

ARTICLE

Service life of reinforced concrete structures made with blended cements and exposed in urban environment

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Abstract

Carbonation-induced corrosion is one of the main causes of degradation of reinforced concrete (RC) structures exposed outdoor in urban environment. To prevent steel corrosion, a durability design, that considers both the initiation and the propagation time, is of fundamental importance. At this aim, the resistance to carbonation of concrete and the steel corrosion rate in the exposure environment need to be known. This paper reports the carbonation coefficient and the corrosion rate of 7-day cured RC specimens made with different binders and exposed outdoor in Milan in unsheltered conditions. Corrosion rates in laboratory conditions with different temperatures and relative humidities are also reported. Experimental data were used to evaluate the service life in unsheltered condition. RC specimens made with Portland cement exhibited the lowest carbonation coefficient and corrosion rate, while specimens with 30% limestone and with 70% ground granulated blast furnace slag the highest.

KEYWORDS

blended cement, carbonation, corrosion rate, service life, unsheltered conditions

1 | INTRODUCTION

One of the major problems faced by reinforced concrete (RC) structures is the corrosion of steel reinforcement induced by carbonation.¹ To prevent the steel corrosion, a durability design is needed and, for carbonation-induced corrosion, both the initiation and the propagation period have to be considered, since both contribute to define the duration of the service life. In such exposure conditions, the alternating wet and dry cycles foster the penetration of carbonation during the dry periods and—

after depassivation—an active corrosion propagation during the wet periods, reducing the duration of the service life. For the design, it is, hence, of fundamental importance, to know both the resistance to carbonation of concrete and the corrosion rate of steel, which are key parameters in defining the duration respectively of the initiation and propagation time. These parameters depend on many factors, among which the exposure conditions and the type of concrete.

Several studies have been carried out to evaluate the resistance to carbonation and the corrosion rate in concrete made with different types of binder. However, these studies are often performed in conditions which are not fully representative of natural exposure.

Carbonation is often evaluated through accelerated tests that are characterized by a duration of the order of

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months and by a CO₂ concentration that is significantly higher than that in atmosphere.²⁻⁵ Those studies have shown that concretes with pozzolanic and hydraulic additions, such as ground granulated blast furnace slag (GGBS) and fly ash, as well as limestone cement have a lower carbonation resistance in comparison to Portland cement concrete, leading, therefore, to a shorter initiation period for carbonation. However, the results of accelerated tests cannot be directly used to design the service life of a RC structure, since they do not take into account the real exposure conditions, for example, the actual relative humidity and temperature. Nonetheless, the behavior of blended cements was confirmed by natural exposure tests.⁶⁻¹¹

As far as corrosion rate is concerned, since the 1980s, several researches have been performed on carbon steel bars embedded in carbonated concrete and mainly exposed to laboratory conditions with different temperatures and relative humidities. Portland cement was often used, while blended cements were seldom considered, especially natural pozzolan, limestone, and silica fume. Very few works allow a direct comparison between concretes made with Portland cement and blended cements.¹²⁻¹⁷ Usually the studies showed higher corrosion rate in blended cement compared to Portland cement.

No publication known to the authors is related to both carbonation and corrosion rate performed on the same concretes. This would allow to properly assess the effect of the environmental exposure conditions and the type of binder on the service life of a RC structure and, hence, to choose the most appropriate binder to guarantee the target durability requirement.

This paper reports results of concrete carbonation and steel corrosion rate obtained on specimens made with different binders, that is, Portland, limestone, fly ash, pozzolan, and GGBS, different water/binder ratios and cured for 7 days and exposed outdoor in Milan in unsheltered conditions. Corrosion rate was also evaluated in laboratory conditions characterized by different temperatures and relative humidities. Experimental data were used to evaluate the service life in unsheltered conditions by means of a probabilistic approach.

2 | MATERIALS AND METHODS

Concretes with three different water/binder ratios, equal to 0.42, 0.46, and 0.61, and four different binder dosages, ranging from 250 to 400 kg/m³ were made with a Portland cement CEM I 52.5R (OPC), according to EN 197-1 standard, and five blended cements. In blended cements, part of the Portland cement was replaced with 15% and

30% of ground limestone (15LI and 30LI), 30% of fly ash of class F (FA), 30% of natural pozzolan (PZ), and 70% of GGBS. Crushed limestone aggregate with maximum size of 16 mm was used, and an acrylic superplasticizer was added to the mixtures to achieve a class of consistence *S4* according to EN 206 standard. Table 1 summarizes the main characteristics of the concrete mixes. Compressive strength and the accelerated carbonation coefficient evaluated on cubes cured for 7 days, whose results were discussed elsewhere^{18,19} are also reported in Table 1. Each concrete is defined through a label that reports the type of cement, the water/binder ratio, and the binder content (i.e., OPC-0.61-300 indicates a concrete made with Portland cement, water/binder ratio of 0.61, and a binder content of 300 kg/m³).

After mixing, concretes were cast in cubic and reinforced prismatic molds, covered with a plastic foil and stored in laboratory at 20°C. After 24 h, specimens were demolded and cured, for 6 further days, at 20°C and 95% relative humidity (R.H.). Unreinforced (i.e., cubic) and reinforced specimens were made at the same time, with the same concrete batch.

One hundred millimeter cubic specimens were used to evaluate carbonation (one specimen for each type of concrete). After curing, they were exposed outdoor in natural conditions, unsheltered from rain and sun—on the roof of the Department of Chemistry, Materials and Chemical Engineering of Politecnico di Milano. Milan average annual temperature is around 13°C, with minimum and maximum average monthly values around -1°C and 30°C. The annual rainfall is about 1000 mm, while the average annual relative humidity is around 75% with minimum and maximum average monthly values around 70% and 85%. The number of days in a year with daily cumulative rainfall higher than 0.2 mm is around 130, and, often, they are consecutive days. It can be estimated that, in a year, the periods of wetting are about 25-35, assuming that aforementioned periods have, at least, two consecutive days with rainfall higher than 0.2.

Four faces of the cubes were masked with epoxy and carbonation was allowed to penetrate only from the cast and a mold surface, which were, during exposure, oriented vertically. After different exposure times up to 1.5 years of exposure, a 20 mm passing-core was drilled and the average value, determined as average between the minimum and the maximum values, of carbonation depths was measured, from the mold surface, with the phenolphthalein test (cores taken at different times were distanced at least 200 mm from each other). After about 15 years of exposure, some specimens were split and the average carbonation depth was measured on the fracture surface. Due to mislabeling caused by the outdoor

TABLE 1 Mix proportion of concrete mixes and results of compressive strength, $f_{c,cube}$ and accelerated carbonation coefficient, k_{acc} evaluated on 7-day cured concretes^{18,19}

Series	OPC (%)	Label	Water/binder ratio	Water (kg/m ³)	Binder (kg/m ³)	Cement (kg/m ³)	Aggregates (kg/m ³)	$f_{c,cube,7}$ (MPa)	$k_{acc,7}$ (mm/year ^{0.5})		
OPC	100	OPC-0.61-300	0.61	183	300	300	1857	45	21.2		
		OPC-0.46-300	0.46	138	300	300	1979	62	12.8		
		OPC-0.46-350	0.46	161	350	350	1868	76	12.9		
		OPC-0.42-350	0.42	147	350	350	1913	83	7.3		
15LI	85	15LI-0.61-250	0.61	152	250	212.5	1983	44	29.5		
		15LI-0.61-300	0.61	183	300	255	1857	38	26.6		
		15LI-0.46-300	0.46	138	300	255	1979	60	11.8		
		15LI-0.46-350	0.46	161	350	297.5	1868	65	12.1		
		15LI-0.42-350	0.42	147	350	297.5	1913	68	11.2		
15LI		15LI-0.42-400	0.42	168	400	340	1815	67	13.6		
		30LI	70	30LI-0.61-300	0.61	183	300	210	1857	31	34.6
				30LI-0.46-300	0.46	138	300	210	1979	47	20.7
				30LI-0.46-350	0.46	161	350	245	1868	49	23.2
				30LI-0.42-350	0.42	147	350	245	1913	57	12.3
FA	70	FA-0.61-300	0.61	183	300	210	1857	30	30.9		
		FA-0.46-300	0.46	138	300	210	1979	52	24.9		
		FA-0.46-350	0.46	161	350	245	1868	52	22.5		
		FA-0.42-350	0.42	147	350	245	1913	66	13.2		
PZ	70	PZ-0.61-300	0.61	183	300	210	1857	35	32.7		
		PZ-0.46-300	0.46	138	300	210	1979	54	18.3		
		PZ-0.46-350	0.46	161	350	245	1868	55	16.6		
		PZ-0.42-350	0.42	147	350	245	1913	54	13.4		
GGBS	30	GGBS-0.61-300	0.61	183	300	90	1857	35	25		
		GGBS-0.46-300	0.46	138	300	90	1979	46	24		
		GGBS-0.46-350	0.46	161	350	105	1868	59	16.7		
		GGBS-0.42-350	0.42	147	350	105	1913	76	13.4		

Abbreviation: GGBS, ground granulated blast furnace slag.

exposure, after 15 years of exposure, carbonation depth was determined only on 15LI, FA, and GGBS concretes. The average carbonation depths, d , measured after different exposure times, t , were interpolated, with the least squares method, to obtain the natural carbonation coefficient, $k_{unsheltered}$:

$$d = k_{unsheltered} \cdot \sqrt{t} \quad (1)$$

Prismatic specimens, 60 mm × 250 mm × 150 mm, were reinforced with three ribbed carbon steel bars with a diameter of 10 mm and a cover depth of 10, 25, and 40 mm. Stainless steel wires, to be used as auxiliary electrodes in the electrochemical measurements, were placed near the rebars (Figure 1). For reinforced specimens,

concretes with only water/binder ratio of 0.61 were made and, for each concrete, two replicate specimens were cast.

After curing and 2 weeks in laboratory conditions, specimens were exposed to accelerated carbonation, with 100% of CO₂, until they were fully carbonated (carbonation was periodically verified with phenolphthalein tests on concrete cores taken from the specimens). The lateral surfaces of the specimens and the external parts of the rebars were then covered with an epoxy coating and one specimen for each concrete was exposed, for about 2 years, outdoor, with the same exposure conditions as the cubic specimens, while the other specimen was exposed to controlled cycles of temperature and relative humidity (the top and the bottom surfaces of the specimens were left uncovered). In particular, the following cycles of temperature and relative humidity were carried

out: $T = 20^\circ\text{C}/\text{R.H.} = 95\%$, $T = 20^\circ\text{C}/\text{R.H.} = \text{submerged}$, $T = 20^\circ\text{C}/\text{R.H.} = 90\%$, $T = 40^\circ\text{C}/\text{R.H.} = 90\%$, $T = 20^\circ\text{C}/\text{R.H.} = 80\%$, $T = 40^\circ\text{C}/\text{R.H.} = 80\%$.

During the exposure, electrochemical measurements of half-cell potential of steel (E_{corr}) versus a saturated calomel electrode (SCE), placed on the top specimen surface in the central part of each bar, and linear polarization resistance measurements (R_p) were performed to monitor the corrosion behavior of steel. Corrosion current density, i_{corr} , was determined from R_p measurements as: $i_{\text{corr}} = B/R_p$, where B was assumed equal to 26 mV. Measurements of conductivity were also carried out between the two wires placed at the different depths, to investigate the concrete electrical resistivity. Cycles were changed, usually after 2–4 weeks, when electrochemical measurements were steady, indicating that stable conditions were reached.

3 | RESULTS AND DISCUSSION

The role of concrete characteristics on the carbonation coefficient in unsheltered natural environment, mainly the type of binder and the water/binder ratio, will be first presented and discussed, also in relation with other properties of the hardened concrete, that is, the compressive strength and the accelerated carbonation coefficient. Then the corrosion behavior of steel bars embedded in carbonated concrete made with the same binders and exposed both in natural conditions and in controlled cycles of T and R.H. will be described. Finally, experimental results of carbonation and corrosion rate will be used to evaluate the impact of the concrete characteristics on the service life of a RC element exposed in unsheltered conditions, by means of a probabilistic approach.

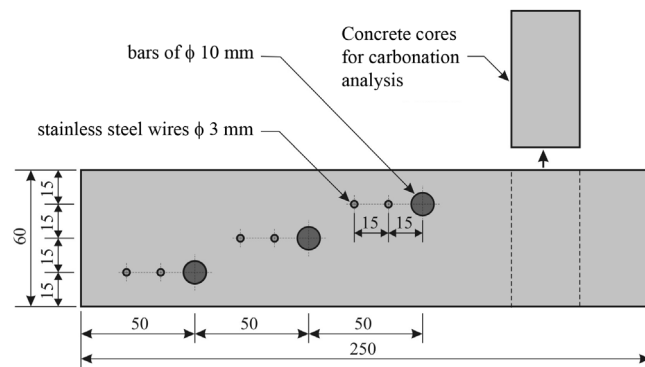


FIGURE 1 Geometry of reinforced concrete specimens (dimensions in millimeters)

3.1 | Carbonation of concrete

Figure 2 shows, as an example, the trend with time of the carbonation depth on concretes with 15% of limestone, moist cured for 7 days and exposed in an outdoor unsheltered environment. Increasing the exposure time, the carbonation depth increased and the highest values were observed on concretes with the highest water/binder ratio; a significant effect of the binder content was not observed. After almost 15 years of exposure, carbonation depths of the order of 7–12 mm were measured on the concretes made with water/binder ratio of 0.61, between 4 and 6 mm on the concretes made with water/binder ratio of 0.46 and around 2.5 mm on the concretes with water/binder ratio of 0.42.

Carbonation depths were fitted through the relationship (1) to determine the carbonation coefficient $k_{\text{unsheltered}}$, and the fitting lines are also reported in Figure 2.

Figure 3 shows the carbonation coefficient as a function of the water/binder ratio and the type of binder (the effect of the binder content was neglected). For each type of binder, the carbonation coefficient and the water/cement ratio were fitted through an exponential relationship and a good relationship can be observed. For instance, for GGBS concretes, the carbonation coefficient increased from 1.5 to 3.6 mm/year^{0.5} when the water/binder ratio increased from 0.42 to 0.61. In Figure 3, the role of the type of binder can be also observed. The OPC concretes showed the lowest carbonation coefficient, with values ranging from 0.35 to 0.78 mm/year^{0.5} when the water/binder ratio varied between 0.46 and 0.61. These values were slightly lower than those evaluated on PZ concretes. It is worth noting that for both types of binder,

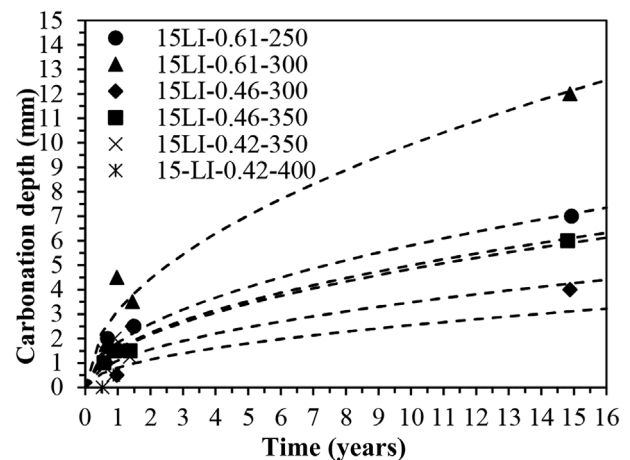


FIGURE 2 Carbonation depth as a function of time of concretes made with 15% limestone cement and exposed to outdoor unsheltered conditions

measurements were carried out only after relatively short exposure times and carbonation depths of only few millimeters were measured, which might have led to a poor estimation of the carbonation coefficients. A higher carbonation coefficient in comparison with OPC concretes can be observed on concretes made with 15LI, FA, GGBS, and 30LI. For instance, the carbonation coefficient of concretes made with water/binder ratio of 0.61 was 1.8, 2.2, 3.6, and 3.6 mm/year^{0.5}, respectively for 15LI, 30LI, FA, and GGBS. Results obtained on the different types of concrete are consistent with those obtained in the studies where the exposure sites had similar climatic conditions (e.g., Lyon), despite the different curing regime.^{7,11} Limiting to concrete made with limestone, values of carbonation rate are consistent with average data obtained within the “Validation testing program on chloride penetration and carbonation standardized test methods” of CEN/TR 17172 on concretes made with CEM II-A/LL—42.5R and water/cement ratio of 0.49 and 0.58 and exposed up to 2 years outdoor in different European locations.²⁰ Higher carbonation rates, at least on OPC concrete, were, conversely, obtained in tropical climate, characterized by highest temperature,²¹ suggesting that temperature, but also relative humidity and time of wetness have a significant impact on carbonation.

To better investigate the impact of the binder on carbonation coefficient, Figure 4 shows the ratio between the carbonation coefficient measured on the concretes made with blended cement, k_{blended} , and the carbonation coefficient measured on OPC concretes, k_{OPC} , at equal water/binder ratio and binder dosage. The $k_{\text{blended}}/k_{\text{OPC}}$ ratio was always higher than 1 (PZ-0.42-350 concrete was an exception). For some water/binder ratios and types of binder, a double, and in some case even higher, value of

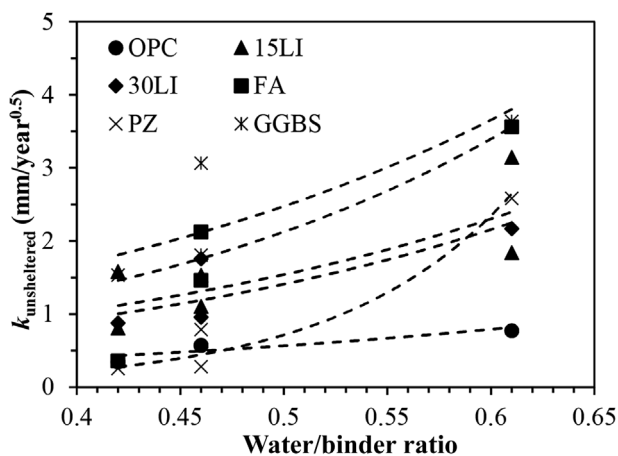


FIGURE 3 Carbonation coefficient in unsheltered environment as a function of the water/binder ratio for different types of concrete cured for 7 days

the carbonation coefficient was observed. The highest ratios were detected for 30LI, while the lowest for PZ, however, as previously observed, for these concretes only data after 1.5 years of exposure were available. These results are in agreement with data reported in the literature, showing a lower carbonation coefficient of concretes with Portland cement in comparison with concretes with pozzolanic and hydraulic binders, that is, natural pozzolan, fly ash, and GGBS and limestone binders.

Correlations between the carbonation coefficient in a natural environment and other hardened concrete properties that are usually easily measured, such as the compressive strength and the accelerated carbonation coefficient, were explored. Figure 5 shows the relationship between the compressive strength and the carbonation coefficient both evaluated on concrete cured 7 days. In general, concretes with a higher compressive strength showed also a lower carbonation coefficient, hence a higher resistance to carbonation. A high variability was detected; however, data seems to be correlated through an exponential relationship, regardless the type of cement, suggesting that, in absence of data, carbonation could be evaluated from compressive strength (Figure 5a). Naturally this correlation depends on the specific climatic conditions where the specimens were exposed. Figure 5b shows the relationship between the accelerated carbonation coefficient obtained through accelerated tests (Table 1), k_{acc} , and the carbonation coefficient measured on specimens exposed outdoor unsheltered from rain, $k_{\text{unsheltered}}$, for the different types of concrete. This correlation, determined for the specific climate conditions considered in the study, is useful since usually the behavior of concrete in a certain exposure condition is evaluated from accelerated tests. Significant

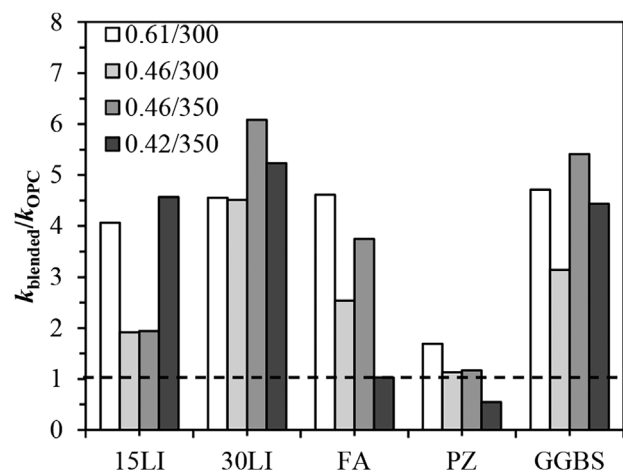


FIGURE 4 Ratio between the carbonation coefficient of blended cement concretes and Portland cement concretes, both in unsheltered environment

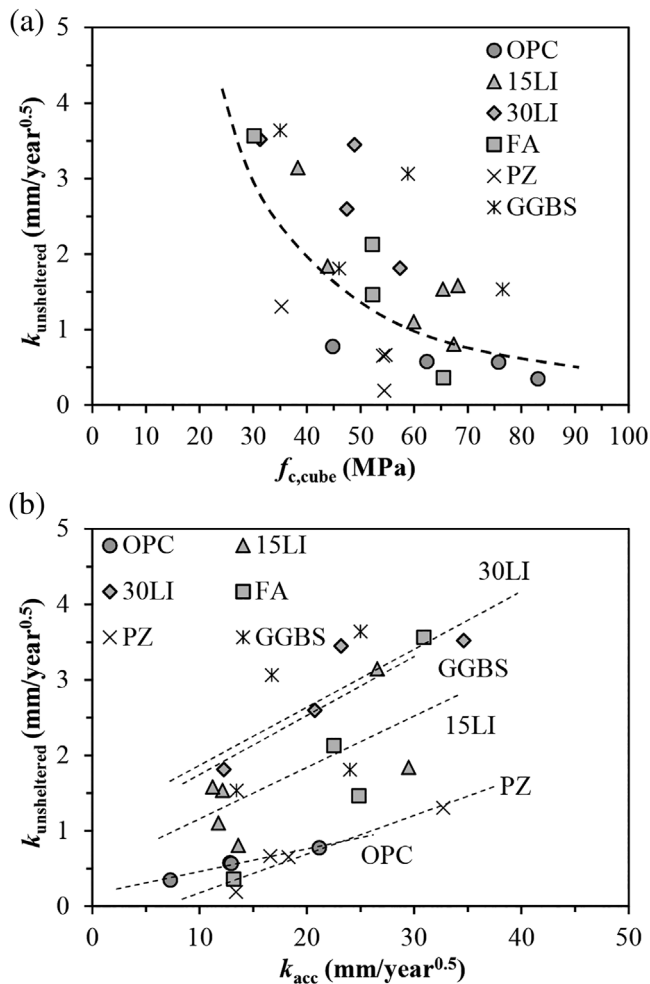


FIGURE 5 Relationship between the natural carbonation coefficient and the compressive strength (a) and the accelerated carbonation coefficient (b) evaluated on 7-day cured specimens

differences, of one order of magnitude, between the accelerated carbonation coefficient and the unsheltered carbonation coefficient can be observed. For instance, the carbonation coefficient measured on 30LI-0.61-300 concrete increased from about 3.5 to around 35 $\text{mm/year}^{0.5}$ when exposure varied between natural and accelerated conditions. Despite the variability of data, a correlation can be determined between k_{acc} and $k_{unsheltered}$, slightly dependent on the type of cement. This suggests that the carbonation coefficient can be estimated from results of accelerated tests; however, it should be taken into account that the correlation is affected by the actual exposure conditions and the cement type and cannot be generalized.

3.2 | Corrosion rate of steel bars in carbonated concrete

Figure 6 shows, as an example, the variation in time of corrosion potential and corrosion current density of the

three rebars embedded at different depths in concrete made with 15LI exposed to controlled cycles of temperature and relative humidity. Resistivity at the same bars depth is also reported. At the beginning of the exposure, that is at the end of the exposure in the accelerated carbonation chamber ($T = 20^\circ\text{C}$, R.H. = 65%), at the depth of the three rebars high electrical resistivity were measured and quite low corrosion current density and quite high corrosion potential on the three bars were determined. The exposure to high relative humidity, that is, 95%, led to a decrease of the electrical resistivity, that approached values of the order of 1000–2000 $\Omega\cdot\text{m}$, an increase of the corrosion current density which, however, remained lower than 2 mA/m^2 and a decrease of the corrosion potential. When exposed to the other controlled environments, all the parameters experienced some variations. Significant modifications occurred in submerged condition, where the highest corrosion current density, of the order of 10 mA/m^2 , the lowest electrical resistivity, of the order of 100 $\Omega\cdot\text{m}$, and the lowest corrosion potential were determined. No significant differences on E_{corr} , i_{corr} , and ρ were observed, in all the environmental conditions, among the bars embedded at the different depths, suggesting that similar humidity and corrosion conditions were present.

To investigate the effect of the exposure conditions and concrete composition on the corrosion potential, corrosion current density, and concrete electrical resistivity, the average values and the range of variability were evaluated for each concrete and condition. Average values of the three parameters were evaluated considering values obtained on all the three rebars and the variability was evaluated considering the maximum and minimum values reached in each exposure condition. Figure 7 shows the average values and the range of variation of E_{corr} , i_{corr} , and ρ as a function of relative humidity and temperature; values obtained outdoor are also included. A significant effect of the relative humidity on the three parameters can be observed. The corrosion potential decreased when the R.H. increased to 95% and in submerged conditions, where values between -750 and -650 mV versus SCE were measured. At 80% R.H., the corrosion potential was higher than -250 mV versus SCE (Figure 7a). Corrosion current densities lower than 1–2 mA/m^2 were determined for R.H. lower than 90%; conversely i_{corr} increased with R.H., reaching values between 3 and 10 mA/m^2 when the specimens were submerged (Figure 7b). Concrete electrical resistivity, as expected, showed the lowest values, between 130 and 480 $\Omega\cdot\text{m}$, when concrete was submerged, and the highest values, between about 4000 and 6800 $\Omega\cdot\text{m}$ when the concretes were exposed to 80% R.H. (Figure 7c). To better illustrate the effect of relative humidity, Figure 8 shows

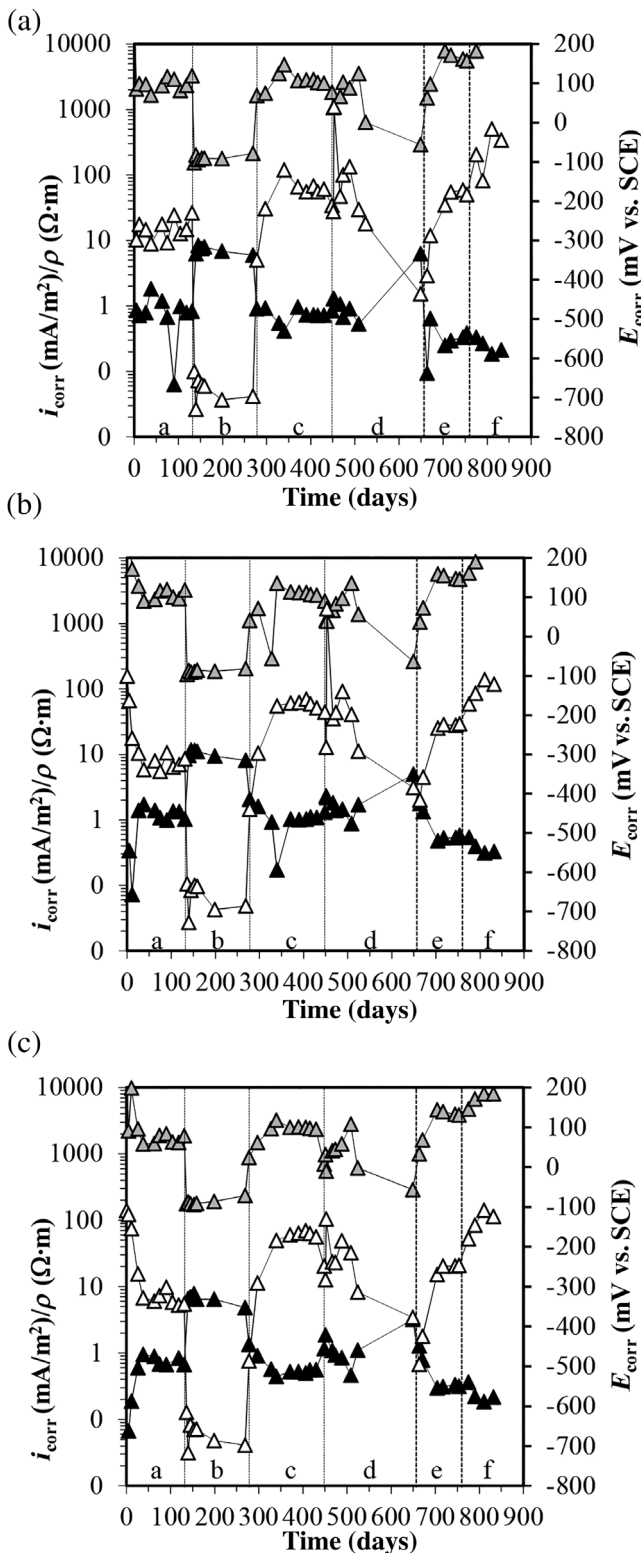


FIGURE 6 Trend in time of the corrosion potential (white symbols), corrosion current density (black symbols) and concrete electrical resistivity (gray symbols) measured at different depths (upper depth, a; middle depth, b; lower depth, c) in concrete made with 15LI after carbonation exposed to different environmental conditions: $T = 20^\circ\text{C}/\text{R.H.} = 95\%$, a; $T = 20^\circ\text{C}/\text{R.H.} = \text{submerged}$, b; $T = 20^\circ\text{C}/\text{R.H.} = 90\%$, c; $T = 40^\circ\text{C}/\text{R.H.} = 90$, d; $T = 20^\circ\text{C}/\text{R.H.} = 80\%$, e; $T = 40^\circ\text{C}/\text{R.H.} = 80\%$, f

the corrosion current density and the resistivity as a function of R.H. (for submerged conditions, an arbitrary value was assumed). It can be observed that i_{corr} increased when the environmental relative humidity increased, while ρ decrease increasing the relative humidity. This clearly indicates that both parameters are affected by the water content in concrete.

Temperature played an effect when R.H. was 90%. For instance, corrosion current density increased from values between 0.4 and 1 mA/m^2 to values between 1.4 and 3.9 mA/m^2 when T increased from 20 to 40°C . Conversely, when R.H. was 80%, low corrosion current density was detected both at temperatures of 20 and 40°C . Also for electrical resistivity a significant variation was observed only when R.H. was 90%; for instance, the electrical resistivity decreased from values between 1700 and 3600 $\Omega\cdot\text{m}$ to values between 215 and 900 $\Omega\cdot\text{m}$ when temperature increased from 20 to 40°C . In outdoor unsheltered conditions, E_{corr} , i_{corr} , and ρ were intermediate to those detected in the controlled laboratory conditions, indicating that the typical exposure conditions that can occur, at least in the temperate climate of Milan, were simulated through the selected cycles of temperature and relative humidity. In outdoor conditions, only the combined effect of relative humidity, raining events, and temperature can be detected. The highest values of corrosion current density, which were measured in outdoor conditions during the rainiest periods, were similar to the highest values measured in submerged conditions. The flat exposure of the specimens in outdoor conditions fostered their wetting during rain events, leading the concrete to conditions similar to saturation. Conversely, the lowest values, which were measured during drying period, were comparable with those measured with R.H. of 80%. A variability of corrosion rate was also observed in other works where specimens were exposed outdoor as shown in the review paper of Stefanoni et al.²² or in the experimental work carried out by Andrade et al.,²³ where beams with mixed-in chlorides were exposed in a climate with dry atmosphere in summer, mild autumn, spring, and winter.

From Figure 7, the influence of the type of binder on the corrosion behavior of carbon steel bars and on the concrete electrical resistivity can be assessed. Portland cement concrete showed, both in laboratory and in outdoor conditions, the lowest corrosion current densities: for instance, in the more aggressive conditions, that is, the submerged ones, average values of the order of 3 mA/m^2 were measured on OPC concrete, whilst values even double were obtained on concretes with blended cement (Figure 7b). Also the highest values of concrete electrical resistivity were measured on OPC concrete. The FA concrete showed, in the different controlled exposure environments, higher resistivity and lower corrosion

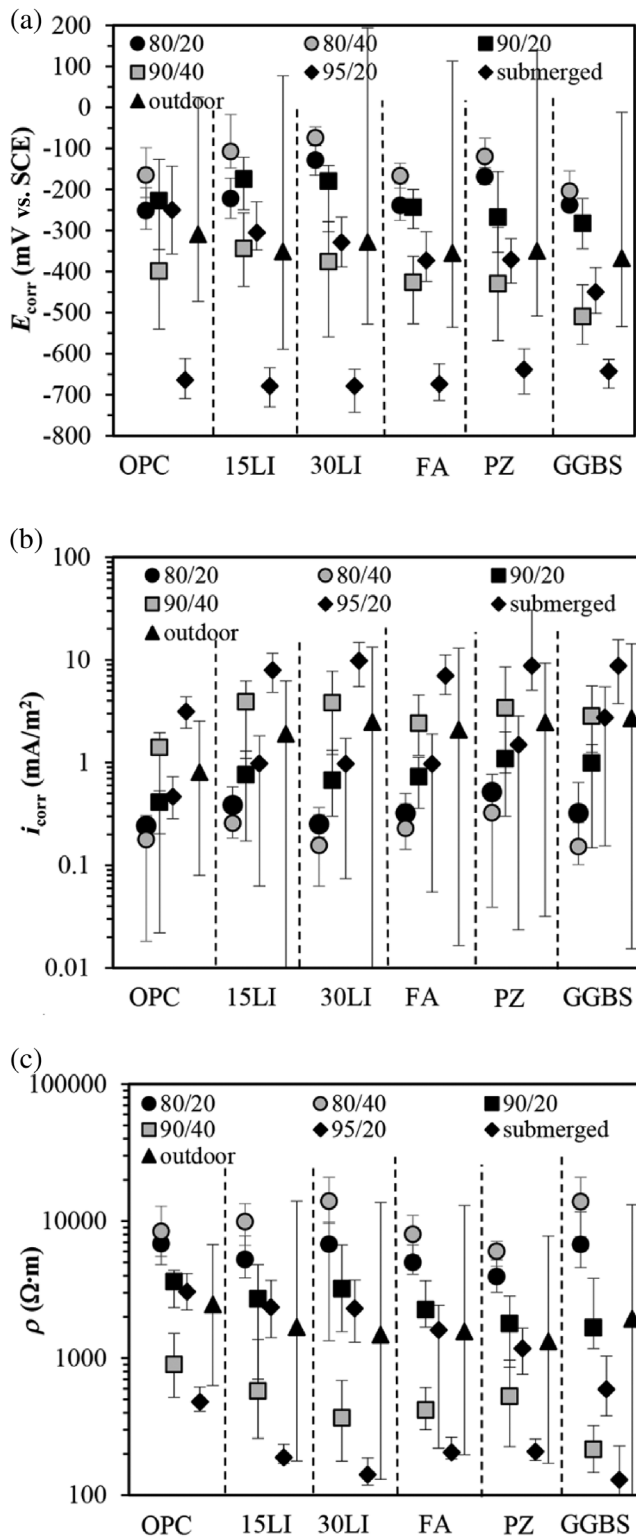


FIGURE 7 Effect of concrete composition and exposure environment conditions on the average values and range of variability of corrosion potential (a), corrosion current density (b), and concrete electrical resistivity (c)

current density in comparison to 15LI, 30LI, PZ, and GGBS concretes. Corrosion rate obtained on bars embedded in OPC concrete was comparable to those

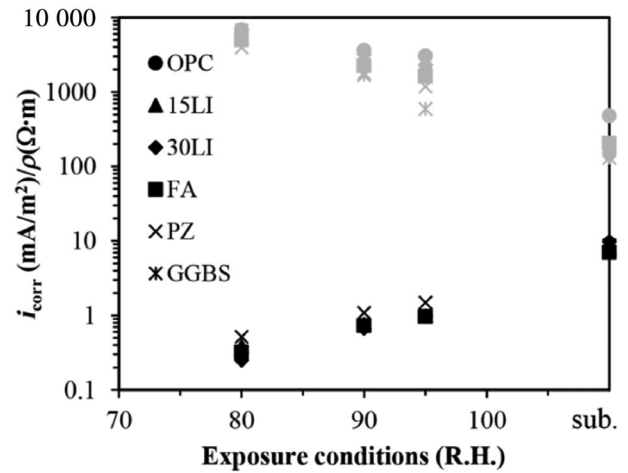


FIGURE 8 Corrosion current density (black symbols) and concrete electrical resistivity (gray symbols) as a function of relative humidity

obtained in the study carried out by Américo and Nepomuceno.²⁴ Lower values of corrosion rate were obtained on bars in FA, GGBS, and PZ concretes in comparison to those reported in the works of Alonso and Andrade,¹² Alonso et al.,^{13,14} and Andrade and Buják.¹⁵

3.3 | Estimation of the service life

Results presented in the previous sections highlighted that concrete characteristics, and in particular, the type of binder, have an impact on the concrete carbonation and corrosion rate. This means that the service life of a RC element can be significantly affected by the type of binder used to cast concrete and that certain types of binder might not be suitable to guarantee, for instance, long service life. To help the designers in detecting the most suitable type of concrete to be used for an element exposed in unsheltered condition, at least in climate conditions similar to that of Milan, the service life that can be reached with the different binders employed in this study was evaluated, by means of a probabilistic approach, starting from the experimental data. For structures exposed in urban environment, the initiation time can be defined as the time required for carbonation to reach the depth of the outermost steel bars, while the propagation period can be defined as the time required to consume the steel bars of an amount that leads to the cracking of the concrete cover. In the probabilistic approach, for the initiation period, the probability of failure, $p_{f,i}$, was evaluated as the probability that the initiation limit state function, g_i , reaches negative values:

$$p_{f,i} = P\{g_i < 0\} = P\{x - k_{\text{unsheltered}} \cdot \sqrt{t_i} < 0\} \quad (2)$$

where x is the concrete cover thickness and t_i is the initiation time. For the propagation period, $p_{f,p}$ was evaluated as:

$$p_{f,p} = P\{g_p < 0\} = P\{P_{\text{lim}} - v_{\text{corr}} \cdot t_p < 0\} \quad (3)$$

where P_{lim} is the limit penetration of corrosion, v_{corr} is the corrosion rate, and t_p is the propagation time. These equations were solved for a RC element exposed outdoor in unsheltered conditions, determining the values for the involved parameters from the experimental results. The carbonation coefficient was described by a normal distribution. The mean value was determined, for each type of binder considered in this work, from data reported in Figure 3 considering a water/cement ratio of 0.5, according to the prescription for these exposure conditions, that is, XC4, of the European Standard EN 206. A SD equal to 0.2 times the mean value was assumed. A mean concrete cover thickness of 30 mm was taken into account, in agreement with the prescriptions of the European Standards for the exposure class XC4, with a SD of 10 mm. As far as the propagation period is concerned, the corrosion rate was described through a beta distribution function, with mean values, determined from the corrosion current density showed in Figure 7 ($v_{\text{corr}} = 1.16 \cdot i_{\text{corr}}$), in outdoor unsheltered exposure conditions, and a SD of 0.25 times the mean value (upper and lower limits equal to 0.1 and 15 $\mu\text{m}/\text{year}$ for all the types of binder, in agreement with the minimum and maximum

values detected outdoor, were considered). Although these data were obtained on concretes with water/binder ratio equal to 0.61, it was observed that the water/binder ratio did not strongly affect the corrosion rate.²⁵ P_{lim} was described by means of a normal distribution with a mean value and a SD respectively equal to 100 and 30 μm , considering as limit state the concrete cracking. Table 2 summarizes the values selected for the different parameters.

Figure 9 shows the probability of failure as a function of the initiation and the propagation time evaluated solving the limit state Equations (2) and (3) through a Monte Carlo method. The probability of failure, that is, the probability of reaching the limit state, increases with the time. Assuming a target probability, P_0 , equal to 10% (red horizontal line in Figure 9), both the initiation and the propagation time can be evaluated as a function of the type of binder. The initiation time varied from about 35 years for 30LI, to 44 years for GGBS, to 60 years for FA and to values even higher for OPC, 15LI, and PZ. The propagation time varied from about 18–20 years for GGBS, 30LI, and PZ, to 24–26 years for 15LI and FA to 60 years for OPC.

From results shown in Figure 9, the service life was evaluated, for each binder, by summing the cumulative distribution functions of the initiation and of the propagation periods (Figure 10). Assuming as the target probability of failure, P_0 , a value of 10%, that is, considering that the cracking of concrete cover occurs on 10% of the concrete surface, the service life was always higher than 60 years. However, significant differences among the binders were observed. The lowest service lives, around 60–70 years, were determined considering 30LI and GGBS concretes, being characterized by the highest

TABLE 2 Values of the selected parameters and types probability density function distribution, PDF, (BetaD = beta distribution; ND = normal distribution) used as inputs in the limit state equations (m = mean value, σ = standard deviation)

Parameter	Unit	PDF	Options	m	σ
$k_{\text{unsheltered}}$	mm/year ^{0.5}	ND	OPC	0.5	0.1
			15LI	1.5	0.3
			30LI	2.8	0.56
			FA	2.1	0.42
			PZ	0.6	0.12
			GGBS	2.5	0.5
x	mm	ND	All	30	10
v_{corr}	$\mu\text{m}/\text{year}$	BetaD ($0.1 < v_{\text{corr}} < 15$)	OPC	0.9	0.24
			15LI	2.2	0.56
			30LI	2.9	0.73
			FA	2.4	0.61
			PZ	2.9	0.72
			GGBS	3.2	0.79
P_{lim}	μm	ND	All	100	30

Abbreviation: GGBS, ground granulated blast furnace slag.

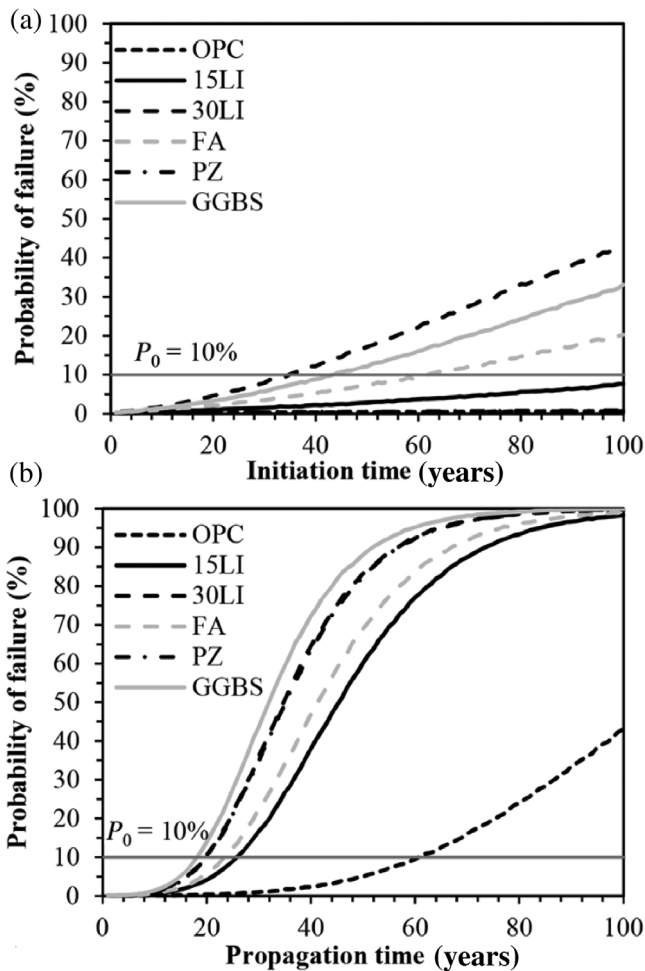


FIGURE 9 Probability of failure as a function of the initiation (a) and propagation time (b) for RC elements exposed in an unsheltered environment and indication of the target probability of failure, P_0 (red line) (concretes with water/binder ratio = 0.5 and concrete cover = 30 mm). RC, reinforced concrete

concrete carbonation coefficient and the highest corrosion rate of steel. The use of OPC concrete led to a service life even significantly higher than 100 years, due to a low concrete carbonation coefficient and a quite low corrosion rate of steel. With FA and 15LI concretes, service lives, relatively long, of the order of 100 years, could be guaranteed. According to these results, the prescriptions provided in the European standards, in terms of water/binder ratio and concrete cover thickness, allowed to guarantee, with a probability of failure of 10%, a service life of, at least, 50 years in an unsheltered environment with all the binders employed in this study.

It is worth noting that these results, and in particular, results on initiation time, were obtained considering experimental data carried out on well-compacted concretes, wet cured 7 days. A significant decrease of the service life might occur if concrete had a higher water/binder ratio, the curing were significantly lower

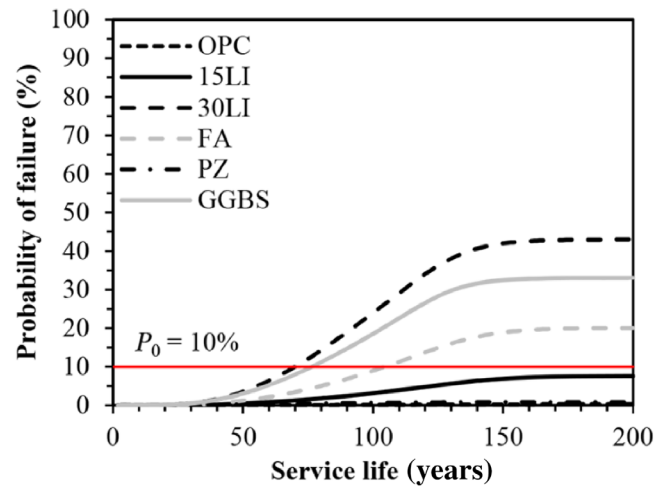


FIGURE 10 Probability of failure as a function of the service life for RC elements exposed in an unsheltered environment and indication of the target probability of failure, P_0 (red line) (concretes with water/binder ratio = 0.5 and concrete cover = 30 mm). RC, reinforced concrete

as well as a lower concrete cover thickness were considered in the simulation. Indeed, experiences on real structures have demonstrated that the corrosion conditions of RC structures are strongly affected by the initial quality of the concrete, the thickness of the concrete cover and the environmental conditions and severe damages were observed on structures made with poor quality concretes and low concrete cover thickness.^{26–30} Nonetheless, within the limits of validity of the results obtained and the conditions considered, the analysis carried out in this paper, provided an example of a performance based approach for a quantitative estimation of the service life.

4 | CONCLUSIONS

Natural carbonation and corrosion rate was studied on 7-day cured specimens made with Portland cement and blended cements, that is, 15 and 30% limestone, 30% fly ash, 30% natural pozzolan, and 70% GGBS and exposed in unsheltered conditions. On the basis of the experimental results and their analysis, the following conclusions can be drawn:

- For each water/binder, concretes made with Portland cement exhibited the lowest carbonation coefficient. The highest carbonation coefficients were observed on 30% limestone, while values slightly lower respect to 30LI were detected on 15% limestone, 30% fly ash, and 70% GGBS. Thirty percentage of natural pozzolan concretes showed comparable carbonation coefficient to

Portland cement concrete; however, only data after 1.5 years of exposure were available. Results were consistent with data obtained by other Authors on concretes with similar composition and exposed to comparable climate conditions. Conversely different climate conditions, for instance in terms of temperature, led to significant differences in terms of carbonation rate;

- The unsheltered carbonation coefficient was well correlated with compressive strength and accelerated carbonation coefficient, suggesting that, in absence of any experimental data on natural carbonation, an estimation of this parameter could be carried out from results of mechanical tests or accelerated tests, taking into account the actual exposure conditions and the type of cement;
- Corrosion potential, corrosion current density, and concrete electrical resistivity were strongly affected by the exposure conditions and, in particular, the relative humidity. At 20°C, appreciable corrosion current density was detected for relative humidity higher than 95%. Temperature played a role only in environments with a high relative humidity (i.e., higher than around 90%). Concrete made with Portland cement had the highest resistivity and the lowest corrosion current density in all the exposure environments. Values obtained in this study were comparable with results obtained in other works for Portland cements while were lower for concrete made with blended cements;
- All the binders employed in this study allowed to guarantee, with a probability of failure of 10%, a service life in an unsheltered environment, with exposure conditions comparable to that of Milan, higher than 50 years, provided that the water/binder ratio was 0.5 and the concrete cover thickness was 30 mm. However, a longer service life can be guaranteed with OPC concretes.

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DATA AVAILABILITY STATEMENT

Research data are not shared.

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