ArupMaterialsConsulting

Outokumpu Stainless

Stainless Steel Reinforcement

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Ove Arup & Partners Ltd
13 Fitzroy Street, London W1T 4BQ
Tel +44 (0)20 7636 1531 Fax +44 (0)20 7755 3626
www.arup.com

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Foreword

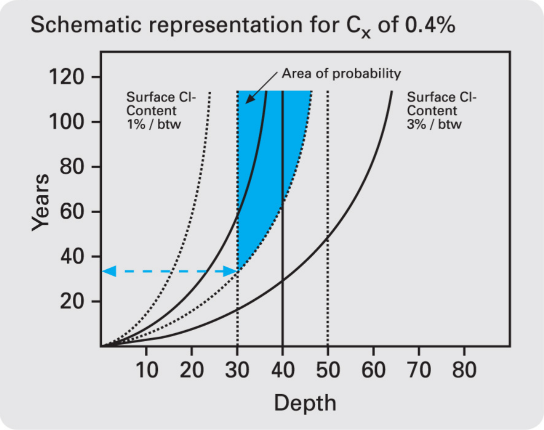
When Outokumpu Stainless Ltd. informed me of their intention to go into full scale production of Stainless Steel Reinforcing Bar (SSR) with their new low nickel duplex alloy, LDX 2101, it was an easy decision for me to make to join them and take up the market development task. For example, corrosion of reinforcement in concrete is all around us, and the only factor apparently holding back the SSR as the solution was the volatility of the nickel price. For this LDX 2101 was the universal answer.

I very shortly discovered though that it wasn't quite as easy to persuade engineers to specify SSR in place of carbon rebar as a corrosion solution. The problem was quite simply that whilst there are standards for SSR the codes of practice for reinforced concrete design said that there is not a corrosion problem and hence there is no need for SSR. Despite all of the visual evidence to the contrary engineers are encouraged to stick to the durability measures in the codes in normal circumstances and ignore the potential of reinforcement corrosion. Not long after, I saw a presentation from Delft University on the application of the DuraCrete model. Encouraged, I quickly researched Life-365 and Concrete Society Technical report TR61 and discovered that not just did these models present designers with a durability solution outside of the normal circumstance of the codes but also challenged the shortcomings of the specifications in the codes themselves. Moreover it became increasingly evident that where SSR was being specified that the designers were using predictive modelling and everywhere else the designers had not heard of predictive models.

The task was simple then: to bring a critical awareness of predictive models to designers around the world and for those already using predictive models to supply quality manufacturers' information on corrosion resistance of the SSR that we produce. In the first instance that is the raison d'être of this report; and in the second it is the subject of ongoing studies at our world renowned Avesta Research Centre in Sweden.

Thankfully my last decision in this task, who to get to produce this report, was as easy as my first. Bryan Marsh is a leading expert in the world of concrete having been instrumental in the development of Standards and one of a handful of individuals producing durability reports on major projects as well as acting as independent checker of others. Suffice to say that where models are being used on major projects there is a good chance that Bryan will be involved. Graham Gedge is a well know materials expert in the world of SSR having written the seminal "Stainless steel reinforcement for concrete; The use of stainless steel reinforcement in bridges" for Outokumpu and subsequently published by the British Stainless Steel Association, and writing the first advisory note on the use of SSR for the UK Highways Authority, BA 84/02 Use of Stainless Steel Reinforcement in Highway Structures.

Finally I think I got lucky when Graham introduced three mathematicians to the project. I was unsure when I sketched out the corrosion profile opposite for our brochure "Activating Your Ideas; Stainless Steel for Corrosion resistant concrete reinforcement" whether it could actually be modelled. Sure, it is a simple way of explaining selective use of SSR but it wasn't in any of the existing models. Thanks to Nathan, David and Shane for this and Bryan and Graham for the report; I hope you the reader will find it useful and informative.



Extract from Outokumpu brochure

Murray A. Adair

Rebar Product Manager Outokumpu Stainless Ltd.

Scope

This report is intended as an aid to designers in the identification of conditions where the selective use of stainless steel reinforcement in concrete structures is pertinent to the achievement of durability. It is intended to be both illustrative and educational, in the sense of being a literary review, rather than developing a new fundamental design tool. It contains a simple deterministic model to describe the ingress of chlorides from sea water or de-icing salt exposure. The model is based on Fick's 2nd law of diffusion, in common with most other common chloride ingress models, but only considers the time to initiation of corrosion and not the propagation period from onset of corrosion until manifestation of damage.

Results from the model in this report should not be used definitely for design purposes and consideration should be given to the limitations of the mathematical modelling of chloride ingress as discussed in this report.

Introduction

The intention of this report is to show how predictive models for deterioration of reinforced concrete can be used to aid the decision making process of whether the selective use of stainless steel reinforcement is a suitable option for avoidance of damage to concrete structures through reinforcement corrosion in chloride environments.

The report briefly reviews the processes that can result in initiation of corrosion of normal carbon steel reinforcement and the most popular mathematical models that have been developed to describe these processes. A simple model is then presented to illustrate the influence of the main parameters, the uncertainties surrounding them, and the effect of using stainless steel reinforcement.

The report provides an introduction to the mechanisms of initiation of reinforcement corrosion in concrete and to the concepts of modelling these processes. It also provides an introduction to the use of stainless steel reinforcement as protection against reinforcement corrosion. It is not the intention of this report to treat any of these subjects in any great detail as the more detailed information is readily available in other publications [e.g. The Concrete Society, 1998; The Concrete Society, 2004].

The report includes a simple representative model (SRM) for prediction of chloride-induced corrosion to illustrate how the selective use of stainless steel reinforcement might be of benefit under certain conditions. The model is based on existing readily available models and is intended to be illustrative rather than attempt to provide a tool for precise analysis. It is intended that this model can be used to compare different combinations of concrete composition and cover to reinforcement in relation to achieving required durability in aggressive chloride environments. These options can in turn be compared with the alternative solution of selective use of stainless steel reinforcement.

Modelling of deterioration processes in reinforced concrete is an imprecise science and is likely to forever remain so because of the inherent variability within materials, exposure conditions and construction quality. Moreover, because of the infinite variety of concrete compositions likely to be produced, and an increasing selection of additional protective measures, it is unlikely that there will ever be a unique solution to achieving durability under a given set of circumstances.

1 The nature of concrete

In its simplest form, concrete comprises Portland cement, fine and coarse aggregates, and water. Nevertheless, most modern structural concrete also contains chemical admixtures and secondary or supplementary cementitious materials such as ground granulated blastfurnace slag (ggbs), fly ash or silica fume. Probably the most common admixtures are plasticisers or water-reducing admixtures which increase the workability of concrete and allow reduction in the free water content required to obtain a certain level of workability. Importantly, from the point of view of durability, these admixtures allow concretes to be produced at much lower free water/cement ratios than would otherwise be practical. Low free water/cement ratios are very significant for concrete as they result in reduced porosity contained within a finer pore structure. This in turn generally results in improved durability and increased strength.

The incorporation of secondary cementitious materials, such as ggbs, fly ash and silica fume, can result in concrete with improved durability through refinement of the pore structure. Well-cured concrete containing adequate proportions of secondary cementitious materials can offer greatly increased resistance to chloride ingress and sulfate attack compared to an equivalent Portland cement concrete. They can also mitigate against damaging alkali-silica reaction (ASR) and delayed ettringite formation (DEF).

Hardened concrete essentially comprises two phases: the aggregate and the matrix. The aggregate can be seen as a filler of varied grading usually from about 20mm down to sub-mm size to ensure effective packing. The role of the matrix is to bind the aggregate particles together and to fill the remaining space between the aggregate particles. The matrix is formed of hydrated cement paste, often containing secondary cementitious materials, but is a porous material. The porosity of the matrix allows passage of water, gases and ions at a rate depending on the size of the pores and the continuity and tortuosity of the pore structure. Porosity often also exists at the interface between aggregates particles and the matrix which can also contribute to the transport properties of the individual concrete. Porosity also exists at the interface between the reinforcement and the matrix which affects the level of protection provided by the matrix.

Transport of fluids through porous media such as concrete is complex and is the subject of books in its own right. Generally speaking, however, the penetrability of concrete to fluids reduces with free water/cement ratio, through the inclusion of secondary cementitious materials, and by prolonged, effective curing which allows hydration reactions to proceed to completion or near completion. The situation is, however, complicated somewhat by interactions of some penetrating media with the concrete itself. Carbon dioxide and chlorides can be bound up to some degree by chemical reaction with components of the hardened cement such as calcium hydroxide and tricalcium aluminate, and water can become physically bound by the very high surface area of the hydration products. Sulfate resisting Portland cement, for example, has a controlled low level of tricalcium aluminate and a consequently low chloride binding capacity which may explain its perceived poor performance in chloride environments.

Concrete is an inherently variable material particularly in relation to its transport properties. This variability can result from many factors including:

- variability within the constituent materials, particularly aggregates and their moisture content
- variation, or even errors, in batching of materials
- variation in compaction
- curing conditions
- environmental effects such as exposure conditions, particularly exposure to moisture

Thus two nominally similar concretes may have significantly different properties. Compare for example a 30% fly ash cement concrete in two different parts of the world. The Portland cement fraction may in one location be of high strength and rapid strength development whereas the other may be low strength and much slower strength development. The fly ash may be very fine, highly pozzolanic and with very little unburnt carbon; on the other hand it could be coarser, of relatively low reactivity and relatively high unburnt carbon. The aggregate could be of high strength and low variability or it could be a much lower quality variable material. The batching plant could be accurate with good control over moisture content of aggregates, or it could be old and unreliable with little control over moisture. Thus it is clear to see that two concretes with the same nominal cement type and free water/cement ratio could vary widely in their properties and performance. It is difficult to allow for such variability within prescriptive specifications for concrete and within models for prediction of performance based on historical data. More accurate use of performance based specifications can be facilitated by prior testing of specific concrete compositions using the actual sources of materials.

2 Transport mechanisms in concrete

Aggressive media can enter concrete by a number of mechanisms including:

- Diffusion – diffusion occurs where a concentration gradient exists across a section such as concrete in constant contact with sea water. The natural tendency is for the aggressive media to spread in an attempt to obtain equilibrium. In the case of dissolved chlorides they require saturated pores to be able to travel into the concrete.
- Permeation – permeation occurs when there is a pressure difference, or hydraulic gradient, across a section which provides a driving force for the movement of water and aggressive media. Common examples would be the walls of deep basements and immersed tunnels, but could include hollow legs of gravity structures such as oil or gas platforms.
- Capillary absorption – capillary absorption occurs where liquids are "sucked" in to empty pores at the contact surface. This process can allow rapid ingress of aggressive media compared to the slower process of diffusion through water-filled pores. Capillary absorption can be of concern for parts of structure subject to infrequent wetting with chloride-laden water after periods of prolonged drying.
- Wick action – wick action is another form of capillary absorption and can occur, for example, where the lower part of an element is in contact with sea water or groundwater and the upper part of the same element is exposed to a strong drying environment. This causes the water, and any aggressive media, to rise up through the concrete. This is of in particular concern in predominantly hot, dry areas such as the Middle East.

In practice, transport of aggressive substances into concrete may be through a combination of more than one of the above mechanisms. A wall in a marine structure, for example, is likely to experience ingress of chlorides at depth by permeation and diffusion whereas in the tidal range ingress is mainly by diffusion. In the splash and spray zones, ingress close to the surface of the concrete may be mainly by capillary absorption but by diffusion at greater distance from the surface where the effect of drying is not felt. Wick action may also transport chlorides from the tidal or submerged zones up into the atmospheric zone. Permeation may be of particular concern in submerged hollow structures, such as immersed tunnels or legs of oil platforms, where a significant hydraulic gradient can exist across the section.

Most chloride modelling is based on an assumption of ingress by diffusion and does not take specific account of other transport mechanisms. It is reported [The Concrete Society, 2004] that the chloride ion profiles in structures where other mechanisms are likely to have been contributory are similar to those resulting solely from diffusion. Nevertheless, the enhanced surface chloride from capillary absorption from extreme environments and/or higher surface porosity is likely to have a significant influence on the notional surface chloride concentration compared with assumed "standard" values.

3 The corrosion of reinforcement in concrete

3.1 Introduction

Steel reinforcement in concrete is normally protected by a passive oxide layer due to the highly alkaline environment of the cementitious matrix. Carbon steel reinforcement will not corrode as long as this oxide layer is maintained. The two main mechanisms by which this protective oxide layer can be broken down are a reduction in concrete alkalinity due to carbonation and the presence of a sufficient concentration of chloride ions (the so-called “threshold level”) at the surface of the reinforcement. For most structures it is possible to protect the reinforcement with an adequate thickness of concrete of sufficient quality to ensure neither of these conditions occur within the intended working life of the structure, or that the progress of any corrosion that is initiated is insufficient to cause disruption of the concrete cover.

Once conditions for corrosion initiation exist, through reduction in alkalinity or chlorides reaching the threshold level at the bar, then corrosion may begin provided there is sufficient moisture and oxygen available at the surface of the steel. This is often represented by a very simple two-stage model originally described by Tutti [1982] as shown in Figure 3.1 [Blin et al., 2008]. In practice, reinforcement at different locations within a structure will, at any one time, be at differing stages within this process due to inherent variations in cover depth, concrete properties, exposure conditions and other factors.

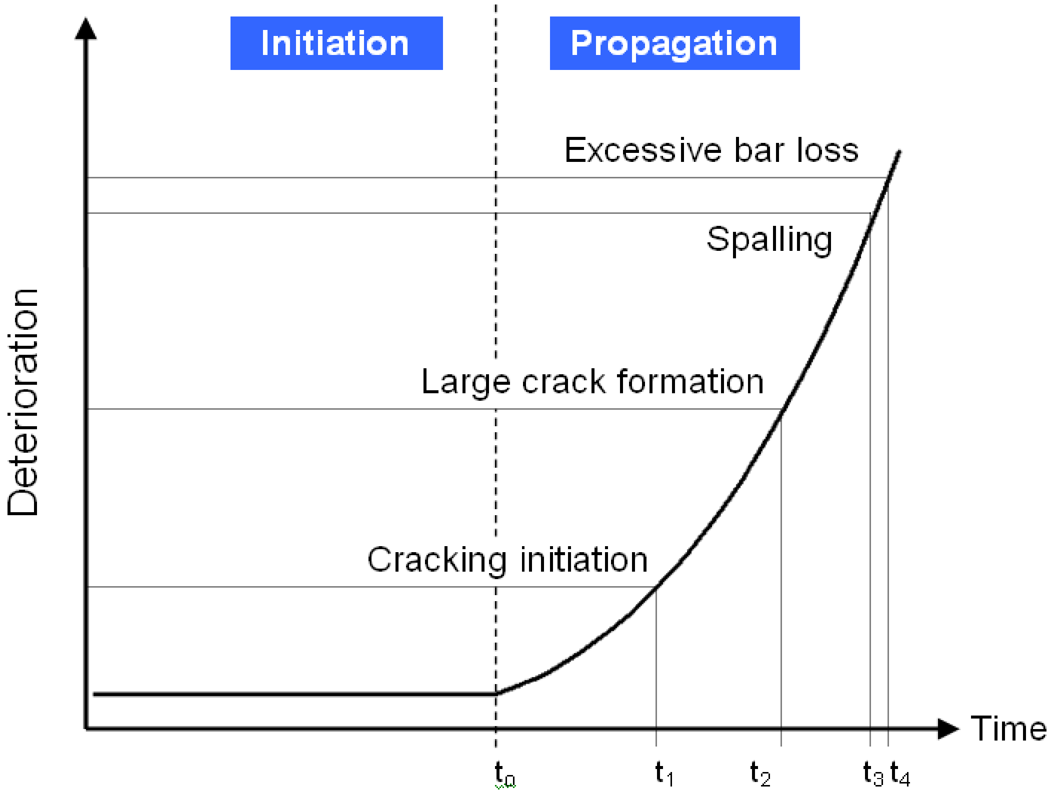


Figure 3.1Simple two-stage model of deterioration of reinforced concrete [Blin et al. 2008]

As is well known, the products of steel corrosion (rust) generally occupy a much greater space than the original steel and their formation thus, unless the surrounding concrete is very porous which should not be the case with structural concrete, creates expansive forces within the concrete cover. The size of these forces will clearly depend on the amount of corrosion products formed which, in turn, will depend on the rate of corrosion and the diameter of the bar. The rate of corrosion will depend at least on the chloride level at the bar, the moisture content (relative humidity) within the concrete, the pH of the concrete, the oxygen availability and the temperature. The ability of the surrounding concrete to resist cracking will also depend at least upon the tensile strength of the concrete and the thickness of the concrete cover. The time between initiation of corrosion and damage to a defined level (e.g. crack width greater than 0.3mm) is known as the propagation period. In some cases such as small diameter bars (e.g. prestressing wires) or localised corrosion (e.g. pitting) the formation of corrosion products may be insufficient to cause cracking.

Once cracks have formed, and where there is sufficient free water movement, some of the corrosion products may be carried through the crack to the concrete surface where they can appear as a characteristic rust staining. Where reinforcement bars are located at the corner of elements the cracks may radiate to the two faces, see Figure 3.2, creating a risk of spalling of the edge concrete. Cracks may also form unseen within the concrete cover zone, parallel to the concrete surface. Where bars are closely spaced these cracks may join up, see Figure 3.3, and cause delamination of the concrete face. In extreme cases whole soffits have become detached through this mechanism.

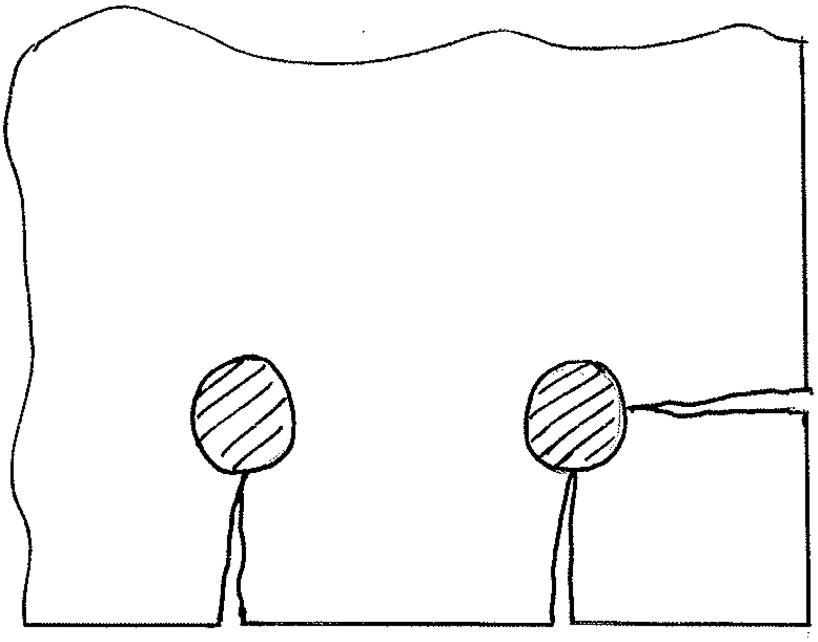


Figure 3.2 Spalling of cover at corner of element

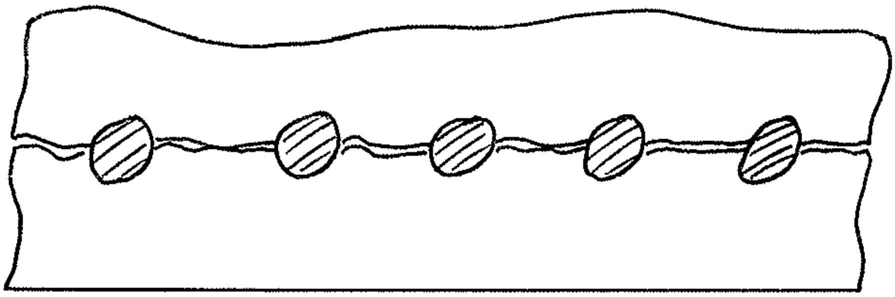


Figure 3.3 Delamination of cover in concrete will close-spaced reinforcement

3.2 Chloride-induced corrosion

There is a notional risk of chloride-induced corrosion of reinforcement in any reinforced concrete in an environment containing chlorides. Such environments include sea water or brackish water exposure, run-off or spray containing de-icing salt, some industrial processes, and groundwater in areas such as sabkas (locations of former salt lakes) in the Middle East. The risk will vary greatly depending on factors such as the concentration of the chloride source and the frequency of contact with the concrete, the composition and thickness of the concrete cover, ambient temperature, and the age of the structure.

Chlorides can enter concrete by various mechanisms - capillary absorption, wick action, diffusion, permeation – depending on exposure conditions, and these have been described in Section 2. The intention of traditional durability design is to select a combination of thickness and quality of concrete cover which is adequate to ensure the chloride concentration at the depth of the reinforcement does not exceed the threshold level within the duration of the design life. The propagation period from corrosion initiation to manifestation of damage is generally not included as it may only be a matter of a few years but is used to provide some margin of safety. Experience has shown that such design has often not been successful and premature deterioration has occurred in many structures such as bridges subject to de-icing salts, and marine structures. Alternative or additional protective measurements, such as the selective use of stainless steel reinforcement, may need to be considered, particularly in extreme exposure conditions, where there are limitations on materials and construction skills, where the design does not permit large cover, or for long design lives.

3.3 Carbonation-induced corrosion

Any concrete in contact with the atmosphere will experience carbonation of the matrix through reaction with the carbon dioxide in air. If there is sufficient moisture present within the concrete there will also be a risk of carbonation-induced corrosion of the reinforcement. The risk of damage due to carbonation-induced corrosion varies widely depending on many factors including CO_2 concentration, ambient temperature and relative humidity, concrete quality, depth of cover and the age of the structure. The intention of traditional durability design is to select a combination of thickness and quality of concrete cover to the reinforcement which is adequate to ensure that corrosion damage is not manifested within the design life. The propagation period from corrosion initiation to manifestation of damage is sometimes included as, under some conditions, the initiation period may be relatively short but the propagation period relatively long. Experience has shown that such design against carbonation-induced corrosion is generally successful and that premature deterioration generally only occurs due to errors in construction such as grossly misplaced reinforcement, or poorly compacted concrete. Alternative or additional protective measurements, such as the selective use of stainless steel reinforcement, will only usually need to be considered in special cases such as elevated CO_2environments (e.g. heavily trafficked vehicle tunnels), or for exceptionally long design lives (e.g. public monuments).

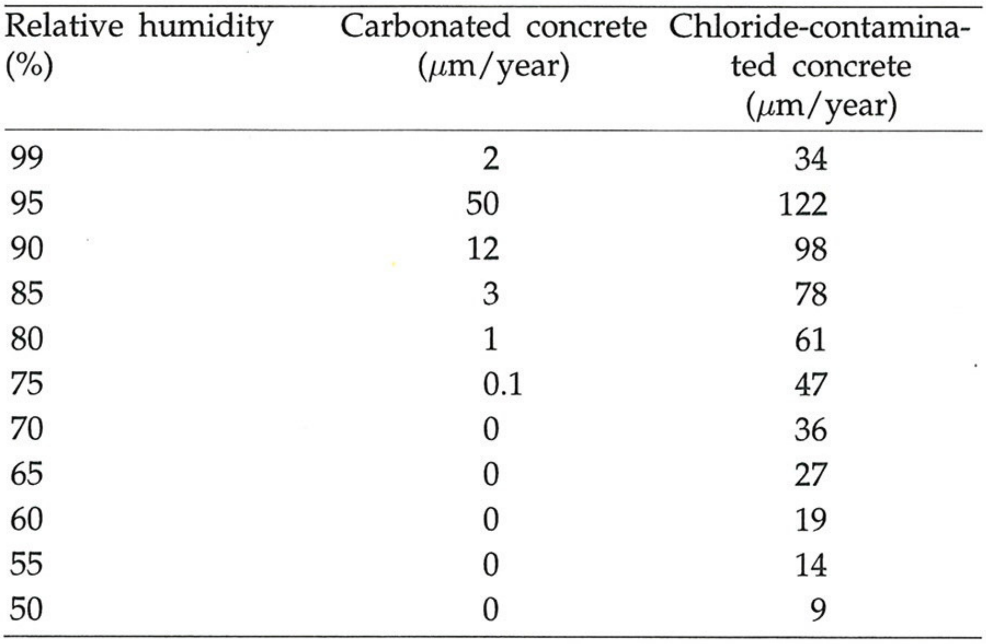
3.4 Influential parameters

3.4.1 Exposure conditions

In the case of chloride environments the severity of the exposure will depend, at least, upon the moisture conditions, chloride concentration and nature of its contact with the concrete, and temperature. Ingress of chlorides is relatively slow where the concrete is saturated, when diffusion is dominant, but can be rapid where chloride-bearing water comes into contact with notionally dry concrete where capillary absorption and wick action can be significant. Permanent or frequent contact with chloride-bearing water will result in a rapid build up in surface chloride level which provides the “driving force” for diffusion. Infrequent or seasonal contact may result in a much slower build-up and rain may act to reduce the surface chloride level through washing out.

For most external structures the CO_2 concentration will be the normal atmospheric level. Carbonation depth is, however, linearly related to CO_2 concentration so can be a major consideration in some enclosed structures such as heavily trafficked vehicle tunnels, particularly if poorly ventilated. Carbonation is slow in saturated concrete and in very dry concrete but will be at a maximum in the relative humidity range of approximately 50-70%. Carbonation-induced corrosion is rarely a problem in structures that are predominantly wet as, although corrosion could be supported, the carbonation process will be very slow and the initiation stage is not reached within the design life. It is also rarely a problem in structures that are notionally dry as, although carbonation may be relatively rapid and the initiation stage reached within the design life, the subsequent corrosion rate will be very low, provided chlorides are absent.

High ambient temperature will increase the rate of reinforcement corrosion, once initiated, but very cold conditions may lead to very low rates of corrosion. Corrosion rates also vary widely with relative humidity but to an extent that differs depending on whether the cause is chloride or carbonation as shown in Table 3.1 [RILEM, 1996].



Relative humidity (%)	Carbonated concrete (μm/year)	Chloride-contaminated concrete (μm/year)
99	2	34
95	50	122
90	12	98
85	3	78
80	1	61
75	0.1	47
70	0	36
65	0	27
60	0	19
55	0	14
50	0	9

Table 3.1 The effect of relative humidity on reinforcement corrosion rates

3.4.2 Design life

Chloride ingress, carbonation and propagation of reinforcement corrosion are all time-dependant processes so longer design lives will generally require more stringent preventative measures. Common nominal design life categories are 50 and 100 years, although 120 years is the accepted design life for bridge structures in the UK and, for many conditions, are covered by design codes. Required lives in excess of this may be termed as “extended” and are outside the scope of design codes so will generally require a special study to decide upon necessary measures.

3.4.3 Cover to reinforcement

Both chloride ingress and carbonation are progressive so cover to reinforcement is a critical parameter in design for a given life; for a given concrete, the greater the cover the longer both the initiation period and the propagation period. The propagation period is increased because the amount of corrosion required to produce sufficient stress to crack the cover concrete is greater.

Accurate fixing of reinforcement is important together with adequate allowance for fixing tolerance in the design. Much premature deterioration is the result of inaccurately placed reinforcement. As an example, a study of nine North Sea offshore platforms [Helland et al, 2008] showed that deterioration was mainly the result of the cover being less than specified or being of poor quality, often due to the difficulty of compaction where the reinforcement was too close to the surface.

There may a practical upper limit to the depth of cover due to considerations such as crack width, spalling on impact or design against deflection, for example, in cantilevers.

3.4.4 Concrete strength class

For a given cement type, resistance to chloride ingress and carbonation will generally increase with compressive strength because of reducing w/c ratio and the consequent improvement in pore structure. Indeed, for a wide range of cements, carbonation rate has been found to be broadly similar for a given strength; exceptions are cements containing high proportions of secondary cementitious materials which generally carbonate at a greater rate. Chloride ingress rates, however, can vary widely for a given strength class depending on cement type.

3.4.5 Cement type

As mentioned above, carbonation rates are broadly similar for a range of cement types at a given compressive strength. High proportions of secondary cementitious materials (e.g. fly ash, ggbs), however, tend to have higher rates of carbonation. Resistance to chloride ingress is highly dependent upon cement type due to differences in pore structure and chloride binding capacity of concretes made with them. Of the cement types commonly used structurally, plain Portland cements tend to have the lowest resistance to chloride ingress and, in particular, sulfate resisting Portland cements which intentionally have a low C_3A content to minimise ettringite formation in contact with sulfates, and consequent low binding capacity. Increasing proportions of fly ash, ggbs and silica fume increase resistance to chloride ingress. Optimal resistance may require triple blends of Portland cement, silica fume and fly ash or ggbs. The resistance to chloride ingress of cements containing secondary cementitious materials tends to continue to improve with time to a much greater extent than plain Portland cements. This is seen later in this report when the age factor is considered in relation to the computational models. National and regional variation in availability of different types of cement and secondary cementitious materials needs to be taken into account. For example, Sweden uses predominantly Portland cements whilst triple blends are common in Australia.

3.4.6 Additional protective measures

Traditional design has relied on a combination of concrete composition and cover to reinforcement to provide the required durability. Experience of premature deterioration in many structures in aggressive environments suggests that this is not always a sufficiently robust approach and that additional protective measures may be necessary. Such measures include:

- Surface protection – semi-permeable or impermeable coatings can prevent or significantly reduce ingress of carbon dioxide or water containing chlorides. Hydrophobic impregnation of the surface can repel chloride-containing waters whilst allowing moisture to dry from the interior of the concrete.

- Controlled permeability formwork (CPF) – controlled permeability formwork allows excess water and air to be expelled from the outer few millimetres of a concrete surface whilst retaining the aggregates, cement and sufficient water for hydration. This produces a surface layer of low w/c ratio, low porosity and increased strength which provides greater resistance to chloride ingress and carbonation.

- Corrosion resistant reinforcement – use of reinforcement with an effective protective coating, non-ferrous reinforcement (e.g. glass-reinforced plastics) or stainless steel reinforcement can be effective through prevention of contact between chlorides and oxygen with the steel, by being non-corrosive, or by having a higher chloride threshold level than carbon steel.

- Permanent formwork – the use of glass-reinforced concrete (grc) or other dense, impermeable formwork which is left in place after the in situ concrete has been placed provides a barrier to ingress of chlorides and carbon dioxide.
- Admixtures – Integral waterproofers or permeability reducing admixtures. Corrosion inhibitors included within the concrete during mixing can help prevent breakdown of the passive iron oxide protective layer on reinforcement when concrete has been neutralized by carbonation or when the chloride level exceeds the normal threshold level for corrosion.

3.4.7 Cracking

There has been much debate over many years about the influence of the presence of cracks in concrete, and their width, on the corrosion of embedded reinforcement. There is considerable evidence from research and practice that the corrosion of reinforcement in concrete in almost any environment is independent of the widths of cracks in the concrete cover. Corrosion initiation may occur in reinforcement at the location of transverse cracks but experience shows that the rate of this corrosion soon declines to an insignificant level. This situation, however, may not necessarily be the same for cracks which are coincident with the reinforcement, for horizontal surfaces with frequent contact with chlorides or where there is flow of chloride-laden water through the cracks. There is also some evidence that the potential for corrosion decreases with increasing crack spacing.

3.4.8 Other influences

Other factors that may influence the rate of carbonation, chloride ingress or reinforcement corrosion include poor compaction; the inherent variability of materials and production; non-inherent variability caused by batching errors, etc. and poor curing, particularly in hot climates.

3.5 The effect of carbonation on chloride-induced corrosion

The effects of carbonation are often not considered in relation to durability in chloride-bearing environments. This is often based on the assumption that severe chloride exposure is usually associated with wet environments, wherein carbonation is slow, or that the effect of the presence of chlorides is dominant. Nevertheless, concrete that is subject to occasional chloride contact under wetting and drying conditions may also be subject to significant carbonation, particularly over long service lives.

Carbonation can result in pore structure refinement whereby resistance to ingress may be improved to some degree but, on the other hand, it can cause the release of chlorides that had been chemically bound during ingress. This increases the free chloride content and the consequent risk of reinforcement corrosion. Another effect that is sometimes overlooked is that the chloride threshold level for stainless steel is significantly reduced in the near neutral pH environment within carbonated concrete. This can be an important consideration where stainless steel reinforcement is being used to provide extended service life in an environment such as the upper tidal marine spray zone in a hot dry climate.

4 Traditional durability design methodology

Most design codes and standards for concrete structures contain a prescriptive or deemed-to-satisfy approach to the design and specification of reinforced concrete in various exposure environments, including exposure to chlorides. This approach generally comprises the specification of limiting values for maximum free water/cement ratio, minimum cement content and minimum cover to reinforcement for various cement types in a limited number of exposure environments. Few such standards contain specific provisions relating to additional or alternative protective measures such as coatings or corrosion-resistant reinforcement. The British Standard for concrete, BS 8500-1, states:

"…there is a degree of uncertainty with the recommendations for an intended working life of at least 100 years in the chloride (XD) and sea water (XS) environments. Reliance solely on cover and concrete quality might not be the most economic solution. In these situations consideration may be given to using other techniques such as stainless steel or non-ferrous reinforcement, barriers, coatings and corrosion inhibitors…"

In many countries' standards a single value of cover and a single concrete composition requirement are given for each exposure class. In the UK, however, a trade-off is permitted between cover and concrete quality whereby lower cover can be used with concrete with high resistance to chloride ingress but larger cover is required for less resistant concrete. Concrete quality is defined by cement type, w/c ratio and cement content; strength class is also included but is only intended as an indirect control on these parameters.

Somewhat surprisingly, despite the obvious time-dependent nature of the risk of reinforcement corrosion, design life has only been included as a specific consideration in the latest generation of standards; Eurocodes and European Standards in particular.

Another recent development in European Standards is specific consideration of the accuracy of placement of the reinforcement. Durability is, of course, dependent on the actual amount of cover achieved in the structure rather than its nominal position assumed in the structural design. Studies have shown [Clark, et al., 1997] that although the mean position of the reinforcement may often coincide with the specified nominal value, the variation about that mean position is often greater than the ±5mm assumed in some older standards such as BS 8110. The European concrete design standard, Eurocode 2, recommends that generally 10mm should be allowed for deviation in cover but some other guidance documents suggest 15mm is more appropriate in areas where construction standards may not be so high.

To ensure the guidance in standards remains manageable it is necessary for exposure environments to be grouped into a relatively small number of standard conditions, and for cement types with broadly similar durability properties to be grouped together. Thus within any exposure class there exists a range of conditions, and within each cement group there exists a range of performance. It is the intention of the standard to ensure the specified recommendations provide the required level of performance for the most severe condition in that exposure class and for the least effective cement type in that group. The safety margin within a durability design using the traditional approach can thus vary widely whilst meeting specified requirements.

Development of the durability provisions in standards has generally not been based on fundamental principles but on the basis of previous experience in practice and knowledge of materials behaviour, but with a significant element of compromise and engineering judgment within code committees. In many environments, such as foundations and building superstructures, experience shows this has worked well with failures often limited to elements which have not built to the design, e.g. low cover. Nevertheless, the failure of some types of structure to achieve their design life without significant unscheduled maintenance or repair, such as bridges in areas of frequent de-icing salt application, has resulted in the belief amongst many durability experts that some requirements in standards are inadequate.

Traditional durability design guidance is usually specific to the area in which the guidance has been developed and may not necessarily be applicable in other locations. A good example of this is the Middle East which, when large-scale development began in the 1960s and 1970s, had no local design codes or standards. Designers often used the design standards from their own countries with no allowance for the often much more aggressive conditions resulting from the high temperatures, aggressive groundwater and high salinity sea water, and often poor construction quality. Much premature deterioration of concrete structures occurred.

5 Conditions where predictive modelling may be warranted

The prescriptive or deemed-to-satisfy route described in the previous section is probably acceptable for structures with an intended working life of, say, 50-60 years in the commonly encountered exposure conditions covered by codes and standards. Nevertheless, a more rigorous approach may be appropriate under circumstances such as where:

- An intended working life significantly greater than approximately 50-60 years is required
- The chloride exposure conditions are significantly more severe than those covered within codes and standards
- The consequences of reinforcement corrosion are high
- Conditions for construction and/or maintenance are particularly difficult

Each of these conditions is considered briefly below. Predictive modelling may also be appropriate where there is a detailed knowledge of the micro-environment.

5.1 Long intended working life

Ingress of chlorides, carbonation of concrete and subsequent corrosion of reinforcement are all time-dependent processes thus it is logical to use a design process that specifically considers this aspect. Traditional durability design methodology often gives little indication of the expected working life and, where it does, generally does not venture beyond 50 or 100 years.

Predictive modelling considers the progress of the deterioration mechanisms with time and thus can be used to predict the condition after any selected duration or to predict the time before a certain defined condition is reached.

5.2 Extreme exposure conditions

Design codes and standards, by their very nature, are usually intended to cover the majority of common situations relevant to that code (e.g. bridges, marine structures). Situations will exist that are uncommon and beyond the scope of the available guidance. Examples might include elements in facilities dealing with high salinity waters, possibly at elevated temperatures. Other examples are found where saline waters come into contact with very dry concrete, such as in dry dock facilities in hot dry climates, where ingress of chlorides by capillary absorption is potentially very rapid.

Extreme conditions can be included by various means including high levels of surface chloride concentration, elevated carbon dioxide contents and increased ambient temperatures.

5.3 High consequences of failure

Corrosion of reinforcement can lead to cracking of the overlying concrete, corrosion staining, spalling and/or loss of reinforcement cross-section. Depending on the expectations and performance requirements for the structure, any of these symptoms may be seen as a failure. Cracking and staining may be unacceptable for a monumental structure whereas, for road or rail tunnels, spalling of even small lumps of concrete may present a risk of serious injury or death to users. Where the consequences of failure are particularly high it is necessary to include a higher degree of reliability, or greater safety margin, into the design. This is generally not readily possible within traditional design other than by simply using a higher specification concrete or greater cover than required by the standard or guidance but with no indication of how much improvement is gained.

Predictive modelling can be used to give an indication of the safety margin after a certain working life or can use conservative values for input parameters to ensure a higher degree of safety than normal.

5.4 Difficult construction conditions

Difficult construction conditions may mean that achieving the required quality of concrete or cover to reinforcement cannot readily be assured. Predictive modelling can be used to determine the effect of low cover or lower quality of concrete.

Certain types of structure may not be conducive to maintenance. Examples might include some nuclear facilities where there may be a danger to health, offshore platforms where access may be excessively hazardous, or industrial process facilities that might suffer large financial losses through temporary shutting down. Predictive modelling can be used to determine after what period maintenance might normally be expected to be necessary. If this is less than the required working life, the necessary conditions can be determined to enable the predicted time to first maintenance to extend to the required working life.

A study of the performance of North Sea oil platforms [Helland et al., 2008] concluded that "…the critical factor has been the execution on site. If this is jeopardized, it does not matter how tight against chloride ingress the mix is designed for and how big the specified cover is". They add in relation to difficult construction conditions that "…the accompanying uncertainty for the quality of the cover zone should be taken into account and compensated for". It is difficult to include poor execution specifically within a predictive model but it is possible to determine how sensitive the design is to variation in concrete quality and cover to reinforcement.

6 Environmental exposure conditions

All exposed conventionally reinforced concrete is, in theory, at risk of corrosion of reinforcement either through loss of alkalinity (carbonation) or through ingress of chlorides (usually from sea water or de-icing salts). That risk can vary from negligible in heated indoor environments to high in hot climate marine structures with a long required service life. It is the responsibility of the designer to ensure the design provides adequate protection against significant damage due to corrosion throughout the required life without the need for excessive or unplanned maintenance.

All uncoated superstructure concrete is subject to carbonation through reaction of the concrete with the naturally occurring carbon dioxide in the atmosphere. The rate of this reaction will depend on the moisture content in the concrete and the ambient temperature. For normal design lives of, say, 50-100 years this is not a difficult exposure condition to deal with. Nevertheless, some structures, such as heavily trafficked and/or poorly ventilated vehicle tunnels, may be subject to elevated carbon dioxide concentrations which will increase the rate of carbonation and need special consideration. Carbonation may also become a significant exposure condition if a particularly long working life is required.

Generally speaking, chloride-bearing exposure conditions are far more severe than carbonation environments. Exposure to chlorides may be through direct or indirect contact with sea water in marine or coastal structures, with de-icing salts in highway structures, or with chloride-bearing waters in industrial processes. The severity of the exposure will depend on several factors including:

- Concentration of chlorides
- Frequency and duration of contact
- Initial moisture content in concrete when coming into contact
- Ambient temperature

Design standards for concrete contain exposure classification systems for the various environmental conditions likely to be encountered by the type of structure covered by the particular code. There is, however, considerable variation between different standards.

Recent standards such as the European Standard, EN 206-1, and the latest revision of the American Standard ACI 318 contain important developments in the classification of exposure.

EN 206-1 contains three exposure classes covering risk of corrosion of reinforcement: one covers the risk through carbonation whilst the other two cover chloride ingress but differentiating between chlorides from sea water and chlorides from other sources. Each class is then divided into sub-classes depending on the moisture content within the concrete. The resultant ten classes are regarded in many of the CEN countries as excessive and national application documents frequently combine some of the classes. In particular, several countries regard it as unnecessary to distinguish between the sources of chlorides and combine the XS and XD classes.

- XC – risk of carbonation-induced corrosion of reinforcement
    - XC1 – dry or permanently wet
    - XC2 – wet, rarely dry
    - XC3 – moderate humidity
    - XC4 – cyclic wet and dry
- XS - risk of chloride-induced corrosion of reinforcement from sea water
    - XS1 – exposed to airborne salt but not in direct contact with sea water

    - XS2 – permanently submerged
    - XS3 – tidal, splash and spray zones

- XD - risk of chloride-induced corrosion of reinforcement from chlorides other than from sea water
    - XD1 – moderate humidity
    - XD2 – wet, rarely dry
    - XD3 – cyclic wet and dry

The most recent version of the American building code requirements, ACI 318-08, has followed the lead of EN 206 in making exposure conditions specific to the deterioration mechanism affecting the concrete.

- C – corrosion protection of reinforcement
    - C0 – concrete dry or protected from moisture
    - C1 – concrete exposed to moisture but not to external sources of chlorides
    - C2 – concrete exposed to moisture and an external source of chlorides from de-icing chemicals, salt, brackish water, sea water, or spray from these sources

In effect, this simply distinguishes between no risk, risk from carbonation, and risk from chlorides with no gradation of the latter two. This doesn't seem to be a very sensitive design tool.

Neither EN 206-1 nor ACI 318-08 takes ambient temperature into consideration.

7 Stainless steel reinforcement

7.1 Stainless steel types and grades

Stainless steels are from a family of steels that are specifically alloyed to provide a combination of corrosion resistance and mechanical properties. Stainless steels used in reinforcement for concrete are of two types, see Table 7.1:

Stainless steel group	Example EN Grades	Example US Grades
Austenitic	1.4301, 1.4436	304, 316
Duplex	1.4462, 1.4362, 1.4162	2205, 2304, 2101

Table 7.1 Stainless steel groups used for reinforcement

The differences between these steels are in the alloying element contents; generally the austenitic steels have higher nickel contents but lower chromium contents than the duplex steels. Nickel does not influence corrosion resistance but is required to promote a particular microstructure. Both austenitic and duplex grades may contain Molybdenum and Nitrogen to provide enhanced localised corrosion resistance.

The alloy content influences not only corrosion resistance and mechanical properties but also cost. Alloys with high Nickel and Molybdenum contents are significantly more expensive than alloys with low contents of these elements. For reinforcement austenitic steels have traditionally been more readily available than duplex steels and have therefore been more commonly used. In recent years development of duplex alloys with comparable corrosion resistance to austenitic steels has occurred and these steels are now widely available. Modern low alloy duplex steels appropriate for use in concrete (with very low Nickel and Molybdenum contents) offer a more economic alternative to established austenitic grades.

7.2 Corrosion resistance of stainless steel

As for carbon steel the corrosion resistance of stainless steel is due to the formation of a passive layer on the surface. Nevertheless, in stainless steel this layer is rich in chromium oxide and prevents corrosion in many natural and man made environments. However, stainless alloys that only contain chromium may be susceptible to localised corrosion particularly in environments that contain chlorides. To improve long term resistance to localised corrosion Molybdenum and Nitrogen are added to the alloys to improve the passive layer.

For steels used in concrete the stability of the passive layer is very strongly influenced by the pH of the environment. In non-carbonated concrete the pH at the level of bar will be high, typically >12 and in these conditions even carbon steel will be passive. However, the presence of significant chloride transport to the bar can result in local loss of passivity and severe attack of carbon steel even at these high pH values. The stability of the passive layer on all stainless steels is very much greater than that on carbon steels and the chloride concentration (threshold level) required to breakdown passivity is probably at least an order of magnitude greater than for carbon steel. In broad terms the stability of the passive layer increases with increasing alloy content but in most applications adequate performance can be achieved with relatively lean and cost effective alloys.

7.3 The selective use of stainless steel reinforcement

Stainless steels are relatively expensive compared to conventional plain carbon steel reinforcement and should therefore be used appropriately to minimise the cost impact on a particular project. Appropriate use is determined from two factors:

- Selection of the appropriate alloy grade for the given service condition. Over specification with high alloy content steels may of little, if any, improved performance but may have a significant impact on cost.
- Use of stainless steels on parts or structures at risk of chloride induced corrosion within the design life, so called selective use.

The concept of selective use was first developed for highway structures and bridges by Arup for the UK Highways Agency. The approach is outlined in UK Highways Agency advice note BA84/02 [Highways Agency, 2002]. This approach has been more generally recognised and used in relation to stainless steel reinforcement. This approach allows the use of ordinary carbon steel for the majority of the structure in conjunction with stainless steel only in those areas that are at risk of chloride induced corrosion from marine environments and/or road de-icing salts and is far more cost effective than the wholesale substitution of carbon steel with stainless steel. BA84/02 advises that total replacement should be limited to major components where the consequences of future repair are likely to be highly disruptive and costly and the possibility of chloride attack is likely. Particular examples, for busy motorways and trunk roads, where this approach is recommended are:

- Parapet edge beams
- Substructures in splash zones adjacent to carriageways
- Substructures in marine environments and superstructures in splash zones
- Below movement joints and on associated bearing shelves

Where stainless steel reinforcement is used it is also permitted to reduce the cover for durability to 30mm, increase the allowable crack width to 0.3mm, and omit the application of a hydrophobic surface impregnation.

7.4 Commonly used stainless steel grades

The wide range of stainless steels that are available, generally and as bar material, can be confusing to the end user/designer unfamiliar with the subtleties of stainless steel compositions and classifications. Over the years the issue has not been satisfactorily resolved by the development of national or international standards and the introduction of new grades is likely to potentially further complicate matters. However, in the context of reinforcement in concrete the situation can be simplified based on the likely exposure condition.

Exposure Condition	**Austenitic Grade**	**Duplex Grade**
Fully cast in concrete with effective temperature <25°C	1.4301	1.4162 or 1.4362
Fully cast in concrete with effective temperature >30°C	1.4404	1.4462
Cast in concrete but spanning an open joint or protruding from the concrete (not in seawater)	1.4404	1.4462
For use on bridge decks without waterproofing membranes	1.4404	1.4462

Table 7.2 Current recommended grades for use in concrete

The grades in Table 7.2 are minimum requirements which will suffice in almost all cases and represent the most effective solution to grade selection. Alternative grades maybe proposed and provided the detailed composition of the major alloying elements are the same or higher than those for the grades given in the table then adequate corrosion resistance should be achieved.

Current advice is prescriptive and does not take account of time, rebar depth and surface chloride concentration that are explained in more detail in other sections of this report. Research is ongoing into these parameters and the development of future advice for inclusion in codes and standards.

7.5 Typical uses of stainless steel reinforcement

Typically stainless steels are used selectively where there is a risk of chloride induced corrosion of reinforcement where part or all of the structure is exposed to salt from either a marine environment or road de-icing salts. Examples include:

- Bridge decks exposed to road de-icing salts where water proof membranes are not used, this occurs mainly in North America.
- Bridge abutments, columns and edge beams exposed to splash waters that contain road de-icing salts.
- The intertidal and splash zones of coastal and marine structures.
- Hard standings in ports and harbours that are exposed to wetting and drying with sea water.

It should be noted that for reinforced concrete structures or parts of structures that are permanently immersed in sea water corrosion risks associated with carbon steel reinforcement are negligible; substitution with stainless steel is therefore unnecessary.

Stainless steel reinforcement may also be used where very long design lives, in excess of 100 years, are required where inspection and maintenance is either impossible or very undesirable, for example structures associated with nuclear power generation.

7.6 Example structures

7.6.1 Stonecutters Bridge, Hong Kong

The Stonecutters Bridge is a cable stayed bridge spanning the Rambler Channel entrance to the sea container port in Hong Kong. The bridge is supported from two mono towers at each end of the main deck. The lower 60m of each tower (below deck level) are constructed from reinforced concrete. The very close proximity to the sea and the requirement for long, maintenance free life for the structure led the designers to develop a particular durability strategy for the reinforced concrete; this included the selective use of stainless steel reinforcement.

Stainless steel grade 1.4301 was used for the outer layers of the main reinforcement in the towers between the pile cap and deck level. The bars were supplied in 9 and 12m lengths in diameters between 25 and 50mm. In total approximately 3000 tonnes of reinforcement were supplied to the project.

Figure 7.1 Fixing Stainless Steel reinforcement for the columns on Stonecutters Bridge

7.6.2 Shenzhen Western Corridor

The Shenzhen Western corridor bridge is a highway structure crossing the sea between Hong Kong and mainland China. It is comprised of multiple span approach bridges and a central navigation span cable stayed bridge. Stainless steel grade 1.4301 was used for the reinforcement of the support structures to the approach bridges in a range of diameters up to 40mm.

Figure 7.2 Stainless steel reinforcement on the Shenzhen Western Corridor

7.6.3 Broadmeadow Bridge, Swords County Fingal Ireland

The bridge was part of the new Northern Motorway. The bridge spans 300m over spanned an environmentally sensitive estuary and flood plane and is constructed from a concrete box girder supported on circular columns. The sensitivity of the area meant that a highly durable structure requiring minimal maintenance was required. To achieve this aim the column reinforcement cages were fabricated from 316L stainless steel.

Figure 7.3 Broadmeadow Bridge and detail of stainless steel reinforcement cage for columns

8 Predictive modelling

Traditionally selection of an appropriate concrete composition required to provide the intended working life, or design life, in the environment to which it will be exposed has been based on empirical relationships between composition and performance. Most design codes and standards for concrete structures contain “deemed-to-satisfy” composition requirements related to exposure conditions but often with no explicit consideration of design life. Experience has shown that this approach is often satisfactory for structures or elements in common exposure conditions over “normal” service lives of, say, 50-60 years. Experience has also shown that this approach may not give satisfactory results under certain conditions such as severe exposure environments or long intended working lives. Predictive models are required to cover such conditions.

Predictive models work by describing the assumed mechanism of deterioration by a mathematical model generally based on observations and measurements from existing structures, or sometimes by attempting to model processes at a fundamental level. Thus it has been found through measurements of chloride concentrations incrementally in concrete exposed to sea water or de-icing salts that the chloride ion concentration profile approximates to that resulting from diffusion.

Having decided on the mathematical description of the principle deterioration mechanism it is then necessary to determine suitable values for the input parameters often from observations on structures or laboratory tests. In some cases, however, particularly in the case of coefficients to take account of factors such as curing, values may be ascribed based on intuition and “engineering judgement”.

Predictive models can be used to estimate service life although, of course, the prediction will be inherently incorrect because of the assumptions involved. It is most important to bear this in mind when using predictive models and not to believe too strongly in a definitive output particularly for deterministic models which provide the output as a single value. Probabilistic models, however, attempt to take account of the potential variability of some or all of the input parameters by considering their statistical distribution. As a result, deterministic models can be simple spreadsheets but probabilistic models require considerably more detailed input.

Most usefully, predictive modelling can be used to compare the effect of factors such as:

- Different concrete compositions
- Cover to reinforcement
- Severity of the environment

Before using predictive modelling, or the output from such, it is important to realise and take account of the limitations involved, including:

Each model is based on a single deterioration mechanism, such as chloride ingress, carbonation, freezing and thawing or chemical attack. In practice more than one mechanism may be present

In the case of chloride ingress, it is generally assumed that the ingress is by diffusion whereas, in practice, ingress may also occur through capillary absorption, permeation or wick action. Modelling of these other processes is extremely difficult as conditions will vary widely between different concrete properties and exposure conditions (e.g. frequency of contact with salt-laden waters).

The quality of the output will depend, not only on the applicability of the chosen mechanism, but on the quality of the input data. In the case of chloride ingress models it is generally necessary to input values for the chloride diffusion coefficient of the concrete, an ageing factor to take account of the meta-stable state of concrete in prolonged contact with water, a surface chloride concentration and a threshold chloride level for initiation of corrosion. Data from structures show wide variation in most, if not all, these parameters for any given concrete or exposure condition yet, with deterministic models, it is necessary to select a single value for each.

The user of the model, or of just the output, may not fully appreciate the practical variability of the input parameters and the magnitude of the effect of this variation on the output.

The output from deterministic models is a single value that can be expressed to any level of precision desired. This can give a false impression of the validity of the number.

There is a high degree of uncertainty over some of the input parameters, particularly surface chloride concentration and threshold chloride levels.

Predictions of long-term behaviour are necessarily based on relatively short-term observations

9 Existing models

9.1 Theory

According to Glass [Glass, 2003] the movement of an ion through a medium, i.e. its flux, is described by a form of the Nernst-Planck equation:

$$J = -D_p(\partial C/\partial x) - \upsilon C(\partial \varphi/\partial x) + Cu \qquad \text{(Eqn 1)}$$

where:

J	Flux
D_p	Diffusion coefficient
C	Concentration
υ	Ionic mobility
φ	Electric potential
u	Average mass velocity
x	distance

This contains three elements to represent the components of the ionic movement due to diffusion, migration and water movement. Models to describe the movement of chloride ions into concrete simplify this by assuming that one of these mechanisms, diffusion, is dominant so only that element need be considered. The movement of chloride ions into concrete is then more simply described by Fick's 2nd law for non steady state diffusion (i.e. where the concentration at any depth is changing with time):

$$\partial C/\partial t = D(\partial^2 C/\partial^2 x) \qquad \text{(Eqn 2)}$$

This equation has been solved using what is known as an error function (also known as a probability integral):

$$C_{x,t} = C_s\,[1-\mathrm{erf}(x/(2\sqrt{DT}))] \qquad \text{(Eqn 3)}$$

where:

$C_{x,t}$	Concentration of chloride at depth 'x' from the surface, at time 't'
C_s	Concentration of chloride at the concrete surface
x	Depth from the surface
D	Diffusion coefficient
t	Time
erf()	Error function

The chloride profile in concrete at any time is then defined by the surface chloride concentration, the chloride diffusion coefficient and the background chloride content as shown in Figure 9.1.

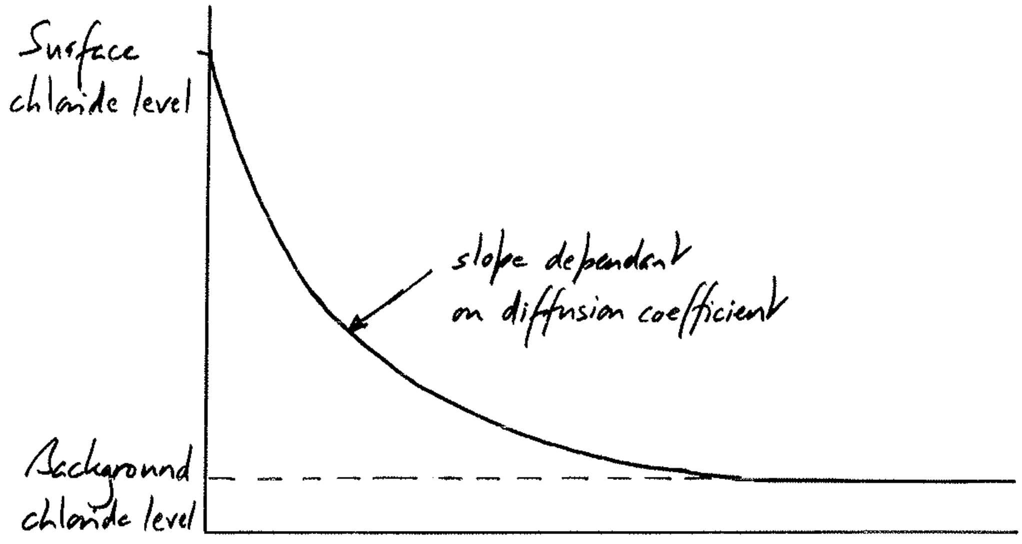


Figure 9.1 Chloride ingress profile in concrete

Nevertheless, this solution necessarily makes certain assumptions:

- The surface chloride concentration is constant
- The diffusion coefficient is constant
- The concentration at time, t = 0, is known (i.e. the background chloride level)
- The concrete is inert, isotropic and homogeneous

The chloride diffusion coefficient describes the rate at which the chloride ions can pass through the pore structure under the influence of a concentration difference. The assumption that concrete is inert is, however, untrue. The pore structure in concrete will change with the increasing degree of hydration of the cementitious matrix. This may occur over several years and to an extent which differs with the type of cement. Also, the matrix will chemically bind some of the ingressing chloride to an extent again dependant on the type of cement. This means that another of the above assumptions is also untrue, that the diffusion coefficient is constant. It is thus necessary to use an apparent or effective diffusion coefficient which changes with time to address the effects of chloride binding and increased hydration. This is typically obtained using the following expression which incorporates an age factor which will depend on the particular type of cement used:

$$D_{ca(t)} = D_{ca(tm)} \left({}^{t}/_{tm}\right)^{n} \qquad \text{(Eqn 4)}$$

where:

$D_{ca(tm)}$	Value of diffusion coefficient D_{ca} at time t_m
n	Age factor

Empirical models use information from measured chloride profiles in existing structures to provide the basis for calculation of values for effective diffusion coefficients. It is interesting to note that the shape of chloride profile in concrete is reported to be often the same even when the ingress was not solely through diffusion.

Substitution of the effective diffusion coefficient thus allows prediction of the chloride concentration at a given depth in the concrete after a given time. This can be used to create a chloride ion ingress profile at a given age or, by assuming a chloride level at which corrosion is expected to commence, to predict the length of the so-called initiation period.

The ingress of chloride and the initiation of corrosion depend on a number of factors that may only be assessed with substantial uncertainty. This overall uncertainty arises from uncertainties and variability in the material characteristics (diffusion coefficient and age factor), the environmental conditions (surface chloride concentration and rate of build-up) and the mechanism of corrosion initiation itself.

This theory only covers the time to initiation of corrosion, at which stage no deterioration has occurred as a result of chloride ingress. Nevertheless, some models also incorporate estimation of the time from initiation of corrosion until some outward sign of deterioration, such as cracking of the concrete cover. The uncertainty involved in such prediction is probably even greater than that for initiation and is not considered in depth within this report.

9.2 Readily available models

Three models are most prominent:

- Life-365
- AGEDDCA
- DuraCrete

Of these only the first two can readily be obtained. The last was available on the internet for a limited period but the DuraCrete website no longer exists. For this reason it is excluded from the main consideration within this report but is described in principle as it contains interesting and important differences from the other two. Life-365 and AGEDDCA are both classed as deterministic as they use set values for each of the input parameters (surface chloride concentration, diffusion coefficient and age factor). DuraCrete takes a probabilistic approach whereby the input parameters are considered as probability functions based on practical experience. The fib Model Code for Service Life Design [fib, 2004] is also discussed as this employs the principles developed by the DuraCrete project.

9.3 Life-365

Life-365 was initially produced for a consortium of two construction products manufacturers and the Silica Fume Association. It was subsequently adapted into the current, second version for a new consortium comprising the Concrete Corrosion Inhibitors Association, the National Ready Mix Concrete Association, the Slag Cement Association, and the Silica Fume Association. The model is intended to provide “guidance in planning and designing concrete construction exposed to chlorides in service” and is aimed at “individuals who are competent to evaluate the significance and limitations of their content and recommendations.” It was produced in response to the need for a “standard model” identified by the Strategic Development Council of the American Concrete Institute (ACI). Development of the model was to be under the jurisdiction of ACI Committee 365 “Service life prediction”. It should, however, be noted that the current Life-365 model is the result of work initiated by the above consortiums and does not yet appear to carry the endorsement of ACI Committee 365.

The user manual points out that the model has many limitations due to the number of assumptions and simplifications that have been made to deal with some of the more complex phenomena and where there is insufficient knowledge to permit a more rigorous analysis.

The model comprises four main elements:

- Prediction of the initiation period from construction to the onset of corrosion
- Prediction of the propagation period from the initiation of corrosion to the time at which it reaches an unacceptable level
- Determination of a repair schedule after first repair, and

- Estimation of life-cycle costs

The last two of these four steps do not appear in most models.

9.3.1 Prediction of the initiation period

The initiation period is defined as the time taken for the chlorides to accumulate in sufficient quantity at the depth of the embedded steel to initiate corrosion. The model uses a simplified approach assuming Fickean diffusion as the chloride ingress mechanism. It does not take account of more complicated effects such as wetting and drying, whereby chlorides are transported by capillary absorption into unsaturated pores, and chloride binding within the cementitious matrix.

The model is based on Fick's 2nd law (equation 5), as used in most common chloride ingress models, but uses finite difference for calculation, with the value of the diffusion coefficient being modified at every step.

$$dC/dt = D \, . \, d^2C/dx^2 \qquad \text{(Eqn 5)}$$

where:

C	chloride content
D	apparent diffusion coefficient
x	depth from the exposed surface
t	time

9.3.2 The apparent diffusion coefficient and the effect of time and temperature

The modifications of the diffusion coefficient in equations 6 and 7 are made to account for the effects of time and temperature, respectively, but the time effect is halted after 25 years.

$$D(t) = D_{ref} \, (t_{ref} /t)^m \qquad \text{(Eqn 6)}$$

where:

D(t)	diffusion coefficient at time, t
D_{ref}	diffusion coefficient at reference time (28 days)
m	constant

$$D(T) = D_{ref} \, . \, \exp[U/R(1/T_{ref} - 1/T)] \qquad \text{(Eqn 7)}$$

where:

D(T)	diffusion coefficient at time, t, and temperature, T
D_{ref}	diffusion coefficient at reference time & temperature (293K = 20°C)
U	activation energy of the diffusion process (35000J/mol)
R	gas constant
T	absolute temperature

The effect of temperature is based on monthly average temperatures for the location under consideration. This data only covers the USA and Canada so the user will need to input their own data to cover other locations, unless one of the preset profiles is felt to be applicable to another region. The diffusion coefficient is based on the 28-day value for a standard Portland cement concrete with no special protective measures applied, assuming the following properties:

D_{ref} $1 \times 10^{(-12.06 + 2.40w/c)}$ m²/s (Eqn 8)

m 0.20

C_t 0.05% by mass of concrete

This is then varied to take account of the type and proportion of cementitious materials in the actual concrete under consideration via the age factor as described in 9.3.7.

9.3.3 The surface chloride level

The surface chloride content and rate of its build-up is determined from the type of structure (e.g. bridge deck, car park), the type of exposure (e.g. marine, de-icing salts), as shown in Table 9.1, and the selected geographic location (for the temperature profile).

Exposure condition	Surface chloride level	
	(by wt of concrete)	(by wt of cement)[1]
Marine splash zone	0.8% instantaneously	5.0% instantaneously
Marine spray zone	1.0% over 10 years	6.25% over 10 years
Up to 800m from the sea	0.6% over 15 years	3.75% over 15 years
800m – 1.5km from the sea	0.6% over 30 years	3.75% over 30 years
Parking structures	0.8-1.0%[2]	5.0-6.25%[2]
Urban bridge decks[3]	0.68-0.85%[4]	4.25-5.35%[4]
Rural bridge decks[3]	0.56-0.70%[5]	3.50-4.40%[5]

[1] Not given in Life-365 but approximate values calculated here assuming concrete density 2350kg/m³ and cement content of 375kg/m³

[2] At a rate of <0.015 - >0.08% wt/year depending on geographic location

[3] assumes no asphalt surface course

[4] 85% of the rate for parking structures

[5] 70% of the rate for parking structures

Table 9.1 Surface chloride levels used in Life-365 depending on exposure condition

9.3.4 User input parameters

The input data required from the user is quite straightforward:

- free water/cement ratio
- the type and proportion of cementitious materials
- the geographic location (or specific annual temperature data)
- the type of structure
- the nature of exposure (marine, de-icing)
- the minimum depth of cover

- details of any other protection measures (corrosion inhibitors, type of steel and coatings, use of membranes or sealers).

9.3.5 The chloride threshold level

The chloride threshold level is assumed to be independent of the cement type and has been taken as the commonly used value of 0.05% by mass of concrete. Higher values are used when the concrete contains corrosion inhibitors or stainless steel reinforcement (see below) but not when epoxy-coated reinforcement is employed (the influence being assumed to be on the propagation period rather than the initiation period).

9.3.6 The effect of free water/cement ratio

The effect of free water/cement ratio on the diffusion coefficient is said to be based on a large database of bulk diffusion tests. Any effect of free water/cement ratio on the age factor is not currently taken into account although it is said that this may be included in a future version.

9.3.7 The age factor

The" diffusion decay index" or age factor, m, in equation 6 varies in the range 0.2-0.6 and depends on the particulars of the mix composition is said to be based on data from the University of Toronto and other published data. The relationship used is:

$$m = 0.2 + 0.4(\%FA/50 + \%SG/70) \qquad \text{(Eqn 9)}$$

where:

FA	fly ash
SG	blastfurnace slag

9.3.8 The effect of cement type

Neither blastfurnace slag nor fly ash is assumed to affect either the basic 28-day diffusion coefficient or the chloride threshold level. Their effect is assumed to be on the constant, m, as shown above for slag contents of up to 70% and fly ash levels up to 50%. Interestingly this expression is capable of coping with triple blends of Portland cement, slag and fly ash although it is not clear whether this was intended or, indeed, there is any background data on such cement types. The incorporation of silica fume (up to 15% by mass of cement) is treated differently than blastfurnace slag or fly ash and results in the reduction of the diffusion coefficient but not the constant, m. The effect attributed to silica fume is large with just 5% silica fume resulting in an almost 60% reduction in the initial diffusion coefficient, and an approx. 80% reduction at 10% silica fume. These factors are said to be based on bulk diffusion data.

9.3.9 Alternative protection strategies

The effect of corrosion inhibitors is said to be based on the documented performance of two particular products, one being based on calcium nitrite, the other based on “amines and esters”. Other types of corrosion inhibitor are not currently covered. The treatment of the two types of admixture differs significantly. The calcium nitrite product is assumed simply to increase the chloride threshold level by an amount dependant on the dosage level. The “amines and esters” product, on the other hand, is assumed to increase the chloride threshold level, reduce the initial diffusion coefficient by 10% and halve the rate of build-up of the surface chloride concentration due to modification of the pore structure. This effect on the pore structure is said to reduce sorptivity and diffusivity.

The influence of membranes and sealers is introduced simply through an increase in the build-up time for surface chloride concentration. Membranes are assumed to deteriorate over a default 20 year period. On application they are assumed to be 100% efficient, allowing no chloride build-up. Their effectiveness is assumed to reduce linearly over their life to the point where the normal rate of surface chloride build-up is unaffected. The user is able to input different values for both initial efficiency and lifetime of the membrane and to reapply the membrane at any stage. The treatment of sealers is similar although the default lifetime is just 5 years; they too can be repeatedly reapplied within the model.

9.3.10 Implicit assumptions

The following assumptions are said to be implicit within the model:

- the concrete is homogeneous
- the surface chloride concentration is constant over the surfaces under consideration at any given time
- properties of the concrete remain constant within each time increment of the finite difference modelling process, and
- the diffusion coefficient is constant over the whole depth of the element under consideration

9.3.11 Probabilistic prediction of initiation period

The Life-365 model also includes probabilistic calculations whereby each of the five input parameters (basic diffusion coefficient, surface chloride concentration, the age factor, chloride threshold level and cover depth) is varied sequentially by 10% from the input value and compared with the result for the basic input data. This comparison involves the estimation of the derivative of the corrosion initiation time with respect to each of the five input parameters to determine the sensitivity of the output to precision of the input. This is then used to determine a single indicator of variability, said to be similar to a standard deviation.

9.3.12 Prediction of the propagation period

No mathematical modelling of the propagation period is attempted within Life-365. A single default value of 6 years is used for all but epoxy-coated reinforcement where it is increased to 20 years. It is, however, possible for the user to select different values.

9.3.13 Determination of a repair schedule

The time to first repair is determined by the model as the sum of the initiation and propagation periods. Nevertheless, the extent of this repair and any subsequent repairs, expressed as the as a percentage of the total area, and the cost needs to be input by the user.

9.3.14 Estimation of life-cycle costs

Total life-cycle costs are calculated as the sum of the initial cost and the future repair costs over the defined service life of the structure. The initial construction costs are simply the sum of the cost of the concrete supplied, the reinforcement and any applied surface protection. The cost of the actual construction process, e.g. formwork, labour, is not included. Future repair costs are based on a “present worth” (sometimes also known as “net present value”) basis taking specific account of price inflation and a discount rate to cover other factors affecting the cost of capital expenditure in the future compared to the same expenditure at the present time.

9.4 AGEDDCA

The AGEDDCA model is presented in Concrete Society Technical Report 61 [The Concrete Society, 2004] and included as a spreadsheet on a CD accompanying the report. It was developed in the UK under a Government and industry partnership funding scheme. The model can be used in two versions. The basic version uses input data derived from the mix constituents and proportions. A more advanced version allows direct input of apparent diffusion coefficient at a defined age and allows the inclusion of additional protective measures such as corrosion inhibitors, controlled permeability formwork, permanent formwork and corrosion-resistant reinforcement.

The model is based on a two-stage deterioration process comprising the initiation period and the propagation period. The initiation period is defined as the time for the level of chlorides at the depth of the reinforcement to reach the threshold level at which the passivating oxide layer on the reinforcement is broken down. The subsequent propagation period is the time from the onset of corrosion to the point at which cracking of the concrete occurs, or, the point at which a specific amount of corrosion chosen by the user to represent the point at which unacceptable damage, such as spalling, is likely to occur.

The background information provided with the model acknowledges that an ideal predictive model for chloride ingress under non-saturated conditions, such as the splash zone of marine structures, should include an initial absorption component followed by further ingress by diffusion. This is, however, not adopted because of the uncertainty in the knowledge of the initial moisture state of the concrete when first in contact with chlorides, the variation in sorptivity of the concrete with depth and the variation in diffusion coefficient with depth. The adopted approach uses an age-dependant apparent diffusion coefficient whereby the value is relatively high to start with but decreases with time to an extent dependent on the cement type.

The general form of the model is reported by its author to be validated by measurements of chloride ion profiles in actual structures and test specimens even in exposure conditions where the concrete is not saturated and diffusion is known to be not the only ingress mechanism.

The age-related apparent diffusion coefficient is then used instead of the constant diffusion coefficient in the error function solution of Fick's 2nd law of diffusion to give:

$$C_{x,} = C_{sn}\,[1\text{-erf}(x/(2\sqrt{D_{ca}}\,t))] \qquad \text{(Eqn 10)}$$

where:

$C_{x,}$	Chloride content at depth *x* at time *t*
C_{sn}	Notional surface level of chloride
x	Depth from the surface
D	Diffusion coefficient
erf()	Error function

Values of surface chloride level and apparent diffusion coefficient have been derived from best-fit analyses of chloride ion profiles at different times. The reduction in apparent diffusion coefficient is said to be on a log-log relationship with time which results in the following expression:

$$D_{ca(t)} = D_{ca(tm)}\,({}^{t}/_{tm})^{n} \qquad \text{(Eqn 11)}$$

where:

$D_{ca(tm)}$	Value of diffusion coefficient D_{ca} at time t_m, and
n	Age factor

The model requires the following input parameters:

- The notional surface chloride level, C_{sn}
- The apparent diffusion coefficient $D_{ca(tm)}$ at time t_m
- The age factor, *n*
- The chloride threshold level, C_t

9.4.1 The notional surface chloride level, C_{sn}

It is noted that the notional surface chloride level required by the model is not the actual chloride level at the concrete surface but the value determined by curve fitting to measured incremental chloride contents, and extrapolation of the curve to the concrete surface. The time taken for the surface chloride to rise to a constant level is considered by the model as insignificant and a constant value is assumed from the time of initial exposure to chlorides. This is likely to be quite an onerous assumption in exposure conditions where the concrete is not in permanent contact with the source of chlorides.

The model uses a single default value of 0.5% by mass of concrete for surface chloride level regardless of exposure condition and cement type, except where modified by certain alternative protection strategies (e.g. coating) as described below. The use of a single value default is surprising as the supporting information in Concrete Society Technical Report No. 61 says that typical average values for predictive purposes, and their 95% characteristic values, in brackets, are:

PC concrete	0.36% (0.75%) by mass of concrete
Fly ash, ggbs, silica fume concrete	0.51% (0.90%) by mass of concrete

The 95% characteristic values are suggested as possibly more appropriate for critical elements or structures because of the high variability of surface levels determined from structures but, as stated above, not used in the model.

Alternatively the user can directly input other values.

9.4.2 The apparent diffusion coefficient $D_{ca(tm)}$ at time t_m

Values of apparent diffusion coefficient have been determined as the values of diffusion coefficient that give the best fit to observed chloride profiles. The apparent diffusion coefficient corresponds to the value of the constant diffusion coefficient in Fick's 2nd law that would have resulted in the same chloride profile after the same exposure period. It is thus equivalent to an averaged, or integrated, value over the period in question. The true diffusion coefficient decreases with time and this is taken into account within the model by the incorporation of an age factor.

The input value for apparent diffusion coefficient can be either from measurement of chloride profile in similar concrete in similar exposure conditions, or an estimate based on the free water/cement ratio and cement type of the concrete as described below:

9.4.3 The age factor, *n*

The age factor is defined as the slope of the log-log relationship between apparent diffusion coefficient and time. The model assumes that the diffusion coefficient continues to decrease without time limit. The following typical values used in the model are said to be based on comprehensive reviews of published data:

Portland cement	-0.264
Fly ash cement	-0.699
GGBS cement	-0.621

It is, however, acknowledged that adjustment of these values may be possible as and when more information becomes available. Selection of a suitable value for silica fume concrete is reported to have been more difficult because of an apparent dependence also on the free water/cement ratio of the concrete. The following expression is used to determine the age factor for silica fume concrete:

$$n = -1 + 1.1(\text{w/c}) \qquad \text{(Eqn 12)}$$

The model is able to use the age factor to calculate the apparent diffusion coefficient at any age from the apparent diffusion coefficient determined from measured in situ chloride profiles from any age of structure using equation 12.

9.4.4 The effect of free water/cement ratio

The following relationships are used to relate the selected free water/cement ratio to a database of diffusion coefficients at an age of 20 years:

Portland cement $\log D_{ca(20)} = -12.926 + 1.999(w/c)$ (Eqn 13)

Fly ash and ggbs cement $\log D_{ca(20)} = -13.325 + 1.409(w/c)$ (Eqn 14)

Silica fume cement $\log D_{ca(20)} = -13.800 + 3.100(w/c)$ (Eqn 15)

The 20 year diffusion coefficients were obtained by normalisation to 20 years of data from various sources for a wide range of concretes obtained over a range of ages from a few weeks up to 10 years. The basic data covered a range of water/cement ratios from 0.33 to 0.70 and is presented graphically within Concrete Society Technical Report 61 [The Concrete Society, 2004]. It can be seen to have high variability and the report points out that measured values of apparent diffusion coefficient may vary by more than an order of magnitude for the same concrete within the same structure.

9.4.5 The effect of cement type

It has been seen above that the broad cement type (Portland cement, fly ash, ggbs and silica fume cements) is taken into account in selection of both the age factor and the effect of water/cement ratio. The magnitude of the effect of the use of cements containing secondary cementitious materials (fly ash, ggbs and silica fume) will, however, depend on the proportion in which these materials are incorporated.

The best-fit curve to results from long-term exposure tests of concrete made with cement containing varying proportions of ggbs is represented by the equation:

$$Dca(ggbs) = Dca(PC) \cdot (Ap^2 + Bp + C) \qquad \text{(Eqn 16)}$$

Where:

Dca(ggbs)	apparent diffusion coefficient of ggbs concrete
Dca(PC)	apparent diffusion coefficient of Portland cement concrete
P	proportion of ggbs (% by mass of total cement)
A, B & C	constants

Similar relationships are said to have been found for fly and silica fume but with different constants reflecting their different degrees of effectiveness. The values for the various constants are given in Table 9.2 below:

Constant Cement type	A	B	C
ggbs	0.0229	-2.9921	100
Fly ash	0.064	-4.6579	100
Silica fume	0.191	-7.5644	100

Table 9.2 Values for constants used in AGEDDCA model

9.4.6 The chloride threshold level, C_t

The chloride threshold level is a function of the cement content within the concrete whereas chloride content is determined as the proportion by weight of the concrete sample. Default values of 2350 kg/m³ for density and 350 kg/m³ for cement content have been assumed to convert values obtained from structures where these values were not known.

Experimental investigations indicate that the chloride threshold level in practice can vary depending on a number of factors including the exposure conditions, water/cement ratio and cement type.

The model admits to a high degree of uncertainty in chloride threshold levels but uses values and functions, given in Table 9.3, which take account of cement type but not exposure condition or water/cement ratio:

Cement type	Proportion	Chloride threshold level (by mass of cement)
Portland cement		0.4%
Fly ash	0-10%	0.4%
	10-50%	0.4[1-0.010(%fa – 10)]
ggbs	0-20%	0.4%
	20-80%	0.4[1-0.005(%ggbs – 20)]
Silica fume	0-10%	0.4(1-0.025%sf)

Table 9.3 Chloride threshold levels used in AGEDDCA model

The user is able to over-ride the above function and input a particular preferred value for chloride threshold level if required.

9.4.7 Corrosion rate

The model uses the following functions to determine the rate of corrosion of reinforcement once the chloride threshold level has been reached, depending on exposure condition and chloride concentration at the depth of the bar:

Wet, rarely dry $CR = 0.84e^{1.45Cx}$ (Eqn 17)

Airborne seawater and cyclic $CR = 0.54e^{1.45Cx}$ (Eqn 18)

Tidal zone $CR = 0.46e^{1.45Cx}$ (Eqn 19)

where:

CR	Corrosion rate
Cx	Chloride concentration at the depth of the reinforcement

Corrosion is assumed to be negligible below a corrosion rate of 1.2µm/year. The model does not take account of other identified influential factors such as climatic conditions (e.g. temperature, humidity, time of wetness).

The increasing chloride concentration at the depth of the reinforcement due to continued ingress is taken into account by calculation of the corrosion rate on an incremental time basis, using the average chloride concentration over that time period.

9.4.8 Time to cracking

The amount of corrosion required to cause cracking, based on the findings of the Brite-Euram DuraCrete project, is assumed to be given by the function:

$$X_c = 83.8 + 7.4\,(c/\varnothing) - 22.6f_{st} \qquad \text{(Eqn 20)}$$

where:

X_c	loss of steel (µm)
c	cover to reinforcement (mm)

$\varnothing$	diameter of reinforcement bar (mm)
f_{st}	splitting tensile strength of the concrete (MPa)

and where the splitting tensile strength can be obtained from the characteristic compressive strength using the following expression:

$$f_{st} = 0.108\{f_{cu}\,[1 + 0.399\,\log(f_{cu})] - 5.334\}^{0.722} \qquad \text{(Eqn 21)}$$

The time to cracking thus depends upon the corrosion rate and size of the reinforcing bar, which determine the amount of corrosion product, and the amount of cover concrete and its strength, which determine its resistance to the forces generated by the formation of the corrosion products. The amount of corrosion required to cause cracking reduces with increasing concrete strength because the resistance to cracking is said to be strain-dependent not strength-dependent. Higher strength concrete has lower porosity and less ability to absorb any of the corrosion products into its microstructure so will crack at a lower degree of corrosion than a lower strength, more porous concrete than can absorb some of the corrosion products.

9.4.9 The effect of temperature

The ambient temperature will affect both the rate of chloride ingress and the rate of any consequent reinforcement corrosion. The influence on the rate of chloride ingress is given by the expression:

$$D_2 = D_1\,(T_2/T_1)\,e^{q(1/T1\ -\ 1/T2)} \qquad \text{(Eqn 22)}$$

where:

D_1	apparent diffusion coefficient at temperature T_1
D_2	apparent diffusion coefficient at temperature T_2
q	w/c ratio factor

and where:

$$q = 1000[-52.5(w/c) + 41.75(w/c) - 2.3] \qquad \text{(Eqn 23)}$$

It is noted that some research has indicated a lack of temperature dependence for bulk diffusion tests as opposed to the behaviour in steady-state diffusion cells. The model thus allows the user the option to remove the temperature dependence and just employ an assumption of 20℃.

The effect of temperature on corrosion rate is not taken into account but the effect on the corrosion on chloride threshold level, said to be based on limited data, is given for the range 0-40℃ by the expression:

$$C_{t(T)} = C_{t(20)}\,[1 - 0.0333\,(T - 20)] \qquad \text{(Eqn 24)}$$

9.4.10 Alternative protection strategies

The model also allows account to be taken of other measures introduced to increase the resistance to chloride-induced corrosion of reinforcement. These measures comprise:

- Integral waterproofers – the surface chloride level is reduced by 10% for a "normal" waterproofer and 20% for a "high range" waterproofer
- Corrosion inhibitors – the chloride threshold is increased by amount dependent on the amount of inhibitor added according to the expression:

  $$C_{tI} = C_{t0} + 0.06\,I\,f_i \qquad \text{(Eqn 25)}$$

  where:

C_{tI}	chloride threshold level with inhibitor

C_{t0}	chloride threshold level without inhibitor
I	inhibitor dosage (l/m^3)
f_i	safety factor

The expression for corrosion rate is also modified to take account of the raised threshold level.

- Type of reinforcement – the chloride threshold level is increased to the following values:

Galvanised steel	1.0% Cl by wt of cement
Stainless steel	3.0% Cl by wt of cement

The value for stainless steel only applies to grade 316 or better. For galvanised steel it is assumed that corrosion will be rapid once initiated so the time from initiation to cracking is taken as zero.

- Controlled permeability formwork – the effect of controlled permeability formwork is taken as equivalent to increasing the depth of cover and a 10% reduction in the surface chloride level. The equivalent cover depth is given by the expression:

$$d_{equiv} = d_{cpf} \sqrt{(D_c/D_{cpf})} \qquad \text{(Eqn 26)}$$

where:

d_{equiv}	equivalent depth of unaffected concrete
D_c	normal effective diffusion coefficient
D_{cpf}	effective diffusion coefficient for the affected zone, taken as $0.45D_c$
d_{cpf} √	depth of zone affected by the CPF, given by: $d_{cpf} = 1 + 40(w/c)$ (Eqn 27)

- Permanent GRC formwork – the effect of a surface layer of glass reinforced concrete (GRC) is also taken as equivalent to an increase in the effective cover to the reinforcement. The extent of the increased cover is dependent on the thickness of the GRC layer and is effective diffusion coefficient. Where the effective diffusion coefficient for GRC is not known it can be calculated from its water/cement ratio using equation 28.

$$d_{equiv} = d_{grc} \sqrt{(D_c/D_{grc})} \qquad \text{(Eqn 28)}$$

where:

d_{equiv}	equivalent depth of unaffected concrete
D_c	effective diffusion coefficient for the normal concrete
D_{grc}	effective diffusion coefficient for the GRC
d_{grc}	thickness of GRC layer

- Coatings and surface treatment – an adjustment is made to the surface chloride level and it is assumed that the coating or treatment is maintained throughout the required working life of the structure, and that the effectiveness does not reduce with time.

9.4.11 User input parameters

- cement type and content
- water/cement ratio
- strength grade
- surface chloride level
- exposure condition
- chloride threshold level
- corrosion rate threshold
- bar diameter and cover
- use of integral waterproofer
- addition of calcium nitrite inhibitor
- type of reinforcement
- use of controlled permeability formwork
- use of GRC permanent formwork
- application of coating or surface treatment

9.4.12 Summary

Chloride ingress is predicted from the user input parameters describing concrete quality and exposure conditions, using background data within the model, and determines the time to initiation of corrosion. The time to cracking is then predicted from the corrosion rate and critical amount of corrosion using the user input parameters of bar size, cover and concrete strength.

The principal output is tabulated values of three serviceability limit states:

- time for chloride to reach the threshold level
- time for the corrosion rate to become significant
- time for sufficient corrosion to cause cracking

9.5 Comparison of the two principal models

9.5.1 Background theory

Both the models are based on the assumption that the chloride ingress is in accordance with Fick’s 2nd law of diffusion although their mathematical solutions are different. AGEDDCA uses an error function approach whereas Life-365 uses a finite difference approach.

9.5.2 Range of exposure conditions

The AGEDDCA model restricts itself to three nominal marine exposure conditions based on the European Standard, EN 206-1, sea water exposure classes XS1-3:

- Wet, rarely dry
- Splash zone
- Tidal

Nevertheless, in actuality, the model makes no distinction between the three conditions.

Life-365 makes actual distinction between four marine and coastal exposure conditions:

- Splash zone
- Spray zone

- Up to 800m from the sea
- 800-1500m from the sea
- It also adds three de-icing salt exposure conditions:
- Parking structures
- Urban bridges
- Rural bridges

9.5.3 Surface chloride concentration

Life-365 and AGEDDCA vary considerably with respect to the surface chloride level and the time taken to build up to the maximum value as shown in Table 9.4:

Exposure condition	Life-365	AGEDDCA
Marine splash zone	0.8% instantaneously	0.5% instantaneously
Marine spray zone	1.0% over 10 years	0.5% instantaneously
Up to 800m from the sea	0.6% over 15 years	0.5% instantaneously
800m – 1.5km from the sea	0.6% over 30 years	0.5% instantaneously

Table 9.4 Surface chloride levels used in different models (by mass of concrete)

9.5.4 Determination of apparent diffusion coefficient

Both Life-365 and AGEDDCA allow for a reduction in the value of the effective diffusion coefficient with time. Nevertheless, Life-365 draws a halt to this reduction after 25 years whilst AGEDDCA allows it to reduce over the whole of the life of the structure although continued improvement in pore structure and further chloride binding would seem unlikely much after about 25 years. This is likely to have a large effect on structures with long design lives such as 100-120 years.

9.5.5 The effect of temperature

Both Life-365 and AGEDDCA have a facility to incorporate the effect of temperature. In Life-365 this is an impact on the rate of diffusion of chlorides and, perhaps surprisingly, not from a lowering of the chloride threshold level. In AGEDDCA the effect is in both increasing the rate of diffusion and lowering the chloride threshold level. Nevertheless, Concrete Society Technical Report 61 makes reference to a paper that concludes temperature has no effect on the diffusion rate; AGEDDCA allows the user to disable this function.

9.5.6 Threshold chloride level

Both Life-365 and AGEDDCA use a single value for the threshold chloride level although AGEDDCA gives reducing values for increasing levels of fly ash, ggbs and silica fume and increasing temperature. The basic value in both models is similar although Life-365 uses a value based on % by weight of concrete whereas AGEDDCA uses a value expressed as % by weight of cement and is thus sensitive to the cement content of the concrete.

9.5.7 Other protective measures

Life-365 allows for the use of corrosion inhibitors, membranes and surface sealers. AGEDDCA covers a wider range, adding integral waterproofers, controlled permeability formwork and permanent formwork.

9.5.8 Effect of cement type

Life-365 starts with the same basic diffusion coefficient at 28 days regardless of cement type but uses different ageing factors. AGEDDCA also uses different age factors for different cement types.

	Portland cement	30% Fly ash cement	50% ggbs cement
Life-365	0.2	0.44	0.486
AGEDDCA	-0.264	-0.699	-0.621

Table 9.5 Effect of cement type on the ageing factor

9.6 DuraCrete

The DuraCrete model differs from Life-365, AGEDDCA and most other chloride ingress models because it is reliability based, rather than deterministic. That is, the variability of each of the input parameters is specifically considered rather than using single assumed values and the output is expressed as a probability rather than an absolute value. In other respects, the model is essentially similar to the AGEDDCA model in the use of the error function solution of Fick's 2nd law to describe the chloride ingress, and the use of an apparent chloride diffusion value coupled with an age-factor.

This approach recognizes that both the environmental load (i.e. its severity) and the resistance of the structure, in practice, will be variable rather than fixed at a single level. In a deterministic approach single values are ascribed and failure is defined as when the environmental load exceeds the resistance of the structure. Consideration of the environment and the concrete as variable means that a single failure point cannot be defined and, instead, must be expressed in terms of the probability of occurrence of the condition defined as failure. Probabilistic design requires that an acceptable probability of failure must be defined. Reports from the DuraCrete programme are no longer readily available so the following description has been taken from one of a number of similar technical papers on reliability-based durability design, although not specific to DuraCrete [Gehlen & Schiessel, 1999].

The reliability of an element or structure, Z, is given by the function:

$$Z = R - S \qquad \text{(Eqn 29)}$$

where:

R	resistance of the element or structure, and
S	environmental load (severity)

Assuming both the environmental load and the resistance of the structure are normally distributed means the reliability will also follow a normal distribution. Knowing or assuming values for the average, μ, and standard deviation, σ, of R and S allows the average and standard deviation to be determined for the reliability:

$$\mu_Z = \mu_R - \mu_S \qquad \text{(Eqn 30)}$$

$$\sigma_Z = \sqrt{(\sigma_R^2 + \sigma_S^2)} \qquad \text{(Eqn 31)}$$

This then allows the calculation of the reliability index, β:

$$\beta = \mu_Z / \sigma_Z \qquad \text{(Eqn 32)}$$

The reliability index can be used for design with the requirement that the calculated reliability index is not less than a prescribed minimum value. That value will depend on the design limit state, such as a serviceability limit state (SLS) of corrosion initiation or spalling or an ultimate limit state (ULS) defined by collapse. For a normally distributed reliability function, the failure probability, p_f, and the reliability index are related as given in Table 9.6 below for different limit states and reliability classes taken from the fib model code for service life design [fib, 2006].

The above description is simplified and in practice there are multiple variables, non-linear functions and non-normal distributions of variables which require the use of sophisticated computer programs.

Generally, design by this method will only consider a serviceability limit state which in the DuraCrete approach is usually chosen as corrosion initiation. The DuraCrete model is then used to determine the required material resistance against chloride ingress to ensure with this defined level of probability corrosion initiation will not occur before the end of the required working life of the structure.

Reliability Class[1]	Limit state	Reliability index, β	Failure probability p_f (%)
All	SLS - corrosion initiation	1.3	10
--	SLS - spalling[2]	2.0-3.0	2.3-0.1
RC1	ULS – collapse	3.7	0.01
RC2	ULS – collapse	4.2	0.001
RC3	ULS - collapse	4.4	0.0001

[1] from fib model code [fib, 2006]

[2] proposed value from [Gehlen & Schiessel, 1999]

Table 9.6 Failure probability and reliability index values from the fib model code

As mentioned above the actual model used for prediction of chloride ingress is based on the error function solution of Fick's 2nd law similar to that of AGEDDCA but introduces separate factors to cover the influence of the environment and curing on the diffusion coefficient. The duration of the initiation period is influenced by the following stochastic variable, each of which requires input of average values and standard deviation:

- Cover to reinforcement
- Chloride diffusion coefficient
- Chloride threshold level
- Age factor
- A factor to transform a measured test value of chloride diffusion coefficient into a practical value for a structure
- Environment factor
- Curing (execution) factor
- Surface chloride concentration

The DuraCrete model uses the above factors to calculate the reliability index at the end of the required working life for comparison with the minimum required level for the serviceability limit state.

A disadvantage with the DuraCrete approach is that there is no readily available database of input parameters and their statistical variation. Companies operating this approach tend to have their own databases, the quality of which will obviously determine the actual, rather than predicted, reliability of their designs. This makes it difficult for a client or independent body to check a design based on this approach.

9.7 FIB Model Code for Service Life Design

The fib Model Code for Service Life Design is contained in fib Bulletin 34 [fib, 2006] and is the latest stage in the development of work on design for durability within the two former organisations, CEB and FIP, that came together to form fib. Interestingly the task group responsible for this work includes individuals also involved in the development of both AGEDDCA and DuraCrete.

The model code considers the four deterioration mechanisms for which it believes broadly accepted models exist:

- carbonation-induced corrosion
- chloride-induced corrosion
- freeze/thaw attack without de-icing agents, and
- freeze/thaw attack with de-icing agents

It divides its approach between two strategies: service life design, and avoidance of deterioration. The latter is covered only briefly but is based on principle that deterioration can be prevented by approaches such as:

- physical isolation of the concrete surface from the exposure environment (e.g. cladding or membranes)
- use of non-reactive materials (e.g. stainless steel reinforcement)
- “separation of reactants” (e.g. maintaining the moisture level in the concrete below a critical level)
- suppression of harmful reaction (e.g. cathodic protection)

The use of stainless steel reinforcement is mentioned as an example of the use of non-reactive materials.

Service life design is covered at three levels:

- full probabilistic design
- partial factor design, and
- deemed-to-satisfy

9.7.1 Full probabilistic design

Full probabilistic service life design according to the model code is based on the following principles:

- models should be sufficiently validated to give realistic and representative results
- input parameters and their associated uncertainty should be quantifiable by tests, observations or experience
- reproducible and relevant test methods should be available to asses the environmental loads and materials resistance
- uncertainties associated with the models and test methods should be taken into account

The code identifies three levels of consequence to form the basis of different levels of reliability depending on the risk to human life or the economic, social or environmental impact, as shown in table 9.7.

Three reliability classes are identified, see Table 9.8, which give specific minimum values for the reliability index depending on the severity of the exposure class and whether the end point of the design is the serviceability limit state defined by depassivation (corrosion initiation) or the ultimate limit state defined as collapse, as discussed in the previous section. The reliability classes can also be associated with the consequence classes.

Consequence class	Description	Examples
CC3	High consequences for loss of human life; or very great economic, social or environmental consequences	Grandstands, public buildings where consequences of failure are high (e.g. concert halls)
CC2	Normal consequences for loss of human life; or considerable economic, social or environmental consequences	Residential and office buildings, public buildings where consequences of failure are medium (e.g. office building)
CC1	Low consequences for loss of human life; and small or negligible economic, social or environmental consequences	Agricultural buildings where people do not normally enter (e.g. storage buildings), green houses

Table 9.7 Consequence classes from the fib model code

Exposure class	Reliability class	Minimum reliability index, β	
		Serviceability Limit State	Ultimate Limit State
		Depassivation	Collapse
XD1 or XS1	RC1	1.3	3.7
XD2 or XS2	RC2	1.3	4.2
XD3 or XS3	RC3	1.3	4.4

Table 9.8 Reliability classes from the fib model code

The probabilistic design approach used in the model code for chloride-induced corrosion of uncracked concrete is the one developed by DuraCrete but slightly revised in the subsequent DARTS research project (Durable and Reliable Tunnel Structures). The critical, or threshold, chloride concentration is compared to the predicted chloride concentration at the depth of the reinforcement at a time, t, using the following equation based on Fick's 2nd law of diffusion:

$$C_{crit} = C_0 + (C_{S,\Delta x} - C_0) . [1- \mathrm{erf}((a - \Delta x)/2\sqrt{}(D_{app.C} . t))] \qquad \text{(Eqn 33)}$$

where:

C_{crit}	critical (threshold) chloride concentration
C_0	initial chloride content in concrete
$C_{S,\Delta x}$	chloride content at depth, Δx, and time, t
x	depth
a	concrete cover to reinforcement
Δx	depth of convection zone (see below)
$D_{app.C}$	apparent chloride diffusion coefficient
t	time
erf	error function

This expression is notable as, although based on Fick's 2nd law of diffusion, it attempts to take account of an outer "convection zone" where the ingress of chlorides is by mechanisms other than diffusion. The apparent diffusion coefficient is given by:

$$D_{app.C} = k_e . D_{RCM,0} . k_t . A(t) \qquad \text{(Eqn 34)}$$

Where k_e is an “environmental transfer variable” taking account of the difference between the ambient temperature experienced by the structure, and the laboratory temperature at which the chloride migration coefficient, $D_{RCM,0}$, will have been determined, and A(t) is an “ageing” coefficient to take account of the change in migration coefficient with time.

The chloride migration coefficient can be determined using the rapid chloride migration test NT Build 492 [Nordtest, 1999] although tabulated values, see Table 9.9, are given for various cement types and free water/cement ratios taken from the DART research programme.

Cement type	Free water/cement ratio[1]					
	0.35	0.40	0.45	0.50	0.55	0.66
CEM I	n/a	8.9×10^{-12}	10.0×10^{-12}	15.8×10^{-12}	19.7×10^{-12}	25.0×10^{-12}
CEM I + fly ash	n/a	5.6×10^{-12}	6.9×10^{-12}	9.0×10^{-12}	10.9×10^{-12}	14.9×10^{-12}
CEM I + silica fume	4.4×10^{-12}	4.8×10^{-12}	n/a	n/a	5.3×10^{-12}	n/a
CEM III/B[2]	n/a	1.4×10^{-12}	1.9×10^{-12}	2.8×10^{-12}	3.0×10^{-12}	3.4×10^{-12}
[1] w/(c + 0.5fa) for fly ash; w/(c + 2.0sf) for silica fume [2] Portland cement + >65% ggbs						

Table 9.9 Default values for chloride migration coefficient

Various other parameters, including critical chloride concentration, are expressed as variables and reference should be made to the model code for details.

9.7.2 Partial factor design

The partial factor approach uses the same deterioration models as the full probabilistic approach but employs assumed characteristic values for environmental loads (actions) and materials properties coupled with partial safety factors to take account of their variability and uncertainty:

$$F_d = \gamma_f \cdot F_{rep} \quad \text{(Eqn 35)}$$

where:

F_d	design value of environmental action
γ_f	partial safety factor
F_{rep}	representative or characteristic value of action

and:

$$R_d = R_k / \gamma_m \quad \text{(Eqn 36)}$$

where:

R_d	design value of material resistance
R_k	characteristic value of material resistance
γ_m	partial factor for material property

It is also possible to introduce another partial safety factor to take account of variability in the deterioration model itself:

$$R_d = R_k / \gamma_M = R_k / (\gamma_m \cdot \gamma_{Rd}) \quad \text{(Eqn 37)}$$

where:

γ_{Rd}	partial factor for uncertainty of the deterioration or resistance model and geometric deviations
$\gamma_M = \gamma_m \cdot \gamma_{Rd}$	partial factor for material property taking account of model uncertainty and dimensional variations

It is intended that numerical values for the partial factors should be obtained either by statistical evaluation of experimental and field data, or on the basis of long term experience.

9.7.3 Deemed-to-satisfy

The deemed-to-satisfy approach relates to the common existing practice of minimum specification requirements for the likes of concrete composition, cover to reinforcement, crack width and curing. Requirements can be determined either on the basis of long term experience, or through statistical evaluation of experimental and field data but using a full probabilistic design approach. In other words, it is intended that the service life design approach should be applied to produce or validate a set of deemed-to-satisfy rules for common applications in normal exposure conditions.

10 Input parameters for models

10.1 Surface chloride concentration

The surface chloride concentration is a defining parameter for determination of the chloride ion concentration profile within concrete. It is what effectively defines the concentration difference between the chloride exposure environment at the surface of the concrete and the, initially, essentially chloride-free interior of the concrete. The value required by the diffusion models is actually a notional value formed by the intercept of the chloride ingress profile with depth with the y-axis, i.e. zero depth, the concrete surface. In an actual structure the chloride ingress profile may not follow this idealized form near to the surface. The chloride concentration at the surface may deviate from the idealized curve due to capillary absorption rather than diffusion being the predominant ingress mechanism over the first few millimeters, or due to periodic washing-out by rain. Direct measurements of surface chloride concentration should therefore be treated with caution and, indeed, reported values [TRL, 1999; The Concrete Society, 2004; Helland et al., 2008] show high variability. It is preferable to obtain the notional value by extrapolation of a measured chloride profile over at least, say, 30 or 40mm depth, ignoring the outer 5mm.

The surface chloride concentration has been found to vary with exposure conditions and the type and content of cement and is consequently very difficult to predict for any given set of circumstances. It is, however, a critical parameter in the modelling of chloride ingress and must therefore be seen as a source of considerable uncertainty and error. Concrete Society Technical Report 61 suggests that for critical applications it may be appropriate to use 95% characteristic values such as 0.75% by weight of concrete for Portland cement concrete and 0.90% for blended cement concretes. Nevertheless, the AGEDDCA model contained in the same report uses the same value of 0.50% for all levels of sea water exposure and for both Portland and blended cement concretes (see Figure 10.1).

The AGEDDCA model assumes that the surface chloride level is achieved instantaneously. This may be realistic for surfaces in frequent contact with sea water (i.e. tidal and splash zones) but under less frequent contact (e.g. spray zone) it is likely that the concentration will gradually increase over a time dependent on the actual location. Life-365 assumes gradual build-up over varying periods depending on distance from the sea.

Estimation of surface chloride levels for de-icing salt is fraught with difficulty as this will obviously vary at least with geographical location, local policy regarding use of de-icing salts, rainfall, speed of traffic and distance from the carriageway.

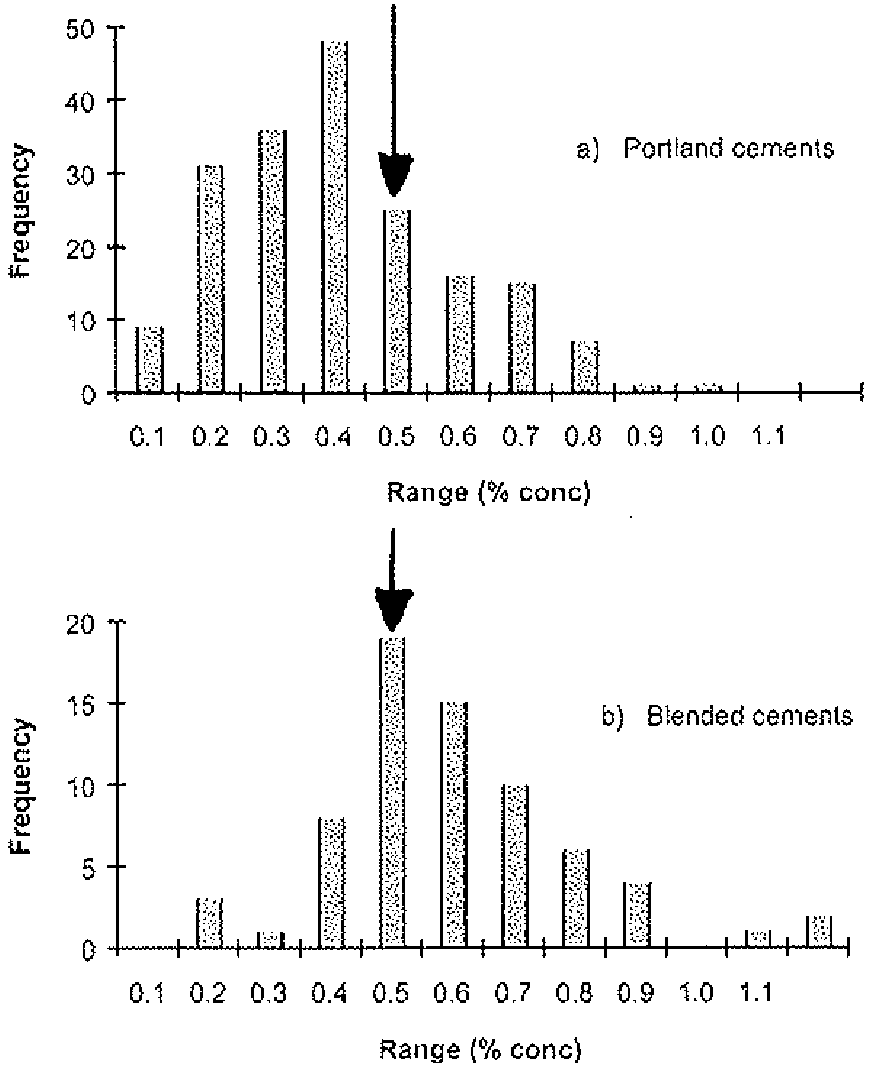


Figure 10.1 The relationship between nominal (indicated by arrow) and measured values for surface chloride concentration

10.2 Effective diffusion coefficient

The mathematical solution of Fick's 2nd law (see Section 9.1) assumes that, although the chloride concentration at any point is changing with time, the surface chloride concentration and diffusion coefficient are constant and that chloride binding is negligible. Nevertheless, chloride binding occurs to differing degrees depending on the cement type. This, together with age-dependent changes in the concrete pore structure, means it is necessary to use an "effective" or "apparent" diffusion coefficient to take account of these age-related effects through inclusion of an ageing factor.

The ageing factor will vary depending on cement type (e.g. presence and proportion of secondary or supplementary cementitious materials). There is clearly some disagreement about the extent of this "ageing" because the AGEDDCA model assumes it continues for the life of the structure whereas Life-365 assumes that it ceases after 25 years. Life-365 reasons that there is some limiting value of diffusion representing the condition of complete hydration at which point further "ageing" would be inappropriate.

Concrete Society Technical Report 61 points out that the value of apparent diffusion coefficient measured from chloride profiles can vary by over an order of magnitude even for the same concrete in the same structure. Even measurements in concrete specimens cast under the closely controlled conditions within a laboratory have been found to differ by a factor of 5 to 6 between the highest and lowest values measured.

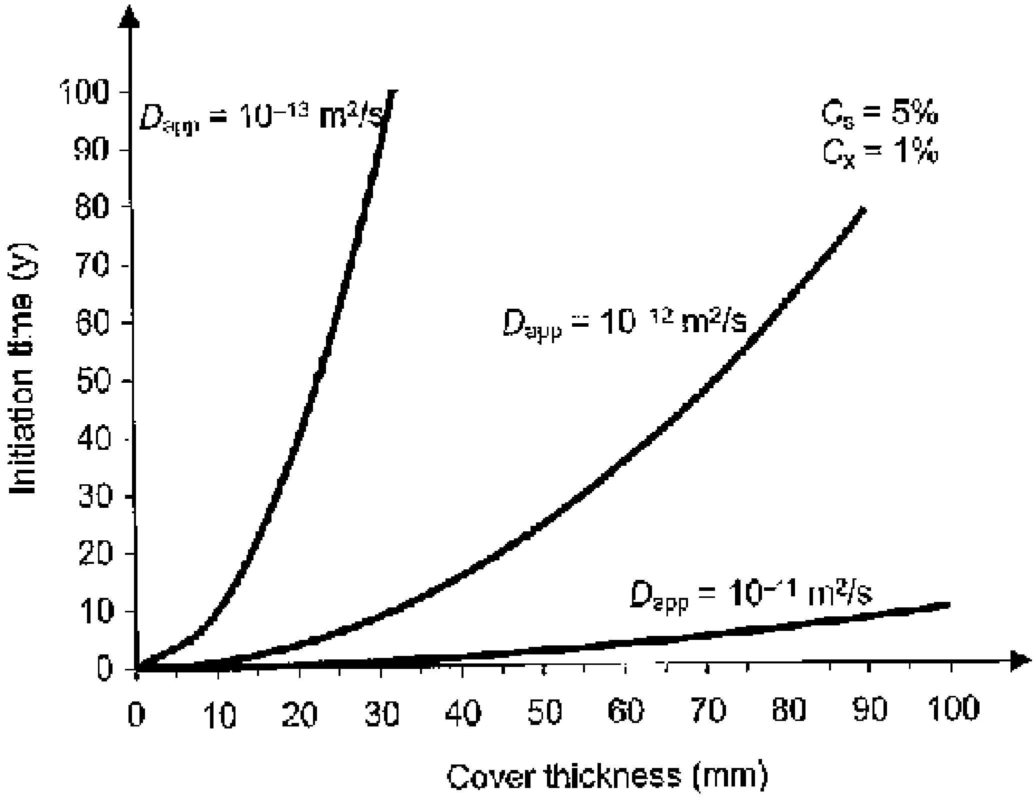


Figure 10.2 Illustration of the effect of variation in the effective, or apparent, diffusion coefficient [Bertolini et al., 2004]

10.3 Background chloride level

All concrete will contain some chlorides within its constituent materials, particularly the aggregates, which will depend on source. Codes and specifications often contain limits for the chloride content of fresh concrete. Unless it is known with confidence that the concrete will be supplied with a lower level, the chloride ingress modelling process should assume the maximum permissible value. Interestingly, in the UK, BS 8500-1 allows the fresh concrete, even for use in chloride exposure conditions, to contain 0.4% by mass of cement as chloride. This value coincides with the commonly assumed threshold chloride level for corrosion initiation for Portland cement concrete and exceeds the value assumed in some models for blended cements. The outcome of any model under these conditions will obviously predict immediate corrosion initiation.

10.4 Chloride threshold level

The so-called chloride threshold level is the level of chlorides at the surface of a reinforcement bar at which the corrosion process is initiated. In practice it varies with a number of factors including cement type and possibly w/c ratio and even exposure conditions. Indeed it has been suggested that there is no single value for threshold chloride level and that it might better be considered in terms of corrosion risk as shown in Table 10.1 [Browne, 1982]:

Nevertheless, deterministic modelling such as described in this report requires selection of a fixed value. This topic is considered in some depth in chapter 12.

Chloride content (% by mass of cement)	Risk of corrosion
< 0.4	Negligible
0.4 – 1.0	Possible
1.0 – 2.0	Probable
2.0	Certain

Table 10.1 Risk of reinforcement corrosion with chloride content

10.5 Illustrative values for variability of input parameters

A paper describing the design of the Western Scheldt Tunnel in the Netherlands [Gehlen & Schiessel, 1999] gives the values in Table 10.2 for average and standard deviation of the main input parameters for chloride ingress modelling used on that particular project:

Parameter	Chosen average value	Standard deviation	Coefficient of variation (%)
Chloride diffusion coefficient (m^2/s)	4.75×10^{-12}	0.71×10^{-12}	14.9%
Chloride threshold level (% binder wt)	0.70	0.10	14.3%
Ageing factor	0.60	0.07	11.7%
Surface chloride level (% binder wt)	4.00	0.50	12.5%
Cover to reinforcement (mm)	50	5	10.0%

Table 10.2 Variability of model input parameters

The actual values for any particular situation will vary depending on concrete composition, cement type, and exposure conditions but the values above serve to illustrate typical assumptions for variability.

10.6 Stainless steel and other corrosion-resistant reinforcement

AGEDDCA takes account of either the use of either galvanised steel or stainless steel reinforcement by higher chloride threshold levels of 1.0 and 3.0 % by mass of cement, respectively. No adjustment is made to the corrosion propagation period.

Life-365 also uses a higher threshold level for stainless steel, 0.5 % by mass of concrete (i.e. 10x that for carbon steel), but not for epoxy-coated bar where enhanced performance is acknowledged through an increase of 20 years in the propagation period.

Neither model assumes an increased propagation period for stainless steel.

11 Other factors affecting predictive models

11.1 Carbonation

All concrete in contact with the atmosphere will carbonate due to reaction of the hydration products with carbon dioxide in the air. The rate of carbonation will depend at least upon the quality of the concrete, carbon dioxide concentration in the atmosphere, ambient temperature and moisture content of the concrete.

As described earlier in the report, carbonation of concrete can create a risk of corrosion of reinforcement. It can, however, also have an impact on the risk of chloride-induced corrosion through:

- Pore structure refinement – carbonation is generally regarded as causing a reduction in pore size within concrete which could cause a reduction in rate of chloride ingress because of a reduced effective diffusion coefficient in the carbonated (outer) layer. Such an effect may be difficult to quantify.
- Release of bound chlorides – carbonation of concrete can cause the release of chlorides that were previously chemically bound into the hydrates. This will have the effect of increasing the effective concentration of chlorides near the surface and increase the rate of diffusion because of the increased concentration difference.
- Reduced threshold level – the chloride threshold level for stainless steel is known to be significantly lower in carbonated concrete than when uncarbonated. No mention of such an effect has yet been found in published literature for carbon steel reinforcement.

The effect of carbonation of concrete cover on chloride-induced corrosion of reinforcement is not specifically considered in the Life-365 or AGEDDCA models. This is probably because most chloride exposure environments feature frequent wetting of the concrete surface, e.g. highway structures splashed by de-icing salt laden water or marine structures subject to contact with sea water or spray. In many climates, exposure to normal rainfall is sufficient to ensure the moisture content in the concrete at the depth of the reinforcement, where adequate cover has been provided, remains constantly high enough to prevent complete carbonation during the life of the structure. Complete carbonation of concrete cover can, however, occur under certain circumstances, such as:

- Very long service life structures
- Predominantly arid conditions with infrequent wetting with chlorides, e.g. dry docks in hot, dry climates
- Elevated carbon dioxide concentrations, e.g. poorly ventilated, heavily trafficked road tunnels
- Inadequate cover and/or inadequate concrete quality

11.2 Cracking and crack widths

There is a widespread perception amongst many that cracking will inevitably cause premature corrosion of reinforcement, and confusion amongst many others about the influence of cracking and crack widths on corrosion initiation and crack rates. Nevertheless, the common predictive models do not explicitly consider the effect of cracking. The fib Model Code [fib, 2006] states that corrosion rates where cracks cross reinforcement is extremely dependent upon the microclimate at the concrete surface, particularly in relation to the orientation of that surface. The most severe conditions are said to exist for cracks in horizontal surfaces subject to frequent chloride exposure (e.g. car park decks in locations where de-icing salts are used in winter). Under these conditions it is suggested that extra precautions are necessary thus implying that chloride ingress modelling under these conditions is inappropriate. For vertical surfaces, and where chloride-laden water is not leaking through cracks, the code suggests that “ordinary crack width limitation” is appropriate implying modelling can be applied under these conditions.

11.3 Execution

Standards of construction will inevitably vary between, and possibly within, structures. Whilst it is probably not reasonable to have to consider gross errors or poor execution with predictive modelling, it may be desirable to consider the effect of curing and method of construction (e.g. in situ or precast). Curing practices vary and may have a significant effect on the chloride diffusion coefficient between say a precast concrete element subjected to short duration steam curing compared to an element cured for a prolonged period by moist coverings or by an efficient curing membrane. The effect of initial curing is likely to be greatest for elements subject to splashing or spray but otherwise in a predominantly dry condition. Frequent wetting or immersion will tend to decrease the influence of curing because of continued hydration under moist conditions. Neither Life-365 nor AGEDDCA explicitly consider the effect of curing although DuraCrete does.

12 Critical Chloride Threshold

Note: This chapter is based on a paper to the ACA November 2009 entitled "Critical Chloride Threshold Levels For Stainless Steel Reinforcement In Pore Solutions" by S. Randström, Avesta Research Center, Outokumpu Stainless AB, Avesta Sweden, and M. Adair, Outokumpu Stainless Ltd, Sheffield, United Kingdom. For convenience, detailed of cited references have been omitted from this report but can be obtained by reference to the original paper which can be found on the Outokumpu web-site.

12.1 Background

As work on predictive modelling has been developmental so too has the approach to evaluating Critical Chloride Threshold (CCT) values. Earlier work was based on determining what concentration of chlorides at rebar depth would break down the passive oxide film on the carbon steel. When this in turn led to the search for more corrosion resistant reinforcement materials it didn’t necessarily mean that the same techniques would transpose themselves onto testing, say, Stainless Steel Reinforcement (SSR). The quest to assess CCT values has therefore also become developmental.

Today SSR is specified around the world to one or the other of two main standards; BS 6744 and ASTM A955. The British Standard, BS6744, allows SSR alloys to be specified by their chemical composition (known as SSR designation) e.g. LDX 2101 (21%Cr, 1% Ni) but also contains an informative table recommending the application of these alloys. This table does not reference any expected chloride level and is therefore no use to a structural designer using predictive models. The American Standard, ASTM A955, carries a table of “typical” alloys and then has a pass or fail corrosion test. Again, as this test cannot be correlated to a percentage of chlorides by mass of cement, it is of no use to designers using predictive models. It is an important difference between the two standards, to note, that the ASTM test is manufacturer specific as opposed the BS table of approved alloys just being chemical composition specific. This is significant as SSR is dependant upon its chromium oxide layer for corrosion resistance and this in turn is dependant upon the pickling process employed by the manufacturer to remove the hot rolled mill scale.

When predictive models are used to enhance the durability of concrete structures by the selective use of SSR, designers first assess where they have to be replace the carbon rebar (assumed in both Life-365 and AGEDDCA to be 0.4% chlorides by weight of cement in temperate climates) and then use chloride depth profiles to assess the CCT value, required at the design life, for the SSR designation to exhibit. Accurate assessment of CCT values by the SSR manufacturers is therefore essential for the use of predictive models. Life-365 acknowledges that there is no standard test for CCT values and proposes a solution. This chapter looks at the issues surrounding CCT values and the developmental work done in assessing them.

12.2 The CCT

Since the maximum tolerated chloride value is dependent on the steel used, attempts to define a critical chloride threshold for different designations of reinforcements have been made. Although many articles, reports etc. have been written on the subject, there are still some issues that remain, namely:

12.2.1 How to define CCT

Although the name itself is a definition, it must be stated that it is the actual free or total chloride level at a specified maximum distance from the reinforcement. Many researchers add chlorides to the concrete and do not mention how much of the chloride that is actually active in the corrosion process compared to how much that is bound as salt in the structure. Although there are correlation factors between total chloride and free chloride, they need to be further investigated for all cement types and measured by different research groups to confirm the results.

12.2.2 How to report CCT

There are mainly two ways of measuring the chloride level in concrete tests, the $[Cl^-]/[OH^-]$ level and "%Cl^- by weight of cement or concrete". There are arguments in favour of both of these methods, and a more expanded discussion can be read in the review by Ann & Song (2007). The $[Cl^-]/[OH^-]$ ratio is suggesting that the inhibiting effect is given by the hydroxyl ion while the detrimental effect is given by the chloride ion. Although this ratio also takes the fact that corrosion is a combined effect of pH and chloride level, the chloride binding is pH dependent, which result in a higher hydroxyl concentration as well as a decrease in chloride concentration when pH is increased. The total chloride level is adopted in many standards as the way to measure chlorides. Although this method does not take the carbonation into account as the $[Cl^-]/[OH^-]$ ratio does, it accounts for local pH drops, that can occur when a pit is nucleated. Moreover, the effect of the cementitious matrix is taken into account, which in a way is a measure of the inhibiting effect.

12.2.3 How to measure CCT

This is the most complex and difficult point to solve. Since concrete structures are expected to have a lifetime of several hundreds of years, accelerated testing is necessary. Accelerated testing itself has some limitations, since it is assumed that there are no long-term effects beyond the test-period. Furthermore, CCT must be measured not only for each steel designation, but also for each cement type, since it has been shown that different cement types have different CCTs. This is mainly explained due to the pH but other effects such as buffering capacity, chloride binding, pore-structure also needs to be clarified.

There are today many different ways to measure the CCT. The difference between measuring in concrete and in simulated pore-solutions is the most obvious. Although measuring CCTs in concrete is closer to the real application, there are also a larger number of uncontrollable parameters, such as voids, water/cement ratios, bound/free chlorides etc., which can cause unexpected results and larger scattering of data. The concrete measurements are also more time-consuming since there is a need to cast all specimens. On the other hand, when measuring in a simulated pore-solution the CCT needs to be "translated" into a concrete-CCT. The concrete also has a certain buffering capacity and lower transport rates, if the chlorides are ponded rather than cast-in, that can cause the CCT level to vary between concrete and pore-solutions.

In natural conditions, the chloride concentration slowly increases with time since it is a result of a diffusion process so many investigations instead use concrete to which chlorides are added to the water added to the cement. For a cast-in method, a diffusion process will be time-consuming and take at least some months. For a pore-solution the problem is less complicated since chlorides can be added but it is then assumed that the effects of the increasing chloride concentrations are the same.

The final critical point is the potential of the rebar. Certain authors define the CCT as the potential where no corrosion process occurs independent of the potential or current density for evaluation, whereas other researchers uses a defined potential for which the current density should be low. Both potentiodynamic as well as potentiostatic testing exist.

Potentiodynamic testing is performed by a very slow increase of the potential (typically 10-20 mV·min-1) from the corrosion potential (open circuit potential) up to a certain potential where a sudden current increase is seen and thereafter the scan is reversed. The formation of a hysteresis loop normally indicates that local corrosion has occurred, while a direct decrease upon a reverse could indicate either oxygen evolution or transpassive corrosion. In the alkaline environments the potential for oxygen evolution is lower than in neutral environments and transpassive corrosion is therefore seldom observed. In a potentiostatic experiment, the potential is increased to a certain potential and thereafter either the temperature or the chloride level is increased to make the environment more severe. The third and most used method for cast-in methods is to study the corrosion at E_{corr}. Although this method is the most realistic method, it is time-consuming and the onset of corrosion is not clearly seen.

12.2.4 What is the significance to the real application

As already mentioned, the most important aspect of the CCT value is that it is applicable for the construction, and that the CCT can rank reinforcement materials and concrete types to guide designers for material selection in a concrete construction. Although this seems to be an advantage of "real" tests, they are time-consuming and many parameters are uncontrolled, which can cause different results, under apparently the same conditions. This makes it more difficult to rank and insert new materials into the standard.

12.3 Factors that influence the CCT

A review was recently published by Ann & Song [2007] concerning the CCT. Below is a short summary of the article as well as other publications.

12.3.1 Measuring the chloride content

Total or free chloride measurements are mainly performed in three ways. The first two methods involve grinding of cement or mortar samples. This powder can thereafter either be boiled in water or dissolved in acid. These methods should give the total chloride content in the concrete. For measurements of the free chloride concentration, pore pressing is used. In this method, the cement paste is placed under high pressure and small volumes of pore solution are pressed out. Although this method only should detect the free chlorides, there is still a risk that bound chlorides are released, causing an overestimation of the free chloride level. Although the pH is not generally influenced by the presence of chlorides, the stability of certain aluminium chlorides is pH-dependent, and hence, a change in pH can cause a change in the chloride concentration since chlorides will be released into the pore-solution.

12.3.2 Additions that affect the CCT

The calcium aluminate (C_3A) and calcium ferroaluminate (C_4AF) salts bind chlorides and thereby increase the CCT in a positive direction. However, since the bound chlorides can be released if the pH is decreased due to carbonation, there might be a risk to add these salts. For the same reason the ground granulated blastfurnace slag (ggbs) has been found to have a small positive effect on the CCT compared to Portland cement (PC). This raises a further question point of how to measure the CCT, since a measurement of the total chloride level should be more independent of the C_3A level than using only the free chloride concentration as the CCT.

The effect of fly ash is not clear. Most authors claim that fly ash has a negative effect on CCT; however, the time to corrosion can still be prolonged. It has been claimed that fly ash has a positive effect on the CCT. Manera et al. discussed in a recent paper the effect of pozzolanic additions on the CCT. Although the pozzolanic addition itself decreased the pH of the cement and thereby decreased the CCT, the refinement of the pore-structure decreased the ingression rate of the chloride and hence caused a delay in the time to corrosion. It is therefore important to make the CCT independent of the effect of ingression rate, since these effects should be handled by correcting the apparent diffusion coefficient rather than the CCT. The original purpose of the article from Manera et al. was to investigate the effect of silica fume in Portland cement. It was found that silica fume had a negative impact on the CCT. It will be reasonable to assume that the CCT of any given rebar type will vary with the cement type.

12.3.3 Voids

In the review by Ann & Song the effect of voids near the rebar was also discussed. Due the lack of cement hydration products close to the rebar, the buffering was lower and thus the pH was decreased more easily. This resulted in a decrease of the CCT. Moreover, the currents could be higher in free solution than in presence of concrete/cement due to the higher conductivity of the pore solution.

12.3.4 Surface finish

Finally it has been found that the surface finish of the reinforcement also plays a role and to determine CCT correctly, the surface finish of the specimens must be similar so as to allow a discussion of any scatter in the data.

12.4 Cast in methods

Cast-in methods are the methods that are closest to the real application, thus the CCT achieved, can directly be translated into the highest chloride level that a certain type of reinforcement can withstand. However, cast-in methods are often more time-consuming than pore solution tests, and uncontrolled parameters such as voids near the rebar can affect the CCT measured, which causes a larger spread of the data. This is the argument between the use of cast-in mortar blocks and cast-in concrete blocks and also the argument against using highly porous concrete or mortar blocks.

12.4.1 Mixed-in chlorides

There are some methods used to determine CCT in cast-in structures. Alonso et al. [2002] published a review of CCT methods, with CCT determined for carbon steel by various methods, both pore-solution tests as well as cast-in methods. In their own study, they used carbon steel cast into mortar specimens (2×2×8 cm). The specimens were placed at RT for 7 days in RH 95%. E_{corr} was measured before the specimens were immersed in 0.5-2 M NaCl solution while I_{corr} was measured, by Linear Polarisation Resistance (LPR). The specimens were removed and the mortar was crushed and chloride content measured. A drawback with this method is that the chloride content measured is the mean concentration, and not necessarily the concentration at the rebar.

Pedeferri et al. in COST 521, 2002 and Andrade & Gonzalez have used the same method as Sederholm et al. used, where the chlorides are added in the mixing water, in the form of $CaCl_2$. Pedeferri et al. do not mention whether the chlorides are measured after the test or that the reported chloride levels are the added chlorides. Not much experimental information is given, but both galvanically coupled and non-coupled carbon steel were investigated, and potentiostatic control was used when the steels not have initiated corrosion at the corrosion potential. In the Andrade experiments, five different cement types are tested with different amounts of pozzolana. In the first graphs, it seems like the pozzolana has a positive effect on the CCTL since lower corrosion currents were measured when pozzolana was used. Garcia-Alonso et al. have also used mixed in chlorides, monitoring the corrosion potential during two years in small mortar samples. Wet sponges were used to enable contact between the specimen and counter electrodes.

Sørensen et al. investigated the CCT in mortar prisms (size 40×40×160 mm) with 0-8% Cl^-. Both carbon steels as well as stainless steels (304, 316) were investigated and the rebars were tested in both welded and unwelded condition in a Portland cement (w/c = 0.5). Two different electrochemical methods were used; potentiodynamic scans from E_{corr} to +200 mV_{SCE} then to –700 mV_{SCE} and back to E_{corr}. In a second series of experiment, a scan from E_{corr} to +200mV_{SCE} and then back to E_{corr} was performed. A series of potentiostatic experiments was also performed where the potential and time were as follow: 0 mV_{SCE} for 5 days, thereafter +150 mV_{SCE} for 1.5 days and +200 mV_{SCE} for 2.5 days. The criterion for corrosion was set as the chloride content where the current was in the order of 10^{-2} $A{\cdot}m^{-2}$. Two sets of experiments were also done in an outdoor environment for 5 months and a carbon dioxide-rich environment respectively and were visually inspected. Furthermore, a comparison with the immersion tests was done, by immersing mortar samples at 0 mV_{SCE} in a solution of 1 M NaCl. The time to corrosion varied between 82-255 days, for those samples that corroded (experiment was interrupted after 175-285 days).

12.4.2 Pond-Test

The pond-test is the method among the cast-in methods that mostly closely resembles the application. A pond of a high chloride-containing solution is put on top of cement or concrete blocks in which rebars are cast in. The method is described in the standard ASTM G109 and has been used by several researchers.

In the ASTM G109 standard, a pond is placed on top of the concrete specimen (101.6x152.4 mm) containing 400 ml of 3 wt% NaCl. Three rebars are placed in the concrete specimen, an upper one that will experience a higher chloride level than the two rebars below. Hence, the upper rebar will be active, while the two lower rebars will be passive and act as cathodes. The voltage and currents are measured and time to failure is defined as the time where the current reaches 10 mA and at least half the samples exhibit currents greater than 10 mA. This method is time-consuming. The concrete specimens are cured for 28 days in a moist room (RH 100%) and thereafter further dried for two weeks (RH 50%) and the test should be started one month after they are taken out from the moist room. The test assumes that a standard level of chlorides reaches the upper bar and the variable is the corrosion inhibitor, which has been added to the mix. It would be possible to vary the chloride levels to the upper bar by increasing the salt solution concentration and number of ponding cycles, but cumbersome for the intent of determining the CCT.

Varying the salt solution concentration was carried out by Lambert et al. that used the pond method. A 5% NaCl solution was used, and the exposure time was up to two years. The specimens were exposed to a weekly or monthly wet-dry cycle in four different concrete types (PC paste, PC limestone, PC quartzite and SRPC quartzite.) To further compare the influence of wet-dry cycles, a set of slabs with admixed chlorides was also used (0-2% NaCl). Measurements of E_{corr} and the polarisation resistance were performed (from which I_{corr} was calculated). The pond test with monthly wet-dry cycles showed the worst performance, and among the concretes, the SRPC-quartzite showed the highest chloride levels.

Castellote et al. used mortar specimens that had a size of 70×70 mm. The specimens were cured for 28 days at RH 95%. A stainless steel plate was placed as a cathode in the pond above the rebar, and a steel plate at the bottom as the anode. Unlike the previous Lambert et al. that used natural diffusion to transport the chloride to the rebar, Castellote et al. used electrostatic method. A potential (up to 13 V) was applied between the stainless steel electrode at the top ant the steel electrode at the bottom. Currents were measured by LPR, which is a common method to measure the corrosion rate.

The pond-test method has also been used by Hartt et al. where both low alkalinity (LA) and high alkalinity (HA) cements have been used. In both cases the w/c ratio was 0.5. The time to corrosion was defined as the first time the potential was below $-280\ mV_{SCE}$. For LA this time was between 13 and 60 days, while in the case of HA it was between 112-197 days. The shorter time in case of the LA cement was explained by the high w/c ratio and an expected dry concrete, which caused fast ingress of chlorides.

The cast-in method used by Manera et al., is an intermediate between the cast-in chlorides method and the pond-test method. The authors use a method where the mixed in chlorides are put in a separate layer between two layers of non-chloride concrete. Measurements of the half-cell potential versus an MnO_2 electrode as well as determination of corrosion currents by LPR were performed during the experiment. The temperature was held at 20 ºC for 3 months; it was thereafter increased to 35 and 50 ºC for 15 days at each temperature. The results showed that the addition of 10% silica fume decreased the CCT, defined as the chloride concentration that macro-cell currents was in the range of 1-2 mA x m^{-2} combined with a potential lower than $-200\ mV_{SCE}$.

12.5 Pore solution tests

The simulated pore solution should resemble the environment to which the rebar is exposed. A homogenous chloride concentration as well as a faster material transport speeds up the test time and significantly limits the amount of uncontrollable parameters.

However, since the CCT in concrete and CCT in pore solutions cannot be assumed to be the same, translation problems between the two are expected.

Although there are a lot of different cements, not many researchers have adjusted the content of the pore solutions to reflect a certain type of cement, except for the used of hydrocarbonate and carbonate ions to simulate carbonation. Some data concerning the chemical content of pore-solutions exist. Andersson et al. pressed pore-solutions out of cement cylinders of different cements to prove chemical composition.

Interestingly, the redox potential (Eh) of the different concretes were measured as well. It was found that the redox potential, i.e., the oxidative effect that the pore solution exerts on the reinforcement, differs between the different cement types. This is something that not has been discussed at all in the CCT discussion and will undoubtedly affect both the corrosion potential and corrosion behaviour, but not the pH. Thus, it is not only the $[Cl^-]/[OH^-]$ that should be noted but also the oxidative effect of the environment in which the reinforcement is placed.

12.5.1 Potentiostatic methods

Potentiostatic methods are used in references by Bertolini et al. and Hurley & Scully, both performed potentiostatic as well as potentiodynamic tests. In the potentiostatic method used by Hurley & Scully, the potential was again set at +200 mV, and the method based on that of Bertolini et al. with addition of long-term measurement of the corrosion potential. The definition of breakdown was defined as the chloride content where the current density exceeded 1-2 mA x cm^{-2}. The chloride content in the solution was increased every 24th hour, starting without chlorides. Constant chloride potentiostatic tests were also performed, using the same test parameters as above. A slightly lower CCT was observed with the constant chloride potentiostatic tests than the incremental chloride tests. However, looking at the diffusion profile of chlorides in the real application, the incremental chloride test should resemble the real application better than the constant chloride test.

The American standard ASTM A955 uses a pore solution where one bar is placed in a pore solution containing 15% NaCl, and two bars are placed in another beaker only containing pore solution at RT. A gel-salt bridge connects the two beakers, and should limit the amount of chlorides transported through the salt bridge. The solution should be exchanged every five weeks. Readings of the voltage drop over a 10-W resistor should be made every day the first week, thereafter weekly. Although this is not a potentiostatic test in the sense that the potential is controlled, the corroding rebar will experience an elevated potential since it is galvanically coupled to passive carbon steel.

12.5.2 Potentiodynamic tests

The potentiodynamic tests used by Bertolini et al. consisted of a slow sweep (20 $mV \cdot min^{-1}$) in the pore solutions. They started 200 mV below the corrosion potential and continued until the current density exceeded 0.5 $mA \cdot cm^{-2}$. The same method was later used by CAPCIS on behalf of CARES UK, when the corrosion resistance of various SSR designations was tested, among them 1.4162. CAPCIS stated that this test method only could be used for ranking of various designations and that it will be difficult to relate it to concrete-CCTL due to the impossibility to convert % chlorides in a solution to % chloride by weight of cement/concrete.

Hurley and Scully used a number of methods to define the chloride threshold. In their potentiodynamic scans, that started 50mV below E_{corr} and was increased in anodic direction at a rate of 10 $mV \times min^{-1}$, their definition of CCT was the chloride concentration where the pitting potential was below 200 mV_{SCE}, well below the oxygen evolution potential, occurring at around +600 mV_{SCE}. There is in the article an extensive discussion about methods as well as the influence of oxygen evolution on the competing corrosion process. Oxygen evolution will significantly limit the effective current density for corrosion if the potential ever will increase above the oxygen evolution potential. The comparison between potentiostatic and potentiodynamic testing revealed that potentiodynamic testing gave different results for the CCT level of SSR than potentiostatic tests. The explanation that was given was that the potentiodynamic test was too fast to cause corrosion, and there was a risk that potentiodynamic testing caused an overestimation of the CCT.

Bautista et al. investigated the influence of welding and weld-cleaning by using potentiodynamic testing in saturated $Ca(OH)^2$ with addition of 0.5% NaCl and with bubbling of CO_2-enriched air to simulate carbonation. The electrochemical measurements were performed by polarisation curves with a scan rate of 10 mV x min^{-1}. Since the purpose of the article was to study the influence of welding and weld cleaning, no CCTL values were published, but instead the pitting potentials were recorded. The results are not very conclusive, but as already known, it was shown that the carbonation decreased the corrosion resistance. Welded material had a lower pitting resistance than unwelded material, and weld cleaning such as sand blasting and pickling increased the resistance to pitting corrosion.

Also Moreno et al. have performed potentiodynamic testing in the pore solutions with the same composition as Bertolini et al. Their criterion for CCT was when a breakdown potential is observed.

12.5.3 Tests performed at the corrosion potential

Moreno et al. also investigated the CCTL in different pore-solutions at E_{corr}, while continuously performing LPR measurements to determine the corrosion rate. Surprisingly they found general corrosion in the mildly carbonated environment, even without chlorides. According to the Outokumpu Corrosion Handbook, even carbon steel should withstand similar environments. Their definition of CCTL from the corrosion potential measurements was a corrosion current density a magnitude higher than it was without chlorides, i.e. in the range of 10-6 A x m^{-2}.

12.6 Summary

It is important to note at this stage that this is an independent report by Ove Arup for Outokumpu Stainless Ltd. and that this chapter is a client contribution based on a recent literary review and experiment set up which has been included because of the interdependence of predictive models and CCT values.

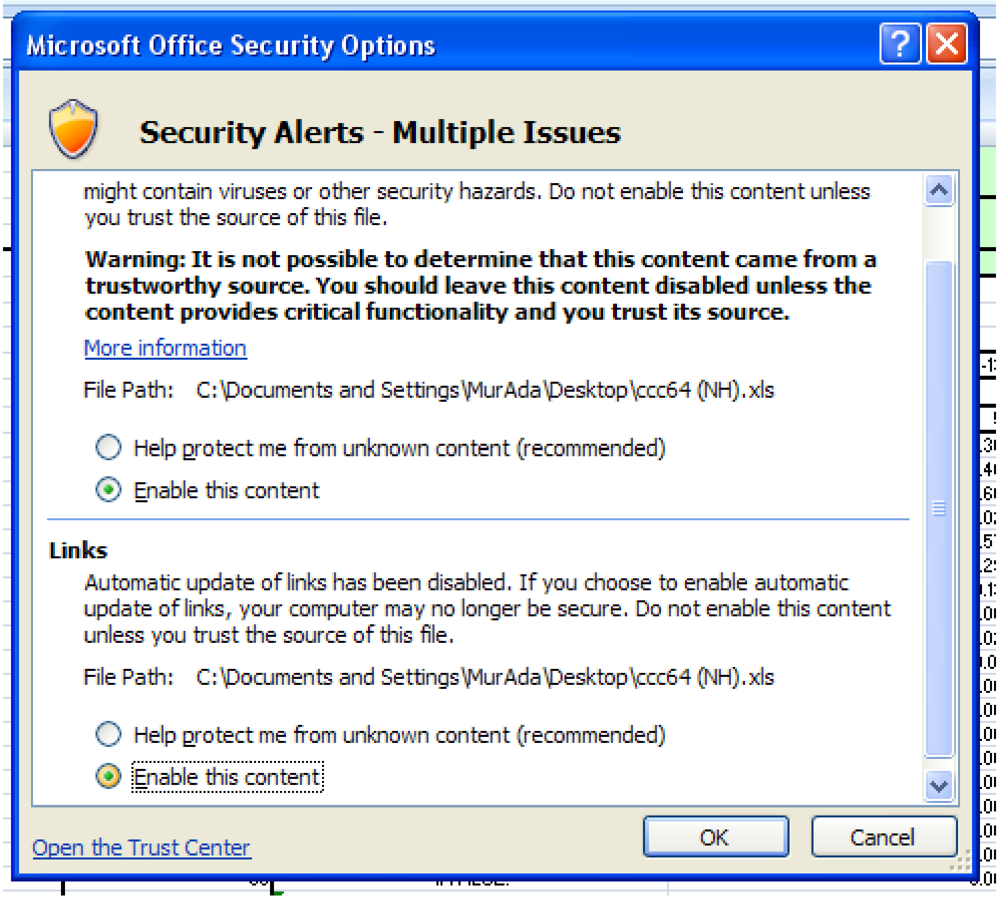
Firstly CCT value assessment is very developmental and not as clear has it may have previously been reported. Secondly whilst potentiostatic testing has advantages over potentiodynamic it needs to be further defined by hold levels. Thirdly whilst cast in samples are more reliable than in solution testing they still need chloride levels measured accurately and correlating to the accelerated in solution testing so that manufactures can certify the corrosion resistance of their SSR

13 A simple representative model

13.1 Overview

It is important to reiterate that the purpose of the Simple Representative Model (SRM)is to help illustrate the aspects of existing models discussed in the body of this report and not to provide definitive answers for durability solutions. It is hoped that the demonstration of the variables will assist engineers' model their own thoughts towards their durability issues.

The SRM is excel based with a visual basic front end. With Microsoft 2007 it will be necessary to enable both content and links at the start of operation.



Microsoft Office Security Options

Security Alerts - Multiple Issues

might contain viruses or other security hazards. Do not enable this content unless you trust the source of this file.

Warning: It is not possible to determine that this content came from a trustworthy source. You should leave this content disabled unless the content provides critical functionality and you trust its source.

More information

File Path: C:\Documents and Settings\MurAda\Desktop\ccc64 (NH).xls

○ Help protect me from unknown content (recommended)

● Enable this content

Links

Automatic update of links has been disabled. If you choose to enable automatic update of links, your computer may no longer be secure. Do not enable this content unless you trust the source of this file.

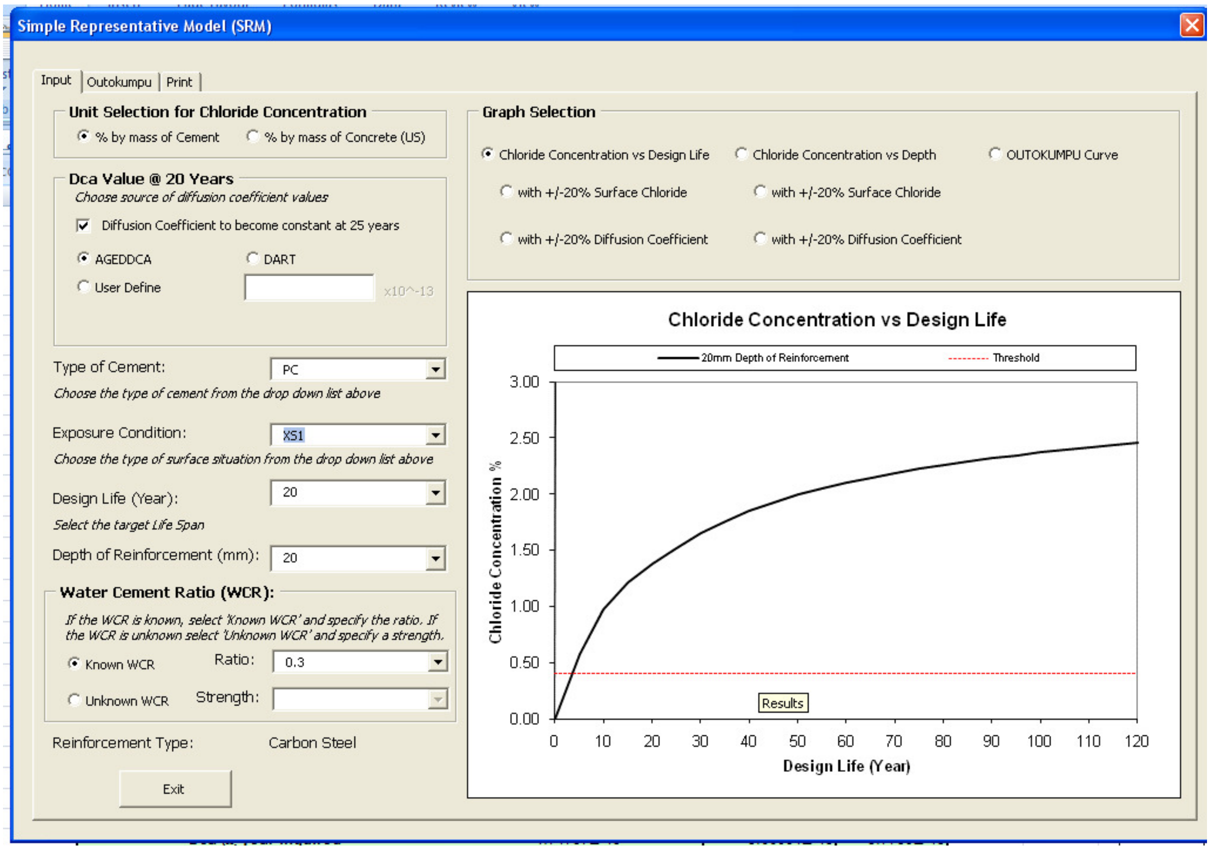
File Path: C:\Documents and Settings\MurAda\Desktop\ccc64 (NH).xls

○ Help protect me from unknown content (recommended)

● Enable this content

Figure 13.1 Possible Security alert message prior to successfully running the model

The SRM has been written to be used from the visual basic front end and not the spreadsheets which sit behind it. The first screen will therefore be as below:



Unit Selection for Chloride Concentration

● % by mass of Cement ○ % by mass of Concrete (US)

Dca Value @ 20 Years

Choose source of diffusion coefficient values

☑ Diffusion Coefficient to become constant at 25 years

● AGEDDCA ○ DART

○ User Define x10^-13

Type of Cement: PC

Choose the type of cement from the drop down list above

Exposure Condition: XS1

Choose the type of surface situation from the drop down list above

Design Life (Year): 20

Select the target Life Span

Depth of Reinforcement (mm): 20

Water Cement Ratio (WCR):

If the WCR is known, select 'Known WCR' and specify the ratio. If the WCR is unknown select 'Unknown WCR' and specify a strength.

● Known WCR Ratio: 0.3

○ Unknown WCR Strength:

Reinforcement Type: Carbon Steel

Graph Selection

● Chloride Concentration vs Design Life ○ Chloride Concentration vs Depth ○ OUTOKUMPU Curve

○ with +/-20% Surface Chloride ○ with +/-20% Surface Chloride

○ with +/-20% Diffusion Coefficient ○ with +/-20% Diffusion Coefficient

Figure 13.2 The default screen settings on the SRM

This screen sets to the defaults as shown above. The first selection for the user is whether chloride values are to be expressed by mass of cement or by mass of concrete. Generally speaking mass of concrete is used in the USA and by mass of cement elsewhere.

The model has been written using data made available by the Concrete Society from the AGEDDCA model. There are strong links between the authors of both AGEDDCA and Life 365 and the table below is common to both CSTR61 and the Life 365 manual. A significant difference between the two models is that Life 365 ceases the ageing effect on effective chloride diffusion coefficient after 25 years, whereas AGEDDCA allows it to continue for the life of investigation. Errors in the ageing factor due to developmental data gathering will therefore be exaggerated the longer the life that the model is predicting if ageing is not truncated.

The SRM gives the user the ability to explore the differences by using the function of Diffusion Coefficient to become constant at 25 years. The SRM also gives a comparison between the AGEDDCA data and the more limited DART data, as explained in the body of the report, as well as a free form Dca entry. It is also important to remember that the analysis using a free entry value of Dca is still being determined using the data from AGEDDCA. The other variables are all on pull down menus and explained in the next section.

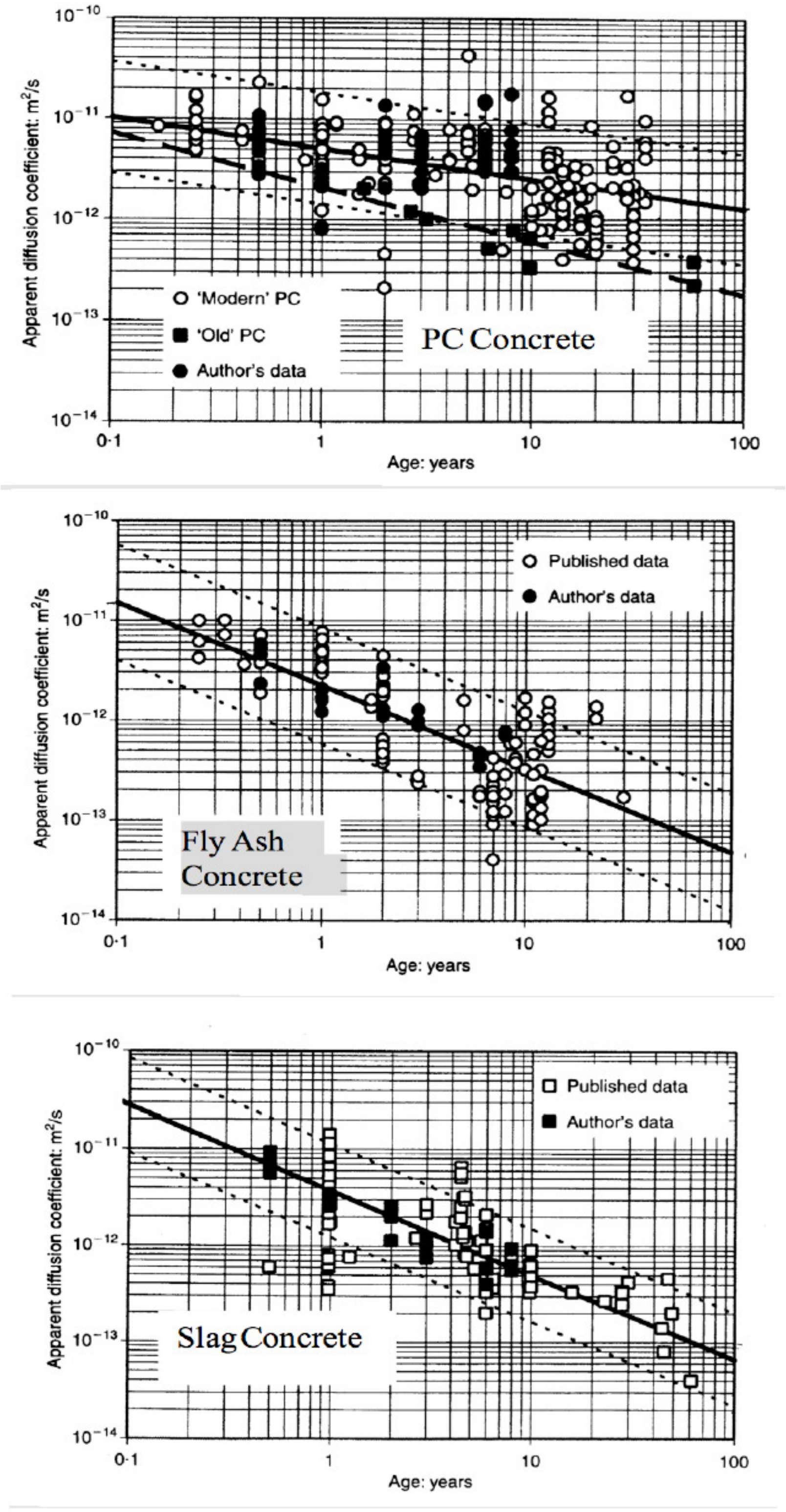
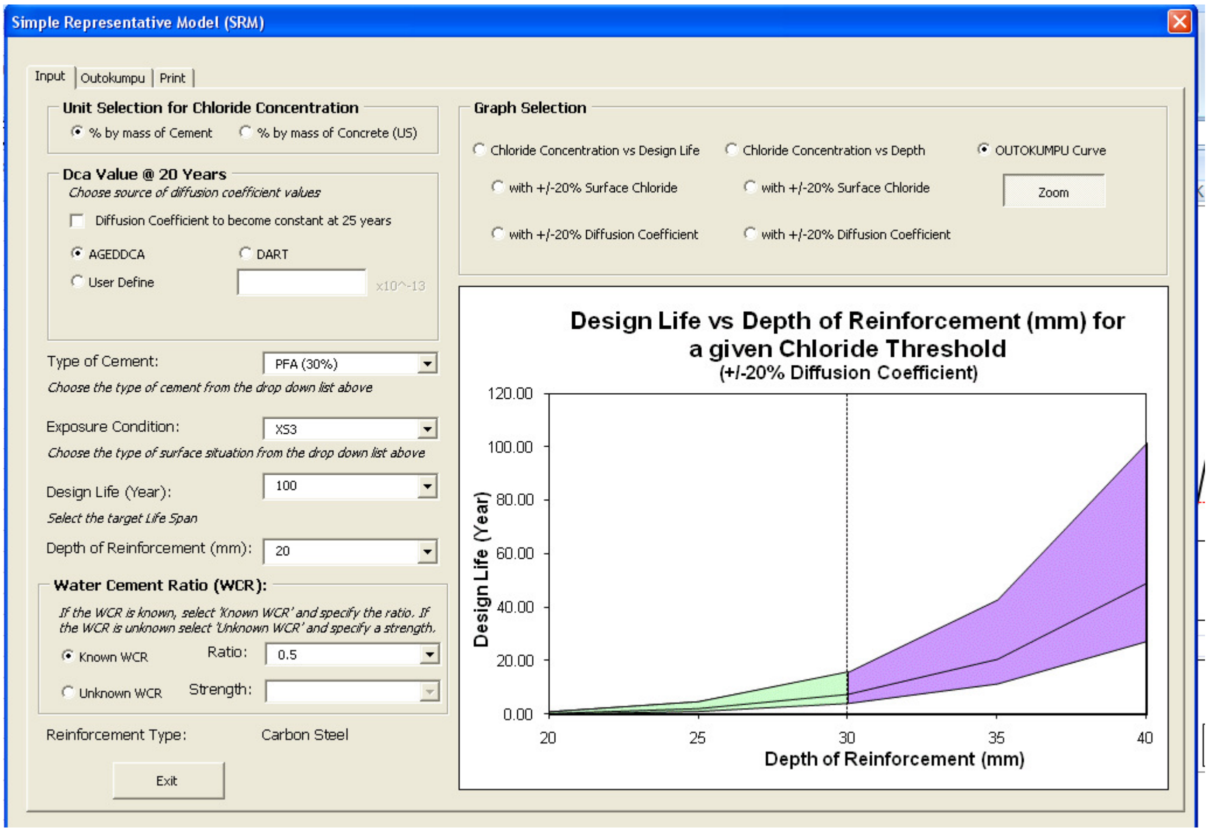


Figure 13.3 Basic data on the effects of ageing from both TR61 and Life 365

13.2 Use of the model

The use of predictive modelling to specify the selective application of Stainless Steel Reinforcement (SSR) is demonstrated by the SRM in two simple steps: Firstly use the “Outokumpu” curve to determine where corrosion of carbon steel rebar, set at 0.4% chlorides by weight of cement , is probably going to occur and where it probably will not occur for a given design life.



Unit Selection for Chloride Concentration

● % by mass of Cement ○ % by mass of Concrete (US)

Dca Value @ 20 Years

Choose source of diffusion coefficient values

☐ Diffusion Coefficient to become constant at 25 years

● AGEDDCA ○ DART

○ User Define x10^-13

Type of Cement: PFA (30%)

Choose the type of cement from the drop down list above

Exposure Condition: XS3

Choose the type of surface situation from the drop down list above

Design Life (Year): 100

Select the target Life Span

Depth of Reinforcement (mm): 20

Water Cement Ratio (WCR):

If the WCR is known, select 'Known WCR' and specify the ratio. If the WCR is unknown select 'Unknown WCR' and specify a strength.

● Known WCR Ratio: 0.5

○ Unknown WCR Strength:

Reinforcement Type: Carbon Steel

Graph Selection

○ Chloride Concentration vs Design Life
○ with +/-20% Surface Chloride
○ with +/-20% Diffusion Coefficient

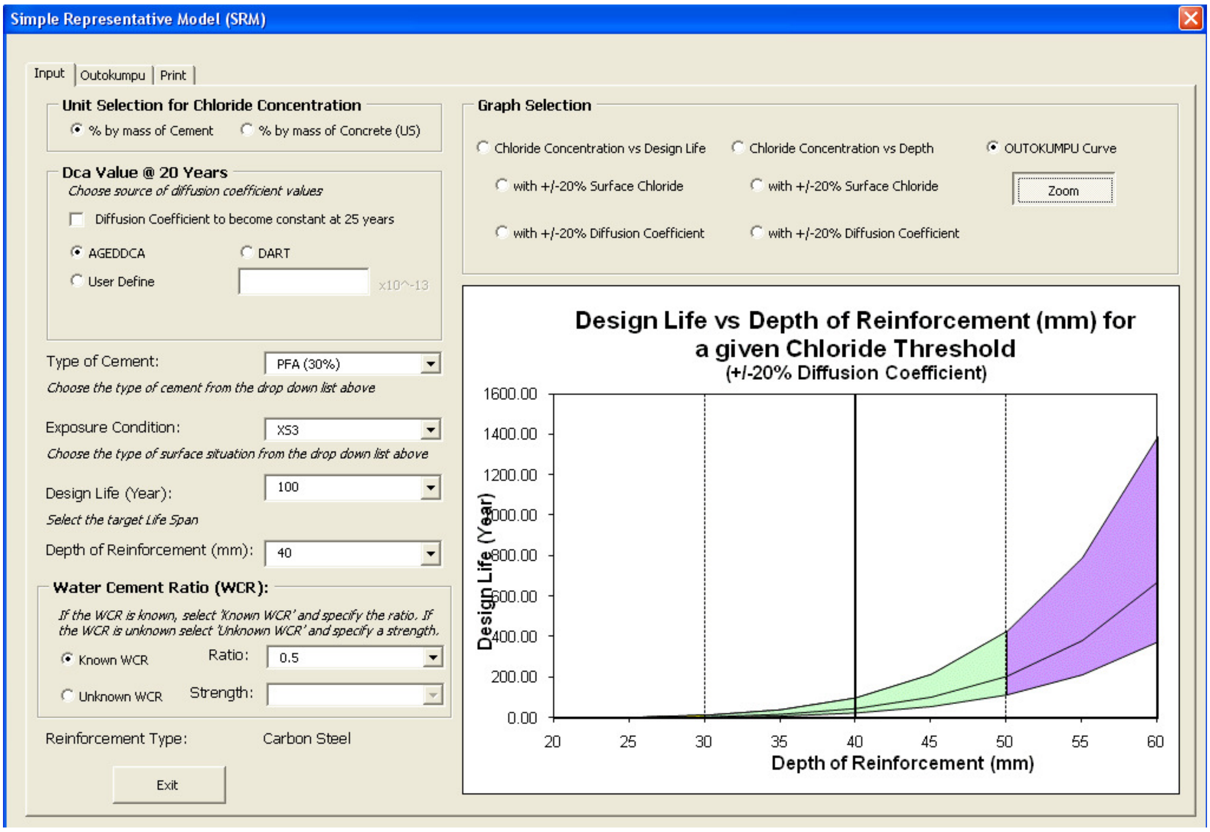
○ Chloride Concentration vs Depth
○ with +/-20% Surface Chloride
○ with +/-20% Diffusion Coefficient

● OUTOKUMPU Curve

Figure 13.4 Working example of where to consider the substitution of carbon steel rebar with SSR.

The use of the zoom function allows a more visible inspection of the SRM's output. A variance of 20% only in diffusion coefficient demonstrates the effect of the inherent variability within concrete when using predictive modelling and it is important to note from the report that research has found variabilities of up to one order of magnitude.

Moving the rebar depth allows investigation of where the carbon steel rebar will probably be free from corrosion. In the example below, when compared with Figure 13.4 above, the implication is that carbon steel rebar will need to be replaced with SSR at 40mm depth but not at depths greater than 50mm for a 100 year life structure, as seen from figure 13.5 below.



Simple Representative Model (SRM)

Input | Outokumpu | Print

Unit Selection for Chloride Concentration
● % by mass of Cement ○ % by mass of Concrete (US)

Dca Value @ 20 Years
Choose source of diffusion coefficient values
☐ Diffusion Coefficient to become constant at 25 years
● AGEDDCA ○ DART
○ User Define x10^-13

Type of Cement: PFA (30%)
Choose the type of cement from the drop down list above

Exposure Condition: XS3
Choose the type of surface situation from the drop down list above

Design Life (Year): 100
Select the target Life Span

Depth of Reinforcement (mm): 40

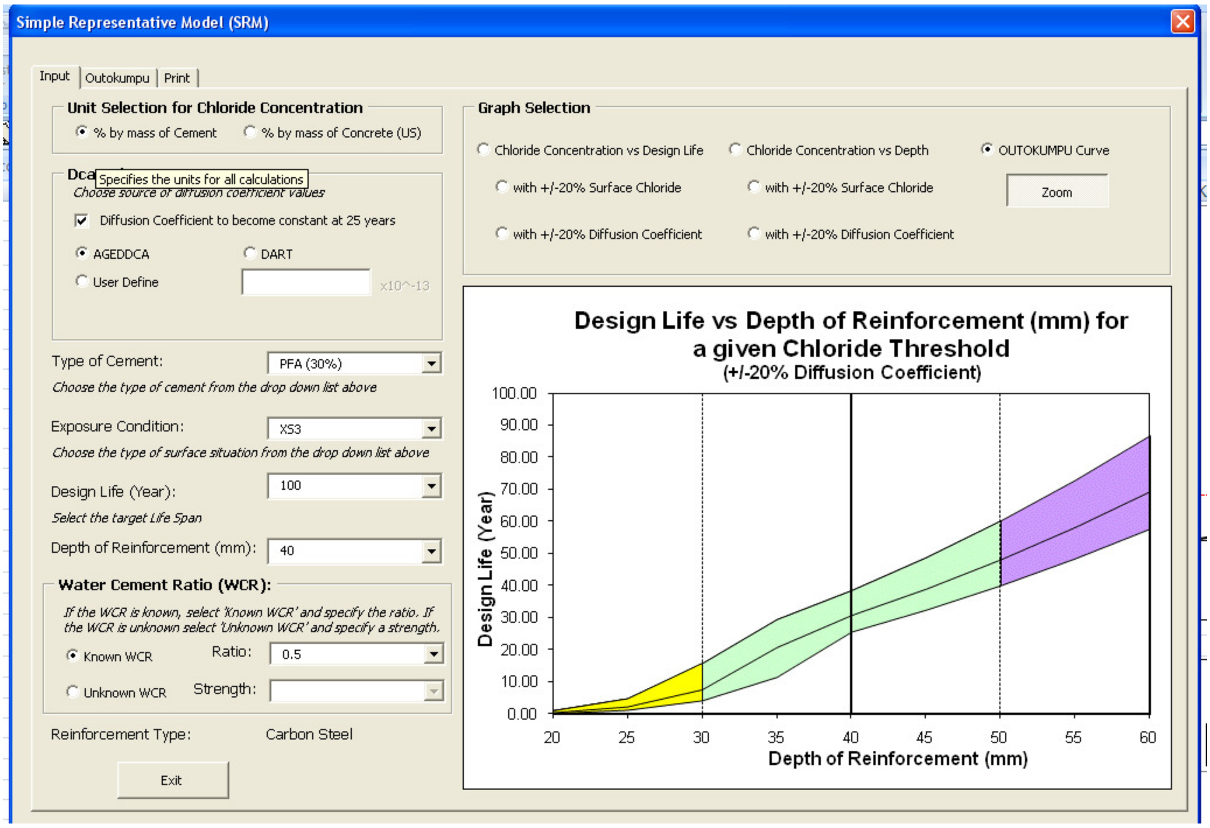
Water Cement Ratio (WCR):
If the WCR is known, select 'Known WCR' and specify the ratio. If the WCR is unknown select 'Unknown WCR' and specify a strength.
● Known WCR Ratio: 0.5
○ Unknown WCR Strength:

Reinforcement Type: Carbon Steel

Graph Selection
○ Chloride Concentration vs Design Life ○ Chloride Concentration vs Depth ● OUTOKUMPU Curve
○ with +/-20% Surface Chloride ○ with +/-20% Surface Chloride
○ with +/-20% Diffusion Coefficient ○ with +/-20% Diffusion Coefficient

Figure 13.5 Examination of where it will be prudent to leave carbon steel rebar in the design.

At this point it is useful for the user to see the effect of ceasing the ageing factor at 25 years as in the example below:



Simple Representative Model (SRM)

Input | Outokumpu | Print

Unit Selection for Chloride Concentration
● % by mass of Cement ○ % by mass of Concrete (US)

Dca Specifies the units for all calculations
Choose source of diffusion coefficient values
☑ Diffusion Coefficient to become constant at 25 years
● AGEDDCA ○ DART
○ User Define x10^-13

Type of Cement: PFA (30%)
Choose the type of cement from the drop down list above

Exposure Condition: XS3
Choose the type of surface situation from the drop down list above

Design Life (Year): 100
Select the target Life Span

Depth of Reinforcement (mm): 40

Water Cement Ratio (WCR):
If the WCR is known, select 'Known WCR' and specify the ratio. If the WCR is unknown select 'Unknown WCR' and specify a strength.
● Known WCR Ratio: 0.5
○ Unknown WCR Strength:

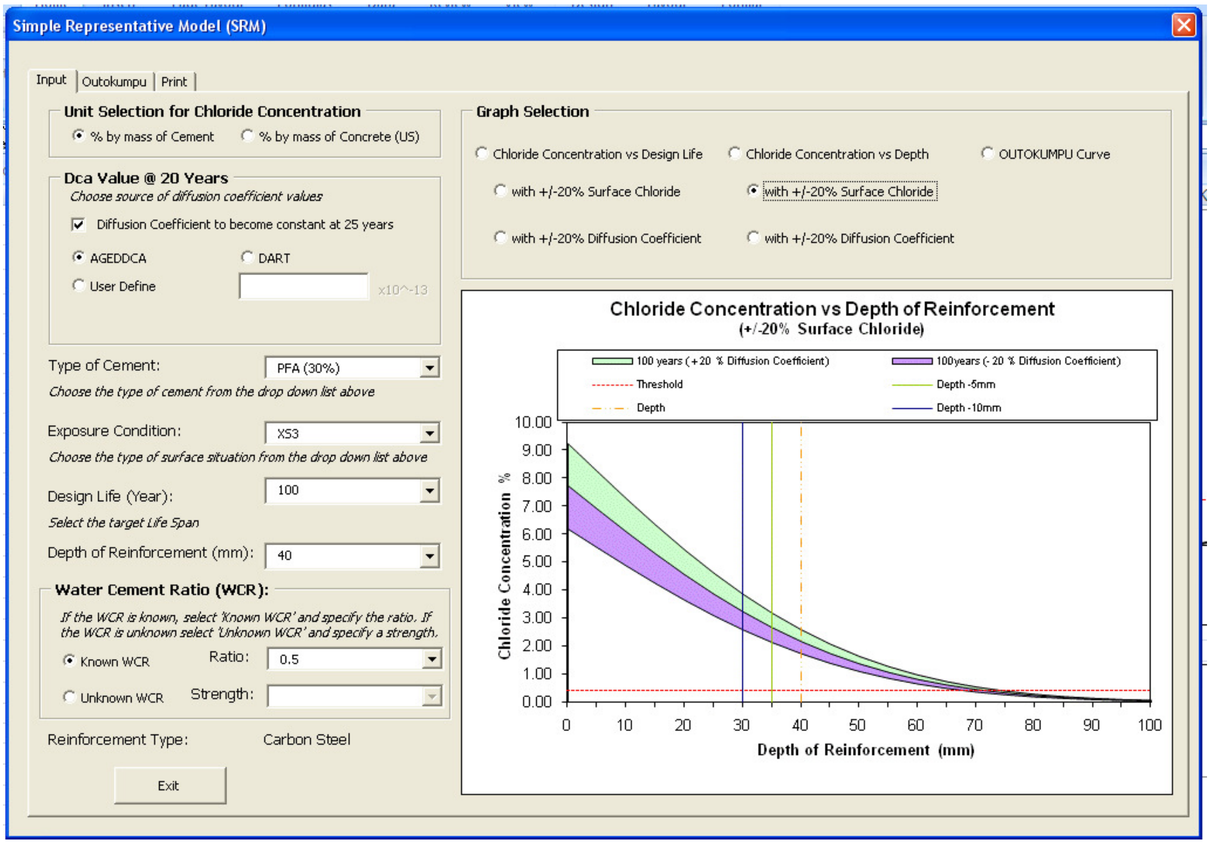
Reinforcement Type: Carbon Steel

Graph Selection
○ Chloride Concentration vs Design Life ○ Chloride Concentration vs Depth ● OUTOKUMPU Curve
○ with +/-20% Surface Chloride ○ with +/-20% Surface Chloride
○ with +/-20% Diffusion Coefficient ○ with +/-20% Diffusion Coefficient

Figure 13.5 The effect of ceasing the ageing at 25 years

The effect of ceasing the ageing is quite considerable, particularly for cement containing supplementary cementitious materials such as fly ash or ggbs, and at this point greater depths should be investigated.

Once the user has determined where they are going to substitute the carbon steel with SSR the second step of the process is to determine what CCT (critical chloride threshold) value of the SSR needs to be thus allowing selection of the most cost effective designation. The SRM has two ways of doing this: either by reviewing chloride concentration versus design life or by chloride concentration versus depth for a given design life. Both are useful and the example below uses the latter:



Simple Representative Model (SRM)

Input | Outokumpu | Print

Unit Selection for Chloride Concentration
● % by mass of Cement ○ % by mass of Concrete (US)

Dca Value @ 20 Years
Choose source of diffusion coefficient values
☑ Diffusion Coefficient to become constant at 25 years
● AGEDDCA ○ DART
○ User Define x10^-13

Type of Cement: PFA (30%)
Choose the type of cement from the drop down list above

Exposure Condition: XS3
Choose the type of surface situation from the drop down list above

Design Life (Year): 100
Select the target Life Span

Depth of Reinforcement (mm): 40

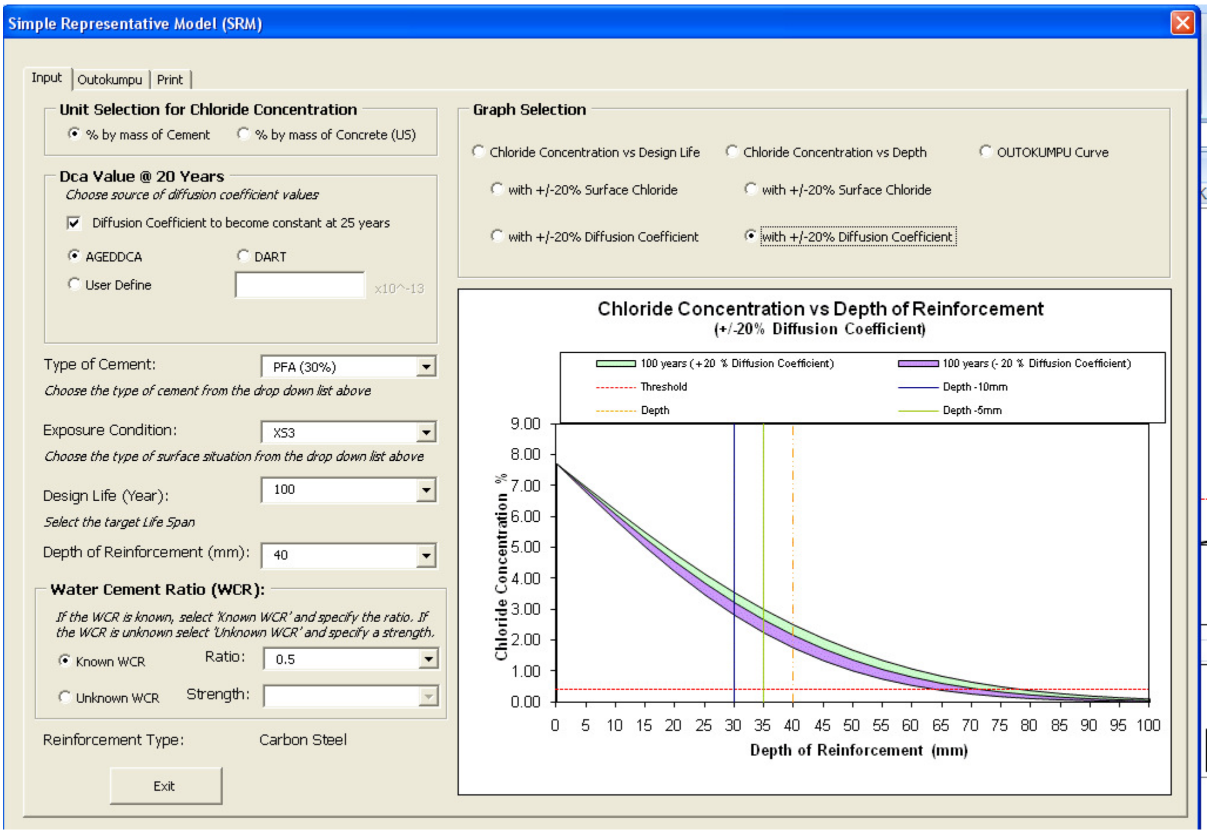
Water Cement Ratio (WCR):
If the WCR is known, select 'Known WCR' and specify the ratio. If the WCR is unknown select 'Unknown WCR' and specify a strength.
● Known WCR Ratio: 0.5
○ Unknown WCR Strength:

Reinforcement Type: Carbon Steel

Graph Selection
○ Chloride Concentration vs Design Life
○ with +/-20% Surface Chloride
○ with +/-20% Diffusion Coefficient
○ Chloride Concentration vs Depth
● with +/-20% Surface Chloride
○ with +/-20% Diffusion Coefficient
○ OUTOKUMPU Curve

Figure 13.6 The working example now determining required CCT value for the SSR

Again the SRM highlights to the user the potential variables in the assumptions. Figure 13.6 above looks at 20% variance on surface contents of chlorides and Figure 13.7 below looks at 20% variance on effective diffusion coefficient.



Unit Selection for Chloride Concentration

● % by mass of Cement ○ % by mass of Concrete (US)

Dca Value @ 20 Years

Choose source of diffusion coefficient values

☑ Diffusion Coefficient to become constant at 25 years

● AGEDDCA ○ DART

○ User Define x10^-13

Type of Cement: PFA (30%)

Choose the type of cement from the drop down list above

Exposure Condition: XS3

Choose the type of surface situation from the drop down list above

Design Life (Year): 100

Select the target Life Span

Depth of Reinforcement (mm): 40

Water Cement Ratio (WCR):

If the WCR is known, select 'Known WCR' and specify the ratio. If the WCR is unknown select 'Unknown WCR' and specify a strength.

● Known WCR Ratio: 0.5

○ Unknown WCR Strength:

Reinforcement Type: Carbon Steel

Graph Selection

○ Chloride Concentration vs Design Life ○ Chloride Concentration vs Depth ○ OUTOKUMPU Curve

○ with +/-20% Surface Chloride ○ with +/-20% Surface Chloride

○ with +/-20% Diffusion Coefficient ● with +/-20% Diffusion Coefficient

Figure 13.7 A second view of CCT value requirement through the SRM

From these two views the designer will be looking for an SSR designation of at least 3% chlorides by mass of cement plus whatever safety factor they feel necessary given the inherent variability and uncertainty in the concrete parameters.

Three more very important facts to recognise from these last two figures are that firstly these models do not take account of any background chlorides which will inevitably be present in the concrete, and these at-depth profiles are the best way to estimate the effect of background chlorides. Secondly, the designer must also specify the temperature and pH of the concrete when specifying the CCT value of the SSR designation as both parameters have a major effect on CCTs. Finally, the cement type has an effect on the original pH of the concrete but it also has an effect on the structures ability to bind chlorides. Whilst this binding capacity has an impact on the effective diffusion coefficient of the concrete it is not clear from the models whether the actual chloride content predicted refers to free chlorides, which will cause the corrosion, or the total chlorides including bound chlorides.

Please ensure that you use the exit button to leave the visual basic front end.

13.3 The model

This model is a relatively simple model based on the basic version of the published AGEDDCA model [The Concrete Society, 2004]. As such it is based on the assumption that chloride ingress from the surrounding environment into concrete can be described by Fick's 2nd law of diffusion and solved with the standard error function commonly used in other such models:

$$C_{x,t} = C_s\,[1\text{-erf}(x/(2\sqrt{DT}))] \qquad \text{(Eqn 38)}$$

where:

$C_{x,t}$	Concentration of chloride at depth 'x' from the surface, at time 't'
C_s	Concentration of chloride at the concrete surface

x	Depth from the surface
D	Diffusion coefficient
t	Time
erf()	Error function

The diffusion coefficient depends on the pore structure of the concrete and the ability with which chloride ions can migrate inwards from the surface. This in turn depends on the type of cement and the free water/cement ratio of the concrete. The actual free water/cement ratio of a concrete is difficult for a non-specialist to predict and, in practice, may be less than any maximum value contained in a specification in order to achieve the required strength class. Conversely, a particular specified maximum free water/cement ratio for durability may result in a lower value than would otherwise be needed just to meet strength requirements. The model thus allows for input both as free water/cement ratio where the value is known (or a maximum value is specified), or simply by strength class where w/c ratio is not known or specified. The model should preferably be run separately for maximum free w/c ratio and strength class where both are specified.

The diffusion coefficient for a given concrete is not constant with time because of interaction between the chlorides and the cement hydrates (chloride binding) and changes in the pore structure due to increased hydration and the reaction of any secondary cementitious materials, where included. The diffusion coefficient used in the model is thus varied according to the expression:

$$^{D1}/_{D2} = (^{T1}/_{T2})^{n} \qquad \text{(Eqn 39)}$$

where:

D1	Predicted diffusion coefficient at time T1
D2	Known diffusion coefficient at time T2
T1	Time T1 at which chloride ingress state is to be determined
T2	Time T2 to which the diffusion coefficient D2 relates
n	Age factor – based on the selected cement type

As described in 13.1 the model allows the user to choose between allowing the ageing effect to continue for the whole time under consideration or to truncate it at 25 years.

The time at which conditions exist for initiation of corrosion is determined by comparison of the predicted chloride content at the specified depth of the reinforcement with the chloride threshold level for the type of reinforcement under consideration.

The model does not include specific consideration of the time from initiation of corrosion to manifestation of damage to the concrete through cracking or spalling, commonly known as the propagation period. This period will depend at least upon:

- Availability of oxygen within the concrete
- Ambient temperature
- Diameter of reinforcement bars
- Depth and strength of concrete cover

In most critical locations it is generally accepted as wise to treat the time marking the end of the initiation period and the start of the propagation period as the limit state for durability design. The propagation period then becomes part of the safety margin but in some conditions, such as marine exposure in hot climates, this may be as low as just a few years.

In less critical locations such as chloride-free, sheltered superstructure the propagation period can be as long, or even longer, than the initiation period but it unlikely that such conditions would warrant consideration of stainless steel reinforcement.

It is possible to include a fixed propagation period by simple subtraction of the chosen value from the input value for time.

The model allows for different exposure conditions, different concrete types and different steel types for reinforcement. It does not allow for other protection strategies such as coatings, hydrophobic surface impregnation, corrosion inhibitors, controlled permeability formwork or permanent formwork. The true effect of such measures is not well known so incorporation into the model would involve large assumptions in the absence of adequate data. Users requiring to investigate such effects are directed to the AGEDDCA model but advised to be cautious in interpretation of any results obtained.

13.4 User input data

To operate the model and predict the ingress of chloride into a particular concrete, the user is required to input various parameters as listed below. All inputs are made by selection from a limited range of available choices. Should a required input value not be included within the available range then this should be regarded as beyond the scope of the model and reference should be made to more detailed models such as “AGEDDCA” in Concrete Society Technical Report 61 [The Concrete Society, 2004]:

Depth – the depth value used in the model is the minimum depth of the outer layer of reinforcement, measured to the near surface of the bar, not the centre line. This is usually specified in increments of 5mm so the user can select from a limited range (20-100mm) of values in 5mm increments.

Time – this will commonly be the required working life (design life) for the structure although it may be decided to deduct a period to take account of the time from corrosion initiation to manifestation of damage to the element. Common required working lives are 50-60 years for building structures and 100-120 years for monumental structures or infrastructure. The user will be able to select from a limited range of values, generally in 20 year increments over the range 20-120 years.

Exposure condition – this dictates the surface chloride concentration which, in effect, provides the driving force behind the chloride ingress. The model allows a choice of conditions based on either the chloride-induced corrosion of reinforcement exposure classes from European Standard EN 206-1 or a descriptive equivalent:

- XS1 sea water exposure – Exposed to airborne salt but not in direct contact with sea water
- XS2 sea water exposure – Permanently submerged
- XS3 sea water exposure – Tidal, splash & spray zones
- XD1 de-icing salt exposure - Moderate humidity
- XD2 de-icing salt exposure - Wet rarely dry
- XD3 de-icing salt exposure – Cyclic wet & dry
- Sea water submerged i.e. XS2
- Tidal/splash/spray zone i.e. XS3
- Coastal i.e. XS1
- De-icing salt splash i.e. XD3
- De-icing salt spray i.e. XD1

Concrete composition – this dictates the diffusion coefficient, and its variation with time, which controls the rate at which the chlorides are able to move into the concrete. The model allows a choice from a limited range of common cement types and either free water/cement ratio or strength class:

- Cement type
    - Portland cement (CEM I)
    - CEM I + 30% fly ash
    - CEM I + 50% ggbs
    - CEM I + 70% ggbs
    - CEM I + 5% silica fume
    - CEM I + 10% silica fume

European cement designations have not been used for the combinations of CEM I and secondary cementitious materials because each designation covers a wide range of composition with a wide range of resistance to chloride ingress, e.g. CEM IIIA covers 36-65% ggbs.

Free water/cement ratio – the user can select a free w/c within the range 0.30 – 0.65 in increments of 0.05. Alternatively the user can specify the w/c ratio indirectly through giving a strength class.

Strength class – the user may not have a feel for water/cement ratio and can thus input strength class instead within the range C30/37 – C50/C60. The model will then select the appropriate value of free w/c ratio, for the chosen cement type, to enable calculation of diffusion coefficient etc. The strength class is expressed using the designation given in European Standard EN 206-1 as explained in Table 14:

Strength class	Characteristic cylinder strength (MPa)	Characteristic cube strength (MPa)
C30/37	30	37
C35/45	35	45
C40/50	40	50
C45/55	45	55
C50/60	50	60

Table 13.1 European Standard compressive strength classes

The selection of free w/c ratio is based on the following stored data for selected cement types as given in Table 13.2:

Strength class	Cement type					
	I	II/B-V (30% fa)	IIIA (50% ggbs)	IIIB (70% ggbs)	II/A-D (5% sf)	II/A-D (10% sf)
C30/37	0.55	0.48	0.55	0.48	0.565	0.58
C35/45	0.50	0.45	0.50	0.45	0.52	0.54
C40/50	0.46	0.41	0.46	0.41	0.48	0.50
C45/55	0.425	0.375	0.425	0.375	0.445	0.465
C50/60	0.39	0.35	0.39	0.35	0.41	0.43

Table 13.2 Assumed values of free water/cement ratio based on strength class for different cement types

13.5 Output

From the user input data the model determines, depending on the user's choice, one of the following:

- the chloride ion profile with depth for the selected design life
- the chloride ion profile with time at the selected cover to reinforcement, or
- a plot of the design life that can be achieved for different cover depths.

This information is presented graphically to give the user a better appreciation of the nature of chloride ingress than if just presenting numerical values. The model is not intended to facilitate calculation of specific design life predictions for given input parameters.

13.6 Background data

13.6.1 Surface chloride concentration

The surface chloride concentration varies with different exposure conditions and there is a degree of disagreement between values used in other models. There is some evidence to suggest that the surface chloride concentration varies with cement type possibly due to differing degrees of chemical binding of chlorides within the different matrices. This model does not use different values for different cement types. The selected values are based on a pragmatic review of other models and published information. The value for the most extreme conditions is the most arbitrary as this is felt to be the area where published information is the least reliable. The selected value has been chosen to represent very onerous conditions. The build-up of chloride at the surface is assumed to be instantaneous for all exposure conditions. This is a conservative assumption, particularly where concrete surface is subjected to occasional exposure to chlorides such as splashing of de-icing salts during the winter months only. The values used are given in Table 13.3.

It should be noted that surface chloride concentration is conventionally expressed as a proportion of the mass of concrete whereas threshold chloride level is conventionally expressed as a proportion of the mass of cement. It is important not to confuse these. A density value of 2350 kg/m^3 has been used in the model to convert between the two.

Exposure condition	Surface chloride concentration (% Cl by mass of concrete)
XS1	0.65
XS2	0.90
XS3	1.15
XD1	0.30
XD2	0.90
XD3	1.15
Sea water submerged	0.90
Tidal/splash/spray	1.15
Coastal	0.65
De-icing salt splash	1.15
De-icing salt spray	0.30

Table 13.3 Surface chloride concentrations assumed for different exposure conditions

13.6.2 Chloride diffusion coefficient

Cement Type (with Mineral Additions)	**Water/ Cement Ratio**	**n - Age Factor**	**Dca @ year 20 (AGEDDCA)**	**Dca @ 28 days (AGEDDCA)**	**DART default values**
PC	0.40	-0.26	7.47E-13	3.25E-12	8.90E-12
	0.45	-0.26	9.40E-13	4.09E-12	1.00E-11
	0.5	-0.26	1.18E-12	5.14E-12	1.58E-11
	0.55	-0.26	1.49E-12	6.48E-12	1.97E-11
	0.65	-0.26	2.36E-12	1.03E-11	2.50E-11
Fly Ash (30%)	0.40	-0.69	1.82E-13	8.25E-12	5.60E-12
	0.45	-0.69	2.14E-13	9.73E-12	6.90E-12
	0.50	-0.69	2.53E-13	1.15E-11	9.00E-12
	0.55	-0.69	2.98E-13	1.35E-11	1.09E-11
	0.65	-0.69	4.15E-13	1.88E-11	1.49E-11
GGBS (70%)	0.40	-0.62	1.75E-13	4.69E-12	1.40E-12
	0.45	-0.62	2.06E-13	5.63E-12	1.90E-12
	0.50	-0.62	2.43E-13	6.75E-12	2.80E-12
	0.55	-0.62	2.86E-13	8.10E-12	3.00E-12
	0.65	-0.62	3.96E-13	1.17E-11	3.40E-12

Table 13.4 Comparison of initial chloride diffusion coefficient data from DART and AGEDDCA

The initial chloride diffusion coefficient is taken from the "AGEDDCA" model in Concrete Society Technical Report 61 [The Concrete Society, 2004]. It varies with cement type and water/cement ratio. Where concrete quality is input as strength class rather than water/cement ratio, the w/c ratio is selected as shown previously in Table 13.2.

This allows the SRM to demonstrate how predictive modelling facilitates the specification of SSR by using a profile not used by the other two main models and to show the effect of variations on assumptions. Some controversy exists over the validity of the CSTR 61 data, particularly in relation to fly ash concrete. For this reason there is a capability within the model to alternatively select the DART coefficient data from table 9.9 or for the user to input their own diffusion co-efficient at 20 years. Values from DART and AGEDDCA are compared in Table 13.4.

13.6.3 Age factor

The age factor, n, is taken from the "AGEDDCA" model in Concrete Society Technical Report 61 [The Concrete Society, 2004] and accounts for changes in the diffusion coefficient either, at the choice of the user, over the whole time under consideration or just a 25 year period whereafter the diffusion coefficient remains constant. The values depend on cement type and those used in the model are given in Table 13.5:

Cement type	Age factor, n
Portland cement (CEM I)	-0.26
CEM I + 30% fly ash	-0.69
CEM I + 50% ggbs	-0.60
CEM I + 70% ggbs	-0.62
CEM I + 5% silica fume	-0.44
CEM I + 10% silica fume	-0.56

Table 13.5 Age factor for different cement types

It should be noted that for the purposes of this simple representative model, the values of age factor for the combinations containing silica fume do not vary with w/c ratio, as they do in the source AGEDDCA model. The values selected here are based on a fixed w/c ratio of 0.4 which is regarded as fairly typical for this type of concrete.

13.6.4 Chloride threshold level

The chloride threshold levels have been selected based on published literature. There is some evidence that chloride threshold level may be lower for cements containing secondary cementitious materials such as fly ash and ggbs, and that it may vary with various other factors. Due to the uncertainty of these views this model uses the single value of 0.4% by mass of cement for carbon steel reinforcement regardless of cement type.

It should be noted that chloride threshold level is conventionally expressed as a proportion of the mass of cement in the concrete whereas surface chloride concentration is conventionally expressed as a proportion of the mass of the concrete. It is important not to confuse these. A density value of 2350 kg/m^3 has been used in the model to convert between the two.

14 Concluding discussion

Experience has shown that chloride-bearing exposure environments can be very aggressive to reinforced concrete, in many cases resulting in deterioration due to corrosion of the embedded steel reinforcement. Premature deterioration of many marine structures and highway and parking structures in regions where de-icing salts are commonly employed has led to a lack of confidence in some deemed-to-satisfy design code recommendations and the desire for improved and more flexible design methods. An immense amount of effort has been expended over several decades in the development of mathematical models to assist in the design of reinforced concrete structures exposed to such conditions.

It is vital when using chloride-ingress modelling that it is clear to the user right from the start that the predictions have a high degree of uncertainty due to the necessary assumptions and simplifications of the nature of concrete and its transport properties, and the limited validity of the input data. Indeed, it can be said that the only thing one knows for certain about the output from such modelling is that it is wrong; the secret to intelligent use of modelling is to be aware of this. Deterministic models are especially dangerous in this respect as they give a single output value often expressed with a high degree of precision but, in reality, carrying a low degree of accuracy. Probabilistic models are an improvement in this respect as they attempt to quantify the variability or uncertainty of the input data and express the outcome in terms of the likelihood of a certain condition existing at a given time.

The use of predictive modelling can, however, be a useful tool to support the consideration of the selective use of stainless steel reinforcement and will develop in its use as greater knowledge of the input parameters render prescriptive measures less necessary.

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