From Cradle to Maturity

A holistic service-life approach for concrete bridges

by Guillermo Di Pace, Luis Ebensperger Morales, Roberto Torrent, and Verónica Bueno

While most taxpayers are probably unaware of the day-by-day degradation process within bridges, the deterioration seriously affects their pocketbooks. After all, they ultimately make the payments needed to support the budgets that governments dedicate to remedy, repair, or retrofit bridges.

A 2001 survey showed that 1/3 of inspected European bridges (predominantly concrete) exhibited symptoms of deterioration, with carbonation- and chloride-induced steel corrosion being the more recurrent cause of distress.[1] Similar data have been reported on other continents.

Examples within Europe include two prominent structures that had to undergo costly repairs after only 20 to 25 years of service, even though they were designed for service lives of 100 years. The alpine Ganter Bridge in Switzerland (Fig. 1) and the marine Krk Bridge in Croatia (Fig. 2) both suffered premature chloride-induced steel corrosion.[3,4] This is strong evidence that the profession of bridge engineering has been far from successful in achieving durability, and this problem deserves further examination.

One explanation is that designing for durability involves physical, chemical, and electrochemical knowledge that many civil engineers lack (although modern curricula of civil engineering are trying to bridge this gap). Probably the main explanation for that failure, however, is the mistaken belief that a bridge in an aggressive environment can achieve a service life of 100 years through the application of conventional design, materials, construction, and inspection practices. This misbelief is fed by conceptually wrong approaches in standards and codes. For instance, Eurocode 2[5] stipulates that 100 years of service life can be achieved by merely increasing the cover depth of steel by 10 mm (0.4 in.) over that specified for 50 years of service life, even though the same concrete specification applies for either service life. This combination of cover depth and concrete quality is clearly insufficient and will not produce a durable structure.[6]

Modern bridge design calls for long service lives—ISO 2394:2015[7] stipulates design service lives of 100 years or more for large bridges. However, because life-cycle cost analyses often show that rehabilitation produces a significant direct cost as well as very high negative externalities such as traffic delays, increased pollution, and vehicle damage, a modern approach calls for designing for more than 200 years of service life.

Fig. 1: Ganter Bridge, Switzerland, undergoing repair

Fig. 2: Krk Bridge, Croatia, shortly before undergoing repair[2]

Achieving a long service life for a bridge exposed to aggressive environments (such as urban, industrial, tropical, marine, and cyclic freezing and thawing) is not an easy task and requires special efforts from all parties involved in its design, construction, inspection, and maintenance. Unless owners, designers, materials suppliers, contractors, inspection and testing bodies, and consultants recognize the challenge and fulfill their tasks with the utmost care and responsibility, the expected durability performance will not be achieved.

Hence the title of this contribution, the objective of which is to review the critical aspects in every step of project development—from birth until maturity—and highlight the actions to be taken to achieve a durable structure, fulfilling the owners' and society's expectations. The focus is placed on durability design against chloride-induced steel corrosion, which is the main deterioration process, although it can be easily adapted to other cases such as carbonation-induced steel corrosion.

Project Phases

Feasibility phase

In this phase, usually starting many years before the initiation of construction, it is important to get accurate information on the macro- and microclimatic conditions to which the bridge will be exposed and on their possible effects on its durability. It is highly recommended to build mock up elements, made not only with durable concretes, but also with advanced systems that may have become industrially available by the time of the bridge construction. These mockup elements should be placed in environments similar to those to be experienced by the real structural elements.

Extensive advanced testing of the mixtures and regular monitoring of the mockup elements should provide useful information on early properties and middle- and long-term behavior of the systems. This information will enable calibration of models and more accurate predictions of the chloride penetration and steel corrosion processes. A good example of such an endeavor is the Hong Kong-Zhuhai-Macao link project, where specimens were installed and monitored for 30 years before the initiation of the construction, providing key information for the service-life design of the different structures composing the link.[8]

Design phase

This is a crucial phase of the project, where the main characteristics of the concrete elements, expected to last 100+ years, are defined and translated into the specifications of the project. This is typically done through predictive modeling. In Fig. 3, we present Tuutti's model of deterioration,[9] applied to steel corrosion due to chlorides or carbonation, and expanded by Nilsson.[10]

As the figure shows, there is an initial period without considerable damage, called incubation, that ends at corrosion initiation time T_i. Within this period, chlorides (or carbonation) penetrate the concrete surface until the front of critical concentration reaches the reinforcing steel (lower part of Fig. 3). The steel then becomes depassivated and prone to initiate the corrosion process. For chloride-induced corrosion, T_i is often associated with the service life of the element. Thereafter, the process is called corrosion propagation. The corrosion propagation time T_p depends on the considered limit state. There is a serviceability limit state (noted SLS in Fig. 3), which can be associated with localized cracking and spalling, and an ultimate limit state (noted ULS in Fig. 3) corresponding to loss of bearing capacity of the structural element due to loss of bond and reduced section of steel and concrete.

Most models developed to predict T_i of structures (DuraCrete,[11] *fib* Bulletin No. 34,[12] and Life-365[13]) are based on the simplified assumption that the penetration of chlorides happens exclusively by nonsteady state diffusion, which obeys Fick's second law of diffusion. If the coefficient of diffusion and the chloride concentration at the concrete surface are constant (none of these assumptions are true), Fick's second law accepts an explicit solution given by Eq. (1)

$$C(x,t) = C_0 + (C_s - C_0)\left[1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right] \quad (1)$$

where $C(x,t)$ is concentration of chlorides in concrete at depth x and time t; C_0 is initial concentration of chlorides in the concrete; C_s is concentration of chlorides at the surface of concrete (assumed constant); erf is error function; and D is coefficient of diffusion to chlorides expressed in length squared over time units, typically m²/s.

T_i happens when the concentration of chlorides at the reinforcement cover depth d reaches a critical value C_{cr}. Assuming that $C_0 = 0$, substituting $C(d,T_i) = C_{cr}$ for $C(x,t)$, and reorganizing the terms, T_i is calculated using Eq. (2)

$$T_i = \frac{d^2}{4D}A^2 \quad (2)$$

where

$$A = \frac{1}{erf^{-1}\left(1 - \frac{C_{cr}}{C_s}\right)} \quad (3)$$

Note that erf^{-1} is the inverse of the error function.

Although Eq. (1) was derived assuming that D is constant, many models are based on the assumption that the coefficient of diffusion of chlorides in concrete decreases with time. This has been observed experimentally, and various justifications for this phenomenon have been proposed (refer to *fib* Bulletin No. 76[14]). A common assumption is shown in Eq. (4)

$$D = D_0\left(\frac{t_0}{t}\right)^a \text{ for } t \le t_a \quad (4)$$

where D is the coefficient of diffusion of chlorides at age t; D_0 is the coefficient of diffusion of chlorides measured at age t_0; a

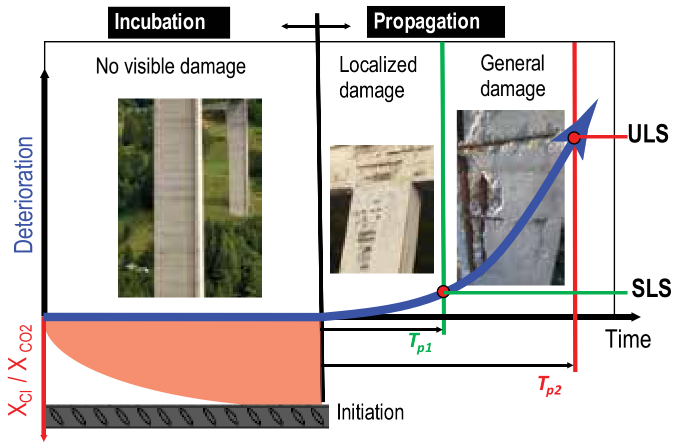


Fig. 3: Tuutti's model of concrete deterioration,[9] expanded by Nilsson[10]

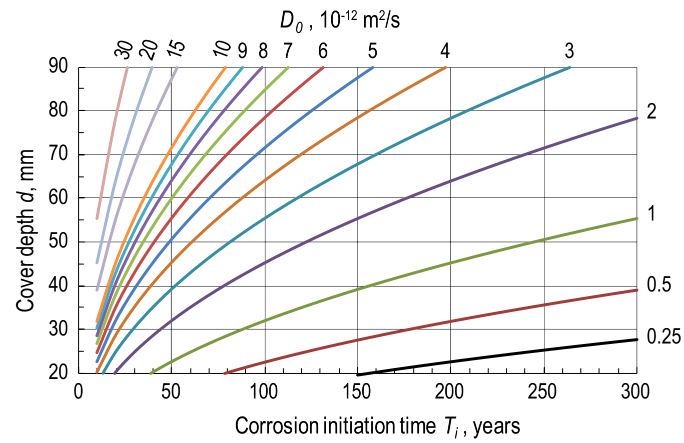


Fig. 4: T_i as a function of d and D_0 for exposure classes XS3 (tidal, splash, and spray marine zones) and XD3 (deicing salts, cyclic wet and dry) in Eurocode 2[5]

is an exponent indicating the intensity of the aging effect; and t_a is the duration of the aging effect on the reduction of D. If $a = 0$, D is constant and not affected by age (the only mathematically correct case).

Equations (2) through (4) constitute the core of the best-known models to predict service life against chloride-induced corrosion (DuraCrete,[11] *fib* Bulletin No. 34[12]) while Life-365[13] follows a more rigorous numerical approach. While getting the pair of values d and D_0 to be specified to achieve the design service life T_i may appear to be trivial, the computed values are extremely sensitive to the elusive parameters C_s, C_{cr}, a, and t_a. Parameters C_s and C_{cr} are very difficult to determine, and their ratio may affect T_i by a factor of 4.[15] Moreover, the validity of the power decay function (Eq. (4)) and of the proposed values of exponent a are the subject of much uncertainty and controversy.[16,17] The effect of a on T_i can be dramatic, leading to orders of magnitude effects on T_i and even to absurd values close to eternity, particularly if t_a is not limited.[18,19]

Recommendations vary widely. Life-365,[13] for example, proposes a t_a value of 25 years, while the DuraCrete[11] (explicitly) and *fib* Bulletin No. 34[12] (implicitly) models extend t_a to the entire T_i of the structure. Some experts suggest, more reasonably, a value not longer than 10 years,[20] and others recommend that t_a is limited to a year or less. In the latter cases, the recommendation is based on the observation that after an appropriate curing period, there will be no substantial decrease in Cl^- permeability under jobsite conditions.

The experimental determination of a is difficult due to the long-term nature of the exponent. Therefore, it is not possible to perform calculations with representative values of a and t_a at the time of the service-life design. Herein, we can see the importance of the long-term tests on mockup elements mentioned earlier in this article. An excellent example of how the information collected from these elements is used to make a well-founded service-life design can be found in Li et al.[8]

In fact, when accurate information on the parameters in Eq. (2) through (4) is not available, the designer has to rely on values proposed by the models, many of which are not the result of experimental data, but primarily based on "expert opinions."[20] The selective choice of these parameters makes the predictions very weak and prone to subjective judgments and manipulations, to "get what you want."

The previously mentioned difficulties in establishing parameters in the model are often disguised behind so-called "probabilistic treatments" by which certain statistical distributions are attributed to each factor in Eq. (2) through (4). These distributions are questionable guesses (again, in many cases the result of "expert opinion"), because the central values of the distributions are not known with certainty (let alone the complete distributions). A certain confusion exists between terms "probability" (or stochastic uncertainty) and "ignorance" (epistemic uncertainty).[21] Perhaps a treatment along the lines of "fuzzy logic" may be more appropriate to deal with such elusive variables, as attempted by Altmann and Metcherine.[22] Another interesting approach is the use of combined multiple models for data interpretation and predictions.[23]

A simple yet robust method has been proposed.[15] The method is based on Eq. (2) through (4), but the need for the elusive parameters is waived. This method can be used to make a preliminary service-life design or to check whether a more sophisticated calculation is providing a reasonable solution. It consists of a simple formula that can be easily presented as a chart, as shown in Fig. 4, for severe marine and deicing salts exposures (XS3—tidal, splash, and spray zones, and XD3—cyclic wet and dry, in Eurocode 2[5]). Similar charts may be built for other exposure conditions.

Associating service life (SL) with T_i can be accepted for structures subjected to severe environments like tidal and splash zones, where T_p is rather short—6 years for Life-365.[13] This is because the wet-dry alternating conditions ensure sufficient supply of oxygen (O_2) and moisture to promote the electrochemical reactions through the low-resistivity, Cl^- contaminated concrete cover. For less severe environments and especially for the case of carbonation-induced corrosion, the criterion of making $SL = T_i$ is too conservative and the propagation time must be taken into account.

The problem of modeling the propagation time is more complex than modeling T_i, because several phenomena have to be considered. For example, corrosion rate, function of resistivity and O_2 supply, and fracture process (concrete crack opening and growth) must be modeled. A review of models can be found in Otieno et al.[24] One of the most widely used models[25] is provided in Eq. (5)

$$T_p = \frac{P_x \rho}{k_{corr}} \tag{5}$$

where P_x is accepted loss in diameter or pit depth in the reinforcing bar; ρ is the electrical resistivity of the concrete; and k_{corr} is a constant.

More recently, a more comprehensive approach has been proposed.[26] Besides resistivity, this model includes fracture mechanics parameters and air permeability as indicator of the O_2 supply through the concrete cover. This holistic approach is based on over 20 years of research performed at the Corrosion Laboratory of the University of South Florida, and it is valid for the worst possible scenario—splash and tidal zones in a marine environment. The corrosion propagation period, in terms of time to cover cracking T_{cr}, can be determined if both the projected average corrosion rate CR and the critical reinforcing bar radius loss X_{crit} are known. The relation between T_{cr}, CR, and X_{crit} is depicted in Eq. (6) and (7)

$$T_{cr}\,(\text{year}) = X_{crit}\,(\text{mm}) / CR(\text{mm/year}) \tag{6}$$

$$X_{crit}\,(\text{mm}) \approx 0.011 \frac{K_{1C}}{K_{1Cref}} \left(\frac{d}{Ø}\right)\left(\frac{d}{L}+1\right)^{1.4} \tag{7}$$

where K_{1C} is concrete fracture toughness in $N/mm^{3/2}$; K_{1Cref} is 35.9 $N/mm^{3/2}$; d is concrete cover in mm; Ø is reinforcing bar diameter in mm; and L is the anode length in mm. CR (in mm/year) is given by Eq. (8)

$$CR = \frac{V_{corr}}{\left(\log\left(\frac{kT_0}{kT}\right)+1\right)\rho} \tag{8}$$

where V_{corr} is the basic corrosion rate, equal to 0.3016 mm/year; ρ is concrete surface resistivity in kΩ·cm, determined per AASHTO T 358[27]; kT_0 is air permeability threshold (0.08×10^{-16} m^2); and kT is air permeability (10^{-16} m^2), determined per SIA 262/1:2013.[28] L can be assumed equal to Ø for localized corrosion and $L \to \infty$ for generalized corrosion. In a practical case L can be assumed to vary between Ø and 1000 mm.

Equation (8) is valid for $kT \leq 0.08 \times 10^{-16}$ m^2. For $kT > 0.08 \times 10^{-16}$ m^2, O_2 availability is assured, so the corrosion rate equation may be simplified as shown in Eq. (9)

$$CR = \frac{V_{corr}}{\rho} \tag{9}$$

When this approach is applied to high-performance concretes in structures with concrete covers in the range of 75 to 90 mm (3 to 3.5 in.), propagation times of more than 20 years are predicted.

These are tools available to make a preliminary SL design, which allows specifying the two main characteristics that define the resistance against the initiation of corrosion: the cover depth d and the chloride diffusion coefficient D_0, typically measured at 28 days. To avoid cumbersome and time-consuming ponding or immersion tests for D_0 determination (AASHTO T 259[29] and ASTM C1556[30]), the current trend is to replace them with rapid chloride migration tests. In these tests, an electric field is established between the faces of a concrete specimen, of which one face is in contact with a NaCl solution. Due to this electric field, which greatly accelerates the process, Cl^- migrate from the anode to the cathode through the concrete sample. The coefficients of diffusion and of migration are directly related by the Nernst-Planck equation, given on page 26 of RILEM Report 40.[31] Two such tests are available: the NT Build 492 method,[32] which assumes conservatively that the coefficient of chloride migration obtained is equal to D_0; and the ASTM C1202 method,[33] which measures the electrical conductivity of the concrete. The latter determines the electrical charge Q (coulombs) passed through the concrete in 6 hours, from which D_0 can be estimated using the relation described in Olek et al.[34] and repeated herein as Eq. (10)

$$D_0\,(10^{-12}\,\text{m}^2/\text{s}) = 0.4 + 0.002Q \tag{10}$$

Still in the design phase, a comprehensive experimental program should be prepared, with guidelines on the mixture characteristics and durability performance required (for example, chloride diffusivity and permeability), to be run by a qualified test laboratory. Interested concrete producers should submit mixture designs together with samples of the raw materials they intend to use for the project. With the test results in hand, the concrete supplier can be selected based on the best technical-economical proposals received. We have designed and supervised such a comprehensive program for the marine Chacao Bridge project in Chile (100 years of service life). The mixtures selected for this project were subjected to a full characterization, including mechanical, rheological, thermal, and durability properties (including long-term tests), to refine the structural and durability design of the bridge.

Construction phase

During the construction phase, a strict concrete testing program should be established, involving not just mechanical testing but also tests that provide durability performance indicators in a matter of a few minutes (electrical resistivity and gas permeability), a few hours (chloride migration), a few days (water penetration), or several months (immersion or ponding Cl^- diffusion tests). Correlations between short-, medium-, and long-term tests, established during the full characterization of the mixtures (design phase), will allow verification that the production process is (or is not) under control.

Quality control is conducted primarily on cast specimens tested under laboratory conditions, the quality of which usually overestimates that achieved on site. Durability depends strongly on the penetrability of the concrete cover that protects the steel. Bad concreting practices have been identified as a major factor in poor durability performance of bridges,[1] as the detrimental effect of poor practices is much stronger on the cover concrete than on the core of concrete elements. Indeed, poor placement and compaction techniques plus a lack of curing (often resulting in greater microcracking) can lead to air permeabilities that are up to 50 times higher than values measured on specimens cast from the same batch and cured under laboratory conditions (refer to Torrent[35] and Ebensperger and Olivares[36]). Additional factors include the variability of conditions under which concrete is processed on site and high variability in cover depth.[37]

The only practical way to have a realistic assessment of the spatial variability of the penetrability and thickness of the concrete cover is by nondestructive (ND) measurements made on site. We have been involved in the development[38] and pioneer application[39] of a ND test method capable of measuring the coefficient of air permeability on site. This method has been a standard Swiss test method since 2003,[28] and the background for the updating of this standard is provided in Torrent et al.[40] There are also electromagnetic instruments capable of nondestructively assessing the cover depth quite accurately.[31] Additional enhancements are available through the use of ground-penetrating radar instruments.

During the construction phase, we believe it is essential to make regular and extensive investigation of the air permeability kT and thickness d of the concrete cover on the exposed surfaces of the bridge elements (Fig. 5). These measurements allow computing a value of service life T_i via the Exp-Ref method described in Torrent.[18]

Because of the quality control testing, vital information is obtained on the main variables affecting service life. This information includes the statistical distribution of such parameters as Cl^- diffusivity and air permeability of the concrete and cover depth. This information allows a more realistic probabilistic service-life assessment, both analytical and Exp-Ref, same as was done for the Hong Kong-Zhuhai-Macao link by Li and Torrent.[41] Figure 6 shows that the Exp-Ref method gives a less optimistic picture than the analytical, not so much in terms of the average service life, but of its scatter, probably due to the higher spatial variability of site quality than of laboratory specimens. This assessment may indicate the convenience of adjusting the concrete mixtures and/or the construction and concreting practices, as was done by one of the authors for the Buenos Aires Metro,[42] or even of applying remedial actions in clearly unacceptable cases. The CTK-ConcreLife Model[43] has been developed using the Exp-Ref method to determine the service life for different exposure conditions related to chloride- or carbonation-induced steel corrosion.

Fig. 5: Measuring *kT* and *d* on site

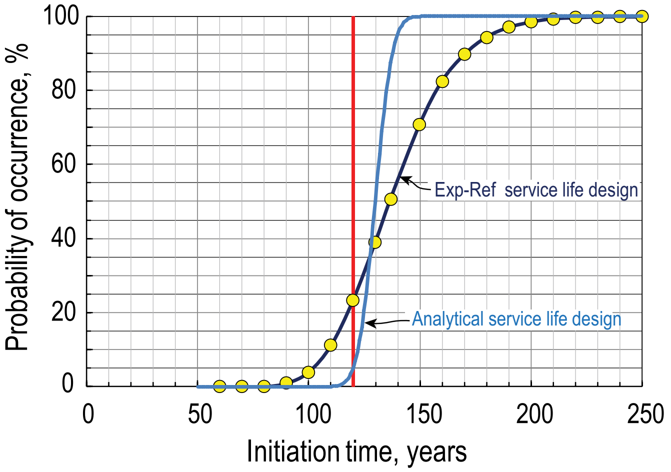


Fig. 6: Analytical (DuraCrete[11]) and Exp-Ref[18] service-life designs for Hong Kong-Zhuhai-Macao link[41]

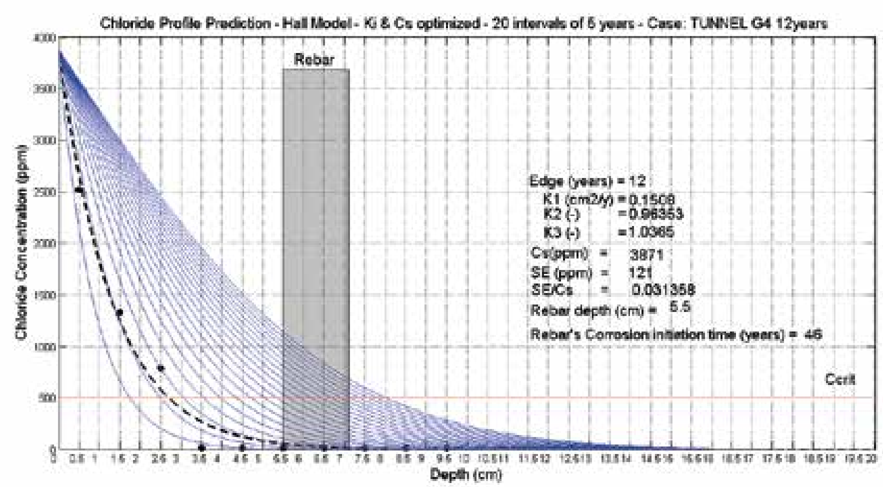


Fig. 7: Experimental Cl⁻ profile at 12 years (dots and broken line) and prediction at later ages with calibrated model (Note: 1 cm = 0.4 in.)

Commissioning and delivery phase

It should be noted that several generations of engineers will take care of the maintenance of any bridge, and each generation will need to have as much documentation as possible to carry out their duties efficiently. All the information collected during design and construction phases must be condensed in a comprehensive report and maintenance manual. An important part of it is the "birth certificate," which, according to *fib* Model Code 2010,[44] "should provide specific details on parameters important to the durability and service life of the structure concerned (for example, cover to reinforcement, concrete permeability, environmental conditions, and quality of workmanship achieved)." The weaker areas of the structure (that is, those showing higher site air-permeabilities and/or lower cover depths) must be identified in the report, focusing the attention of the maintenance engineers. Contour mappings of expected service lives, calculated using the Exp-Ref method, may facilitate maintenance planning.

Operational phase

The maintenance plan should include monitoring the long-term performance of the real structural elements and of mockups, made under the same conditions as the bridge elements and identically exposed. The monitoring systems may include embedded sensors, applying ND tests (electrochemical tests), or getting chloride profiles from cores drilled from mockups.

These measurements are essential for judging and improving the accuracy of the predictive models used during the design—they serve as calibration benchmarks. The measurements can also be used to evaluate the future performance of the real structure. For example, a finite element method program has been developed to solve Fick's second law differential equation,[45] and this allows a service-life prediction, based on chloride profiles obtained at a certain age, under several possible assumptions (that is, variable surface Cl^- concentration and diffusion coefficient variable with depth, Cl^- concentration, and time). Figure 7 shows an example of the service-life prediction for the Naxberg Tunnel in Switzerland, exposed to Cl^-, based on data from Bisschop et al.[46] The model is calibrated with 12-year experimental chloride profile data and is used to predict T_i equal to 46 years for a cover depth of 55 mm (2 in.) and critical chlorides concentration C_{cr} = 500 ppm (0.05% of concrete mass).

Conclusions

The traditional approach to durability design, predominantly prescriptive-based, has proved inadequate to guarantee the design/expected service life of bridges. This failure has had onerous consequences for the owners and taxpayers. The application of existing models to predict service life is difficult for civil engineers due to the need for defining critical elusive parameters, which are hard to determine even for experts. Some are called probabilistic, but, in fact, this term disguises ignorance as a form of probability. Moreover, most of the models rely on results of laboratory testing of cast specimens that do not reflect the true durability of the structure, influenced greatly by concreting practices. Achieving long service lives of bridges exposed to aggressive environments is only possible if all players in the concrete construction chain recognize the challenges and do their utmost to build a bridge of top quality and monitor its performance. This paradigm has to supersede the one that may have been behind the recent partial collapse of Polcevera Viaduct, Genoa, Italy; built in 1967: "At that time, it was believed that concrete would last forever."[47]

The comprehensive approach proposed in this article is an interactive process, involving measures to be taken before, during, and after the construction of the bridge. It applies robust solutions (not affected by subjectivity or manipulation), based on simple but rigorous principles easily grasped by civil engineers. It places strong emphasis on the quality of the end product, emphasizing equally the quality of the concrete mixture and of the jobsite practices. It is expected that this holistic concept will increase service life of concrete bridges, reducing costly early interventions and benefiting owners as well as users.

References

1. "Bridge Management in Europe (BRIME) – Deliverable D14," Contract No. RO-97-SC.2220, 2001, 221 pp.

2. Stipanovic Oslakovic, I.; Bjegovic, D.; and Mikuli, D., "Evaluation of Service Life Design Models on Concrete Structures Exposed to Marine Environment," *Materials and Structures*, V. 43, No. 10, Dec. 2010, pp. 1397-1412.

3. Brühwiler, E., "A Structural Engineer's View: More Attractive Value-Added Solutions for Concrete Structures," Holcim Workshop on VAS, Zürich, Switzerland, Dec. 2007, 44 slides.

4. *fib* Bulletin No. 44, "Concrete Structure Management: Guide to Ownership and Good Practice," *fib* (International Federation for Structural Concrete), Lausanne, Switzerland, 2008, 210 pp.

5. EN 1992-1-1, "Eurocode 2: Design of Concrete Structures – Part 1-1: General Rules and Rules for Buildings," CEN (European Committee for Standardization), Brussels, Belgium, 2004, 225 pp.

6. Torrent, R.J., "Bridge Durability Design after EN Standards: Present and Future," *Structure and Infrastructure Engineering*, Jan. 2018.

7. ISO 2394:2015, "General Principles on Reliability for Structures," ISO (International Organization for Standardization), 2015, 111 pp.

8. Li, Q.; Li, K.; Zhou, X.; Zhang, Q.; and Fan, Z., "Model-Based Durability Design of Concrete Structures in Hong Kong-Zhuhai-Macau Sea Link Project," *Structural Safety*, V. 53, Mar. 2015, pp. 1-12.

9. Tuutti, K., "Corrosion of Steel in Concrete," *Research Report No.4.82*, CBI (Swedish Cement and Concrete Research Institute), Stockholm, Sweden, 1982, 468 pp.

10. Nilsson, L.O., "Transport Processes in the Microstructure of Concrete and Their Relevance for Durability," Second International Conference on Microstructural-related Durability of Cementitious Composites, Amsterdam, The Netherlands, 2012, 8 pp.

11. Engelund, S.; Edvardsen, C.; and Mohr, L., "General Guidelines for Durability Design and Redesign: DuraCrete—Probabilistic Performance Based Durability Design of Concrete Structures," Contract BRPR-CT95-0132, Project BE95-1347, V. 15, Lyngby, Denmark, 2000, 109 pp.

12. *fib* Bulletin No. 34, "Model Code for Service Life Design," *fib*, Lausanne, Switzerland, 2006, 116 pp.

13. Life-365 Service Life Prediction Model™ and Computer Program for Predicting the Service Life and Life-Cycle Cost of Reinforced Concrete Exposed to Chlorides, Life-365 Consortium II, 2012, 80 pp.

14. *fib* Bulletin No. 76, "Benchmarking of Deemed-to-Satisfy Provisions in Standards," *fib*, Lausanne, Switzerland, 2015, 191 pp.

15. Torrent, R., "The Reference Approach to Service Life Design," *High Tech Concrete: Where Technology and Engineering Meet*, D.A. Hordijk and M. Luković, eds., Maastricht, The Netherlands, 2017, pp. 2224-2233.

16. Gulikers, J., "Considerations on the Reliability of Service Life Predictions Using a Probabilistic Approach," *Journal of Physics IV France*, V. 136, Nov. 2006, pp. 233-241.

17. Gulikers, J., "Practical Implications of Performance Specifications for Durability Design of Reinforced Concrete Structures," *Proceedings of the* fib *Workshop on Performance-Based Specifications for Concrete*, F. Dehn and H. Beushausen, eds., Leipzig, Germany, 2011, 10 pp.

18. Torrent, R., "Service Life Prediction: Theorecrete, Labcrete and Realcrete Approaches," SCTM3 Conference, Kyoto, Japan, 2013.

19. Torrent, R., "Robust, Engineers' Friendly Service Life Design," Keynote Lecture, 15th East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-15), Xi'an, China, Oct. 11-13, 2017.

20. Andrade, C., "Prediction of Service Life by Considering the Initiation and Propagation Periods," *Durability and Sustainability of Concrete Structures*, SP-305, M.A. Chiorino, L. Coppola, C. Mazzotti, R. Realfonzo, and P. Riva, eds., American Concrete Institute, Farmington Hills, MI, 2015, pp. 4.1-4.10.

21. Altmann, F., and Metcherine, V., "A Fuzzy Probabilistic Durability Concept for Strain-Hardening Cement-Based Composites," *8th RILEM International Symposium on Fiber Reinforced Concrete: Challenges and Opportunities (BENFIB 2012)*, J.A.O. Barros, ed., Guimarães, Portugal, 2012, pp. 660-670.

22. Altmann, F., and Metcherine, V., "Durability Design Strategies for New Cementitious Materials," *Cement and Concrete Research*, V. 54, Dec. 2013, pp. 114-125.

23. Taffese, W.Z., and Sistonen, E., "Significance of Chloride Penetration Controlling Parameters in Concrete: Ensemble Methods," *Construction and Building Materials*, V. 139, May 2017, pp. 9-23.

24. Otieno, M.; Beushausen, M.; and Alexander, M., "Prediction of Corrosion Rate in RC Structures – A Critical Review," RILEM Bookseries 5—*Modelling of Corroding Concrete Structures*, C. Andrade and G. Mancini, eds., RILEM, 2011, pp. 15-37.

25. Andrade, C.; D'Andrea, R.; Castillo, A.; and Castellote, M., "The Use of Electrical Resistivity as NDT Method for the Specification of the Durability of Reinforced Concrete," Non-Destructive Testing in Civil Engineering (NDTCE '09), Nantes, France, 2009, 6 pp.

26. Di Pace, G., and Torrent, R., "A Model for Corrosion Propagation Time and Real Application Example," RILEM 4th International Conference on Service Life Design for Infrastructures, Delft, The Netherlands, Aug. 27-30, 2018, pp. 592-600.

27. AASHTO T 358, "Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration," AASHTO, Washington, DC.

28. SIA 262/1:2013—"Betonbau – Ergänzende Festlegungen (Concrete Structures – Supplementary Specifications)," SIA (Swiss Society of Engineers and Architects), Zurich, Switzerland, 2013 (in German).

29. AASHTO T 259, "Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration," AASHTO, Washington, DC.

30. ASTM C1556, "Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion," ASTM International, West Conshohocken, PA, 2011, 7 pp.

31. RILEM Report 40: "Non-Destructive Evaluation of the Penetrability and Thickness of the Concrete Cover," *State-of-the-Art Report of RILEM Technical Committee 189-NEC*, R. Torrent and L. Fernandez Luco, eds., RILEM, 2007, 223 pp.

32. NT Build 492, "Concrete, Mortar and Cement-Based Repair Materials: Chloride Migration Coefficient from Non-Steady-State Migration Experiments," NORDTEST, Espoo, Finland, 1999.

33. ASTM C1202, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration," ASTM International, West Conshohocken, PA, 2017, 7 pp.

34. Olek, J.; Lu, A.; Feng, X.; and Magee, B., "Performance-Related Specifications for Concrete Bridge Superstructures, Vol. 2: High-Performance Concrete," *FHWA/IN/JTRP-2001/08*, Indiana Department of Transportation, Indianapolis, IN, 2002, 192 pp.

35. Torrent, R., "Gas-Permeability of High-Performance Concretes−Site and Laboratory Tests," *High-Performance Concrete: Performance and Quality of Concrete Structures*, SP-186, V.M. Malhotra, P. Helene, L.R. Prudencio Jr., and D.C.C. Dal Molin, eds., American Concrete Institute, Farmington Hills, MI, 1999, pp. 291-308.

36. Ebensperger, L., and Olivares, M., "Especificación y Control de Durabilidad del Hormigón in-Situ con Ensayos No-Destructivos: Metodología Integral de Estimación de Vida Útil con Mediciones de Permeabilidad al Aire y Espesor de Recubrimiento," 2nd International Bridges Congress, Santiago, Chile, Oct. 2017.

37. Neville, A., "Concrete Cover to Reinforcement—Or Cover Up?," *Concrete International*, V. 20, No. 11, Nov. 1998, pp. 25-29.

38. Torrent, R., "A Two-Chamber Vacuum Cell for Measuring the Coefficient of Permeability to Air of the Concrete Cover on Site," *Materials and Structures*, V. 25, No. 6, July 1992, pp. 358-365.

39. Torrent, R., and Ebensperger, L., "Study on Methods to Measure and Evaluate the Characteristics of the Cover Concrete on Site," *Report 506*, Swiss Federal Highways Office, Bern, Switzerland, 1993, 119 pp. (in German).

40. Torrent, R.; Denarié, E.; Jacobs, F.; Leemann, A.; and Teruzzi, T., "Specification and Site Control of the Permeability of the Cover Concrete: the Swiss Approach," *Materials and Corrosion*, V. 63, No. 12, Dec. 2012, pp. 1127-1133.

41. Li, K., and Torrent, R., "Analytical and Experimental Service Life Assessment of Hong Kong-Zhuhai-Macau Link," *The Eighth International Conference on Bridge Maintenance, Safety and Management (IABMAS 2016)*, Paper 427, Foz do Iguaçú, Brazil, 2016.

42. Di Pace, G.; Calo, D.; and Torrent, R., "Assessment of Concrete Permeability in Tunnels," *SACoMaTIS 2008 International RILEM Conference*, L. Binda, M. di Prisco, and R. Felicetti, eds., Varenna, Italy, V. 1, 2008, pp. 327-336.

43. CTK-ConcreLife Model Concrete Service Life Design and Control, Construtechnik Ltda, Santiago, Chile, 2018.

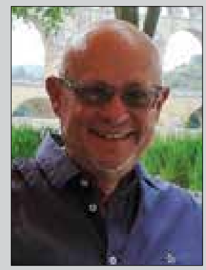
44. "Model Code 2010 – Final Draft, V. 1," *fib*, Lausanne, Switzerland, 2010, 350 pp.

45. Bueno, V., "Evaluación del Uso de Volúmenes Finitos en la Estimación del Tiempo de Iniciación de la Corrosión en Armaduras," Dissertation in chemical engineering, University of Zulia, Maracaibo, Venezuela, 2005.

46. Bisschop, J.; Schiegg, Y.; and Hunkeler, F., "Modelling the Corrosion Initiation of Reinforced Concrete Exposed to Deicing Salts," *Report 676*, Swiss Federal Highways Office, Bern, Switzerland, 2016, 99 pp.

47. Ceriolo, L., "'Damals glaubte man, Beton halte ewig,' Ein Gespräch mit dem Hochbauingenieur Michel Virlogeux, ('At that time, it was believed that concrete would last forever,' an interview with structural engineer Michel Virlogeux)," TEC21, N0. 7-8/2019, pp. 26-29 (in German).

Selected for reader interest by the editors.



Guillermo Di Pace is an International Concrete and Cement Consultant, involved in several emblematic large infrastructure projects. His area of expertise includes corrosion of steel in concrete. He was a Visiting Professor at the University of South Florida, the University of Texas at Austin, and University of California, Berkeley. He received his civil engineering degree from Universidad de Buenos Aires, Buenos Aires, Argentina, and his PhD from the Catholic University and Polytechnic Universities of Guayaquil, Ecuador.

Luis Ebensperger Morales is a consultant in concrete technology and construction, and Director of Construtechnik Consultants, Chile. His expertise includes durability, service-life design and control, and thermal cracking of mass concrete structures. A member of RILEM, ASTM, and IABSE, he received his civil engineering degree from Universidad Católica de Chile, Chile, and his PhD from Technical University of Munich, Munich, Germany.

ACI member **Roberto Torrent** is a consultant in cement, concrete, and concrete construction technology and Director of Materials Advanced Services Ltd., Buenos Aires, Argentina. His research focuses on durability testing and service-life design of concrete structures. He is also an Honorary Member of RILEM and a member of *fib* and SIA. He received his civil engineering degree from Universidad de Buenos Aires and his PhD from University of Leeds, Leeds, United Kingdom.

Verónica Bueno is a specialist in metal corrosion, particularly in corrosion of steel in reinforced concrete, and a Consultant at Materials Advanced Services, Switzerland. Her research focuses on durability and corrosion testing and modeling of chloride penetration into concrete. She is a member of RILEM. She received her chemical engineering degree from University of Zulia, Zulia, Venezuela, and her PhD from Politecnico di Milano, Milan, Italy.