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Corrosion process and structural performance of a 17 year old reinforced concrete beam stored in chloride environment

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Abstract

The long-term corrosion process of reinforced concrete beams is studied in this paper. The reinforced concrete elements were stored in a chloride environment for 17years under service loading in order to be representative of real structural conditions. At different stages, cracking maps were drawn, total chloride contents were measured and mechanical tests were performed. Results show that the bending cracks and their width do not influence significantly the service life of the structure. The chloride threshold at the reinforcement depth, used by standards as a single parameter to predict the end of the initiation period, is a necessary but not a sufficient parameter to define service life. The steel–concrete interface condition is also a determinant parameter. The bleeding of concrete is an important cause of interface de-bonding which leads to an early corrosion propagation of the reinforcements. The structural performance under service load (i.e.: stiffness in flexure) is mostly affected by the corrosion of the tension reinforcement (steel cross-section and the steel–concrete bond reduction). Limit-state service life design based on structural performance reduction in terms of serviceability shows that the propagation period of the corrosion process is an important part of the reinforced concrete service life.

Keywords: Crack detection (B); Corrosion (C); Durability (C); Reinforcement (D); Concrete (E)

1. Introduction

Corrosion of steel reinforcements represents the major cause of degradation of reinforced concrete structures. The corrosion process leads to several coupled effects: longitudinal cracking of concrete cover due to expansive corrosion products; steel cross-section reduction; and the degradation of steel–concrete bond. As a result of these effects, the service life and the load-bearing capacity of reinforced concrete elements are considerably reduced.

Many studies have already been carried out on parameters which influence the onset of steel corrosion in concrete containing flexural cracking. These cracks, to the result of mechanical loading, represent a preferential route for penetration of chloride and reduce the initiation period of the corrosion process [1–4]. However, other authors showed that, if corrosion firstly appears on the steel located at the flexural crack tip, a self-healing phenomenon [5] due to the crack filling by the corrosion products slows down the migration of chlorides [6], and reduces strongly the corrosion activity [7,8]. The influence of the flexural cracking on corrosion is still a contentious subject. Chloride content at the depth of the steel bars in the concrete is an important major parameter for corrosion onset. RILEM [9] and ACI [3] define threshold values to forecast the beginning of the propagation period of the corrosion process in reinforced concrete.

This paper deals with the study of beams naturally exposed to a salt fog for 17years. At different stages, beams have been extracted to perform experiments in order to collect data: corrosion cracking maps, chloride content at the level of the reinforcement, corrosion distribution assessed from corrosion cracks width, and the mechanical performance. From these results, the influence of flexural cracks and chloride content on corrosion initiation period is discussed including the influence of other parameters as the steel–concrete interface condition. During the propagation period, the evolution of corrosion with

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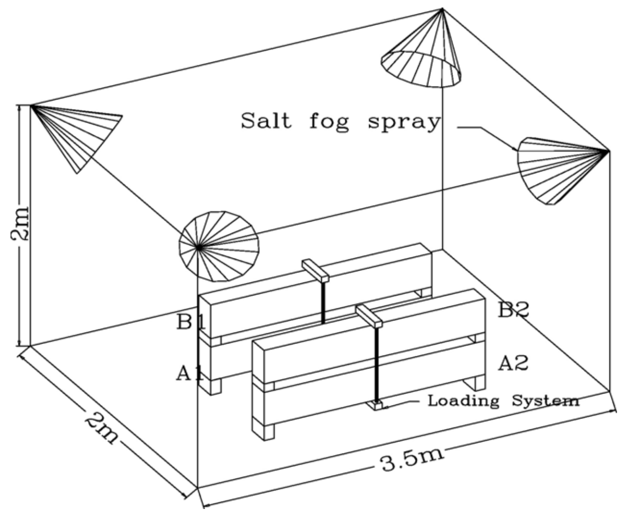

Fig. 1. Experimental setup and exposure conditions.

time and its influence on the structural performance under service load (i.e.: bending stiffness) is studied.

2. Experimental context

A long-term experimental program was initiated in 1984 at Laboratoire Matériaux et Durabilité des Constructions (L.M.D.C.). The aim was to improve the comprehension of the steel corrosion process in reinforced concrete elements and its incidence on the structural performance. 36 reinforced concrete beams (300×28×15cm), a typical size of the elements used in the construction industry of the time were cast and then stored in a chloride environment under service load to take into account the influence of the flexural cracks. At different stages, experimental studies were performed on beams to assess the development of flexural and corrosion cracking, to measure chloride content and to analyze the evolution of the mechanical behavior.

The aggressive environment is a salt fog (35g/l of NaCl corresponding to the salt concentration of sea water) generated through the use of four sprays located in each upper corner of a confined room (Fig. 1). After 6years of storage, the beams were submitted to wetting–drying cycles in order to accelerate the corrosion process:

- 0 to 6years: continuous spraying under laboratory conditions ($T° \approx 20°C$),
- 6 to 9years: cycles spraying under laboratory conditions ($T° \approx 20°C$), one week of spraying and one week of drying,
- 9years to now: cycles spraying, one week of spraying and one week of drying, however the confined room was transferred outside, so the beams were exposed to the temperature of the south-west of France climate, ranging from −5°C to 35°C.

Although this represents an accelerated version of the real process, the corrosion obtained was much closer to that actually observed in natural conditions, with respect to corrosion distribution, corrosion type and the oxides produced, than those resulting from the use of an impressed current or a $CaCl_2$ admixture in concrete [10].

The beams were divided in two groups, the type A and type B beams, which have different reinforcements, but use the same ordinary reinforcing steel (yield strength=500MPa). Beams A and B correspond to a 40mm maximum and 10mm minimum concrete cover respectively (Fig. 2), in accordance with the French regulations at the time of manufacturing [11]. The beams were loaded in a three-point flexure by coupling a type A beam with a type B beam. According to French standards, the level 1 loading (M_{ser1}=13.5kN m) corresponded to maximum loading versus durability in an aggressive environment for the type A beam (serviceability limit-state requirements in an aggressive environment) and to maximum loading versus resistance (ultimate load limit state in a non-aggressive environment) for the type B element. The beams stored under sustained level 1 loading are called A1 and B1. The level 2 loading value for the type B beam presented in this paper (M_{ser2}=21.2kN m) corresponded to 80% of the failure load and was equal to twice the design service loading in aggressive environment according to French standards. For the type A beam, it corresponded to maximum loading versus resistance (ultimate load limit state in a non-aggressive environment). The beams stored under sustained level 2 loading are called A2 and B2.

Concrete composition and cement chemical composition are given in Table 1. The water/cement ratio was 0.5 but the water content was in some case adjusted (i.e. 0.49 or 0.48) to obtain a constant workability of 7cm in slump test.

The average compressive stress and the elastic modulus obtained on ϕ11×22cm cylinder specimens were respectively 45MPa and 32GPa at 28days. The tensile strength, measured using the splitting test, was 4.7MPa. Porosity was 15.2%.

In order to realize a comparative study, several control beams have been cast with beams destined to aggressive environment. The control beams were called B2T whereas the corroded beams were called B2CL. The control beams have the same concrete composition and lay-out of the reinforcement, but were stored in a

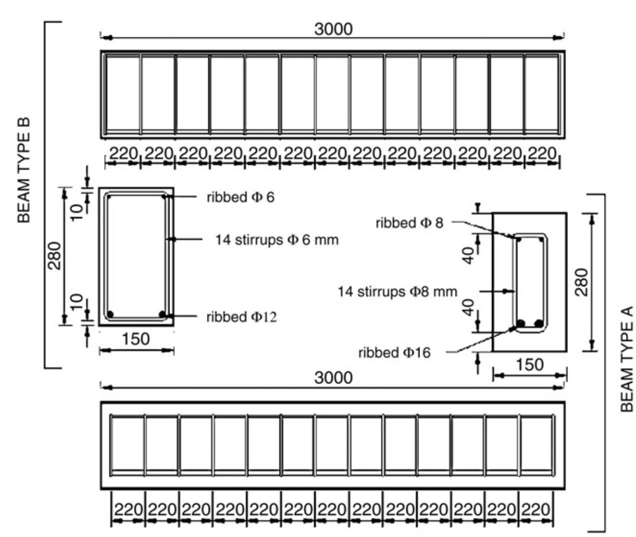

Fig. 2. Lay-out of the reinforcement (all dimensions in mm) for type B beams.

Table 1
Concrete and cement composition

Mix component		
Rolled gravel (silica + limestone)	5/15 mm	1093 kg/m^3
Sand	0/5 mm	734 kg/m^3
Portland cement: OPC HP (high performance)		358 kg/m^3
Water		179 kg/m^3

Cement composition

	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Na_2O
% Weight	21.4	6.0	2.3	63.0	1.4	3.0	0.5

50% of R.H. and 20°C laboratory room. The average compression stress and elastic modulus at 28days were similar to B2CL values.

In this paper, the analysis of one of the B2CL corroded beam (beam B2CL2) extracted from the confined room at different stages, is presented. This paper aims to analyze and understand the corrosion process during initiation and propagation period versus different parameters such as flexural cracking, total chloride content at the depth of the reinforcement, steel–concrete interface condition…, and its impact on the structural performance under service load. To follow the evolution of the mechanical characteristics of the concrete and the chloride penetration through the concrete cover versus time, destructive tests have been performed on other equivalent type B2 corroded and control beams at 6, 14 [12] and 17years. No destructive tests have been performed on B2CL2 which is still aging in salty environment in order to follow the mechanical behavior in the future. The mechanical characteristics of the concrete were measured by drilling cores near the support where the flexural load does not affect the concrete. The average compressive strength and the elastic modulus were stabilized after 6years and equal respectively to 63MPa and 35GPa for corroded beams, and 65MPa and 36.3GPa for control beams.

3. Experimental program

3.1. Cracking maps

Cracking maps were drawn with the locations of flexural transverse cracks and longitudinal corrosion cracking at four ages, 28days, 6, 14 and 17years. The crack widths were also measured using a binocular lens with an accuracy of 0.02mm.

3.2. Total chloride content

During the first years, the total chloride content was studied by microprobe [13]. Chloride content was measured on 10mm diameter cores sawn in 5mm thickness pieces. This measure was local and led to a large scatter. This method only gives information on the presence of chlorides, not on quantity. At the date of 5years, drilling was used to collect powder from a 10mm diameter core, at different depths. However, the 10mm diameter was also too small in comparison with the largest aggregate dimension. So, the accuracy of total chloride measurement at this date is rather weak, and difficult to assess. After 5years, the total chloride content was assessed using a profile grinder GERMANN INSTs [14]. Concrete samples were also collected at the two later dates (14 and 17years) using the profile grinder method. This method consists in extracting fine concrete powder (< 80µm) from the lateral surface of the beam over diameter disk of 30mm (about three times the size of the largest aggregate) and on few millimeters depth to obtain representative quantity of powder (about 4g). Nitric acid is used to extract total chloride which is then measured by potentiometric titration using $AgNO_3$, and reported as a percentage per mass of cement. Concrete powders were taken in a compressive and a tensile concrete areas located in the central part of the beam, at the depth of the reinforcements, i.e. 16mm. Samples were collected from at less 50mm from any concrete crack (corrosion or flexural cracks).

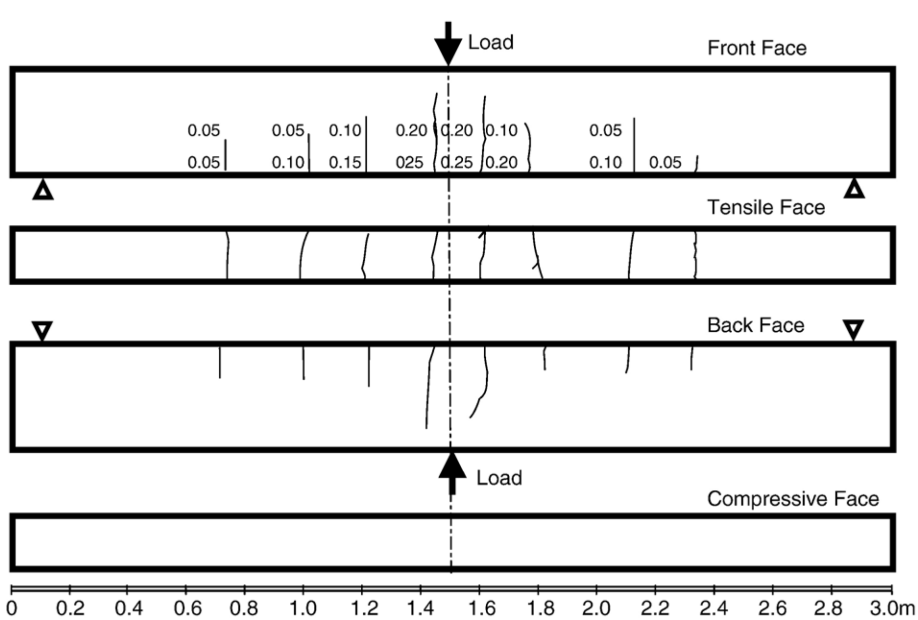

Fig. 3. Cracking map of beam B2CL2 after 28 days of storage.

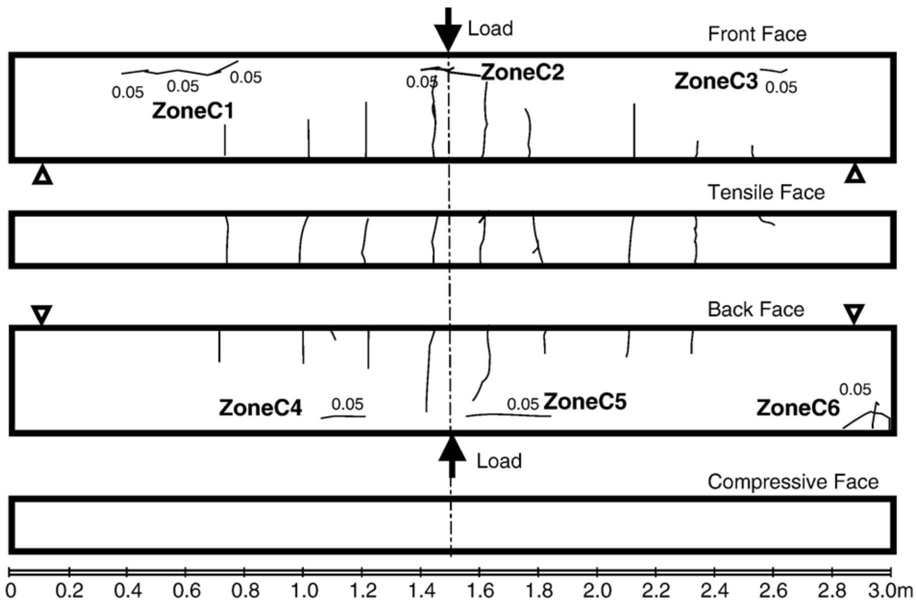

Fig. 4. Cracking map of beam B2CL2 after 6 years of storage.

3.3. Mechanical behavior

The global mechanical behavior under service loading of the element is analyzed through the study of mid-span deflection after 14 and 17years for the B2CL2 corroded beam and B2T control beam, loaded in the three-point flexure. The maximum loading applied during these tests was 13.5kN m. It is reminded that this loading corresponds to the maximum loading versus resistance (ultimate load limit state in a non-aggressive environment) for the type B element. The objective was to study the stiffness and to avoid the damage of the corroded beam with a higher loading level.

At 28days and 6years, the deflection was recorded using an analog flatbed plotter with 150mm full scale. This method was not enough adequate to give good accuracy for the displacement under service loading. At 14 and 17years, the recording system was different. A numerical sensor measured the displacement with an accuracy of 0.01mm. Consequently, only the results of mid-span deflection at 14 and 17years will be presented.

4. Experimental results

4.1. Cracking maps

The cracking states of each face of the beam are presented on cracking maps, for the four ages (Figs. 3–6). The first map (Fig. 3) has been drawn after the first mechanical loading, at 28days. On these maps, the thicker lines mean that opening widths are superior to 1mm.

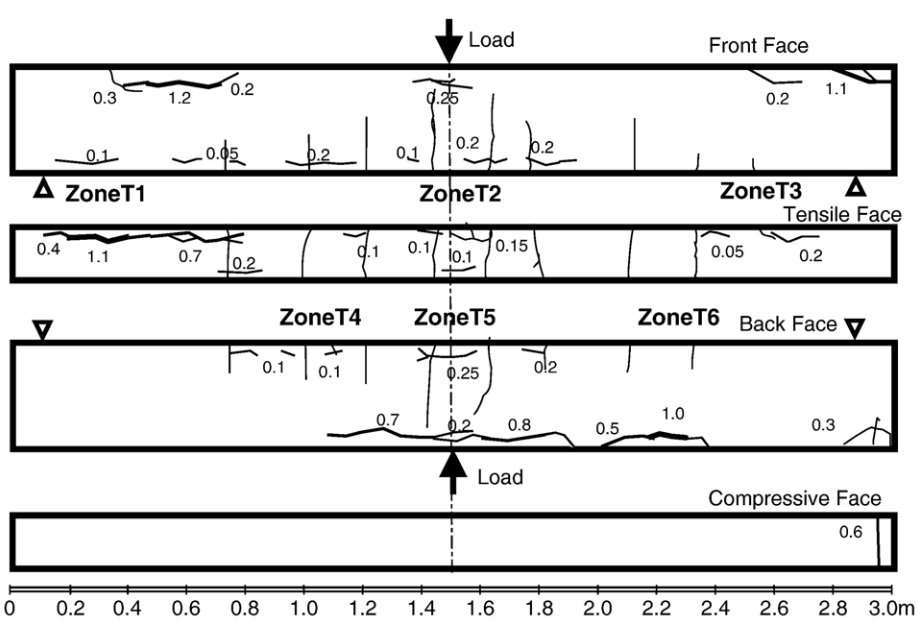

Fig. 5. Cracking map of beam B2CL2 after 14 years of storage.

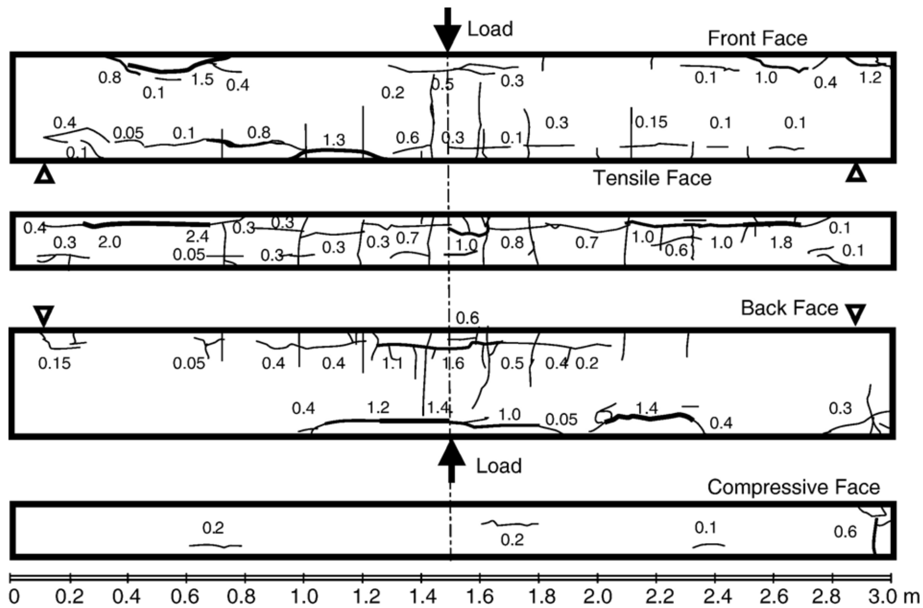

Fig. 6. Cracking map of beam B2CL2 after 17 years of storage.

Flexural cracks are generated at 28days by three points loading. They appear in the tensile central part of the beam where stresses exceed concrete tensile strength. Their widths (in mm) are specified at the bottom of front face and at the reinforcement level. The maximum value is 0.25mm and remains inferior to the 0.3mm limitation recommended by Eurocode 2 [15]. For the other stages, the width of flexural cracks are not indicated since they do not get widen along the time.

The next cracking map (Fig. 4) has been realized after 6years of storage, when first visible corrosion cracks appeared. On the contrary of transversal straight lines in the central area corresponding to the flexural cracks, cracks induced by steel corrosion are longitudinal and parallel to reinforcements. On this map, six corrosion cracks are visible in the compressive zone. Three are located on the front face (called ZoneC1 to 3), and three on the back one (called ZoneC4 to 6).

After 14years of storage (Fig. 5), corrosion cracking has increased. The corrosion cracks located in the compressive zone widened, but had not grown significantly in length. In the tensile zone, cracking has appeared but no preferential zone can be observed. Six cracking areas in the tensile zone are marked (T1 to T6). The evolution of these six corrosion zones will be studied further in the paper. There is no evidence of cracking associated with stirrup corrosion.

The cracking map at 17years (Fig. 6) shows only a few new longitudinal cracks due to steel corrosion in the compressive zone. On the contrary, in the tensile zone, a great number of cracks have spread out all along the beam. They have connected and widened. Cracking has generalized along the front tensile reinforcement, but is more located in the central part of the back face.

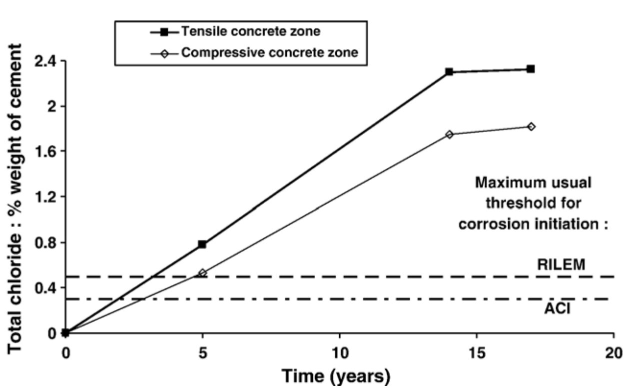

Fig. 7. Total chloride content at the depth of the reinforcement (16 mm) in the compressive and tensile zone of the corroded beams, at the different stages.

4.2. Evolution of the total chloride content

The development of the total chloride content in the tensile and the compressive concrete zones at 16mm depth (i.e. at the reinforcement depth), is shown in Fig. 7.

After 5years, the chloride content reached and exceeded the RILEM and ACI threshold values, respectively 0.5% and 0.3% by weight of cement, generally used to forecast corrosion initiation in reinforced concrete [3,9]. The total chloride content seems to increase faster in the tensile zone. This is probably due to a mechanical damage of the tensile concrete which leads to a significant increase of the chloride diffusion coefficient [16]. However, at this date, the accuracy of the measurement was weak. Indeed, the concrete powder was extracted from 10mm

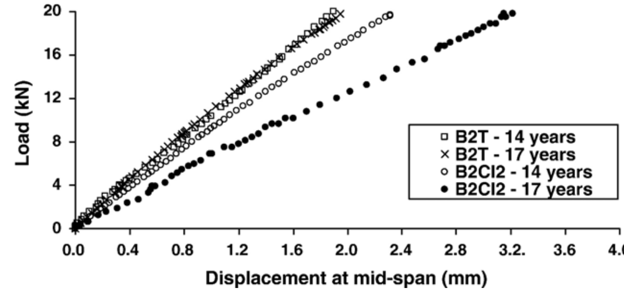

Fig. 8. Deflections at mid-span after 14 and 17 years of storage.

diameter core. So this dimension was too small in comparison with aggregates one. So, it may occur that the concrete powder was not representative if drilling was made on an aggregate. In that case, the chloride content would be underestimated since the chloride is located in the cement paste. But, the accuracy is rather difficult to calculate.

Between 14 and 17years, the total chloride tends to stabilize. In particular, the value seems to reach a maximum chloride threshold which can be contained by the concrete. The total chloride is composed of free and bound chlorides. The level of free chlorides depends on salt fog chloride concentration and concrete apparent porosity. The values of these different parameters are the following:

- apparent porosity: 15.2%
- NaCl concentration of salt fog: 35g/l, i.e. almost 22g/l of chlorides
- concrete density: 2240kg/m^3
- masses ratio of concrete on cement: 6.25.

So, the maximum free chloride percentage by cement mass is $15.2\% \times 22 \times 6.25 / 2240 = 0.93\%$. The level of bound chlorides depends on the interaction isotherm between chlorides and cement. From the analysis of this interaction isotherm, François et al. [7] showed that the ratio between bound and free chloride is about 1.5. So, the theoretical maximum total chloride percentage for this beam concrete is: $0.93 + 1.5 \times 0.93 = 2.3\%$. This value is close to experimental ones. This tends to prove that, whatever the location, compressive or tensile zones, the concrete is saturated with chloride after 14years. As the scatter can be estimated to 5% of the percentage of chloride by weight of cement, the difference between compressive and tensile zone can be considered as significant at 14years [16].

4.3. Mechanical behavior

During a three-point flexure tests after 14 and 17years, the deflection at mid-span was recorded for B2T control beam and B2CL2 corroded beam (Fig. 8).

The B2T deflection was identical after 14 and 17years. Comparing B2T control beam and B2CL2 corroded beam after 14years, the deflection increase due to corrosion was about 20%. After 17years it reached 70%. This degradation of mechanical behavior is directly linked with the increasing cracking development due to reinforcements corrosion, particularly in the tensile region.

5. Discussion

5.1. Influence of flexural cracking on corrosion process

Flexural cracks represent a way of preferential penetration for chlorides to reach the re-bar and reduce the initiation period of corrosion process [1–4]. However, the evolution of corrosion cracking of B2CL2 seems to prove that the presence or the width of flexural cracks do not play a significant role on corrosion process. The results obtained on other corroded beams from the same experimental program with weaker flexural crack widths confirm this conclusion [12]. In spite of the presence of flexural cracks, no sign of corrosion is observed on the tensile zone of the beams after 6years of storage. The first visible longitudinal corrosion cracks appear in the compressive zone. After 14years, corrosion cracks also appeared in the tensile zone, but not exclusively in flexural cracking zone as the main crack is located near the support (ZoneT1 in Fig. 5). Between 14 and 17years, the corrosion cracking increases in the tensile zone but this corrosion propagation cannot be correlated to chloride penetration through bending cracks as, at this stage, the concrete is saturated by chlorides all along the reinforcement and particularly between the bending cracks.

Some authors showed that, if corrosion firstly appears on the steel located at the flexural crack tip, a self-healing phenomenon [5] due to the crack filling by the corrosion products slows down the migration of chlorides [6], and reduces strongly the corrosion activity [7,8]. This interpretation is in accordance with the experimental results obtained on B2CL2 which can be generalized to the whole set of 36 beams studied at L.M.D.C. with crack width ranging between 0.1mm and 0.4mm [12]. The current standards, American [3], European [15] and French [17] standards, recommend to limit, directly or indirectly, the flexural cracks widths in order to prevent the corrosion effects. This study, confirms that the influence of flexural cracks on the corrosion process is not determinant when widths are less than 0.3mm.

5.2. Relationship between total chloride content and corrosion cracking

First, ACI and RILEM limitations have been exceeded after a short lapse of time at the level of the reinforcing bars, about 4 to 5years. From the different locations of chloride extraction obtained on other equivalent beams stored in the same conditions [18], we could conclude that this high total chloride content is generalized all over the beam at the depth of the steel. However, the corrosion cracks did not develop homogeneously all over the longitudinal reinforcement length. Previous studies [18,19] carried out on other beams from the same experimental program have confirmed the heterogeneous development of the reinforcement corrosion. Thus, in spite of an important chloride content after 5years, only a few corrosion cracks in the compressive zone can be noted. Moreover, this corrosion cracking appeared first in the compressive zone after 6years in spite of a total chloride content higher in the tensile zone.

The stirrups which are the steel reinforcement closest to beam faces have not developed enough corrosion products to involve concrete cover cracking which is another paradox.

Therefore, in accordance with other studies [4,18,20,21] the chloride threshold seems to be a necessary but not sufficient parameter to forecast accurately the steel corrosion onset in concrete. A determinant parameter which can explain those paradox is the steel–concrete interface condition. The presence of voids (de-bonding) at the interface allows a better environment for corrosion: accessibility for oxygen, water and chloride to the reinforcing bars [16]. These voids are determinant for corrosion to

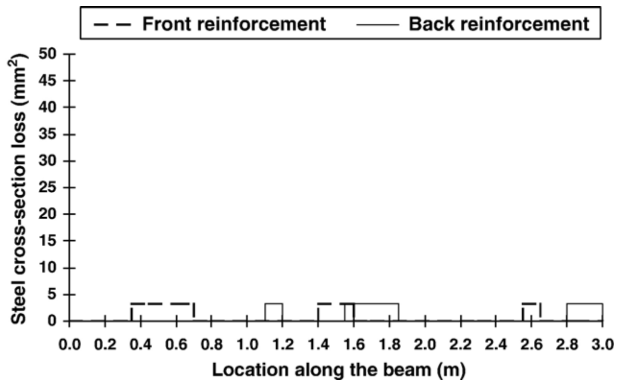

Fig. 9. Distribution of corrosion along B2CL2 beam, for reinforcements in compressive concrete zone, after 6 years of storage.

start and contribute to reduce the initiation period of the corrosion process. On the contrary, a good steel–concrete interface condition delays the corrosion onset.

6. Calculus and study of steel loss evolution during propagation period

In a previous study [19], a non-destructive method to assess the reinforcement corrosion was proposed. This method is based on empirical relationships which allow us to calculate the local steel cross-section reduction from the corrosion crack width. Firstly, the corrosion crack maps after 6, 14 and 17years of exposure (Figs. 4–6) are used to calculate the distribution of the local cross-section reductions along each longitudinal reinforcement of the beam B2CL2. Firstly, the evolution of the local reinforcement cross-section reductions along the re-bars versus time are discussed (local analysis). Secondly, the evolution of the global mass loss of the reinforcement located in tensile and compressive zone is discussed (global analysis). The relationships between corrosion cracking and corrosion of stirrups are also commented. Finally, the influence of corrosion propagation on beams' structural performance (bending stiffness) and service life are discussed.

6.1. Corrosion of the reinforcing bars located in compressive concrete zone

The distribution of cross-section loss due to corrosion is calculated from this empirical linear expression [19]:

$$w = K(\Delta A_s - \Delta A_{s0})$$

where

w	is the crack width in mm
ΔA_s	is the steel loss of cross-section in mm^2
ΔA_{s0}	is the steel loss of cross-section initiating cracking in mm^2
K	0.0575.

The calculated corrosion distributions (steel cross-section loss) along both reinforcement in the compressive concrete zone are exposed in the Figs. 9–11 respectively after 6, 14 and 17years.

After 6years of exposure (Fig. 9), the first corrosion cracks occur in the compressive concrete zone. Only six corrosion pit attacks have led to concrete cracking. The corrosion is rather randomly distributed, and there is no corrosion preferential area along the reinforcement. At this stage, the chloride threshold at the depth of the reinforcing bars is almost the same all along the beam and exceeds ACI and RILEM limitations. As mentioned previously, in spite of a uniform total chloride content along all the steel bars, corrosion cracking occurred after 6years exclusively along the bars in the compressive concrete zone and not along the tensile bars. The steel–concrete interface condition can explain this phenomenon. The early corrosion of the bars in compressive zone can be correlated to the Top-bar effect resulting from concrete bleeding. The Top-bar effect leads to voids under horizontal bars located in the upper part of the beam with regard to casting direction [22–26]. These voids are determinant for corrosion to start because they lead to a better access for oxygen, water and chloride to the reinforcing bars and then reduce the initiation period of the corrosion process [4,18,20].

After 14years of storage (Fig. 10), only two new pit attacks leading to concrete cracking at location 2.2 and 2.8m along the beam are detected. The losses of steel cross-section have increased in the existing corroded zones. This tends to prove that for longitudinal corrosion cracks, there is no self-healing as in the case of flexural cracks. The access to the longitudinal re-bars is easier for the chlorides, the oxygen and the humidity in the case of longitudinal cracks along the bars than in the case of transversal ones. The longitudinal cracks are following the whole length of the reinforcements.

After 17years (Fig. 11), the corrosion has gone on increasing in terms of intensity, but no more new pit attacks leading to concrete cracking appeared. The corroded areas have a little extended in the central part of the back reinforcement. The growth of the cracks between 6 and 17years is very heterogeneous. Whereas the reductions of steel cross-section were the same after 6years, the ratio values range between 1 and 3 after 17years.

6.2. Corrosion of the reinforcing bars located in tension zone

The calculated corrosion distributions (steel cross-section loss) along both tension reinforcement are shown in the Figs. 12

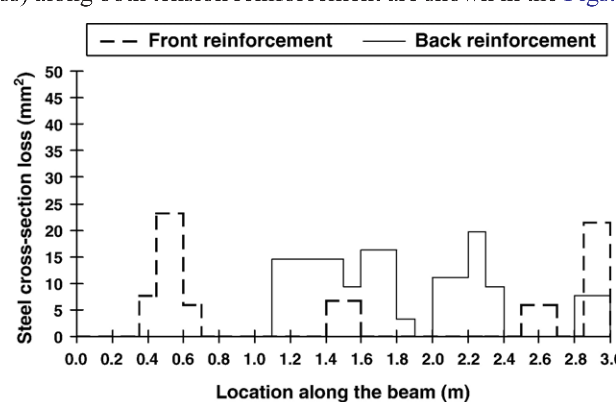

Fig. 10. Distribution of corrosion along B2CL2 beam, for reinforcements in compressive concrete zone, after 14 years of storage.

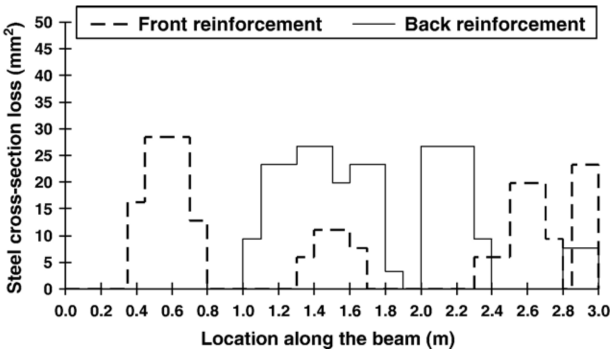

Fig. 11. Distribution of corrosion along B2CL2 beam, for reinforcements in compressive concrete zone, after 17 years of storage.

and 13 respectively after 14 and 17years. After 6years of storage, no longitudinal corrosion cracking occurred in the tension concrete zone, whereas the total chloride threshold necessary for corrosion onset was reached. It is possible that the corrosion had begun, but the volume of oxides produced was not sufficient to generate the concrete cracking. Contrary to reinforcing bars in the compressive concrete zone, the tensile rebars are located in the bottom part of the beam with regard to the casting direction. Thus, an explanation could be that the interface has better quality for tensile reinforcement than for compressive ones because of bleeding. So, the corrosion propagation of tensile bars was delayed.

After 14years of storage, corrosion pits leading to cracks appeared in the tensile concrete zone (Fig. 12). The corrosion distributions along both tensile reinforcements are not similar. Some localized corrosion attacks can be observed in the B central part of the back and front reinforcements. The central part corresponds to the zone where the load is a maximum. In the case of the front reinforcement, corrosion also appeared in the areas close to the supports (zones A and C). The A area, corresponds to the most corroded area. We remark that the corrosion developed in a quite heterogeneous way, in terms of location and intensity (Figs. 14 and 15). The explanation is rather complex: quality of the interface, local humidity, orientation of the concrete surface versus salt fog sprays, …

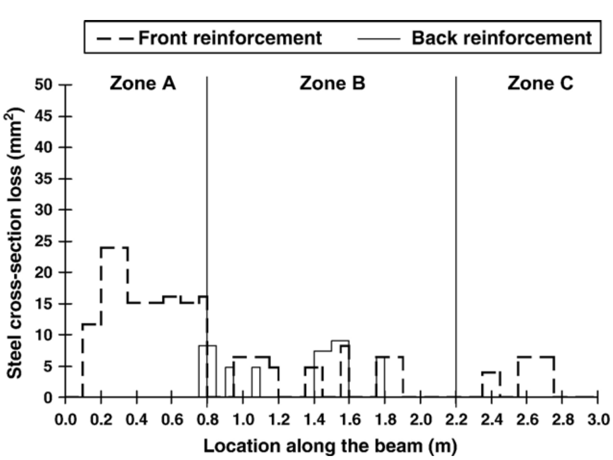

Fig. 12. Distribution of corrosion along B2CL2 beam for reinforcements in tensile concrete zone, after 14 years of storage.

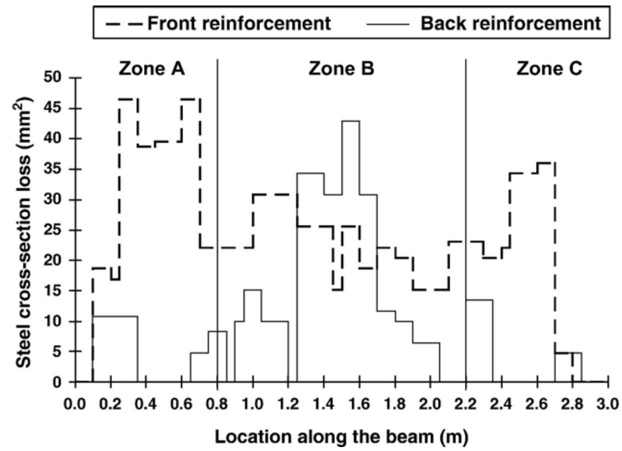

Fig. 13. Distribution of corrosion along B2CL2 beam for reinforcements in the tensile concrete zone, after 17 years of storage.

After 17years of storage (Fig. 13), the corrosion was generalized all along the front reinforcement, but the maximum cross-section reductions were still located in zone A, close to the left support. Along the back reinforcement, a large increase of corrosion occurred in the B central part of the beam, with a 380% increase between 14 and 17years at mid-span. Three new local pit attacks also appeared over this period of three years near each support (zones A and C). The corrosion propagation is very heterogeneous:

- the evolution is different between the two tensile reinforcements, the maximum loss of steel cross-section occurs near the left support for the front bar, and in the central part of the back one.
- the corrosion is not uniform considering only one reinforcing bar.
- between 14 and 17years, the increase of steel cross-section losses varies considerably versus the location along the bars.

6.3. Global analysis of corrosion propagation versus time

Fig. 16 represents the evolution of the percentage of total steel mass loss versus time. This percentage corresponds to the

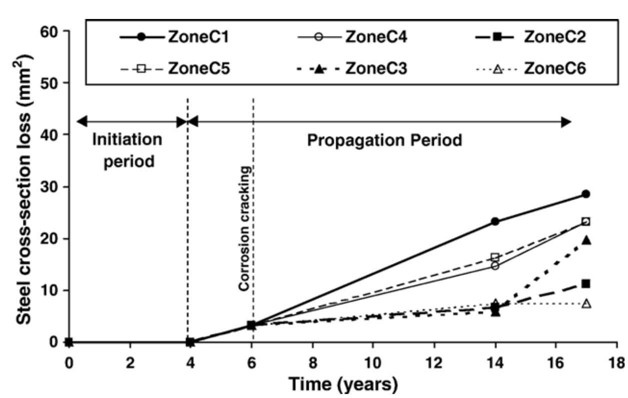

Fig. 14. Evolution of the local loss of steel cross-section versus time for the six locations in the compressive zone. Zones C1 to C6 are shown on the cracking maps.

ratio between the global loss of steel mass and the total mass of reinforcements. The global loss of steel mass corresponds to the total area located under the curve presenting the steel loss mass distribution along the reinforcement, i.e. multiplication of the volume of steel loss by steel density. The results are presented for reinforcements located in tensile and compressive concrete zone separately.

Because of their small initial cross-section (diameter = 6 mm), the percentage of steel loss is always higher for the bars in the compressive concrete zone than for the tension re-bars. For the bars in the compressive concrete zone, after the end of the initiation period (about 4years) which is an extrapoled value, the global corrosion rate is almost constant until 14years of exposure. After 14years the corrosion rate slightly increases. For tension re-bars, after the end of the initiation period (about 5years), the corrosion rate remains low until 14years of exposure. Finally, between 14 and 17years, probably because of corrosion concrete cracking widening (width > 0.2mm), the corrosion rate considerably increases. This may be due to an easier oxygen access.

This global analysis of B2CL2 corrosion process shows that, firstly, the initiation period which is shorter for the bars in the compressive zone may be caused by the lower quality of the interface as mentioned previously.

In the propagation phase, the shape of both curves is globally similar with an increase of the corrosion rate between 14 and 17years of exposure. Then, this global approach of the corrosion process shows that the corrosion in the propagation phase is comparable for all the reinforcing bars of the beam which is coherent as the exposure condition and the concrete characteristics and cover are similar. But it is important to keep in mind that the local analysis of the corrosion propagation along the beam (Figs. 14 and 15) has shown very important scatters on the distribution of the steel cross-section losses, even along the same re-bar.

6.4. Analysis of the structural performance versus corrosion

Fig. 17 allows to analyze the evolution of the stiffness in flexure, i.e. the slope of the curve mid-span deflection-load (Fig. 8), versus time. Experimental values have been measured

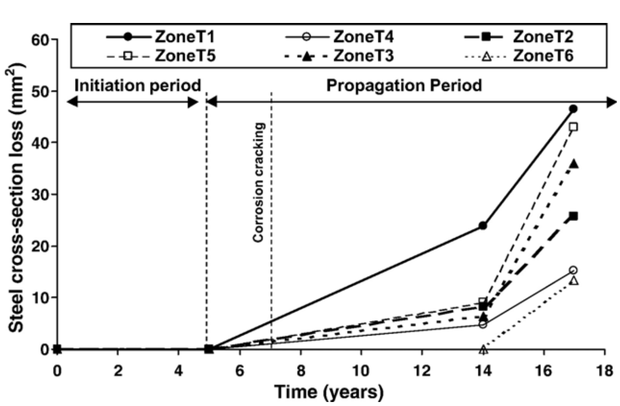

Fig. 15. Evolution of the local loss of steel cross-section versus time for the six locations in the tensile zones. Zones T1 to T6 are shown on the cracking maps.

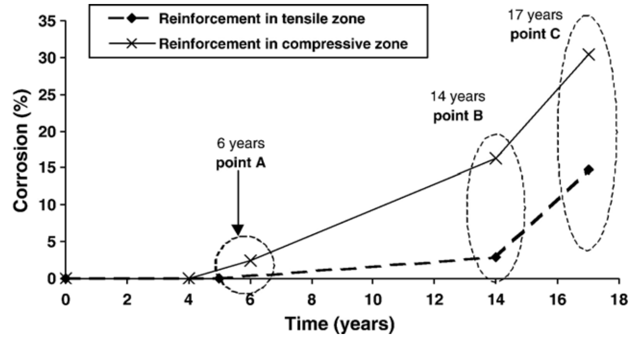

Fig. 16. Evolution of the global corrosion versus time for reinforcements in tensile and compressive concrete zone.

at 14 and 17years. Theoretical results calculated from a model proposed by Castel et al. [27] are also presented for the B2T control beam and B2CL2 corroded beam. This model is based on the assessment of the tension stiffening effects and on the coupled effects of steel corrosion: the steel cross-section loss and the steel–concrete bond damage in the tensile zone. However, the model does not take into account the effects of corrosion cracking in the compressive zone which is not significant under service load [28]. Indeed, this cracking can have an influence on the ultimate load of the beams but not on the flexural stiffness under service load [28,29].

Before 6years, no experimental data are available. The model does not predict any difference between B2T and B2CL2 as no significant corrosion in the tensile zone of B2CL2 occurred. A slight increase of the stiffness appears due to the rise of the concrete elastic modulus. Between 6 and 14years, a reduction of stiffness of about 20% was recorded experimentally between B2T and B2CL2. For B2CL2, the bending stiffness (theoretical and experimental) obtained after 14years is equal to that at 28days: the effect of corrosion counteracts the increase of the concrete elastic modulus between 28days and 6years.

Finally, the most important reduction of the bending stiffness of B2CL2 is observed between 14 and 17years. This degradation of the structural performance under service loads can be clearly correlated to the corrosion propagation in tensile reinforcements. The reduction of the bending stiffness shown in Fig. 17 is correlated with global corrosion propagation of the tensile re-bars shown in Fig. 16 (point B to point C). The bending stiffness

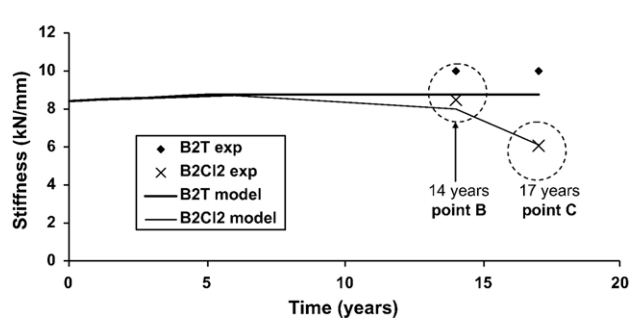

Fig. 17. Evolution of the model and experimental stiffness versus time of B2T and B2CL2 beams.

is then mostly affected by corrosion propagation of the tension re-bars where the bending moment is the most important. The corrosion in these areas leads to the steel cross-section loss and the degradation of the steel–concrete bond [28,30–32]. It is also important to note that this stiffness reduction cannot be attributed to the presence of new flexural cracks.

The concrete cracking due to corrosion of the bars in the compressive zone has a weak influence on the structural performance under service loading. This cracking can lead to a reduction of load-bearing capacity and also a loss of beam ductility due to compressive concrete spalling [29].

6.5. Limit-state service life design

The development of limit-state service life design is presently of a strong interest. One approach has been in quantifying when an unacceptably wide corrosion crack occurred [33]. The second approach could be when and how cracking adversely affects structural performance in terms of serviceability. According to B2CL2 corrosion process, the structural performance is not significantly affected before 14years of exposure. This period includes the initiation phase of the process and a large part of the propagation period leading to concrete corrosion cracking. Then, for this beam, initiation period was about 4 to 5years, the corrosion cracking occurred after 6 to 7years and the first significant structural performance reduction occurred after 14 to 15years of exposure, according to the experimental conditions of this study. In this case, the propagation period could be considered as the longest part of the service life of the structural element. Of course, this is due to the small concrete cover which reduced the initiation period but, even with larger concrete cover, the corrosion propagation phase is a consequent part of the reinforced concrete service life.

7. Conclusion

The main conclusions of this study were:

- Development of corrosion cracks during the time do not seem to be correlated to initial flexural cracks. Furthermore, test made on other set of beams lead to the same results [12]. Transversal flexural cracks and thier widths (< 0.4 mm) do not influence significantly the corrosion process of tension reinforcing bars and then the service life of the structure.
- The chloride threshold at the reinforcement depth, used as a single parameter to predict the end of the initiation period, is a necessary but not a sufficient parameter to define service life. The steel–concrete interface condition is also a determinant parameter.
- The bleeding of concrete is an important cause of interface debonding which could lead to an early corrosion propagation of the reinforcements located in the compressive zone compared to tensile bars in the case of simply supported beams.
- During the propagation period, in spite of very controlled environmental conditions, the corrosion distribution and evolution along the all reinforcing bars is quite different and heterogeneous. This observation concerns the location, the intensity, and the corrosion rate of corroded areas on a same re-bar and also between the two different reinforcement. It highlights the difficulty to model corrosion evolution with time, versus the numerous parameters involved as environmental ones (humidity, temperature, conditions of exposure...), concrete characteristics (physical, chemical, mechanical, ...) and steel–concrete interface condition.
- The structural performance under service load (i.e., stiffness in flexure) is affected by the corrosion of the tension reinforcement and not significantly by the concrete cracking due to the corrosion of the steel bars located in the compressive zone. The reduction of the stiffness results from two coupled effects: the steel cross-section reduction and the steel–concrete bond loss due to steel corrosion between the bending cracks.
- Limit-state service life design based on structural performance reduction in terms of serviceability shows that the propagation period of the corrosion process is a consequent part of the reinforced concrete service life.

References

[1] W. Danilecki, An investigation into the effect of crack width on the corrosion of reinforcement in reinforced concrete, RILEM Symposium on the Durability of Concrete, in: RILEM Publications (Ed.), 1969, Prague.

[2] P. Duffaut, L. Duhoux, B. Heuze, Corrosion des aciers dans le béton armé. Essais réalisés dans l'estuaire de la Rance de 1959 à 1971, Annales de l'TBTP 305 (1973), (in French).

[3] ACI committee 222, Corrosion of metal in concrete, Journal Proceedings 82 (1) (1985) 3–32.

[4] N. Otsuki, S. Miyazato, N. Diola, H. Suzuki, Influences of bending crack and water–cement ratio on chloride-induced corrosion of main reinforcing bars and stirrups, ACI Materials Journal 97 (4) (2000) 454–464.

[5] S. Jacobsen, J. Marchand, L. Boisvert, Effect of cracking and healing on chloride transport in OPC concrete, Cement and Concrete Research 26 (6) (1996) 869–881.

[6] C.Q. Li, Corrosion initiation of reinforcing steel in concrete under natural salt spray and service loading — results and analysis, ACI Materials Journal 97 (6) (2000) 690–697.

[7] R. François, G. Arliguie, J.C. Maso, Durability of Reinforced Concrete, Syntheses Report, L.M.D.C. I.N.S.A.-U.P.S., 1994, Toulouse, France (in French).

[8] P. Schiessl, M. Raupach, Laboratory studies and calculations on the influence of crack width on chloride-induced corrosion in concrete, ACI Materials Journal 94 (1) (1997).

[9] RILEM technical committee 124-SRC, Draft recommendation for repair strategies for concrete structures damaged by reinforcement corrosion, Materials and Structures 27 (1994) 415–436.

[10] F. Paradis, V. Lapointe, M. Jolin, J. Marchand, Accelerated test methods to investigate chloride-induced steel corrosion damage in reinforced concrete - a brief review, in: V.M. Malhotra (Ed.), Seventh CANMET/ACI International Conference on Durability of Concrete, 2006, pp. 357–373, May 28–June 3, Montreal.

[11] B.A.E.L., French Regulations for Reinforced Concrete Structures, 1983.

[12] R. François, G. Arliguie, Influence of service cracking on reinforcement steel corrosion, Journal of materials in civil engineering, ASCE 10 (1) (1998) 14–20.

[13] R. François, G. Arliguie, J-C. Maso, Durabilité du béton armé soumis à l'action des chlorures, (in French), Annales de l'I.T.B.T.P. 529 (1994) 1–48.

[14] E. Poulsen, Chloride Profiles — Analysis and Interpretation of Observations, AEC Consulting Engineers Ltd, Vedbaek Denmark, 1993.

[15] EUROCODE 2, Design of Concrete Structures — Part 1.1: General Rules and Rules for Buildings, December 2004.

[16] A. Castel, O. Francy, R. François, G. Arliguie, Chloride Diffusion in Reinforced Concrete Beam under Sustained loading, in: V.M. Malhotra (Ed.), Fifth CANMET/ACI International Conference, Recent Advances in Concrete Technology, 2001, pp. 647–662, ACI SP 200, July 29–August 1, Singapore.
[17] B.A.E.L., French Regulations for Reinforced Concrete Structures, 1999.
[18] A. Castel, T. Vidal, R. François, G. Arliguie, Influence of steel–concrete interface quality on reinforcement corrosion induced by chlorides, Magazine of Concrete Research 55 (2) (2003) 151–159.
[19] T. Vidal, A. Castel, R. François, Analyzing crack width to predict corrosion in reinforced concrete, Cement and Concrete Research 34 (2004) 165–174.
[20] T.U. Mohammed, N. Otsuki, M. Hisada, Corrosion of steel bars with respect to orientation in concrete, ACI Materials Journal 96 (2) (1999) 154–159.
[21] T.A. Soylev, R. François, Quality of steel–concrete interface and corrosion of reinforcing steel, Cement and Concrete Research 33 (9) (2003) 1407–1415.
[22] G.B. Welch, B.J. Patten, Bond strength of reinforcement affected by concrete sedimentation, Journal of the American Concrete Institute 62 (2) (1965) 251–263.
[23] J.O. Jirsa, J.E. Breen, Influence of casting position and shear development and splice length-design recommendations, Center for Transportation Research, Bureau of Engineering Research, University of Texas Austin, Research Report 242-3F, Project 3-5-78-242, 1981, 46 pp.
[24] B. Brettmann, D. Darwin, R.C. Dohaney, Bond of reinforcement to superplasticized concrete, ACI Journal (1986) 98–107.
[25] S.H. Kosmatka, Bleeding significance of tests and properties of concrete and concrete making materials, ASTM Special Publication, Philadelphia, STP 169C, 1994, pp. 8–111.
[26] K.H. Kayat, Use of viscosity-modifying admixture to reduce top-bar effect of anchored bars with fluid concrete, ACI Materials Journal, 95 (2) (1998) 158–167.
[27] A. Castel, R. François, G. Arliguie, Mechanical behavior model of corroded reinforced concrete, Comptes Rendus de l'Académie des Sciences — Série IIb/Mechanics, vol. 330, ELSEVIER, 2002, pp. 45–50, Editions.
[28] A. Castel, R. François, G. Arliguie, Mechanical behavior of corroded reinforced concrete beams — part 1: experimental study of corroded beams, Materials and Structures 33 (2000) 539–544.
[29] J. Rodriguez, L.M. Ortega, J. Casal, J.M. Diez, Corrosion of reinforcement and service life of concrete structures, Durability of Building Materials and Components 7 (1) (1996) 117–126.
[30] J.G. Cabrera, Deterioration of concrete due to reinforcement steel corrosion, Cement and Concrete Composites 18 (1996) 47–59.
[31] S. Yoon, K. Wang, J. Weiss, S.P. Shah, Interaction between loading, corrosion, and serviceability of reinforced concrete, ACI Materials Journal 97 (6) (2000) 637–644.
[32] Y. Ballim, J.C. Reid, A.R. Kemp, Deflection of RC beams under simultaneous load and steel corrosion, Magazine of Concrete Research 53 (3) (2001) 171–181.
[33] Duracrete Final Technical Report, Probabilistic Performance based Durability Design of Concrete Structures, The European Union — Brite EuRam III, May 2000.