic material [33], or types of corrosion products formed at the concrete–steel interface.
(iii) Isolation of different corrosion-affecting factors during the modelling or testing process.

It is also apparent that the majority of the available models are deterministic and do not account for the stochastic nature of the corrosion process and the model input parameters. The result is that point estimates are obtained as opposed to interval estimates [18] that more closely describe the corrosion phenomenon and the RC structure's response to corrosion-induced damage. This can greatly affect the formulation of proactive maintenance and repair strategies with respect to action times.

Finally, it is clear from this review that both field and laboratory data should be combined with numerical modelling to formulate models for the prediction of the time to pre-defined limit states, or to estimate the time to necessary maintenance and/or repair. Maintenance and repair strategies can therefore be formulated based on predictions of such models. However, this is only possible if the adopted limit state is easily quantifiable and representative of the actual structure's response to corrosion-induced damage, e.g. deflections and corrosion-induced cracking of the concrete cover.

5. Conclusions

This paper has presented a critical review of the modelling of corrosion propagation in RC structures, citing areas that still require further research. The following conclusions can be drawn:

(i) Modelling the corrosion of steel in RC structures requires a good understanding of both the corrosion process and the associated effects on the structure.
(ii) The validity of most prediction models is always limited to the set of conditions under which it was developed. The end-users of such models should be made explicitly aware of such conditions to avoid improper predictions.
(iii) Model validation using data from natural corrosion assessments on real structures should be preferred to the use of laboratory accelerated tests.

(iv) Prediction models should account for:
  (a) the effects of both load- and corrosion-induced cracking on corrosion rate and,
  (b) variability of both concrete as a material and the corrosion process. This should encompass both the variability in the input parameters, spatial and temporal variability of the RC structure. Temporal variability can be taken into account by carrying out a time-dependent analysis, i.e. re-evaluation of the time-dependent parameters (e.g. polarisation of steel surface, type and amount of rust production, pH of the pore solution) at every time-step of analysis. In general, service life prediction of corrosion-affected RC structures should adopt a probabilistic approach to account for the variability in the influencing parameters.

(v) The formulation of optimal proactive maintenance and repair strategies for corrosion-damaged RC structures is highly dependent on results of prediction models. Therefore, limit states used to denote acceptable corrosion-induced damage should be easily quantifiable and representative of the actual state of the structure with respect to its structural and durability performances.

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