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EXTENT OF CORROSION DAMAGE FOR RC STRUCTURES EXPOSED TO CHLORIDE-BEARING ENVIRONMENT

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Abstract. In industrialized countries, most of the reinforced concrete (RC) structures and infrastructures have more than 40 years, since they were built around 1960 – 1980, and were designed for a service life of 50 years. Consequently, in the next years, the number of structures and infrastructures that will need to be repaired will steeply increase. Several techniques, characterized by a different durability, environmental impact and economic impact, are available for the repair of a reinforced concrete structure damaged by the corrosion of reinforcement. To help the designers in the choice of the most suitable one, a preliminary assessment of the condition of the structure is essential, aimed at diagnosing the causes of deterioration, the extent of damage, i.e. the extent of corroding reinforcement, and its evolution in time. An approach, with a wide consensus, for the evaluation of the extent of corroding reinforcement is available for carbonation-induced corrosion, whilst it is still lacking for chloride-induced corrosion. In this paper two approaches for the evaluation of the extent of corroding reinforcement for a RC structure subject to chloride-induced corrosion are presented and compared, showing similar results. These approaches can be useful to properly plan a restoration intervention as well as to assess the reliability of the recently proposed model for service life design.

Keywords: Assessment, corrosion, chloride.

1. Introduction

In industrialized countries, most of the reinforced concrete (RC) structures and infrastructures, were built around 1960 – 1980 and, in general, were designed to last for a period of 50 – 100 years [1]. More and more of these structures are approaching the end of their designed service life, and due to their exposure conditions, i.e. typical urban or rural environment or marine exposure, often suffer damage due to carbonation- or chloride- induced corrosion of reinforcement, such as concrete spalling or cracking or rust stains that may affect their serviceability. A proper maintenance and rehabilitation of reinforced concrete structures could extend their service life, by restoring the structural safety and preventing the future damage. This would avoid to demolish them and build new ones, leading to a lower social, environmental and economic impact, due, for instance, to a less hindrance to traffic and waste of materials [2].

Several techniques are available for the rehabilitation, which vary from keeping the conditions of corrosion and serviceability under control by some form of monitoring, when the extent of damage is limited or the remaining period of use is short, to the conventional repair technique, which requires the replacement of carbonated concrete cover or chloride contaminated cover with a cementitious mortar, to electrochemical techniques or hydrophobic treatments, that can control corrosion of steel bars without requiring the removal of the original concrete cover, except the cracked concrete [3–5]. Each of these strategies is characterizes by a different durability, environmental impact and economic impact, and the choice of the most suitable protection system should be a compromise among the different requirements [4].

Hence, it appears essential to choose and plan the most suitable restoration intervention, a careful condition investigation that cannot be limited to a visual inspection by an expert, but should include, beside the visual inspection and the examination of design documents, field tests as well as sampling for laboratory tests, i.e. carbonation depth, chloride profile and cover depth. The inspection should be aimed at estimating if, both at the time of inspection and at the end of the designed lifetime, the reinforcement is still passive, i.e. corrosion has not initiated since carbonation or a critical amount of chlorides has not reached the steel surface, or the reinforcement is corroding but the propagation is in the early stages, e.g. concrete cover is not cracked and reduction in cross section of rebars is negligible [6]. Since concrete quality, mainly the permeability, nature and intensity of cracks, and concrete cover are spatially variable over the concrete surface, due to different concrete batches and the heterogeneity of workmanship, the corrosion conditions will vary accordingly [7–9]. An evaluation of the extent of corroding reinforcement, i.e. of the amount of steel bars that are no more longer passive, and its evolution in time, is, then, needed. For carbonation-induced corrosion the extent of corroding reinforcement can be assessed by comparing measured carbonation depth with measured cover depths

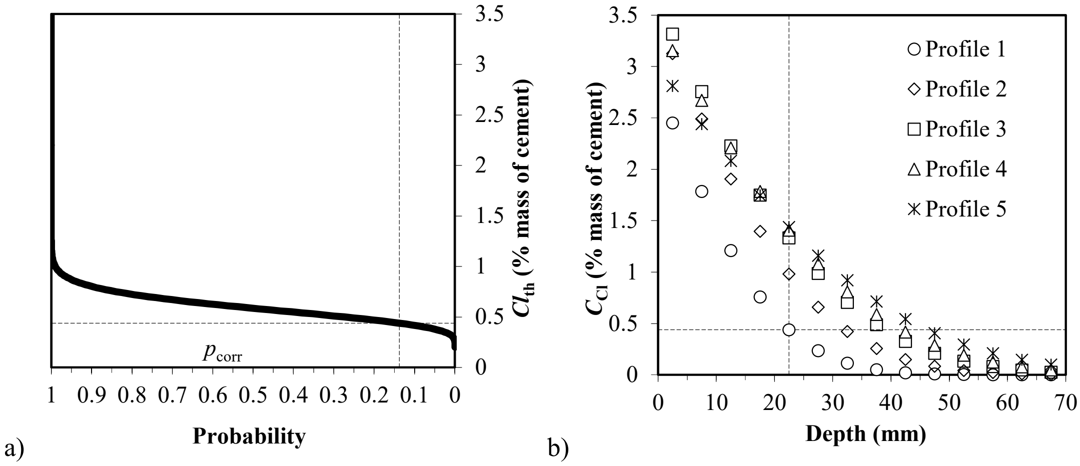

FIGURE 1. Cumulative distribution function of the critical chloride content (a) and imaginary chloride profiles measured on a RC structure (b).

in the structure and in the literature a statistical approach with a wide consensus was proposed for its evaluation [10]. For chloride-induced corrosion, to evaluate the extent of corroding reinforcement, the chloride content at the depth of the bars needs to be compared to the critical chloride content, since corrosion occurs when at the bar depth a chloride content equal to the critical chloride content is reached. The variability of these parameters makes it difficult to assess the extent of corroding reinforcement and, unfortunately, an approach with a wide consensus is still lacking.

This paper presents two mathematical approaches for the evaluation of the extent of corroding reinforcement for a reinforced concrete structure subjected to chloride-induced corrosion. A fictitious example is provided to compare the results of the two approaches and to discuss their real application.

2. MATHEMATICAL APPROACHES FOR THE EVALUATION OF THE EXTENT OF CORRODING REINFORCEMENT

2.1. STATISTICAL APPROACH

In order to assess the extent of corroding reinforcement, R_{dep}, of an existing structure, the critical chloride content, Cl_{th}, the chloride content, C_{Cl}, and the depth of the bars, c, need to be known. The critical chloride content depends on several factors, and, due to its stochastic nature, can be properly defined only through a probability density function. According to the Model Code for Service Life Design, proposed in 2006 by the International Federation of Concrete, fib [11], Cl_{th} for ordinary carbon steel bars, can be described by a Beta Distribution, lower and upper limited. Figure 1a shows the cumulative distribution function of the critical chloride content, assuming that the average value and the standard deviation are respectively 0.6 % and 0.15 % vs mass of cement (values of 0.2 and 2 % vs mass of cement have been assumed for the lower and upper limits).

The chloride content at the different depths and the concrete cover thickness can be determined during the inspection of the RC structure, through field and laboratory tests. In order to assess R_{dep}, and to increase its reliability and accuracy, a significant number of measurements of these parameters is required. As an example, Figure 1b shows imaginary chloride profiles measured on five concrete samples taken from the structure after 10 years from the construction. In the example the total chloride content was evaluated at depth intervals, Δc, of 5 mm. From the concrete cover depth measurements, carried out *in situ*, a frequency analysis can be performed (the depth intervals should be equal to those of the chloride profiles) (Figure 2).

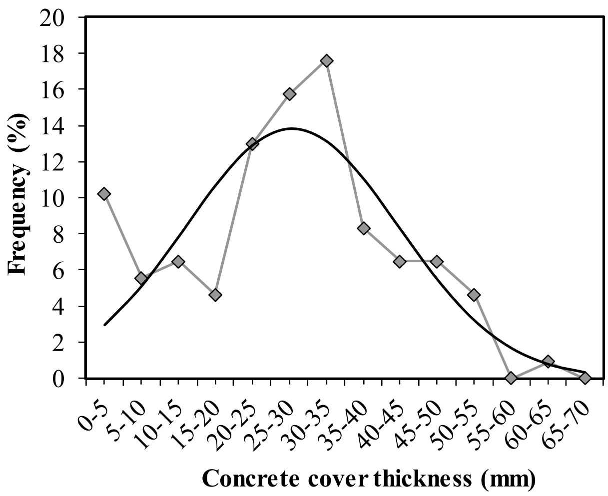

FIGURE 2. Imaginary frequency analysis (grey symbols) and probability density function (black line) of the concrete cover thickness.

In order to evaluate the extent of the depassivated bars, initially the variability of the concrete cover

Profile	C_{cl} (% mass of cement)	$p_{corr,\,i}$
1	0.44	0.14
2	0.98	0.99
3	1.33	1
4	1.41	1
5	1.44	1

Table 1. Chloride contents at the depth $c = 22.5$ mm and calculated values of $p_{corr,\,i}$.

Δc	[mm]	$0-5$	$5-10$	$10-15$	$15-20$	$20-25$	$25-30$	$30-35$	$35-\ldots$
f_c	[%]	10.2	5.6	6.5	4.6	13	15.7	17.6	26.9
p_{corr}	[%]	100	100	99.9	97	82.5	73.2	55	41
R_{dep}	[%]	10.2	5.6	6.5	4.5	10.7	11.5	9.6	3.2

Table 2. Proportion, f_c, of imaginary cover depth measurements falling within the range Δc, probability of occurrence of corrosion, pcorr, and extent of corroding reinforcement, R_{dep}, in each depth Δc.

thickness, c, is neglected and it is considered that the bars have the same concrete cover (i.e. the concrete cover of all the bars falls within the same cover depth interval, Δc). Moreover, it is initially supposed that, at the bars depth, c, a unique value of the chloride content, C_{cl}, is available. The probability that corrosion initiated, p_{corr}, corresponds to the probability of occurrence that the variable Cl_{th} is equal to the considered value of chloride content, C_{cl}. The probability that corrosion occurred is an estimation of the extent of corroding reinforcement. This means that, if at the bars depth a chloride content lower of 0.2 % mass of cement is measured (that corresponds to the lower limit of the Beta distribution), p_{corr} should be null and hence all the bars would be passive, conversely, if the chloride content were higher than 2 % mass of cement (that corresponds to the upper limit of the Beta distribution), p_{corr} would be equal to 100 % and, hence, all the bars would be actively corroding. As an example, a concrete cover thickness equal to 22.5 mm (as indicated by the vertical dotted line in Figure 1b) and the first profile, i.e. profile 1, can be considered; at this depth a chloride content of about 0.44 % mass of cement can be evaluated (horizontal dotted line in Figure 1b). At this chloride content corresponds a probability p_{corr} ($= R_{dep}$) equal to 14 %, given by the intersection between the horizontal dotted line and the Cl_{th} cumulative distribution function, as shown in Figure 1a.

Considering that more measurements of chloride content at the bars depth, c, are available (as in the example of Figure 1b), p_{corr} can be evaluated through the equation:

$$p_{corr} = \frac{\sum_{i=1}^{n} p_{corr,\,i}}{n} \tag{1}$$

Where n is the number of chloride profiles and i represents the $i-$th profile.

Table 1 reports the chloride contents at the concrete cover $c = 22.5$ mm, shown in Figure 1b, and $p_{corr,\,i}$ evaluated for each profile. Applying equation 1 it can be evaluated that, for this depth interval, p_{corr} is about 83 %.

However, in a real structure, it has to be taken into account also the variability of the concrete cover thickness, as shown for instance in Figure 2 and reported in Table 2 in terms of percentage of measurements, f_c, falling within different depth intervals. Hence, p_{corr} has to be calculated, through equation (1), for all the concrete cover depth intervals, Δc, as shown in Table 2.

$R_{dep,\,j}$ can be, then, evaluated, for each Δc, as:

$$R_{dep,\,j} = p_{corr,\,j} \cdot f_c \tag{2}$$

Where f_c is the proportion of cover depth measurements falling within the range Δc and j is equal to the given interval depth, Δc.

Hence, the extent of corroding reinforcement can be calculated by summing up corrosion proportions over all the depth intervals:

$$R_{dep} = \sum_{j=1}^{m} \left[f_{c,\,j} \cdot \left(\sum_{i=1}^{n} \frac{p_{corr,\,i}}{n} \right)_j \right] \tag{3}$$

Where n is the number of chloride profiles and i represents the $i-$th profile, m is the number of cover depth intervals, Δc, and j represents the $j-$th depth.

By applying equation 3 to the given example, a value of R_{dep} about 62 % can be evaluated.

2.2. Probabilistic approach

To evaluate the extent of corroding reinforcement, a probabilistic approach, similar to that used for the design of new RC structures, can be also used. In the probabilistic approach for the design of new structures, the extent of corroding reinforcement corresponds to the probability, p_{dep}, that the limit state

Profile		1	2	3	4	5
D_{app}	$[10-12\,\mathrm{m}^2/\mathrm{s}]$	0.4	0.7	1	1.2	1.6
C_s	[% mass of cement]	2.8	3.45	3.6	3.4	3

Table 3. Diffusion Coefficient, D_{app}, and surface chloride concentration, C_s, of imaginary chloride profiles.

Parameter	Unit	Type of distribution	Value	
			m	σ
D_{app}	$[10-12\,\mathrm{m}^2/\mathrm{s}]$	normal	0.98	0.46
C_s	[% mass of cement]	normal	3.25	0.33
C	[mm]	normal	27.9	14.3
Cl_{th}	[% mass of cement]	beta	0.6	0.15
T	[year]	-	10	-

Table 4. Values of the parameters and types of statistical distribution used as inputs in the probabilistic approach (m = mean value, σ = standard deviation).

equation for the initiation of corrosion, G, is not satisfied, i.e. that $p_{dep}\{G\} < 0$. For chloride-induced corrosion, the limit state equation for the initiation of corrosion can be described by means the solution of Fick's second law:

$$\begin{aligned} p_{dep} &= \{Cl_{th} - C_{cl}(x,t)\} \\ &= p\left\{Cl_{th} - C_s\left[1 - \operatorname{erf}\frac{c}{2\sqrt{D_{app}\,t}}\right]\right\} < 0 \end{aligned} \tag{4}$$

where: Cl_{th} is the critical chloride threshold, C_s is the surface chloride concentration, c is the concrete cover depth, D_{app} is the diffusion coefficient and t is the time. The variables Cl_{th}, C_s, c and D_{app} are stochastic variables, whilst the time, t, is a deterministic variable.

To evaluate the actual corrosion conditions of an existing structure, reliable values for the input parameters need to be provided. As far as the time, t, is concerned, it corresponds to the time between the year of construction and the considered time (in the example, 10 years). The critical chloride threshold, as previously observed, can be described for carbon steel bars through a Beta distribution, with a mean value of 0.6 % mass of cement. The other parameters are related to the considered existing structure and to its exposure conditions and can be determined, if results of a detailed inspection are available, from the experimental measurements. The concrete cover distribution can be evaluated by interpolating the experimental measurements and determining the distribution parameters, i.e. the mean value and the standard deviation. From the imaginary concrete cover thickness shown in Figure 2, a normal distribution, with a mean value of 27.9 mm and a standard deviation of 14.3 mm can be determined (black line in Figure 2).

The interpolation of the chloride content at different depths, at time, t, by means of the second Fick's Law, allows to calculate, for each profile, a value of D_{app} and C_s. From the chloride profiles shown in Figure 1b, data summarised in Table 3 can be evaluated. From these values, both for the diffusion coefficient and the surface chloride concentration, the probability density function, with its characterizing parameters, can be determined. Assuming for both parameters a normal distribution, for the diffusion coefficient a mean value and a standard deviation respectively equal to 0.98 and $0.46 \times 10^{-12}\,\mathrm{m}^2/\mathrm{s}$ can be calculated, whilst for the surface chloride concentration a mean value and a standard deviation equal to 3.25 % and 0.33 % mass of cement can be determined. Table 4 summarizes the values of the parameters involved in equation 4, to be applied for the evaluation of the extent of corroding reinforcement of the example. Solving equation 4 through a probabilistic technique, as the Montecarlo method, with the values presented in Table 4, a p_{dep} equal to 61 % can be evaluated.

3. Discussion

Both mathematical approaches allow to evaluate the extent of corroding reinforcement i.e. the percentage of reinforcement likely to be non-passive, for a reinforced concrete structure subjected to chloride-induced corrosion, by comparing the chloride content at the depth of the reinforcement, measured during an inspection, with the critical chloride threshold, taking into account the variability of each involved parameter. The extent of corroding reinforcement could be evaluated both at the time of the inspection and in the future, by estimating the evolution in time of the chloride content at different depths. The approaches provide only indications on the amount of actively corroding bars, but do not give any indication on their localization. Furthermore they might overestimate the percentage of bars where initiation of corrosion really occurred. Because of the mechanism of chloride-induced corrosion, in fact, in the surrounding zones of corroding bars that usually have a chloride content higher than the chloride threshold, the steel benefits of the formation of macrocell that provides cathodic polarization, preventing the initiation of corrosion [3].

The statistical approach appears to be easily applicable since the punctual value of the chloride content is directly compared with the critical chloride threshold, whilst the probabilistic approach requires the elaboration of the data obtained through experimental measurements, i.e. the concrete cover thickness and the chloride profile, to determine the probability density function of the input parameters. However, in principle, the probabilistic method could be used for a preliminary estimation of the extent of corroding reinforcement even in absence of any experimental data derived from an inspection. Indeed, the input parameters involved in equation 4 could be determined from the design data and the environmental exposure conditions. As concrete cover thickness, the design concrete cover could be considered, whilst from the information related to the construction materials, i.e. the w/b ratio and the type of cement, and the exposure condition, the diffusion coefficient and the surface chloride concentration could be estimated. Of course, in this case, the reliability of outcomes will depend on the uncertainty in the parameters chosen for the calculations.

From their application, values of 62 and 61 % of extent of corroding reinforcement were respectively obtained with the statistical approach and the probabilistic approach, suggesting that the two approaches are comparable. The accuracy and reliability of the outcomes of both approaches depends on the available number of experimental measurements of concrete cover thickness and chloride content at different depths and their representativeness of the whole structure. The reliability of these methods could be validated by comparing their results with a careful corrosion potential mapping that allows to detect the depassivated area.

The availability of a methodology that allows to evaluate the percentage of corroding reinforcement, provided that it is representative of the corrosion conditions of the whole structure, could help the designer in the choice of the most suitable repair technique. The percentage of depassivated area and its evolution in time will represent the amount of concrete, even that sound, that must be removed in the conventional repair, since the concrete must be removed in all areas where the chloride threshold is reached at the depth of the reinforcement or it is expected to be reached during the design life of the repair. This repair technique, that also requires the careful cleaning of the surface of the reinforcement to remove all chloride contaminated rust, appears to be suitable when the amount of concrete to be removed and replaced is limited. Conversely when large amount of sound concrete should be removed, i.e. when the damage is extensive, other repair methods, such as cathodic protection or electrochemical chloride removal, that require the removal of the only cracked concrete, may be considered.

Finally such kind of approaches can be also useful in validating the outcome of the recently proposed probabilistic models for service life design as, for instance, the fib Model Code for Service life design. In these models the probability of failure, i.e. the probability of occurrence of corrosion, is evaluated as the probability that the initiation limit state function reaches negative values and, in case of chloride-induced corrosion, it is defined as the difference between the critical chloride threshold and the chloride content at the depth of the bars. The probability of failure has the same meaning of the percentage of corroding reinforcement. Hence, the application of the models for service life design to existing structures and the comparison of the results to the real corrosion conditions of these structures might allow a preliminary understanding of their reliability. Usually in these models the concrete performances in relation to the resistance to chloride penetration are evaluated through accelerated tests. The results of accelerated tests are then modified through a series of corrective factors that are directly provided by the models themselves. These accelerated tests were usually not carried out for structures built in the past and, hence, to apply these models to an existing structure, attention should be paid in their definition. An inaccurate estimation of the parameters chosen for the calculation could compromise the comparison with inspection results.

4. Conclusive remarks

This paper presented two approaches to evaluate the extent of corroding reinforcement i.e. the percentage of reinforcement likely to be non-passive, of a reinforced concrete structure exposed to chloride-induced corrosion, from the results of an inspection, provided that its results are representative of the whole structure. In the statistical approach the extent of corroding reinforcement is calculated by summing up corrosion proportions that correspond to the probability that the critical chloride content is equal to the chloride content at a certain depth interval, over all the considered depth intervals. In the probabilistic approach the extent of corroding reinforcement, corresponds to the probability that the difference between the critical chloride threshold and the chloride content at the depth of the bars, both considered as stochastic parameters, reaches negative values. Their application to a fictitious example showed similar results, suggesting that the two approaches are comparable.

These approaches can be useful to properly plan a restoration intervention, since the outcomes represent the amount of concrete, even sound, that must be removed in the conventional repair, providing then indications on the suitability of such kind of repair technique rather than other treatments, such as the cathodic protection or the electrochemical removal of chlorides, that require the removal of the only cracked concrete.

Finally, these approaches can be used to assess the reliability of the recent proposed models for service life design, by comparing the outcomes of their application to existing structures with the results to the real corrosion condition of these structures.

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