



# Durability Modeling to Determine Long Term Performance of Historic Concrete Structures

Gina Crevello<sup>(✉)</sup>, Irene Matteini, and Paul Noyce

Echem Consultants LLC, Poughkeepsie, NY, USA  
{gcrevello, imatteini, pnoyce}@e2chem.com

**Abstract.** Understanding the impact of material degradation, rates of deterioration, and condition state changes are critical in making sound repair choices for historic concrete structures. Forecasting remaining service life and planning for durability is critical, particularly when the subject structure is an irreplaceable landmark. By monitoring the long-term performance of a structure, a corrosion and degradation rates can be established. Multiple parameters on the subject reinforced concrete structure are tested and monitored. The data is then utilized in durability and service life models to understand where the structure is in regard to critical performance thresholds and when failures may occur. This approach, when applied to historic structures, can help provide an understanding of ‘anticipated remaining service life’ and to assist in developing a proactive repair. This will minimize future degradation to the historic building fabric. For this paper, an approach to carbonation-based durability models will be presented in relation to historic concrete buildings.

**Keywords:** Corrosion monitoring · Historic concrete · Durability engineering  
Service life modeling · Carbonation

## 1 Introduction

Understanding the rate of material degradation can be critical in preserving the fabric of historic concrete buildings. Durability models can be utilized for predicted performance of specified materials within their given environment. This allows for the determination of remaining service life. By forecasting the probabilistic long-term behavior of corrosion related degradation, preventative maintenance and proactive repairs can be planned.

Durability modeling is based on a detailed material assessment that focuses on structure specific conditions. The assessment identifies mechanisms of deterioration, including physical, chemical, structural, mechanical and electrochemical damage, which is incorporated into mathematical models. The models are based on predetermined degradation thresholds. Long-term monitoring systems can be installed to provide extensive data ranges and higher statistical reliability for the analysis. This approach allows owners to make informed decisions on the best repair choice for the life extension of the buildings and can also be successfully used to determine how repairs perform.

## 2 Historic Concrete and Deterioration

### 2.1 Preserving Concrete

Reinforced concrete revolutionized construction of the 20th century. Design, construction methods, materials, and standards of workmanship will vary enormously according to the date of construction. In addition, location and climate will play a part in the deterioration. All these factors will affect the durability of the structure [1].

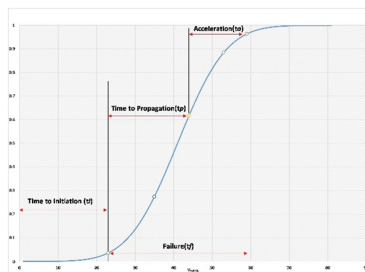
As concrete buildings begin to age, those deemed architecturally significant will require material conservation. Challenges exist in the assessment and repair of historic concrete buildings, however an appropriate repair program addresses existing distress and reduces the rate of future deterioration [2]. Related disciplines such as construction history, conservation philosophy, concrete durability and concrete repair [3] must be integrated to properly assess performance.

### 2.2 Establishing Condition States

To understand durability, a condition state rating system and service life analysis will define the current condition of the structure. Visible defects are identified as known conditions. The unknown conditions, or the areas that are not yet visibly defective, is where durability modeling and a service life analysis is required. Data collection and subsequent analytical procedures provide timeframes on deterioration critical to future performance. The service life assessment should provide a time line of when thresholds are reached in the structure and when obsolescence will occur [4]. When studying behavior of historic concrete, condition state parameters should include carbonation, temperature, relative humidity and corrosion.

### 2.3 Corrosion Deterioration

Corrosion of reinforcing steel is arguably the most detrimental deterioration mechanism to concrete. It is not a question of if a concrete structure will corrode; it is a question of when it will corrode. Traditional corrosion models established by Tuutti [5] define three



**Fig. 1.** Modified Tuutti model

modes of time based deterioration in reinforced concrete structures:  $t_i$ ,  $t_p$ ,  $t_f$  (see Fig. 1).

Where:

$t_i$  = Time to Corrosion Initiation

$t_p$  = Time to Corrosion Propagation

$t_f$  = Time to Failure.

The corrosion process for steel reinforced concrete can be simplified into a two-stage process, namely the ‘initiation phase’ and the ‘propagation phase’. The initiation phase is the time taken for conditions to become conducive to corrosion through carbonation or chloride ingress. The propagation phase is the period in which the accelerated corrosion of the steel reinforcement ultimately leads to rust staining, cracking and spalling of the cover concrete [6].

When corrosion of steel is concerned, the service life [ $t_1$ ] is assumed as the sum of an initiation period and a propagation period:  $t_1 = t_i + t_p$  [5]. Initiation of corrosion is usually chosen as limit state for chloride-induced corrosion [or carbonation]... As a consequence, the service life [of the reinforced concrete structure] is assumed to be coincident to the initiation time (i.e.,  $t_1 = t_i$ ) [7]. In chloride rich environments, the service life or  $t_1$  has proven to be very short in some instances and corrosion is certainly anticipated on structures within 20 years or less.

Carbonation induced corrosion, which tends to initiate at cracks and defects, typically has an extended  $t_i$ . Rates of carbonation are variable, and depend upon material characteristics of the concrete, such as:

- (1) the level of pore water, i.e. relative humidity;
- (2) grade of concrete;
- (3) permeability of concrete;
- (4) whether the concrete is protected or not;
- (5) depth of cover; and
- (6) time [8].

Typically,  $t_1$  for historic concrete buildings subject to carbonation is longer than structures subject to high chloride loads (i.e. bridges, parking garages, marine structures).

### 3 Establishing the Parameters

#### 3.1 Establishing a Carbonation Protocol

Concrete is highly alkaline in nature. Carbon steel in alkaline concrete will have negligible rates of corrosion while the  $\text{Fe}_3\text{O}_4$  (passive layer) is intact. Carbonation is the reaction between the cement hydration products in the concrete and atmospheric carbon dioxide ( $\text{CO}_2$ ), which leaves the steel vulnerable to corrosion [9]. The resulting drop in pH leaves the steel unstable, thus subject to corrosion in the presence of oxygen

and moisture. Carbonation rates generally follow parabolic kinetics where the depth of the reaction on a sample or structure is proportional to the square root of time (Eq. 1):

$$d = At^{0.5} \quad (1)$$

where:

d = carbonation depth,

t = time and

A = constant, generally of the order 0.25 to 1.0 mm.year<sup>-0.5</sup> [10].

The progress of the carbonation front is the most common means of expressing the extent to which carbonation has occurred [9]. Fridh and Lars-Olof suggest that carbonation is a moving boundary where the rate is dependent upon the amount of CO<sub>2</sub> in the air, the diffusion coefficient of the carbonated concrete (established by the NT 492 Test), as well as the amount of CaO which can be carbonated [11]. Simple phenolphthalein tests which are spray or brush applied indicate when pH has dropped below 8.3, however the loss of the passivity usually occurs around pH 11 [8] to 11.5 [9]. A carbonation front can be 5 mm ahead of the delineator. Therefore, phenolphthalein test results are not sufficient to gauge actual carbonation depths. Once carbonation occurs at the steel corrosion may ensue.

### 3.2 Temperature and Relative Humidity

Corrosion kinetics are influenced by oxygen availability, temperature and moisture. The three environmental factors that have the largest impact on the rate of carbonation are CO<sub>2</sub> concentration, relative humidity [RH] and temperature [9]. Therefore, these factors are to be considered when modeling future carbonation progress and corrosion risk. The overall effect is that the optimum rate of carbonation occurs at 55% RH [9]. RH values greater than 75% will provide sufficient moisture for corrosion to initiate in carbonated concrete. The most aggressive corrosion activity will occur in carbonated concrete with RH values between 85% and 95%, at temperatures above 20 °C [12].

Since the deterioration of concrete is based on diffusion or chemical reactions, the deterioration progress can also be considered as having temperature dependency. Generally, the rate of material diffusion and the chemical reactions rise with increases in temperature [13].

### 3.3 Site Specific Corrosion Data

To establish site specific condition states from corrosion activity, Linear Polarization Resistance (LPR) testing is required. In any real corrosion system, corrosion current ( $I_{\text{corr}}$ ) and corrosion rate are a function of many system parameters, including type of metal, composition of the electrolyte solution, concrete type, temperature, movement of the solution, material resistivity, etc. LPR measures both Half-cell potential ( $E_{\text{corr}}$ ) and  $I_{\text{corr}}$ .  $I_{\text{corr}}$  is the loss of electrons occurring at the anodic site of the corrosion cell and is measured in microAmps per square centimeter ( $\mu\text{A}/\text{cm}^2$ ) [14].

The values of  $I_{\text{CORR}}$ , are used to then calculate steel section loss (corrosion rates) using Faraday's Law of Metal Loss, and measures in microns per year ( $\mu\text{m}^{\text{yr}^{-1}}$ ). The rate of degradation of the steel and the predictive condition state can be established, as seen in Table 1. As LPR is variable, more accurate models can be generated if monitoring probes can be installed. Otherwise, seasonal measurements should be acquired to understand  $I_{\text{CORR}}$  values. In buildings where RH is 85%, corrosion rates of  $3 \mu\text{m}^{\text{yr}^{-1}}$  would be typical, at 90%  $12 \mu\text{m}^{\text{yr}^{-1}}$  and at 95% 50 respectively [12]. These values drop to  $2 \mu\text{m}^{\text{yr}^{-1}}$  or less in saturated concrete.

**Table 1.** Corrosion rates and condition states [14]

Condition state	Corrosion rate ( $\mu\text{m}/\text{yr}^{-1}$ )	Expected section loss (%)	Comments
5	<0.1	<1	Very low
4	0.1–1.16	1–9	Low to medium
3	1.16–3.34	10–17	Medium
2	3.34–11.6	18–25	Medium to high
1	>11.6	>25	Very high

## 4 Case Study

### 4.1 Concrete Building Case Study

A reinforced concrete museum in a metropolitan area, constructed in 1954, underwent a major repair campaign in 2007–2008. The building was identified to be suffering mildly from carbonation induced corrosion. The walls comprise 12.7 cm thick gunite which are rigidly connected to a ramp slab (floor) and web walls. Original construction records indicate that the walls were reinforced with two layers of 50 mm ( $2 \times 2$ ) mesh, plus 2 layers of No. 3 bars at 30 cm centers horizontally, 2 layers of No. 4 bars vertically, and top and bottom continuous No. 4 bars—all secured to a framework of 38 mm 'tees' which are in turn tied into the structure. The outside surfaces were smoothed after the forms were removed, but while the gunite was still green [15]. Soon after construction the exterior walls were painted with a marine coating.

### 4.2 Verification of Conditions

Prior to the corrosion monitoring program, a full condition assessment and structural analysis were performed. The review indicated carbonation had diffused between 1 and 6 mm into the surface of sound concrete from the exterior and up to 17 mm from the interior face of the gunite wall. Corrosion had been documented along the cracks which were parallel with the reinforcing steel of the rotunda dating back to 1989, thirty years after completion. Reports up to 2006 confirmed visual correlation between hairline cracking, and minor corrosion activity at the reinforcement. Reinforcing steel cover varied, however low cover was approximately 25 mm.

### 4.3 The Condition Survey

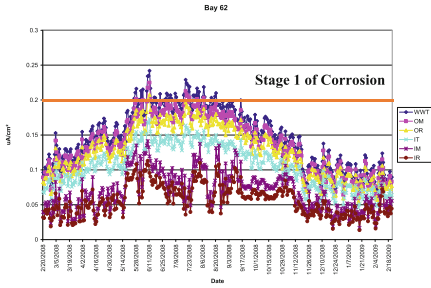
A corrosion condition assessment was performed with half-cell ( $E_{\text{corr}}$ ) and corrosion rate ( $I_{\text{corr}}$ ) values measured and mapped on two areas of the building. This included the 6<sup>th</sup> floor rotunda (external), and the 2<sup>nd</sup> floor gallery wall (internal). The analysis of half-cell potentials values, and potential gradient mapping provided visual support of corrosion activity. The corrosion assessment indicated that only 4% of the area tested exhibited corrosion rates greater than  $2.2 \mu\text{m}^{\text{yr}^{-1}}$  (Condition State 3). This would be considered moderate [16] in most structures. However, higher rates measured during the testing program from Condition State 2 ( $4 \mu\text{m}^{\text{yr}^{-1}}$ ) indicated ongoing damage in the form of cracking within 10 years at areas of low cover. This level of degradation is significant for a landmark concrete structure. The client required on-going and continued use of the facility. As the data from the survey indicated future risk, a long-term corrosion monitoring system was installed to better understand the performance of the structure, and post-repair conditions.

### 4.4 Monitoring System

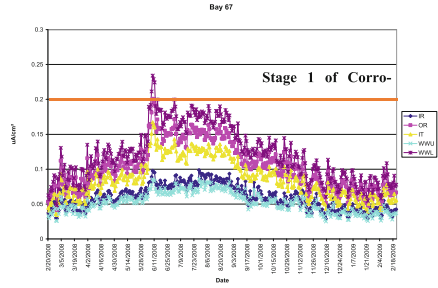
The subject areas of the monitoring program included the survey areas at 6th floor building envelope walls (Rotunda) and the 2nd floor gallery walls. The system consisted of embedded temperature and humidity probes and LPR probes comprised of Silver/Silver Potassium Chloride (0.5 M) reference electrodes, each with an auxiliary Mixed Metal Oxide Titanium electrode. Both half-cell potential and corrosion rate values could be collected and measured from the same probe. Probes were installed at high risk locations with two controls at an internal web wall.

The locations deemed high risk for the 6<sup>th</sup> floor Rotunda were in two 'Bays' identified as 62 and 67. 62 is a West facing bay with a neighboring adjacent high-rise building, and Bay 67 is south facing, where the highest environmental exposure was noted. LPR Probes were placed to measure the performance of the following steel components: the Web Wall Double 'Ts' i.e. the internal steel connection angles at transverse web wall and exterior wall; Intermediate 'Ts'; Inner Rebar, Outer Rebar; Inner Mesh, Outer Mesh. Data for the initial year is provided in Figs. 2 and 3, with statistic of corrosion rate, temperature and humidity in Tables 2, 3, 4, 5 and 6.

Temperature and humidity probes were installed in three 'spaces' within the two bays. The probe placement was (1) embedded within the walls; (2) within the interstitial space at the internal wall face enclosed within a protective box, and (3) within the insulation under the air barrier for internal ambient. The data has been collected over an eight-year period, however for the purposes of brevity the first year of corrosion data is being reviewed.



**Fig. 2.** Bay 62 LPR monitoring data over 1 year



**Fig. 3.** Bay 67 LPR monitoring data over 1 year

**Table 2.** Statistical data corrosion current Bays 62

Bay 62	WWT	OM	OR	IT	IM	IR
	uA/cm <sup>-2</sup>					
Min	0.064	0.059	0.055	0.046	0.017	0.013
Max	0.242	0.225	0.208	0.173	0.141	0.113
Average	0.148	0.137	0.127	0.106	0.070	0.056
Range	0.178	0.166	0.153	0.127	0.124	0.099

**Table 3.** Statistical data corrosion rate Bay 62

Bay 62	WWT	OM	OR	IT	IM	IR
	μm <sup>-yr</sup> <sup>2</sup>					
Min	0.74	0.69	0.63	0.53	0.19	0.16
Max	2.81	2.61	2.41	2.01	1.64	1.31
Average	1.72	1.59	1.47	1.23	0.81	0.65
Range	2.07	1.92	1.77	1.48	1.44	1.15

**Table 4.** Statistical data corrosion current Bays 67

Bay 62	IR	OR	IT	WWU	WWL
	uA/cm <sup>-2</sup>				
Min	0.029	0.041	0.035	0.025	0.048
Max	0.099	0.201	0.167	0.086	0.234
Average	0.060	0.104	0.087	0.052	0.121
Range	0.070	0.159	0.133	0.061	0.186

**Table 5.** Statistical data corrosion rate Bay 67

Bay 62	IR	OR	IT	WWU	WWL
	$\mu\text{m}^{-\text{yr}^2}$				
Min	0.33	0.48	0.40	0.29	0.56
Max	1.14	2.33	1.94	1.00	2.72
Average	0.69	1.21	1.01	0.61	1.41
Range	0.81	1.85	1.54	0.71	2.16

**Table 6.** Embedded temperature and humidity data Bay 62 and 67

Data set	Temperature		RH	
	62A	67A	62A	67A
Min	24	25	76	61
Max	94	91	100	98
Average	61.18	58.06	86.45	74.5
Range	70	66	24	37

## 5 Conclusions

The long-term data acquired from the monitoring system illustrated the improved performance of the building envelope. The values of the data collected from the LPR readings indicates that over the course of the first year of monitoring, the building shifted corrosion risk categories. Prior to the repairs, the average values were 2.2 microns. After the monitoring system was installed, the average corrosion rates were measured at less than 0.106 micro/Amps per square centimeter ( $<0.1 \mu\text{A}/\text{cm}^2$ ) or 1.24 microns ( $\text{m}^{-\text{yr}}$ ) section loss for Bay 62 and 0.08 micro/Amps per square centimeter ( $<0.1 \mu\text{A}/\text{cm}^2$ ) or 0.98 microns ( $\text{m}^{-\text{yr}}$ ) section loss. This data indicates that the reinforcing steel in the location of the probes to be a low risk of corrosion. Using the new inputs in the crack propagation models, anticipated damages were predicted to occur  $>25$  years. The decrease in the wall RH from 100% at the time the coating was applied, to values less between 76% and 65% over a 1-year period indicated that corrosion condition states were decreasing and building envelope conditions had improved. The information indicated a reduction in deleterious conditions impacting corrosion.

The general assumption made by the various system reviews supported that the stable conditions remain and advancing conditions states were not posing a risk to the building elements which were monitored. These assumptions can be based on reducing the movement of the carbonation front, reducing moisture and measuring on going corrosion activity. In this instance, coating integrity is required to minimize further corrosion propagation. The monitoring allowed for a determination of repair performance to be assessed and to scientifically gauge whether further interventions were required.



The comprehensive approach can provide a timeline for degradation, budgeting and interventions to landmark concrete structures prior to damage occurring on the structure. In this instance, understanding the risks affecting advancing condition states, or a reduction, allowed the team assurance in their chosen repair. The work demonstrated that the conditions on the structure stabilized over time, further reducing moisture, and subsequent corrosion in the building envelope. By understanding the reactions driving corrosion kinetics and how active corrosion will become once initiated, allows time frames for intervention to be developed.

**Acknowledgments.** The authors wish to acknowledge the building owners and design teams which were instrumental in the overall assessments.

## References

1. MacDonald S (ed) (2003) Concrete building pathology. Blackwell Science, Oxford
2. Gaudette P, Slaton D (2007) Preservation briefs 15: preservation of historic concrete. United States Department of the Interior. National Park Service
3. Heinemann H, Zijlstra H, van Hess RJJ (2012) From concrete repair to concrete conservation: how to preserve the heritage values of historic concrete. In: Grantham M, Mechtcherine V, Scheck U (eds) Concrete solutions: proceedings of concrete solutions, 4th international conference on concrete repair, Dresden, Germany, 26–28 September 2011. CRC Press, Boca Raton
4. Crevello G, Noyce P (2015) Service life prediction for reinforced concrete. *Structure Magazine*, October 2015
5. Tuutti K (1982) Corrosion of steel in concrete. Swedish Cement and Concrete Institute, Stockholm
6. British Research Establishment (BRE) (2013) Client Report. Residual life models for concrete repair: assessment of the concrete repair process. BRE, London
7. Bertolini L et al, Milan Politecnico di Milano, Italy (2016) Corrosion of steel in concrete and its prevention in aggressive chloride-bearing environments. In: 5th international conference on durability of concrete structures June 30–July 1 2016, Shenzhen University, Shenzhen, Guangdong Province, P.R. China
8. Ware T (2013) Diagnosing and repairing carbonation in concrete structures. *J Build Surv Appraisal* Vol 1(4):338–344. Henry Stewart Publications 2046-9594
9. Dyer T (2014) Concrete durability. CRC Press/Taylor and Francis Group, Boca Raton
10. Revie W (2011) Uhlig's corrosion handbook, 3rd edn. Wiley Press, The Electrochemical Society, Boca Raton
11. Fridh K, Lars-Olof N (2014) A model of carbonation and a technique to determine the degree of carbonation. In: XIII international conference on durability of building materials and components, September 2–September 5 2014, RILEM Proceedings Pro 96, San Paulo, Brazil
12. Sarja A, Vesikari A (1996) Rilem Report 14: durability design of concrete structures. E&F Spon

13. Otsuki N et al (2009) Temperature dependency on chloride induced corrosion. *Jpn Inst Concr J Adv Concr Technol* 7(1):41–50
14. Crevello G, Matteini I, Noyce P (2016) Understand rates of corrosion in concrete structures. *Structure Magazine*, April 2016
15. Cohen G (1958) Frank Lloyd Wright's Guggenheim Museum: he adds new dimensions to use of poured concrete in building construction. *The Aberdeen Group*
16. Broomfield J (1997) *Corrosion of steel in concrete: understanding, investigation and repair*. E&FN Spon, London