# Some remarks on the seismic assessment of RC frames affected by carbonation-induced corrosion of steel bars

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## Abstract

In Europe, a significant number of existing buildings have been built in the two decades following WW2 and, hence, they are often affected by degradation phenomena, which result in reducing the resisting sections of structural members. Moreover, in areas currently classified as earthquake-prone, the same buildings were originally designed by either considering only gravitational actions or assuming outdated seismic design criteria. Therefore, the effect of degradation on under designed structure is a subject of concern and needs to be properly addressed with the aim to achieve a realistic assessment of the current safety level of existing RC frames in seismic areas. The present paper presents some numerical results of seismic analyses carried out on structural models including the effect of carbonation-induced corrosion of steel bars in RC members.

## 1 Introduction

Durability is recognised as a major issue in civil engineering structures, as during the past decades the common observations have clearly shown that the classical procedures adopted to design concrete structures have often failed to achieve sufficiently durable performance [1]. Specifically, both designing durable reinforced concrete (RC) structures [2] and handling degradation phenomena possibly developing in existing ones [3] are timely challenges in modern structural engineers. Deterioration processes, generally due to various environmental phenomena (such as corrosion possibly affecting reinforcing steel bars, frost actions, alkali aggregate reactions and sulphate attack, etc.) often lead to serious degradation in concrete members and structures [4]. In this context, the well-known Tuutti's model for degradation of structure is a classical conceptual tool intended at describing the time-evolution of the effects of concrete degradation in structural members and, particularly, in RC sections [5].

A wide and consistent classification of the exposure classes are currently adopted for determining minimal requisites for concrete based on the environmental "action" which is actually expected to be exposed to [6]. Furthermore, EN 1994-1-1:2004 [7] defines "structural classes" with the aim to provide designers practice-oriented criteria for adopting proper thickness of the concrete cover depending on both the exposure class and the design service life.

More recently, new conceptual frameworks have been formulated with the aim to design service life of new RC structures and analyse existing ones by taking into account the degradation processes possibly induced by environmental exposure [8]. The fib Model Code 2010 [9] provides researchers and practitioners with a wide report of the most recent models available in the literature for simulating the degradation processes and their consequences on the structural response of members and structures.

As a matter of principle, the safety and serviceability assessment of RC structures should consider the time-dependent variation of the structural response due to degradation phenomena. Actually, the increasing deterioration of concrete as well as the progressive corrosion of reinforcing bars may usually lead to significant reductions of the safety margins, with respect to the initial values at both ultimate and serviceability limit states [10].

In RC structures, the most serious deterioration mechanisms are those leading to reinforcement corrosion, which may occur only after DE passivation due to carbonation of the concrete cover, penetration of chloride ions, or a combination of both. Experimental tests carried out on RC members have shown that their load carrying capacity together with their ductility properties decrease as the level of rebar corrosion increases [11].

The present study specifically deals with the degradation of RC structures due to oxidation and corrosion of the internal steel rebars induced by carbonation [12]. Although this exposure class may induce milder degradation effects than other phenomena (e.g. chloride ingress) possibly affecting steel rebars [13], it is generally relevant for all RC structures as they “live” in an atmosphere that is rich in $CO_2$. Moreover, the increase in $CO_2$ concentration in the atmosphere and the rise of temperature associated with global warming can further increase the likelihood of carbonation-induced corrosion. Furthermore, the rise of average temperature can also increase corrosion rates in steel rebars [14]. Therefore, the impact of climate change on existing and new infrastructure is considerable, as corrosion damage is detrimental for both safety and aesthetics for structures [15].

This paper provides an overview of the current state of knowledge on carbonation-induced degradation of RC structures (Section 2) and summarizes the results of parametric analyses intended at figuring out the possible consequences of degradation on the ultimate capacity of members (Section 3) and on the seismic response of frames (Section 4). The main conclusions of this preliminary study are remarked in Section 5.

## 2 Simulation of carbonation-induced degradation of RC members

Carbonation is the chemical reaction of carbon dioxide ($CO_2$) with calcium hydroxide ($Ca(OH)_2$), the latter being part of the cement paste in concrete whereas the former is present in the atmosphere. It is well known that carbonation is affected not only by the concentration of $CO_2$, but also by other environmental parameters such as relative humidity (RH) and temperature [14]. Moreover, the penetration of carbonation in concrete depends on relevant materials properties, like porosity.

The present study is based on a simplified conceptual description of degradation phenomena possibly developing both inside concrete cover and in steel rebars. Fig. 1 depicts the current assumptions about the diffusion of carbonation and the nominal width of cracks possibly developing within the concrete cover. Three main stages of the process (which can be ideally associated to relevant limit states) can be defined as follows:

- depassivation, which occurs at time $t_d$, when the depth of the carbonated concrete $x_c$ layer equals the cover thickness $a$: at this stage, oxidation is triggered in steel rebars, which results in an initial expansion of their volume and a progressive loss of resisting area;
- cracking of concrete cover, which occurs at time $t_{cr}$, when the tensile stresses induced by the expansion of steel rebars reaches the tensile strength of concrete and first cracks of significant width $w_{cr}$ are formed;
- spalling of concrete cover, which occur ideally at time $t_s$, corresponding to a substantial loss of the static contribution of the concrete cover as the developed cracks achieve a critical width $w_s$.

Fig. 2 shows the conceptual assumptions for the time evolution of degradation in concrete cover and bar radius induced by carbonation.

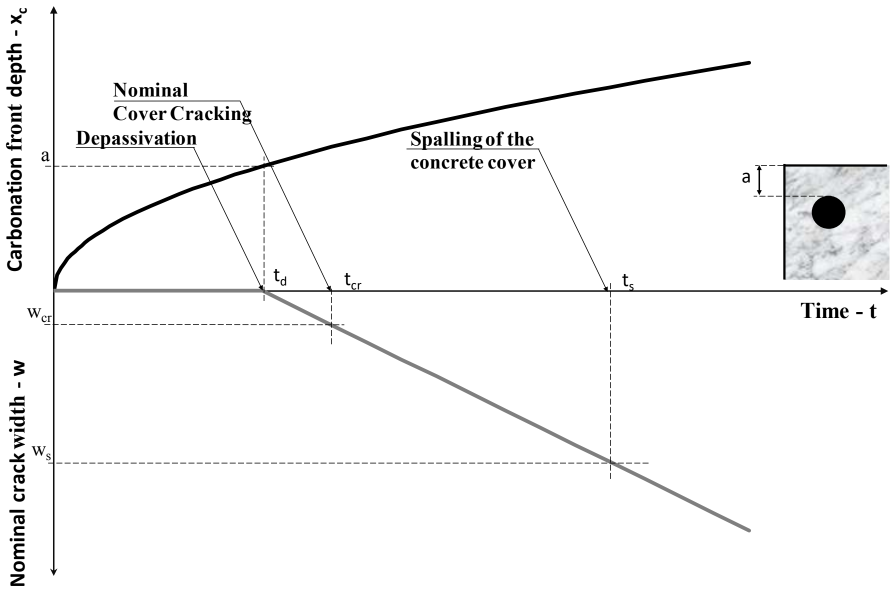


Fig. 1 Conceptual description of the time evolution of carbonation-induced effects inside concrete [12].

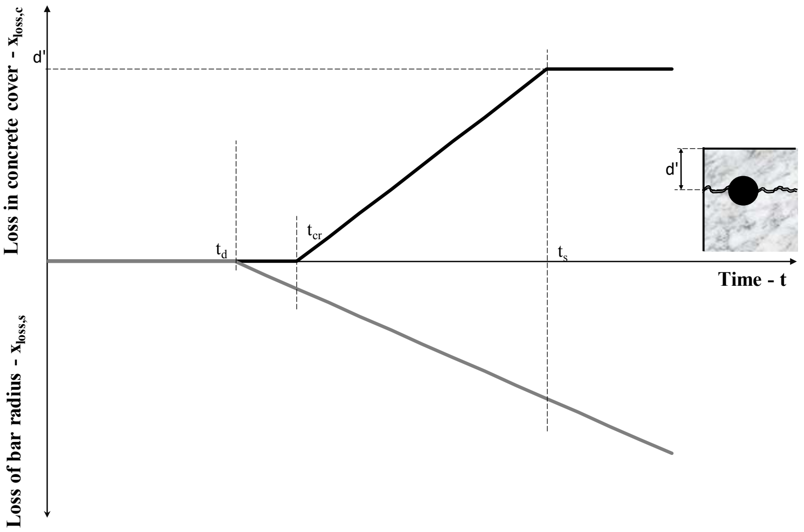


Fig. 2 Time evolution of concrete cover and bar radius loss due to carbonation [12].

Several models have been proposed simulating the diffusion of carbonation inside concrete cover. However, as they generally derive from the well-known Fick's law, the depth of the carbonated layer of concrete can be expressed as a function of the square root of time *t* and the following expression can be obtained for the depassivation time $t_d$ [8]:

$$t_d = \left( \frac{2 \cdot k_e \cdot k_c \cdot (k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot C_s \cdot t_0^{2w}}{a^2} \right)^{\frac{1}{2w-1}} \tag{1}$$

where:

- $k_e$ is environmental function [-];
- $k_c$ is execution transfer parameter [-];
- $k_t$ is regression parameter [-];
- $R_{ACC,0}^{-1}$ is inverse effective carbonation resistance of concrete [(mm$^2$/years]/(kg/m$^3$));
- $\varepsilon_t$ is error time;
- $C_s$ is the CO$_2$ concentration [kg/m$^3$];
- W(t) is weather function [-].

Due to space restrictions, no details are reported hereafter about mathematical expressions and statistical definitions assumed for of the above parameters. Further relevant information can be found in the original work [8].

Depassivation ideally triggers oxidation and, hence, corrosion of steel reinforecemtns. A liner model can be considered to descrive the time evolution of bar radius loss $x_{corr}$ as follows [8]:

$$x_{corr}(t) = V_{corr} \cdot w_t \cdot (t - t_d) \tag{2}$$

where the two parameters $V_{corr}$ and $w_t$, representing the rate of corrosion and a wheather function, depend on the environmental exposure class which the member is subjected to.

Table 1 Distribution of $V_{corr}$ and $w_t$ for different exposure class [16].

| Exposure class | | $V_{corr}$ [mm/year] | | $w_t$ [-] |
|---|---|---|---|---|
| XC1 | Dry | 0 | 0 | 0 |
| XC2 | Wet-rarely dry (unsheltered) | 0.004 | 0.003 | 1 |
| XC3 | Moderate humidity (sheltered) | 0.002 | 0.001 | 0.5 |
| XC4 | Cyclic wet-dry (unsheltered) | 0.005 | 0.003 | 0.75 |

As for concrete cracking and spalling, crack width is soposed to grow linearly right after $t_d$ and a conventional value $w_{cr}$=0.05 mm is assumed to define "visible" crack opening. Specifically, the following expression in assumed in the present study:

$$w = 0.05 + \beta \cdot \left[ p(t - t_d) - p_0 \right] \tag{3}$$

where:

- $\beta$ is a parameter controlling propagation [-];
- $p(t-t_d)$ is a measure of the propagation phenomenon [mm], which can be equalled to the loss of radius in steel bars $x_{corr}(t)$.

Moreover, the following expression is assumed for $p_0$:

$$p_0 = a_1 + a_2 \cdot \frac{a}{\phi} + a_3 \cdot f_{t,sp} \tag{4}$$

where:

- $\phi$ is the bar diameter [mm];
- $f_{t,sp}$ is the splitting strength of concrete.

In this formulation, the value $t_{cr}$ can be obtained by solving eq. (2) with respect to time t and after imposing $x_{corr}(t_{cr})=p_0$ given by eq. (4).

The values assumed for both β in eq. (3) and $a_1$, $a_2$, $a_3$ in eq. (4) are consistent with the mean and regression ones determined as part of DuraCrete Project [17]. Then, the splitting condition in the concrete cover can be determined when w in eq. (3) reaches a given threshold limit, which in the present study is assumed $w_s$=1 mm. The value of $t_s$ in Fig. 2 can be easily derived by solving eq. (3) with respect to t, for w=1 mm.

Therefore, eqs. (1)-(4) can completely describe the time evolution of the relevant degradation phenomena driven by concrete carbonation, which lead, on the one hand, to a reduction in bar radius (from $t_d$ on) and, on the other hand, to the (linearly) progressive loss of concrete cover between times $t_{cr}$ and $t_s$.

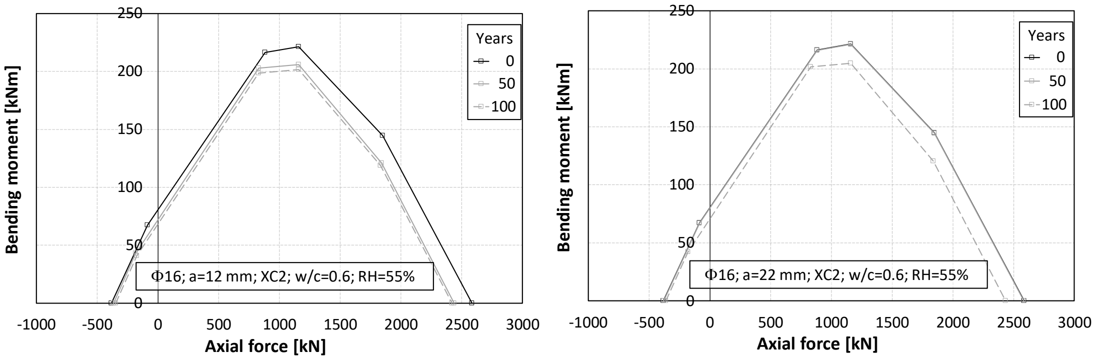
Finally, it is worth highlighting that the models summarised above are utilised in a deterministic way, with the aim to understand the influence of the relevant parameters of the member and structural response in RC frames. However, the paremeters controlling eqs. (1)-(4) need to be defined in statistical terms with the aim to cover both uncertainty and randomness affecting their predictions.

## 3 Consequences on capacity: N-M interaction curves of RC sections

The degradation model outlines in Section 2 can be employed with the aim to describe the time evolution of ultimate strength of a RC section. Specifically, the case of a 30x50 cm$^2$ RC section reinforced with a total of 10 longitudinal bars uniformly distributed along the perimeter is considered as a case study. Variable exposure conditions, bar diameters $\phi$ and concrete cover thickness $a$ (or, similarly, design cover d'=a+ϕ/2) are considered in the following parametric analysis. The effects of a variation of these parameters at the boundaries of the range considered in the present study is reported hereafter. For the sake of simplicity, a uniform degradation process is assumed both on all sides of the cross section and throughout the whole element length.

The results in terms of N-M interaction curves for the aforementioned RC section, reinforced by ϕ=16 mm rebars and fully exposed to XC2 conditions are reported in the following. C20/25 concrete and FeB38k rebars are in the analysis with the aim to reproduce typical material properties of existing RC buildings realised in the 70s' and 80s' of the last century that are suppose to be close to the end of their theoretical service life (i.e. 50 years, in ordinary buildings).

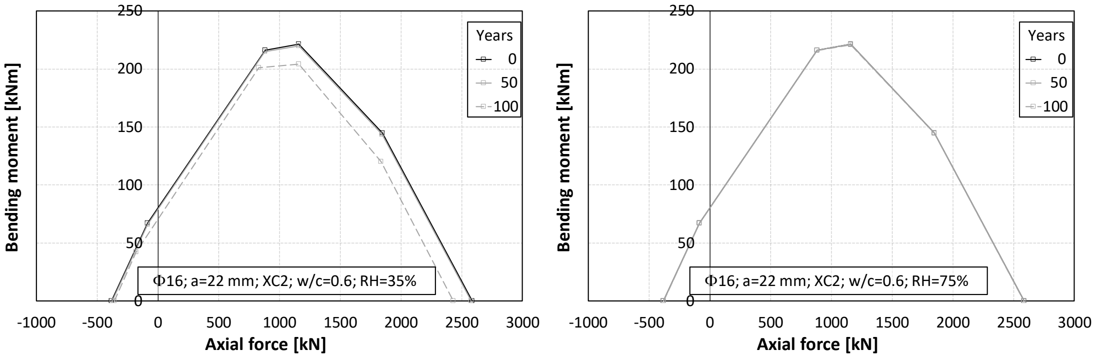
Fig. 3 shows the N-M curves determined after 50 and 100 years of service life, in comparison with the one determined for the sound section, in the two cases of thin and thick concrete covers; moreover, XC2 exposition class and RH=55% are assumed. The two N-M curves ofbtained for thin (d'=20 mm or a=12 mm, Fig. 3a) and thick (d'=30 mm or a=22 mm, Fig. 3b) concrete cover, confirm that the latter play a signifcant role in controlling the time evolution of RC sections. Specifically, premature degradation can be observed in Fig 3a after 50 years, wherease no substantial difference can be seen in Fig. 3b between the N-M curves referring to the sound section and the one after 50 years.



a) d'=20 mm b) d'=30 mm

Fig. 3 N-M interaction curves of rectangular RC section subjected to carbonation-induced degradation phenomena in concrete cove and steel rebars (XC2, RH=55%, $\phi$=16 mm)

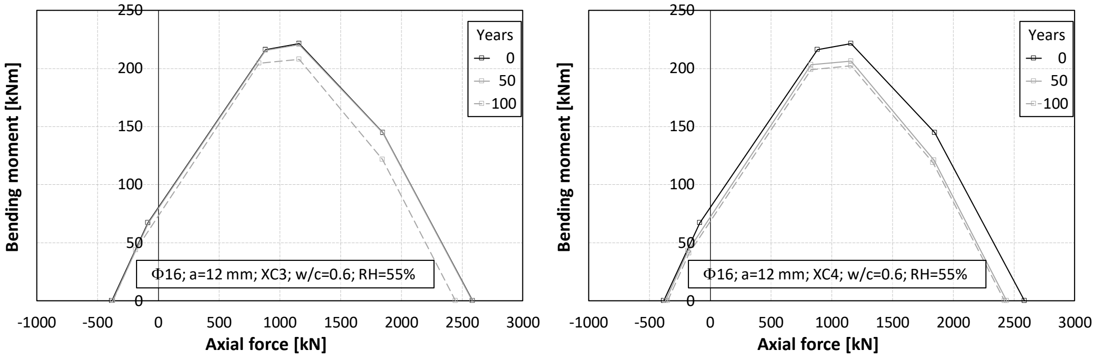
Fig. 4 shows the influence of the average relative humidity RH of the force capacity of the RC section with 30 mm of concrete cover. It shows a similar behaviour for both 35% (Fig. 4a) and 75% (Fig. 4b) with a slightly higher degradation observed for dry environment (RH=35%).



a) RH=35% b) RH=75%

Fig. 4 N-M interaction curves of rectangular RC section subjected to carbonation-induced degradation phenomena in concrete cove and steel rebars (XC2, d'=30 mm, $\phi$=16 mm)

Similar considerations can be derived by observing the time evolution of the RC section (this time assumed with a thin concrete cover) in the cases of exposure to XC3 (Fig. 5a) and XC4 (Fig. 5b).



a) XC3 b) XC4

Fig. 5 N-M interaction curves of rectangular RC section subjected to carbonation-induced degradation phenomena in concrete cove and steel rebars (RH=55%, d'=20 mm, $\phi$=16 mm)

Finally, it is worth highlighting that the premature degradation observed in the RC sections under consideration confirms that C20/25 concrete cannot generally guarantee durability in RC members exposed to XC2, XC3 and XC4 conditions, as they often show a non negligible strength degradation after 50 years of service life, especially in the case of thin concrete cover.

## 4 Seismic analysis: pushover analysis and capacity curves

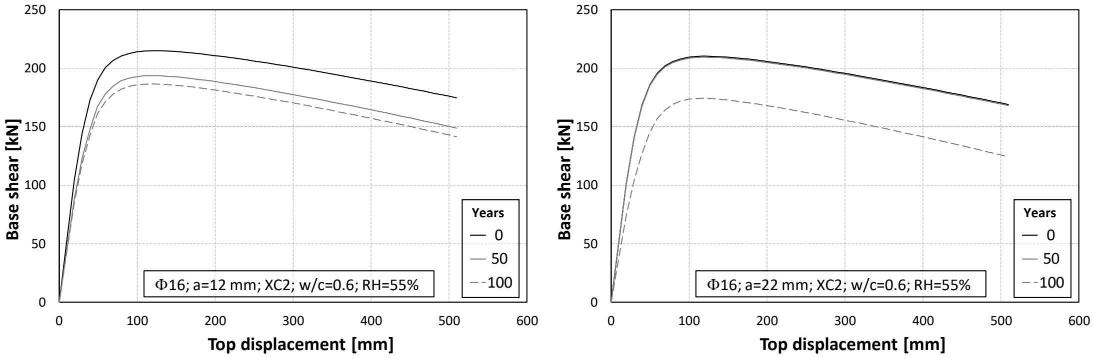
Degradation of RC section affects the resulting structural response of RC structures subjected to seismic actions. To quantify the influence of carbonation-induced degradation on the seismic capacity, a series of pushover analyses, run in OpenSEES [18], have been carried out at by considering reduced values of both steel reber area and concrete cover layer at 50 and 100 years, according to the degradation models outlined in Section 2 and already applied to the derive the time evolution of the N-M iiteraction curves in Section 3.

To do so, a 4 bay-4 storey RC plane frame is considered in the following with the aim to point out the influence carbonation-induced degradation on the seismic capacity of structures. They have uniform bay width of 4.50 m and story height of 3.50 m. For the sake of simplicity, the RC section already analysed in Section 3 is assumed for beams and columns. Static pushover analyses are carred out by considering both the sound transverse sections and the ones affected by degradation possibly developed after 50 and 100 of service life. Degradation of materials, which can be represented by a reduction in both concrete cocer thickness and steel rebars area, can be easily implemented by modifying the geometric parameters of the fiber discretisation of transverse section apotped in the OpenSEES model.

Fig. 6 shows the results of pushover analyses carried out on the frame under consideration exposed to XC2 conditions and RH=55%. It reports two graphs referred to either thin (d'=20 mm, Fig. 6a) or thick (d'=30 mm, Fig. 6b) concrete cover.

The pronounced capacity degradation already observed on the generic RC section with 20 mm concrete cover (Fig. 3a) reverberates its effects upwards at the structural scale, as the pushover curves represented in Fig. 6a show a significant reduction in lateral stiffness and maximum force already for 50 years. In fact, in this case, cracking is expected to occur at after 19 years and cover is expected to spall out after 39 years; then, the 50-year pushover curve already takes into account both phenomena, whereas the further reduction observed between 50 and 100 years is only due to corrosion that keeps developing in steel rebars.

Conversely, in the case of 30 mm concrete cover (Fig. 6b) cracking is expected to initiate at after 56 years and, hence, only a slight corrosion effect can be seen between the two (almost overlapped) curves referred to the sound structure and the one after 50 years of service life. Then, although concrete spalling is expecetd to occur later on (afer 76 years), it results in a more significant reduction of the transverse section which leads to a pushover curve (the dashed grey line) that is even lower than the corresponding one in Fig. 6a.

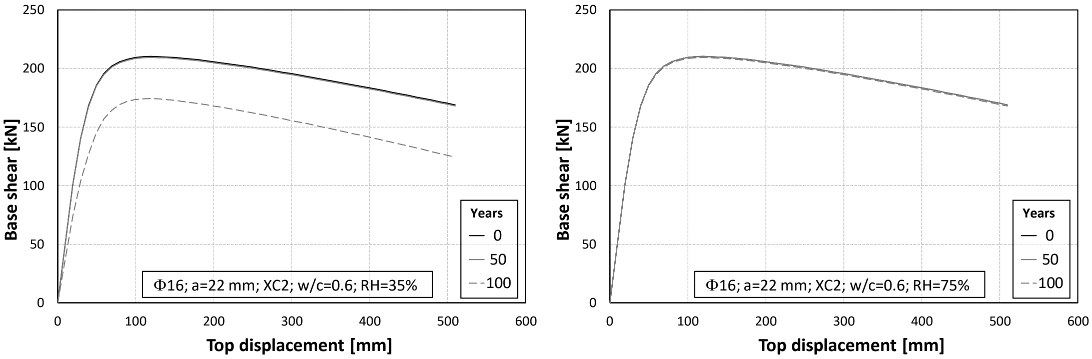


a) d'=20 mm b) d'=30 mm

Fig. 6 Time evolution of the capacity curve for a 4 bay-4 storey RC frame (XC2, RH=55%)

Fig. 7 aims at pointing out the role of relative humidity on the time evolution of pushover curves. Specifically, the cases of dry (RH=35%, Fig. 7a) and humid (RH=75%, Fig. 7b) environmental conditions are considered, as already assumed in Fig. 4 with the aim to investigate the time evolution of the force capacity in the generic RC section.

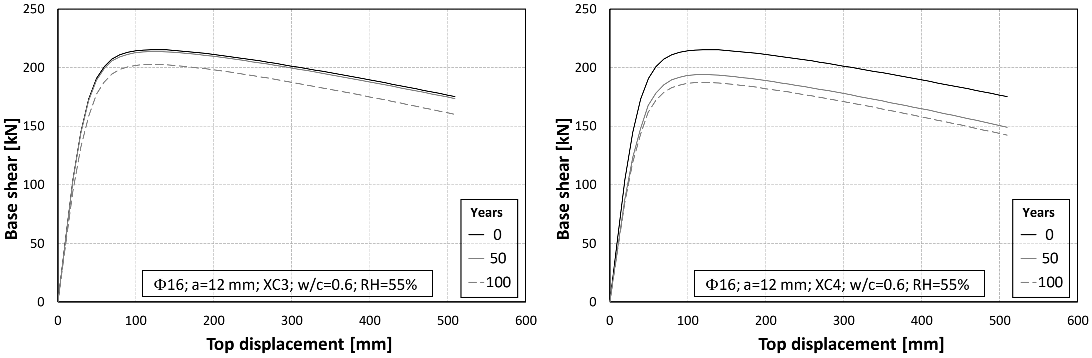
Fig. 7a shows no substantial differences with respect to the Fig. 6b, the latter being obtained for RH=55%. Conversely, Fig. 7b confirms that higher values of relative humidity have a delation effect of the penetration of carbonation and, consequently, on the degradation of both the RC section strength and structure capacity. In fact, in this case depassivation is expected to occur after 94 years and, hence, at 100 years the structure is supposed to be almost unaffected by corrosion.



a) RH=35% b) RH=75%

Fig. 7 Time evolution of the capacity curve for a 4 bay-4 storey RC frame (XC2, d'=30 mm)

Finally, Fig. 8 shows the structural scale counterpart of Fig. 5, as it analyses the effect of different exposure conditions (namely, XC3 and XC4). Similar considerations can be done for the former as they are compared to the corresponding N-M interaction curves represented as part of the latter.



a) XC3 b) XC4

Fig. 8 Time evolution of the capacity curve for a 4 bay-4 storey RC frame (RH=55%, d'=20 mm)

## 5 Conclusions

This paper aims at investigating the influence of carbonation-induced degradation phenomena on the structural performance of RC frames. The time evolution of both carbonation diffusion inside concrete and corrosion affecting oxidised steel rebars results in three relevant stages of the RC section response, namely depassivation, concrete cracking and cover spalling, which are progressively achieved over time, depending on both environmental conditions and relevant geometric and material properties.

The paper presented a concise overview of the available models intended at simulating these phenomena, along with a practice-oriented mathematical description of the evolution (generally step-wise linear in time) of relevant geometric properties, such as effective concrete cover thickness and radius loss in steel rebars. The consistency the proposed approach has been demonstrated by reporting some results of analyses carried out at both the member- and structural-scales for an RC frame.

As for the member-scale analysis, N-M interaction curves have been drawn for the considered RC section in the various stages of their degradation configuration and for some conventional service values of life (namely, 50 and 100 years). The reported results show that degradation leads to significant reduction in terms of section capacity subjected to normal stresses (expressed by axial loads and benfing moments): concrete cover can delay the development of degradation that, however, is mainly influenced by the actual environmental conditions which the element is exposed to.

Moreover, the effects of degradation phenomena at the global level have been investigated through pushover analyses, which are generally adopted to simulate the behavior of frames subjected to earthquake actions. A 4 bay-4 storey RC frame has been considered in the present study and the time evolution of its capacity curves under different environmental conditions (described in terms of Relative Humidity and Exposure Class) has been investigated. A non negligible loss in terms of lateral strength of frames has been observed and this sheds a new and concerning light on the actual level of safety of existing structures, which are often affected by the degradation phenomena considered in this work.

Finally, the results obtained in all the proposed analyses outline that the effect of corrosion induced by carbonation-induced degradation on RC sections, members and frames cannot be generally neglected. Thus, further studies are needed to achieve a comprehensive quantification of such effects and formulate sound design/assessment criteria aimed at preserving safety in RC structures exposed to aggressive environmental conditions.

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