CHALLENGES IN PREDICTING SERVICE-LIFE OF CONCRETE STRUCTURES EXPOSED TO CHLORIDES

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Abstract

Several service life models exist for prediction of time-to-corrosion of concrete structures exposed to chlorides. Some are more sophisticated than others in terms of addressing the various chloride transport mechanisms, while others assume transport by diffusion alone.

As well, the input variables have uncertainty associated with them such as boundary conditions, cover depths, diffusion coefficients, time-dependent changes, and rates of build up of chlorides at the surface.

But there is disagreement, for example, on how to measure time-dependent changes in diffusion properties, on how long it continues, as well as to what properties influence the time-dependent values. As well, imperfect curing can result in depth-dependent effects on diffusion properties. How experimental data is obtained as well as how models handle these properties has a dramatic impact on predicted service lives.

In addition, construction detailing, defects and construction practices can have a significant influence on actual penetration rates, but are rarely quantified or modeled. Very few models deal with the influence of cracks or the fact that concrete in the cover zone will almost certainly have a higher diffusion coefficient than the bulk concrete as the result of imperfect curing or compaction. While many models can account for variability in input properties, they will never be able to account for extremes in construction defects. Therefore, to ensure the reliability of service life predictions and to attain a concrete structure that achieves its predicted potential, designers and contractors need to work together to ensure proper detailing, minimize defects, and adopt adequate, yet achievable, curing procedures. As well, concrete structures are often exposed to other destructive elements in addition to chlorides (eg frost, ASR) and this adds another level of complexity.

This contribution will discuss some of these issues as well as the need to use performance-based specifications together with predictive models.

1. INTRODUCTION

One of the challenges to both builders and owners of concrete structures is the ability to predict the time to maintenance, or rehabilitation. To address this, predictive models have been developed, especially related to the time-to-corrosion of reinforced concrete exposed to marine or de-icer salts. Early chloride ingress models, based on Fick's second law of diffusion were overly simplistic, but since the early 1990's, models were improved to account for time-dependent changes in diffusion [1], the time to build up of surface chloride concentration, chloride binding and, in some cases, depth-dependent diffusion [2, 3]. However, diffusion is only one mechanism of ingress of fluids including aggressive ions such as chlorides. Other mechanisms including capillary absorption, permeability, and wick action can greatly accelerate ingress of chlorides, and some models have added terms to account for their effects. Taking a different approach, more fundamental multi-species models have been developed (such as Stadium [4, 5]) that use effective diffusion values for different ions and account for their interactions and the nature of the pore structure and transport processes.

In addition, many models are deterministic which, given the level of uncertainty in concrete composition, rebar placement, and transport properties, only give average predictions which are likely not realistic. Several models, such as DuraCrete [6], LIFE-365 Version 2 [7], and the draft ISO16240 [8, 9, 31] are probabilistic, or semi-probabilistic and typically input standard deviations as well as average values for each input value.

Models for prediction of other time to damage from other mechanisms of degradation, such as alkali-silica reaction and frost, have been developed, and these are useful for prediction of maintenance of existing structures. For reasons of space, these are not covered here. For new construction, these concerns can largely be avoided by better testing of aggregates or the use of appropriate mitigative measures in the case of ASR, and for frost by use of air-entrainment, frost-resistant aggregates and prevention of premature exposure to freezing during construction.

2. TRANSPORT MECHANISMS AND TESTS

However, diffusion is only one mechanism of ingress of fluids including aggressive ions such as chlorides. Capillary absorption into unsaturated near-surface zones can allow rapid ingress of dissolved ions in the so-called convection zone of the concrete cover. In some cases, pressure heads are also involved so permeability is an issue that will accelerate ingress. In the cases of tunnel liners, pipes, and slabs-on-grade, wick action from the air boundary can evaporate water from the near-surface pores, leaving behind a build up of precipitated chloride salts [10].

Limits on specific standard penetration resistance properties (eg. Bulk chloride diffusion in Nordtest NT Build 443 (ASTM C1556), water sorptivity in ASTM C1585), or vapour transport (ASTM E96) may be useful for pre-qualification of concrete mixtures, and such fluid penetration resistance test limits have been specified internationally in many high-profile, long-service life structures (such as the Confederation Bridge in Canada, the Tsing Ma Bridge in Hong Kong, and the Oresund Bridge between Denmark and Sweden, to name a few). Test values for relevant transport properties are needed as inputs to many of the service life models currently being used. For example, the original Version 1 of LIFE-365 life cycle cost model [11], widely used in North

America uses relatively simplistic inputs of 28-day bulk chloride diffusion values (Da) as well as a time dependent coefficient (m) established by measuring chloride diffusion values at different ages. To make it simpler to use, this model also contains a data base of default Da and m values for different types of concretes, so a user can simply input a concrete mixture.

However, before tests are adopted in specifications, they must not only be shown to be useful and reliable, they must also be standardized and include precision data based on interlaboratory evaluations, in order to develop confidence in the results and to be able to set realistic specification limits that take account of the test variability [12]. This has not always been the case, as several international bridge and tunnel projects adopted chloride diffusion limits based on an ad-hoc bulk diffusion method which was only published as a loose copy and apparently was not published in the proceedings of the conference where it was presented [13]. While that ad-hoc test may have been useful, it was applied on several international projects without going through a standardization process. Many tests have been proposed by various researchers, but only a few have been adopted in recognised standards.

There are limitations to practical use of rigorous permeability and diffusion-based testing beyond prequalification of concrete mixtures, and there is an important role for rapid index tests for chloride penetrability for quality assurance during construction. As a result, for acceptance, a rapid permeability-index test, such as ASTM C1202 or the Rapid Chloride Migration Test, Nordtest NT Build 492 (also AASHTO TP-64), may need to be adopted. Because it is relatively simple and rapid, the ASTM C1202 test has become widely used for this purpose. A similar approach but using other durability indicators for quality control purposes during construction has been proposed elsewhere [14].

One type of rapid index test that is currently receiving a lot of attention is electrical resistivity [15, 16]. While resistivity is also influenced by the conductivity of the pore fluid, for most concrete mixtures a measure of the bulk conductivity is sufficiently influenced by the porosity and continuity of the pore structure that it is useful for characterizing the fluid penetration resistance [17].

As an interim measure, in 2004 the Canadian CSA A23.1 concrete standard adopted the ASTM C1202 rapid chloride penetration index limits for prequalification of concrete mixtures to meet (a) C-1 exposure conditions (concrete exposed to freezing in a saturated condition with de-icer salts: 35 MPa, air-entrained, 0.40 w/cm max.) of 1500 coulombs at 56 days, and (b) C-XL exposure (like C-1 but where extended service life is required, 50MPa, air-entrained, 0.37 w/cm max.) of 1000 coulombs at 56 days. In 2009, CSA A23.1-09 [18] was modified to include a maximum single value limit as well as average value coulomb limit when this test is to be used for acceptance during construction (eg. For the C-1 exposure, a single test value is allowed to be up to 1750 coulombs as long as the average value remains below 1500 coulombs at 56 days). This allows for test variability and is similar to the statistical approach used for strength acceptance.

This should not be construed as ignoring other more rigorous test methods or use of service life modeling. Bulk diffusion and other transport property tests useful for service life model inputs can be specified during prequalification, but results should then be related to the index test values to be used for practical quality assurance during construction. The adoption of the ASTM

C1202 index test in the CSA standard also allows attention to be paid to fluid penetration resistance in a much wider number of construction specifications where service life modeling would not typically be included.

3. CHLORIDE SERVICE-LIFE MODELS

Since corrosion of reinforcement is the largest single cause of deterioration of reinforced concrete structures, most models have focused on this, and mainly due to chloride ingress. The earliest chloride ingress models assumed that diffusion is the only mechanism of chloride ingress. Numeric solutions to Fick's 2nd law were then used to predict time to critical chloride concentrations at the depth of cover [19]. Later models included time-dependent (m) [20] and depth-dependent [2, 3] diffusion coefficients and time-dependent build up of surface chlorides (Cs(t)) [21]. Some models use apparent or bulk diffusion coefficients (Da) based on total (acid soluble) chloride penetration profiles and make corrections for chloride binding while others have used effective diffusion coefficients (De).

One variable is whether acid soluble or water soluble chlorides are used in the calculations. It is far easier to obtain reliable measurements of total chloride content acid soluble chloride contents by dissolving samples in nitric acid. Unfortunately, this will include bound chlorides and not accounting for this has a significant impact on service life predictions [22]. While it is only the water-soluble chlorides that will act to depassivate the steel reinforcement, the methods for obtaining water-soluble chlorides are somewhat arbitrary and are difficult to define. Under some conditions, such as carbonation [23], almost all bound chlorides can be released. Another issue is that all materials contain some chlorides and there will be a background level of chloride throughout the concrete. For example, in Toronto and Chicago, the crushed limestone coarse aggregates typically contain significant chlorides, but unless the aggregate is crushed to powder, these chlorides are effectively insoluble inside the coarse particles. If the background chloride in concrete is effectively insoluble, then a correction should be made to subtract these background values when acid soluble chlorides are being determined; for models this typically has to be done anyway as diffusion rates are impacted by the chloride gradient.

Some models have included terms accounting for permeation and absorption while a few can account for conditions of less than saturated pore systems.

Where boundary conditions warrant, it is important to consider other mechanisms than diffusion such as absorption into unsaturated surfaces is a rapid process with time spans in the order of minutes or hours while diffusion ingress is measured in time spans of years. As well, evaporation due to wick action [10, 24, 25] can act to concentrate deleterious levels of precipitated salts in pores below the evaporative surfaces (although that mechanism is less active with concretes having a discontinuous capillary pore structure. ie. $w/c < 0.45$). Wick action is also important for damage due to physical sulfate and other types of salt attack.

It is known that there is variability in all properties of reinforced concrete, so models, such as DuraCrete [6], and Version 2 of LIFE-365 [7] have taken probabilistic approaches using inputs of both average and standard deviation for each input value. A typical example used in LIFE-365 is shown in Table 1. More simplistic stochastic models only give average time predictions, well beyond the decision point required for structural repair.

Once the transport processes are modeled, another area that needs further attention is the critical chloride threshold for initiation of corrosion. Published values vary by more than an order of magnitude. More recent work has recognized the usefulness of the chloride to hydroxyl ion ratio as being more meaningful (taking into account different cement contents and types of cementing materials), but even these values vary by more than an order of magnitude [an extensive review is given in [26]. This subject is the study of the RILEM committee CTC formed in 2009.

A few sophisticated models, such as STADIUM [4, 5], use a more materials science-based approach and account for multi-species ionic movements (since chlorides are not the only potentially damaging ions in pore solutions and to maintain charge balance, different cations also impact rate of diffusion.

The FIB Task Group 5.6 Model Code on Service-Life Design of Concrete Structures (Chaired by P. Schiessl) FIB Bulletin 34, 2006 [8] (also see [9]) is currently being drafted by ISO TC71/SC3/WG4 into ISO WD/16204 [27] (not yet completed) and when issued will be the most advanced approach for achieving service life of new structures subject to reinforcement corrosion and for prediction of residual service life of existing structures.

Table 1: Example Average Values and Standard Deviations for Inputs in LIFE-365 [7]

Variable	Units	Average Value	Standard Deviation	Coefficient of Variation, %
Chloride Bulk Diffusion, Da at 28 days	m^2/s	8.87×10^{-12}	2.22×10^{-12}	25
Time-dependent coefficient, m	-	0.20	0.05	25
Max. Surface Chlorides, Cs	% Cl	1.0	0.30	30
Background chlorides, Ct	% Cl	0.05	0.01	20
Cover Depth	mm	60	5	8

4. MATERIALS VARIABILITY AND CONSTRUCTION DEFECTS

Construction detailing and practices as well as defects can have a significant influence on actual penetration rates of aggressive ions and fluids, but are rarely quantified or modeled. Very few models currently deal with the influence of cracks or the fact that concrete in the cover zone will almost certainly have a higher diffusion coefficient than the bulk concrete as the result of imperfect curing or compaction. An example of variation in properties of similar bridge structures is given by Tikalsky et al [28].

It is unpredictable areas of poorly compacted, poorly cured, or cracked concrete with less than the design depth of cover which will severely shorten the predicted the time to corrosion, regardless of what model is used. Therefore, one of the most effective ways to obtain the model-predicted service life of a structure is to address these site issues prior to and during construction. Pre-construction and pre-pour meetings are an effective approach to make sure that the contractor and sub-contractors understand the issues and have the required labour, materials and equipment on site to ensure best practices are followed.

As well, inspection of formwork placement and reinforcement cover depths prior to each placement of concrete can be used to correct areas of low cover deficiencies. In probability-based models, a common approach is to assume an average and range of properties such as concrete cover depth. But by correcting cover deficiencies prior to concrete placement, the variability in predicted service life can be significantly reduced. For example at the newest airport parking garage in Toronto, this process led to a standard deviation in cover depth of only 3 mm over a huge deck area (Figure 1) (the specified cover was 40 mm), well below the CSA A23.1 allowable variation of 10 mm.

The ability to adequately place, vibrate and compact concrete in areas of congested reinforcement also needs to be established, preferably with test of mock ups but also by letting the contractor select the required workability so that the concrete supplier can design the mixtures for the correct workability rather than the one specified.

To address compaction and curing, testing of cores extracted from the in-place concrete can be used since that most closely measures the performance of the final structural element (to avoid drilling cores from precast elements, extra element units can be cast along with the structural elements for this purpose).

A number of researchers have studied the effects of cracks on permeability and chloride ingress [29, 30], but little of this has been integrated into predictive models. Part of this stems from the inability to predict the number and widths of non-structural cracks that may occur in practice.

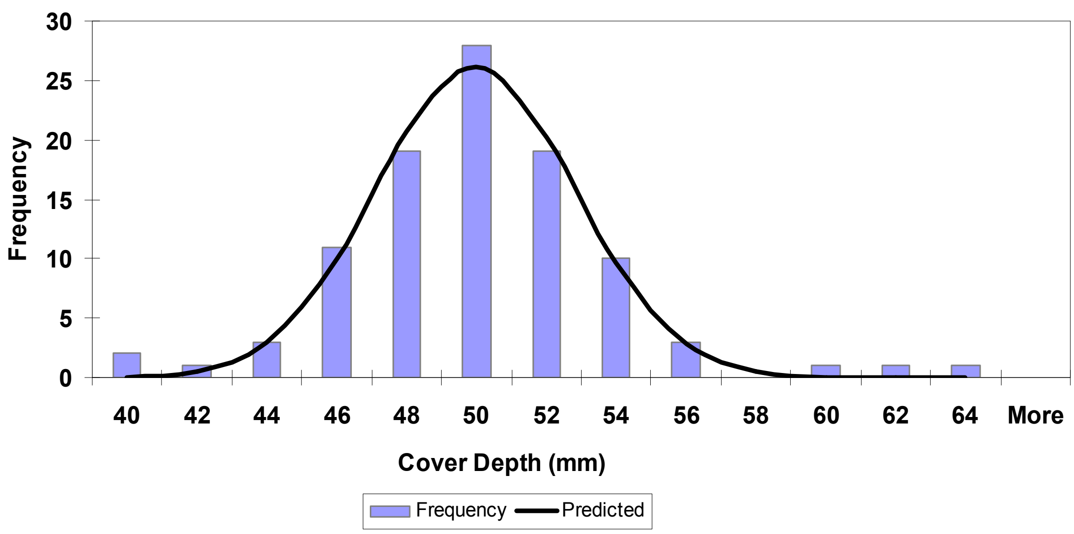

Figure 1: Range of Measured In-Place Concrete Covers on Toronto Airport Parking Garage

As stated earlier, probabilistic predictive models can account for variability in concrete properties and uncertainties in cover depth but will never be able to account for extreme variations caused by construction defects. Therefore, to minimize these extremes (the outliers beyond 90% confidence limits), in addition to use of sophisticated models, designers and design professionals representing the owner need to be proactive in ensuring that acceptable performance is achieved during construction. That is why effective performance specifications are an important component in achieving convergence between predicted and actual durability performance.

For example, to minimize the need to minimize the number and impact of unintended cracks (ie non-structural cracks), during prequalification, concrete mixtures need to be evaluated for their cracking potential in addition to fluid penetration resistance. In general, reduction of paste contents by optimized aggregate gradations (along with use of the maximum practical coarse aggregate size) and use of water-reducing admixtures is necessary to reduce the potential for both thermal and shrinkage cracking. From a testing point of view, restrained ring shrinkage cracking tests for concrete have been standardized (ASTM C1581) as have tests for autogenous shrinkage of different paste compositions (ASTM C1698). Heat of hydration can also be evaluated (ASTM C1702 for cements and C1679 for cement-admixture combinations).

In addition, designers can reduce crack widths by improved detailing of reinforcement to for example incorporate smaller diameter bars or by use of prestressing.

In summary, to ensure the reliability of service life predictions and to attain a concrete structure that achieves its predicted service life without excessive maintenance or repair, designers, contractors and concrete suppliers need to work together to ensure proper detailing, minimize defects, and adopt adequate, yet achievable, curing procedures. This is best achieved through use of performance specifications.

5. PERFORMANCE SPECIFICATIONS

Traditional standards and specifications for concrete have largely been prescriptive, (or prescription–based), and can sometimes prevent attaining the expected performance of concretes by requiring limitations on, for example, minimum cement contents and SCM replacement levels. There is a current trend away from prescriptive towards performance specifications in North America and around the world. Prescriptive specifications are the norm and have been developed by local experience but are often conservative. They also often inhibit innovation since new materials and methods do not fit into the prescriptive mould. However, adoption of true performance-based specifications presupposes that we have a clear understanding of all the performance issues that can affect concrete. It also assumes that there are appropriate performance test methods in place to evaluate all of the performance issues for: concrete materials, fresh concrete, hardened concrete, and durability. It also assumes that performance can either be measured in time to affect the outcome, and/or can be used to pre-qualify concrete mixtures. Most parties to construction are familiar with testing for fresh properties and strength of concrete, but the biggest challenges in this regard relate to requirements for durability [31].

While there are many types of aggressive exposures which might require a multitude of durability tests, the common element is that most aggressive exposures require that the permeability or fluid penetration resistance of concrete be minimized. Therefore adoption of one or more tests for penetration resistance is fundamental to ensuring durable concrete.

The Canadian concrete standard CSA A23.1-09 [18] outlines the requirements and responsibilities for use in performance-based concrete specifications. The responsibilities of the various parties need to be clearly defined with a performance specification. This has been attempted in CSA A23.1-09, the essence of which is shown in Table 2. In addition, an Annex to that standard was added to that standard to explain each of these issues in more detail.

The onus for meeting performance clearly rests with the producer up to point of placement. Since in-place performance is also affected by the contractor's placement methods, the producer must work with the contractor to ensure the owner's performance requirements are achieved: eg. the contractor (not the owner/specifier) should set the target slump to allow for proper placement and compaction for the situation, and the producer needs to design and provide this without reducing the intended performance of the hardened concrete.

Performance tests are performed at various stages before or during construction:

1. Pre-qualification: To provide a mixture that when placed under defined conditions can meet the specification.

2. Quality Control: To document that a) materials supplied meet spec. b) the concrete supplied is equivalent to that which was pre-qualified (sometimes called identity testing), c) pre-qualified placing practices are being followed. (ie. test at each change of ownership, such as point of discharge from truck)

3. In-Place Testing: Using NDT and/or tests on cores extracted from the structure to ensure that the concrete supplied and the placement methods meet owner-defined performance levels as required in several highway agency End Result Specifications (ERS).

Concrete producers and contractors are often just interested in prequalification and quality control testing. However, owners are interested in performance of the hardened concrete in the structure, a number of highway agencies in North America have adopted or are currently considering the use of end-result specifications (ERS) where contractors are paid based on consistently meeting specified performance requirements using in-place testing of the structure. A number of these agencies have developed ERS with defined financial penalties for failure to meet the in-place requirements, some of which exceed the cost of the concrete and if performance is lower than a certain threshold, removal is required. However, for many non-public works projects, it is likely that a combination of prequalification and as-delivered quality control tests will be the default option. Regardless, of the type of performance specification adopted, the acceptance criteria and the responsibilities of the various parties in case of failure need to be clearly defined.

While various performance tests can be used for pre-qualification, quality assurance, or in-place testing, there are far more issues which have to be addressed to obtain desired performance in aggressive environments. This type of information is detailed in an Annex to CSA A23.1-09.

A few of these points are listed below:

- Require all contract bidders to attend a pre-bid meeting to hear about special requirements—so they can't complain afterwards that they missed some of the performance requirements.

Table 2: Prescriptive vs Performance Specification Responsibilities (after CSA A23.1-09)

Alternative	The owner shall specify	The contractor shall	The supplier shall
(1) Performance: When the owner requires the concrete supplier to assume responsibility for performance of the concrete as delivered and the contractor to assume responsibility for the concrete in place.	(a) required structural criteria including strength at age; (b) required durability criteria including class of exposure; (c) additional criteria for durability, volume stability, architectural requirements, sustainability, and any additional owner performance, pre-qualification or verification criteria; (d) quality management requirements (e) whether the concrete supplier shall meet certification requirements of concrete industry certification programs; and (f) any other properties they may be required to meet the owner's performance requirements.	(a) work with the supplier to establish the concrete mix properties to meet performance criteria for plastic and hardened concrete, considering the contractor's criteria for construction and placement and the owner's performance criteria; (b) submit documentation demonstrating the owner's pre performance requirements have been met; and (c) prepare and implement a quality control plan to ensure that the owner's performance criteria will be met and submit documentation demonstrating the owner's performance requirements have been met.	(a) certify that the plant, equipment, and all materials to be used in the concrete comply with the requirements of this Standard; (b) certify that the mix design satisfies the requirements of this Standard; (c) certify that production and delivery of concrete will meet the requirements of this Standard; (d) certify that the concrete complies with the performance criteria specified; (e) prepare and implement a quality control plan to ensure that the owner's and contractor's performance requirements will be met if required; (f) provide documentation verifying that the concrete supplier meets industry certification requirements, if specified; and (g) at the request of the owner, submit documentation to the satisfaction of the owner demonstrating that the proposed mix design will achieve the required strength, durability, and performance requirements.
(2) Prescription: When the owner assumes responsibility for the concrete.	(a) mix proportions, including the quantities of any or all materials (admixtures, aggregates, cementing materials, and water) by mass per cubic metre of concrete; (b) the range of air content; (c) the slump range; (d) use of a concrete quality plan, if required; and (e) other requirements.	(a) plan the construction methods based on the owner's mix proportions and parameters; (b) obtain approval from the owner for any deviation from the specified mix design or parameters; and (c) identify to the owner any anticipated problems or deficiencies with the mix parameters related to construction.	(a) provide verification that the plant, equipment, and all materials to be used in the concrete comply with the requirements of this Standard; (b) demonstrate that the concrete complies with the prescriptive criteria as supplied by the owner; and (c) identify to the contractor any anticipated problems or deficiencies with the mix parameters related to construction.

- Make contractors, including subcontractors, detail in their bid how they intend to meet the special requirements part of the bid submittal. eg. Concrete placement methods, protection, curing, hot/cold weather provisions.
- Do not accept low-price bids that are not responsive to the special requirements.
- Once work has commenced, require pre-pour meetings for important placements: The contractors, the suppliers, the subcontractors, including the finishers need to be aware of what needs to be done to ensure that the concrete can be delivered, placed, compacted, protected, finished, and cured to achieve the performance objectives. Even the person who will be fog misting , or applying other protective measures needs to be there to understand why it is important.

Achieving the owner's performance requirements requires more cooperation between the concrete suppliers, the contractors, and concrete finishers than often exists in typical practice.

6. COMBINED FORMS OF DEGRADATION

In addition to designing for the primary form of aggressive exposure, concrete elements must be designed to resist all other forms of expected aggressive exposure. Often several destructive mechanisms can act together, or damage due to one mechanism can allow moisture ingress that enables another form of damage (eg. frost damage to concrete previously cracked by ASR). The Eurocode 2 design code makes use of EN206.1 exposure classifications to determine limits on required concrete properties. In the 2008 version of the ACI 318 Code [32], exposure classifications were adopted and the engineer-of-record must now define all the potential exposures for each concrete element in a structure including listing them as 'zero' exposure classes if they are not in any form of aggressive exposure. For decades the CSA A23.1 standard [18] has made use of a table defining durability exposure conditions (adapted here as Table 3). For a particular exposure condition, requirements are then placed on concretes for maximum w/cm, minimum strength, limits on type of cementitious materials (for sulfate exposures only, and there is also a performance test option), maximum chloride penetration resistance, and minimum periods and types of curing may apply. In addition, the CSA A23 concrete standard has adopted effective test methods and contains extensive sections on practical ways to avoid damage due to alkali-aggregate reactions, based on performance or by an experientially driven risk minimization model (similar approaches are now being considered by RILEM committee TC219-ACS and by ASTM Committee C09).

7. PREVENTATIVE MAINTENANCE

Often after structures are built, little or no maintenance is performed to address parts that wear out. Expansion joints, drains, seals, external membranes and sealers can degrade long before the structure and need to be cleaned, replaced or reapplied at regular intervals to prevent premature deterioration of the structure (similar to preventative maintenance on a car). Sometimes, there is an awareness of this need by both the owner and design professional but limited maintenance budgets do not allow for it, but in other cases the owner can be blissfully ignorant of the potential problems that may result from lack of maintenance. It has been suggested that structures should come with an owner's manual, similar to that provided with cars and this is the approach taken in the current draft of the *fib* Model Code 2010 [27] (based on [8]) where as-built documentation (also called birth certificate documentation) by inspection and measurement of in-place properties is required and a service-life file is to be developed after maintenance activities. Service life models are not able to address these sorts of issues.

8. PREDICTING RESIDUAL LIFE OF EXISTING STRUCTURES

The prediction of residual service life of existing structures adds additional challenges as often either the required fluid transport properties at time of construction were not measured or concrete data are no longer available. In such cases, one approach is to obtain current chloride diffusion data and back-calculate the early-age values needed for current model inputs using time-dependent coefficients in reverse, eg as done in [33]. Alternately, the currently obtained chloride penetration profiles can be used in combination with current diffusion values as models inputs, but residual time-dependent changes may not be as large as initially used values and may need to be modified. In structures where an initial service life prediction was made, long-term performance data can be used to improve predictions of remaining service life (or time to corrosion) and the remaining time period to be extrapolated will be much smaller. While this is only a cursory discussion of this topic, the proposed draft ISO 16204 Model Code will apparently be applicable to prediction of residual service life of existing structures [8, 27].

9. SUMMARY

There have been impressive achievements in the development of predictive service-life models in the last 20 years. This is especially true in the area of time to onset of reinforcement corrosion where models have developed beyond application of very simplistic Fick's 2nd Law methods to much more sophisticated multi-mechanistic, time-dependent, probabilistic transport models. Models are on the verge of being adopted in some model building codes, and that will lead to further developments as issues related to their use are addressed. Unfortunately, North American design Codes are along way from that at the current time. Test methods have been developed to provide input values, but often these tests are time-consuming and suffer from high levels of variability. Faster, more reliable test methods will provide better predictions and will be better suited for quality assurance purposes during construction. In the mean time, rapid index tests will likely be used for that purpose.

Performance-based models also require performance-based specifications and currently there are few truly performance-based specifications. With the rapid pace of developments in this area, the next few decades will almost certainly lead to very exciting changes to the way supplied concrete is documented and construction processes are followed in order to meet the new model-driven durability performance requirements of building codes and specifications.

Table 3: Exposure Classes adapted from CSA A23.1-09

Definitions of C, F, N, A, and S classes of exposure (1)	
C-XL	Structurally reinforced concrete exposed to chlorides or other severe environments with or without freezing and thawing conditions, with higher durability performance expectations than the C-1, A-1, or S-1 classes.
C-1	Structurally reinforced concrete exposed to chlorides with or without freezing and thawing conditions. Examples: bridge decks, parking decks and ramps, portions of marine structures located within the tidal and splash zones, concrete exposed to seawater spray, and salt water pools.
C-2	Non-structurally reinforced concrete exposed to chlorides and freezing and thawing. Examples: garage floors, porches, steps, pavements, sidewalks, curbs, and gutters.
C-3	Continuously submerged concrete exposed to chlorides but not to freezing and thawing. Examples: underwater portions of marine structures.
C-4	Non-structurally reinforced concrete exposed to chlorides but not to freezing and thawing. Examples: underground parking slabs on grade.
F-1	Concrete exposed to freezing and thawing in a saturated condition but not to chlorides. Examples: pool decks, patios, tennis courts, freshwater pools, and freshwater control structures.
F-2	Concrete in an unsaturated condition exposed to freezing and thawing but not to chlorides. Examples: exterior walls and columns.
N	Concrete not exposed to chlorides nor to freezing and thawing. Examples: footings and interior slabs, walls, and columns.
A-1	Structurally reinforced concrete exposed to severe manure and/or silage gases, with or without freeze-thaw exposure. Concrete exposed to the vapour above municipal sewage or industrial effluent, where hydrogen sulphide gas may be generated. Examples: reinforced beams, slabs, and columns over manure pits and silos, canals, and pig slats; and access holes, enclosed chambers, and pipes that are partially filled with effluents.
A-2	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure. Examples: reinforced walls in exterior manure tanks, silos, and feed bunkers, and exterior slabs.
A-3	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure in a continuously submerged condition. Concrete continuously submerged in municipal or industrial effluents. Examples: interior gutter walls, beams, slabs, and columns; sewage pipes that are continuously full (e.g., forcemains); and submerged portions of sewage treatment structures.
A-4	Non-structurally reinforced concrete exposed to moderate manure and/or silage gases and liquids, without freeze-thaw exposure. Examples: interior slabs on grade.
S-1	Concrete subjected to very severe sulfate exposures (2).
S-2	Concrete subjected to severe sulfate exposure (2).
S-3	Concrete subjected to moderate sulfate exposure (2).

Notes: (1) “C” classes pertain to chloride exposure; “F” classes pertain to freezing and thawing exposure without chlorides; “N” class is exposed to neither chlorides nor freezing and thawing;
(2) another table then defines sulfate concentration limits for each sub-category

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