Performance Based Specifications for Durability of Concrete Structures – Opportunities and Challenges

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ABSTRACT: Despite numerous reported cases of failure of concrete structures the durability of concrete is still compromised due to lack of a clear strategy for ensuring its performance in the exposure environment. This is someway addressed in EN 206-1: 2000, by specifying 6 different exposure classes with 23 different sub-classes in total. However, the approach to ensure durability in these service environments is still prescriptive or "deemed-to-satisfy" manner. Therefore, a performance-based strategy to specify concrete and a set of performance tests to assess its performance in the service environment are essential so that reliable prediction of its service life could be made. Such an approach is likely to contribute to improving the sustainability of concrete infrastructure, by minimising premature replacement of structures and allocating resources in a timely and planned manner. This strategy is introduced in this paper and suggestions for making further progress are made.

Keywords: Transport mechanisms, air permeability, sorptivity, diffusivity, carbonation, chloride ingress, electrical resistivity, sensor systems, structural health monitoring, durability assessment, service life prediction, performance-based specification

1 INTRODUCTION

Reinforced concrete as a composite construction material is distinguished for its very long service life in comparison to other building materials. However, in reality when these concrete structures are subjected to extreme environmental conditions, such as those experienced in marine environments, they tend to deteriorate rather in an alarming rate. The most commonly reported deterioration mechanisms affecting concrete durability are: (a) corrosion of reinforcement (account for more than 50% of the reported cases) as a result of carbonation, chloride ingress and leaching, (b) freeze-thaw deterioration, (c) crystallisation of salts in pores, (d) sulphate attack, (e) acid attack, (f) alkali attack, (g) alkali-silica reaction, (h) cracking in both the pre-hardening and hardened states, (i) fire damage and (j) abrasion (British Cement Association 1987). The movement of aggressive gases and/or liquids from the surrounding environment into the near surface zone of the concrete (Figure 1), followed by physical and/or chemical changes in their internal structure is primarily responsible for the premature deterioration of reinforced concrete structures by one or a combination of the above mechanisms (Figure 2) (Basheer & Nolan 2001; Basheer *et al.* 1994). The movement of these aggressive substances occurs due to differentials in humidity, ionic concentration, pressure and temperature within the microstructure of concrete.

The damage caused by these deteriorations is of either physical or chemical in nature or even both. The chemical deterioration can be caused by external attack that mainly occurs through the action of aggressive ions, such as chlorides, sulphates and other salts, originating from sub surface soil and sea water, penetration of carbon dioxide from atmosphere resulting in the carbonation of concrete and the action of acid rain. The physical deterioration effects can arise due to changes in thermal expansion of water in concrete as a result of the 'freeze-thaw' process, stresses in pore structure caused by crystallisation of salts and fatigue caused by thermal stresses. Cracking resulting from the deterioration processes in concrete is more likely to result in non-serviceability of the structure and if remained unattended can eventually lead to its collapse. The current situation in most developed countries is that repair and rehabilitation costs of structures far exceed the total budget for capital development programmes. Therefore, an important consideration for the design and construction of sustainable concrete structures is to ensure their durability in the service environment. The longer a structure lasts, less money is used to repair and replace the structure and hence more resources become available for capital projects and to contribute to developmental activities of a country. That is, designing and constructing durable concrete structures not only contributes to social sustainability, but also to the economic sustainability.

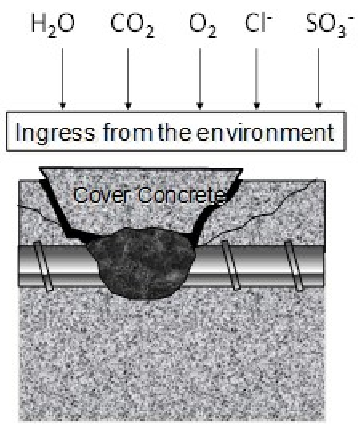


Figure 1. Environmental penetrations into concrete and relevance of assessing quality of the concrete cover

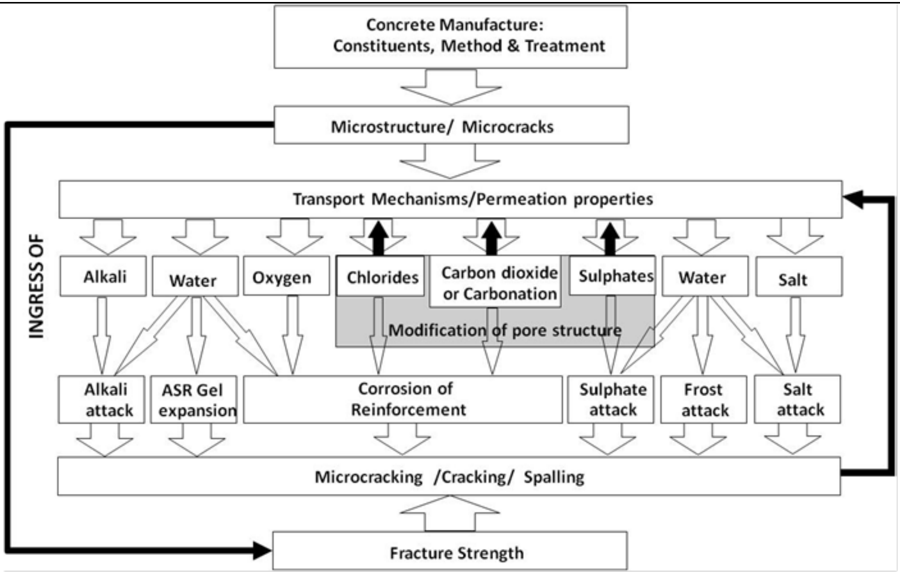


Figure 2. Interactions between concrete microstructure, permeation properties and deterioration due to exposure to service environments (Basheer *et al.* 1994)

This paper examines the concept of performance-based specifications for ensuring the durability of structures and introduces a strategy for performance testing using both in situ test techniques and sensors embedded in concrete. The data thus obtained could be used in service life models for predicting the likely durability of the concrete.

2 DURABILITY SPECIFICATIONS IN CURRENT EUROPEAN AND BRITISH STANDARDS

In December 2003, EN206-1:2000 (BSI 2003) was launched to supersede the previous standards to specify concrete. Concrete designers, producers and users alike now have the opportunity to use a wide range of cement and aggregate types for concretes in various exposure classes. The new Code applies to *in situ* and precast concrete structures and specifies constituent materials of concrete, properties of fresh and hardened concrete and verification of these properties using appropriate tests (Annex J of the Standard). EN 206-1: 2000 now specifies concrete based on its performance against specific deterioration mechanisms, as discussed below. In addition, limitations for concrete composition, specification of concrete, delivery of fresh concrete, production control procedures and conformity criteria and evaluation of conformity are also included.

As per EN 206-1: 2000 there are 23 exposure classes in total, spilt into six groups covering:

(i) No risk of corrosion
(ii) Carbonation-induced corrosion
(iii) Chloride-induced corrosion resulting primarily from de-icing salts
(iv) Chloride-induced corrosion resulting from seawater exposure
(v) Freeze-thaw attack
(vi) Chemical attack

These exposure classes are further sub-divided as shown in Table 1. The exposure classes to be selected depend on the provisions valid in the place of use of the concrete. Furthermore, the concrete may be subject to more than one of the actions described in Table 1 and the environmental conditions to which it is subjected may thus need to be expressed as a combination of the exposure classes.

In EN 206-1: 2000 the durability of concrete relies on prescriptive specification of minimum grade, minimum binder content and maximum water-binder ratio for defined environmental classes in Table 1. However, in the development of this standard, it has not proven possible within the European Committee for Standardisation (CEN) to agree common values for the specification parameters to cover the wide range of climates and cements in use in the EU Member States. Therefore, the Standard includes indicative values, and it is left to individual Member States to specify national values where they require them to differ from the indicated values. The Standard expects designers to define their required characteristics in terms of minimum strength class, maximum water-binder ratio and minimum binder content. Consistence specification is left to the judgment of the contractor/user so long as the above parameters are not compromised. Therefore, as a complementary to EN206-1: 2000, and to ease the turnover to EN206-1 and contain UK provisions on materials, methods of testing and procedures which are outside the scope of EN206-1: 2000 but within UK national experience, BS 8500 (BSI 2006) was produced. Furthermore, whilst BS EN 206-1 gives guidance for CEM I concrete, the more comprehensive BS 8500 includes additional cements CEM II, III and IV (BSI 2000) and provides a flexible trade-off between cover depth and concrete quality.

Table 1. Exposure classes from EN 206-1: 2000

Class	Environment	Examples
1. No risk of corrosion or attack		
X0	Concrete with no embedded metal (except where there is freeze/thaw, abrasion or chemical attack) For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity.
2. Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity, External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3. Corrosion induced by chlorides other than from seawater		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools, Concrete exposed to industrial water containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements, Car park slabs
4. Corrosion induced by chlorides from seawater		
XS1	Exposed to airborne salt but not in direct contact with seawater	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spry zones	Parts of marine structures
5. Freeze/thaw attack with or without de-icing agents		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agent	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation, with de-icing agent or seawater	Road and bridge decks exposed to direct spray containing de-icing agents and freezing. Splash zones of marine structures exposed to freezing.
6. Chemical attack		
Where concrete is exposed to chemical attack from natural soils and ground water is given in Table 2, the exposure shall be classified as given below. The classification of seawater depends on the geographical location; therefore the classification is valid in the place of use of the concrete.		
XA1	Slightly aggressive chemical environment according to Table 2	
XA2	Moderately aggressive chemical environment according to Table 2	
XA3	Highly aggressive chemical environment according to Table 2	

Table 2. Limiting values for exposure classes for chemical attack from natural soil and groundwater

(i) The aggressive chemical environments classified below are based on natural soil and ground water at water/soil temperatures between 5ºC and 25ºC and a water velocity sufficiently slow to approximate to static conditions. (ii) The most onerous value for any single chemical characteristics determines the class. (iii) Where two or more aggressive characteristics lead to the same class, the environment shall be classified into the next higher class; unless a special study for this specific case proves that it is not necessary.				
Chemical characteristic	Reference test method	XA1	XA2	XA3
Ground water				
SO_4^{2-} mg/l	BS EN 196-2	≥ 200 and ≤ 600	> 600 and ≤ 3000	> 3000 and ≤ 6000
pH	ISO 4316	≤ 6.5 and ≥ 5.5	≤ 5.5 and ≥ 4.5	< 4.5 and ≥ 4.0
CO_2 mg/l aggressive	BS EN 13577:1999	≥ 15 and ≤ 40	> 40 and ≤ 100	> 100 up to saturation
NH_4^+ mg/l	ISO 7150-1 or ISO 7150-2	≥ 15 and ≤ 30	> 30 and ≤ 60	> 60 and ≤ 100
Mg_2^+ mg/l	ISO 7980	≥ 300 and ≤ 1000	> 1000 and ≤ 3000	> 3000 up to saturation
Soil				
SO_4^{2-} mg/kg[a] total	BS EN 196-2[b]	≥ 2000 and ≤ 3000[c]	> 3000[c] and ≤ 12000	> 12000 and ≤ 24000
Acidity ml/kg	DIN 4030-2	> 200 Bauman Gully	Not encountered in practice	
[a] Clay soils with a permanently below 10-5 m/s may be moved into lower class. [b] The test method prescribes the extraction of SO_4^{2-} by hydrochloric acid; alternatively, water extraction may be used, if experience is available in the place of use of the concrete. [c] The 3000 mg/kg limit shall be reduced to 2000 mg/kg, where there is a risk of accumulation of sulphate ions in the concrete due to drying and wetting cycles or capillary suction.				

The British Cement Association and Building Research Establishment have reviewed the UK literature and, where required, the international literature to establish the minimum specifications for concrete necessary to achieve durable concrete in the exposure conditions defined by EN 206-1 for the broader range of binders used in the UK (Hobbs 1998). These were obtained for 50 and 100 years of service lives. Typical minimum requirements for concrete to resist corrosion induced by carbonation and chlorides from sea water, when the minimum cover to reinforcement is 30mm, are summarised in Tables 3 and 4. Similar tables are available for other forms of exposure. The minimum requirements for concrete to resist different exposure regimes summarised in this document could be used as a guide whilst designing concrete mixes for the different exposure classes in EN 206-1.

Table 3. Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by carbonation for an intended working life of at least 50 years

Exposure class	Dry XC1	Wet, rarely dry XC2	Moderate humidity or cycle, wet/dry XC3/XC4		
Minimum strength class	C20/25	C20/25	C30/37	C35/45	C35/45
Maximum W/C	0.65	0.65	0.60	0.35	0.45
Minimum cover to reinforcement (mm)	15	30	30	30	30
Cements*	All		I, I/SR, II/A-S, II/B-S, II/A-D, II/A-V, II/B-V, IV/A[1]	IV-B[1]	III/A[2] III/B[2]
For combinations Slag (%) Fly ash (%)	 ≤ 80 ≤ 55	 ≤ 80 ≤ 55	 ≤ 35 ≤ 35	 - 36-55	 36-80 -

[1] Siliceous fly ash

[2] Limiting values for CEM III/A and III/B cements based on limited data

* See EN 197-1 (BSI 2000) for classification of cements.

Table 4. Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by chlorides from seawater for an intended working life of at least 50 years

Exposure class	Moderate humidity	Wet, rarely dry		Cyclic wet/dry	
	XS1[1]	XS2A	XS2B	XS3[2]	
Minimum strength class	C35/45[3]	See Table 5	No guidance given	C40/50[3]	C35/45[3]
Maximum W/C	0.50			0.45	0.50
Minimum cover to reinforcement (mm)	40mm (*in-situ*); 55mm (*in-situ* top horizontal surface); 30mm (factory)			50mm (*in-situ*); 65mm (*in-situ* top horizontal surface); 40mm (factory)	50mm 65mm 40mm
Cements	All			I, I/SR, II/A-S, II/A-V, IV/B[4]	II/B-S, II/A-D, II/B-V, III/A, III/B, IV/A[4]
For combinations Slag (%) Fly ash (%)	 ≤ 80 ≤ 55			 ≤ 20 ≤ 20 and 35 to 55	 21 to 80 21 to 35

[1] Distances from the sea ranging from 100 to 3000m.

[2] Distances from the sea ranging from 0 to 100m.

[3] For II/B-V, IV/A and IV/B concretes the minimum grades may be reduced by one grade.

[4] Siliceous fly ash.

Table 5. Maximum water-cement ratios and minimum cement content for concretes subject to exposure class XS2A

Cement	Fully compacted concrete made with 20mm nominal maximum size aggregate	
	Cement content not less than	Free water-cement ratio not more than
CEM I, II/A-S, II/B-S, II/A-D, II/A-V, II/B-V, III/A, III/B	330	0.50
II/B-V (fly ash ≥ 25% by mass of nucleus), III/B (slag ≥ 74% by mass of nucleus), III/C (slag ≥ 90% by mass of nucleus)	300	0.55
CEM I/SR	280	0.55

3 NEED FOR PERFORMANCE-BASED SPECIFICATIONS AND PERFORMANCE TESTING

As demonstrated in section 2, the European Standard EN206: 'Concrete: Specification, performance, production and conformity' deals with durability of concrete entirely on the basis of prescriptive specification of minimum grade, minimum binder content and maximum water-binder ratio for a series of well-defined environmental classes, although Annex J within this code does give brief details of the approach and principles for performance-related design methods with respect to durability. Although numerous attempts have been made to introduce performance-based specifications, this has been hampered by the lack of reliable, consistent and standardised test procedures for evaluating concrete performance (Nanukuttan *et al.* 2010a, b, c).

The Committee responsible for developing EN-206 recognised that an appropriate testing technology has not been sufficiently developed to satisfy performance-based philosophy (Hobbs 1998). In this respect, there is widespread recognition that central to the concept of performance-based specifications is the requirement for reliable and repeatable test methods which can evaluate the required performance characteristic(s) along with performance compliance limits, which should take into account the inherent variability of the test method. It is evident that test procedures are required such that those properties of concrete which ensure long-term durability can be determined very early on in the life of a structure and that it will meet specified requirements (DuraCrete 1999). The lack of adequate performance-related test methods is one of the main factors inhibiting the move from the prescriptive, *deem-to-satisfy*, approach to performance-based specifications and forms the focus for this paper. Specifically, as it is the cover-zone concrete which provides the first line of defence against the environment (Figure 1), attention is directed towards assessing the performance of this zone (Basheer *et al.* 2007).

Additionally, there is also an intense need to evaluate the concrete earlier to obtain an early indication of potential concerns or, conversely, to gain early confidence that all is well. The sooner information is obtained about the early-age properties of any given batch of concrete, the sooner adjustments can be made to the materials, proportions or processes for subsequent concrete placement and the sooner remedial measures can be initiated on the concrete already installed or construction practices altered (e.g. extended curing). Early-age testing is useful in this regard and, indeed, absolutely essential as the consequences of unsatisfactory concrete discovered at a later stage becomes expensive. The term *identity testing* is used in BS 8500-1:2006 to describe testing to validate the identity of the mix. Identity testing attempts to verify some key characteristic of the concrete that relates to the desired performance and could take the form of a slump, flow, density, strength, water-content or some non-destructive or in-place method. For example, consider a performance requirement for a concrete mix of an in-place chloride-ion diffusivity of, say, 3×10^{-12} m^2/s, assuming that the mix had been pre-qualified based on preconstruction testing. During actual construction the challenge is to perform a test - or suite of tests - on concrete sampled at the time of placement, which can then be used to verify that the concrete, as delivered, is substantially the same as the concrete that had previously been shown to meet the 3×10^{-12} m^2/s diffusivity requirement. There would, in addition, be a need to assess that the in-place concrete has a diffusivity of 3×10^{-12} m^2/s using appropriate testing techniques.

Figure 3 illustrates the usefulness of continuous monitoring of performance of concrete structures in three different stages during the service life of structure, viz. initial stage where material properties of concrete change, second and crucial stage where initiation of deterioration happens and third stage where propagation of deterioration takes place. The conventional approach for the diagnosis of deterioration process involves performing chemical analyses using cores cut from the structure. There are specialised non-destructive and partially-destructive techniques available to assess the condition of concrete depending on the type of deterioration involved. However, these techniques can only provide information on condition of concrete on that particular day and time of testing and it would be an expensive task to test frequently the condition of concrete. A better understanding on the deterioration process and hence the condition of concrete can only be attained by studying the history of temporal and seasonal changes that takes place in the cover zone of concrete. Figures 1 and 2 suggest that most structures are likely to encounter one or more forms of deterioration, unless preventive maintenance are carried out on an ongoing basis to increase the time before the initiation of deterioration. During the initial phase (Figure 3), information on both the initial characteristics of concrete and the ingress of deleterious substances into the structure, and/or their effects on the microstructure of concrete, can be used to determine t_0. The data thus obtained are invaluable for scheduling when to carry out cost-effective repair and rehabilitation works by using appropriate service life models.

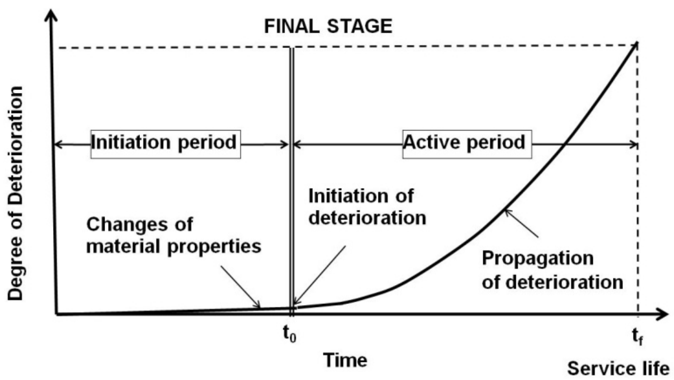


Figure 3. Usefulness of continuous monitoring of structures

4 PRINCIPLE BEHIND THE DEVELOPMENT OF A PERFORMANCE TESTING STRATEGY

It is set against the background in sections 2 and 3 that a strategy to exploit novel testing techniques which can then be used in a performance-based testing protocol for the assessment of concrete durability in marine environments is suggested. The techniques utilise both electrical property measurements and permeation characteristics of concrete as a means of assessing durability and hence long-term performance. In this section, the principle behind the development of the performance testing strategy is given, including how data could be used in service life predictions.

4.1 *Electrical Property Measurements*

As the flow of water under a pressure differential (hence permeability) or the movement of ions under a concentration gradient (hence diffusion) is analogous to the flow of current under a potential difference (hence electrical resistance), it is axiomatic that the electrical properties of concrete could serve as a simple, yet effective, methodology for assessing transport processes, hence durability. To illustrate this, conventional treatment of rock conductivity data has been to use the term formation factor (F), which is defined as the ratio of the conductivity of the saturating liquid (σ_o) to the bulk conductivity of the saturated rock (σ) and linked to porosity, ϕ, through the empirical (Archie's Law) relationship: $F = \sigma_o/\sigma = \phi^{-m}$. In this relationship, the exponent *m* is related to the tortuosity and connectivity of the pore structure within the rock with *m* values lying in the range 1.5-2.5. Millington and Quirk (1964) have presented a relationship relating rock permeability, k, to F and the average pore radius, r, within the rock determined by mercury intrusion porosimetry as: $F = r^2/8k$. The Katz-Thomson model (Katz & Thompson 1986) for saturated porous systems has been developed on percolation concepts and the term *critical pore diameter* is introduced. This model establishes a relationship between the permeability, k, formation factor, F, and the critical pore diameter, d_c, within the system as: $F = d_c^2/226k$. In this approach, fluid flow and electrical conduction through a porous material are determined by the same dimension d_c. Finally, provided that the solid phase can be regarded as an insulator in comparison to that of the interstitial aqueous phase, diffusion and ionic conductivity of a saturated porous system are connected through the Nernst-Einstein relationship: $Q = \sigma/\sigma_o = D_{eff}/D_o = \tau$, where D_{eff} is the effective diffusion coefficient of the material; D_o the diffusion coefficient of the ion (e.g. Cl^-) in the free electrolyte, and τ is termed the tortuosity; the ratio σ/σ_o thus represents the reciprocal of the formation factor noted above. Research (Brite EuRam 1998) also indicates that the single most important factor which influences the corrosion rate of depassivated reinforcement is the conductivity of the surrounding concrete as this controls the magnitude of the corrosion current.

The relationships presented above serve to highlight the direct link between electrical conductivity and those properties of concrete which are of importance in relation to durability namely, diffusion and permeability. The detailed programme of work will serve to exploit the inter-relationships between electrical properties at the macro- and meso- scale levels (discussed below), permeation characteristics and degradation processes. Table 6 (McCarter *et al.* 2000) presents the diffusion coefficient, D_{eff}; permeability, k; bulk conductivity, σ; and formation factor, F, after 450 days hydration for standard mortar samples with different binder types as indicated. It is evident that there is a strong correlation between transport properties and the electrical parameters discussed above, indicating that electrical properties could be developed as a means of ranking the performance of different cementitious systems.

Table 6. Values of D_{eff} (Cl^-), water permeability (k); bulk conductivity, σ, and Formation Factor (F)
MK = metakaolin; MS = micro-silica; GGBS = ground granulated blast-furnace slag; PC = Portland cement (CEM I)

Binder Type	$D_{eff} \times 10^{-12}$ m^2/s	$k \times 10^{-13}$ m/s	$\sigma \times 10^{-3}$ S/m	F (=1/Q) σ_o/σ
PC	2.42	30.5	14.4	126
PC/MS	0.38	5.23	2.0	620
PC/MK	0.41	3.42	4.4	295
PC/GGBS	0.45	8.54	6.0	255
PC/GGBS/MK	0.14	1.21	0.6	1416

Bulk conductivity represents one electrical parameter at a fixed frequency, however, studying the electrical response of concrete over several decades of frequency could give further insights. Concerning performance-based testing, some of the key questions which need to be addressed include: can early-age electrical measurements (< 1-day) be used to predict values at later stages in the hardening process? Further to this, can early-age measurements be used to assess the quality of the as-batched/delivered concrete? Can the electrical response of hardened concrete at the macro- or meso- scale be correlated with sorptivity, diffusivity, corrosion activity and the protective qualities of the concrete cover? What is the interrelationship between electrical properties/response and surface permeation properties?

The sensor systems (Figures 4 and 5) developed by McCarter *et al.* (1992, 1995, 2008) monitors the spatial distribution of electrical conductivity within the cover zone of concrete, which is based on the inter-relationship between electrical properties of concrete with ionic diffusion and corrosion dynamics. These sensors can be embedded in the cover zone of concrete; are robust enough to withstand the harsh environments in concrete as well as the service environment, cost effective and easy to log and store the data; and have good sensitivity and repeatability.

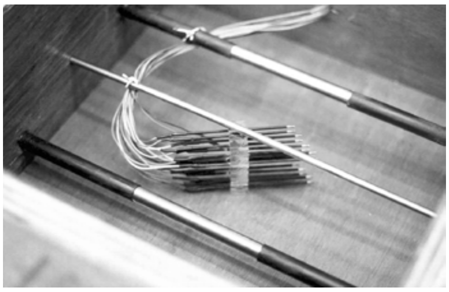


Figure 4. Covercrete electrode array (McCarter *et al.* 1992)

Figure 5. Corrosion and electrical resistance sensor (McCarter *et al.* 2008)

The covercrete electrode array was used in a set of 3 pier stems located on the east coast of Scotland exposed to the North Sea. The piers were constructed at the same time as the Dornoch Firth Bridge. They were constructed for the purpose of experimental work and as such used the same concrete mix design and formwork as the bridge. Construction of the piers was completed in 1992. Figure 6 displays a schematic of the pier-stems at three different locations to provide XS1, XS2 and XS3 exposure conditions. The pier-stem positions are designated high- , mid- and low- level with one each of the untreated PC concrete, Caltite and silane treated pier-stems placed at each location. Figure 7 shows the pier-stems in position at the exposure site. In addition, 18 specimens each manufactured using PC (CEMI), GGBS and PFA concretes were transported and placed at the exposure site and secured in galvanised steel frames (Figure 7); six specimens per mix were positioned at three exposure environments.

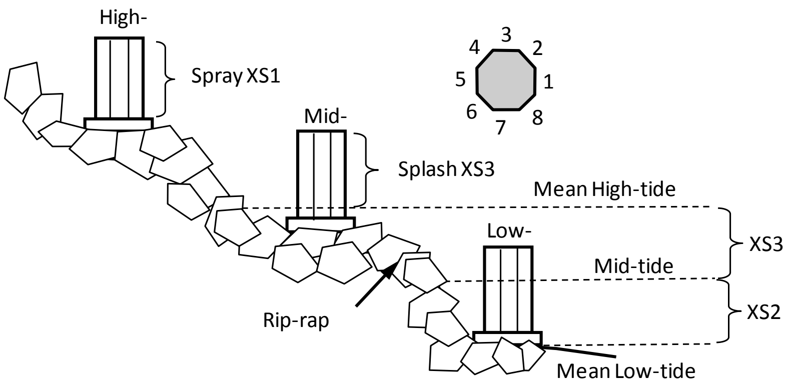


Figure 6. Schematic of the pier stems exposed at Dornoch, Scotland

Figure 8 reports results of the normalised conductivity for PC, GGBS and PFA concretes. The normalised conductivity, N_C, is defined as the ratio of standardised conductivity at a particular electrode position on the sensor at time, t, to the conductivity measured at that respective electrode position taken at a datum point in time. N_c values thus allow relative changes in conductivity to be studied. It is clear that both GGBS and PFA concretes improved in their resistance to chloride transport as their conductivity values decreased with time. However, in the case of the PC concrete, this did not happen. Additionally chloride sampling was done from different levels and the data thus obtained are compared in section 4.3.

4.2 *Measurement of the Transport Properties*

The transport properties (i.e. diffusion, absorption and permeability) of concrete can be related to its microstructure and degree of saturation, hence an assessment of the durability of concrete structures can be made in terms of these parameters (Basheer *et al.* 2006). For instance, if corrosion has been caused by the ingress of chlorides in a cyclic wetting and drying regime, there is excellent correlation between the time to initiation of corrosion and the sorptivity of the concrete (Figure 9). A similar approach incorporating different transport properties can be used in developing performance-based tests for the durability of reinforced concrete structures in exposure classes described in EN206.

Figure 7. Concrete pier stems and concrete blocks exposed to different exposure classes

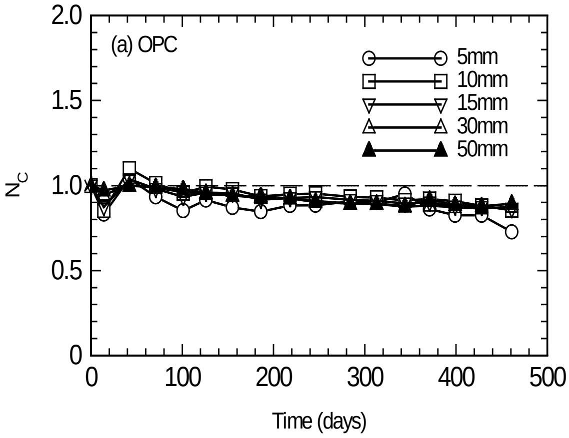


Figure 8(a). Normalised conductivity of PC concretes exposed to marine environment in Dornoch, Scotland

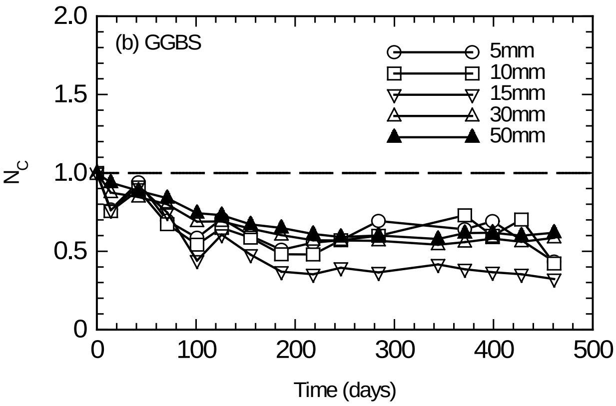


Figure 8(b). Normalised conductivity of GGBS concretes exposed to marine environment in Dornoch

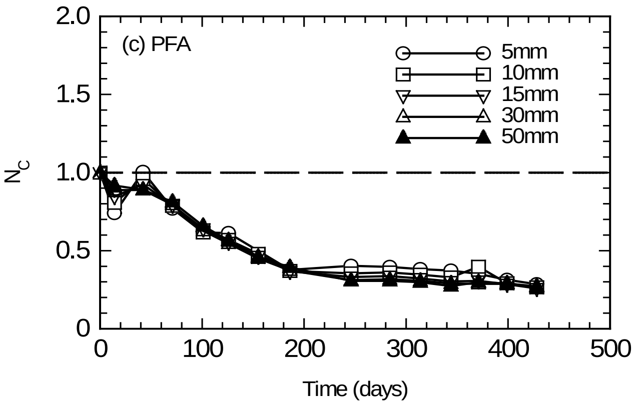


Figure 8(c). Normalised conductivity of PFA concretes exposed to marine environment in Dornoch

The Autoclam Permeability System developed by Basheer *et al.* (1994) for measuring the air and water permeability and sorptivity of concrete *in situ* (Figure 10) and the Permit Ion Migration Test developed by Nanukuttan *et al.* (2008) for measuring the chloride diffusivity (Figure 11) in combination with the pull-off apparatus (Long & Murray 1984) to measure the fracture strength of the cover concrete (Figure 12) can be used as performance tests. Data obtained with these instruments form part of the strategy of performance testing of concrete structures. Typical correlations between permeation characteristics and durability parameters are shown in Figure 13. These data would suggest that performance of concrete structures can be specified in terms of their permeation characteristics, which can be measured during their service life using non-destructive tests, such as Autoclam or Permit.

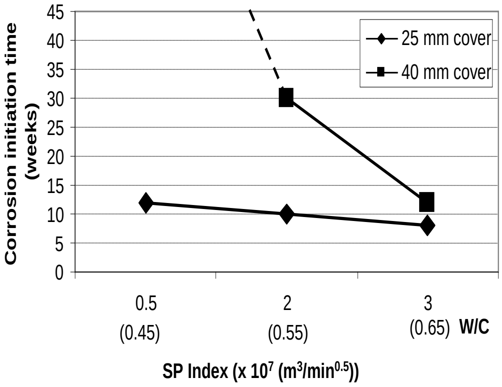


Figure 9. Relationship between time to initiation of chloride induced corrosion and sorptivity index (Basheer *et al.* 2006)

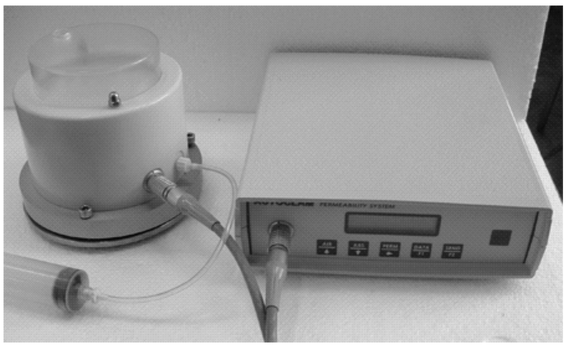


Figure 10. Autoclam Permeability System

Figure 11. The Permit Ion Migration Test

Figure 12. The Limpet Pull off Test

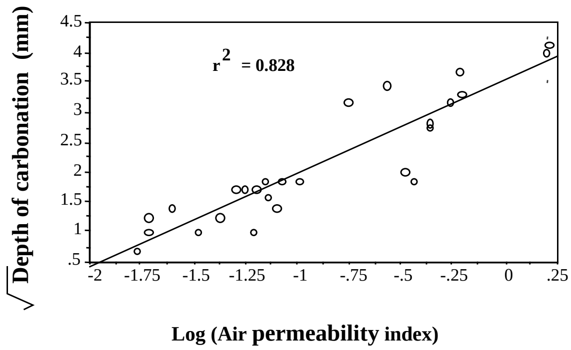


Figure 13(a). Correlation between Autoclam air permeability index and depth of carbonation (Basheer *et al.* 2006)

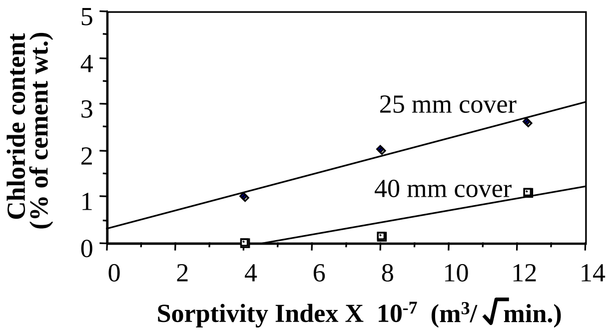


Figure 13(b). Correlation between Autoclam sorptivity index and chloride penetration after 10 weeks of cyclic ponding (Basheer *et al.* 2006)

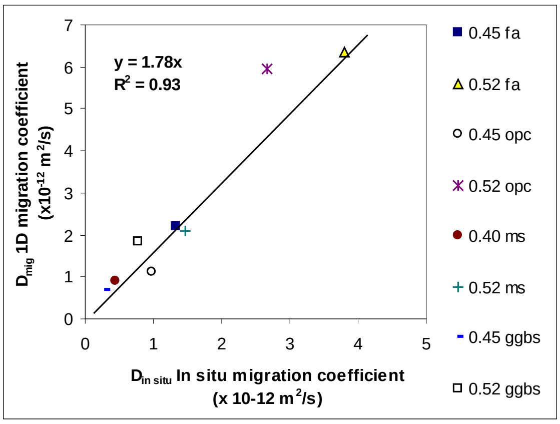


Figure 13(c). Correlation between Permit *in situ* migration coefficient and 1D migration coefficient for concretes containing supplementary cementitious materials (Basheer & Nanukuttan 2007)

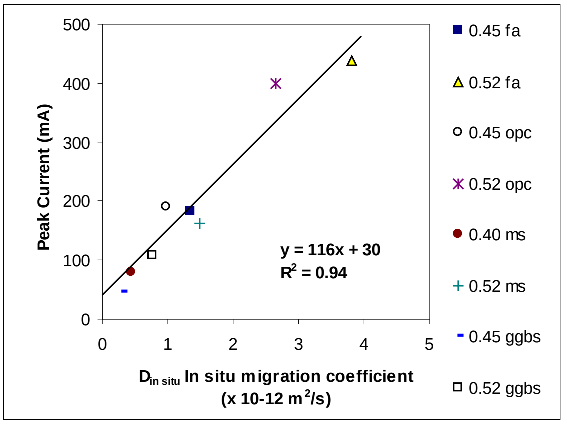
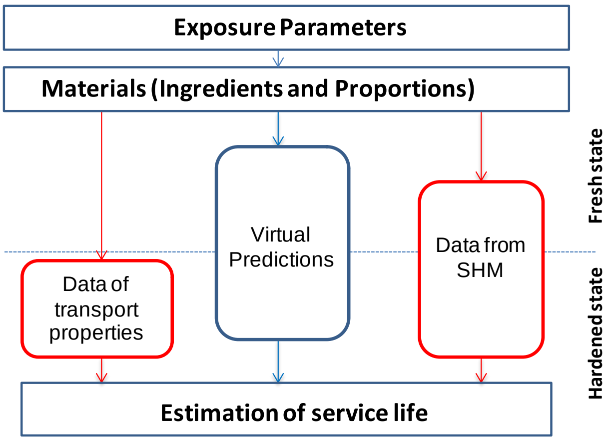


Figure 13(d). Peak current versus Permit *in situ* migration coefficient (Basheer & Nanukutta 2007)

4.3 *Models for Predicting Transport Properties and Service Life of Concrete Structures*

The National Institute of Science and Technology (NIST), Gaithersburg, USA has developed models to predict transport properties from mix parameters and the degree of hydration of the cementitious material (Bentz 2008). Many European researchers have been refining these models for the range of cements specified and used across Europe. In an EU project (Chlortest 2006) it has been found that the "ClinConc" predictive model (Bentz & Tang 2006) was able to estimate the chloride flux for concretes exposed to both northern and southern European environments (Tang *et al.* 2012). The lack of suitable data from real structures and concretes exposed to various environments limited the scope of this project. As summarised in Figure 14, numerous models are available for predicting both fresh state and hardened state properties of concrete and for estimating its service life. Some of these models predict the performance of fresh concretes and some others are used to determine properties of the hardened concretes, both of which then lead to the estimation of the service life.

Accuracy of the models can be improved by using transport properties and/or data from structural health monitoring (SHM).



Service Life Models

Hymostrut (Bentz 2008)
μiC (Bishnoi and Scrivener 2009)
Stadium (Samson & Marchand 2006)
NIST (Bentz 2008)

Life365 (Ehlen *et al.* 2009)
Stadium (Samson & Marchand 2006)
NIST (Bentz 2008)
ClinConc (Bentz and Tang 2006)
DuracCrete (2000)

Figure 14. Service life models and their use. Note: SHM - Structural Health Monitoring

The authors have applied the ClinConc model to 7-year data for concretes in submerged condition at the marine exposure site on the Dornoch Firth (NE, Scotland) (Figure 7). Figure 15 illustrates the chloride penetration in the pier stem in XS2 class using the ClinConc model alongside the measured chloride concentration data at 7.17 years. Figure 15(a) shows some deviation from the measured data, nevertheless the prediction is within the scope for error outlined by ClinConc. The cumulative graph in Figure 15(b) shows an overview of the chloride penetration; from this perspective the ClinConc model is very close to the measured data with just a slight overestimation. These graphs show that the ClinConc model is quite an accurate model for estimating the chloride transport in a marine environment.

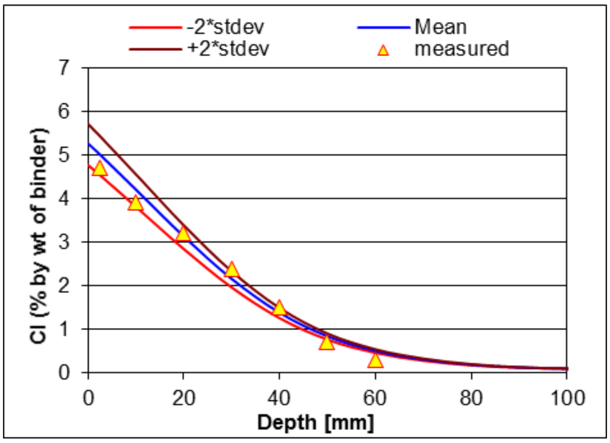


Figure 15(a) The ClinConc service life model compared with the measured data based on chloride profiles.

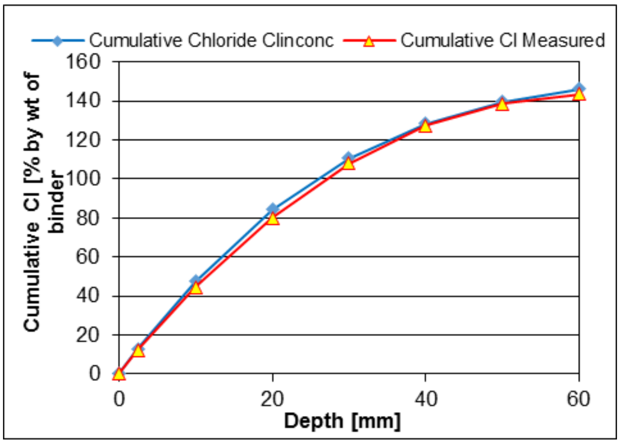


Figure 15(b). Comparison of the model and measured data using a cumulative area approach

Whilst these models are extremely useful for understanding the contribution made by different cementitious materials and various mineral admixtures to the hydration and microstructure of cementitious systems and for the design of concrete mixes with different cementitious materials, they have not been integrated in a performance-based service life design of concrete structures. In this regard, questions which need to be addressed are: can these models provide transport properties which would predict the chloride flux after a certain period of exposure in a marine environment or depth of carbonation in service environments? What modifications are needed to reconcile predictions based on hydration models and electrical measurements of the plastic concrete? How they compare with transport properties measured using in-place NDT methods? How reliable are the models to predict the service life based on initiation of deterioration or end of life? Further research is needed to illustrate the usefulness of not only the SHM data but also the service life models in the context of performance testing and sustainability.

5. PRINCIPLES OF SUSTAINABILITY OF CONCRETE CONSTRUCTIONS

Sustainability is normally assessed at societal, environmental and economic aspects. In the context of sustainable concrete infrastructure, this means that all these three aspects should be given due importance and some simple but relevant questions need to be asked regarding various stages of construction, including those related to the performance of a structure in the service environment and demolition or reuse (Figure 16). A balanced approach needs to be adapted for different structures to address the social and environmental impacts. For example, a bridge may have significantly low environmental impact when reduction in travel time and associated CO_2 emissions are considered. For the same reason it will also improve the living condition of all users and communities in the nearby region, thereby contributing to a large social impact. In most constructions, however, the economic impact alone is given the utmost importance. Figure 17 identifies a closed-loop approach for achieving infrastructure sustainability. Considering the 50-100 years of service life of civil infrastructures the impact related to the construction and decommissioning stages is minimal. Therefore, it is the maintenance stage which needs much attention so that the structure performs satisfactorily with minimum social and environmental impacts. With the help of innovative service life design principles, performance testing and modelling concepts, sustainable concrete infrastructures can be constructed.

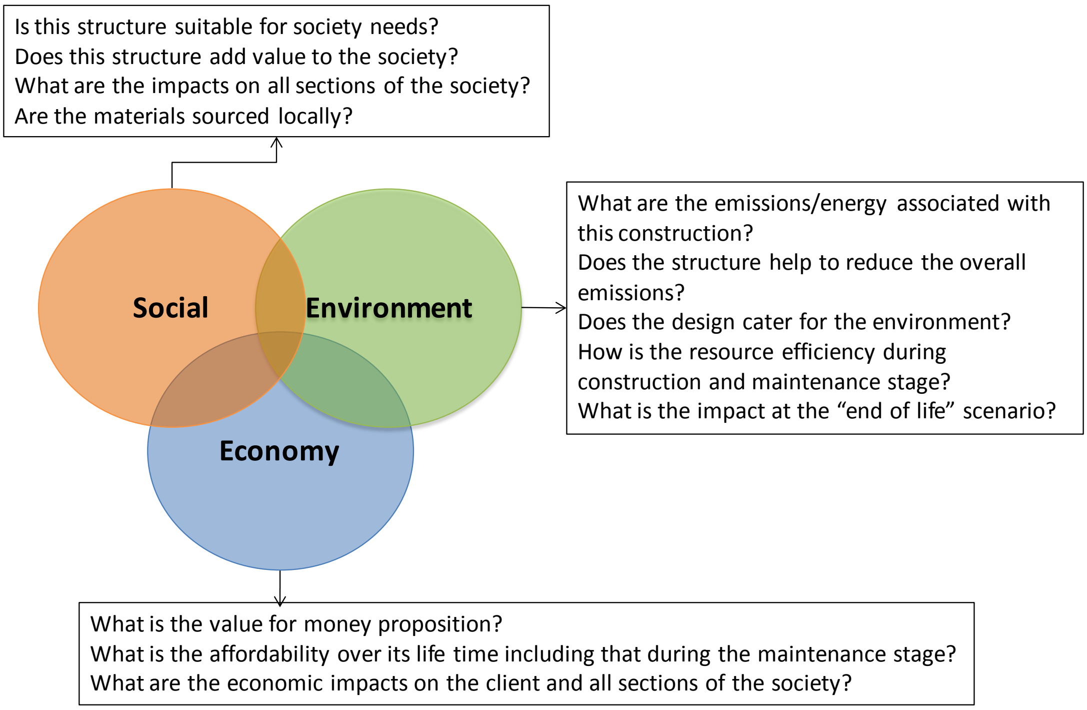


Figure 16. Aspects of sustainability to be addressed in the context of concrete structures

6. ROLE OF THE PERFORMANCE-BASED STRATEGY IN SERVICE LIFE DESIGN OF CONCRETE STRUCTURES

The objective of service life design is to ensure that a structure has adequate durability for its intended service environment (Clifton 1990). Durability is defined here as the ability of a structure or element to perform its required function for its required life with only planned maintenance, and durability is not an inherent property of a material, but it depends on the context in which it is used. Therefore, to ensure the durability of structures, they must be designed to take structural as well as environmental loads (Fohnsdorff & Masters 1990). Section 2 summarised the "deemed-to-satisfy" approach used in Codes of Practice (BSI 2003). However, until recently most codes of practice had no guidance on the lifetime that could be expected from structures designed according to them, or any guidance on how to design for different service lives. Occasionally long-term experience of the user or the user community could be used to ensure the durability, but a proper process to ensure the durability is by resorting to what is known as service life design.

Service life designs comprise essentially four stages (Clifton 1990; Frohnsdorff & Masters 1990; Singh 1991): a design process; identification of the minimum performance requirement (durability); a specified minimum life; and characterisation of the exposure environment. The SLD process comprises: (i) identification of the degradation factors; (ii) identification of deterioration mechanisms resulting from the degradation factors; (iii) identification of critical areas for durability; (iv) establishment of the required service life; (v) establishment of the required condition at the end of the service life; (vi) establishment of acceptable level of maintenance; (vii) establishment of acceptable level of performance; (viii) selection of durability strategy; and (ix) determination of material requirements.

The first step in the process is to identify the degradation factors highlighted in section 1 (Figure 2), such as carbonation, chloride ingress, freezing and thawing, and various forms of environmental penetrations. This information can then be used to identify the likely deterioration mechanisms, such as those shown in Figure 2. It is not necessary that all degradation factors will result in significant deterioration, but consideration of the exposure classes in EN 206-1: 2000 would help to identify probable causes of deterioration and design the structure.

It is important that SLD efforts are concentrated on those parts of the structure which are critical to its performance. The next step in the process is the establishment of the required service life. For instance, buildings and other common structures are designed normally for a service life of 50 years, whereas bridges and other civil engineering structures are designed for 100+years of service life. Consideration is given then to end of service life options, such as no residual life (or demolish), potential for re-use after significant refurbishment or minor maintenance and re-use.

The service life of some materials, elements and structures may be greatly enhanced by regular maintenance, such as re-application of protective coatings. If maintenance is considered, this should be taken into account in SLD while predicting the service life, but one issue is that the effect may not be quantifiable easily. Figure 18 demonstrates the influence of regular maintenance in restoring the performance of a structure (curve 3). It is possible for a structure to be designed for its intended service life without the need to carry out planned maintenance or repair, as shown by curve 1. In this case, the cost involved needs to be justified for a maintenance-free design. As shown by curve 2, if adequate attention is not given to SLD, it is possible for the structure to collapse prematurely as a consequence of either structural failure or lack of durability.

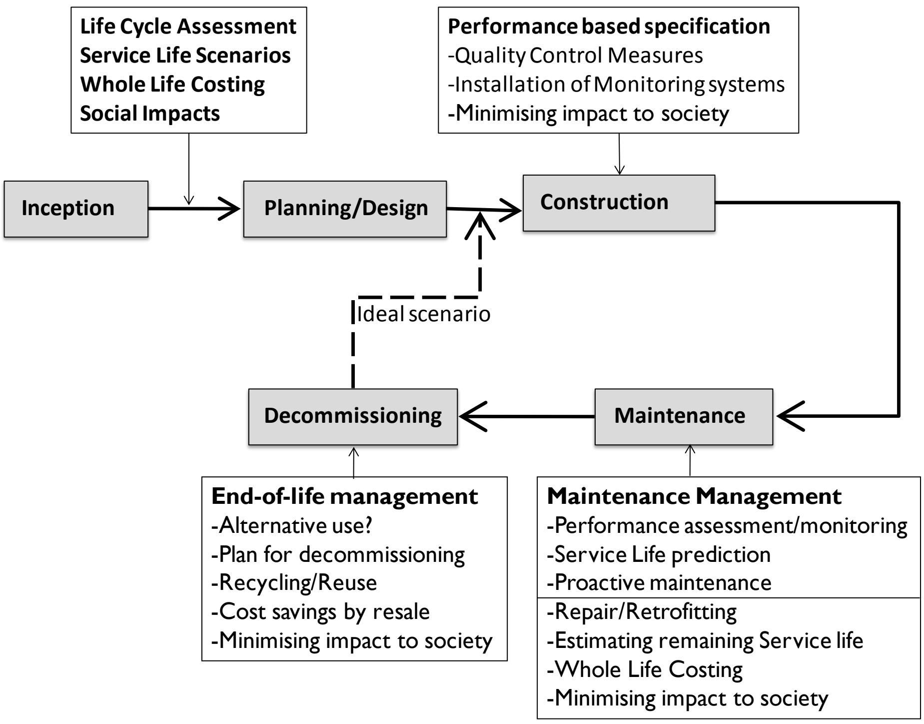


Figure 17. An approach to deliver sustainable infrastructure

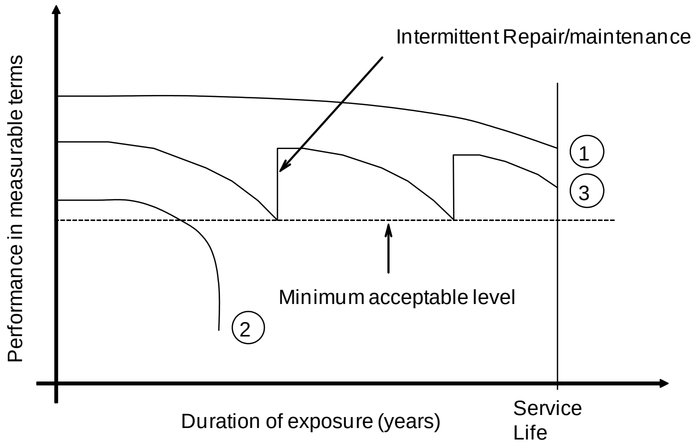


Figure 18. Influence of intermittent repair/maintenance on service life (Somerville 1992)

Establishing the minimum acceptable level of performance depends on the type of structure, its function and the particular requirement of the client. For instance, crack width requirements could be different for reinforced and prestressed concrete structures. Once a decision on the above parameters has been reached, it is essential next to consider appropriate durability strategies, depending on the degradation factors and resultant deterioration mechanisms. Possible strategies could include the use of non-corroding reinforcement, designing adequate cover to provide additional protection to steel against the ingress of harmful environmental penetrations, use of special materials inhibiting deterioration mechanisms from occurring (such as corrosion inhibitors and air-entraining agents), application of surface treatments, or a combination of some of these strategies.

As stated in section 4.3, mathematical models could be used predict material properties as well as deterioration for mechanisms such as carbonation, chloride ingress, rate of corrosion, etc. For the prediction of deterioration, either a deterministic approach or a probabilistic (stochastic) approach could be used (Dempsey *et al.* 2010). The deterministic approach uses specific (mean) values for loads (exposure), for their effect and for service life in a mathematical deterioration model, such as those listed in Figure 14. This approach does not take account of statistical distribution of any of these factors, but a safety factor can be applied to the end result to account for real life variability in performance. On the other hand, the probabilistic approach considers statistical distributions for loads, their effect and for service life. It employs the same mathematical models as the deterministic approach, but considers the probability of failure. It is necessary to know, or reliably predict, the statistical distribution of these parameters.

The foregoing discussion would highlight that key to successfully applying SLD process to ensure durable concrete structures is to identify reliably the environmental loads, isolate the degradation factors and their likely effect on the deterioration process, and model the performance either using deterministic or probabilistic approach for the materials and their expected performance in the service environment. As highlighted in previous sections, the selection of materials depends on performance-based specifications and the reliability of the predicted performance in the service environment influences the estimated life. In relation to the latter, performance tests have a role to play, as stated in section 3.

7. CONCLUDING REMARKS

In this overview paper, the current strategies for durability specifications and design are briefly described first. Limitations of these are highlighted and an approach to make use of both performance-based specifications and performance testing concepts is introduced. A number of the performance tests is introduced and their relevance demonstrated. Finally, the concept of service life design of concrete structures and how it helps to contribute to the sustainability of concrete infrastructure are described. The principles introduced in this paper have recently been investigated in a recently completed project which demonstrated how progress could be made in designing durable and sustainable concrete infrastructure in marine environments (McCarter *et al.* 2017). That is, there is now a real possibility for introducing performance-based specifications for the design of reinforced concrete structures and predicting the service life based on data obtained from *in situ* performance tests.

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