

Durability of Reinforced Concrete Structures, Theory vs Practice

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Abstract: The durability of concrete as a construction material and the durability of reinforced concrete structures under different environmental conditions are probably the least understood attributes of concrete. No simple theory applies and only incremental advancement based on experiences gained through practice can be achieved. This paper addresses this issue, with particular emphasis on the possible improvement of durability through better concrete mix design (or more specifically use of high-performance concrete), more careful crack control, use of corrosion inhibitors and use of protective coatings.

1. Introduction

The durability of a structure is its ability to serve its intended purposes for a sufficiently long period of time, or at least during its expected service life. However, although a durable structure is expected to serve without deterioration to the extent that major repair is required before expiry of its design life, it must not be presumed that durability is a substitution for good maintenance. Even for a structure that has been designed and constructed to a high durability standard, regular inspection and routine maintenance to restore the fitness for purpose of the structure are needed. Hence, when we come to the durability design of a structure, we must bear in mind the future need for maintenance and take into account the cost of maintenance in the life cycle cost analysis of the structure.

A reinforced concrete structure may deteriorate because of deterioration of the concrete itself or because of corrosion of the steel reinforcing bars inside the concrete. Common causes of the deterioration of concrete include alkali-aggregate reaction, chemical attack, freezing and thawing action, abrasion and fire. There is often the misunderstanding that carbonation and chloride attack would cause deterioration of concrete. In actual fact, carbonation and chloride attack would not cause any harm to

the concrete itself; they mainly cause de-passivation of the steel in concrete, which then leads to steel corrosion. Deterioration of concrete is not a common problem. In most cases, if there is any problem with the durability of a concrete structure, it is the corrosion of the steel reinforcing bars that is most likely to be the major cause. This paper focuses on the durability of reinforced concrete structures in relation to corrosion of steel reinforcement.

The corrosion of steel is an electro-chemical reaction, similar to what is happening in a galvanic cell (i.e. a battery). As presented by Neville (Properties of Concrete, 4th Edition, 1995), a brief description of the corrosion phenomenon is as follows. When there exists a difference in electrical potential along the steel bar in concrete, an electro-chemical cell is set up: there form anodic and cathodic regions, connected by the electrolyte in the form of pore water in the hardened cement paste. The positively charged ferrous ions Fe^{++} at the anode pass into the pore solution while the negatively charged free electron e^- pass through the steel into the cathode where they are absorbed by the constituents of the electrolyte and combine with water and oxygen to form hydroxyl ions $(\text{OH})^-$. These travel through the electrolyte and combine with the ferrous ions to form ferrous hydroxide, which is converted by further oxidation to rust. The electro-chemical reactions involved are as follows:

Anodic reactions:



Cathodic reaction:

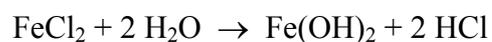


It can be seen that oxygen is consumed and water is needed for the process to continue. There is no corrosion in dry concrete probably below a relative humidity of 60% nor is there corrosion in concrete fully immersed in water, except when water can entrain air, for example by wave action. The optimum relative humidity for corrosion is 70 to 80%. At higher relative humidities, the diffusion of oxygen through the concrete is considerably reduced. The differences in electro-chemical potential can arise from differences in the environment of the concrete, for example when a part of it is permanently submerged in sea water and a part is exposed to periodic wetting and drying. A similar situation can arise when there is a substantial difference in the thickness of cover to the steel.

Fortunately, even with a continuous supply of oxygen and water, the steel in concrete does not necessarily corrode. The concrete cover protects the steel from corrosion not just by hindering the ingress of deleterious fluids but also by means of passivation. As explained by Neville (Properties of Concrete, 4th Edition, 1995), steel embedded in hydrating cement paste rapidly forms a thin passivating layer of oxide, which strongly adheres to the underlying steel and gives it complete protection from reaction with oxygen and water, i.e., from corrosion. This state of the steel is known as passivation. Maintenance of passivation is conditional on an adequately high pH (or, in other words, high alkalinity) of the pore water in contact with the passivating layer. Basically, concrete is alkaline because of the presence of lime, i.e. Ca(OH)₂, which is liberated as a byproduct during cement hydration. The pH of the pore water in hardened Portland cement paste is initially around 12.6 to 13.5. This pH is high enough to offer passivation protection to the steel.

However, there is everywhere carbon dioxide, i.e. CO₂, in the air. Carbon dioxide reacts with moisture to form carbonic acid, which then reacts with the lime in the pore water of concrete to form calcium carbonate, a neutral product. This is called carbonation. As a result, the alkalinity of the concrete gradually drops and once the pH is reduced to below around 10, the passivation protection to the steel will be gone and if there is oxygen and water, the steel will start to corrode.

Apart from carbon dioxide, chloride ions may also destroy the protective passivity layer on the surface of embedded steel thereby causing de-passivation. Chloride ions activate the surface of the steel to form an anode, the passivated surface being the cathode. The reactions involved are as follows:



Thus, the chloride ion Cl⁻ is regenerated so that the rust contains no chloride, although ferrous chloride is formed at the intermediate stage.

Rust has a lower density than steel. Hence, as corrosion takes place, the volume increases and since the expansion is restrained by the surrounding concrete, bursting stresses are induced, resulting in cracking, spalling or delamination of the concrete cover. This makes it easier for aggressive agents to ingress towards the steel, with a subsequent increase in the rate of corrosion. In other words, once corrosion starts, it will accelerate.

2. Permeability and Diffusivity of Concrete

With the exception of mechanical damage, all the adverse influences on the durability of concrete involve the transport of fluids or ions through the concrete. There are four fluids/ions principally relevant to the durability of concrete: water, oxygen, carbon dioxide and chloride ions. They can move through concrete in different ways:

- *permeation* (flow under pressure gradient);
- *diffusion* (flow under concentration gradient);
- *sorption* (flow under capillary suction).

As sorption is normally insignificant, the major factors influencing the durability are the permeability and diffusivity of the concrete.

The permeation of fluids through a porous medium generally follows the Darcy's law:

$$\frac{dq}{dt} \frac{1}{A} = \frac{k\rho g}{\eta} \frac{dh}{dL}$$

in which $\frac{dq}{dt}$ = rate of flow, A = sectional area, k = intrinsic permeability of the medium, ρ = density of fluid, g = gravitational acceleration, η = viscosity of the fluid and $\frac{dh}{dL}$ = pressure gradient. When referring to a particular fluid, the term $\frac{k\rho g}{\eta}$ is usually replaced by K , the coefficient of permeability of the medium to the fluid being referred to, so that the above equation is simplified as

$$\frac{dq}{dt} \frac{1}{A} = K \frac{dh}{dL}$$

In actual practice, the term K is determined directly by tests.

When the above law is applied to concrete, however, the following difficulties are encountered:

- (1) There are several different types of pores in concrete: gel pores, capillary pores and air voids in the hardened cement paste and pores in the rock aggregate, each of different pore size and having a different porosity and therefore contributing differently to the overall permeability of the concrete.

- (2) The spatial distribution of pores is not uniform; it is well known that the porosity of the hardened cement paste within the transition zones is much higher than that of the bulk of the hardened cement paste.
- (3) In reality, the permeability of a porous system is dependent not only on the porosity of the medium, but also on the connectivity of the pores. Among the various types of pores in concrete, the air voids are usually isolated (except in honeycombs) while the capillary pores are generally interconnected in the form of capillaries; even at the same porosity, they contribute quite differently to the permeability of concrete.

Despite the above difficulties, it may be said that the pores relevant to the water permeability of concrete are those that are interconnected and with a size of at least 120 nm (Neville, 1995). Hence, the gel pores (about 9 nm in diameter) and air voids (mostly isolated) may be ignored. The pores that contribute most to the permeability of concrete are the capillary pores.

The relation between the permeability and capillary porosity of a cement paste is shown in Figure 1.

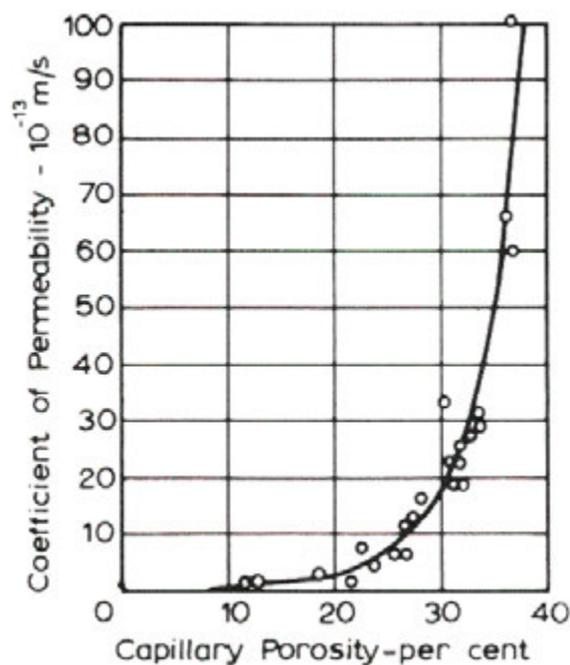


Figure 1 Effect of capillary porosity on permeability (Powers, 1958)

The capillary porosity is in turn dependent on the water/cement (W/C) ratio and the degree of hydration. When fully matured, the permeability of a cement paste is mainly a function of the W/C ratio, as shown in Figure 2.

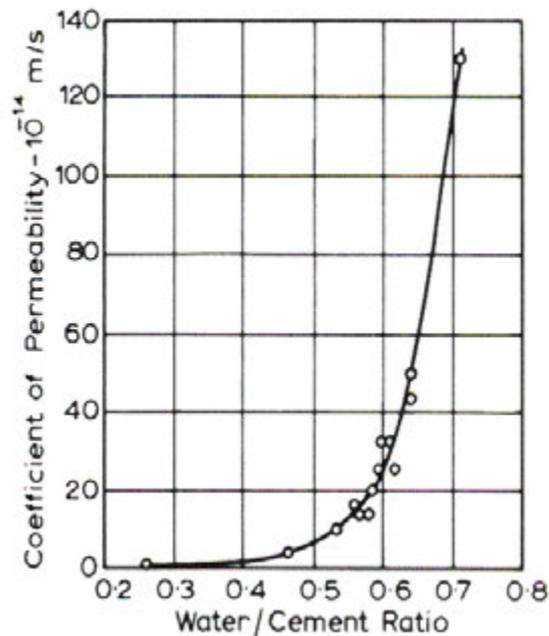


Figure 2 Effect of water/cement ratio on permeability (Powers et al, 1954)

It can be seen from the above that the permeability of a fully matured cement paste would be reduced from about 25×10^{-14} m/s to around 9×10^{-14} m/s when the W/C ratio is decreased from 0.6 to 0.5. Hence, the permeability of a cement paste may be effectively reduced simply by using a lower W/C ratio.

Table 1 Comparison between permeability of rocks and cement paste (Powers, 1958)

Type of rock	Coefficient of permeability (m/s)	W/C ratio of mature paste of the same permeability
Quartz diorite	8.24×10^{-14}	0.42
Marble	2.39×10^{-13}	0.48
Marble	5.77×10^{-12}	0.66
Granite	5.35×10^{-11}	0.70
Sandstone	1.23×10^{-10}	0.71
Granite	1.56×10^{-10}	0.71

However, it must not be forgotten that the rock aggregate is also to some extent permeable and the overall permeability of concrete is dependent on both the permeability of the hardened cement paste and the permeability of the rock aggregate. The coefficient of permeability of some common rocks and the corresponding W/C ratio of cement pastes having the same permeability are listed in Table 1. It is noteworthy that the permeability of most rocks is of the order 10×10^{-14} to 100×10^{-14} m/s, which is similar to that of hardened cement paste.

Of particular interest is that the permeability of granite, a commonly used rock for aggregate in Hong Kong, turns out to be higher than 1000×10^{-14} m/s, which is many, many times higher than that of hardened cement paste. The quality of the only type of rock available and in fact 100% in use for aggregate in Hong Kong is not really that high from the durability point of view. Please, therefore, do not blame the Hong Kong engineers if the durability of the reinforced concrete structures in Hong Kong is not meeting your expectation. The Hong Kong engineers do not really have any other alternative, unless the people of Hong Kong is prepared to pay for the high cost of importing better quality aggregate from elsewhere. Specification writers should also bear in mind that, regardless of whether you like it or not, there is a certain physical limitation to the durability standard that can be met in Hong Kong; if physically impossible or impracticable standards are specified, there will be endless multi-million dollars claim cases to be resolved at the end.

On the other hand, the diffusion of gases/vapours/ions through a porous medium generally follows the Fick's law:

$$\frac{dq}{dt} \frac{1}{A} = D \frac{dc}{dL}$$

where $\frac{dq}{dt}$ = rate of mass transport, A = sectional area, D = coefficient of diffusivity of the medium and $\frac{dc}{dL}$ = concentration gradient.

As in the case of permeation, the diffusion of gases/vapours/ions through a porous medium is dependent on the size, porosity and connectivity of the pores. Since the same geometric factors influence both permeation and diffusion, permeability and diffusivity are inter-related and it may be said that when the permeability is high the diffusivity is also likely to be high and vice versa. In fact,

there are test results showing that they are roughly proportional to each other (Lawrence, 1984). Consequently, they are often loosely treated as equivalent.

However, unlike permeation, the diffusivity of gases or ions through concrete or any porous medium is quite sensitive to the relative humidity or the degree of saturation of the pores. The diffusion of gases, such as oxygen and carbon dioxide, through dry pores is much faster than through wet pores or through the pore water. In contrast, the diffusion of ions, such as chlorides and sulphates, takes place only through the pore water and thus ionic diffusion is possible only when the pores are saturated or at least partially saturated.

Both the permeability and diffusivity are greatly affected by curing. An example is shown in Figure 3. Good and prolonged curing substantially reduce the permeability and diffusivity of the concrete within the surface zone, which protects the steel reinforcing bars from the aggressive environment and from corrosion. Hence, the importance of adequate curing cannot be over-emphasized. The existing common practice of removing the formwork within a short time in order to speed up the construction is not really appropriate.

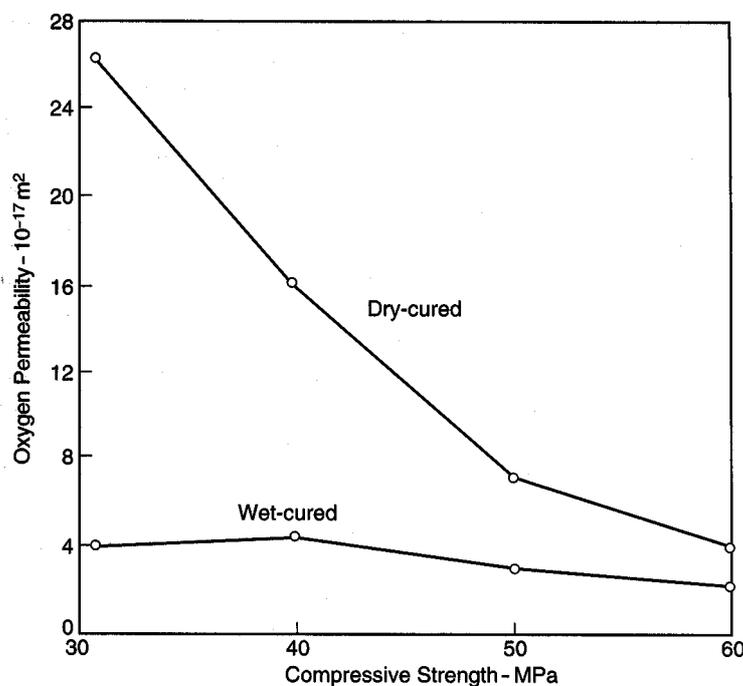


Figure 3 Effect of curing on oxygen permeability (Bassat et al, 1990)

3. Carbonation and Chloride Attack

Carbonation, i.e. the reaction of the alkalis in concrete with the carbon dioxide in air, does not cause deterioration of concrete but it has important effects on the durability of reinforced concrete structures. It gradually reduces the alkalinity of concrete to a pH value of about 9 and once the pH of the concrete surrounding the steel drops to below 10, the steel will become de-passivated and if water and oxygen are available the steel will start to corrode.

Carbonation takes place even at small CO₂ concentrations such as are present in rural air, where the CO₂ content is about 0.03%. In an unventilated room, the content may rise to above 0.1%. In large cities it is on average 0.3% and exceptionally up to 1%. The highest concentration of CO₂ is probably encountered by the lining of vehicle tunnels. Carbonation occurs progressively from the outside of concrete exposed to CO₂ but does so at a decreasing rate because CO₂ has to diffuse through the pore system, including the already carbonated surface zone of concrete. Such diffusion is a slow process if the pores in hydrated cement paste are filled with water because diffusion of CO₂ in water is 4 orders of magnitude slower than in air. On the other hand, if there is insufficient water in the pores, CO₂ remains in gaseous form and does not react with the hydrated cement. It follows that the rate of carbonation depends on the moisture content of the concrete.

The highest rate of carbonation occurs at a relative humidity of around 70% (very close to the yearly average ambient relative humidity of Hong Kong). Under steady conditions, the depth of carbonation increases in proportion to the square root of time, as depicted by the following equation:

$$d_c = C\sqrt{t}$$

in which d_c = depth of carbonation, C = carbonation coefficient and t = time of exposure. Apart from the environmental conditions, the carbonation coefficient is dependent mainly on the quality, particularly the diffusivity, of concrete. Broadly speaking, in concrete with a W/C ratio of 0.6, a depth of carbonation of 15 mm would be reached after 15 years, but in concrete with a W/C ratio of 0.45, the same depth of carbonation would not be reached until after 100 years.

Since the diffusivity is a function of the pore system of the hardened cement paste, the type of cement, the W/C ratio and the degree of hydration are the relevant

factors influencing carbonation. As these factors influence also the strength of concrete, it is often said that the rate of carbonation may be simply taken as a function of the strength grade of concrete. Whilst this postulation applies quite satisfactorily to laboratory cured concrete, in field cast concrete, which is usually provided with less than ideal curing, the curing conditions also have great effects on the rate of carbonation, as illustrated in Figure 4.

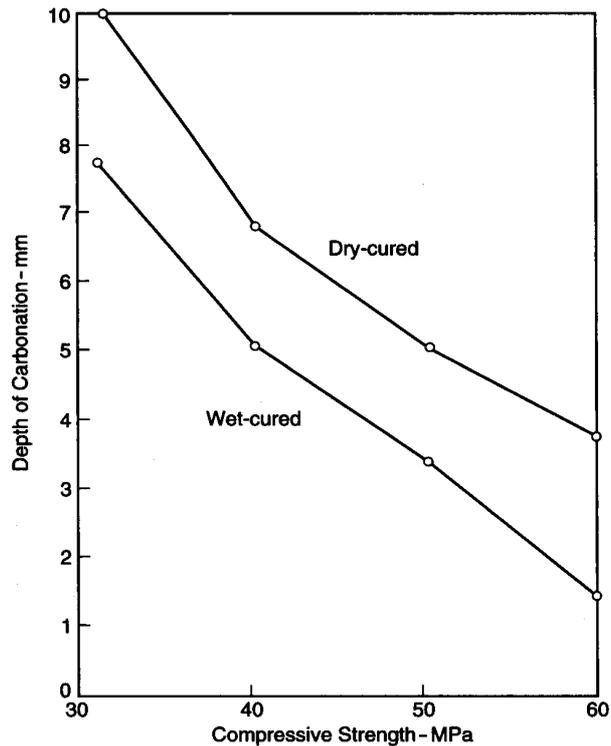


Figure 4 Effect of curing on carbonation depth (Bassat et al, 1990)

With mineral admixtures (also called supplementary cementitious materials), such as pulverized fuel ash (PFA) and condensed silica fume (CSF), added to the concrete mix, the contents and dosages of mineral admixtures also have significant effects on the rate of carbonation. Mineral admixtures are basically pozzolanic materials, which can react with Ca(OH)_2 to produce gel in the cement paste. Consequently, a concrete containing mineral admixtures will have a lower Ca(OH)_2 content in the hardened cement paste so that a smaller amount of CO_2 is required to carbonate all the Ca(OH)_2 and the depth of carbonation will tend to increase at a faster rate. On the other hand, however, the addition of mineral admixtures generally results in a denser microstructure of the hardened cement paste so that its diffusivity is reduced and the depth of carbonation will tend to increase at a slower rate.

The question is: which of the above two opposing effects is dominant? Conflicting test results have been obtained by different researchers and according to Neville (Properties of Concrete, 4th Edition, 1995), the key factor is the quality of curing. Since the pozzolanic reaction of mineral admixtures is generally slower than the hydration reaction of cement, concrete mixes incorporating mineral admixtures generally require longer periods of curing but the required curing was not always provided in previous tests thereby leading to misleading test results. No general conclusion can be made. But for concrete with up to 30% PFA added and with actual strength higher than 35 MPa, there should be no increase, or only a marginal increase, in depth of carbonation compared to that of concrete containing no PFA, if adequate curing has been provided. However, if the required prolonged curing has not been provided, the carbonation resistance of concrete containing PFA could be worst.

Like carbonation, chloride attack does not harm the concrete. Chlorides lead to corrosion of steel inside concrete and it is the corrosion that causes deterioration of reinforced concrete structures. There are two possible sources of chlorides: internal from the ingredients of the concrete mix and external from the environment. The use of ingredients containing chlorides for concrete is nowadays severely restricted and thus it may be assumed that the major source of chlorides is from outside.

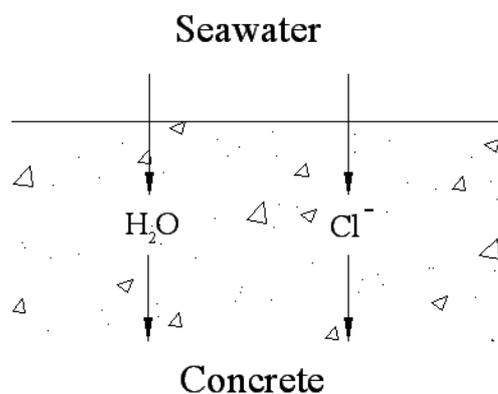
Two major types of structures are subjected to chloride attack: highway structures treated with de-icing salts and marine structures in contact with seawater. De-icing salts are not used in Hong Kong and thus only marine structures need to be considered here. However, land structures at close proximity to the sea have also been found to be contaminated with chlorides, which probably were raised from the sea by turbulence and carried by wind in the form of air-borne seawater droplets. Air-borne chlorides can travel substantial distances and up to 2 km has been reported (Neville, 1995). The first author had himself encountered this air-borne chloride problem at a distance of 30 m from the coast. Hence, it may be advisable to design all land structures within, say, 100 m from the seashore to a marine structure standard. Field tests are needed to find out the actual variations of the air-borne chloride content with the height of the sampling location and with the distance of the structure from the shore so that a more scientific solution can be worked out.

Whatever their external origins, chlorides penetrate concrete by transport of water containing the chlorides, as well as by diffusion of the chloride ions in the pore water and by absorption. When a reinforced concrete structure is permanently submerged in seawater, chlorides penetrate to a considerable depth and such ingress

can, with time, result in a high concentration of chloride ions at the surface of the reinforcing steel. However, unless oxygen is present, there will be no corrosion. Hence, marine structures that are permanently immersed in deep water, where the supply of oxygen is limited, do not in general have corrosion problems as serious as those in shallow water or within the tidal zone.

In a reinforced concrete structure that is sometimes exposed to seawater and is sometimes dry, the ingress of chlorides is progressive. When wet, the seawater and chlorides penetrate to a certain depth through the capillary pores and cracks in the concrete, and when dry, the water moves outwards and evaporates from the ends of capillary pores and cracks open to the ambient air. It is, however, only pure water that evaporates, leaving behind the chlorides in the concrete. Thus, the concentration of chlorides in the pore water increases near the surface of the concrete. The concentration gradient so established then drives the chlorides in the pore water near the surface towards the interior zones of lower concentration by diffusion. In effect, the water moves outwards and the chlorides inwards, as shown in Figure 5. The next cycle of wetting brings more seawater and chlorides into the pores and the subsequent drying leaves the chlorides behind, thereby gradually building up the chloride concentration in the concrete. Added with the plentiful supply of oxygen during the dry periods, marine structures within the tidal and splash zones, which are subjected to regular wetting and drying, generally have the most serious corrosion problems.

When wet :



When dry :

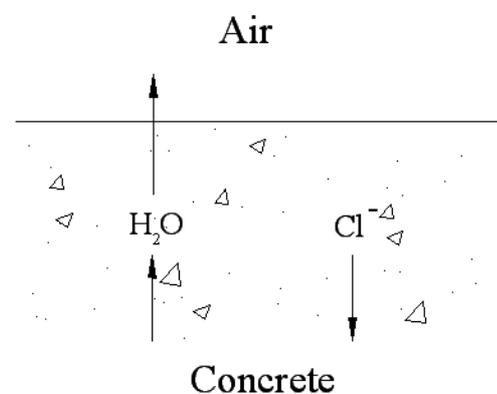


Figure 5 Ingress of chlorides in concrete subjected to regular wetting and drying

Given a sufficiently long period of time, say 10 or more years, depending on the quality of concrete and the thickness of concrete cover, the chloride concentration in the concrete at the surface of the steel reinforcing bars may build up to a level high enough to cause de-passivation of the steel (this level is called the threshold chloride level). A typical chloride content profile within the surface zone of concrete is shown in Figure 6. However, it must be borne in mind that the repeatability of chloride content profiles obtained from saltwater immersion or spraying tests is very low and the test results may not be representative of what happens in the real structure. The situation in the field is extremely complicated as tidal variation of seawater level, wind and sunshine can all affect chloride ingress and the variation in chloride content profile from one location to another of the same structure can be very large.

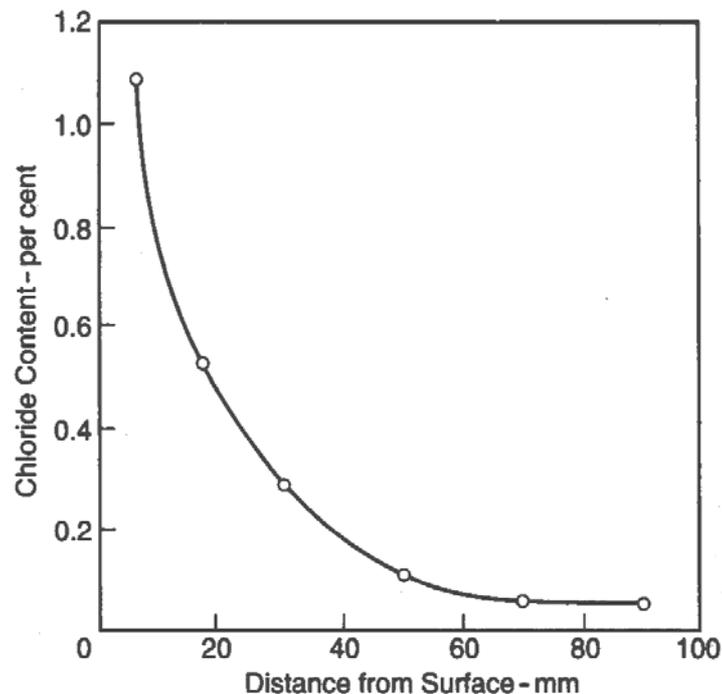


Figure 6 A typical chloride content profile in concrete (Neville, 1995)

Regarding the threshold chloride level, Neville (Properties of Concrete, 4th Edition, 1995) was of the view that it is difficult to establish a threshold chloride concentration level below which there is no corrosion. This threshold depends on a number of factors, which are still imperfectly understood. He also added that there may not exist such a fixed and unique threshold level at all. For practical purposes, the thickness and quality of the concrete cover to steel may be more important.

4. Improving Durability by Use of High-performance Concrete

The various adjustments to the mix parameters of a concrete that promote strength, such as lowering of W/C ratio and incorporation of CSF, generally also promote durability. Hence, there is the common postulation among engineers that *a stronger concrete is more durable* and therefore the use of a higher grade concrete is the simplest way to improve durability. Whilst some theorists like to dispute this postulation, this is in fact mostly the case except for the resistances against freezing and thawing action and certain forms of chemical attack. Nevertheless, to play safe, both strength and durability should be considered explicitly at the design stage.

As explained before, the carbonation resistance of concrete is influenced greatly by the mix composition of the concrete. The mix parameter having the greatest influence is the W/C ratio because it affects directly the pore structure and diffusivity of the concrete. Generally, lowering of the W/C ratio can substantially increase the carbonation resistance. Since the W/C ratio also influences the strength of concrete, the carbonation resistance may be correlated to the strength of the concrete, as presented in Table 2, where it is evident that the use of a higher grade concrete can effectively improve the carbonation resistance.

Table 2 Correlation between depth of carbonation and strength (Parrott, 1987)

28-day strength (MPa)	Depth of carbonation after 30 years (mm)
20	45
40	17
60	5
80	2

The presence of mineral admixtures also has some effects on the carbonation resistance. With good curing, the addition of PFA or GGBS (ground granulated blastfurnace slag) should have only marginal effects on the carbonation resistance. However, the addition of these mineral admixtures would demand better and longer periods of curing and if such curing is not provided the carbonation resistance may become worst. On the other hand, the addition of CSF would significantly increase the carbonation resistance because of two reasons: (a) CSF is finer than PFA and GGBS and is thus more effective in densifying the microstructure of the cement paste matrix, (b) a smaller dosage of CSF is generally needed and hence the reduction in lime content is smaller (Sulapha et al, 2003).

The chloride resistance of concrete is governed primarily by the pore structure and diffusivity of the concrete. Any factor that improves the pore structure and reduces the diffusivity of concrete would increase the chloride resistance. Therefore, lowering of W/C ratio and addition of mineral admixtures, such as PFA, GGBS and CSF, would all improve the resistance of concrete against chloride attack. The effects of adding mineral admixtures, especially those of high fineness, are two-fold. Firstly, incorporation of mineral admixtures of high fineness improves the packing of the cementitious materials and reduces the pore volume and size in the bulk of cementitious powders. Secondly, the silica content of the mineral admixtures reacts with the lime in the concrete to form further gel products (this is known as pozzolanic reaction) thereby converting the soluble lime (which tends to leach out leaving behind pores in the concrete) to insoluble gel and reducing the porosity of the concrete. Among these mineral admixtures, CSF is particularly effective. It has been shown that the addition of 9% CSF would reduce the chloride diffusivity by a factor of about 5 (Gjorv et al, 1994). There is also the view that as the beneficial effects of the various mineral admixtures are so significant, their use in reinforced concrete liable to corrosion in hot climates should be made mandatory and Portland cement alone should not be used (STUVO, 1986).

Summing up, it may be said that the durability of a reinforced concrete structure may be improved by changing to the use of a high-performance concrete, or more specifically, a higher strength concrete incorporated with various kinds of mineral admixtures. A successful example in Hong Kong is the Recommended Specification for Reinforced Concrete in Marine Environment published in the newest Port Works Design Manual: Part 1, which is summarized in Table 3.

Table 3 Specification for marine concrete in Port Works Design Manual

Mix parameter	Acceptable limits
Water/cementitious content ratio	Not exceeding 0.38
Cementitious content	380 – 450 kg/m ³
Supplementary cementitious materials	Either PFA or GGBS, and CSF to be incorporated
If PFA added, PFA content	25 – 40 %
If GGBS added, GGBS content	60 – 75 % (normal application) or 60 – 90 % (low heat application)
CSF content	5 – 10 %
Characteristic strength	45 MPa

5. Improving Durability by More Careful Crack Control

No matter how good the quality of the concrete is, if the concrete cracks extensively, the water-tightness of the concrete cover to the steel will be impaired and the durability of the concrete structure will be adversely affected. Unfortunately, material engineers tend to think that the control of cracking is the responsibility of structural engineers whilst structural engineers tend to think that it should be the responsibility of material engineers. Moreover, the consultants blame the contractors for bad workmanship while the contractors blame the consultants for inappropriate specification and design. At the end, everyone suffers and we are imparting a bad image to the general public. Compared to the effort that we have spent on improving the quality of concrete, we have not really done enough to control cracking of concrete, which is important not only from the durability point of view, but also for structural integrity, water-tightness and aesthetic.

Broadly speaking, there are two categories of cracks: non-structural cracks (also called restraint-induced cracks) and structural cracks (also called load-induced cracks). Non-structural cracks are due to sedimentation, shrinkage or thermal movement of the concrete, which if restrained, would induce tensile stresses large enough to cause cracking. Structural cracks are due to tensile stresses developed in the concrete resulting from the applied loads.

Non-structural cracks can appear during the plastic stage, curing stage and long-term stage. Those appearing during the plastic stage include plastic settlement cracks (due to external restraint of the sedimentation movement of the fresh concrete mix by the reinforcing bars and the formwork) and plastic shrinkage cracks (due to self-restraint of the drying shrinkage movement of the top surface of the fresh concrete because of rapid evaporation). Plastic cracking can generally be avoided by improving the mix design of the concrete (basically reducing the water content and increasing the fine powders content) so as to reduce sedimentation and shielding the top surface of the fresh concrete from rapid evaporation.

Non-structural cracks appearing during the curing stage are mainly early thermal cracks. They are due to internal or external restraints against the thermal movement of the concrete as the temperature of the concrete changes due to the heat generated from the chemical reactions of the cementitious materials. Internal restraint (also called self-restraint) is the major cause of cracking in massive concrete structures while external restraint is the major cause in concrete structures cast against

rigid movement restraints. If internal restraint is the major cause, then insulation would help to reduce thermal cracking. However, if external restraint is the major cause, no insulation should be applied as insulation would actually aggravate the problem. Many engineers do not distinguish the two types of restraints and specify insulation to be applied in all cases, which is wrong and is the root cause of many early thermal cracking problems in Hong Kong (Kwan and Ng, 2004). In any case, regardless of whether internal and/or external restraints exist, internal cooling by air/water/liquid nitrogen would always help to mitigate the early thermal cracking problem, but most contractors do not like this because of the trouble and cost involved, especially if the cooling is not separately priced for as a BQ item by itself.

Non-structural cracks appearing during the long-term stage include thermal movement cracks and shrinkage movement cracks. They are due to the thermal/shrinkage movement of the concrete structure being restrained by rigid walls or supports. One way of alleviating the problem is to provide movement joints. If shrinkage movement is the major cause of cracking, then the provision of late-cast strips and the addition of shrinkage reducing agent to the concrete should also be considered as possible mitigation measures.

On the other hand, structural cracks are almost unavoidable, unless the structure is to be completely redesigned or prestressed, which is usually not practicable. Nevertheless, the crack widths can be controlled by limiting the tensile stresses developed and putting in crack control reinforcement.

Lastly, the authors would like to emphasize that crack control is not a simple matter. We need to rewrite many of the existing specifications, train the site staff about appropriate methods of curing, develop methods of temperature control during curing, and conduct more research on thermal and shrinkage movement analysis of concrete structures. Quite sadly, however, this topic seldom receives its deserved attention partly because cracking rarely leads to immediate danger and partly because many researchers who never go to site are ignorant of the problem.

6. Improving Durability by Use of Corrosion Inhibitors

Certain chemicals, if incorporated into the concrete during mixing so that they become part of the concrete mass, have the capability of inhibiting steel corrosion. Nitrites of sodium and calcium have been found to be effective in laboratory tests

(Neville, 1995). The action of the nitrite is to convert ferrous ions at the anode into a stable passive layer of ferric oxide (instead of ferrous oxide which is less resistant to chloride attack), with the nitrite ions in the concrete consumed for reacting preferentially to the chloride ions. The concentration of nitrites must be sufficient to cope with a continuing ingress of chloride ions. Indeed, it is not certain whether corrosion inhibitors are effective indefinitely and do not simply delay corrosion. Other chemicals, which may also be used as corrosion inhibitors, are being searched and tested for their effectiveness (Nmai et al, 1992).

It must be emphasized, however, that inhibitors are no substitute for good quality concrete (concrete of low permeability and diffusivity); they serve only as an additional safeguard. There are already corrosion inhibitors available in the local market. According to the supplier, the level of corrosion protection increases in proportion to the dosage, which should be sufficient to offset the anticipated chloride ion content of the concrete over the design life of the structure. It should also be noted that the effectiveness of a corrosion inhibitor could vary with the type of cement used, the quality of the concrete, the thickness of cover provided and probably also the environmental conditions. Field trials are needed to evaluate the effectiveness of various corrosion inhibitors under the local conditions.

7. Improving Durability by Use of Protective Coatings

Application of protective coatings, which act as surface barriers to stop ingress of aggressive fluids/ions, to the surface of concrete is an effective measure for improving the durability of reinforced concrete structures. Ideally, if affordable, the protective coatings could be applied to all surfaces of the concrete structure. If, however, for sake of economy, the protective coatings are to be applied only to certain parts of the concrete structure where the corrosion is expected to be more serious, then careful consideration is needed because application of a barrier at the anode rather than the cathode would increase the ratio of the size of the cathode to the anode and consequently increase the rate of corrosion (Neville, 1995).

Other considerations concerning the application of coatings include special surface preparation that may be required, UV resistance of the coating if exposed to sunshine, and crack bridging capability of the coating. It should be advisable to always carry out field trials before embarking on the use of any new product. Some coatings may not last the whole design life of the concrete structure and the necessity

of re-applying the coating after certain period of service should be taken into account in the life cycle cost analysis of the structure.

Protective coatings may also be applied directly to the steel reinforcing bars. Coatings that have been in use include galvanizing zinc and epoxy. Zinc coatings have only limited effective life because the zinc is offering corrosion protection by serving as a sacrificial (and therefore consumable) anode. Epoxy coatings do not have this problem but the reinforcement details need to be redesigned to cater for the reduction in bond strength.

8. Concluding Remarks

As implied by the title "... theory vs practice", which was set by the SCCT rather than by the authors, the durability of concrete needs to be dealt with by both theoretical studies and practical evaluations. Whilst academics in universities can help to carry out theoretical studies and controlled experiments in laboratories, engineers may contribute more to field tests and monitoring which are necessary to evaluate the actual performance of the various possible durability improvement measures. Deep collaboration between university and industry is the key to the ultimate solution. Academics should go to site more often to improve their understanding of the practical aspects of the problem whereas the industry could provide more opportunities for field trials and perhaps sponsorship for research.

More specifically, future research and development works should focus on the concrete itself, curing and temperature control methods, control of cracking, and various methods of corrosion protection. Regarding the concrete, the newly adopted specification for marine concrete is a good starting point. Since the grade of the concrete produced as per the specification requirements should be at least 50, the marine concrete might be better specified as a Grade 50 concrete. Further studies aiming to develop reliable test methods for measuring the carbonation resistance and chloride diffusivity of concrete are needed so as to pave a way for gradual migration to performance specification (the current prescriptive specification is simply too difficult to check for compliance). The existing specifications for curing, particularly those on period of curing and temperature control, also need to be reviewed and, if necessary, redrafted. Lastly, control of cracking and methods of corrosion protection are for long-term research because only incremental advancement based on feedback from field applications can be achieved.

Acknowledgement

The theory part of this paper is based largely on Neville's famous book on concrete technology (Neville, Properties of Concrete, 4th Edition, 1995), which contains almost everything about concrete and should be referred to if readers are interested in in-depth knowledge of concrete.

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