ADVANCES IN DESIGN AND RESIDUAL LIFE CALCULATION WITH REGARD TO REBAR CORROSION OF REINFORCED CONCRETE

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ABSTRACT: The increasing amount of structures presenting distress due to reinforcement corrosion is urging the establishment of more accurate calculation methods for the service life of concrete structures. In the present paper, a summary of the different approaches is presented that are able to calculate the expected life of new structures, in certain aggressive environments or the residual life of already corroding structures. The methods for the initiation period are based on the proper calculation of the carbonation front or chloride penetration and on the steel corrosion rate. The methods for the residual load-bearing capacity calculations are based in the use of "indicators" or in the evaluation of the reduced section and a structural recalculation.

KEYWORDS: Service life prediction, reinforced concrete, durability design, residual life, corrosion.

INTRODUCTION

When reinforced concrete started to be industrialised, the pioneers were convinced to have found a material with unlimited durability, as concrete supposes a chemical protection for the steel in addition to providing the rebar with a physical barrier against contact with the atmosphere.

However, in spite of the good performance noticed over this century it is also evident that the amount of concrete structures presenting insufficient durability is increasing. The deterioration can be due to the chemical and physical attack to concrete or to the corrosion of reinforcement. Concrete deterioration can be prevented by using appropriate mix proportions and by selecting the cement type, which is relatively inexpensive. However reinforcement corrosion, being the main distress observed results of much higher economical investment.

Therefore, durability studies and service life calculation is becoming an area of increasing interest. Standards and Codes still do not incorporate sufficiently practical rules to assure the needed service life in aggressive environments and the design of new concrete structures is still based on the traditional methodology of deemed to satisfy rules, fixing limiting values for one or several concrete requirements: concrete grade, minimum cement content, maximum water/cement ratio, air content, cover thickness, and maximum structural crack width.

This approach for providing certain durability has demonstrated to be insufficient and new methods have to be developed in order to predict and quantify the structural service life. In present paper is addressed the definition of service life with regard to corrosion of reinforcements and mention is made to some models of calculating the service life as well as mention is made to the structural consequences of corrosion

Service life modelling

The most well known service life model for concrete reinforcements is that of Tuutti (1) In it, he initiation period comprises the time taken by the aggressive (chlorides or the carbonation front) to reach the reinforcement and depassivate the steel. It is known as the period of propagation from the reinforcement starts to corrode. This corrosion may produce the collapse of the structure and therefore from a certain level of deterioration repair has to be undertaken. This apparently simple model has been refined in several during the past years (2-4). At present the main steps to calculate the service life of reinforced structures are:

1) Definition of the length of the service life and of the limit state of durability,
2) Identification of aggressivity of the environment,
3) Calculation of attack progression,
4) Establishment of concrete specifications and supplementary protection methods, if necessary.

1. Length of Service Life

Service Life is defined as the period in which the structure keeps its security, functionality and aesthetics without unexpected costs of maintenance. Typical lengths considered are of 50 years for normal buildings and 100 years for bridges and other important structures whose replacement is costly. Other structures have shorter nominal services lives (nuclear plants, industrial installations as might be those of the agriculture, temporal buildings,...) when they are located in very aggressive environments or the evolution of the technique or the change in use recommends lives of 25 or 30 years. The establishment of the service life period has technical supposes for the designer not only an economical reference but it has legal implications due to the professional responsibilities that are involved if the structure does not achieve the nominal service life. Therefore, it has to be defined as a period for the calculation of the durability and not a period in which the professional responsibilities hold.

The definition implies as well that of which the state that means the failure of the expected service life is. That is, a *limit state*, has to be defined on order to identify when the design requirements are not fulfilled and major repairs are necessary. Figure 1 shows the several limit states that can be considered in the case of reinforcement corrosion (5,6).

The verification of the limit state lies in the fulfillment of the expression: $R \geq S$, being R=

to the property related to the resistance against the attack and S the property related to the environmental aggressivity. The limit state can also be verified by the expression: $t_L \geq t_d$ where t_L is the time taken by the agressivity to reach the limit state and t_d is the target service life. Assuming a certain propagation of the corrosion : $t_L = t_i + t_p$.

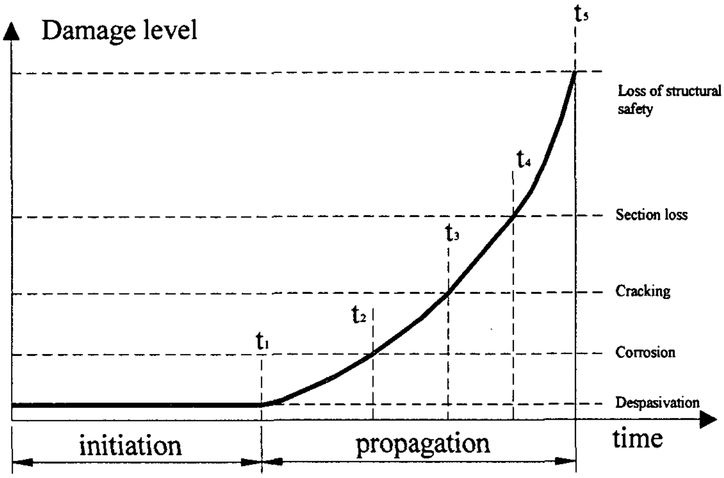

Figure 1- Service life model and limit states related to reinforcement corrosion.

$$t_L = service\ \ life = t_i + t_p = K_c\sqrt{t} + \frac{PL}{CR} \qquad (1)$$

where: t_i=corrosion initiation time, years, t_p=corrosion propagation time in years, V_c= carbonation or chloride penetration rate, PL= Penetration limit, mm, CR = Corrosion rate, µm/year

2. *Identification of aggressivity of the Environment*

Environmental (climatic and chemical mostly) actions are the responsible of the shortening of the durability of reinforced concrete. In general, no significant damage is noticed in dry indoor conditions, although indoor environment may be very different depending upon heating regimes or external climate.

In outdoor environment, the main aggressive agents in relation to steel corrosion are: carbon dioxide (CO_2) concentration, chloride (Cl^-) proportion and RH-T° (humidity-temperature) cycling. Proper values of CO_2, and Cl^- are needed in order to be introduced into the mathematical expressions that will be described later. RH-T° cycling are also very relevant as they fixe the concrete moisture content. A certain amount of evaporable water is necessary for any degradation process to progress.

The environmental aggressivity is usually defined by classifications that are given in the Standards or in Codes (7,8). They are formulated in the form of tables.

3. *Calculation of attack progression*

Two aspects have to be considered: 1) The identification of the mechanism of the attack and the estimation of the arte of advance of the damage. Regarding **mechanisms of attack**, there are two main processes: carbonation and chloride penetration, by which the aggressive agents may penetrate into concrete.

Carbonation usually progresses by a diffusion mechanism while chlorides may penetrate also with a combination of absorption and diffusion (tidal or splash zones). Both are known to follow the law of the "square root of time":

$$x = V\sqrt{t} \tag{2}$$

where: x = attack penetration depth, mm, t = time, s, and, V= constant depending on concrete and ambient characteristics

This general expression can be formulated in several manners by means of more sophisticated models. As examples can be mentioned those of Tuutti (1), Bakker (9) and Parrott (10). The three models have as input factors the concrete requirements and climatic (humidity) loads. As an example equation 2 shows the expression due to Tuutti (1):

$$\frac{C_s}{C_x} = \sqrt{\pi}\,[\frac{x/\sqrt{t}}{2\sqrt{D}}]\exp[\frac{x^2}{4Dt}]\ erf(\frac{x/\sqrt{t}}{2\sqrt{D}}) \tag{3}$$

C_s= CO_2 concentration in the atmosphere, mol/kg
C_x = amount of bound CO_2 (cement phases plus pore solution), mol/kg,
D = CO_2 diffusion coefficient, m^2/s
x = Carbonation depth, mm, and
t = time, s

When properly solved, the different models may give very similar results. The choice of preference depends on the available input data.

In the case of **chlorides** the most used mathematical model is in second Fick's law for the case of semi-infinite medium (1).

$$C_x = C_s(1 - erf\frac{x}{2\sqrt{Dt}}) \tag{4}$$

where C_s= surface chloride concentration, %, C_x= proportion of chlorides at a certain depth, %, D= chloride diffusion coeficient, m^2/s, x= depth of penetration, m, t=time, s, In spite of the wide use of this model, it has numerous limitations, that can be summarized as follows:

a) The C_s is not always a constant as it may increase with time
b) if C_s is not constant, the D value cannot be used for comparative purposes as it depends on the C_x/C_s ratio.
c) The D value is not a constant. It changes with the proportion of chlorides and time.
d) No absorption period effect is considered in this model.
e) It has not been related D to the concrete mix proportions.

All these shortcomings limit the use of the model for predictive purposes as the D value should be established in previous laboratory experiments. The trials undertaken until now indicate that D values obtained in young concrete are much higher than values obtained in cores taken from old real structures [11]. Other more comprehensive and sophisticated models, [12] as well as accelerated methods based on the application of an electrical field [13], are being now experimented.

4. Concrete requirements

The availability of refined calculation methods provide the possibility of trying different concrete qualities in order to obtain the same durability. This is known as a "performance based approach" dealing to the use of different concrete dosifications. The different concrete qualities should be linked to the cover thckness. As an example, to avoid depassivation due to carbonation during 50 years with a concrete cover of 30 mm; the V_c would be: ~2.3 mm/year$^{0.5}$. This penetration rate can be achieved with different cement types, w/c ratios and proportions of cement.

On the other hand, the environmental aggressivity will play as well an important role, as it conditions the rate of advance of the aggreesive substances from the exterior of the concrete The final aim of using refined methods is the adequate selection of concrete characteristics and of minimum cover for a particular environment. At this respect it would be an inetresting advance to hacve a comprehensive parameter of easy measurement for the quality control and verification during the concrete fabrication of its expentacies regarding durebility. The concret parameter proposed with this purpose is the electrical resistivity (14).

Supplementary Protection Methods

In very aggressive environments it may happen that concrete cover itself is not sufficient protection to the designed life time. Then, at the design phase it will be necessary to define the supplementary protection methods to provide the structure with the adequate durability. The main additional protection methods are: 1) cathodic protection, 2) galvanising, 3) stainless steel rebars, 4) epoxy coated rebars, 5) corrosion inhibitors, 6) concrete coatings. The description of these protection methods is out of the scope of the present paper although Figure 2 summarises their main features.

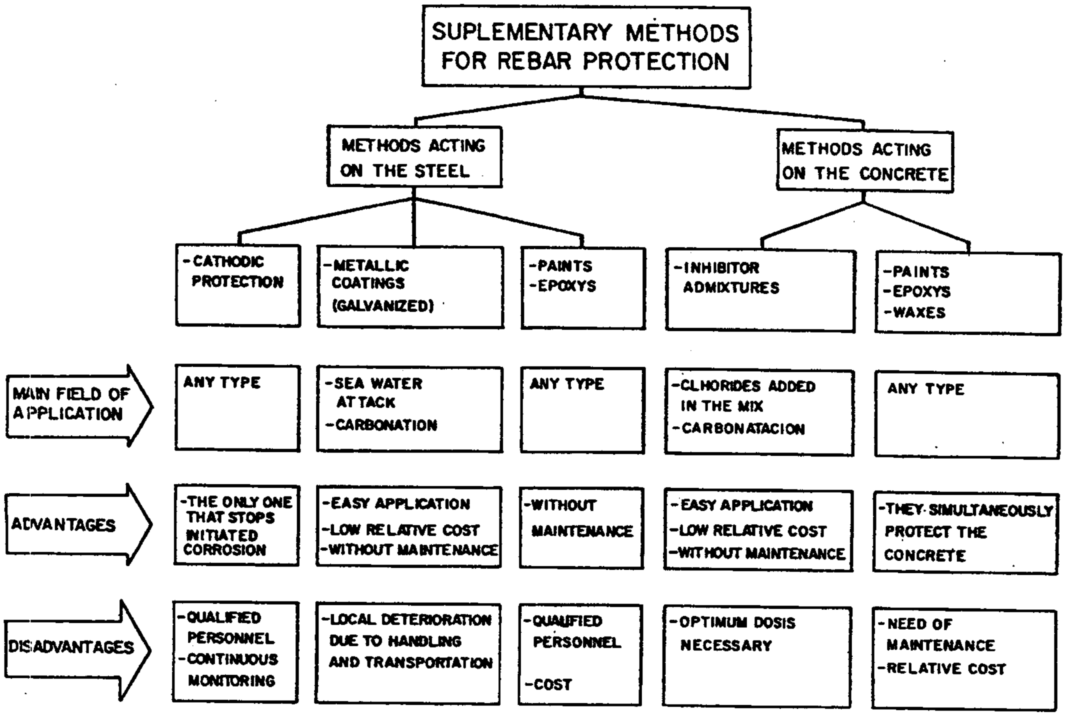

FIG. 2. Summary of supplementary methods for rebar protection.

RESIDUAL LIFE PREDICTION OF STRUCTURES SUFFERING CORROSION

The main consequences of corrosion regarding the structural behaviuor shown in figure 3 are (15):

- In the reinforcement: 1) loss of load-bearing section (5) and 2) loss of ductility (15).
- In the concrete interface and the cover: 3) loss in bar/concrete bond (16) and 4) concrete cracking (17).

Several efforts have been made during last years to quantify these effects. Examples are the FIB bulletin no. 243 (1) or the Swedish Road Administration Manual (2). The simplified procedure here proposed, has been developed in the BRITE – EURAM project BE – 4062. Other parteners in this project have been: BCA (UK), Lund University (Sweden) Cementa (Sweden) and CBI (Sweden)

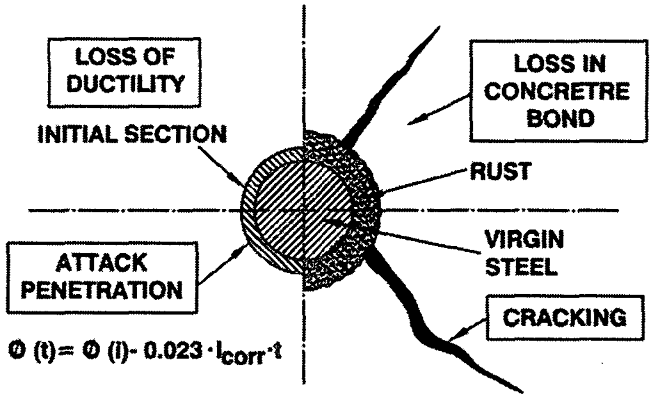

FIG. 3. Consequences of rebar corrosion.

These immediate consequences of the corrosion will affect the load-bearing structural capacity in different ways, which are still under study. Two are the main approchaes to deal with the structural assessment:

1) That used at present, based in the experience and rough evaluation of the structural load-carrying capacity, which may be labelled "simplified method".

2) That, which in opposition, may be named "detailed method". It is based in the establishment of the reduced load-bearing section or geometry and the verfication of the serviceability and ultimate limit states with the expected corrosion rate.

Simplified Method of Residual Life Assessment

The needing of adequate tools for a fast, economic and safe enough assessment of a structure is shown by an increase in the structural management programs developed last years.

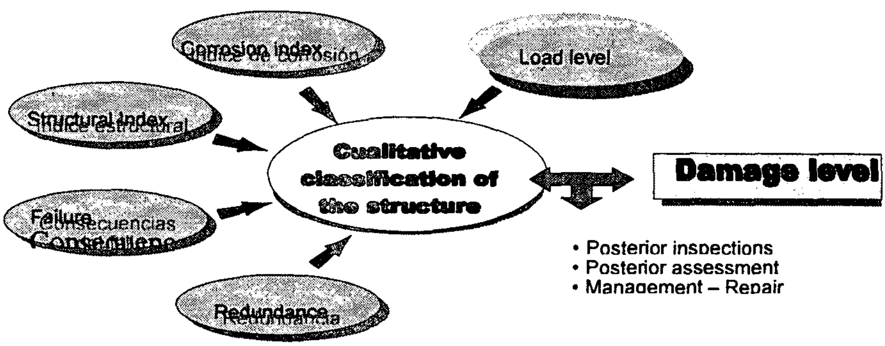

Figure 4. Simplified assessment general schema.

As it is shown in figure 4, the simplified assessment developed is an empirical procedure based on the application of several indexes, experimentally developed, that can reflect the principal aspects involved in the residual life calculation. These indexes are:

- Corrosion Index (deterioration mechanism evolution): It tries to represent the damage level caused by corrosion on reinforcement. This index is related to the measurement of corrosion rate or corrosion current in the structure and the type of corrosion (generalised or localised). In addition, visual inspection can provide an external damage classification in different levels. These indexes can vary from *Negligible* (when no external damage are detected or small rust spots are presented) to *High* (when a generalised *spalling* and cracking are detected).

- Structural index: Their function is to take into account the influence of corrosion in the structural typology studied. This parameter depends on the type of the element (beam or column) and the geometrical and mechanical characteristics of the element (longitudinal reinforcement, transversal reinforcement, size, general load level). For bending moments figure 5 shows the source of the Structural index.

In addition, visual inspection can provide an external damage classification in different levels, as is shown in table 1. These indexes can vary from *Negligible* (when no external damage are detected or small rust spots are presented) to *High* (when a generalised *spalling* and cracking are detected).

Table 1. Visual damage rating

External damage	Description
Negligible	No external damage, few external rust spots.
Low	Small damages (crack width < 0.3 mm)
Moderate	Crack width > 0.3 mm following reinforcing map
High	Generalised cracking and spallig.

Source: BE-4062 (1995)

In column elements, the proposed procedure is similar to that given for beams. A transversal reinforcement index is obtained, function of the rebar diameter and the spacing. This value tries to characterise the possibility of buckling in the longitudinal bars. A second factor, function of the cover/column size ratio represents the loss of bearing capacity due to *spalling*. In addition, it has to be taken into account:

- Failure consequences, determined through the importance of the structure or their risk of victims.

- In the case of assessment of a part of a whole structure, the hyperstaticity is considered in the final value of the Damage level.

- Also, it is necessary to take into account the actual load level of the element, due to possible existence of oversize elements.

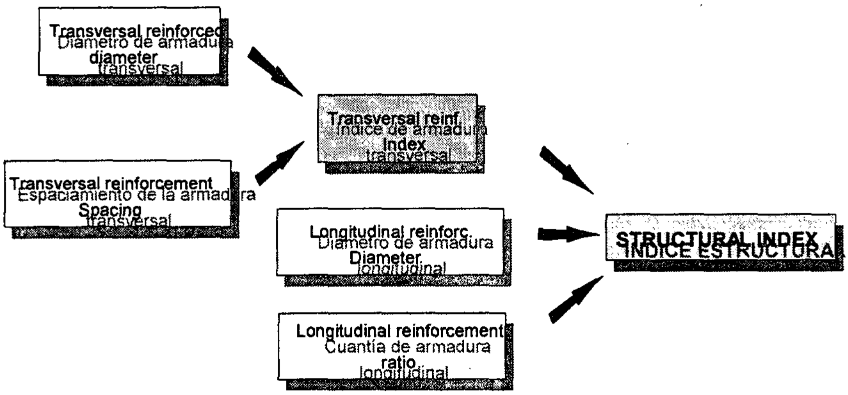

Figure 5. Structural Index procedure for beams.

Table 2. Damage classification and urgency of intervention in years.

External Damage	**Damage Level**			
	Negligible	Medium	Severe	Very Severe
Negligible	> 10	6-10	4-6	
Low	6-10	4-6	2-4	1-2
Moderate	4-6	2-4	1-2	1-2
High		1-2	0-1	0-1

These indexes (corrosion and structural) are weighed adequately in order to obtain a Damage Level of the whole structure, which is ranked into *Negligible* , *Medium* , *Severe* , *Very Severe,* leading to the definition of the "urgency of intervention" in years.

Detailed Method of Residual Life Prediction

The main objective is residual safety level determination, in order to establish and adequate intervention program with a high degree of information available. On the other hand, it can be also used for the calibration procedure of the indexes above presented. Three are the main aspects to be analysed in a structural assessment. The two last aspects will be extended below for structures affected by reinforcement corrosion:

- Action, or better action effect, evaluated on the structure.
- Deterioration process evaluation.
- Safety and serviceability limit states verification.

The attack by corrosion will be appraised by using a *penetration attack,* **Px**, which is the loss of reinforcement radius. **Px** is the main parameter that will allow a correlation with the general effects previously mentioned on the composite section concrete. This parameter can be measured visually from the residual diameter or estimated by means the corrosion rate $\mathbf{I_{corr}}$ **Px** is related with $\mathbf{I_{corr}}$ through the expression :

$$P_X[mm] = 0{,}0116\,\alpha\, I_{corr} t \tag{5}$$

where t is the time since the corrosion started α is the pitting factor which takes into account the type of corrosion (homogeneous or localised).

The corrosion rate is the amount of metal lost in area and time units (g/cm^2 year or μA/cm^2 or μm/year). This progressive loss can be measured currently by means of an electrochemical method based on the Polarisation Resistance, R_p, technique which is non-destructive. Measurements carried out on real structures (15) confirm previous findings in the laboratory and enable the determination of the normal values of corrosion rates: values smaller than 1μm/year mean negligible corrosion and higher than 10 μm/year have to be considered as high.

The reduction in steel cross section is calculated through expression (6) where α is the "pitting factor". The α values are different if the corrosion is homogeneous ($\alpha = 2$) than for pitting corrosion ($5 < \alpha < 10$).

$$\Phi = \Phi_0 - \alpha P_x \tag{6}$$

Cover cracking (18,19). The oxides generated in the corrosion process provoke an tensional state in the concrete cover that will produce final cracks, reducing consequently the cross section of the concrete element and therefore their load bearing capacity. Several empirical expressions have been developed, that can evaluate the crack width of the cover, as a direct function of the corrosion attack **x** and several geometric and mechanical paramenters.

$$w = 0.05 + \beta\left[P_x - P_{x_0}\right] \qquad [w \leq 1.0\ \text{mm}] \tag{7}$$

Where:

- **w** is the crack width in mm,
- $\mathbf{P_x}$ is the attack penetration in microns.
- $\mathbf{P_{X0}}$ is an attack corresponding to the crack initiation and,
- β is a factor depending on the bar position. (table 3)

Table 3. β Values for crack width calculations

	Mean Values		**Characteristic Values**	
	Upper Bars	Lower Bars	Upper Bars	Lower Bars
β	0.0086	0.0104	0.01	0.0125

The value of $\mathbf{Px_0}$ can be estimated trough expression on table 4.

Table 4. $\mathbf{Px_0}$ Expressions for crack initiation

	Mean Values	**Characteristic Values**
$\mathbf{Px_0 = a + b_1\, c/\phi + b_2\, f_{c,sp}}$ **(3)**		
a	74.5	83.8
$\mathbf{b_1}$	7.3	7.4
$\mathbf{b_2}$	-17.4	-22.6

Where:

- $\mathbf{Px_0}$ is the attack in μm,
- $\mathbf{c/\phi}$ is the cover diameter ratio.

Loss of bond. (20) The concrete – steel bond is the responsible of the bar anchorage in the element ends and the composite behaviour of both elements. However, corrosion provokes a reduction in bond due to the cover cracking and stirrups corrosion. Finally a limit state of bond can be achieved. Three main aspects should be considered:

- *Residual bond assessment.* Table 5 shows empirical expressions obtained that allow to obtain realistic residual bond values. All of them are expressed depending on the attack penetration $\mathbf{P_x}$.

Table 5. Relationship between bond and P_x in mm

Values	Bond strength (MPa)	
	With stirrups	No stirrups
Mean values	$5.25 - 2.72\ P_x$	$3.00 - 4.76\ P_x$
Characteristic values	$4.75 - 4.64\ P_x$	$2.50 - 6.62\ P_x$

For intermediate cases where the amount of stirrups is low, bellow the actual minimum, or the stirrups capacity can be strongly reduced by corrosion effect, expressions of table 6 may be applied.

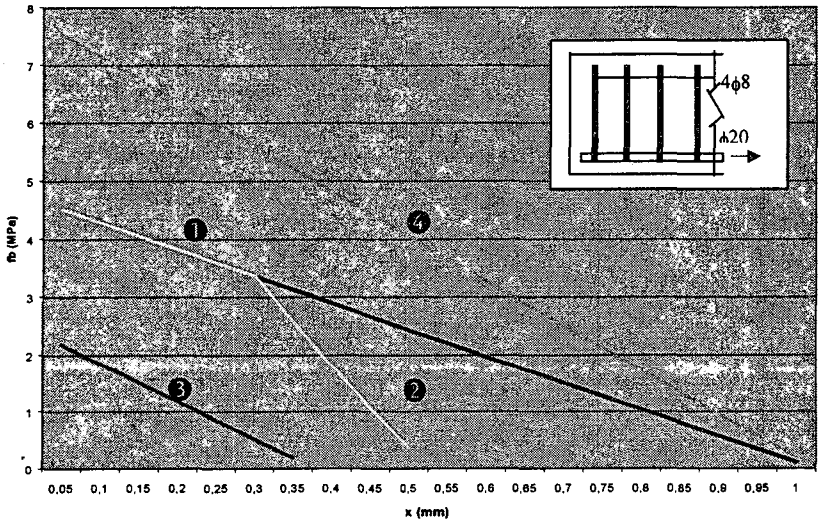

Figure 6. Residual bond as a function of $\mathbf{P_x}$

Table 6. Bond values for cases of intermediate amount of stirrups

	Mean Values	Characteristic Values
f_b	$8.25 + m(1.10{+}P_x)$	$10.04 + m(1.14{+}P_x)$
m	$-4.76 + 2.04(\rho/0.25)$	$-6.62 + 1.98(\rho/0.25)$
ρ	$n\ [(\phi_w - \alpha x)/\phi]^2$	$n\ [(\phi_w - \alpha x)/\phi]^2$

Where:
- ϕ is the initial longitudinal diameter in mm.
- ϕ_w is the transversal diameter in mm.
- **n** is the number of transversal reinforcements.

- α depends on the type of corrosion.
- f_b bond strength

These expressions are of application with P_x values between 0,05 and 1 mm with $\rho \leq 0.25$.

- *Influence of external pressures* that can be present due to external supports. In this case, expressions similar to that presented in Eurocode 2 have been developed by (5). These are:

$$f_b = (4.75 - 4.64\, P_x)/(1 - 0.098\, p) \tag{8}$$

Where f_b is the bond in MPa, P_x is the corrosion attack in mm y **p** the external pressure in the bond zone (MPa). This expression can be used for the bond evaluation of the rebar at element ends.

Figure 5 shows an application in a reinforcing bar diameter of 20mm without stirrups (curve 3) or with 4ϕ8 stirrups. Curves 1 and 2 correspond to the bond with a reduction at the end of the element without pressure (1 with homogeneous corrosion and 2 with pitting corrosion), curve 4 corresponds to an external force of 5 MPa.

- *Relationship between bond and crack width.* Several expressions have been developed for relating the residual bond with the crack width (table 7).

Table 7. Relationship bond f_b (MPa) and crack width w (mm).

	Stirrups	**No stirrups**
Mean values	f_b =18 - 0.52 w	f_b =3.19 - 1.06 w
Characteristic values	f_b =4.66 - 0.95 w	f_b =2.47 - 1.58 w

Limit state verification

Ultimate limit state

For slab and beams, a conservative value of the ultimate bending moment can be achieved by using the classical models but reducing the steel section and the concrete section spalled or cracked. A possible reduction due to bond deterioration must be considered, specially if the corrosion attack is on the tensile zone of the beams.

Although, shear and bending moment are supposed to have the same safety in the design design phase, for beams without corrosion the shear formulation is considered to be conservative, whereas for corroded ones, several factors may induce a premature fail of shear, such as:

- Small diameters on stirrups.
- Lower cover for stirrups.
- Spalling of cover.

In order to check the ultimate axial effort of a column element, (12) the reduction should be applied on the reduced concrete section in the case of spalling and if there are not stirrups, a reduction in the longitudinal bars subjected to compression due to risk of buckling should be also taken into account.

Serviceability limit state

The serviceability limit states to be checked should be:

- Exterior aspect of the structures (rust, spalling).
- Cracking of cover due to corrosion or excessive loading.
- Excessive deflections.

For the deflection and crack checking due to loading, the same expressions provided by Eurocode 2 can be used, but reducing the steel section and that of the concrete due to spalling.

Conclusions and final remarks

At present there are not adequate recalculation tools for the economic and safe management of concrete structures. Such a tool is still in its beginnings and only empirical or qualitative appraisal methodologies can be found in the literature.

Through the EU projects "The residual service life of concrete structures" (BRITE 4062) and the Contecvet, an important advancement from present situation has been made. Thus, methodologies for simplified and refined assessment have been developed and recalculation tools have been derived based in an extensive research on the structural behaviour of corroding elements.

The methodology starts by a preliminary inspection, that tries to identify the damage level of the structure and the environmental characteristics that surrounds the structure. Then, it is needed a structural assessment on the element in order to evaluate the intervention urgency. This structural assessment can be performed at two different levels:

- **Simplified assessment**: An empirical procedure using several indexes from which, a general damage level of the structure is obtained. As a direct result of their application the specialist should be able to decide if more studies are needed.

- **Detailed assessment** That allows a complete verification of the element safety in a similar manner as that proposed by the Limit State theory. The aspects covered are:

1. Action effect assessment.
2. Material properties and their damage level.
3. Load bearing capacity and serviceability verification

This detailed assessment can be performed using classical design concrete models but, reducing adequately the final characteristics of the composite section reinforcement – concrete.

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