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Durability and Sustainability of Concrete Structures – Workshop Proceedings

SP-305

Editors:

Mario Alberto Chiorino, Luigi Coppola, Claudio Mazzotti,
Roberto Realfonzo, and Paolo Riva



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Cover photo: Interior of the concrete dome of Small Sports Palace in Rome, Italy, 1957, by Pier Luigi Nervi with Annibale Vitellozzi. Courtesy of PLN Project Association

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Preface

With the dawn of twenty-first century, the world has entered into an era of sustainable development. The main challenge for concrete industry is to serve the two major needs of human society, the protection of the environment, on one hand, and - on the other hand - meeting the infrastructural requirements of the world growing population as a consequence of increase in both industrialization and urbanization. In the past, concrete industry has satisfied these needs well. Concrete is an environmentally friendly material useful for the construction of vast infrastructures. Skyscrapers, highway bridges, roads, water retaining structures and residential buildings are all testimonials to concrete's use and versatility. However, for a variety of reasons the situation has changed dramatically in the last years. First of all, the concrete industry is the largest consumer of natural resources. Secondly, portland cement, the binder of modern concrete mixtures, is not as environmentally friendly. The world's portland cement production, in fact, contributes to the earth's atmosphere about 7% of the total CO₂ emissions, CO₂ being one of the primary greenhouse gases responsible for global warming and climate change. As a consequence, concrete industry in the future has to face two antithetically needs. In other words how the concrete industry can feed the growing population needs being – at the same time - sustainable?

ACI Italy Chapter has been playing a significant role in the last years in the broad area of concrete technology in Italy and, in particular, in the field of concrete durability and sustainability. ACI Italy Chapter has become increasingly involved in research and development dealing with durability and sustainability issues such as reduction in CO₂ emissions, use of recycled materials and innovative products, design of durable structures and maintenance, repair and refurbishment of concrete infrastructures.

In October 2015, the American Concrete Institute Italy Chapter (ACI IC) and the Department of Civil, Chemical, Environmental, and Material Engineering (DICAM) of the University of Bologna sponsored the First International Workshop on "Durability & Sustainability of Concrete Structures" in Bologna (Italy). The workshop was co-sponsored by the American Concrete Institute and ACI Committee 201. The proceedings of the workshop were published by ACI IC as SP305. The proceedings consist of forty-eight refereed papers concerning reduction in green house gases in cement and concrete industry, recycled materials, innovative binders and geopolymers, Life Cycle Cost Assessment in concrete construction, reuse and functional resilience of reinforced concrete structures, repair and maintenance, testing, inspection and monitoring.

Many thanks are extended to the members of the technical paper review panel. Without their dedicated efforts it would not have been possible to publish the proceedings. The cooperation of the authors in accepting reviewers' comments and suggestions and in revising the manuscripts accordingly is greatly appreciated.

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Mario Alberto Chiorino
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ANALYSIS AND REHABILITATION OF A R.C. HIGH-RISE STRATEGIC BUILDING IN MILAN

Maurizio Acito, Fabio Cavagnera, Claudio Chesi, Vito Lavermicocca, Valentina Sumini*

Synopsis: In relation to reinforced concrete high-rise buildings built in the Fifties and Sixties of the 20th Century, it has acquired importance, in the last few years, the analysis of the capabilities to withstand various kinds of environmental risks, defined according to actual parameters. The provisions prescribed by new structural design codes practiced today, indeed, have substantially changed both design actions and verification procedures as well, if compared to the building criteria in use in the past. This kind of analysis gives evidence to specific design performances which are seen as prevalent nowadays but were not considered in older versions of the codes, as the earthquake loads. In the present work this problem is discussed with reference to the case study offered by the Milan Municipality 25 story r.c. building erected in Milano in the '60s. Typically, this kind of buildings were designed for the effect of vertical loads and wind lateral loads only. At present, after being recognized of strategic importance for the society, they have to be verified also for the seismic resistance. Although the seismic hazard is classified as low in the area of Milano, design seismic forces are a little more severe than wind actions for this building, due to the limited ductility resources available in the structural elements, mainly in the shear walls. Consequently, the value which can be assigned to the load reduction factor is extremely low.

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Keywords: Reinforced concrete, historical high-rise building, structural performance, seismic action.

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Vito Lavermicocca is Structural engineer and collaborator of the ABC Department of Politecnico di Milano in the role of EXPO quality control consultant for the Italian Pavillion structure assesment.

ACI IC member **Valentina Sumini** is an engineer and structural research collaborator at Politecnico di Milano and Politecnico di Torino, dealing with the structural performance of reinforced concrete skyscrapers and large-scale seismic vulnerability assessment methods for urban centers. She received her BS from Politecnico di Torino; MS from Politecnico di Torino, Politecnico di Milano and Alta Scuola Politecnica; and PhD in Preservation of Architectural Heritage from Politecnico di Milano. She is a member of ACI Italy Chapter, since 2014, after having been awarded for her Ph.D. thesis ("Innovation in Concrete Structures and Cementitious Materials").

INTRODUCTION

The problem of the safety verification of buildings designed and realized over fifty years ago is becoming a main issue, in consideration of a couple of main circumstances. On the one side, the duration of fifty years has been commonly adopted as the conventional value for the nominal life of a structure, thus becoming a standard reference for the definition of design actions; on the other side, a large number of buildings realized after WW2 are now reaching the age of fifty. On this basis, cities like Milan are now defining rules and starting extended programs for the verification of the safety level of buildings falling in this category, which comes out to be fairly large.

The problem deserves special attention in relation to the substantial modification in the design approach to safety verification, which has been changed, through the last fifty years, from the allowable stresses method to the limit state design. Also, an important modification has taken place in relation to the definition of the design actions, with an increasing concern about the seismic excitation.

The opportunity offered by the re-analysis of the building that houses still nowadays the technical offices of the Milan Municipality, realized in Milan in the sixties and classified as of strategic importance, has shown a good basis for the discussion of all this matter. In particular, it has shown the complexity of the reference framework of building rules which have to be considered. Also, it leads to an interesting debate about the decision of designing and planning suitable interventions, in relation to the basic issue of structural safety.

THE DESIGN OF THE MILAN MUNICIPALITY HIGH-RISE BUILDING

The building, rising to one side of the main square of the *Centro Direzionale*, the large business center in Milan designed in the Fifties, houses the technical offices of the Milan Municipality (**Figure 1**). The tall building design begun in 1955 and lasted almost ten years. The architects involved in the design are Vittorio Gandolfi, Renato Bazzoni, Luigi Fratino and Aldo Putelli. The building consists of two bodies: a tall one (25 stories), containing all offices where public traffic is limited, and a four-story one with heavy-traffic offices. A glass-panelled body connects the bridge-building with the tall one. An interesting aspect is that the bridge-building has a span of about 25 meters (82 ft) as it has to surmount the entire Melchiorre Gioia avenue. The shorter sides of the two buildings are respectively 20 meters (65.6 ft) and 16 meters (52.5 ft). Both these structures are in reinforced concrete and have been designed by the engineer De Bernardinis.

The entire design was based on a module, with a length of 1.60 m (5.24 ft), that corresponds to an employee workspace. The module dimension is reported identical in the window, doubled in the window posts of the curtain wall, quadrupled in the r.c. columns span along the longitudinal directional and five times in the r.c. columns span in the transversal one. Moreover, the reinforced concrete structure is extremely interesting as presents some similarities with the Pirelli building (designed by the architect Gio Ponti and the engineers Pier Luigi Nervi and Arturo Danusso), conceived in the Fifties. The ground floor and the typical floor plans (plant dimensions are about 50 m × 17 m (164 ft × 56 ft)) in **Figure 2** show the column and wall disposition in the

reinforced concrete skeleton of the building. The wall system, consisting of two cross shear walls (with openings and connecting beams at the upper levels) and two cores, located in correspondence of the elevator shafts, has been designed to mainly resist to wind loads while the columns have to bear the vertical loads [2]. Each plan of the building has been divided with the same square module (1.60 m × 1.60 m (5.24 ft × 5.24 ft)) with movable panels, made of aluminum frames, that are both transparent and opaque. Moreover, all the technological plants have been located inside the workspaces, according to the same grid dimensions [3, 4, 5].

THE REASONS FOR RE-EVALUATING THE STRUCTURAL SAFETY MARGINS

As it will be discussed in the following, the original design actions for this structure are not in line with the requirements of the current building code for the design of new structures. Specifically, the seismic action was not considered in the original design. The evolution in the design rules, however, does not automatically imply, for existing structures, that the action effects are updated and, consequently, the structural safety checked again; this is clearly specified in the Italian Building Code [9] at Chapter 8, which is dedicated to existing structures. On the other hand, a new safety evaluation is required if the normal usage conditions of the building are modified and the structural mass increased; in this case, structural modifications might also be required. For the analyzed study case, however, the change in the usage condition has not resulted into the modification of the live loads; also from this point of view, therefore, no additional safety verification is strictly required. Such a need was rather originated by the prescription introduced in 2003 by the special government document, [6], which imposed seismic safety verifications for buildings of special importance covering a strategic role, as the one under consideration. The dead line for verifications was initially set five years later, but has been continuously delayed, due to the lack of concern into this matter by public administration offices, which were also afraid about the verification outcomes in terms of possible interventions.

ORIGINAL DESIGN ACTIONS AND SAFETY EVALUATION

The two typical phases in the design process, i.e., action definition and verification criteria, have substantially changed in the last fifty years. As to actions, code prescriptions at design time covered only the vertical loads; due to this, the terminology “gravity design” was commonly in use. No design prescription was given for wind loads which, especially in the case of high-rise buildings, were defined according to widely accepted standard criteria, well documented in classical engineering manuals, like [7] and [8]. Probably, for the building here considered the criterion indicated in [7] was used, corresponding to the pressure distribution specified in **Table 1**.

The corresponding vertical profile of the wind pressure is shown in **Figure 3**, where the one relative to the present code is also shown for comparison. In both cases, the global pressure comes from the superposition of the windward and the leeward components, corresponding, respectively, to 2/3 and 1/3 of the global pressure. More precisely, according to the present code, the two components are obtained from a reference pressure value, multiplied by coefficients (0.8 and 0.4 respectively) reflecting the building typology and geometry.

As it appears from **Figure 3**, at a height of 87 m (285.4 ft), i.e., at the top of the building, the wind pressure value according to [7] is 2.166 kN/m² (44.36 lbf/ft²), much higher than the value corresponding to the present code [8,9], 1.63 kN/m² (33.39 lbf/ft²). Looking, however, at the resultants of the two pressure distributions on the main façade (87 × 50 = 4350 m² (46823 ft²)), the value corresponding to the old criterion is only 10% larger than the one coming from the new one.

In the original conceptual design, the wind force in the building short direction had to be entirely resisted by the two main shear walls. In the spirit of the working stress approach, according to rules dating back to 1939 (see [10]), a normal design procedure consisted in computing the maximum shear stress as

$$\tau_{\max} = \frac{V}{b \cdot z}$$

(with V indicating the shear action, b indicating the section width and z the internal lever arm) and checking that the corresponding value was less than 0.4 N/mm² (58 psi), or 0.6 N/mm² (87 psi) in case of high strength concrete. In this situation, it was allowed to assume that shear was entirely resisted by concrete; no specification was given for minimum reinforcement requirements. In the problem here considered, the adopted design criterion was probably of this kind.

With reference to the direction normal to the building long dimension (Y), corresponding to the maximum wind action, the wind bracing system is based on the two lateral walls and is almost symmetrical. With reference to the wind shear action on a single wall (5490 / 2 = 2745 kN (617 kips)), a value of 0.335 N/mm² (48.6 psi) can be estimated for the maximum shear stress, a little lower than the limit value of 0.4 N/mm² (58 psi).

The designer adopted a minimum amount of shear reinforcement corresponding to Ø 10 mm (0.394 in.) horizontal bars at a spacing of 60 cm (12.99 in) at ground level, while at upper levels spacing is reduced to 33 cm (23.62 in).

As already mentioned, in the absence of structural modifications, the present building code does not require the re-analysis of the building for wind actions, even if both the action definition and the safety verification criterion have changed from the design time. A different problem has to be faced if the building seismic safety, which was not considered at design time, has to be checked; in this case, the situation requires a careful discussion.

THE SEISMIC PROBLEM: ACTION DEFINITION AND SAFETY VERIFICATION

According to the present building code [9], the verification of an existing structure for seismic safety has to be done, normally, through the limit state approach and the shear resistance of columns, walls and beams has to be developed by contributions coming from both concrete and reinforcement. It should be considered, indeed, that the code still allows, in some specific cases, the adoption of the working stress approach; this is limited, however, to low seismicity areas and to buildings of normal importance. According to a recent regional update of the seismic zones in Lombardy [12], the city of Milano has been assigned to a medium-low seismicity zone; as a consequence, safety verifications must follow the limit state approach.

In relation to this, it must be considered that [9] still allows, in some specific cases, the adoption of the working stress approach for safety verifications; this, indeed, is restricted to seismic areas classified into the 4th zone and is limited to buildings of normal importance. According to a recent regional update of the seismic zones in Lombardy [12], the city of Milano has been assigned to the 3rd zone. As a consequence, safety verifications must follow the limit state approach.

The implication of this in the problem under study is that, in the presence of a shear collapse modality for concrete, a brittle behaviour has to be considered. In line with the code prescriptions for existing structures, this leads to the adoption of a behaviour factor as low as 1.5 in the evaluation of the seismic load.

Coming then to the detailing of the seismic forces, the acceleration response spectrum has to be defined, in line with the current hazard map imposed for the Italian territory. This requires to choose, first, a value for the building nominal life; in this case the default value of 50 years has been used, amplified through a “usage coefficient” of 2, corresponding to the situation of buildings covering a strategic role. This results, for the city of Milan, into the evaluation of an earthquake return period of 949 years. The associated peak ground acceleration is 0.059 g, which has to be amplified by a soil coefficient of 1.5, corresponding to a C class soil (see [9]). The design spectrum is therefore obtained from the elastic response spectrum dividing by the behaviour factor ($q = 1.5$).

On this basis, through a finite element model of the entire building, developed with the commercial software system Midas Gen, a modal analysis with response spectrum has been performed. The main result from this is given by the total shear force at the base of the building for a seismic motion acting in the short direction. A total value of 8723 kN (1961 kips) has been obtained, larger than the design wind action in the same direction (see **Table 3**).

The conclusion of all this is clear: the structure can not be considered as positively verified for the design seismic action. The decision to be taken in relation to this is discussed in the following section; as a support to it, it is interesting to comment on the modal analysis results about the global behaviour of the building, in terms of vibration periods, natural modes, and shear distribution at the base.

In general terms, a regular behaviour of the building can be observed, reflecting a regular distribution of masses and stiffness. Periods are shown in **Table 2**; the main period, 2.403 s, is associated to a translational mode in the short direction and is in line with the provisions based on the use of simple formulas. The first torsional mode corresponds to the 3rd mode; the cumulated effective mass is over 90% in both the main directions at the 12th mode. The modal shape for the first two modes are shown in **Figure 4** and **Figure 5**.

Looking at the distribution of the shear action in the short direction at the base (see **Table 3**), it can be observed that it is resisted at 93% by the two main shear walls. This is a clear confirmation of the correctness of the above mentioned design assumption. It is also of interest to look at the distribution of the shear action along the building height, for both wind and earthquake actions (**Figure 6** and **Figure 7**). The diagram for the earthquake shear clearly reflects the effect of higher modes, also in line with the stiffness reduction occurring over the 15th floor.

THE DECISION ABOUT STRENGTHENING

In the case under study, which corresponds to the situation of an existing building not in line with the design prescriptions for new buildings, three different strategies are possible, in principle: leave the structure as it is, improve the resistance to some level, pursue full structural strengthening, in the sense of reaching the safety level which is required for new structures. Within the Italian framework of code prescriptions, which is not clearly defined in all the details at the moment and, mainly, in the perspective of structural safety, the choice of the correct strategy deserves a careful debate, with the consideration of several aspects.

This specific problem has to be analyzed from the point of view of the building refurbishment in relation to the new usage condition, on one side, as well as of the government prescription dating back to 2003, which requires the evaluation of seismic safety for buildings of strategic importance, on the other side.

As to the more recent problem, i.e., remodelling, the code clearly states that full strengthening is not required as long as the vertical loads do not change and the structural elements do not undergo any modification. This is exactly the situation in the case under study. In relation to the design actions, moreover, the code specifies that strengthening is required if man controlled actions change, whereas resistance does not have to be upgraded if new reference values are introduced for environmental actions, which are associated to high uncertainty and are not man controlled. At the same time, however, it is also remarked that strategic functions should be assigned only to buildings totally in line with seismic safety requirements.

On the other side, looking at the government prescription, it has to be considered that the specific requirement was limited to safety evaluation; all the resulting decisions had to be care of local public administration offices. On this matter, the Civil Protection Agency also expressed an opinion, asserting that the owners of buildings of strategic importance are responsible only for safety evaluation activities, while no specific strengthening intervention is required in case verifications are not satisfied. All the same, Civil Protection encourages the definition and planning of seismic strengthening interventions in public buildings.

From a careful reading of all the relevant documents, therefore, it looks clear that full strengthening, in this case, is not imposed, but simply encouraged. This is fully in line with the local administration trend, which aims at a general safety improvement for its real estate.

On this basis, the decision was taken to proceed to the design of full strengthening interventions.

A side consideration has to do with code prescriptions specific for the monumental heritage. In this case full strengthening interventions are normally considered not compatible with the needs of conservation and simple improvement interventions are therefore accepted. In the specific case, where the special situation of historic interest does not apply, any discount on safety did not look justified.

In cooperation with the Milan Municipality offices, suitable strengthening works have therefore been designed, in order to increase the shear resistance of the two main shear walls, as briefly described in the following.

Considering that the critical aspect in the shear walls had to do with the lack of shear reinforcement, an external concrete layer has been added, with a thickness of 7 cm (2.76 in), hosting a suitable amount of reinforcing bars, both vertical and horizontal. In this way, the total area of the cross section has increased by 20 %. Special concern has been given to the need for a full cooperation between the concrete internal core with the new external one; to this purpose, connecting bars have been inserted. Also the original concrete external surface has been corrugated.

CONCLUSIONS

Based on the results related to the structural performance of the Milan Municipality tall building, the following conclusions are drawn:

1. the evaluation of the real seismic safety for existing buildings of strategic importance has become an important issue and has to do with a number of specific indications in the building code and in the connected documents;
2. as it is shown by the analysis of this typical case, also in places like Milan, characterized by low seismicity levels, the seismic load may acquire more relevance than the wind load if a brittle collapse mechanism is present and, consequently, a reduced value has to be assumed for the behaviour factor. The basic problem, in this, does not come from the lack of consideration of horizontal loads in the original design but, rather, from the evolution in the criteria for the verification of concrete sections in shear;
3. the general conclusion has to do with the lack of shear reinforcement in the shear walls designed about fifty years ago. It is believed that such verification problem might become very common in the next years, leading to a systematic reinforcing of concrete walls.

ACKNOWLEDGMENTS

The authors wish to express their gratitude and sincere appreciation to MM SpA, for having allowed the publication of this research. This work reflects the first results of an ongoing collaboration between the Politecnico di Milano and MM SpA (responsible for the contract is Dr Massimo Recalcati), with the aim to evaluate the seismic vulnerability of reinforced concrete existing buildings.

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TABLES AND FIGURES

Table 1– Wind pressure variation according to [7].

Height from the ground level [m]	Pressure [kN/m ²]
0 ÷ 10 (0 ÷ 32.8 ft)	0.50 or less (in urban areas) (10.4 lbf/ft ²)
10 ÷ 50 (32.8 ÷ 164 ft)	1.00 (20.9 lbf/ft ²)
50 ÷ 100 (164 ÷ 328 ft)	1.50 ÷ 2.40 (31.3 ÷ 50.1 lbf/ft ²)
>100 (>328 ft)	2.40 (50.1 lbf/ft ²)

Table 2– Natural modes: periods and effective masses

Mode	Periods [s]	Tran - X		Tran-Y		Rotn-Z	
		Mass [%]	Sum [%]	Mass [%]	Sum [%]	Mass [%]	Sum [%]
1	2.403	7.39	7.39	54.09	54.09	0.03	0.03
2	2.023	57.48	64.87	6.50	60.59	0.49	0.52
3	1.561	0.30	65.17	0.14	60.74	64.87	65.39
4	0.611	3.38	68.55	14.83	75.57	0.18	65.56
5	0.521	8.52	77.07	3.91	79.47	4.87	70.43
6	0.480	6.12	83.18	0.51	79.98	9.63	80.07
7	0.262	1.93	85.11	4.78	84.76	0.32	80.38
8	0.234	0.46	85.58	0.91	85.66	0.63	81.02
9	0.226	2.80	88.38	1.45	87.11	0.22	81.24
10	0.208	1.61	89.99	0.13	87.24	0.50	81.74
11	0.162	1.22	91.21	0.17	87.41	1.23	82.97
12	0.144	0.08	91.29	3.52	90.93	0.15	83.12
13	0.114	3.64	94.93	0.26	91.19	0.35	83.47
14	0.092	2.54	97.47	0.01	91.19	0.66	84.13
15	0.074	0.00	97.47	6.19	97.39	0.01	84.14

Table 3–Shear force distribution in walls and columns.

Loads		Shear force	Shear distribution
X direction	Wind	2407 kN (541115 lb)	86% in walls
			14% in columns

Ydirection	Earthquake	9319 kN (2094994 lb)	85% in walls 15% in columns
	Wind	8220 kN (1847929 lb)	93% in walls 7% in columns
	Earthquake	8723 kN (1961008 lb)	93% in walls 7% in columns



Figure 1–Photographic image of the tall building, housing the technical offices of the Milan Municipality [1].

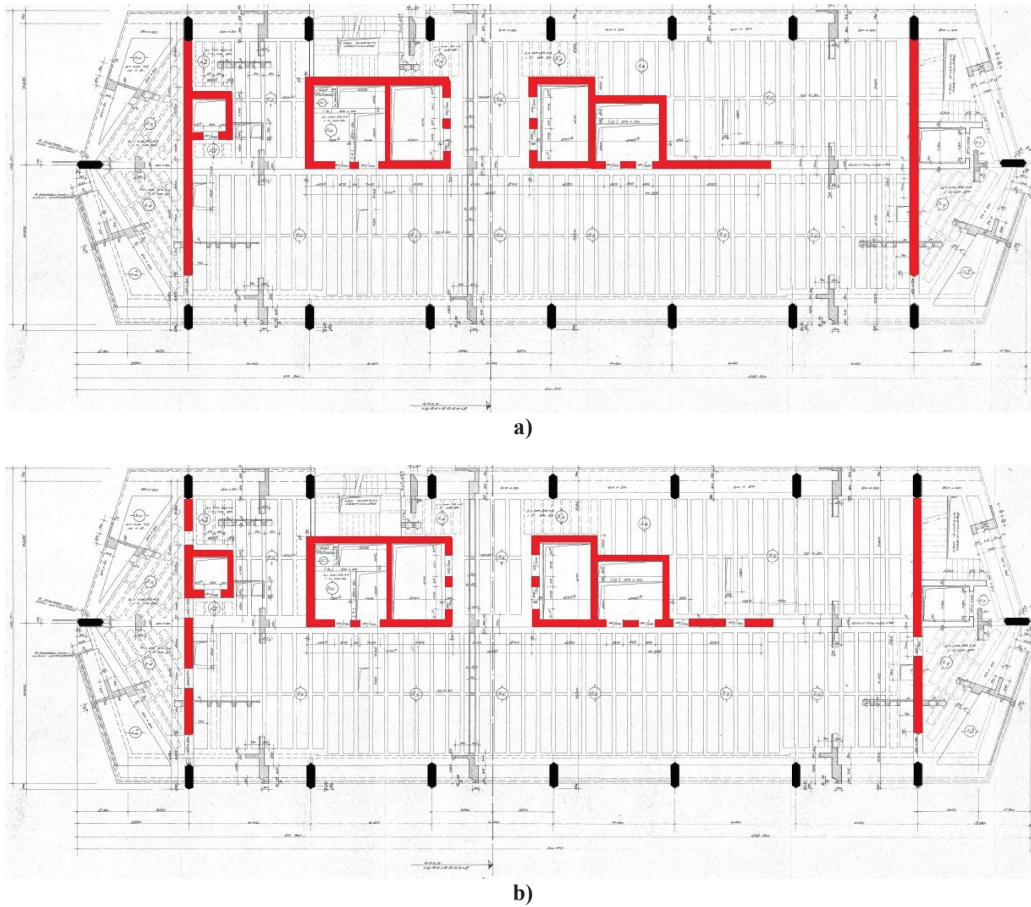


Figure 2—Building plant view with the wall system and the frame elements: a) ground floor; b) typical floor.

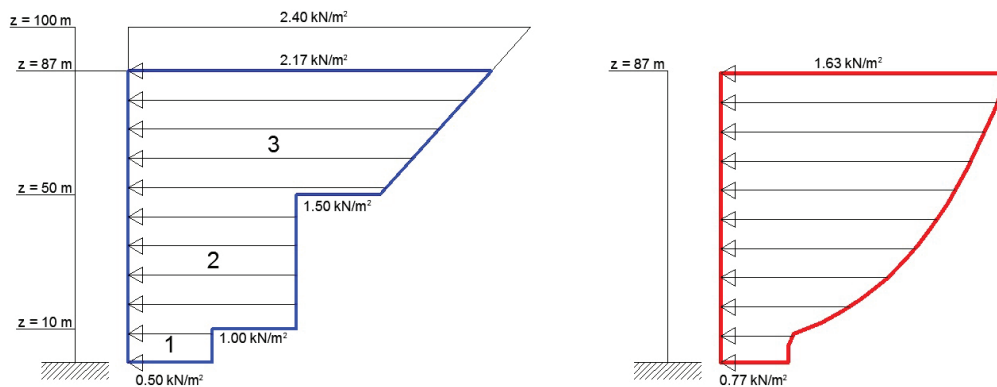


Figure 3—Wind load profiles.

* The z values are: 10 m (32.8 ft), 50 m (164 ft), 87 m (285.4 ft) and 100 m (328 ft).
 The pressures are: 0.50 kN/m² (10.4 lbf/ft²), 0.77 kN/m² (16.1 lbf/ft²), 1 kN/m² (20.9 lbf/ft²), 1.5 kN/m² (31.3 lbf/ft²), 1.63 kN/m² (34 lbf/ft²), 2.17 kN/m² (45.3 lbf/ft²), 2.40 kN/m² (50.1 lbf/ft²)

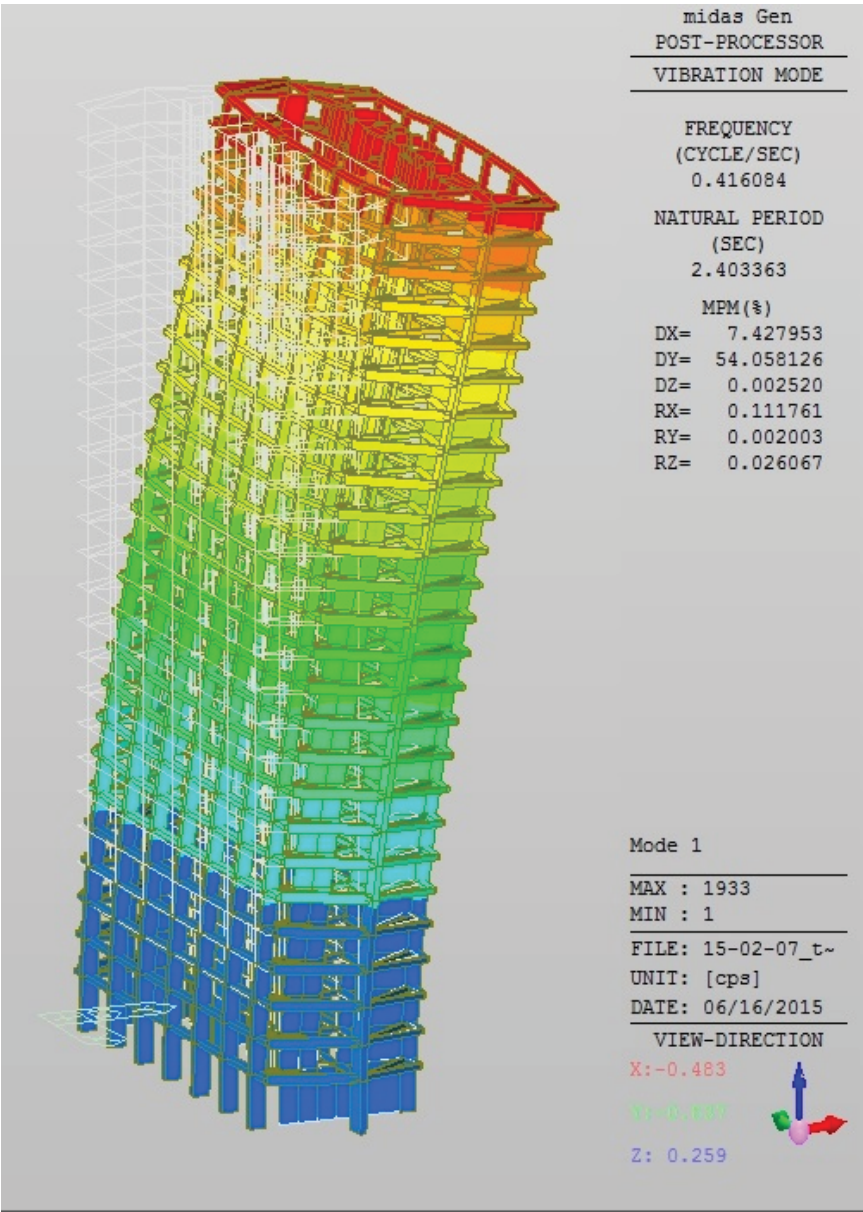


Figure 4–Modal analysis: first mode of vibration.

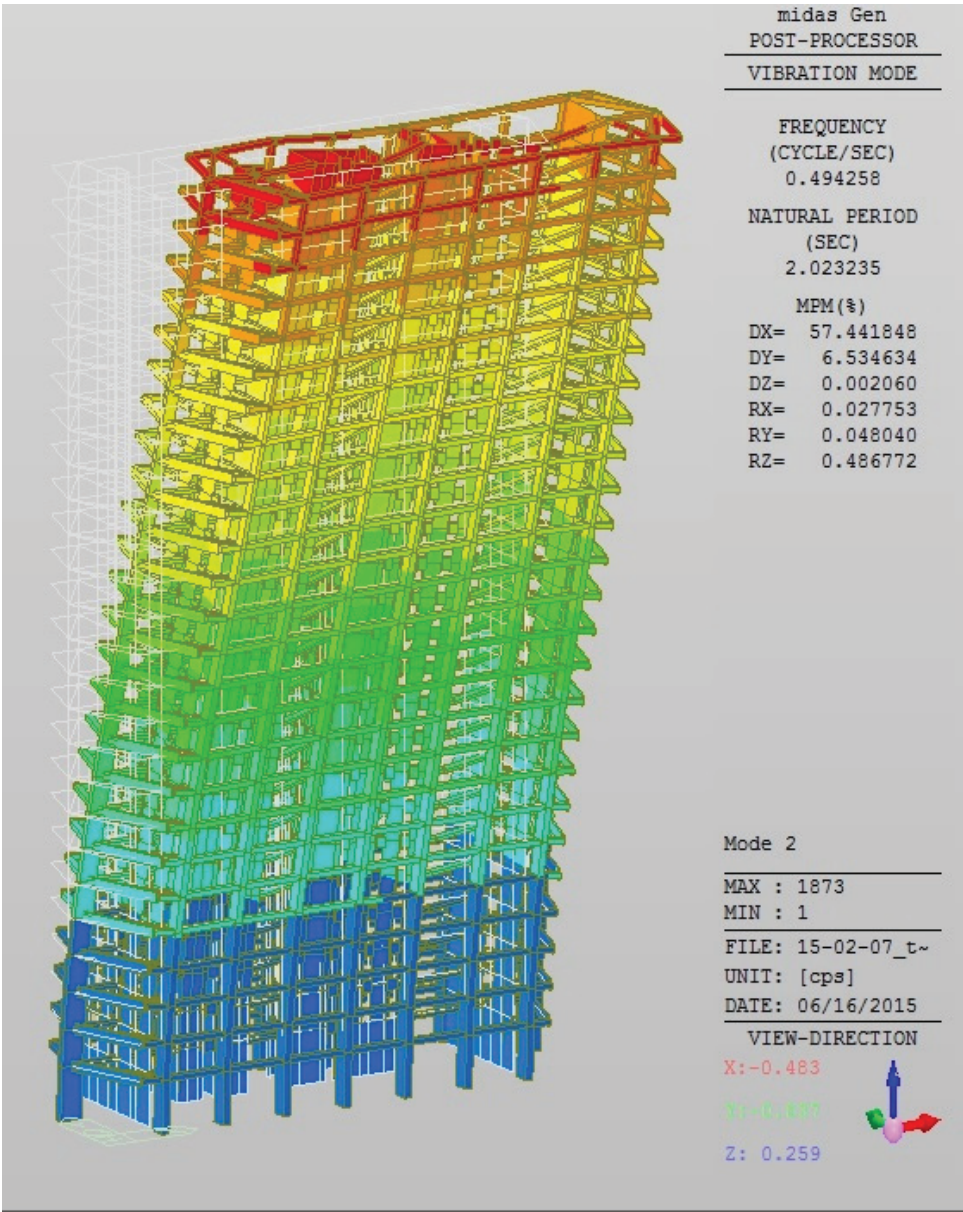


Figure 5– Modal analysis: second mode of vibration.

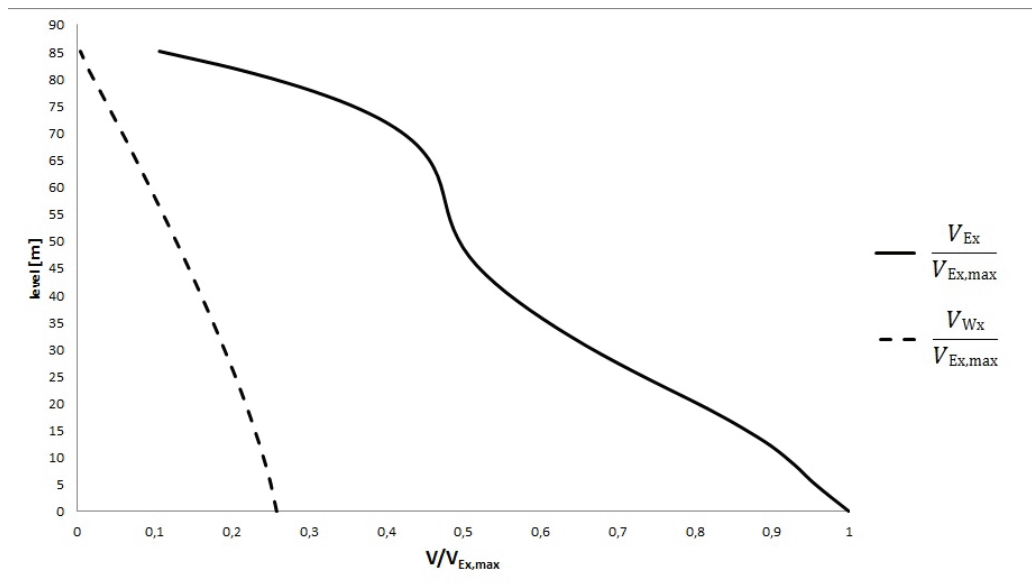


Figure 6— Shear force diagram for wind and earthquake in X direction.

* The level range is 0-285 ft

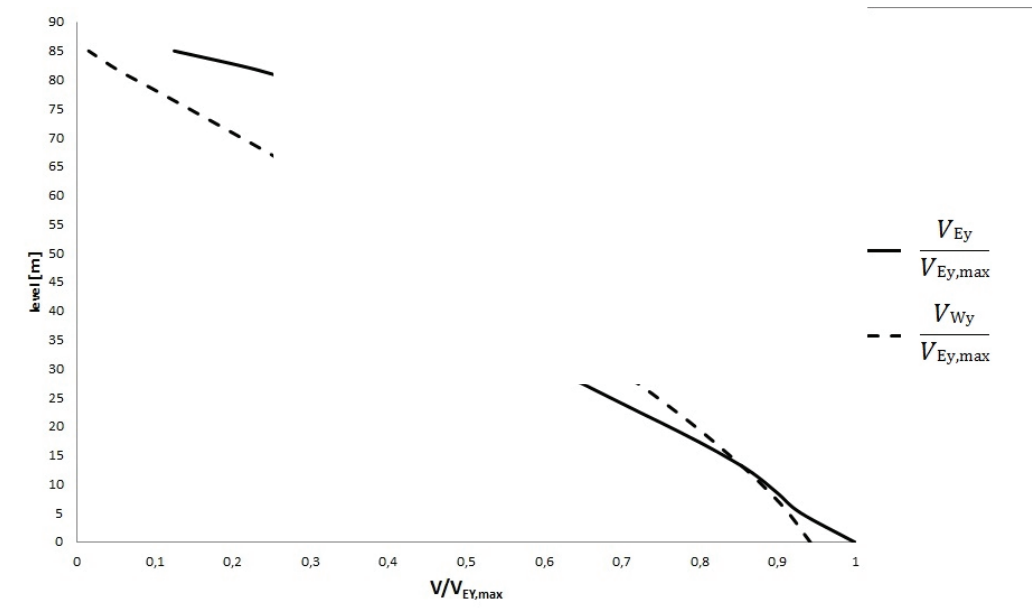


Figure 7— Shear force diagram for wind and earthquake in Y direction .

* The level range is 0-285 ft

Acito et al.

NATURAL CARBONATION OF CONCRETE

Federico M. Aguayo, Thano Drimalas, Kevin J. Folliard

Synopsis: In this paper, the carbonation resistance of various concrete mixtures incorporating supplementary cementitious materials exposed to atmospheric CO₂ concentration in Austin, Texas, USA, was investigated. The paper provides a detailed description on the creation of an outdoor exposure site to place and monitor concrete specimens under ambient air exposure conditions. Two exposure conditions were investigated, including specimens that were placed outdoors unsheltered and in sheltered environments protected from direct rainfall. The depth of carbonation was measured after 730 days of exposures. Additionally, relative humidity (RH), temperature, and environmental CO₂ concentrations in the air were also monitored at the site to provide a general indication of the conditions at the exposure site. Various factors were investigated including water-cementitious materials ratio, type and dosage of supplementary material, and cement type. The depth of carbonation of mixtures without supplementary material was moderate. The addition of supplementary material significantly increased the carbonation depth, especially when replacement levels exceeded 30% of cement by mass. The results were exacerbated for those concrete specimens that were placed outdoors but sheltered from direct rainfall.

Keywords: (Concrete, CO₂ concentration, Exposure Site, Natural Carbonation, Supplementary Cementitious Materials)

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INTRODUCTION

In reinforced concrete structures, the surface of the steel rebar is normally protected from corrosion by a thin passive layer of iron oxide formed by the high alkalinity ($\text{pH} \approx 13$) of the concrete's pore structure. This phenomenon, known as passivation, will protect the steel indefinitely unless the passive layer is destroyed either by the presence of an excessive amount of chloride at the location of the steel and/or by the reduction of the pH in the vicinity of the steel due to the diffusion of gaseous carbon dioxide from the environment reacting with the hydration products (a process known as carbonation).

In North America, it has typically been chloride-induced corrosion that has been the leading cause of corrosion (Basheer et al. 1996). However, in recent years, there has been increased concern over carbonation-induced corrosion for concrete mixtures using supplementary cementing materials (SCMs) at relatively high replacement ($> 30\%$) by mass of portland cement. Concrete mixtures using SCMs as a partial replacement have typically been considered less resistant to carbonation than mixtures with ordinary portland cements (OPC) due to lower portlandite ($\text{Ca}(\text{OH})_2$) content (Hewlett 1998) and subsequent consumption by pozzolanic reaction. Hence, the extent of chemical interaction (and binding of CO_2) will be reduced compared with OPC concrete (Thomas and Mathews 1992). Additionally, the lower portlandite content provides an initial lower pH in the pores solution and may lead to carbonation of other concrete products such as calcium silicate hydrates (Roziere et al. 2009; Houst and Wittmann 2002).

While the mechanism of degradation is well known in OPC systems, recent studies have focused on the effects of SCM replacement on carbonation depth on the properties intrinsic to the concrete, i.e. the concrete's porous structure. Several studies have shown increasing rates of carbonation depth with decreasing moist curing age (Thomas and Mathews 1992; Jia et al. 2012). Because carbonation is related to permeability of concrete, higher carbonation should be expected in inadequately cured concrete, with or without fly ash (Sisomphon and Franke 2007). However, increasing levels of fly ash, i.e. greater than 30%, significantly increase the rate of carbonation. However, Atis (2003) reported contradictory results with regards to the carbonation of concrete made with high volume fly ash. It was concluded that fly ash concrete made with 50% replacement showed lower or comparable carbonation depth when compared to OPC concrete. In addition, Weshe (1991) presented several publication reporting contradicting results with regards to the carbonation of fly ash concrete. Nonetheless, several studies (Naik and Kumar 2010; Galan, et al. 2010) still demonstrate increase in carbonation depth with replacements by mass of cement with SCMs.

As the carbonating process is so slow, various attempts have been made to determine the carbonation resistance of concrete mixtures using relatively high carbon dioxide concentration in the atmosphere as a means of accelerating the test. Thereafter, modeling techniques such as Fick's first law (Tuutti 1982) can be used to predict carbonation depth. In many cases, the concentration of carbon dioxide can be considered highly aggressive compared to natural conditions and influence the concrete's carbonation resistance (Visser 2013). This is especially the case when incorporating the addition of SCMs to the binder. Sanjuan et al. (2003) have reported large discrepancies between accelerated and natural carbonation of concrete using SCMs. Thus, there still exist few publications that report the effect of concrete mixtures using SCMs on carbonation, especially in the natural environment.

This paper presents the creation of an outdoor natural carbonation site in Austin, TX, USA, to assess the carbonation resistance of concretes prepared with binders containing various replacements of SCMs by weight of cement. Several parameters were selected to be evaluated including mixtures with varying w/binder, varying

supplementary cementitious materials, and exposed under two environmental conditions: sheltered and unsheltered. Carbonate content was measured as a function of depth transverse to the face of the specimen using a solution of phenolphthalein on a freshly fractured surface. Relative humidity (RH), temperature, and CO₂ concentrations were also measured at the exposure site to give a relative indication of the natural conditions at the site.

RESEARCH SIGNIFICANCE

Because the reliability of modeling techniques used to predict the performance and service life of concrete structures requires documented cases of field performance of concrete elements, there is an increasing demand for outdoor exposure site performance data. Evaluating carbonation in an outdoor environment is the most reliable method, which could be sufficiently repeated and reproduced, to assess the carbonation resistance of concrete. This paper focuses on the development of an outdoor exposure site to evaluate the rate of carbonation of concrete mixtures exposed to natural carbon dioxide conditions in Austin, Texas. In addition, a comparison is made between carbonation resistance of concrete with and without SCMs.

EXPERIMENTAL PROGRAM

Materials

Two ordinary portland cements were used throughout the program: a general use (GU) and general use limestone (GUL) cement were used, with the GU cement used in the majority of the mixtures. Both cements originated from the same plant and conform to the ASTM C1157 (equivalent to CSA A3000-08) requirements; the chemical compositions and analyses are provided in Table 1. Three supplementary cementitious materials including a low-calcium and high-calcium fly ash and a granulated blast furnace slag (GBFS) were also incorporated at various replacement levels as a percent weight of cement. Table 1 includes detailed chemical analysis for each of the SCMs used throughout the program.

The aggregate used in the program were of crushed and manufactured limestone for the coarse and fine aggregate, respectively. The materials were procured from a local source in central Texas. In addition, the use of a polycarboxylate high-range water reducer (HRWR) provided by a local admixture company was used in order to achieve a target workability between 140-170 mm (5.5 – 6.70 in) for all mixtures, if necessary.

Concrete Mixtures

In order to study the effects of each SCM and cement type on carbonation, six series of concrete mixes, designated LC, HC, S, T, GU, and GUL were cast. The first three series include a single source of SCM either a low-calcium (LC) or high-calcium (HC) fly ash, and granulated blast furnace slag (S), replaced at three different amounts of cement mass. Two ternary mixtures (T) using fly ash and GBFS were also included in the program to study the effects on natural carbonation. The latter two series included the control mixtures using only the two cement types included in the program. In the case of cement replacement, GU cement was used throughout the program to study the effect of SCMs on carbonation.

Three cement contents were used for each of the series including 310, 340, & 400 kg/m³ (523, 573, & 674 lb/yd³). The mix proportions are given in Table 2.

The aggregates were mixed for 30 seconds. Then half of the water was added into the drum while continuing to mix over a period of 30 seconds, followed by an additional 30 seconds of mixing. The rest of the dry materials were added and allowed to mix for 30 seconds at which point the remaining water, containing the HRWR (if necessary), was added. All materials were mixed continuously for a further four minutes with a three-minute resting period in between.

Carbonation test of concrete in a natural environment

The mixtures were cast in a prism of 100 x 100 x 400 mm (4 x 4 x 16 in) and de-moulded after being stored at 23°C ± 3°C (73°F ± 5°F) under wet burlap and plastic for 24 hours. Specimens were then labeled and placed outdoors either sheltered or unsheltered environments until the age of testing. No moist curing was provided following de-moulding of specimens to study the carbonation characteristics of insufficiently cured concrete under natural conditions (Jia et al. 2012).

The depth of carbonation was measured perpendicular to the face by spraying the freshly split face with a phenolphthalein pH indicator. A single 85 mm (3 in) slice was cut from the specimen at different ages for testing using a block cutter. After splitting the concrete prisms, the freshly split surface was cleaned and sprayed using a phenolphthalein 0.5% ethanol solution with a 0.5 g phenolphthalein and 90 ml 95 V/V% ethanol diluted in water to 100 ml (Chang and Chen 2006). In the noncarbonated part of the specimen, where the concrete was

still highly alkaline, a purple-red color was obtained. In the carbonated part of the specimen where the alkalinity of concrete is reduced, no coloration occurred. The depth of the uncolored zone was measured to the nearest 0.5 mm (0.02 in) made at five points along each side of the square cross-section prism slice for a total of 20 values and average presented.

Following the testing of the freshly broken specimen, a thick layer of an acrylic polymer-based aqueous dispersion was applied to the surface of the remaining part of the prism. The polymer forms an elastic waterproof and gas proof membrane that will protect the broken surface from carbonation. The concrete prisms were then returned back outdoors into the exposure site until the next measurement.

Outdoor Natural Carbonation Exposure Site

The development of an outdoor exposure site to investigate the performance of hardened concrete exposed to natural carbonation was created in 2011. Figure 1 shows the exposure site initially created in 2011 for the concrete prisms manufactured for the program.

Exposure methods were based on the specification given in the European standard CEN/TS 12390-10 (2008) *Determination of The Relative Carbonation Resistance of Concrete*. In the standard, two procedures are provided to measure the relative carbonation resistance of concrete:

- Accelerated carbonation carried out under controlled exposure conditions using natural levels of carbon dioxide; or
- Exposure to carbonation under natural condition sheltered and unsheltered from direct rainfall

The accelerated procedure uses an environmental chamber exposing the hardened concrete to a much higher concentration of carbon dioxide than would normally in the field providing faster results. The latter procedure provides a closer indication to field performance and thus, was determined to be one used for the project. Detailed information on the specification used to design the site as well as the monitor used to measure the ambient carbon dioxide levels are presented next.

Exposure Site Design - The exposure site was constructed in an open field directly across from the Construction Materials Research Group (CMRG) at J.J. Pickle Research Campus. All site preparation and construction was completed by the research team. The exposure site consists of one 0.150 x 7.62 x 7.62 m (0.5 x 25 x 25 ft) square deck frame. A Bobcat skid-steer loader was used to clear all vegetation and level the site. The deck was raised using various aggregates piled at the center of railroad ties anchored into the ground. After spreading the aggregate across the interior of the square frame, a tamper was used to properly level the aggregate height in order to reduce the gradation of the site.

Specimens that were prepared for sheltered outdoor exposure were to be stored in a purpose-built Stevenson Screen. The main purpose of the Stevenson Screen was to protect the specimens from any weather elements and compare differences between the two concrete exposures.

The Stevenson Screen constructed at CMRG was designed using a rough appearance cedar wood boards and timbers purchased from local hardwood store. The beams spanning the Stevenson Screen were nominal 51 x 102 mm (2 x 4 in) boards and nominal 102 x 102 mm (4 x 4 in) cedar wooden posts were served as the columns.

Design of Wall Panels - The four sides of the walls were to be constructed to allow free interchange of atmosphere whilst preventing specimens being exposed directly to precipitation. Stevenson Screen slats angled at 45° were to be constructed to allow for free interchange of atmosphere. In order to place the slats appropriately, 45° isosceles wooden triangles were sawed from a wooden panel with a thickness equal to the wooden posts serving as the columns and nailed gun to the each end of the wooden frames. Each triangle was leveled in order to ensure a 45° angle for the slats. The wooden slat strips were sawed from a thin 6.35 mm (0.25 in) thick wood panel. Each slat was cut to a width of 80 mm (3.25 in) to allow for a few millimeters of overhang from the exterior face of the posts. The slats were cut to size of each wall panel and stapled gun to the each end with the addition of wood glue to secure a tight bond. Figure 2 illustrated the wooden slat strips used in the final design of the long span frame including the 45° isosceles wooden triangles.

Design of Roof - Sheets of 6.35 x 1219.2 x 2438.4 mm (0.25 x 48 x 96 in) High Density Polyethylene (HDPE) purchased at local plastic supplier in Austin, Texas were selected as part of the cover design. The sheets were selected to reduce maintenance and replacement costs from the long-term exposure to the high aggressive Texas

heat. In addition, each sheet of HDPE (as well as the entire Stevenson Screen) was white in color in order to minimize the effect of solar radiation.

CEN/TS 12390 does not specify a slope grading for the roof thus, a modification was employed to add a 10% grading across the one-half the length of the Stevenson Screen. This grade was chosen arbitrarily and used to prevent the ponding of water on the surface. Wood panels cut to the slope grade were used on each end of the long span. In order to stabilize the self-weight of the HDPE sheets, square wood panels were used. Figure 3 illustrates the design of the slope grade and wood panels used for each sheet.

After attaching the wood panels to the frame, two sheets of HDPE were cut to cover the frames as well as provide a few inches of overhang similar to the slats. Each sheet was nailed to the three wood panels on each end as shown in the right photo in Figure 3. Each sheet had to be properly sized so that the door frames could fully open as well as not overlap with each other. To ensure that lids were not damaged from abruptly opening due to strong winds, each lid was securely tied to the middle of the Stevenson Screen using a steel cable. The cable was cut to a desired length allowing for enough room to access the inside.

Carbon Dioxide (CO₂) Monitor - After completing the construction of the Stevenson Screen, monitor was placed inside the Stevenson Screen to measure the atmospheric temperature, relative humidity, and carbon dioxide levels outside. The monitor was chosen to be placed inside the Stevenson Screen to protect it from rainfall or other weather elements that could potentially damage the device. In addition, typical meteorological reading stations place their instrumentation inside and measure atmospheric readings from here.

RESULTS AND DISCUSSION

Natural CO₂ Concentration

The monthly average CO₂ concentration and relative humidity (RH) measured at the exposure site in Austin, TX is shown in Figure 4. In addition, the progress of CO₂ concentration throughout the day is shown in Figure 5. The time of day chosen were based on when traffic was at and/or around peak time with the latter 10PM during calm periods. The CO₂ concentration measured at the site represents a relative idea of the environmental conditions at the site in 2012. A summary of the RH, temperature, and environmental CO₂ concentration measured in the test period of the specimens fabricated for the study are shown in Table 3. Data regarding the, environmental conditions including precipitation for the site are also compared from a local weather station in the area (Underground 2015).

The CO₂ in Austin has a minimum concentration at the very beginning of the year and a maximum during middle of the summer. Typically, it is during the middle of the summer that temperatures in Austin can exceed 35°C (95°F) frequently and may have some influence on the increase in CO₂ levels in the atmosphere. Additionally, Austin experiences lower humidity levels during the middle and end of year (Cedar Lake Ventures, Inc 2012). Reasons to believe the spike in concentration could also originate from the increased activity level that occurs at the research lab during the summer. Increased concrete production from various research projects is at its highest peak in the summer. In addition, the location of the lab is relatively near an active train line that could further promote higher than normal CO₂ concentrations, especially during the summer.

Average CO₂ concentration for 2012 measured at the site was approximately 389 ppm. Interestingly, the average concentration exhibited at the site was reasonably close to that of the national average at 394 ppm (Earth's CO₂, 2007) which can confirm reliability in the CO₂ monitor used at the site.

It is evident that the CO₂ concentration varies significantly throughout the day (Figure 5). In the early morning rush hour, the exposure site experiences the highest CO₂ levels during the day with significant decreasing levels as the day progressed right before the afternoon commute. Interestingly, the results at 10PM presented higher levels than the afternoon (4PM) indicating the significant influence the commute has on the CO₂ concentration.

Overall, the results of the natural CO₂ concentration provide a general indication of the condition at the outdoor exposure site. However, the concentration does not reflect those that are commonly used in accelerated test methods and could influence the rate and mechanism of carbonation (Atis 2003; Ohga and Nagtaki 1989; Neves, Branco and Brito 2013). As a result, it is important to assess the carbonation resistance of concrete in natural conditions. Atmospheric CO₂ is continuously increasing at an alarming rate at more than 2.1ppm annual average increase (Earth's CO₂ 2007). Additionally, CO₂ concentration are now consistently averaging greater than 400ppm per year. Continued exposure will encourage further carbonation of concrete specimens at increased levels.

Carbonation Depth

The carbonation depth of the concrete specimens exposed outdoors unsheltered after two year is shown in Figures 6 to 8. Results comparing the two cements used in this study show very little differences in the amount of carbonation depth from one to two year exposure ages. Prisms using GU cement had slightly higher carbonations depths at one year compared to GUL cement; however, prisms showed similar carbonation depths regardless of cement type after two years of exposure. Figures 6 & 7 also demonstrate that the carbonation rate decreases slightly with decreasing w/binder. The reduction in carbonation depth from the reduced w/binder is not as pronounced as expected. This may be attributed to the portlandite content from the increase in cement content for the mixture. The results also demonstrate that the carbonation rate at an early age is very fast in insufficiently cured concrete.

The pronounced effect on the rate of carbonation due to the addition of SCMs in the mixture can be seen from Figure 8. All prisms containing 30% fly ash or more experienced significantly higher carbonation depths than their control after only two years of exposure. The effects are more pronounced with low-calcium fly ash having the least resistance to carbonation followed by high-calcium and GBFS. Interestingly, mixtures containing GBFS showed lower carbonation depths than their control at 15 and 30% replacement while a sharp following the addition of 50% GBFS. The lower replacement using GBFS may be providing more hydraulic properties early on providing more resistance; however, at 50% replacement, the hydration process, rather than the carbonation process, became the dominating factor in concrete. Nonetheless, all mixtures incorporating SCMs experienced increasing carbonation depths with increasing SCM replacement. Additionally, mixtures using SCMs have a more pronounced effect in carbonation depth from one to two year exposures when compared to their control.

It is also interesting to note the carbonation resistance of ternary mixtures under natural exposure. The two ternary mixtures demonstrated similar performance after two years of exposure regardless of fly ash type. It is evident from Figure 8 that concretes performed similar to those binary mixtures incorporating 30% fly ash. Suggesting that there is a relationship between replacement levels of SCMs and their carbonation resistance. Ternary mixtures have typically been known to provide increase resistance to environmental causes of deterioration due to their increased long-term strength and low permeability (Erdem and Kirca 2008). However, it is clear that the poorly cured concretes did not demonstrate better performance when compared to their control.

The initial curing period is the most important factor influencing the carbonation of concrete. It can be seen from the Figures 6 to 8 that the insufficient cured concrete has significantly influenced the rate of carbonation regardless if SCMs used. The longer the moist curing period for concrete, the higher the degree of hydration of the cement and the lower the porosity and permeability of the concrete is (Jia et al. 2012).

It is generally admitted that concrete exposed outdoors, but sheltered from rain, carbonates more quickly than concrete which is not sheltered (Houst and Wittmann 2002). This is due to the pore blocking by periodic rain water. Figure 9 shows the variation in carbonation resistance at various SCM replacements including the effect of no SCMs (controls). Specimens stored interannally (sheltered from rain) carbonate on average 40% more than the same concrete stored unsheltered. However, concretes containing no SCMs has a pronounced effect on carbonation when sheltered. The depth of carbonation after 2 years is around twice that of concrete exposed directly outdoors for both cement types. The results in Figure 9 clearly indicate that carbonation is increased considerably for concrete sheltered, especially when poorly cured. Regardless of exposure conditions, however, similar trends when using SCMs can be deduced from the results.

FURTHER RESEARCH

Concrete specimens stored outdoors will be continued to be monitored and assessed for their long-term carbonation resistance at future intervals. Efforts should be made to increase the size of the exposure site to further investigate other various curing conditions prior to exposure. Additionally, initial results on the carbonation depth prior to exposure (post curing) would also be beneficial to the study. Experimental and modeling techniques will be included as a part of the study to determine appropriate carbonation coefficients, especially when using high replacement of SCMs. Lastly, carbonation resistance of similar mixtures under accelerated conditions will be examined in the future.

CONCLUSIONS

This paper presented a detailed description of an outdoor natural carbonation exposure site. Additionally, results are discussed on the carbonation resistance of various concrete mixtures Austin, Texas. The following are the key conclusions to date from this testing:

1. CO₂ concentration in Austin, Texas was at a minimum during the beginning of the year and reaches a maximum during the middle of the summer. Variations of the CO₂ concentrations were noted with the maximum concentration exhibited during the morning commute.
2. Concrete prisms placed outdoors with no SCMs (GU and GUL) exhibited similar carbonation resistance after two years of exposure. Reduced carbonation depth was exhibited with decreasing w/binder content; however, the reduction in carbonation depth was considered almost negligible.
3. The dosage of SCM was the dominating influencing factor for concrete carbonation both in the unsheltered and sheltered environment. This is still the case in ternary mixtures with relatively high carbonation depths exhibited after two years of exposure.
4. The insufficient curing period is apparent in all mixtures including those with no SCM, especially when considered sheltered environments.
5. In general, concrete prisms exhibited the most severe case with regards to carbonation resistance in sheltered environments using 30% or more SCM replacement. In order of greatest durability for each SCM observed, GBFS, high-calcium, and low-calcium had the highest, moderate, and lowest durability performance, respectively.

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TABLES AND FIGURES

Table 1–Physical and chemical compositions of cement and slag

	GU	GUL	LC	HC	S
Specific gravity	3.13	3.08	2.45	2.5	2.95
Blaine fineness, cm ² /g	3790	4460	3010	3840	4420
Loss of ignition	2.38	5.54	2.4	0.37	-
SiO ₂ , %	19.7	18.83	46.029	37.942	32.76
Al ₂ O ₃ , %	4.84	4.6	22.397	19.712	6.994
CaO, %	61.83	60.57	4.367	20.85	48.067
MgO, %	2.91	2.7	0.891	3.735	10.374
SO ₃ , %	3.71	3.33	0.513	1.733	0.037
Na ₂ O, %	0.18	0.16	1.258	1.348	0.302
K ₂ O, %	0.76	0.77	1.568	0.627	0.333
TiO ₂ , %	0.31	0.38	1.09	1.35	0.465
MnO, %	0.29	0.24	0.04	-	-
Fe ₂ O ₃ , %	2.97	2.86	15.67	4.933	0.457
Alite	50.5	47	-	-	-
Belite	25.3	24	-	-	-
Aluminate	3.9	2.9	-	-	-
Ferrites	11.8	11.2	-	-	-
Calcite	3.1	9.4	-	-	-

– = not measured items.

Table 2–Details of concrete mixture proportions

Mix ID	Mix Series	W/binder	Low-calcium (LC), %	High-calcium (HC), %	Granulated Blast Furnace Slag (S), %	Mix Proportions: kg/m ³			
						OPC	FA1	FA2	S
LC1	LC	0.5	15	-	-	310	46.5	-	-
LC2			30	-	-		93	-	-
LC3		0.45	50	-	-	340	155	-	-
LC4			15	-	-		51	-	-
LC5			30	-	-		120	-	-
HC1	HC	0.5	-	15	-	310	-	-	-
HC2			-	30	-		-	-	-
HC3		0.45	-	50	-	340	-	155	-
HC4			-	15	-		-	51	-
HC5			-	30	-		-	120	-
S1	S	0.5	-	-	15	310	-	-	46.5
S2			-	-	30		-	-	93
S3		0.45	-	-	50	340	-	-	155
S4			-	-	15		-	-	51
S5			-	-	30		-	-	120
T1	GU-T	0.5	20	-	20	310	62	-	62
T2			-	20	20	310	-	62	62
GU1	GU	0.5	-	-	-	310	-	-	-
GU2		0.45	-	-	-	340	-	-	-
GU3		0.45	-	-	-	400	-	-	-
GUL1	GUL	0.5	-	-	-	310	-	-	-
GUL2		0.45	-	-	-	340	-	-	-
GUL3		0.45	-	-	-	400	-	-	-

Table 3–Environmental Conditions during Test Period (Underground, 2015)

	Outdoor Monitor	Local Weather Station
Temperature, °F	101– 29	109 – 27
Temperature, °C	38 – –2	43 – –3
RH, %	23 – 100	9–100
CO ₂ , ppm	200 – 698	N/A
Yearly Precipitation, mm	N/A	837.33



Figure 1—Outdoor natural carbonation exposure site located in Austin, TX.



Figure 2—a) Wooden slat strips used in the long span of the Stevenson screen. Each slat was cut to fit inside each wall panel, b) long span wooden frame illustrating the isosceles triangles at the ends, c) final design of Stevenson Screen frame



Figure 3: Design of roof system

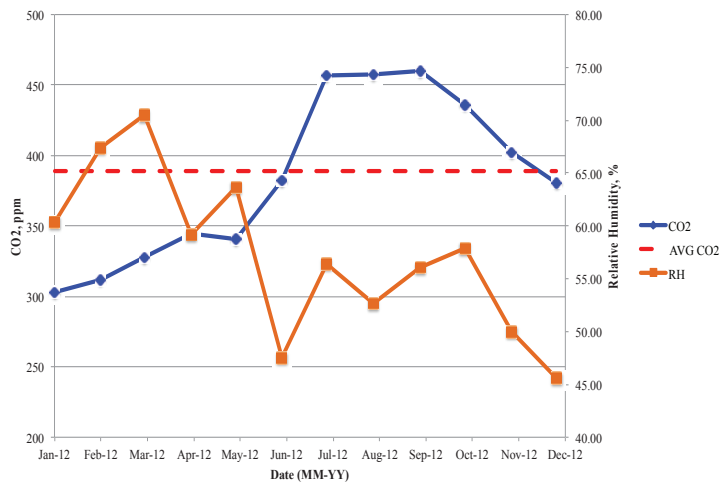


Figure 4–Monthly Relative Humidity (RH) and Environmental CO2 Conditions Outdoors Sheltered

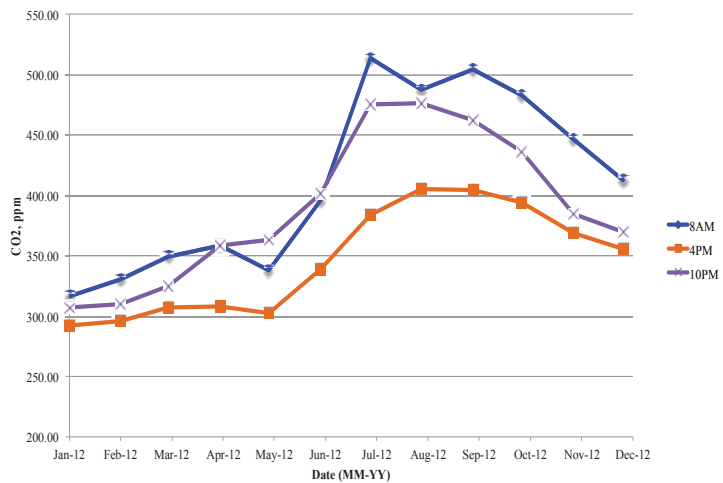


Figure 5–Monthly Environmental CO2 Progression Outdoors Sheltered

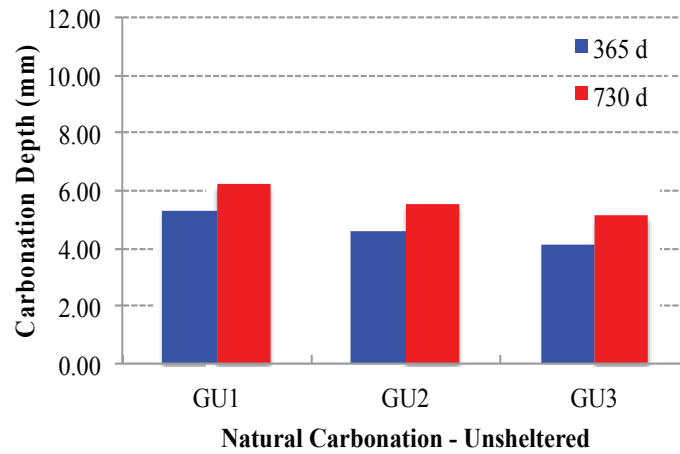


Figure 6–Depth of Carbonation of concrete mixtures using GU

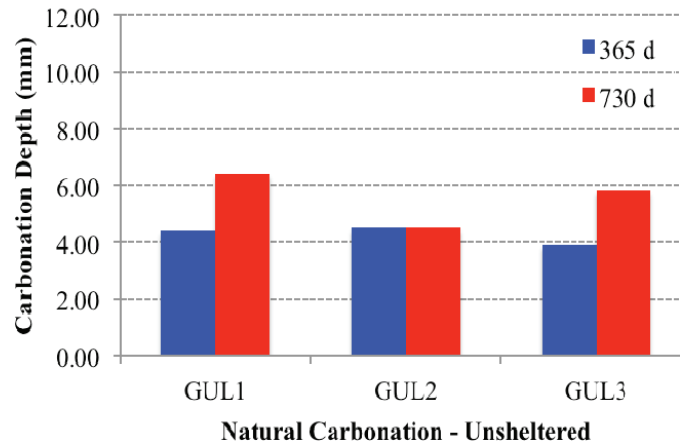


Figure 7–Depth of Carbonation of concrete mixtures using GUL

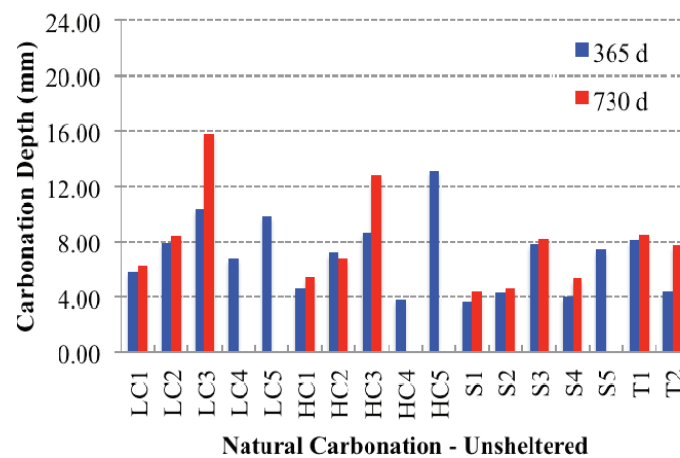


Figure 8–Depth of Carbonation of concrete mixtures using SCMs

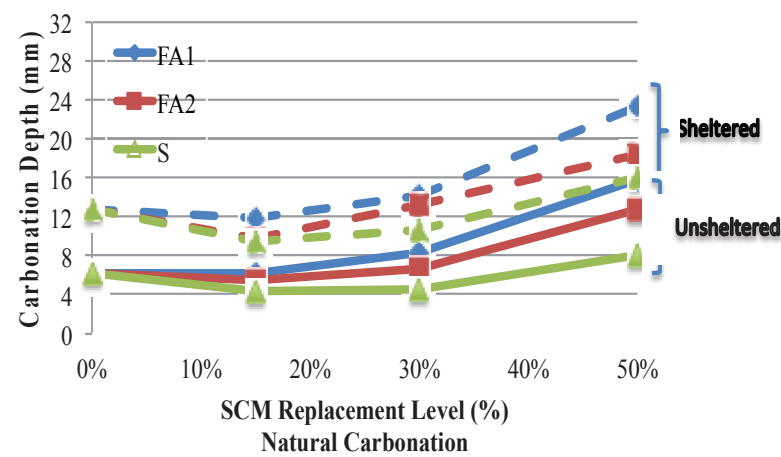


Figure 9–Depth of Carbonation of concrete exposed under unsheltered and sheltered condition at various SCM replacement levels

THE DURABILITY OF RESTORATION OF EXPOSED CONCRETE CASE HISTORIES COMPARED

Francesca Albani

Synopsis: Architectural works by major architects from the twentieth century have often been subject to restoration campaigns intended to redeem them from decay (sometimes from neglect) in order to present them as “monuments” of the twentieth century. In particular, in architecture in which exposed reinforced concrete plays a key role in architectural, figurative, symbolic and cultural terms, the theme of repairing/replacing the concrete cover is a fundamental issue in the restoration project.

Analyzing the methods, materials and techniques of intervention used in several campaigns of restoration of buildings built before World War II (the church of Notre-Dame du Raincy by Gustave and Auguste Perret in France and Rudolf Steiner’s Goetheanum at Dornach near Basel in Switzerland) or after the War (Figini and Pollini’s housing built in Milan and the Olivetti Factory in Crema by Zanuso) provides an opportunity to identify best practices, and also critical factors and weaknesses of the different approaches used in the restoration of these buildings from the 1980s to the present.

Keywords: durability, exposed concrete, historic buildings, maintenance, re-alkalization, repair, restoration

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INTRODUCTION

For decades now the community engaged in the restoration of historic buildings has had to deal with the issue of the decay of exposed concrete. Rust, cracks, delamination and the spalling of the concrete cover, caused by corrosion of the reinforcing bars, are well-known^{1,2,3,4,5} and codified^{6,7,8,9} pathologies.

The main problem, when intervening on buildings in which exposed concrete plays a central expressive and formal role, is how to reconcile the preservation of exposed concrete surfaces with repairs or restoration made necessary due to the aging of the building.

Is it possible to restore exposed concrete façades, ensuring a certain durability of the intervention and preserving the surfaces of the historical concrete?

The question has already troubled many architects, curators, conservation workers and laboratories. Through the analysis of a series of buildings constructed in different periods, it is possible to present a series of reflections on strategies for intervention, the materials and techniques of repair used, highlighting their strengths and weaknesses. The case studies selected concern architecture created by architects whose work has characterized their time, in which exposed concrete is a key expressive element. They have all been subjected to one or more restoration projects that were preceded by preliminary investigations (measurements, diagnostics in situ and in the laboratory, etc.) involving experts from various fields, including university, institutional or private laboratories and the decisions were sometimes taken after public debates.

Analyzing and presenting a critical reflection on works like these, where the level of care and attention is very high given the nature and significance of the building, can help not only to highlight the state of the art for ways of dealing with the problem, but could be a first phase in which to identify the best practices to be adopted in relation to these specific issues.

LARGE SCALE CONCRETE REPAIR/ DEMOLITION RUDOLF STEINER'S GOETHEANUM IN DORNACH, 1924-1928

The Goetheanum in Dornach, Switzerland, designed by Rudolf Steiner between 1924 and 1928¹⁰, is the international forum for the dissemination of anthroposophy. It was built after an earlier wooden building was destroyed by fire in 1913. This building is not only the expression of Steiner's anthroposophic conception and the spiritual and ethical consequences of architecture on the human mind, but is also one of the most important examples of architecture in reinforced concrete from the twenties.

Restoration and repairs

The first signs of degradation (delamination and spalling of the concrete cover) were detected during the 80s. The diagnosis made showed that the level of carbonation of the concrete was between 30 and 80 mm (1,18 and 3,14 in) thick, and therefore that at many points it had exceeded the thickness of the concrete cover of a maximum of 20-50 mm (0,78-1,96 in).

In 1988 Fosroc NCT (Norwegian Concrete Technologies) proposed a system that at the time was innovative and minimally invasive, the electrochemical re-alkalization¹¹. Two samples of some 100 square meters (393,70 in) were chosen, one on the north side, the other on the east. Unfortunately it failed to produce positive and satisfactory results.

After some years it was decided to apply a protective coating on a silicone resin basis to reduce the capacity of absorption of the concrete, together with other traditional techniques (removal of damaged concrete, treatment of the rebars and repair with alkaline mortar). As soon as it was realized that the operation was going to affect the significance of Steiner's architecture, the work was suspended, leaving the façades with exposed rebars. It was then decided to resume the electro-chemical repairs, but a new problem emerged. The experiments and tests conducted did not produce the expected results because the surface had been impregnated with a silicone-based agent which had partially sealed the pores¹².

In the early 90s the Eglin Ristic engineering firm in Basel was placed in charge of the restoration. The work on the northeast, east and southwest façades was carried out between 1993 and 1996. The concrete was removed to a depth of 4 cm (1,57 in) and a new layer of concrete 7 cm (2,75 in) thick was cast in situ using silicon resin molds, about 3.20 m wide and 1.10 high, 12,59 and 4,33 in) set into the formwork. The purpose was to reproduce on the new concrete surface the marks of the shuttering used during construction. Despite the efforts, the marks left by the shuttering created a surface texture different from the original. This profoundly altered the expression and sense of the historical concrete made with a wooden formwork.

Before opting for this type of intervention a symposium was organized with the participation of several international experts and discussions and consultations were held with the cantonal and federal authorities in

charge of conservation. This sophisticated procedure, based on the undifferentiated demolition of the whole concrete surface, raised many questions concerning the need for such an invasive intervention.

A SAMPLER OF SOLUTIONS THE CHURCH OF NOTRE DAME DU RAINCY, 1922-1923

“Many of the clergy appreciate and will always be grateful to Perret for ensuring that a church is not a nondescript building but the house of God.” These words were written in 1932 by the Abbé Orel in a commentary on the church of Raincy Montagny in the special issue of “L'architecture d'aujourd'hui” devoted to Perret. They testify to an awareness of the innovative character of this building, which is much more than a simple “modern” reflection on the theme of the church on a longitudinal plan, just as the architecture cannot be read just as the expression of a new material, reinforced concrete. Auguste Perret knew this well, and for this reason he described the project as the one he “was proudest of”¹³.

Project and realization

Having taken over the commission from the original designer, the engineer Guyot, the challenge that faced the Perret brothers was to build a place of worship rapidly and cheaply with a material hitherto used principally for industrial buildings. They turned out a series of projects at breakneck pace between March and August 1922.

The second version of the project made substantial changes to the façade and steeple, but only in August did they attain the final project: he made the church larger, increasing it by one bay (56 x 19 m or 2,20 x 0,74 in.), the number of columns was doubled and there was a complete dissociation of the structure from the curtain walls. The 28 identical columns made of reinforced concrete, shaped and tapering upwards, support the roof, which consists of double segmental barrel vaults over the nave and smaller vaults set orthogonally over the aisles. Technically Perret could have covered the whole span with a smaller number of columns or have used parabolic forms – he was working on the Esders factory at the same time. But this decision to multiply the columns, creating central bays almost square in form and lateral bays about half of those of the nave, created a rhythmic sequence of structural elements, which means, in accordance with a widely Medieval and Renaissance logic, an awareness of the scale of the building in the worshippers using the church.

The Perrets had already tackled the theme of religious architecture in 1908 when, together with M. Ballu, they built Oran cathedral. Here they decreed the birth of the concrete “claustra”, stimulating a series of reflections on the potential that this artefact opened for light and architectural expression. The Perrets referred to the continuous tracery screen by the rarely used term “claustra”, previously used for the Maison de Santé at Chareton by E. J. Gilbert, published in 1852. It was then taken up by Labrousse and his followers to describe his rectangular marble slabs pierced with geometric patterns. In the case of the Church of Notre Dame du Raincy, perforated panels about 60 cm (2,36 in) per side were prefabricated in five shapes: round, triangular, square, rectangular and cross-shaped. Arranged in different ways they created the patterns of the lateral façades. The panels are supported and jointed by means of ribbing raised in relief above their surfaces. The cladding and its load-bearing structure are therefore not set in the same plane. The division of the elements between the columns is based on those in the second version of the project, which divided them into three sectors of equal sizes. This clearly brings out the complete independence of the supporting structure from the supported elements. The main façade is characterized by the presence of the imposing steeple. At its sides, instead of large corner pillars, it has four clusters of five columns made by using the same formwork as for the pillars of the nave.

Restoration and repairs

The building was listed as a “historic monument” on 29 June 1966. It was subjected to several restorations between 1988 and 1996, to the point where it can be seen as a sampler of diagnostic techniques and solution for the problems of restoration and conservation of structures using exposed reinforced concrete.

In 1976, a series of surveys were conducted in situ and in the laboratory, including sonic probes of beams and pillars, examination of the specifications of the mortars and concretes used, with analysis of the particle size of the aggregates, of the physical characteristics and of the state of the components. Between 1978 and 1983 numerous surveys were made of the vaults, including the mapping of fissures.

Finally in 1986-1987 Pierre Bonnard directed the most recent informative campaign which identified the exact composition of the different kinds of concrete, in which lime was used as a binder with a cement and clay aggregate. The survey also brought out its too rapid installation, as shown by the numerous defects – segregation in the concrete and bleeding – which over the years have speeded up the process of degradation. The phenomena of degradation were already substantially present less than fifty years after the building was erected, with the principal being due to carbonation of the concrete (40 to 75 mm or 1,57 to 2,95 in). In the presence of water and oxygen, this leads to the oxidation of the reinforcement bars, so exerting an expansive action that causes the spalling of the concrete covering.

A first round of work in 1988 and 1989, directed by André Dozet, dealt with the roofing, in particular with a remake of the waterproofing as well as solving the problem of the variation in thermal expansion between the inner and outer vault.

In 1991 the steeple was restored. It was a very complex operation that took into account the different kinds of degradation on the basis of the exposures and specific situations. These processes affected the concrete, whose surface was still intact, involved new techniques – that are still nowadays under research.

– for re-alkalinizing the concrete. Such systems are aimed at the re-alkalinization of the concrete (or chloride extraction), as well as protecting against the corrosion of the reinforcement bars without damaging the surface of the historical concrete. The main purpose of this procedure is to restore the depleted alkalinity surrounding the reinforcement, so a new passive alkaline film can form around the steel bars. These systems require all the damaged areas to be firstly removed and repaired. The surgery consists of placing an electrolytic layer of cellulose fiber sprayed onto the concrete historic surface, while a wire mesh of steel or titanium is set into it. The negative pole of a rectifier is connected to the bars and the wire mesh to the positive pole of the rectifier. Hence the reinforcement serves as a cathode and the wire mesh as an anode. The electrolytic layer is immersed in an aqueous sodium carbonate solution prior to applying an electric charge, which is supposed to lead to the electrolysis of the water in the pores surrounding the reinforcement bars. In the end the electrolyte is removed. After the treatment the phenolphthalein tests showed the carbonation was already to a deep of 60 mm (2,36 in). The results presented in LRMH report concluded that the procedures for re-alkalinizing (without alcalins because the aggregates were sensible to them) the concrete didn't guarantee a durable protection to the rebars¹⁴.

The façades, however, were restored in three different campaigns between 1992 and 1996 using completely different methods. The claustra in the south façade were replaced with new panels made of concrete with the addition of resin and steel reinforcement. Many glass elements in the bay with the baptismal font to the right of the steeple were preserved where they were installed, while the north side was subjected to a more conservative intervention that benefitted from the earlier experiences¹⁵. The cost of survey, analysis and restoration works from 1988 to 1996 was about 18,45 million French Francs (about 3 million dollars)¹⁶.

SMALL SCALE CONCRETE REPAIRS/TRADITIONAL TECHNIQUE HOUSING BY FIGINI AND POLLINI IN MILAN, 1963-1969

Obviously modes of intervention that focus on local repairs are certainly to be preferred in the case of historically important buildings. An example is a residential building constructed in Milan by Luigi Figini and Gino Pollini in 1963-1969¹⁷. It belongs to a series of buildings that Figini and Pollini built in the 50s and 60s, characterized by a grid facade with small loggias that create an effect of light and shade and give depth to the composition. The similarity with the pattern of a Scottish tartan is immediate, even in the building in Via Mazzini, where the grid of the main structure in reinforced concrete has two more grids superimposed on it: an outer one consists of monolithic pillars in Baveno granite and concrete beams, while the other more internal grid is formed by the window frames.

Restoration

Because of falling fragments of concrete, in December 2003, work began on the restoration of the external façades¹⁸. Diagnostics in situ and in the laboratory concerned both the structure of the façade and the innermost main structure (position of the reinforcing rods, measurements of the thickness of the concrete cover, measurements of the potential of corrosion of the reinforcements, detection of ultrasound velocities, sclerometric surveys, phenolphthalein tests, analysis for the presence of chlorides). The tests by the Chemistry, Material and Chemical Engineering Department "Giulio Natta" of Politecnico di Milano indicated that the external structure, with obvious signs of degradation over about 90% of the structure, presented carbonation to a depth of 40-50 mm (1,57- 1,96 in) and the thickness of cover was from 5 to 29 mm (0,19 to 1,14 in)¹⁹.

The objective of the choices of action were intended to preserve the greatest amount of historical concrete and, simultaneously, to ensure a certain durability for the restoration work. It was decided to carry out a traditional type of operation (removal of the carbonated concrete and its replacement with a mortar) at the ends of the beams of the structure of the façade, which were by this time in a state of advanced decay, but not to intervene on the more internal principal structure which was better protected from the weather and therefore unaffected by the pathologies of degradation.

In the specific case two problems emerged that led to the choice of the materials used. A first problem was the impossibility of reconstructing a concrete cover thick enough to ensure durable protection for the reinforcement bars, for reasons related to the need to preserve the geometry of the façade characterized by the co-planarity of the stone elements in relation to the concrete **showed in Figure 1**. In many areas the thickness of the concrete cover was less than 10 mm and therefore totally inadequate to guarantee an acceptable degree of durability. An example is showed in **Figure 2**. To cope with this problem a coating was applied to the reinforcement bars. Protection from corrosion was therefore entrusted to the coating itself and not to the repair mortar which, due to the small thickness of the concrete cover, would not have ensured lasting protection.

A second problem concerned the choice of the type of mortar used for reconstructing the concrete cover. The objective was to identify a cement mortar with a composition and chromatic characteristics compatible with the existing concrete.

The costs of survey, analysis and restoration works were about 1.200.000 Euro (about 1.300.000 dollars).



Figure 1–Figini & Pollini housing building before restoration, 2003



Figure 2–Thickness of concrete cover is less than 10 mm (0,39 in)

DECAY OF CONCRETE SURFACE AND PREFABRICATED BUILDING OLIVETTI FACTORY BY MARCO ZANUSO IN CREMA, 1968-1971

The Crema factory is also an outstanding example of an architectural heritage built by techniques of prefabrication, which in Italy after the war down to the first mid-1970s was characterized by a different procedure from that found in other European countries, such as France, Germany or Britain²⁰. After an initial phase, the immediate post-war period, which was characterized by a substantial time lag in this sector, Italian architecture succeeded in producing examples of excellence through industrial processes applied to the building sector combined with the sophistication of the industrial design of functional objects. The work of renowned designers, including Marco Zanuso, designer of the Crema factory, blazed a trail of excellence that succeeded in finding expression in the sector of industrial architecture, largely built in northern Italy, in keeping with an idea of the project interpreted as the whole set of decision-making activities, ranging from the first planning phase to all the subsequent stages of the project including construction and management. The structure is simple, rapid to assemble and capable of allowing for future expansions, while guaranteeing a certain flexibility. For the purpose a trilithic system was developed, based on a pillar with a shaped profile wedged into the foot, the main beams with an inverted Y-shaped section and secondary beams with a triangular cross-section closed at the top with reinforced concrete roofing panels inside which were laid the thermo-ventilation ducts. The Crema complex, which covers about 48,000 square meters, was the facility within the group devoted to the engineering and production of typewriters²¹.

Restoration

In January 1994, to try and overcome the crisis caused by the closure of the Olivetti factory, the public authorities, including the Lombardy Region, the Province of Cremona and the City of Crema, established an agency, Reindustria, to enhance and develop the resources of the territory. For the former Olivetti site a program was devised for its reconversion to an "Information Technology Pole at the Regional Level" based principally on the installation of the University of Milan's Faculty of Information Technology and Sciences on the site together with a number of companies operating in the computer sector.

In October 2009, when the teaching and research pole were already fully active, in a neighboring building near the factory containing the assembly line, a roof panel collapsed. To ensure the safety conditions necessary for the university to continue its activities, a survey was undertaken to establish the causes of the collapse and define a project for the safety and consolidation of the structure. Numerous tests were conducted in situ and in the laboratory by the P. Pisa laboratory at the University of Brescia²² on the main beams and secondary beams, making it possible to focus on a range of issues and observe the fragility of the structure present since its construction. The Olivetti company was well aware of the problems and remedied them by careful maintenance work. The principal problem lies in the roofing: because of a complex set of factors (buckling of the side walls, which resulted in the separation of the roofing plates or damage to the waterproofing), they fail to carry off the rainwater properly, so allowing it to stagnate in the structure, especially in the hollow secondary beams and a severe problem of corrosion in the reinforcing bars and the conservation of the exposed concrete surfaces.

The restoration of exposed concrete was characterized from two different approaches. A part of the complex was restored with traditional method and techniques. After the demolition of the damaged or carbonated concrete and treating the reinforcing bars, the cover was replaced with an alkaline mortar. After, a pink or blue colored coating was put on the concrete surfaces with the purpose of protecting and making them uniform **showed in Figure 4 and 5**. This last operation permanently disrupts the sense of a structure that was conceived to be built out of exposed concrete²³.

In the other parts of the complex the restoration of exposed concrete was not realized and the priority was the structural consolidation **showed in Figure 7**. This consisted in the reinforcement of the secondary beams to prevent the side walls from sagging by installing a "hoop" and the strengthening of the principal walls, because the existing reinforcement bars have proved insufficient and allow slippage of the wings at the head of the beam **showed in Figure 8**. The intervention, with invasive effects in formal terms, proved capable of solving the structural problems. Moreover, this intervention is not capable of ensuring safety in case of an earthquake. The static structure of the factory, which consists of pillars embedded at the base and simply supported beams, if subjected to horizontal actions by an earthquake, will not be capable of preventing the structures from being free to slide over each other, regrettably, in compliance with the regulations still in force, which classify Crema as an area of low seismic risk.



Figure 3—Olivetti factory in Crema in 1975 (Archivio del Moderno, Mendrisio)



Figure 4, 5 -The parts of the complex with blue and pink colored coating



Figure 6 –The part of the Olivetti factory with structural consolidation works



Figure 7 – Connection between pillar and Y-shape beam. Detail of the decay of concrete structures
CONCLUSIONS

After analysis and comparison of case studies, selected from architectural works in which exposed reinforced concrete facing has a central role in the expression of the architecture and which were the subject of one or more restoration campaigns, a number of conclusions can be drawn.

1. After the experiences of the 1980s, there is widespread support – though with processes of maturation and differentiated degrees of awareness – for the position that sees the preservation of historic surfaces in exposed concrete as a major goal. There is a clear awareness of the difficulties this entails, especially concerning the durability of the intervention.
2. There is no single answer to the question whether it is possible to restore exposed concrete surfaces of façades while preserving the historical concrete, because there are many factors and circumstances that determine the choices and outcomes of the work: the type of decay that affects the surfaces (specifications and extent); the way the historical concrete was made (materials and techniques); the role that the exposed concrete surfaces play in the architectural work.
3. The effective comparison between the restoration of historic buildings is impossible because every building has specific characteristics and problems. Is only possible make a reflection about the methodology and the goals of restoration works.
4. The type of approach to the problem adopted, for some time now, is to proceed with the preliminary investigation before finalizing the design choices, making a diagnosis in situ and in the laboratory to identify and analyze the causes, mechanisms and level of deterioration, characterized by an interdisciplinary approach.
5. There are two principal ways of restoring exposed concrete façades. Firstly, the traditional one is based on the application of repair mortar for local interventions or greater extent, after demolishing the damaged or carbonated concrete and treating the reinforcing bars. There are various methods for installing the new mortar/concrete, as well as its composition. In many cases this intervention is unfortunately followed by the application of a surface coating that has the purpose of protecting the surface and making it uniform. This last operation permanently disrupts the sense of a structure that was conceived to be built out of exposed concrete.

The second concerns concrete which seemingly is not suffering from problems of degradation, namely without the expulsion or delamination of the concrete cover. This method consists of using electrochemical processes for the re-alkalization of the carbonated concrete, followed by the addition of an “invisible” layer to protect the surface with a “corrosion inhibitor” or water repellent impregnation (hydrophobation). In this case the existing concrete is retained. The corrosion inhibitors are chemical substances (amino alcohols, sodium monofluorophosphate, etc) that, if they are present in a sufficient concentration, can slow down the corrosion of the reinforcement bars. They are applied to the existing concrete surface as an aqueous solution and transported to the bars through capillary absorption.

6. The issue of the durability of the repair in this case is still undecided. Many doubts exist both towards the electrochemical procedures of re-alkalization and towards corrosion inhibitors.
7. Probably the most interesting approach, in the light of the immense literature, numerous studies and experience gained in over 35 years of restoration of “monuments”, is intervention in a locally diversified way in different parts of the building depending on the problems of degradation present, on the basis of a careful and thorough campaign of preliminary investigations. Obviously specific interventions should be favored that repair damaged areas or surfaces by re-alkalizing them while still intact, thus ensuring the conservation of the historic surfaces of formal, cultural and historical significance.

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PREDICTION OF SERVICE LIFE OF REINFORCED CONCRETE BY CONSIDERING THE INITIATION AND THE PROPAGATION PERIODS.

Carmen Andrade

Synopsis: Service life calculations are now treated more in detail in standards and in particular in fib Model Code 2010. The new performance is introduced in addition to probabilistic calculations. In spite of the advances made it is necessary to apply the new approaches with care because any of the existing models have been calibrated in the same concrete more than a couple of decades and then, the predictions may present substantial errors. In addition, the calculation of the propagation period and the limit state of corrosion are aspects not well defined as the MC2010 does not considers any model for the propagation stage. In present paper are analysed some aspects of service life models proposing improvements. A universal statistical distribution of chloride threshold is presented and the consideration of an Initiation Limit State as defined by ISO 13283 is proposed for the depassivation onset. Finally, are illustrated trough examples that the probability of failure for deterioration processes should not be a fixed value but it would depend on the rate of deterioration.

Keywords: chloride threshold; corrosion probability; limits state; reinforcement corrosion, service life.

Biography:

Carmen Andrade is Dr. in Industrial Chemistry, Research Professor of the National Research Council of Spain (CSIC). Her work has been of on durability with particular attention at the phenomenon of corrosion of steel in reinforced concrete. She is author of numerous publications and has several national and international patents. She has been President of UEAtc, RILEM, WFTAO and Liaison Committee. She is Doctor Honoris Causa by the University of Trondheim, Norway and in 2006 by the University of Alicante. She has been General Director of Technological Policy of the Ministry of Education and Science.

INTRODUCTION

Reinforced concrete can remain unaltered during long periods of time, however the concrete can suffer degradation processes and steel can corrode when the carbonation front and the chloride threshold reach the rebar. The corrosion attack has developed numerous studies to find economical preventive methods and repair systems. On the other hand, the advances on high performance concretes are promoting the design requirement of 100 years of service life or more for important structures. These trends are taken into consideration in the fib Model Code 2010¹ in which a performance approach is introduced for the verification of service life together with the classical specifications on concrete mix proportions. For the verification of durability, MC 2010 proposes four routes (chapter 4.3.2): 1) avoidance of damage, 2) deemed-to-satisfy approach, 3) probabilistic safety format and 4) partial safety factor. In the performance treatment, full probabilistic or semi-probabilistic formats are illustrated. This induces the need to define a value for the service life with its failure probability. The optimal corrosion threshold and probabilities have to be selected in function of the consequences of corrosion and of the importance of the section or element and its influence in the structural performance.

Although to specify a target service life and a mechanism of attack seems and advance with respect to the classical approach of deemed-to-satisfy rules for concrete mix, the situation from a practical point of view is however complex, because there are still several undetermined parameters in the application of chloride or carbonation models^{2,3} which being assumed introduce uncertainties. Additionally, as the models are relatively new, they are not calibrated in the same concrete with long term records, which means that there are not cases in which short term tests were performed time ago and verification now could confirm the adequacy of the prediction. The longest timeframe for the calibration of models is around of 20 years^{4,5}.

One of the problems in the models is related to that the initial and boundary conditions of the solution of the diffusion law^{7,8,9} are not fulfilled in the real structures. This is due to both, the surface concentration, C_s , and the diffusion coefficient, D_{ap} , are not constant along the structure's life^{5,9}. This lack of constancy depends on the environmental conditions⁵ and of the cement type⁴. Another key aspect of the calculation of service life in chloride bearing environments is the selection of a chloride threshold, Cl_{th} ¹⁰, which is not a fixed value together with the definition of full absence of corrosion as limit state. In the MC 2010 is proposed a probability of 10% (a reliability index $\beta = 1.3$) typical of serviceability limit states, SLS. The meaning of the probability percentage of depassivation has not been justified¹¹ in terms of real structural performance.

Finally the MC 2010 does not propose any model for the propagation stage of corrosion. The evolution of corrosion depends on the concrete porosity and its moisture state and models have been defined time ago by the author^{12,13}. In present paper are shortly analyzed, due to paper length constrains, several aspects of service life prediction, in particular the corrosion onset and how to link this "limit state of corrosion" to the corrosion propagation and the classical verification of the serviceability and ultimate limit states (SLS & ULS).

CRITICAL ANALYSIS ON CORROSION DURABILITY MODELS IN MC 2010

For the sake of shortening the presentation only chloride ingress process will be used for the sake of illustration. Similar deductions can be made for the carbonation process.

Initiation period

The typical solution for chloride penetration into concrete is the "error function" whose exact analytical solution can be found only for few boundary conditions. One exact analytical solution is found when the external concentration, C_s and the diffusion coefficient are constant, where x is the penetrated distance, is:

$$C(x, t) = C_s \left(1 - \operatorname{erf} \frac{x}{2\sqrt{D_{ap}t}} \right) \quad (1)$$

This solution is modified in MC 2010 by considering that D_{ap} decreases with time¹⁴:

$$D_{ap}(t) = \left[D_{ap}(t_0) \right] \left(\frac{t}{t_0} \right)^{-n} \quad (2)$$

In Bulletin 34¹⁵ is also suggested that x may be affected by a so called "convection zone", Δx (concrete layer, up to which the process of chloride penetration differs from Fick's 2nd law of diffusion).

Prediction Of Service Life Of Reinforced Concrete By Considering The Initiation And The Propagation Periods

Next it will be commented these modifications of equation 1 together with other that make unfeasible its exact analytical solution.

1. Variation of the C_s : Although not included in equation 1, also C_s evolves by increasing with age. This increase is attributed to the binding capacity of the cement¹³. It has an impact on the D_{ap} calculated because, as shown in figure 1, as higher the C_s is, higher is the C_x for a fixed time and D_{ap} . Then, the predictions will be much influenced by the selected values of C_s and therefore, any specification for service life should fix not only the only the D_{ap} , but also of the expected C_s value or its evolution law with time.
2. “Convection zone” Δx ¹⁵: is the thickness of the concrete “skin” zone where a maximum of the chloride concentration is found beyond the concrete surface.

$$x = a - \Delta x \quad (3)$$

Where a is the cover depth and x the distance between the maximum in chloride concentration and the external surface of the bar. It is important because the maximum beyond the concrete surface appears in many occasions and then second Fick's law cannot be directly applied to the whole chloride profile through equation 1. It appears not only due to wet-dry cycles, but also due to the progressive leaching and carbonation that seem to play a role. Until now, the mechanism why in this surface zone the chloride concentrations are increasing instead of decreasing has not been neither determined nor quantified. Then the values given for the thickness Δx are assumptions or empirical.

3. Decrease D_{ap} with time: it depends on the hydration progress and the chloride binding ability of the cement. It has been quantified by equation 2¹⁴ in which the exponent n is called “aging factor”. This empirical factor demands to assume a value for its calculation and to define during which period of time to apply it. The impact of the value of the aging factor n is shown in figure 2. In the figure the decreasing lines indicate the progressive lower values of D_{ap} following the aging factor (which are those used when numerical methods are employed for the calculation) and the horizontal lines are the values calculated from equation 2 (used when a mathematical analytical calculation is used) applied in the whole period. This figure shows that the D_{ap} can change one or more orders of magnitude with a relatively small change of n and then, it results crucial its accurate determination. One possibility would be the measurement of the D_{ap} at two or more ages, which on the other hand makes more expensive and longer the testing.

An additional deduction from the identified real variations of C_s and D_{ap} with time is that for solving equation 1 it is more rigorous the use of FEM numerical methods because there is not exact mathematical analytical solution for that equation with these variations in time. When comparing both mathematical tools for the same conditions, the service life results shorter when using numerical methods.

A final comment on Chloride testing which is other important aspect because the *test type* and the *time* at which to perform the experiments will influence the value of the input parameters. Two types of tests can be used: natural diffusion or accelerated or migration experiments. In the case of chlorides, the migration experiments are performed in short time at a particular concrete age, while the natural diffusion tests give the integration of the evolution of the profiles during the test time period. At European level the migration experiments have not been considered due the impossibility they measure the aging factor in contact to chloride solutions for long time. The proposed method is based in natural exposure to a chloride solution during at least 90 days. A recent decision has been to standardize the measurement of the electrical resistivity as it is a Non-destructive method and can be applied in the same specimen several times.

Then it can be concluded that the prediction of the time to depassivation through present models included in MC 2010, and widely used, are far from being accurate and their results need either to be used only as informative or to be applied by experimented specialists having additional expert opinion on concrete durability and reinforcement corrosion.

Probabilistic treatment

Another present trend is related to the application of structural reliability principles to service life as incorporated in Model Code 2010¹. The structural design is made through the identification of limit states which are defined through Limit States Functions. The simplest LSF defines safety as the requirement that resistance, R , is greater than or equal to the total action effect, S :

$$R - S \geq 0 \quad (4)$$

The probability of failure, P_f , is equal to the probability that S is greater than R . The verification of structural safety consists of demonstrating that the probability of failure, P_f , is smaller than an acceptable target value. For practical application the failure probability is better represented by the *reliability index*, β , through the equation 5 where Φ is the standard normal probability distribution function and P_f is the failure probability corresponding to a specified reference period:

$$\beta = -\Phi^{-1}(P_f) \quad (5)$$

This can be visualized for service life modelling in figure 3 where the initiation and propagation periods have been represented with a mean value and a statistical distribution.

For chloride attack, the Limit State Function suggested is expressed as that the critical chloride concentration must be equal or higher than the chloride concentration at the rebar surface level at the target life time:

$$P\{\} = P_{dep} = P\{C_{crit} - C(c, t_{SL}) < 0\} < P_0 \quad (\text{eq.7.8-11}) \quad (6)$$

where:

$P\{\}$ is the probability that depassivation occurs

C_{crit} is the critical chloride content to achieve depassivation of the reinforcement (wt.% binder content)

$C(c, t_{SL})$ is the chloride content at the depth c and time t (wt.% binder content)

c is the concrete cover [mm]

t_{SL} is the design service life [years]

P_0 is the target failure probability

The difficulty of this approach is the lack of data on the statistical distributions of the input parameters. As mentioned the surface concentration and the D_{ap} value evolve with time and additionally they have a scatter which is unknown because no records exist. On the concrete cover there are more studies showing that a coefficient of variation from 15% to 30% can be reasonable.

Regarding the critical chloride content is known that it is not a fixed quantity as it is influenced by several variables, The MC 2010 literally says¹ «...The critical chloride content, C_{crit} , when the depassivation of the reinforcement occurs, is also a variable depending on factors like steel properties, type of cement and additions, moisture content etc. For practical design, a realistic distribution has to be applied. The fundamental knowledge on this property is still limited, and most numbers reported in the literature are based on in-field observations. Often these observations are grouped in tables with qualitatively described ranges as “negligible risk of corrosion”, “possible risk of corrosion” etc. When performing a probabilistic service life design, it is imperative to make clear if the actual limit state is linked to occurrence of corrosion (based on the mean and the statistical spread of C_{crit}), or to a conservative deterministic value of for example “negligible risk” or “possible risk”... Then, as in the other input parameters the designer has no clear indication on which value and which statistical distribution to adopt. This makes the design very expert-dependent.

Finally regarding the corrosion propagation MC 2010 indicates that not an agreed model was found and no indications on how to calculate the progression of corrosion is incorporated.

CONTRIBUTIONS TO IMPROVE SERVICE LIFE CALCULATIONS

New models or improvements of present ones are a necessity because a more accurate calculation of service life is part of the trend for more sustainable concrete structures. In present paper are described briefly some few aspects that can make less uncertain the predictions in the case of chloride ingress. The aspects to be commented are: a) the statistical distribution of the chloride threshold, b) the need to consider the depassivation onset not as SLS but as a Limit state of initiation of deterioration and c) the consideration of a model for corrosion propagation.

Statistical distribution of the chloride threshold

The normal value considered in numerous standards is of 0,4% by weight of cement. The service life is considered ended when this amount reaches the external part of the bar. However, the process is not simple because the pits are initially of microns in size and progressively the corroded area is enlarging. Due to the bar has a diameter of around 10 mm or more, the depassivation of the whole perimeter of the bar that is the progression of the threshold front until the internal surface of the bar, may take decades if the diffusion coefficient is low.

Then, in order to treat this situation different approaches are feasible: one would be to consider the whole bar diameter and identify its middle point as that where the deterministic value of the chloride threshold has to be verified. That is, to consider the bar position, not as the external surface but as its middle point in diameter. Our approach has been to take the corrosion onset as a random process which has to be defined with a statistical distribution. Statistical distributions of the Cl_{th} are few in the literature^{16, 17, 11}. That of Hausmann is expressed in function of the Cl/OH ratio and then it is more difficult to be used. In figure 4 are reproduced two ones^{11, 17} which are expressed in function of the chloride concentration alone. That from Markeset is a lognormal distribution and was collected in real structures in Norway with a mean value of $0,77 \pm 0,24\%$ by cement weight while that of Izquierdo et al. is a normal distribution with a mean of $0,70 \pm 0,20$ and was built from laboratory results made on mortar specimens fabricated with different cement types and submitted to potentiostatic tests. In spite of the very different sources of the data, they seem coherent from the point of view that in real conditions and real concrete, the amount of chloride needed for producing corrosion is a bit higher than in the laboratory