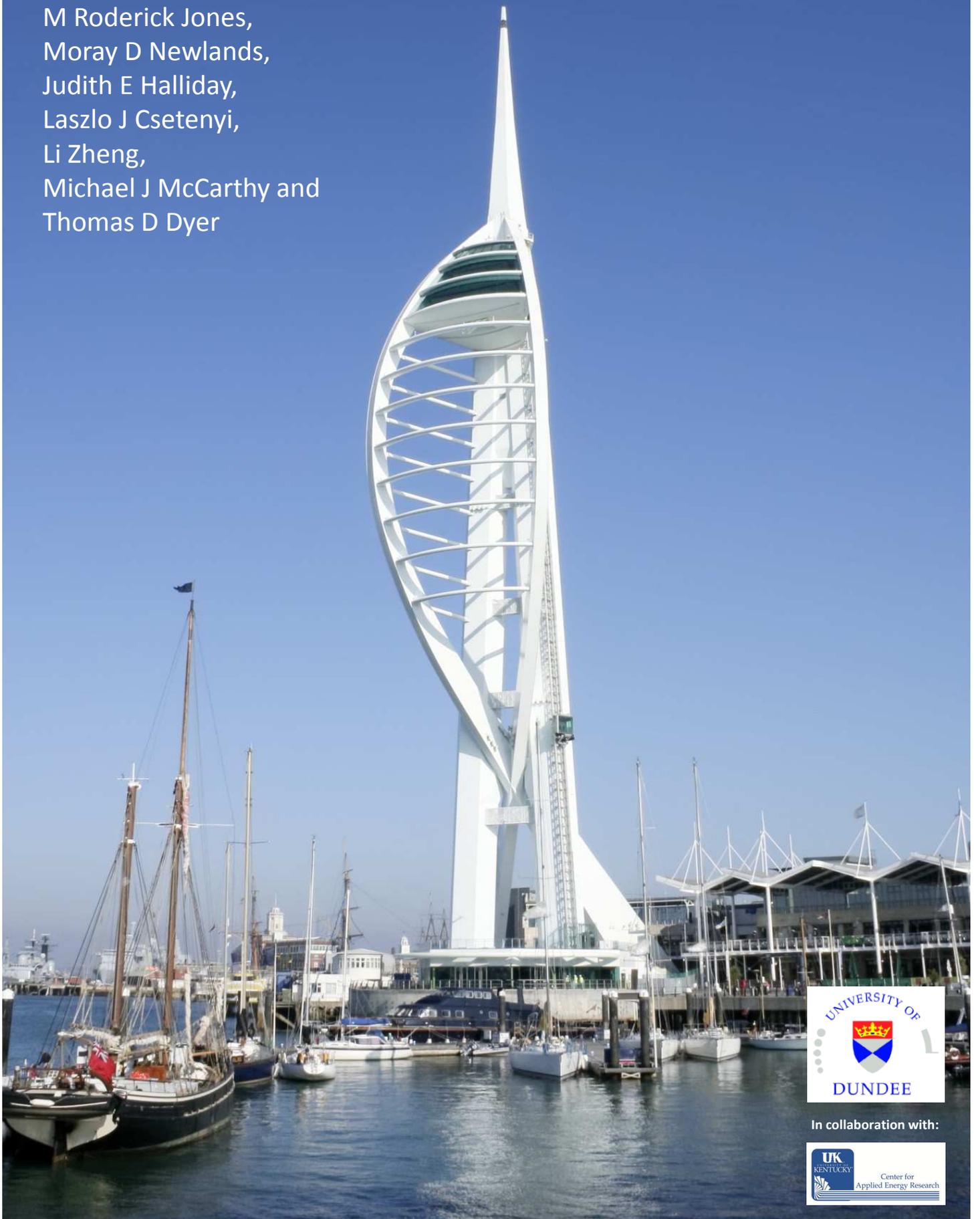


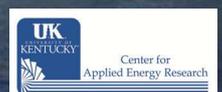
Concrete in the Low Carbon Era

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Proceedings of the International Conference
held at the University of Dundee, Scotland, UK
on 9 - 11 July 2012

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Durability Performance: Models & Test Methods

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For reinforced concrete the environmental conditions that lead to corrosion of reinforcement, such as carbonation or chloride ingress, are recognized as well as those environmental conditions such as aggressive soil or freezing and thawing that lead to the deterioration of concrete itself. Although a complete understanding of all the deterioration mechanisms that affect concrete is some way off there are simplified models for some aspects of deterioration. The simplified models will only be useful where they can incorporate materials performance coefficients that can be obtained in a robust, repeatable and reproducible manner. For this reason it is important to continue to develop and standardize performance tests and models such that the results are both reliable and meaningful. At this time tests for chloride diffusivity, carbonation resistance and freeze thaw parameters have yet to be fully standardized at the European level, although European Technical Specifications are available and are being assessed. In some European Countries an equivalent performance concept has been adopted, a concept that formalizes testing of a proposed concrete to demonstrate performance of not less than a reference concrete of established durability. Formalised guidance for this concept at the European level is likely to be the most practical move towards performance based specification prior to the establishment of indicative performance criteria.

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Keywords: Carbonation, Chloride, Freeze-thaw, Standards, Testing

INTRODUCTION

As stated in the Standard itself consideration is being given to detailing a performance - related approach to the specification of durability in the European Standard for Concrete EN 206-1 [1]. Although some European member states have developed tests and criteria in which they are confident for use in their own country, the necessary Europe-wide consensus for the detailed requirements of performance based specifications is unlikely to be developed in the near future. This is unfortunate as 'sustainable development' is becoming a primary design consideration and the level of green-house gas emissions embodied in construction materials as well as during the in-use phase is already a requirement of an increasing number of buildings and infrastructure projects. Sustainable development should mean that all buildings and structures remain serviceable for their intended working life, preferably with a minimum level of maintenance or refurbishment. For this reason it is very important to have a comprehensive understanding of the likely deterioration processes and how the appropriate quality of material may be specified to provide the required durability.

Currently the UK has comprehensive guidance within the British Standard for Concrete BS 8500-1 [2] in terms of limiting values for intended working life of up to at least 50 or 100 years for concrete exposed to corrosion of reinforcement induced by carbonation or chloride ingress. The problem with the limiting value approach is that the values adopted may be accepted as safe, but there is no way of verifying if they are safe or overly-conservative. In previous years an overly-conservative specification for durable concrete may not have been of great concern as the cost of reinforced concrete construction was relatively inexpensive. The cost of concrete construction is increasing but there is also the added consideration of environmental impacts and social considerations. Environmental impacts and other sustainability aspects are of increasing significance, accepting it is currently difficult to make a technically sound comparison for either buildings [3], or structures such as bridges [4], as the metrics required have yet to be adequately defined.

To ensure that building and structures are not over-designed with respect to durability then it is necessary to develop a performance based specification, where the essential aspect is that the performance can be modelled and that the necessary materials coefficients can be identified and measured.

MODELLING DURABILITY

Exposure classes

There can be no single concrete durability model as the significant deterioration mechanisms will depend on the environment to which the concrete element or structure is exposed. According to European and British Concrete Standards there are five main classes of exposure, these are set out in Table 1.

Test methods

A requirement of any modelling process is the identification of relevant materials coefficients and the ability to measure them, preferably with a standard test method. Around the world there are many test methods pertaining to carbonation, chloride ingress, freeze-thaw and chemical attack. Unfortunately none of these have been developed into a European EN

standard but there are three test methods set out as CEN Technical Specifications. These are CEN/TS 12390-9 for freeze-thaw resistance - scaling [5], CEN/TS 12390-10 for relative carbonation resistance [6] and CEN/TS 12390-11 for the chloride resistance of concrete by unidirectional diffusion [7]. There is no European Technical Specification for testing concrete to measure its resistance to chemical attack. This is not because of a lack of interest rather than procedures have been established at National level where it would be near impossible to agree a test at European level that may result in significantly different guidance. For example, in the UK BRE Special Digest 1 [8] sets out comprehensive guidance on concrete in aggressive ground, but this is almost entirely based on BRE test procedures, experience largely from the UK, and for UK ground conditions. As this paper is limited to Europe wide developments it is only freeze-thaw, carbonation and chloride ingress that will be considered in greater detail.

Table 1 European concrete standard exposure classes

BROAD CLASS DESIGNATION AND DESCRIPTION		CLASS DESIGNATION AND DESCRIPTION	
XC	Corrosion induced by carbonation, where concrete containing reinforcement or embedded metal is exposed to air and moisture	XC1	Dry or permanently wet
		XC2	Wet, rarely dry
		XC3	Moderate humidity
		XC4	Cyclic wet and dry
XD	Corrosion induced by chlorides other than from sea water, where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than sea water	XD1	Moderate humidity
		XD2	Wet, rarely dry
		XD3	Cyclic wet and dry
XS	Corrosion induced by chlorides from sea water, where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides from sea water or air containing salt originating from sea water	XS1	Exposed to airborne salt but not in direct contact to sea water
		XS2	Permanently submerged
		XS3	Tidal, splash and spray zones
XF	Freeze-thaw attack, where concrete is exposed to significant attack from freeze-thaw cycles whilst wet	XF1	Moderate water saturation without de-icing agent
		XF2	Moderate water saturation with de-icing agent
		XF3	High water saturation without de-icing agent
		XF4	High water saturation with de-icing agent
XA	Chemical attack, where concrete is exposed to chemical attack	XA1	Slightly aggressive chemical environment
		XA2	Moderately aggressive chemical environment
		XA3	Highly aggressive chemical environment

Freeze-thaw resistance

For freeze-thaw resistance CEN/TS 12390-9 sets out one reference test method, the slab test, as well as a cube test and a capillary suction frost (CF/CDF) test. In all cases the sample preparation is the same with curing under polythene for the first day and then six days under water at $20 \pm 2^\circ\text{C}$. Thereafter the test specimens are cured in a climate controlled room or chamber at $20 \pm 2^\circ\text{C}$ and an evaporation of $45 \pm 14 \text{ g}/(\text{m}^2 \text{ h})$ for 28, 20 or 21 days depending on the particular test. The evaporation rate is normally achieved with a wind velocity of $\leq 0.1 \text{ m/s}$ and a relative humidity (RH) of $65 \pm 5\%$. After this the specimens are either ponded or immersed with the fluid and left to stabilise for 3, 1 or 7 days prior to the freeze-thaw regime. For each test the freeze-thaw cycle is from -20°C to $+20^\circ\text{C}$, where the duration of one cycle is 24 hours for the slab and cube tests but only 12 hours for the CF/CDF test. The significant variations in test regime are summarized in Table 2 together with recommendations for limit values as discussed by Boos and Giergiczny [9] who tested a range of concretes containing various cement types using the CEN/TS 12390-9 suite of tests.

Table 2 CEN/TS 12390-9 Freeze thaw test methods

TEST, SPECIMENS SIZE AND SATURATION TECHNIQUE	TEST AGE	FLUID	FT CYCLES	CRITERIA	CLASS
Slab test: 150 x 150 x 50 mm sawn surface exposed to 3 mm deep pond of liquid	31 day min. 28 65%RH, 3 ponded	W			
		S	56 (24 h)	<1.0 kg/m ²	
Cube method: 2 x 100 mm cubes, formed surfaces fully immersed in liquid	28 day min. 20 65%RH, 1 immersed	W	100 (24 h)	<10%	XF1
		S	56 (24 h)	<5%	XF3
Capillary suction frost test (CF/CDF Test): 150 x 150 x 70 mm PTFE formed surface immersed 5 mm deep in 10 mm reservoir of liquid	35 day min. 21 65%RH, 7 immersed	W	28 (12 h)	<1.0 kg/m ²	
		(CF) S (CDF)	28 (12 h)	<1.5 kg/m ²	

Notes:
W = De-ionised water
S = 97% tap water and 3% NaCl by mass

Boos and Giergiczny note that the loss of up to 10% by mass of the cubes after 100 cycles of the cube test with de-ionised water is a concrete suitable for XF1 exposure, and at up to 5% by mass then XF3 exposure. This closes the circle in that there is a model, there is a test and there are limits that may directly pertain to an exposure class. Figure 1 show results from Romero et al [10] where two types of concrete were tested using the CEN/TS 12360-9 CDF test. The authors indicated that a scaling of less than 1.5 kg/m² after 28 (12 hour) freeze-thaw cycles is indicative of acceptable performance. As the density of concrete will be no more than 2500 kg/m³ then a material loss of 1.5 kg/m² is equivalent to an average erosion of 0.6 mm, which is the loss of some cement matrix and finer particles of sand.

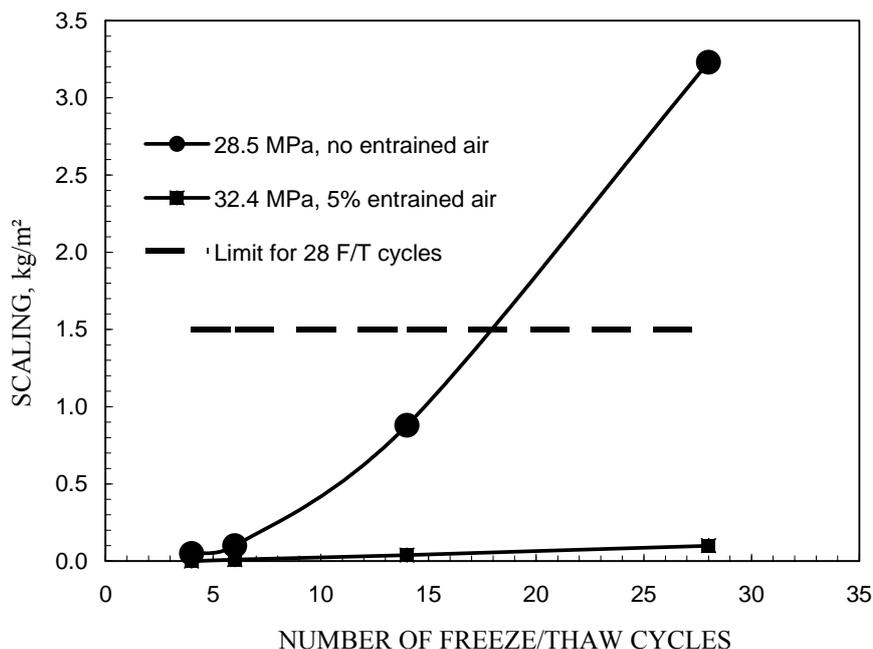


Figure 1 EN 12390-9 CDF performance of non- air entrained and air-entrained concrete

It is important to note that the CEN/TS 12390-9 tests are accelerated tests and that 28 freeze-thaw cycles in the test do not equate to real freeze-thaw cycles. In particular the test samples although cured for one week under water they are then exposed to three weeks curing at 65% RH. An exposure at 65% RH is about the optimum for carbonation and this may mean that concretes incorporating higher levels of ggbs or fly ash as part of the cement may not give test results that reflect their performance in real external environment where the ambient RH may be considerably higher. In addition for the CD/CDF test the samples are immersed in the fluid for seven days so they may be saturated before freezing, and then the temperature cycle goes from +20 to -20°C and back to +20°C in less than 12 hours.

Dransfield [11] summarized a record of freeze-thaw cycles over a six year period from covering the winters from 2006 to 2010 for a location close to a concrete specimen exposure site. This site is at an altitude of ~400 m and registered 182 freeze-thaw cycles defined as an air temperature drop to less than -1°C and then a rise to more than +1°C. The maximum recorded temperature drop over the five years was to -10°C, where it is important to note that the temperature of the test specimens would not have fallen as low as the air temperature. Dransfield did not quantitatively assess the level of attack on the exposure site specimens but did set out a visual ranking assessment after the specimens had been exposed to around 1100 air temperature freeze thaw cycles over nearly 12 years.

Figure 2 shows the NW corner of a nominal 30 MPa, non-air entrained, Portland cement concrete where the comparative assessment stated that this was one of the most severely attacked. Other specimens were either at a higher compressive strength or air-entrained.



Figure 2 Non air-entrained 30 MPa Portland cement concrete exposed for 12 years and around 1100 freeze thaw cycles as measured by air temperature

Interestingly it is a vertical face that exhibits more freeze thaw damage than the near-horizontal surface in the foreground. Current understanding of the freeze thaw mechanism is that a horizontal surface is more likely to become highly saturated and therefore more at risk of damage. It may be that at the particular location the combinations of air-frost and prevailing winds are more severe than ground-frost on a well drained albeit near horizontal surface.

Accepting that there is only a nominal similarity between the non-air entrained Portland cement concrete reported by Romero et al [10] and the field specimen shown in Figure 2 it is interesting to compare performance. The Romero et al concrete lost around 3 kg/m^2 of surface in 28 freeze-thaw cycles, equivalent to an average depth of 1.2 mm. The Figure 2 field specimen has probably not lost as much depth as this over most of the exposed surfaces, except as shown in the top corner closest to the viewer where damage is deeper as some of the coarse aggregate particles are partially exposed.

A conclusion of the comparison between the accelerated CEN/TS 12390-9 freeze thaw scaling test and the single field specimen is that there is no clear relationship. This does not mean that freeze-thaw testing is not useful because it is, for example the test and the performance limit of 1.5 kg/m^2 can differentiate between an air-entrained and non air-entrained concrete with respect to resisting damage by freezing and thawing.

Carbonation resistance

Carbonation is important with respect to durability as it reduces the alkaline environment of concrete from around a pH of 12 to below 9 where ferrous reinforcement is susceptible to corrosion provided there is sufficient moisture and air.

For the determination of the relative carbonation resistance of concrete CEN/TS 12390-10 sets out a method for comparing the carbonation resistance of one concrete with another concrete of the same 28 day reference strength, and where both concretes are cured in sealed polythene bags to an age where 50% of the reference strength is achieved. Once the 50% of the reference strength is achieved the specimens are exposed to one of two environmental conditions, A or B. Exposure A is a climate controlled chamber where temperature is maintained at $20 \pm 2^\circ\text{C}$, relative humidity $65 \pm 5\%$ and carbon dioxide $0.035 \pm 0.005\%$. This

level of carbon dioxide is about the same as the natural atmospheric level but maintaining RH at around 65% means that carbonation is maximised. Exposure B is where the specimens are exposed to an external atmosphere but are protected from any precipitation. This container has an impermeable lid and Stevenson screen slats at the side to protect from driving rain but allow the free circulation of air. As the samples are kept dry the rate of carbonation under exposure B will be greater than most concrete exposed to the natural environment. As this test is reliant on curing each concrete until 50% of its reference strength is achieved then it is representative of very good curing. Using the data presented by Clear [11] a CEM I or CEM II/A type cements takes around 2 days moist curing at 20°C is required to attain 50% of the 28 day strength. Similarly for combinations of CEM I with 50 to 70% ggbs then around 4 days moist curing, and with 50% fly ash 5-6 days moist curing, are required. Carbonation depths are measured at 182, 273, 365, 547 and 730 days ($\pm 2\%$) for type A exposure. For exposure B measurements are at 365 and 730, and then as specified thereafter in days ($\pm 2\%$) Carbonation measurements are made by splitting the specimens and spraying with an indicator liquid that does not produce a colour change where the concrete is carbonated to a pH below around 9.

From the above summary of the CEN/TS 12390-10 test procedure it is evident that as the early curing is controlled to match the type of cement, and the exposure conditions are not representative of actual structures, then the results cannot be directly related to a service life performance. This is accepted as the declared aim of the full test is to show that one concrete may perform as well as another rather than for modelling quantitative performance.

With respect to the service life of a structure the depth of carbonation with time is only part of the information required to assess durability, as can be clearly seen in the traditional corrosion type model described by Somerville [12] and depicted in Figure 3. In this Figure the time taken for the carbonation front to progress from the surface of the concrete to the level of the reinforcement is called the initiation period. During this time there is no visible deterioration and hence damage = 0. Once carbonation is at the level of the reinforcement then it may start to corrode and a level of damage can be measured as the expansion caused by the formation of rust. The period of increasing damage is know as the propagation period as this is when crack will start to form above the reinforcement and may eventually lead to spalling. At some point the damage will reach a level that is unacceptable and this is the end of the service life, unless the damage is repaired.

Numerous studies have been carried on concrete carbonation where a recent study by Jones et al [13] is valuable in that it includes results from long term outside exposure for a wide range of materials. Figure 4 shows the carbonation depths of concrete for specimens externally exposed but protected from wind and rain, so in accordance with the CEN/TS 12390-10 type B requirements but the experiments were started prior to publication of the TS. The period of curing prior to exposure was recorded as up to one month sealed cure as the specimens were made around the UK and then transported to the various exposure sites. The two main exposure sites are Dundee in Scotland and Ringwood in the Southern counties of England. The range of materials include various aggregate and cementitious types where most mixes were made as plain concrete, with a water reducing admixture and a high range water reducing admixtures to achieve w/c ratios from 0.5 to 0.7. Figure 4 shows the relationship between carbonation depth measured at 5-6 years, and the 28 day cube strength.

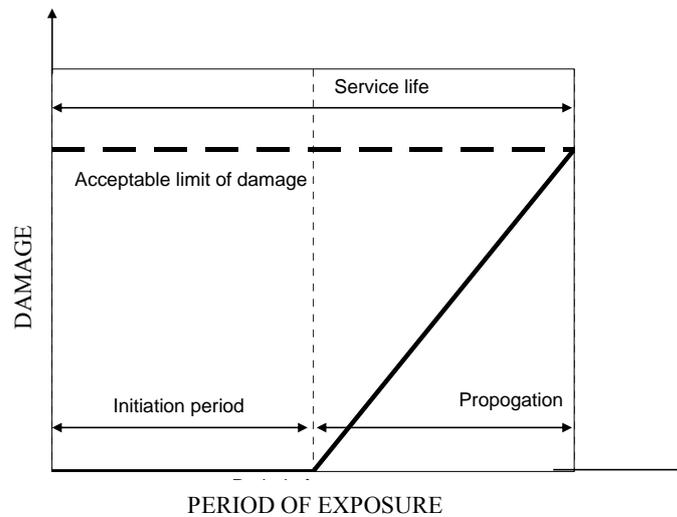


Figure 3 Traditional model for corrosion

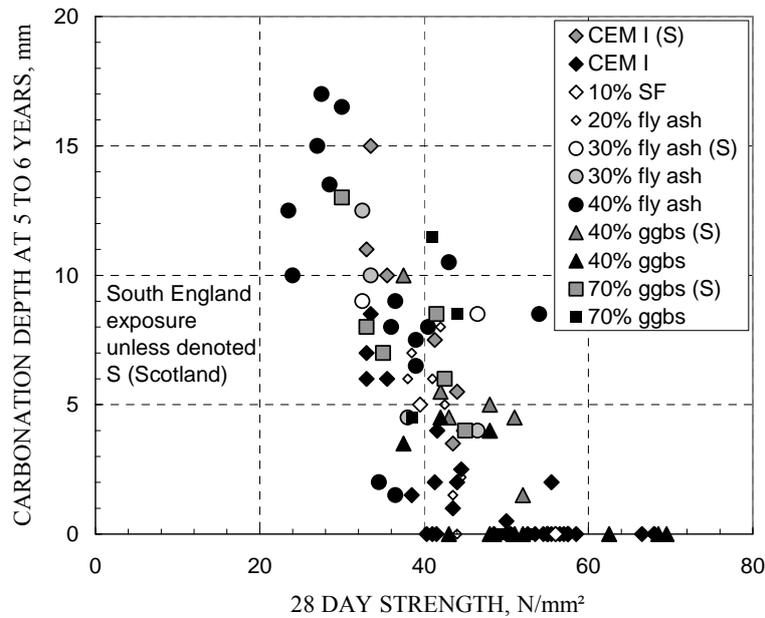


Figure 4 Relationship between carbonation depth and 28 day strength, after Jones et al [12]

From Figure 4 it is evident that there is a relationship between 28 day strength and depth of carbonation irrespective of cementitious material, accepting quite a high scatter of results. Even for the lower strength 25 MPa concrete the depth of corrosion is not much over 15 mm after five years. It is established that carbonation rate is not linear but decreases with time, where it is often quoted a being proportional to the square root of time. Jones et al confirm a similar relationship between the depth of carbonation and time as shown in equation:

$$x_c(t) = K \times t^{0.4}$$

Where x_c is the depth of carbonation at time t and where K is a constant.

A typical concrete strength for building is a C25/30 and from Figure 4 a 30 MPa concrete carbonates to a depth of 15 mm after more than 5 years. This indicates that K is $7.9 \text{ mm}/t^{0.4}$ and so after 50 years a carbonation depth of 38 mm may be expected.

At 38 mm means that the carbonation could have progressed through 30 mm of cover and almost enveloped a 10 mm diameter reinforcing bar.

However, carbonation is only the initiation part of corrosion, as shown in Figure 3 where it takes some time for damage to propagate. As well as simple carbonation Jones et al [12] also investigated the propagation period on pre-carbonated specimens containing reinforcing bars, where the specimens were exposed to an external unsheltered environment to ensure there was sufficient moisture for corrosion to progress at a measurable rate within a year or so. Based on these results they conclude that a minimum propagation period of 15 years is reasonable, and Mattila and Pentti [14] who considered a carbonation worse case of balcony façade elements suggest anything from 5 to 50 years.

So, the important observation is that although the carbonation resistance of concrete is important it is only part of the information required with respect to estimating the service life of a structure exposed to the atmosphere, even in a comparatively simple model.

Chloride ingress resistance

Within the alkaline environment of concrete a passive layer forms around any embedded ferrous reinforcement that protects it from corrosion. This passive layer breaks down if the concrete matrix carbonates or broken down by the ingress of chloride ions. As highway and marine structures will be exposed to the risk of chloride ingress then it is an important factor affecting their durability.

A method for the measurement of the chloride resistance of concrete by unidirectional diffusion is set out in CEN/TS 12390-11, where specimens are cured at $20 \pm 2^\circ\text{C}$ under water for 28 days, conditioned and prepared for at least one day and then exposed for a minimum of 90 days to a chloride solution. The test is defined as non-steady state as the ingress of chloride into the sample is progressive, and there is also a likely to be a significant amount of continued cement hydration which will reduce the chloride diffusion coefficient over the 90 day test period. There are chloride diffusion tests where a thin sample of concrete is exposed to the diffusion of chloride from a defined high chloride concentration on one side of the sample to a low concentration on the other.

In this steady state test the diffusion coefficient is calculated once a stable diffusion rate is established, and as this is usually within hours or days then the effects of continued hydration can be regarded as insignificant. It is also important to note that in the standard non-steady state test the formed or trowelled surface is discarded and it is a cut or ground surface originally 10 mm within the specimen, which is exposed to chloride ingress.

Figure 5 shows a core or a cylindrical specimen but cube specimens are equally acceptable where the chloride ingress would normally be from a side face of the cube.

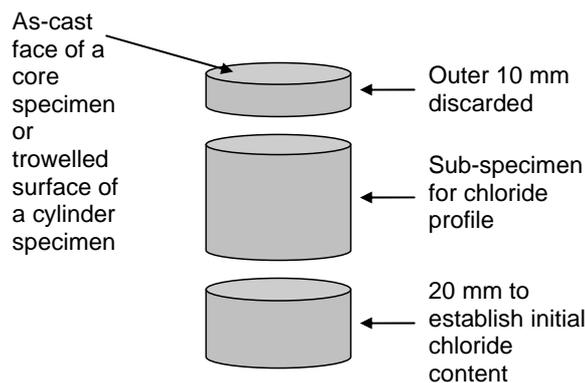


Figure 5 Specimen for determination of chloride diffusion coefficient

To determine the chloride diffusion coefficient at least 8 chloride content measurements are made from successive layers of the profile sub-specimen, where the measurements are expressed as percent chloride per mass of concrete. Guidance on the depth of each layer is provided in term of the w/c ratio and cement type. This is necessary as for low w/c ratio concrete, and/or concrete made with ggbs, fly ash or silica fume as part of the cement the chloride may not penetrate more than 10 to 20 mm. A typical chloride profile is shown in Figure 6 and the chloride diffusion coefficient is calculated by fitting a specified equation to the relevant points by means a non-linear regression analysis by least squares. In the example shown the actual measured surface chloride level, at an average 0.5 mm from the cut surface, is 0.96% and the 'zero point' chloride level 0.015%.

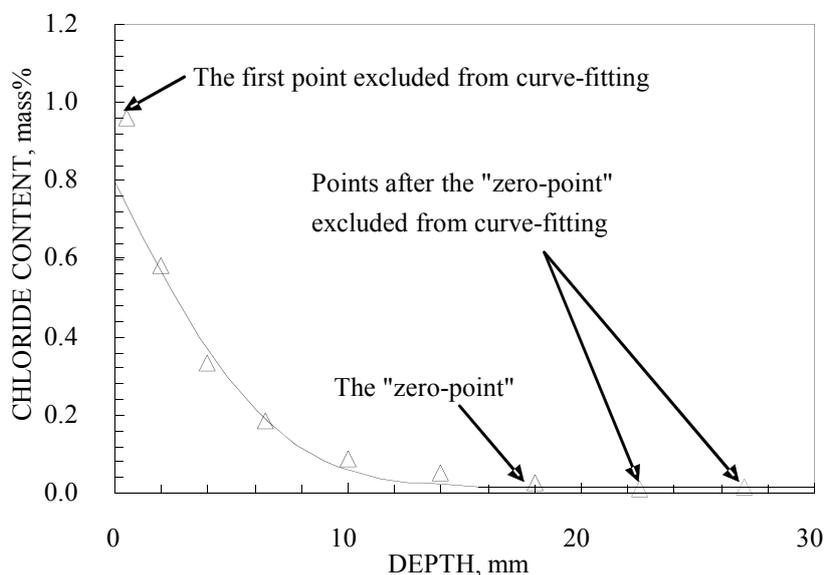


Figure 6 Specimen for determination of chloride diffusion coefficient

The curve fitted surface chloride C_s is 0.8% and the calculated non-steady state diffusion coefficient D_{nss} is $4.5 \times 10^{-12} \text{ m}^2/\text{s}$. As can be seen the determination of non-steady state diffusivity coefficient requires at least four months, and then preparation and analysis of numerous sub-samples as well as quite sophisticated calculations. Despite this it appears to be favoured for modelling the durability performance of reinforced concrete as described by what is a state-of-the-art report by Bamforth [15].

Measuring chloride diffusivity of concrete is essentially subjecting a specimen to XS2 exposure, which is permanently submerged, whereas in a real structure the most severe risk of corrosion is where the concrete is exposed in a tidal zone or subject to spray. This is one reason why chloride diffusivity can only be a partial indication as to the durability of reinforced concrete exposed to sea water, or chlorides other than seawater. In a similar manner to carbonation the progress of chloride to the reinforcement is only important for the initiation stage, and it is not clear how quickly damage is likely to propagate after chloride has reached the reinforcement. Bamforth points towards a minimum, or threshold, level of chloride before the rate of corrosion is significant and this threshold level may depend on the w/c ratio and type of cementitious material.

DISCUSSION AND CONCLUSIONS

The strength of the freeze-thaw scaling resistance test to CEN/TS 12390-9 is that the model of deterioration, the surface damage caused by a number of freeze/thaw cycles, appears to be similar to field conditions. Curing at 65% RH followed by saturation and then undergoing freeze-thaw under saturated conditions is a more severe test than that experienced in the field.

Measuring carbonation on exposure to a protected external environment or a 65% RH climate controlled environment to CEN/TS 12390-10 will also give a more severe degree of carbonation than field exposure. Results indicate that to get a measurable carbonation depth for 40 MPa concrete may take 5 years so this is a major drawback. Another weakness is that the test only gives result pertaining to the initiation period of corrosion, and a significant part of the useful service life may be during the propagation period where it may take some time for significant damage to develop. It is also noticeable that the optimum RH for carbonation is lower than the RH level at which corrosion of reinforcement is likely to occur at a significant rate.

Like carbonation resistance measuring the resistance to chloride ingress using the CEN/TS 2390-11 test only gives an indication of the initiation period under simulated permanently submerged conditions. In addition the test does not include the formed or trowelled surface of concrete where it is known that the cementitious content is likely to be high and hence enhance overall resistance to chloride ingress.

From this brief review of durability test methods it is noted that in each case the property being measured is only a part of the information required to model the service life of a concrete element or structure. It is also evident that the test conditions are generally more severe than those experienced by real structures. Having stated this it should be noted that useful and repeatable tests will always fall short of producing totally authentic materials coefficients often needed to enumerate durability models. This should not mean that durability testing is not carried out, just that when an Engineer or Designer requires a particular durability performance then he or she should be aware of the tests available and the limitations of the results produced.

Where an Engineer or Designer does not feel confident to specify a particular performance directly, then there is the option of specifying that the performance should not be less than that of a concrete where there is general acceptance of performance. This is the basis of procedure being developed by Harrison [15] where a candidate concrete may be proposed on

the basis that it can be demonstrated that it has a durability test performance of not less than that of a reference concrete. It is not difficult to envisage that if this procedure is only adopted for a minority of projects then it would not take long for the Engineer or Designer to establish direct performance values for the range of concretes they specify for particular applications.

ACKNOWLEDGEMENTS

The hard work and expertise of all the Research Team at the Dundee University Concrete Technology Unit over many years is gratefully acknowledged, particularly their contribution to the development of European standards for concrete test methods.

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