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fib
CEB-FIP

Structural Concrete

Textbook on behaviour, design and performance

Second edition

Volume 3

Structural Concrete

Textbook

on behaviour, design and performance

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December 2009

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Manual, Guide (to good practice) or Recommendation	approved by the Technical Council of ***fib***
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Preface to Chapter 5

This document seeks to give an appreciation of the wide range of issues associated with the design of durable concrete structures and matters concerned with service life design. The material reported here is drawn from many sources. Hopefully all of which have been adequately acknowledged. The document attempts to reflect recent and evolving developments and thereby give an up to date overview on matters concerned with service life design.

The document contains 19 main sections. Section 1 provides an introduction and sets out the scope of the issues, providing an overview of the document. It notes that modern concrete forms a family of materials and seeks to indicate why is durability of concrete structures is important. It reviews previous experience in terms of the in-service performance of concrete structures and the need to take a holistic view when creating durable concrete structures. Section 1 also notes the importance of variations in concrete properties and the durability critical role of the cover concrete, it outlines some simple conceptual models for the deterioration caused by the corrosion of reinforcement. It introduces the benefits of considering through-life performance, whole-life cost and sustainability perspectives when undertaking outline service life design and developing durability and service life design concepts. Section 1 provides definitions and terminology, before giving an overview of the various approaches to service life design and drawing parallels between contemporary structural and probabilistic based service life design concepts.

Section 2 goes on to provide an overview of the service life design, construction and through-life care process. It examines the role and importance of the client brief and the definition of performance expectations, before headlining environment aggressivity classification, conceptual and detailed design for durability and the deemed-to-satisfy durability solutions design given in codes and standards. Section 2 introduces the concept of probabilistic performance-based service life design, before examining the way project specifications can be used by client / owner's as a tool to achieve the desired durability and the importance of the execution of the construction works upon through-life care and maintenance aspects.

Section 3 examines through-life performance, whole-life cost and sustainability issues seeking to draw out wider societal sustainability perspectives, as well as consideration of whole-life cost issues.

Section 4 reviews mechanisms that may cause deterioration or damage to concrete structures; providing an overview of deterioration and damage mechanisms (but excludes damage arising from accidental actions) and outlining at the role of water and moisture transport mechanisms in this. Section 4 looks in detail at mechanisms that may cause deterioration or damage classifying these into four groups; those causing physical deterioration and damage processes in concrete; those associated with chemical deterioration processes; those associated with biological deterioration processes in concrete and the corrosion of reinforcement. Consideration is briefly given to deterioration mechanisms acting in combination.

Section 5 reviews some factors which influence the durability of concrete structures. These include the geometrical form and architectural detailing of the structure; the cement type, mix design and concrete quality; the reinforcement type and the depth and quality of the concrete cover; together with the potential influence of cracking, crack width and crack orientation.

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Section 6 examines issues associated with environmental aggressivity looking at moisture driven deterioration processes, atmospheric induced deterioration, the role of temperature induced effects and a contemporary system for the classification of environmental exposure.

Section 7 reviews the recommendations made in some standards and codes of practice; looking at those given in the CEB-FIP Model Code 1990, the recent *fib* Model Code for Service Life Design 2006 (published in *fib* Bulletin 34); together with the guidance given in EN 1992: Concrete structures (Eurocode 2) and associated product standards. The potential approach and anticipated content of the *fib* Model Code 2010 is also examined.

Section 8 provides a brief overview of modelling of deterioration processes, with the main focus being upon carbonation and chloride induced corrosion of reinforcement in uncracked concrete. Consideration is also given to the modelling of other mechanisms of deterioration such as frost attack, sulfate attack, alkali-aggregate reaction (AAR), leaching and surface weathering and abrasion by ice. The application of deterministic and probabilistic models is discussed, together with the partial factor method. The results for reinforcement corrosion obtained from a deterministic service life design model are compared with those from a probabilistic model.

Section 9 explains the use of the factorial approach to estimating service life, making reference to a process used for combining additional protective measures to extend service life.

Section 10 examines the service life design process and related considerations outlining the main steps in a service life design procedure, the approaches which may be adopted for detailed service life design, together with the definition of the target service life and environmental aggressivity. The various steps in the overall service life design procedure are introduced, but with more detailed discussion being provided about the adoption of a multi-layer protection approach, with an example of this concept being given for multi-layer protection of pre-stressing tendons (as given in *fib* Bulletin 33). Additional observations are for durability and service life design issues for environments where de-icing salt is applied and for structures in a marine environment. The concepts associated with 'Birth certificate' documentation and reliability updating are also explained.

Section 11 presents detailed recommendations for measures to enhance resistance or avoid reinforcement corrosion on the basis of the selection of cementitious materials, the use of admixtures and fibres, ways of enhancing the resistance of the surface of the concrete, ways of enhancing tolerance to carbonation and chlorides, together with the avoidance approach (i.e. the 'design out' approach). The section concludes with an overview of measures to enhance resistance or avoid reinforcement corrosion.

Section 12 summarises some measures to enhance resistance or avoid forms of deterioration other than reinforcement corrosion.

Section 13 examines the influence of some design, execution and workmanship issues upon durability of concrete structures. The matters considered include the influence of locally available concrete materials and labour, the dimensioning of structural elements, reinforcement detailing and congestion, the compaction and curing of concrete, the use of controlled permeability formwork (CPF), self-compacting concrete (SCC), high performance concrete (HPC); plus the durability problems associated with spacers and chairs to support reinforcement and the use of inserts and fixtures. The benefits arising from the use of stainless steel reinforcement are discussed, along with the role of the use quality assurance and quality control and the potential importance of aesthetics and appearance issues.

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Section 14 examines construction quality issues and, in particular, the role of the project execution specification in creating greater certainty of achieving durable concrete structures. The approach adopted is centred on the European construction standards and the CEN Execution Standard, with observations being made about how these link with the quality management standards contained in the ISO 9000 series.

Section 15 discusses the potential benefits of pre-construction planning and trials as a means of improving the durability of concrete structures, particularly upon major infrastructure projects. Six steps to delivering improved certainty of achieving durable concrete construction are set down.

Section 16 describes the condition control processes that may be adopted for planned through-life structure management and care, linking these into an overall process of condition survey and monitoring activities, condition assessment and condition evaluation. These are based upon the concepts developed in the work undertaken for *Chapter 9: Conservation* of the future *fib* Model Code 2010. This section outlines the role of Classes of condition control within the process of through-life management of a structure, describing the associated Condition control levels / inspection regimes. Whilst a strategy using proactive condition control measures is recommended for new concrete structures, the approach outlines allows for reactive condition control measures and for situations where condition control measures are not feasible (i.e. foundations).

Section 17 describes methodologies for the monitoring of durability and performance of a concrete structure. It provides advice upon the locations for surveys, testing and monitoring activities, condition survey and monitoring activities, tools and techniques for surveys and monitoring and the gathering data for condition control purposes. As with Section 16, the approach is based upon the concepts developed in the work undertaken for *Chapter 9: Conservation* of the future *fib* Model Code 2010. Section 17 also briefly considers issues associated with the automated monitoring of concrete structures and how these techniques can link to the updating of service life predictions.

Section 18 presents details of two service life design examples from practice; the Great Belt Fixed Link, Denmark and the Western Scheldt Tunnel, the Netherlands.

Section 19 provides a future look examining some potential developments which may influence durability and service life design in the future. The topics considered include the forthcoming *fib* Model Code 2010, the need for improved scientific understanding of deterioration processes, anticipated developments in cementitious and concrete materials, as well as the further development and application of service life design principles.

The document provides an extensive list of references and bibliography, as well as a list of abbreviations and a glossary.

Stuart L. Matthews, Chair of *fib* Commission 5

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5 Design of durable concrete structures

5.1 Introduction

5.1.1 Background

Concrete is the most extensively used construction material (in terms of annual tonnage used). It has been used successfully for an innumerable number of buildings and structures for almost 2000 years. It has shown that it can be a very durable material. For example, the Pantheon in Rome was rebuilt in the early 2nd century (completed 126 AD) utilising Roman concrete. The building acted originally as a Roman temple and is still being used today as a working church (Fig. 5.1-1). However, it should not be assumed that concrete has everlasting properties and that all concretes will be durable in all circumstances. These issues are the focus of this document.

Fig. 5.1-1: The Pantheon, Rome

The introduction of reinforcing steel dramatically improves the tensile strength of concrete, enabling slender structures with much longer spans and higher working stresses to be built. Most of today's concrete constructions rely on the composite interaction of concrete and steel, which is aided by the near equivalence of their thermal expansion characteristics.

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Structural (reinforced) concrete made from Portland cement has dominated the construction environment for the last 100 years or so. One of the reasons for this success is that concrete is very adaptable in that it can be cast into almost any shape or form. Today concrete provides a unique building material for the majority of structures world-wide.

With correct design, specification and construction concrete structures provide high performing durable assets with long service lives. The vast majority of concrete structures perform satisfactorily and are adequately durable for their service environment.

It is clear that the future development of the world economy will require significant investment in a wide range of assets and buildings and other industrial facilities in many different geographic areas and climatic zones, but particularly in energy generation and transport infrastructure. This will involve the creation of a diverse range of long-life, and in some cases ultra-long life, assets. Energy generation and transport infrastructure will potentially include facilities such as nuclear power plants, barrages for electrical energy generation, offshore wind turbine farms, bridges, tunnels, structures for offshore installations, and many more – all of which will be situated in extremely aggressive environments and will be subject to very demanding performance requirements. Durability will be an important consideration.

These facilities will require a diverse range of concrete structures, which will involve the use of large volumes of different types of concretes. In these developments it will be important to get an appropriate balance between potentially conflicting requirements associated with matters such as functional performance, adequate durability (to achieve the long design service-life needed with many types of infrastructure), long-term holistic sustainability issues, whole-life cost and effective through-life management of these assets. With long-life assets it is necessary to achieve a different balance between these influencing factors to that associated with other forms of construction which have a shorter-life and lower expectations for long-term durability, such as housing and conventional buildings.

Thus around the world concrete has provided a very adaptable means of creating the facilities and assets necessary to meet the needs of society. This has been done largely using the locally available material resources (i.e. sand and rock as fine and coarse aggregates, water etc). This is a great advantage. Thus concrete can be tailor-made for a great variety of uses; it can be made almost anywhere and from a host of indigenous materials. So far concrete has been made mainly using Portland cement as the binder.

Whilst such concrete has a significant amount of embodied carbon dioxide (CO_2), as the manufacture of Portland cement involves sizeable CO_2 emissions, the future use of alternative cements (which are currently under development) could reduce this substantially. These changes and developments will undoubtedly pose new and different challenges in respect of the design of durable concrete structures. These issues reflect the developing tension between global goals and more local societal needs and aspirations.

This document on durability aims to provide a basic understanding of the complex set of phenomena governing durability and long-term performance of concrete structures and how this forms a basis for service life design. Whilst consideration is given to concrete as a material, the focus is upon the behaviour of the concrete structure and its interaction with its environment. This document should not only assist the designer to improve the future durability performance and reliability of concrete structures, but should also assist engineers involved with the assessment, maintenance and extension of life of existing concrete structures.

Details on concrete technology, material properties and deterioration mechanisms have been described in detail in Section 3.1. Cracking and crack control have been described in

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Section 4.3.2. These chapters include essential information related to durability. Chapter 8 is concerned with the assessment, maintenance and repair of existing concrete structures.

5.1.2 Concrete – A family of materials

Considerable developments in concrete technology over recent years have created an extended family of different concrete types which have very different properties, characteristics and performances; for example concretes can be made with an intended compressive strength which ranges from as little as 2 N/mm^2 to over 200 N/mm^2, that is by two orders of magnitude. Such concretes are used in very different applications.

These technical developments have effectively created a range of new concrete materials which include high strength concretes, high performance concretes, lightweight concretes, gas concretes, high density concretes, flowable and self-compacting concretes, coloured concretes, concretes made with recycled and/or waste materials from other industries, fibre reinforced concretes, concretes using corrosion resistant metallic reinforcement, concretes using non-metallic reinforcement or externally bonded reinforcement, and so on.

Future developments in concrete materials are expected to involve new cements and other forms of binder which provide alternatives to the ubiquitous Portland cement we use now. These materials would have significantly lower carbon emissions and environmental impacts associated with their manufacture (i.e. their embodied CO_2 would be lower). The performance and durability implications of these new binders have yet to be fully evaluated.

There have also been considerable technical changes associated with the use of concrete as a structural material; such as the evolution of design concepts and analytical methods, and allied advances in construction procedures, reinforcement and prestressing systems which all have a bearing upon achieving adequate durability for the required performance of the structure concerned.

5.1.3 Why is durability important?

The problems which can arise when concrete structures lack sufficient durability and fail prematurely are dramatically illustrated by the structural failure of the parking deck shown in Fig. 5.1-2. The failure was caused by the deterioration of the embedded steel reinforcement in the concrete structural elements which were not adequately durable for the harsh environment associated with a parking structure.

In most instances a lack of durability does not produce such a dramatic outcome, with the attendant threat to life safety. However, a lack of durability potentially has economic implications for society, both from direct costs associated with repair of the damage, but also from the indirect losses and disruption to business and society. Fig. 5.1-3 shows the less dramatic image of delamination and spalling of a concrete slab soffit caused by reinforcement corrosion. However, this would still be expensive to repair and disruptive to building users.

A lack of durability potentially also has significant environmental impacts in a variety of ways, such as through the need to undertake repairs which use more virgin materials and other resources, uses more energy and results in more CO_2 emissions, creates additional waste materials – some of which may need to go to landfill, causes disruption and supplementary environmental impacts associated with the disruption to society caused by repair works, etc.

Because of these considerations concrete structures, along with other forms of infrastructure upon which our society depends, commonly need to achieve high levels of

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reliability and long service lives. Thus it is essential that mechanisms and circumstances causing deterioration are properly understood so that we adopt appropriate design methods, construction procedures and material compositions, as well as maintenance and repair procedures, to ensure we achieve adequately reliable structures in the future.

Fig. 5.1-2: Failure of parking deck due to deterioration of the concrete structure which was not sufficiently durable for the aggressive environment (image courtesy Arminox)

Fig. 5.1-3: Severe delamination and spalling of a concrete slab soffit caused by reinforcement corrosion [BRE Digest 444: Part 1 (2000)]

Although all structural materials deteriorate with time, with the rate and extent depending upon the environmental circumstances, structural concrete which is cast on site has some important characteristics which differ from the properties and performance of other structural materials. These characteristics include the following:

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- Assumptions are made during the design stage about the quality and the performance of the concrete which will be achieved in the finished structure. These assumptions cannot be verified until the concrete has been cast on site and there has been sufficient time for it to develop strength and for its other properties to have matured.
- The development of the concrete's properties through the hydration process takes time. For example, the compressive strength of a concrete may continue to increase for perhaps a year or more after casting, depending upon the mix design and various other factors. The characteristics which affect durability also change with time.
- The durability of the concrete is influenced by various factors including the mix design, the cement type and the amount of water added to the fresh concrete.
- The actual quality and performance characteristics of the concrete are determined by the execution process during construction on site. Hence, it is this very short period of time which constitutes a critical phase in ensuring that the finished structure meets the required durability and performance requirements. Thus it is necessary to ensure that all aspects of the execution processes are "done right first time", as it can be extremely difficult to overcome or compensate for durability critical deficiencies when the concrete structures is situated in an extremely aggressive environment.
- The concrete is subject to environmental effects which over time influence its durability and performance, with there commonly being an extended period when these environmental effects slowly interact with and change aspects of the internal environment of the outer layers of the concrete. For example, during the 'initiation' periods associated with carbonation and chloride ingress there is no visually observable change to the concrete. It is often assumed by 'lay observers' that there is no change or deterioration of the concrete in this phase; which is not correct.

Thus if the achieved durability performance of a concrete structure in an aggressive environment turns out to be sub-standard, this commonly is not apparent until some time has passed due to the nature of the deterioration processes that concrete structures generally experience. Premature deterioration of a concrete structure may not become apparent until the contractual liability periods of the contractor and the designer have passed. However, this period could be shorter than the notionally 'trouble-free' service life envisaged by the owner.

5.1.4 Previous experience – Reviews of the in-service performance of concrete structures

Various studies have been made into the in-service performance of some concrete structures (see below). These studies have given insight into aspects of some of the inadequacies in performance, lack of durability, types of deterioration and circumstances of their occurrence. They have provided understanding of some factors contributing to the observed problems and inadequacies; sometimes estimating the relative contribution of design and construction activities which caused the deficiencies. Taken together the studies paint a complicated picture, often presenting different views on the influencing factors and means by which they might be addressed.

The studies of in-service performance of concrete structures include the following:

- Idorn's 1967 study of the durability of concrete structures in Denmark;
- Hauser's 1979 study into the lessons to be learnt from European failures;

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- Fraczek's 1979 study into errors associated with concrete structures in the USA;
- Wallbank's 1989 study of the in-service performance of 200 UK concrete bridges;
- Clark et al.'s 1997 study of the quality of UK concrete construction;
- Jones et al.'s 1997 study of how to ensure the durability of new concrete construction;
- Bamforth et al.'s 1997 study of chloride ingress into structural concrete;
- Tajalli and Rigden's 2000 study of the performance of 40 UK concrete bridges;
- *fib* Bulletin 26's review in 2003 of the occurrence of stress corrosion cracking and associated failures within prestressed concrete construction located in Germany;
- Helland et al.'s 2008 study of chloride penetration into HSC & HPC structures.

These studies are reviewed in detail in Chapter 8, but here we will simply overview the recommendations made in these studies about the steps which might be taken to improve the in-service performance of concrete structures. It is reasonable to observe that a wide range of approaches have been recommended and tried over the years to reduce the incidence of specific problems and inadequacies which occur with some concrete structures.

However, it is worth noting at this point that numerous site surveys and studies have been conducted over the years into the causes of quality failings in concrete construction, particularly the failure to achieve the specified reinforcement cover: refer Marosszeky & Chew (1990), King & Dakin (2001) and Clark et al. (1997). These studies have shown that the failure to achieve the specified reinforcement cover does not solely arise from poor standards on site, but that the difficulties causing this outcome (i.e. this non-compliance with one of the key quality requirements of the process for reinforced concrete construction) can result from measures taken at all stages of the construction process. Fig. 5.1-4 presents a breakdown of the perceived occupational origin of these problems as established in the study by Clark et al. (1997).

It will be noticed that the origin of these problems is about equally divided between design and management processes and those associated with the operatives fixing the reinforcing steel or the formwork or placing and compacting the concrete. More specifically:

- Management, including the activities of designers and suppliers 47%
- Site operative activities 53%

Fraczek (1979) reports on a survey of consulting engineers and government agencies into errors associated with concrete structures in the USA. This work was conducted by American Concrete Institute Committee 348, which reviewed 277 cases of errors. The survey found that about three-quarters of the errors were discovered as a result of either distress of the affected structure (172 cases), or by the collapse of the structure (39 cases). The types of distress exhibited included cracking, spalling, deflection, settlement, rotation and leakage. The results suggested that of the errors studied about half occurred during the design stage and about half occurred during the construction stage. Each stage was responsible for about the same number of structural collapses.

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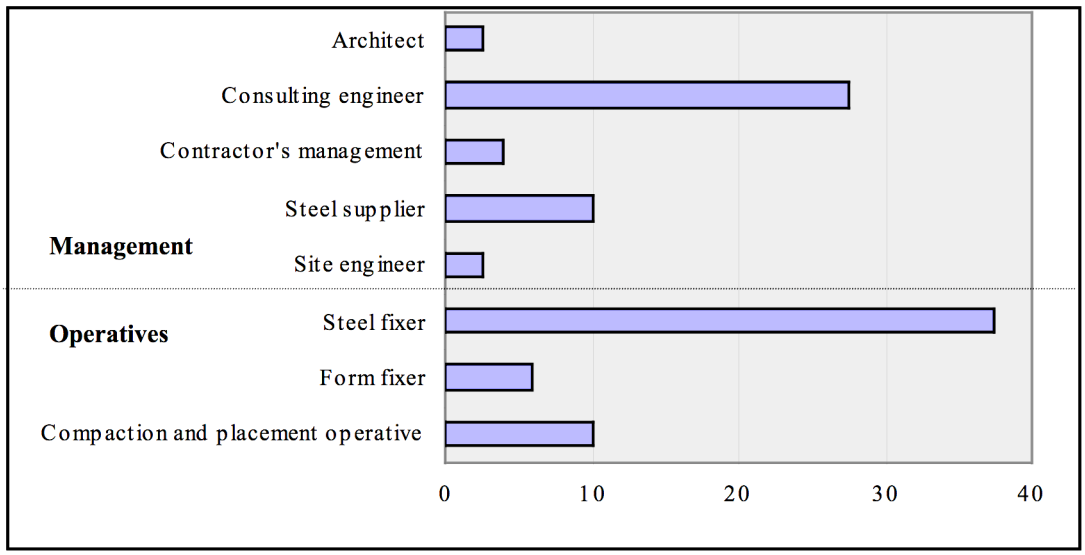

Fig. 5.1-4: Perceived occupational origin of the problems of achieving adequate cover depth in concrete structures [Clark et al. (1997)]

Of the errors due to design, most were discovered during in-service and most resulted in serviceability problems. Of the errors due to faulty construction, nearly three-quarters were discovered during construction and about half resulted in either structural distress or collapse. However, Fraczek (1979) notes that the survey undertaken was not a true reflection of the actual incidents of error occurring in the process of concrete construction. The errors reported in the study were the ones which escaped detection by the standard error detecting procedures used by industry in the overall design and construction process. Thus the picture presented is biased to reviewing the small proportion of errors which the normal industry review procedures failed to capture.

Hauser (1979), who reported on the lessons learnt from a study of some 800 European structural failures in which 592 people were injured and 504 were killed, suggested that the main issues associated with structural safety were a combination of ignorance and gross errors made by individuals. Of the 493 cases of structural failure where unfavourable conditions or circumstances had not been properly taken into account (either totally or incorrectly considered), Hauser (1979) found that the errors were attributable as follows:

- 36% to the design team, of which 8% (of the total) were attributable to the architect;
- 33% to the contractor;
- 11% to the actions associated with both the contractor and the engineer; and
- 20% to other causes, of which (5% of the total) related to the users of the structure.

Of the errors which occurred during the designing/planning phase of the works:

- 34% were due to errors made in the conceptual phase of design;
- 34% were associated with errors in structural analysis;
- 19% with the production of the drawings;
- 9% were associated with the planning of the construction works; and
- 4% were due to combinations of the above.

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Hauser (1979) suggests that most errors leading to structural failures are avoidable and that a small additional effort spent checking data and the planning of proposed activities would help reduce the number of problems considerably.

Thus these three studies spaced over two decades and involving several geographic areas appear to have come to largely similar conclusions about the origin of the problems which result in premature deterioration and potentially unplanned expenditure on maintenance and remedial works to concrete structures.

Jones et al. [BCA (1997)] reports the findings of a comprehensive study by the British Cement Association of in-service performance inadequacies and premature deterioration of concrete structures. This work was associated with the development of a holistic approach to ensuring the durability of new concrete construction, by utilising cycles of continuous improvement to tackle the underlying issues affecting durability.

Jones et al. [BCA (1997)] reviewed published data upon 271 cases of deterioration of concrete structures and found that 50% of the cases of deterioration considered were caused by corrosion of reinforcement due to either chloride penetration or carbonation. However, they felt that it was not possible to attribute these instances to any one member of the supply chain/construction team, because of the interaction of potential contributing factors. Amongst other findings, Jones et al. [BCA (1997)] attributed the causes of in-service performance inadequacies and premature deterioration to be as follows:

- 39% were due to environment effects;
- 16% were due to poor quality concrete;
- 13% were due to the selection of the wrong/an inappropriate material;
- 12% were due to inadequate/low cover depth;
- 8% were due to a failure of the waterproofing system;
- 7% arose from poor quality detailing;
- 4% were due to poor workmanship;
- 2% arose from poor construction specifications.

Jones et al. [BCA (1997)] contended that management efforts to control quality in construction are typically based upon the application of inspection and correction procedures. As a consequence, the causes of the defects within the construction process were not identified and therefore remained unresolved.

They make the observation that achieving good quality concrete structures which are durable is commonly impeded by the procurement process used, which typically treats the matter simply as the purchase of a ‘product’, rather than recognising and engaging in the ‘process’ which is required to deliver the desired construction outcome. The mind set associated with the purchase of ‘product’ (i.e. a finished item with the expectation of compliance with known or pre-determined performance characteristics) is inconsistent with the prototype nature of major construction projects. The problem with the ‘product’ approach, which is commonly cascaded down through the supply chain for the purchase of specific services, is that it ignores the interface issues between the different steps in the overall construction process and, in so doing, potentially the overall objective (e.g. producing a durable concrete structure).

Over the years various approaches which have been recommended and tried with varying degrees of success to reduce the incidence of specific problems and inadequacies which occur with some concrete structures. These have (broadly) included:

- recognition that concrete, although generally durable, is not necessarily a maintenance-free material in all environments and that it may require repair [Idorn (1967)];

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- changes in technical procedures [Wallbank (1989)];
- packages of improvement in industry related processes – education, contracts, tools – and in the inter-actions and relationships between stakeholders [Clark et al. (1997)];
- adoption of a holistic process-based approach utilising cycles of continuous improvement to tackle the underlying issues affecting durability [BCA (1997)];
- better understanding of in-service performance and how to respond to the insight given by this [Tajilli & Rigden (2000)];
- achieving a high standard of execution to ensure durability [Helland et al. (2008)].

These approaches have met with varying success. It appears that improvement may be achieved in a particular area for a period of time, then further change in the circumstances occurs for some reason and the cycle seemingly must begin again. Factors such as regulation, client interest, stakeholder education, best practice expectations, control of execution processes and post-construction verification appear to be critical in driving change and then in seeking to sustain improved industry performance.

Interestingly some of the above perceived in-service performance deficiencies were attributed to causes which are similar to the reasons cited for inadequate structural performance in the 1981 report "*Structural failures in buildings*" published by the Institution of Structural Engineers (1981). This report encompassed all types of buildings constructed in all materials. The most significant reason for failure was identified as a lack of understanding of the loading conditions and the real behaviour of structures. Ineffective interactions (communication) between stages in the design and construction process were also cited as an important issue, representing breakdowns in communication within the overall process. The analysis carried out highlighted the significance of communication deficiencies occurring between the concept phase and subsequent phases. The study did note that the major feature of the structural failures and the in-service deficiencies considered was not injury or loss of life, but the wider economic consequences of the lack of performance of the structures concerned. Thus there are issues common to all material types which need to be addressed.

Whilst most structural problems considered were discovered during the first 10 years of service, with very few arising after 20 years service; durability related problems tended to occur at a considerably later age. The Institution of Structural Engineers' report (1981) noted that most durability related repairs are apparently made when structures are between about 10 and 50 years old.

Whilst a number of the problems identified in the Institution of Structural Engineers' report (1981) have been addressed by changes implemented over the intervening years, many of the suggested rectification actions are similar to those identified in later studies more concerned with durability. For example, that a project should be supervised by an appropriately experienced competent person and that design concepts should be similarly checked. Other issues identified included clarity in responsibilities and a focus on quality management. The study also implied that local weakness in the experience and understanding held within the owner's/professional team was more important than general deficiencies in codes of practice.

The above studies suggest that although the technical ability exists to achieve the required level of durability and, in spite of progress having been made over the years, elements of further work remain to be done. Service life design concepts and, perhaps most importantly, taking a whole-life perspective on the performance of structures and associated costs of ownership could provide a basis to improve the certainty of achieving durable concrete structures which require a long service life.

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Emmons (2006) has reported that some US$ 18-21 billion is spent annually on the repair and maintenance of concrete and concrete structures in the USA. Chapter 8 presents his estimate (as of 2006) of the sums spent annually on the repair and maintenance of concrete and concrete structures by major USA market sectors. Taken together these provide an indication of the costs associated with maintenance, damage, premature deterioration and the extension of service life of various types of concrete structures in the USA. These estimates may also provide some insight into the likely levels of attention these matters may need in other parts of the world.

In summary: the reviews of (a) the in-service performance of concrete structures and (b) of previous experiences in seeking to effect change in construction industry processes have demonstrated that, in addition to defining appropriate technical requirements, there are significant issues to be addressed in respect of people and process considerations. These are commonly referred to as 'soft' issues, as opposed to the 'hard' issues associated with the specific technical requirements. People issues include factors such as number of people available to undertake the task (the resource pool) and their competence. People issues include factors such as communication, co-operation and co-ordination, procurement procedures, using experience and applying lessons from the past, etc. 'Soft' issues are considered further in later Sections.

5.1.5 Creating durable concrete structures – The need to take a holistic view

There are a number of phases in the development and use of a concrete structure. Typically the sequence of events through the life of a concrete structure is as follows:

- ***concept phase*** – where the owner's basic requirements and needs are established;
- ***design*** – usually involving preliminary and detailed design phases (see Chapter 1);
- ***construction*** – the process whereby the asset is built;
- ***operation and use*** – through-life performance and maintenance of its functionality;
- ***disposal*** – the process by which the asset is either sold, decommissioned or removed.

The way the structure is to be procured, designed, constructed, used and disposed of will have impacts in the various phases in the life of a concrete structure. These need to be understood, particularly in terms of any implications there may be for the durability requirements for the structure.

As part of the conceptual design process, a performance plan can be developed which provides a simple outline of the design service life requirements of the various elements of the structure. These include non-concrete elements of the structure or building concerned. The principle is illustrated in Fig. 5.1-5 for the example of a bridge. In this case, the foundations are required to last the life of the structure (e.g. nominally 120 years for a UK bridge) whilst the other components are intended to either be maintained or replaced at shorter periods within the design life of the bridge. This approach, when taken in conjunction with an appropriate understanding of the aggressivity of structure's service environment, helps the early development of a holistic view of the durability related issues.

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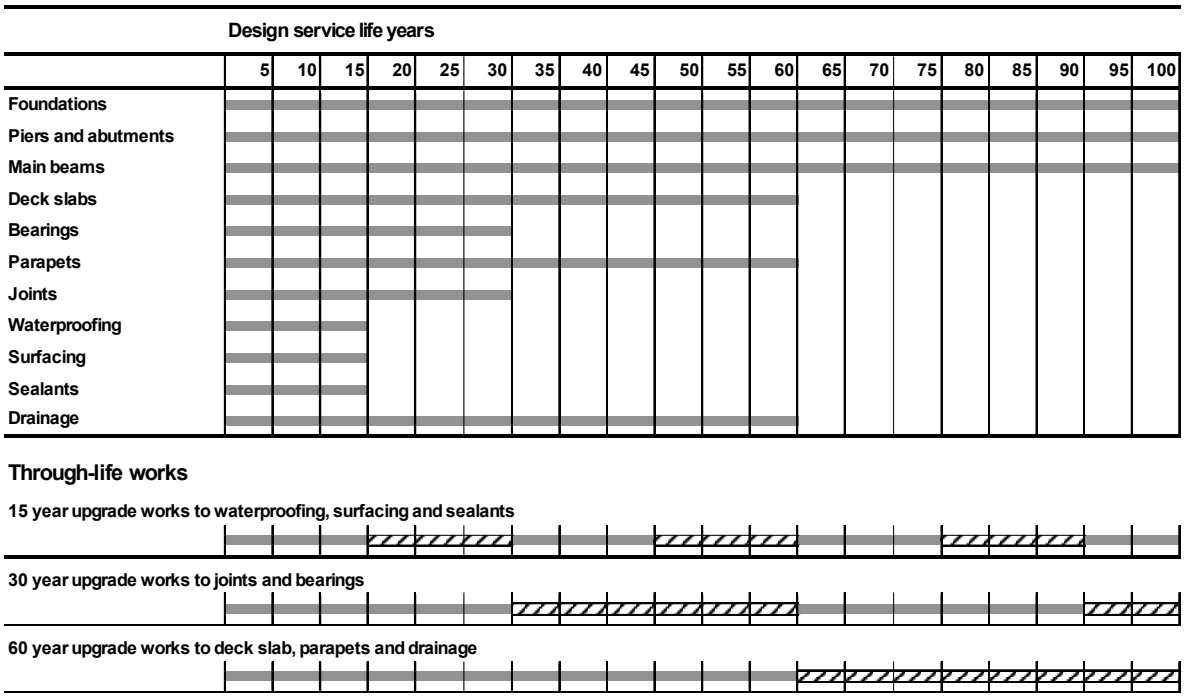

Fig. 5.1-5: A design service life performance plan for elements of a bridge [after Nolan et al. (2006)]

The performance plan approach provides a record these early decisions and a means to:

- record the design intent with regard to service life throughout the evolving design;
- check the anticipated maintenance requirements against the client's expectations for maintenance and functionality;
- record any maintenance related decisions taken during the service life design process;
- record those aspects of the design intent which need to be passed onto the client/user of the structure, perhaps by way of a birth certificate or an owner's/operator's manual for the structure.

Approaches to protecting the structure identified at the conceptual design stage will need to be further developed as the design process evolves into the detailed design. The approach taken will depend on the nature of the structure and its service environment. In many instances the provision of basic resistance measures, in combination with addressing potential construction quality and buildability issues, will be sufficient to ensure that the structure is adequately durable.

However, there may be situations where a more rigorous approach to service life design will be required, for example where:

- The service environment is particularly harsh.
- The functionality of the structure or its component parts cannot be disrupted.
- A long service life has been requested by the client.

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Consequences of durability failure	**Maintainable**	2	1	1
	Durability sensitive	**3**	2	1
	Durability critical	**4**	**3**	2
		High	**Medium**	**Low**
		Degradation risk level posed by service environment		

Key:	
1	No additional measures required at concept design stage.
2	Additional protective measures may be adopted at concept design stage. Engineer's judgement.
3	Additional protective measures recommended at concept design stage. Engineer's judgement.
4	Additional protective measures should be considered at concept design stage.

Fig. 5.1-6: Decision matrix for consequence of failure and durability risk [after Nolan et al. (2006)].

Fig. 5.1-6 presents a decision matrix to assist the designer in identifying those elements that are at potential risk from premature durability failure and illustrates their importance to the function of the structure. This assessment requires characterisation of the service exposure environment and the assessment of the criticality of the component parts of the structure. An iterative approach can be used where significant risk/consequence combinations are identified, allowing the designer to alter the form and detail of the structure to achieve sufficient durability for the structure, especially for the critical parts.

Criticality is a measure of the consequence of durability failure upon the functionality of a structure or a component part. The designer should therefore classify the component parts in the design into one of the following categories:

Durability critical: where failure of a component part would result in a significant and adverse affect upon the functionality of the structure, such that it is unable to provide the desired minimum level of performance. Durability failure therefore has the potential to cause major disruption and unacceptably high costs. These costs would typically comprise direct costs (i.e. to undertake the repair) and consequential costs (i.e. those due to disruption/loss of service). Durability critical component parts of a structure are therefore required to last the expected life time of the structure.

For example, the failure of a series of bridge beams may be durability critical where their failure causes unacceptable disruption to the functioning of a roadway underneath. Also if works to maintain a component part are expected to unacceptably impact on the functionality of the structure, then the component part would need to be classified as durability critical. This would of course depend on the client's requirements for continuity of the function of the structure. Durability critical structures or component parts may require additional protection measures to reduce the risk of durability failure or to minimise the need for intervention within the required service life.

Durability sensitive: where failure of a component part would adversely affect upon the functionality of the structure, but not to such an extent that it is unable to provide the desired minimum level of performance. Although the efficiency of operation or the functionality of the structure might be reduced; maintenance, remedial or replacement works could be carried out at an acceptable cost.

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However, if the necessary remedial or other works were delayed excessively or not undertaken for some reason, it is conceivable that the functionality of the structure could be adversely affected in the longer term. For example, if the joint sealants in the cladding of a tall reinforced concrete building were to experience deterioration the structural functionality of the building would not be immediately impaired. However, this situation might permit the ingress of moisture which would potentially cause corrosion and spalling of concrete, which could pose a hazard to pedestrians passing the building. This would be deemed to be an unacceptable hazard and would therefore potentially affect the functionality/use of the building.

Maintainable: where deterioration of a component part does not impact upon the performance of the structure such that the functionality of the structure is adversely affected. Thus maintenance, remedial or replacement works can be carried out without undue inconvenience or disruption and at an acceptable cost.

In making this categorisation, all component part should be considered carefully and the designer should be mindful of the consequences of failure of seemingly unimportant items. The designer should also consider the level of redundancy in the structure (e.g. simply supported structures are likely to have less redundancy than a continuous beam). The smaller the degree of redundancy in a structure the more critical durability failure tends to become. However, the designer should be aware that unchecked deterioration of a component part of a structure with a high degree of redundancy, while possibly taking longer to fail than a similar component in a structure with a lower degree of redundancy, could eventually result in a more catastrophic failure.

5.1.6 Variations in concrete properties and the durability critical role of the cover concrete

The normal variations which occur in concrete properties have an influence upon concrete durability. These need to be recognised and may need to be taken into account to achieve a durable concrete structure. This means recognising that the properties of concrete change across the section of a concrete element and within a particular pour of concrete (i.e. relative to height up the concrete element/individual pour). In addition, there are differences between the concrete found within actual structures and laboratory samples of concrete taken for control and verification purposes.

Of particular interest are the factors which influence the quality, and hence the durability, of the concrete in the near-surface zone. These matters are discussed by Dewar (1985) and these basic differences are illustrated in Fig. 5.1-7. Dewar (1985) refers to the concrete in the near-surface zone as the ‘covercrete’ and the concrete forming the bulk of the concrete element as the ‘heartcrete’.

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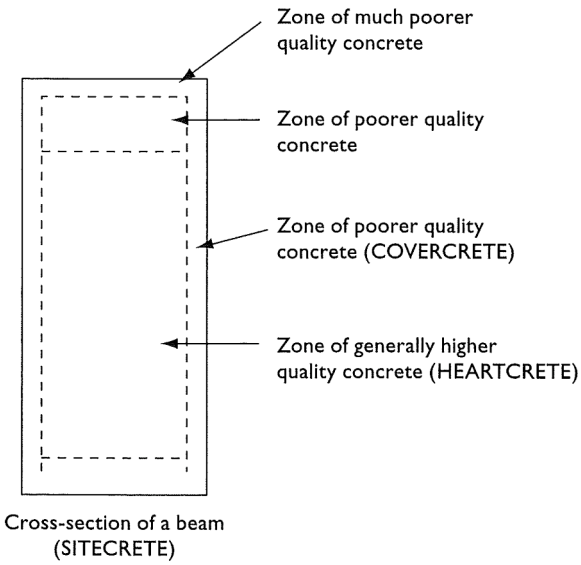

Fig. 5.1-7: Normal variations in concrete quality across the section of a concrete beam [after Dewar (1985)]

A critical factor is that the near-surface zone tends to exhibit poorer durability qualities, e.g. higher surface absorption, than the concrete forming the bulk of the concrete element. Importantly these variations occur even when normal good concrete construction practices and procedures are employed. These factors influence the likelihood that the reinforcement, which is protected only by the concrete in the near-surface zone, will corrode.

These variations in quality arise for a number of reasons. A very significant influence is the water gain effect which occurs, for example, in the upper levels of a concrete pour and against normally formed concrete surfaces. The water gain effect is particularly pronounced at the top surface of a concrete pour. There are also the difficulties encountered in compacting concrete in the cover zone between the reinforcement and the form face.

The greater the resistance of the near-surface zone to the ingress of chlorides, carbonation and moisture, the better the durability of a concrete structure.

5.1.7 Conceptual models for the deterioration caused by the corrosion of reinforcement

Studies of the deterioration of concrete structures [for example BCA (1997)] have found that 50% of the cases of deterioration considered were caused by corrosion of reinforcement due to either chloride penetration or carbonation.

Fig. 5.1-8a portrays the general two-phase conceptual model first proposed by Tuutti (1982) that is commonly used to represent the development of corrosion of reinforcement embedded in concrete; being applied for corrosion induced by both carbonation and by chloride ingress. The vertical axis portrays increasing corrosion damage (increasing towards the top) and the horizontal axis increasing period of exposure (increasing towards the right).

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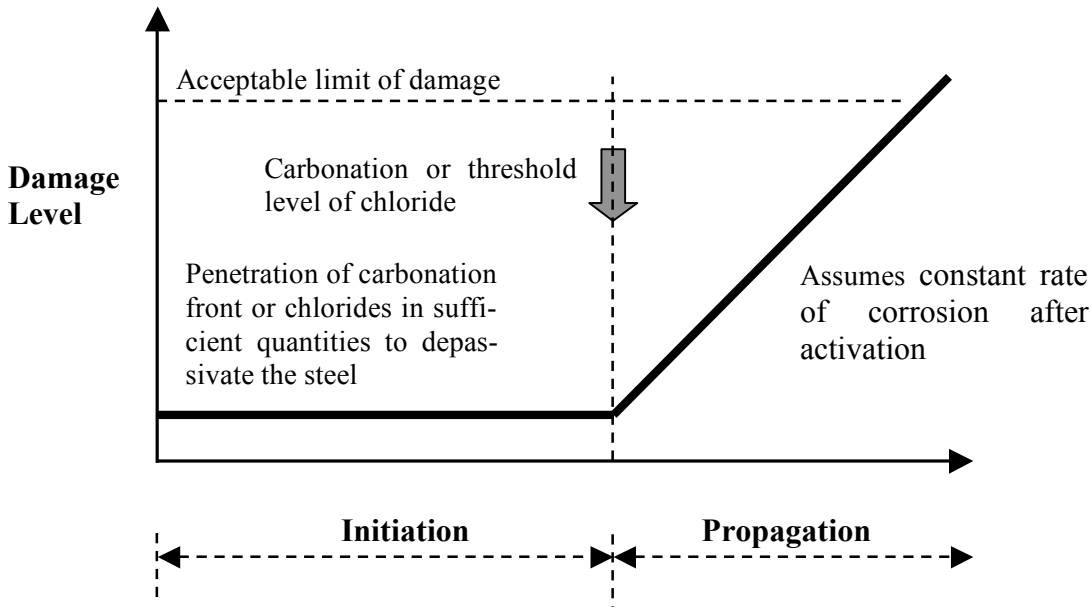
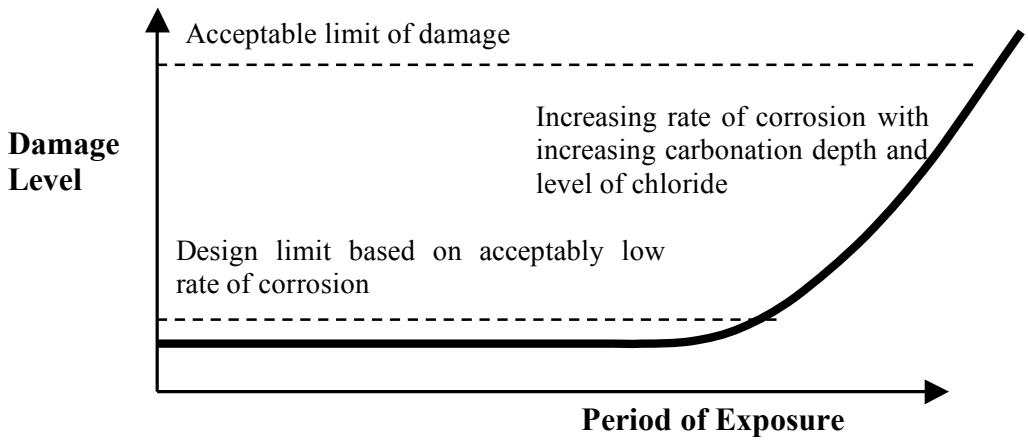


a) Traditional model of corrosion (after Tuutti)



b) Progressive corrosion model (after Bamforth)

Fig. 5.1-8: Simplified models for the corrosion of reinforcement (after Bamforth (2004)]

In the early life of the structure (the initiation phase) the ingress of aggressive species occurs through the cover concrete (i.e. carbon dioxide and chlorides). After some time the surface of the reinforcement becomes depassivated, permitting corrosion to begin. At this point the corrosion propagation phase is entered and various corrosion products are produced, with cracking and spalling of the concrete following at some later time. It should be noted that until cracking and spalling of the concrete have started to occur, there are no external visible signs of the ongoing deterioration process.

Thus the traditional model defines a distinct period of initiation, during which the reinforcement is still in a passive condition, followed by a propagation period when the reinforcement is corroding (i.e. in an active condition). This is a greatly simplified representation of the complex interaction of processes that leads to corrosion activation, determines subsequent corrosion rates and defines the severity and nature of subsequent corrosion induced damage.

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In reality the corrosion propagation process is much more complex. For example, the face of the reinforcing bar closest to the exposed surface of the concrete will become activated first, and corrosion will begin over a very small area. As the carbonation or chloride front penetrates further into the concrete, an increasing proportion of the reinforcing bars will become active.

If one considers the progressive involvement of the complete circumference of the reinforcing bars in the corrosion process, the activation period itself may be spread over many years. Furthermore, in conditions of exposure to external chlorides, the level of chloride at the reinforcement will increase with time. As the chloride level influences the corrosion rate, the latter will also increase in time. Accordingly the rate of corrosion is unlikely to be constant, as represented by the line of constant slope in Fig. 5.1-8a. The gradually increase activation of the reinforcing bar suggests that there is likely to be an extended transition between the initiation and propagation phases, rather than the abrupt change in behaviour represented by the change in gradient of the line in Fig. 5.1-8a. Accordingly, the actual behaviour is likely to be better represented by the revised conceptual model shown in Fig. 5.1-8b.

5.1.8 Through-life performance, whole-life cost and sustainability perspectives

Service life design can play a key role in the delivery of sustainable concrete construction. It also provides a basis for considering through-life performance of a structure and linking this with whole-life costs. To get the greatest benefits, it is essential that service life and durability issues are considered from an early stage in the design and construction process.

Service life design processes can contribute to sustainable development by helping to meet the client's needs in a cost-effective manner, seeking to avoid premature and costly repairs that may arise from a lack of durability or poor quality design. In some instances it may also be appropriate to avoid the excessive use of resources, such as if a structure is over-designing from a durability perspective. Importantly, a structured approach to service life design provides the opportunity to encourage the uptake of other sustainability-related initiatives at appropriate stages of the construction process.

These matters are discussed further in Section 5.3.

5.1.9 Durability and service life design concepts: Definitions and terminology

ISO 13823, "*General principles on the design of structures for durability*" [ISO (2006)], defines design service life as *"the specified period of time for which a structure or a component is to be used for its intended purpose without major repair being necessary.*"

ISO 13823 [ISO (2006)] also defines durability as the "*capability of a structure or any component to satisfy with planned maintenance the design performance requirements over a specified period of time under the influence of the environmental actions, or as result of a self-ageing process*."

From the above it will be seen that design service life and durability are concepts that are inextricably linked. In simple terms, paraphrasing the above, and assuming that appropriate maintenance activities are undertaken:

- the design service life of a structure is the specified period of time over which it is expected to satisfy the design performance (functional) requirements without unplanned major remedial works, and that:

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- a structure will be deemed to be durable when it has satisfied the design performance (functional) requirements for the design service life.

Thus durability is essentially a retrospective view on the performance of a structure. The expectation that a structure might be durable can only be assessed by use of models to represent the deterioration processes to which the structure is likely to be subjected. Accordingly the deterioration models need to be appropriate to the hazards in the service environment in which the structure will be situated. To undertake design a forward looking methodology is required, which is the service life design process.

Service life is usually expressed in years. The design service life is commonly taken to be 50 years for most types of buildings and 100 years, or perhaps significantly longer, for civil engineering structures and other elements of infrastructure such as bridges. Service life may also be referred to in a variety of other ways. For example, ISO 2394 [ISO (1998)] uses the term "design working life". Box 5.1-1 gives further information on design service life/design working life and presents example design working life values taken from ISO 2394 [ISO (1998)] for various types of structure.

Box 5.1-2 explores the idealised in-service behaviour of a structure which is experiencing deterioration as time passes, such that structural capacity reduces as time passes. In this case the performance requirement employed (structural capacity) is concerned with the safety of the structure. Thus, in this example structural safety is diminishing with time as deterioration takes place. This change is presented graphically, showing a notional reduction in structural capacity (safety) with time. At some point in time the structural capacity (safety) is reduced to the minimum acceptable performance level. This situation defines the actual service life of the structure, which may be longer or shorter than the specified length of the design service life depending upon the actual rate of deterioration experienced.

ISO 13823 [ISO (2006)] defines service life as "*the actual period of time during which a structure or any of its components satisfy the design performance requirements without unforeseen major maintenance and repair.*"

It is perhaps interesting to note at this juncture that almost all current structural design codes make no allowance for the effects of deterioration during the lifespan of the structure. This includes the structural Eurocodes developed by CEN, which are generally considered to be the most technically advanced suite of structural design codes currently available. It is assumed that the intended performance level will be maintained by appropriate through-life management, care and maintenance activities. Unfortunately, this is not always the case.

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Box 5.1-1: WHAT IS DESIGN SERVICE LIFE / DESIGN WORKING LIFE?

Design working life is defined by ISO 2394 – *General principles on reliability for structures* as:

The assumed period for which a structure or a structural element is to be used for its intended purpose without major repair being necessary.

NB. This assumes that appropriate maintenance activities will be undertaken.

Table 1 of ISO 2394 gives examples as follows:

Class	Notional design working life (years)	Examples
1	1-5	Temporary structures
2	25	Replacement structural parts
3	50	Building structures and other common structures
4	100	Monumental building structures, bridges and other special or important structures (Large bridges)

Some owners have been known to specify a service life / design working life of perhaps 300 years or more on some forms of very long-life infrastructure. The actual end of use can be determined by a number of factors including changes of use and economics, as well as durability limitations, or failure of the structure or its parts.

There are many factors in the design and construction process that can determine whether a structure will meet its design service life. These problems highlight the need for:

- A holistic approach to design embracing the entire construction process and explicitly addressing service life.
- Whole life costing and evaluation of sustainability impacts in assessing the cost performance / environmental impacts of constructed work and in deciding between alternative options for achieving the owner's objectives.

This approach may, at first sight, seem overly sophisticated for the majority of structures, which are presently built to a simple prescriptive design. However, even a simple, straightforward, consideration of service life concepts at the design stage can radically improve the through-life performance of a structure.

Service life design demands a sophisticated understanding of the actions that can lead to premature failure and how those vary with time. We now have a good understanding of the mechanisms that can lead to deterioration and loss of serviceability, but our ability to reliably relate these to service life is still in need of improvement. There are three key steps to defining and achieving the required service life:

- Identifying, as precisely as possible, what service life is required of the structure.
- Identifying, again as precisely as possible, the actions that could result in the structure failing to achieve its service life and what constitutes the end of service life.
- Ensuring that at each stage of the construction process, design, execution and through-life maintenance and management, steps are taken to address and overcome

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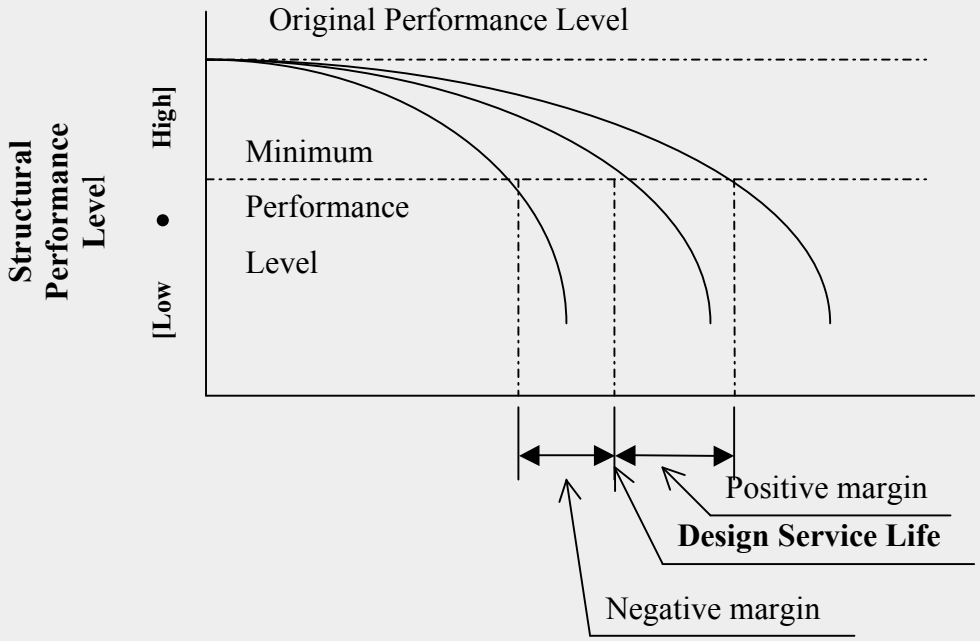
Box 5.1-2: IDEALISED SERVICE LIFE BEHAVIOUR

The following diagram presents an "idealised deterioration curve" which expresses a simplified notional theoretical relationship between design service life and structural performance level; and the (actual) service life and structural performance level.

An explicit definition of what is meant by the "design service life" is required before a "behaviour or deterioration curve" can be established for a particular structure. This necessitates both an understanding of the mechanisms of deterioration acting upon the particular structure and also an appropriate model to represent through-life behaviour. Box 5.1-1 considers several issues relating to design service life. However, these definitions are not always set-down in enough detail to tightly establish what condition will trigger the notional end of service life. There are a range of possible interpretations, which makes the potential outcome uncertain. Such a situation is unsatisfactory.

The actual service life of the structure will not necessarily be equal to the originally specified design service life. The difference is known as the "time margin", as shown in the figure below. If the actual service life achieved is longer than the design service life, there will be a "positive time margin". Conversely, if the actual service life achieved is shorter than the design service life, there will be a "negative time margin".

It is necessary to have an adequate "time margin" to be confident of achieving the specified design service life. The issue needs to be considered not only from the engineering perspective, but also from the viewpoints of the economic and other non-technical requirements. Based on the scenario design chosen, the owner will be able to establish more precisely the maintenance strategy required for the concrete structure(s) concerned.



Box 5.1-2 Fig.: Idealised deterioration curve for a concrete structure

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The principal performance requirements adopted in service life design commonly relate to structural safety, in conjunction with the serviceability and the appearance of the structure. However, various other functional or performance requirements may be used as the basis for defining service life. These may be expressed in terms of satisfying one or more fundamental properties (e.g. acting as proxy criteria for service life and durability issues). Examples of functional requirements and corresponding fundamental properties are:

- ***A minimum load carrying capacity.*** The corresponding fundamental properties include the strength of the concrete and steel, the loss of reinforcing bar cross-section due to corrosion, loss of reinforcement bond and anchorage as a result of corrosion and depth of the spalling/delamination of the concrete surface affecting the ultimate load capacity of the structure.
- ***A maximum penetration of gaseous or liquid substances into the concrete.*** The corresponding fundamental properties include the permeability, capillarity and diffusivity of the concrete, as well as the size and location of cracks. These parameters are in turn are functions of factors such as the standard of execution, as well as material properties including the water-cement ratio, aggregate type, cement type, curing method, degree of hydration, etc. These parameters could have an influence upon the durability of the structure, depending upon the severity of the service environment.
- ***A maximum acceptable deformation.*** The corresponding fundamental properties include the elastic modulus of the concrete, shrinkage, creep, thermal movement, cracking, as well as differential settlement and movement effects.

Thus in the above, the first two items can be linked to potential durability requirements. However, whilst the third item could be linked to serviceability considerations, it could also be associated with deterioration causing a reduction in the effective flexural stiffness of the structural element concerned – resulting in excessive deflection.

The basic elements of the relationship between "durability" and "performance", together with the associated influencing factors, were helpfully defined diagrammatically in the CEB "*Design guide for durable concrete structures*" [CEB (1989)] and later incorporated into the CEB-FIP Model Code 1990 [CEB (1992)]. This is reproduced as Fig. 5.1-9. Whilst this captures the concept of functional requirements and corresponding fundamental properties, it does not portray the time dimension discussed above and which forms a key facet of the service life design concept. However, the time dimension is incorporated in the diagrammatic representation of service life, linking performance and time, presented as the figure in Box 5.1-2.

The elements of design, materials selection, execution and curing which determine the "quality" of the concrete are illustrated in Fig. 5.1-9. Essentially the quality of the concrete is dominated by the nature and distribution of pores in the cast concrete, as these characteristics control the potential transport mechanisms that permit the movement of gases, water and ions into or out of the body of notionally uncracked concrete forming the structure. Consideration may also need to be given to the influence of cracking upon the potential performance and durability of the structure.

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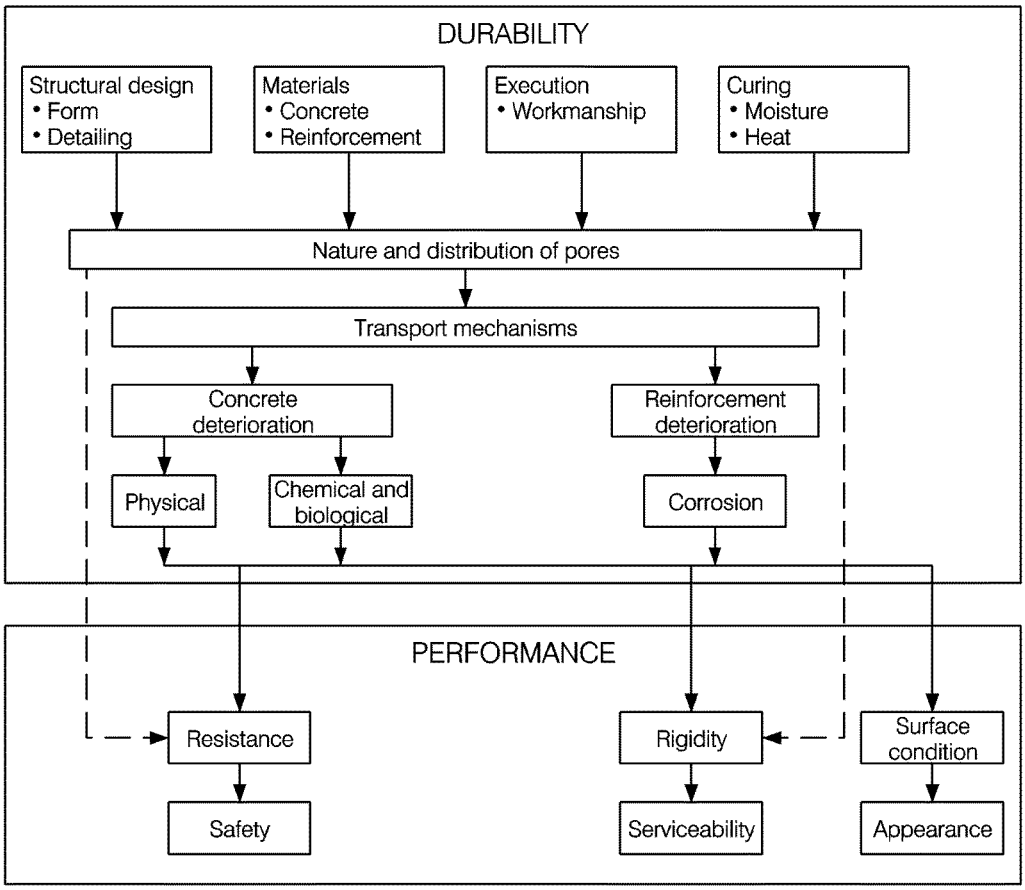

Fig. 5.1-9: Relationship between durability and performance [CEB (1992)]

Service life design is therefore the process by which the designer chooses fundamental properties of the materials and other characteristics of the structure in order to fulfil the functional requirements for the specified period of time (i.e. the design service life). Thus service life design is intended to provide a rational way to ensure that a structure has sufficient resistance against the deterioration mechanisms associated with the service environment, or that it will not be affected by them. Summarising, this process typically involves the following steps:

1. Identification and appropriate formulation of the functional/performance requirements which must be met.

2. Definition of the aggressivity of the service environment, both generally (macro-scale) and at component/local level (micro-scale) where appropriate.

3. Use of deterioration models appropriate to the hazards present in the service environment in which the structure will be situated. The models describe the interrelations between environmental conditions, material properties and deterioration mechanisms. Engineering judgement may be needed to compensate for inadequacies in the models representing the physical deterioration processes, or even the absence of such models.

4. Alternatively, identify materials or compositions which will not react or be adversely affected by the hazards present in the service environment and which will not significantly deteriorate over the required service life (i.e. the design-out concept).

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5. Periodically updating the estimated service life of the structure by measuring parameters which reflect the in-service progress of the deterioration processes. This may be undertaken by means of installed sensors or by non-destructive testing.

Thus performance based service life design enables, amongst other things, the point to be determined at which a particular performance/functional requirement or fundamental property criterion is (or is not) being achieved.

In practice there are various service life criteria which may be used, depending upon the aspect of performance being considered. Three possible service life criteria are:

- ***Technical service life:*** the actual time in service until a defined minimum acceptable performance (functional) state is reached, perhaps due to deterioration associated with the service environment. This might be the initiation of corrosion (see below).
- ***Functional service life:*** the actual time in service until the structure becomes obsolete due to changes in the performance (functional) requirements.
- ***Economic service life:*** the actual time in service until replacement of the structure is economically more advantageous than keeping it in service.

In regard to the definition of the end of technical service life; appropriate criteria can be developed from the various states or conditions that might arise during the life of a structure. These criteria need recognise the degradation processes that could affect the ability of an individual element or the overall structure to perform its intended functions. The states are typically defined in terms of key performance or functionality requirements. For example, the end of technical service life in a structure subject to deterioration caused by reinforcement corrosion might potentially be defined by one, or possibly more, of the following limit states:

- the point at which corrosion is initiated;
- the first appearance of cracking visible with magnification;
- cracking due to corrosion visible to the naked eye;
- the first appearance of spalling;
- the development of excessive deflection under a particular loading configuration;
- the assessed load capacity being reduced to less than a predetermined value;
- the probability of failure under the design or other specified loading exceeding a predetermined value, perhaps expressed as a reliability index.

This approach to service life design does not result in a single value, but recognises that the outcome may vary depending on a number of factors. These might include the type of element or structure, the associated performance requirements and the maintenance regime that is to be adopted. In addition, environmental and aesthetic aspects can strongly influence considerations about what comprises acceptable technical performance. Thus these are additional parameters that may need to be addressed.

It has been suggested that nearly all of the types of deterioration mechanism which affect concrete structures might potentially be modelled in a (very) simplified way by the two-phase deterioration model concept employed by Tuutti (1982) to represent corrosion induced deterioration. The two-phase curve used by Tuutti (1982) is illustrated in Fig. 5.1-8a, with a more realistic conceptual model being presented as Fig. 5.1-8b. However, limited work appears to have been undertaken with respect to the development of service life design models for deterioration mechanisms such as alkali-silica reaction (ASR)/alkali-aggregate reaction (AAR),

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acid attack, sulfate attack, leaching, biological processes or the two forms of frost attack that involve internal frost damage and surface scaling.

5.1.10 Overview of approaches to service life design

Service life design approaches may be divided into two principal design strategies:

Service life design strategy A	Avoiding deterioration by the design-out approach.
Service life design strategy B	Providing resistance to the deterioration mechanisms active in the service environment.

The two principal service life design (SLD) strategies can be sub-divided into a number of different methodologies for achieving the service life design objectives of the particular design strategy. The overall list of service life design strategies and associated methodologies is given below. There is some overlap between the different methodologies; for example SLD strategy B3 potentially overlaps with activities in B1, B2 and B4. The service life design approaches assume that appropriate maintenance activities, such as clearing drains to ensure that water is shed from the structure quickly, will be undertaken in support of the chosen strategy.

Strategy A: Avoiding deterioration by the design-out approach on the basis of:

A1 Changing the service environment to remove the deterioration mechanisms.

A2 Using non-reactive materials to avoid the deterioration reactions.

A3 Inhibiting the deterioration reactions.

Strategy B: Providing resistance to the deterioration mechanisms active in the service environment on the basis of adopting:

B1 Deemed to satisfy (code) provisions.

B2 A single or multi-stage (barrier) protection strategy.

- B2.1 Basic resistance using a single protection strategy.
- B2.2 A multi-stage protection strategy.

B3 The factorial method. (Adapted from ISO 15686: Part 1).

B4. A reliability-based methodology:

- B4.1 Full probabilistic design.
- B4.2 Deterministic partial-factor design (semi- probabilistic design).

The different methodologies outlined in respect of Strategy A – "*Avoiding deterioration by the design-out approach*" cannot be guaranteed to provide complete protection. In most cases they seek to minimise the risk of deterioration occurring under a range of envisaged circumstances. In some respects methodologies A1-A3 might be considered to be alternative protection schemes.

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SLD strategy A1: Actions taken to change the service environment: These might involve the provision of membranes, coatings or some other form of surface barrier between the concrete and the environment within which the structure or component is situated. The effectiveness, longevity and reliability of the particular measure adopted will depend upon a range of factors. In the case of a coating these factors might include its thickness, the permeability of the coating relative to that of the concrete, the standard of workmanship achieved, the expected life of the coating in the environment, etc.

SLD strategy A2: Use of non-reactive materials: The actions which might be taken to avoid the deterioration reactions by using non-reactive materials could involve the use of stainless steel or non-metallic reinforcement to minimise the likelihood of reinforcement corrosion. Another option could be to re-design that component of the structure so that it could be constructed as plain concrete, without reinforcement thereby removing the corrosion risk –Fig. 5.1-10. Other examples include the use of non-reactive aggregates (in respect of ASR/AAR), low alkali cement (in respect of ASR/AAR) and sulfate resisting cements (in respect of sulfate attack).

Consensus national and international guidance on these measures typically seeks to achieve a suitable balance between the risk of deterioration and implied economic burden associated with the restrictions, such that the recommended measures are reasonably applicable to the overall population of structures.

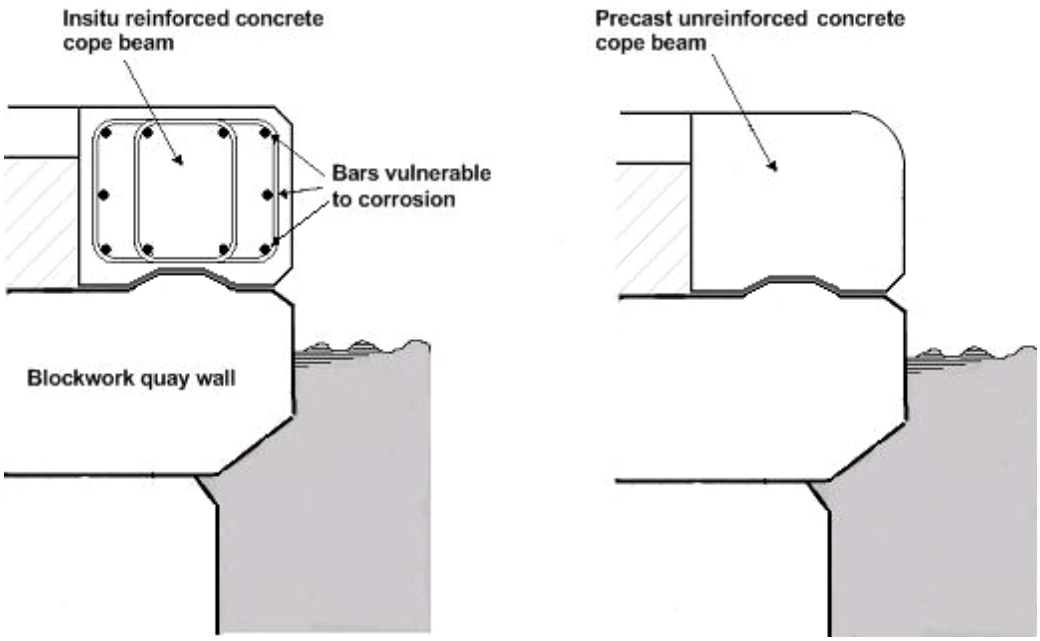

Fig. 5.1-10: Replacement of reinforced quay wall cope beam by an unreinforced precast concrete capping block (CIRIA and Concrete Society (2008)]. Reproduced with permission.

SLD strategy A3: Inhibiting the deterioration reactions: This could, for example, involve the application of cathodic prevention/protection or corrosion inhibitors to suppress reinforcement corrosion or the use of air entrainment to reduce the risk of frost attack.

In regard to Strategy B – "*Providing resistance to the deterioration mechanisms active in the service environment*", the characteristics of the four main methodologies (B1 – B4) are described below. In some respects methodologies B2.1 and B2.2 might be considered to be alternative protection schemes.

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SLD strategy B1: Deemed to satisfy provisions: This approach uses the recommendations which are typically given in tabulated form within codes of practice for structural design and/or the associated materials standards. These relate the provision of resistance (e.g. the depth of cover, concrete grade etc) to the aggressivity of the service environment and the length of design service life. These provisions generally result in concrete structures that are durable in most service environments. However, these provisions may not be adequate in the more severe environments.

SLD strategy B2: Use of single or multi-stage protection: This methodology seeks to select appropriate measures to reduce the likelihood of deterioration, working either singularly or in combination to slow down the penetration of aggressive species into the concrete. These measures are commonly classified under the headings of materials, design and construction (Somerville (1999)]. The main steps in the process are to:

- identify the nature of the deterioration mechanisms (hazards) present in the service environment, their aggressivity and the transport mechanisms responsible for creating the problems;
- establish what parts of the structure may be at risk;
- select means of creating barriers to prevent or slow down the transport mechanisms.

The means to be considered include changing the selected values for the depth concrete cover, its quality, cement type and mix additions, the type of reinforcement, control of crack width and occurrence, the use of protective measures such as coatings, surface treatments, etc. The design skill comes in choosing the best combination of measures for the particular circumstances. It is generally most effective to combine measures from the different groups, namely from the materials, design and construction classifications.

In order to examine different scenarios and options for the through-life care and management of the structure, this methodology could use deterioration models to represent the physical deterioration processes active in the service environment. The deterioration models employed obviously need to be appropriate to the particular hazards present in the service environment and their aggressivity; they could be either deterministic or probabilistic in their formulation. The modelling and options evaluation processes would be expected to include consideration of cost, ideally whole-life cost rather than simply just first cost. These considerations are also increasingly likely to take into account the environmental and wider sustainability impacts of the different scenarios and options for the through-life care and management of the structure.

SLD strategy B3: The factorial method: These concepts have been adapted from those set out in ISO 15686: Part 1 entitled "*Buildings and constructed assets. Service life planning*". There are two ways in which the factorial method has been employed in service life design. These are (a) to make a *basic estimate of service life*, and (b) to make a *refined estimate of service life*. The latter is able to take account of measures that provide additional resistance or protection to the concrete structure or component to enhance (extend) its service life.

The *basic* factorial method uses a reference service life for the concrete structure or component in question, which is derived from the codes of practice for structural design and/or the associated materials standards (i.e. as used in SLD strategy B1). The reference service life is adjusted by means of seven factors (A to G) which represent different influencing parameters upon the service life of the structure or component. These factors have a value of less than one (unity) in circumstances where the influencing parameter in question does not or is unlikely to meet the requirements specified in the codes of practice for structural design and/or

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the associated materials standards. The value can be greater than unity when the influencing parameter exceed these requirements. This process produces an *estimated service life* for the concrete structure or component in question. This will be shorter than the reference service life when the level of the influencing parameters are below those required by the codes of practice for structural design and/or the associated materials standards. The *basic* factorial method might be considered to be a development of the "deemed to satisfy provisions" discussed in SLD strategy B1.

The *refined* factorial method also uses the reference service life for the concrete structure or component (as above), but in this case the adjustment factors take account of the influence of explicit concrete mix and construction parameters such as cement content, cement type, mix additions, water – cement ratio, cover depth; corrosion inhibitors, reinforcement type and the use of controlled permeability formwork. Account may be taken of additional protection measures, such as permanent formwork and the provision of a surface coating, and service environment influences such as surface chloride levels. The values of the adjustment factors used may depend upon the service environment class – for example, these might be classified in accordance with the provisions of the codes of practice for structural design and/or the associated materials standards (i.e. as used in SLD strategy B1).

Consideration of additional protection measures links to the adoption of a multi-stage protection strategy (SLD strategy B2). On this basis the *refined* factorial method might be thought of as being a 'bridge' between the simplicity of the "deemed to satisfy provisions" (SLD strategy B1) and the complexity involved in the use of a deterministic or a probabilistic deterioration model (SLD strategy B4).

SLD strategy B4: Reliability-based (probabilistic) methodology: This method involves the use of a deterioration model to represent a physical deterioration process active in the service environment.

Currently fully probabilistic reliability-based models and the associated analyses are complicated activities. Generally these can only be undertaken by a very limited number of specialists. In most instances there is insufficient data available to truly represent the statistical variability of the input parameters to the deterioration model to portray actual 'real-life' behaviour and experiences. As a result simplifying assumptions have to be made. This generally involves making assumptions about the theoretical statistical (probabilistic) distributions of these input parameters/variables. For example, a normal Gaussian probability distribution might be assumed for parameters such as cover depth variability. However, this would cause analysis difficulties if the assumption resulted in hypothetical negative cover depths (i.e. reinforcing bars notionally located outside the concrete surface). To overcome such problems, other theoretical probability distributions may be assumed (e.g. lognormal, Weibull and rectangular distributions).

A simpler approach is to employ a deterministically based deterioration model. This might be formulated in a partial safety-factor format, again utilising assumed probability distributions to obtain the values of the partial safety factors. Such an approach is likely to assume that a normal Gaussian probability distribution is applicable.

Section 5.1.11 examines the parallels between contemporary structural and service life design approaches. Whilst contemporary structural design uses a probabilistic approach, using a partial safety-factor format applied to a deterministic model, it adopts the simplifying assumption that the variables are not time dependent. This simplification cannot be used for modelling deterioration processes where both 'loads' and 'resistances' can change with time, which greatly adds to the complexity of the analysis.

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Clearly the validity of the method is dependent upon how appropriate the deterioration model is for the particular hazard. If there are multiple hazards present in the service environment, consideration will need to be given to their relative aggressivity and the timescales that the individual deterioration processes operate over. There may also be synergistic effects between different deterioration processes, such as between carbonation and chloride induced corrosion of reinforcement. For example, there can also be linkages between deterioration processes that induce cracking in concrete, such as alkali-silica reaction, and subsequent corrosion of reinforcement. However, work on synergistic effects and on probabilistic deterioration models that might represent them is at a very early stage.

Observations: In some circumstances, the adoption of a multi-stage (barrier) protection strategy may be referred to as being a first generation service life design methodology/strategy; and the use of a reliability-based or probabilistic methodology may be referred to as being a second generation service life design methodology/strategy.

Only SLD strategy B4 explicitly uses a deterioration model. As discussed above, there are various ways to take account of uncertainty. The approach makes these considerations central to the overall analysis and service life design process, offering both deterministic or probabilistic approaches for their consideration. This provides a direct relationship to the physical deterioration processes active in the particular service environment, their variability and their aggressivity.

Although the other methods (SLD strategies B1-B3) recognise these factors, they operate in an empirical/implicit manner based upon previous experience and an understanding of likely performance derived from material science. However, these other methods are considerably simpler and quicker to use than the probabilistic deterioration model; which may require a significant amount of input data (unlikely to be easily obtained and would require substantial time for its collection), together with a significant degree of expertise to obtain a satisfactory output, as well as to then correctly interpret and use the information produced.

The *basic* factorial method, which produces a single figure estimate for the service life, is limited in that it does not take account of or represent the variability of the factors which influence the physical deterioration processes. However, the *refined* factorial method does seek to account for the influence of explicit concrete mix and construction parameters, together with some aspects of service environment characteristics.

As noted above, SLD strategy B2, which concerns the use of single or multi-stage protection measures, potentially overlaps with activities in SLD strategies B1, B3 and B4. For example, the modelling of deterioration processes could provide insight into the choice and combination of protection measures. Thus some methodologies can be used together, so the classification of methodologies used (SLD strategies B1-B4) should not be considered to be a list of mutually exclusive alternatives.

Of the various service life methodologies providing resistance (SLD strategies B1-B4); the deemed to satisfy provisions is the most easily used approach, followed by the factorial method and then by the use of a deterioration model approach – as shown below. Conversely, the sophistication of the methods and their ability to make predictions of the performance of the structure in its service environment, are ranked in the reverse order.

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Deemed to satisfy provisions	Factorial method	Use of a deterioration model
Easy to use → → →		More difficult to use
Increasing sophistication and ability to predict performance in the service environment → →		

Clearly the deterioration model employed needs to be appropriate to the particular hazards present in the service environment and their aggressivity; deterioration models can be either deterministic or probabilistic in nature. Hypothetically the use of a deterioration model would be expected to give the most accurate prognosis of future behaviour. However, there are two main aspects to these considerations. Firstly, different models for the same deterioration process have been known to produce markedly different predictions, so there may be issues of consistency in the prognoses made and also in the sensitivity to the choice of input parameters for the various models. Secondly, there is the accuracy of the prognoses relative to the (potential) real behaviour of the structure in the actual circumstances and service environment. The second issue concerns the validation of a particular deterioration model.

Practitioners accept that there is uncertainty in this regard and they recognise the need to improve their models and methodologies. However, predicting future behaviour is a difficult task because the properties of materials and the characteristics of the service environment exhibit random variation; whilst these can be analysed statistically they cannot be predicted precisely. It is standard practice for the normal probabilistic variation in the properties of the materials and the characteristics of the service environment to be defined by their average values and standard deviations. Typically this is based upon limited data sets.

The deterioration modelling approach introduces the possibility of undertaking design for specific circumstances, that is to make tailor made designs based upon performance characteristics of materials and the characteristics of the specific service environment, plus the chosen approach to the through-life care and maintenance of the structure. This potentially provides the possibility for optimising particular aspects of the design. It also facilitates the comparison of alternative design options.

Notionally service life design approaches might be classified as being either prescriptive or performance-based.

- The ***prescriptive design*** approach uses a 'recipe' or rule based procedures given in authoritative guidance documents such as codes of practice for structural design and/or the associated materials standards. Thus avoidance of deterioration by design-out (SLD strategy A) and the deemed to satisfy provisions (SLD strategy B1) would be classed as prescriptive design approaches.
- The ***performance-based design*** approach is essentially concerned with fitness for purpose or the function to be fulfilled by the structure or component part; rather than prescribing how it is to be formulated or constructed. Thus the focus is on fulfilling requirements set for the performance of the structure or component part and is based upon measurable parameters/outcomes. On this basis, perhaps only SLD strategies B3-B (the *refined* factorial method) and B4 (reliability-based design method) would be classed as performance-based design approaches.

However, a number of the service life design methodologies combine the use of the prescriptive and performance-based design approaches. For example, in the case of the use of multi-stage protection measures (SLD strategy B2) and the *basic* factorial method (SLD strat-

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egy B3-A), aspects of these methodologies could be classed as being prescriptive, but other aspects are more performance-based in their nature.

It is suggested that designers do not get too 'anxious' as to whether a service life design methodology is classed as being either prescriptive or performance-based.

In the service life design approach some form of target life is essential to enable alternative design options for meeting the required design service life to be evaluated at the conceptual design stage. It is essential that service life design should be treated as an integral part of the overall design process rather than as a standalone 'bolt-on' to be carried out as part of an ancillary quality assurance scheme.

In this process consideration needs also to be given to issues of first- and whole-life cost, in conjunction with sustainability related matters. These topics are discussed in Section 5.3.

5.1.11 Parallels between contemporary structural and probabilistic based service life design

Up to now design for durability has been primarily based upon the prescriptive approach, with specification limits in the main being given on matters such as the concrete mix constituents and proportions, the depth of cover to reinforcement and the permissible width of cracks. This has utilised 'recipe' and rule based procedures. If the prescribed limits are met, it has been deemed that the structure will be durable. This has not always the case in aggressive environments. In these procedures no direct account is taken of the environmental 'loading' or of the potential 'resistance' of the structure.

Conversely, in the case of structural design the procedures do take account of both the 'loading and actions' which may be applied to the structure and also of the potential 'resistance' of the structure. Thus (in the main) the procedures for structural design are considered to be rational and objective; being based upon the principles of reliability theory to ensure that probabilities of failure, or of non-compliance with specified criteria, are kept within acceptable bounds. The procedures recognise the uncertainty which exists in the magnitude and nature of the applied loading and in the strength of the materials. In essence the generally accepted aim of structural design is to achieve an acceptable probability that the structure being designed will perform satisfactorily during its intended service life.

Use of a reliability-based methodology provides two principal options:

- Full probabilistic design: However, it will seldom be possible to employ this approach for new structures because there is a lack of suitable statistical data. The approach might be adopted for special long-life or monumental type structures where the necessary investigations and research work could be justified. In addition, the time necessary to undertake the required data gathering and associated investigations would need to be available.
- Partial factor design (semi- probabilistic design): In this approach the necessary factors to produce acceptable levels of safety have been calibrated by reference to probabilistic design studies of the overall population of structures. This approach is based upon design values for loads, capacities and geometrical characteristics and is widely used, forming the basis for most contemporary codes for structural design. It uses a deterministic model to represent the behaviours of interest.

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When undertaking contemporary structural design the partial factor design approach will invariably be used. The designer first defines the loads and actions to be resisted and, in recognition of the uncertainty which exists in respect of the magnitude and nature of the applied loading, partial-safety factors are applied, to provide a margin (safety factor) on the performance of the structure. Similar procedures are employed in respect of material strengths, the geometry and other qualities; these limit the stresses employed to an acceptable level. The factored loads are resisted by the structure by selecting an appropriate combination of structural system, element geometry, material type and strength. The structural design process seeks to ensure that the probability of the loading exceeding the resistance (capacity) of the structure is maintained at an acceptably low level.

When it comes to service life design to verify that the intended life can be achieved with an acceptable level of reliability, durability design can be undertaken using the same concepts as those employed for structural design and as described above. However, probabilistic durability design is more complicated than standard structural design because the loading and resistance functions may be time dependent (either one or both); whereas the parameters used in structural design are assumed to be time independent. This simplification greatly reduces the complexity of the analysis used for structural design.

5.2 Overview of the service life design, construction and through-life care process

5.2.1 Introduction

Service life design should be treated as an integral part of the overall design process, rather than as a standalone ‘bolt-on’ to be carried out as part of a quality assurance scheme. However expressed, some form of target life is essential to enable alternative design options for meeting the required service life to be evaluated at the conceptual and later design stages.

Fig. 5.2-1 presents a simplified representation of the main stages in a service life design approach, and its interaction with structural design. The system is based on well established concepts and current good design practice:

- The client brief: defining client requirements and expectations.
- Assessment of environmental loads: characterising the exposure environment.
- Conceptual design: Providing a concept scheme suitable for meeting the client’s requirements.
- Design: designing and detailing for durability and buildability.
- Execution strategy: designing for buildability and onsite supervision levels.
- Maintenance strategy: ensuring continuity of functionality of the structure and providing the user with the required information.

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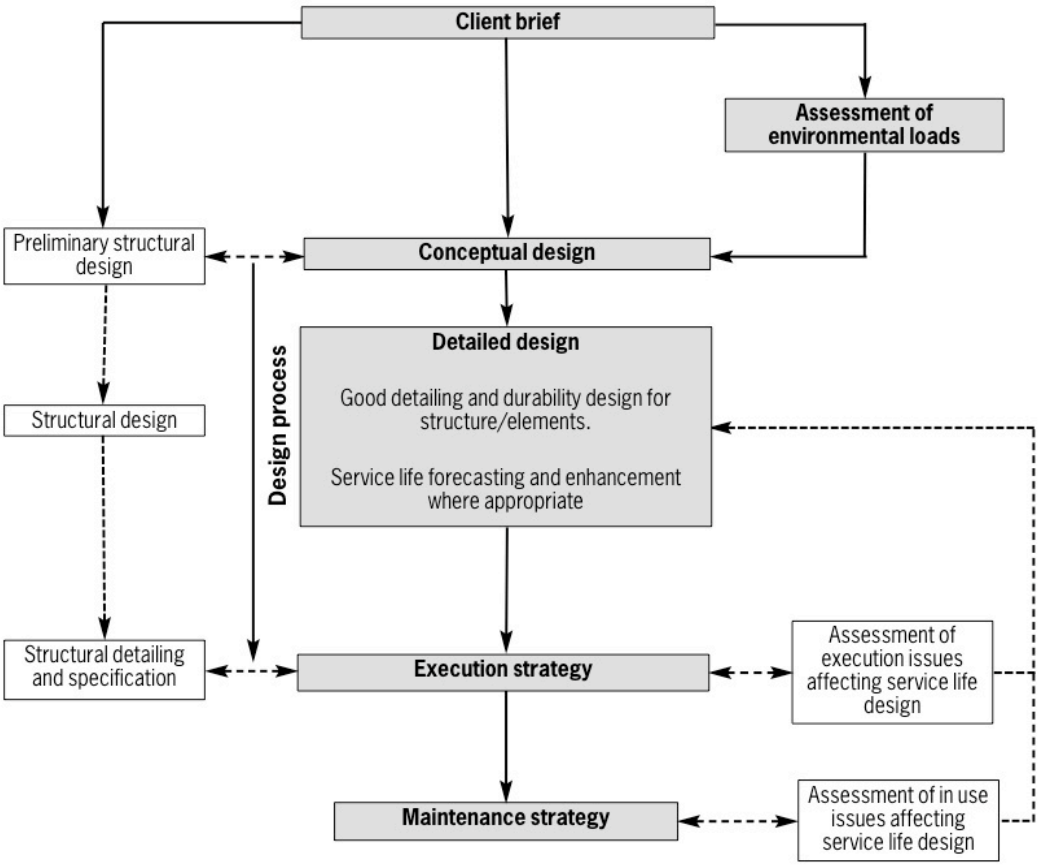

Fig. 5.2-1: Simplified service life design procedure [BRE (2006)]

The following, however, may be new to some designers:

- Adopting a holistic approach, ensuring that service life and durability are considered throughout the structural design process. For example, consideration should be given to the way in which the structure interacts with the environment and how maintenance will affect operation.
- The concept of estimating the likely service life based on the design, its environment and other factors.
- Enhancing the design or specification in the light of specific advice based on state of the art knowledge of materials to meet the design service life.

All the following parties should contribute to the service life design process in different ways. They all have important contributions to make.

- The client/owner by defining clear performance requirements and taking an appropriate whole-life (long-term) perspective upon decisions, costs and benefits to be accrued.
- The designer and other technical professionals by converting the owner's performance requirements into design concepts, strategies, technical specifications and conditions.
- The contractor and material suppliers by using appropriate materials and processes in a careful manner that follows the technical specifications and design concepts in order to maximise the likelihood that the structure will meets the owner's performance requirements and be adequately durable.

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- The operator or user (if not the owner) by providing through-life care and management, undertaking timely maintenance and interventions upon the structure to ensure that the intended service life is achieved without unforeseen or unnecessary costs being incurred.

Any of these parties may – by their action or inaction – contribute to a lack of durability and excessive costs for remedial works upon the structure, with the possible disruption costs to users associated with any loss of service or functionality that this may involve.

However, first and foremost the client/owner must demand good quality construction, he must check the quality received, he must be prepared to pay an adequate sum to get the required quality (i.e. the initial construction cost), he must maintain the structure in a condition satisfactory to him and be prepared to bear the costs of this [Rostam and Schiessl (1994)].

The philosophy behind such an approach was more elegantly described by John Ruskin in:

The Common Law of Business

It's unwise to pay too much, but it's worse to pay too little.
When you pay too much, you lose a little money – that is all.
When you pay too little, you sometimes lose everything, because the thing you bought was incapable of doing the thing it was bought to do.
The common law of business prohibits paying a little and getting a lot – it can't be done.
If you deal with the lowest bidder, it is well to add something for the risk you run.
And if you do that, you will have enough to pay for something better.

John Ruskin, 1860

5.2.2 Client brief and definition of performance expectations

The client brief addresses the relevant needs and aims of the construction project, resources to be provided by the client, the details of the project and any appropriate design requirements within which all subsequent briefing (when needed) and designing can take place. This stage of the design system should highlight the important issues for the client (and parties advising and assisting the client in the preparation of the brief) to address in order to enable effective service life design. It has been sub-divided into two parts; the clients' basic needs and the consequent performance requirements.

Clients vary greatly in their understanding of how to formulate and express their needs and a key role of the designer/consultant is helping the client to understand and express his/her needs in terms that can be translated into clear performance requirements.

Client's basic needs: The client should consider a number of basic issues that can influence the design and the performance requirements for the structure. These include:

- what type of structure is needed and where is it to be located;
- the planned function(s) of the structure and its component parts;
- the performance requirements for the structure;
- any desired sustainability credentials;
- appearance or aesthetic requirements, both initially and through-life;

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- requirements for usable space, dimensions, services and fittings;
- the period of tenure and the requirements for the structure at the end of this period;
- future changes of use to increase flexibility and minimise the risk of obsolescence;
- restrictions to the design (e.g. planning regulations).

Definition of the client's basic requirements will enable the performance characteristics for the structure to be specified. These performance requirements, together with an assessment of the environmental loads, will form the basis of the design process and should include:

- safety and serviceability requirements;
- service life and clarity upon what constitutes the end of service life;
- importance of continuity of function and flexibility to accommodate changes of use;
- management and maintenance requirements;
- acceptable level of inspection, maintenance, repair or replacement;
- acceptable whole life costs (capital and operating costs);
- design restrictions;
- agreed sustainability credentials.

The issue of whole life cost encapsulates an understanding of how design decisions affect not only the initial construction costs, but the costs of operating the structure throughout its life. It is expected to become a significant influence in the design process, along with sustainability considerations. However, as with service life design, there is still much work to be done to translate ongoing research in these areas into a useable system.

5.2.3 Environment aggressivity classification

The environment to which the concrete will be exposed is a key factor in designing for a given service life. Relevant factors include:

- The general environment conditions of the area in which the structure is situated (macro-climate).
- The specific location and orientation of the concrete surface being considered and its exposure to prevailing winds, rainfall etc (meso-climate).
- Localised conditions (micro-climate) such as surface ponding, exposure to surface run-off and spray, aggressive agents, regular wetting, condensation, etc. These aspects include those arising as a consequence of the interaction between the structure and the environment (e.g. cladding to keep the structure dry or ponding due to poor detailing, etc.). It is essential that the interaction between the structure and its environment is addressed when seeking to design durable structures. The influence of an aggressive environment on potentially vulnerable and critical components can be reduced by appropriate design. These issues would be considered in the 'conceptual design' and 'detailed design' phases of the service life design process.
- The required performance under the defined environmental loads.

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Environmental loads are defined as aggressive actions, natural or man-made, transmitted to the structure through ground, water or the atmosphere, which, directly or indirectly, can attack structural components, through chemical or physical means, thereby reducing their load-bearing capacity, stability or functional serviceability.

In the service life design process the assessment of the environmental loads is brought together with outputs from the client brief to provide a definition of the required performance under these loads. This forms the starting point for the durability-related aspects of the conceptual design stage in which the basic defence strategy is formulated.

Fig. 5.2-2 illustrates some possible micro-climate conditions for a bridge subjected to de-icing salts. Water can be present on the structure as a result of ponding, rundown and spray effects. Water can also be available in the form of vapour, condensation, as well as from leaks and seepage through joints and cracks. The effect that water from the different sources has upon the durability of the structure will vary, principally depending upon how they contribute to the different transport mechanisms facilitating the penetration of de-icing salts or other aggressive species into the structure. Bridge orientation and the direction of the prevailing wind will also influence micro-climate conditions on the bridge. However, many of the potential problems can be alleviated by good detailing, particularly in terms of sloping surfaces to avoid ponding, by making provision for movement and by removing water from the structure as quickly as possible.

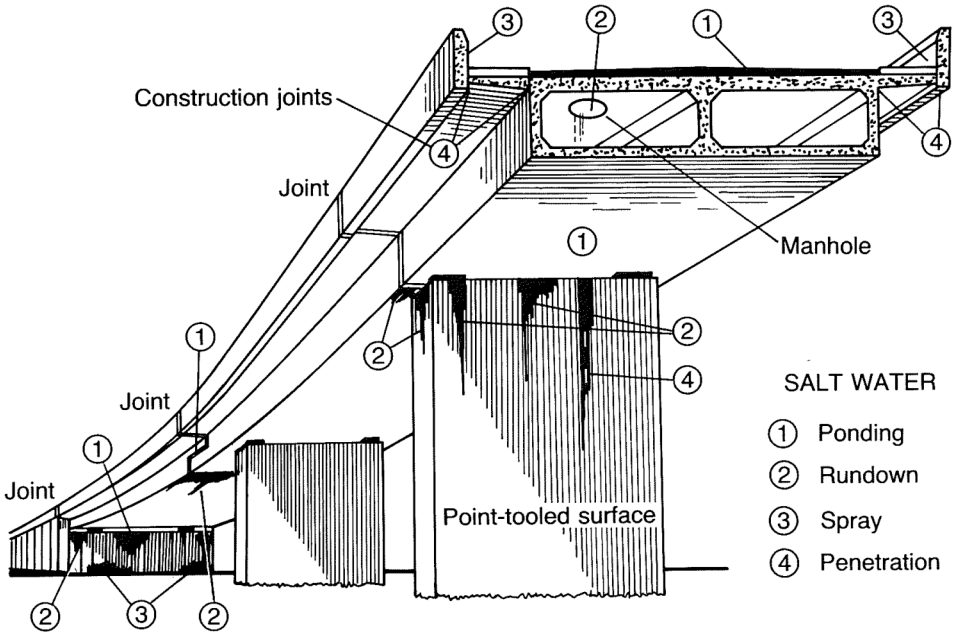

Fig. 5.2-2: Some micro-climate conditions on a bridge subject to de-icing salts [Pritchard (1992)]

5.2.4 Conceptual design for durability

Conceptual design forms the core of the service life design process. Based on an assessment of loads and of the clients requirements key decisions are made regarding how best to resist the environment and the choice of structural form to minimise deterioration. The aim should be to minimise the effects, which are influenced by wind and temperature, of water by ensuring that water drains off the structure quickly and by reducing penetration in durability-critical areas. For routine structures this may be straightforward, involving some combination

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of appropriate prescriptions with good detailing practices. In more extreme situations greater rigour may be needed, perhaps using a design-out approach or a multi-layer protection system – or possibly a combination of the two concepts with good detailing practices, utilising either the factorial or reliability-based methodologies to evaluate alternative options.

At this stage decisions need to be made to maximise the sustainability of the structure, based on the sustainability credentials agreed in the performance requirements. In particular issues such as designing for reuse, minimising waste and energy in use need to be addressed at this point. In the context of concrete construction, these could include concretes made using recycled and secondary aggregates or 'low energy' cements (which alternatively may be referred to a low carbon or low embodied CO_2 cements).

The likely level of workmanship and the quality of materials need to be taken into account. Decisions made at the conceptual design stage will also influence the maintenance and management strategies. Parts of the structure that may be vulnerable to deterioration need to be identified, along with their criticality, and whether it is possible for them to be maintained or whether they may need to be replaced during the design service life. Whole life cost assessments of potential design options may be carried out prior to selection of the most appropriate option. These assessments may require initial forecasts of component service lives in their specific exposure environment. These assessments may be carried out in more depth at the detailed design stage.

The ease with which a design can be translated into reality (i.e. its 'buildability') needs to be assessed throughout the design process. Structures and details which are difficult to build may be constructed to a lower quality than those that are easier to build, with potentially significant consequences for durability. It is advisable that the construction team are able to comment on the buildability of the design. It is also important for the designer to be aware of the preferred construction methods and to know their advantages and constraints.

The selection of the basic defence strategy during conceptual design is one of the key decisions affecting durability. As noted previously (Section 5.1.10), there are two main service life design strategies that can be used and, depending on the circumstances, a number of options (methodologies) within each which can be employed to achieve the design objectives.

Service life design strategy A: Design-out

- changing the service environment to remove the deterioration mechanisms including the exclusion of water, e.g. provision of membranes, coatings, enveloping cladding system, continuous structures avoiding joints, etc.;
- using non-reactive materials to avoid the deterioration reactions (e.g. non-ferrous reinforcement, sulfate-resisting cement, low alkali cement, non-reactive aggregates, etc.;
- inhibiting the deterioration reactions, e.g. cathodic prevention, corrosion inhibitors, etc.

Service life design strategy B: Provide resistance

- adoption of deemed to satisfy provisions;
- single-layer protection, e.g. depth of cover, use of low permeability materials;
- multi-layer protection, e.g. coatings, corrosion inhibitors etc in addition to good quality concrete and appropriate cover depth, etc.;

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- service life design using the factorial method: (a) basic method and (b) refined method;
- service life design using a reliability-based (probabilistic) methodology, which may either involve a full probabilistic approach or a simpler deterministic partial safety-factor design (semi- probabilistic design) approach.

5.2.5 Detailed design for durability

In this stage, the conceptual service life design is developed alongside the conventional structural design process into something that can be built. Conventional structural design is essentially a numerical process involving structural analysis and section design undertaken to satisfying the specified limit states, but also there are the additional and very important facets of dimensioning and detailing the structure. The following items will need to be included in the output from structural design process, as these are critical to the durability of the structure:

- appropriate concrete cover (also relevant to bond and fire resistance considerations);
- the required concrete strength grade and concrete composition;
- reinforcement arrangements to limit flexural cracking under dead or imposed loads, particularly crack widths, allow proper compaction of concrete, etc.;
- shapes, dimensions, jointing of components and general detailing; to provide adequate stiffness and stability, to limit deflections, control water flow and exposure to aggressive species, but also to ensure ease of construction and buildability, etc.;
- consideration of movement, under the effects of temperature, wind, etc.

Any areas that are critical to durability and that require particular attention need to be identified and addressed during the detailed design stage. These should be as easy to build as possible. Minor elements with short service lives need to be easy to replace. Inspection, maintenance, repair and replacement plans should also be developed at this stage. These plans serve to help ensure that the performance of the structure matches that required by the client.

5.2.6 Codes and standards – Deemed-to-satisfy durability design

The deemed to satisfy provisions are typically given in tabulated form within national and international codes of practice for structural design and/or the associated materials standards. Generally these relate the provision of resistance (e.g. cement type and quantity, maximum water-cement ratio, the depth of cover, concrete grade, minimum air content, type of curing, control of early age cracking, limitation of crack widths, etc) to the aggressivity of the service environment and the length of design service life.

The deemed to satisfy provisions are generally accepted as being the minimum requirements or provision that designers are expected to meet. The deemed to satisfy provisions may not reflect current best practice measures because of the time it takes to achieve consensus and to go through the relevant standardisation procedures associated with the periodic updating of such guidance. However, the deemed to satisfy provisions in the more modern and international codes of practice for structural design do generally result in concrete structures that are durable in most service environments. However, these provisions may not be adequate in the most severe or aggressive environments.

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The deemed to satisfy provisions within national and international codes of practice for structural design are expected to remain the principal basis for the durability design of most concrete structures in 'normal' service environments for the foreseeable future.

Historically, and for reasons of tradition, national codes of practice for structural design and/or the associated materials standards differ considerably from country to country. Examples of these differences include whether it is permissible to:

- add calcium chloride to the concrete as a set accelerator;
- employ high water-cement ratios;
- use a very small depth of concrete cover in externally exposed concrete structures.

All these are examples of conditions or measures that are counter-productive to producing durable concrete structures with a long service life.

Typically national codes and standards for structural design do not yet include service life design approaches using a reliability-based (probabilistic) methodology which are becoming increasingly used for structures with long or very long service life and/or are exposed to severe/aggressive environments.

Deemed to satisfy provisions do not provide a means of predicting the service life of a concrete structure based upon data gathered about a particular service environment. For the same reason, they do not permit service life updating where a revised estimate of the service life of the structure is made based upon data gathered from in-service measurements on the structure.

5.2.7 Probabilistic performance-based service life design

A comparison is made in Section 5.1.11 of the parallels between contemporary structural and probabilistic performance-based service life design. This identifies the limitations of the widely used prescriptive approach to durability design, noting the need to recognise the inherent variability in the environmental 'loading' applied to the structure and also of the potential 'resistance' of the materials used to form the structure. This variability is similar to that which occurs in the mechanical properties dealt with in the structural design process.

For structural design it is usual to assume that the strength (resistance) remains constant, and that the loads, even if fluctuating, can be characterised by a single value. Thus the (load and resistance) variables are assumed not to vary with time. Partial safety factors are applied to both the load and response functions, to take account of the variability and uncertainties, to produce values that can be used for design.

Probabilistic performance-based service life design takes the concepts employed in contemporary structural design and extends them to the durability design of concrete structures. As durability involves time-dependent parameters, the simplifying assumptions used in structural design cannot be employed in durability design. Accordingly the procedures employed in probabilistic performance-based service life design are somewhat more complicated than those employed for contemporary structural design, but there are software packages available which perform the necessary calculations. In addition, there may be several serviceability related limit states associated with the end of the technical service life to consider when designing for durability – refer Section 5.1.9.

The concepts for probabilistic performance-based service life design are set down in the *fib Model code for service life design* (MC-SLD) [*fib* (2006)]. This document contains five

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chapters and four informative annexes. The MC-SLD explains the basis of design; these being the requirements to be met, the limit states concerned, the basic variables to be considered and procedures for verification. The MC-SLD also gives detailed procedures for the verification of service life design, as well as requirements to be addressed during execution of the works and for quality management. The MC-SLD also defines responsibilities for maintenance and condition control of the structure in service. More details are given in Section 5.7.3.

The reliability-based methodology incorporated into the MC-SLD recognises the use of two principal options; the use of a full probabilistic design approach and the deterministically based partial safety-factor (semi- probabilistic) design approach. It is suggested that the full probabilistic design approach is only likely to be employed for special long-life or monumental type structures, because there is a lack of suitable statistical data to undertake the necessary calculations. The adoption of the full probabilistic design approach is likely to require simplifying assumptions and the adoption of assumed probability distributions. Conceptually the partial safety-factor design approach is similar to the approach used for contemporary structural (strength) design.

Probabilistic performance-based service life design concepts have also been included in the forthcoming *fib* Model Code 2010; which is a revision, updating and extension of the earlier CEB-FIP Model Code 1990 [CEB (1992)]. This means that service life design will be integrated into the general process for the design of concrete structures. This will permit design for structural safety and for durability to be performed using conceptually similar procedures.

5.2.8 Project specifications – A client/owner's tool

The durability of a structure and its associated through-life costs and environmental impacts are largely determined by the achievement of appropriately specified durability related requirements (or conversely the failure to define and/or to achieve them) set down in the project specification for the design and construction of a concrete structure. The project specification, incorporating the execution specification and project-specific quality plan requirements, provides the client/owner and his professional team with a means of defining and achieving the standard of performance they require.

It is critical that appropriate information is included in the tender and contract documents which provide the legal and financial foundation for procuring the desired concrete structure. Thus clients/owners and their professional teams need to know how to draw up appropriate tender and contract documents to ensure that problems do not occur which could diminish the actual durability of the structure below the level that is required. The centrepiece of these documents is the project specification. The goal is that these documents should incorporate the necessary preventive action requirements and effective control procedures and that they should not simply create a "forms for forms sake" based approach to quality management.

These issues relate particularly to the execution of construction and to quality plan requirements when the structure is being built.

The term execution is used to refer to all the physical activities undertaken for the physical completion (construction) of the works. These activities include procurement, scaffolding, falsework, formwork, reinforcement, concreting, curing etcetera; as well as the related inspection and documentation of those activities. The usage of this term is consistent with the convention adopted in European and international technical construction standards.

These matters are discussed further in Sections 5.14 and 5.15.

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5.2.9 Execution of works

The execution phase is crucial in determining whether the structure will be durable for the design service life. There have been numerous examples of durability problems due to poor concrete, inadequate cover to reinforcement, chlorides in the concrete or combinations of these. For example, these deficiencies have led to various forms of corrosion induced damage, such as cracking and spalling of concrete, reductions in structural capacity, as well as creating the need for repair works and created a larger than anticipated maintenance burden.

The quality of workmanship is influenced by the quality of design and detailing. Some designs are difficult to build, even with reasonable care and standards of workmanship on site. Many of the construction problems can be avoided by involving the contractor in the design process, in accordance with modern concepts of partnering. Cover, curing, compaction, supervision and communication also influence quality – as has been discussed in Section 5.1.4.

There are two complementary ways of addressing construction quality and variability, especially in providing resistance to corrosion. One is to attempt to raise standards over time to an acceptable and sustainable level by encouraging better design and detailing, communication, education and training, benchmarking of best practice, and the use of method statements and certification. The other is to assume current variability and standards, and consequently relies on strategies such as 'design-out' and the use multi-layer protection systems.

From an environmental perspective it is important to note that there is considerable wastage of materials on site, unused during the construction process. In the case of UK construction, it is reported that nearly 20% of all materials delivered to site are wasted. Much of this waste is avoidable and has been estimated to reduce the already small profits of construction companies by up to 25%. Whilst it may be less of a problem with concrete than with some other construction materials, it is essential that we establish how and why fresh concrete is wasted on site, and how to minimise waste. One such approach is summarised in Box 5.2-1.

Box 5.2-1: AUDITING AND MINIMISING CONSTRUCTION WASTE ON SITE

There are various site based methodologies to audit waste streams generated during the construction process, with the goal of targeting reductions in these. These methodologies are intended to provide robust and accurate mechanisms by which wastes arising can be benchmarked and categorised by source, type, amount, cause and cost. Audits are typically undertaken for activities such as construction, demolition, refurbishment, manufacturing and prefabrication. The data is a springboard to identifying and prioritising actions to reduce waste (which is a producer responsibility), seek to achieve re-use of waste streams at source (the proximity principle), and maximise recovery to extend the effective life-cycle of all types of construction materials. The software tools provided often enable the potential cost savings to be identified and the reduction, reuse, recycling and recovery options of materials to be maximised.

One such system is SMARTWaste™ (Site Methodology to Audit, Reduce and Target Waste) which was developed by the UK Building Research Establishment (BRE).

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5.2.10 Through-life care/maintenance

If the structure is to meet the defined performance requirements it is necessary to ensure that the structure is being used in a way that is compatible with the design intent and that inspection, maintenance, repair and replacement are conducted in accordance with the plans developed under the detailed design stage. End of life decisions, such as the implications of obsolescence, possible changes of use and the loss of fitness for purpose must also be addressed. These issues influence the design and so must also be considered as part of the client brief and conceptual design.

5.3 Through-life performance, whole-life cost and sustainability

5.3.1 Introduction

So far the above discussions have mainly focused upon issues relating to the functional requirements for the concrete structure. These are sometimes referred to as the technical performance requirements. However there are other wider economic, socio-cultural and environmental factors (sometimes referred to as non-technical factors or issues) that may have an important bearing upon decisions for designing and managing a concrete structure.

Clearly design and management decisions need to be undertaken in a holistic way, balancing these different considerations. It is necessary to do this within a suitably broad framework. Previous work relating to life-cycle analysis of assets and structures has provided various frameworks to help define, understand and measure these components. Fig. 5.3-6 illustrates one classification of these components, utilising the four headings:

- functional requirements;
- economic and financial requirements;
- societal and cultural aspects;
- environmental considerations.

Collectively the later three factors are commonly known as “sustainability”, but definitions of the scope and meaning of this term are perceived to vary significantly around the world. Sustainability and environmental impacts are key considerations in the world economy and in the creation of assets and facilities required by modern society. This Section seeks to bring together considerations of through-life performance, whole-life cost and sustainability; whilst giving some insight how they impact upon the service life design of concrete structures and the attendant requirement for durable concrete structures.

5.3.2 Service life and whole-life cost issues

Fig. 5.3-1 illustrates schematically the relationship between the main phases in the life of a concrete structure (on the horizontal axis) and two factors denoted below relative to a particular phase (time) in the life of the concrete structure, namely:

1A the level of knowledge available about the performance of the concrete structure, and

1B the cost of achieving any change in the performance requirements.

Factors 1A and 1B are portrayed in Fig. 5.3-1. This shows a low influence/impact in the early stages of the life of the concrete structure, but with an increasing influence as the concrete structure gets into the later stages of its life. Clearly the level of knowledge available

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about the potential performance of a concrete structure would be expected to rise with time, increasingly greatly as it was designed and constructed.

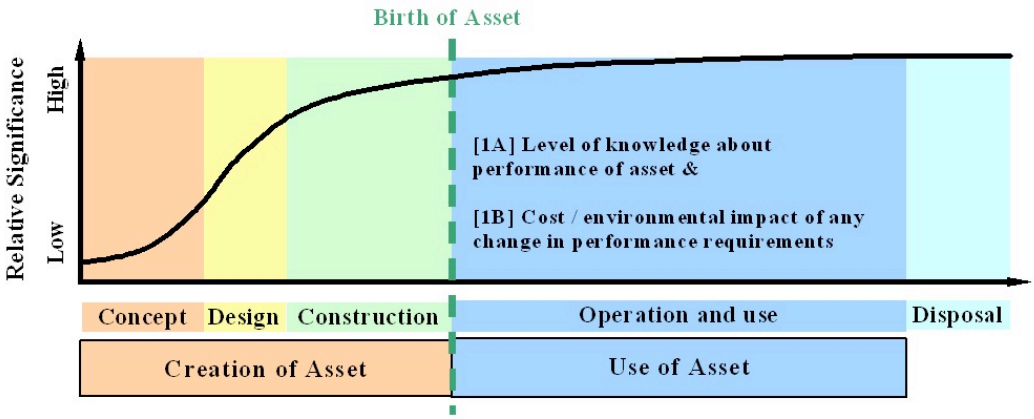

Fig. 5.3-1: Relative level of knowledge about the performance of a concrete structure and the cost and environmental implications of the timing of making a change.

Fig. 5.3-2 illustrates the relationship between the main phases in the life of a concrete structure (on the horizontal axis) and two factors denoted below relative to a particular phase (time) in the life of the concrete structure, namely:

2A the importance of decisions made at different stages in the life, and

2B the influence that these decisions have on the life-cycle cost of a concrete structure.

Factors 2A and 2B are portrayed in Fig. 5.3-2. This shows a high influence or impact in the early stages of the life of a concrete structure, but with a decreasing influence/impact as the concrete structure passes into the later stages of its life. The influence and impact of the timing of decisions upon total direct life-cycle costs of a facility will clearly be greatest when the conceptual development is being undertaken. Impact diminishes greatly for decisions made during the later stages of the life-cycle.

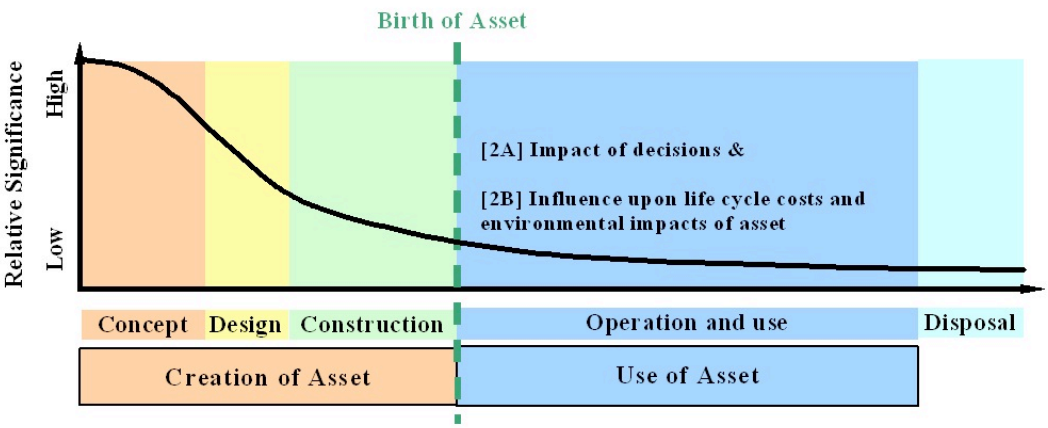

Fig. 5.3-2: Relative impact upon the performance of a concrete structure and upon life-cycle costs and environmental impacts of the timing of making a change

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ISO 15686: Part 5 (2007) suggests that up to 80% of the operation, maintenance and replacement costs of a building are influenced by the first 20% of the design process. Figure 7 in ISO 15686: Part 5 presents a curve indicating the scope for life cycle cost savings during the various life cycle phases of project for the construction of an asset. This curve defines a similar relationship to that portrayed in Fig. 5.3-2.

It will be noted that there is an inverse relationship between the magnitude of the influence of decision being made (Fig. 5.3-2) and the level of knowledge about the performance of the concrete structure (Fig. 5.3-1) at that particular phase in the life of the concrete structure. These would typically be expected to "cross-over" towards the end of the design phase. The conflict which arises between the magnitude of the influence of decision being made (Fig. 5.3-2) and the level of knowledge about the performance of the concrete structure (Fig. 5.3-1) is often referred to as the "Decision Paradox".

Total direct life-cycle operational costs could be an order of magnitude (or more) greater than the construction cost, but depends on the nature of the concrete structure being considered. (*Note: Figs. 5.3-1 and 5.3-2 consider only direct costs and do not address the much larger business related expenditures associated with the use of the facility, as discussed above.*)

Design and through-life management strategies for concrete structures can be summarised somewhat crudely by two conflicting ideologies, namely:

Approach 1. Buy cheap (low initial or first cost) and pay more at a later stage via higher through-life operational cost.

Approach 2. Pay more initially (higher initial or first cost), but gain from a reduced through-life operational cost.

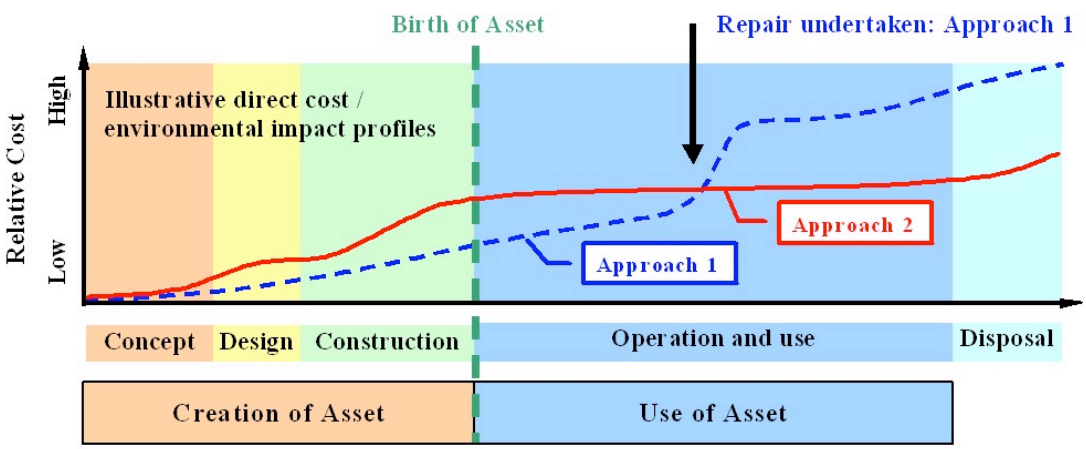

Fig. 5.3-3: Illustrative cumulative direct cost and environmental impact profiles of a concrete structure for two design and operation strategies

Fig. 5.3-3 shows notional cumulative costs and environmental impacts; it illustrates the importance of early decisions. It illustrates the value of committing significant pre-construction funding to the acquisition of adequate knowledge about the through-life performance of a structure and its component materials and the value of controlling and verifying construction processes in an effective manner. This is necessary in order that the (post-construction) behaviour of the structure during operation and use will conform to the required performance levels. In Fig. 5.3-3 the blue line represents Approach 1 described above; with the red line representing Approach 2. The difference between the red and blue lines symbol-

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ises the additional investment or saving being made. Where the red line is above the blue line this implies that extra expenditure is being incurred during planning and stages of construction. Later when the red line is below the blue line, this symbolises that a through-life return is being gained on the additional investment made earlier.

A practical illustration of the balance between first-cost and through-life costs is given by the example of two marine piers constructed in Progreso in Yucatán, Mexico on the shores of the Gulf of Mexico (89°W, 21°N). The marine environment is very aggressive with high levels of chlorides present, as well as high atmospheric temperatures. The first pier was built in the 1940s and the designers foresaw that the concrete used in construction would be of low quality. However stainless steel reinforcement was specified, which incurred higher construction costs (the red line in Fig. 5.3-3; Approach 2) than if normal carbon steel had been used (the blue line in Fig. 5.3-3; Approach 1). The 1940s pier has performed very well for over 60 years and there has been little deterioration or need for maintenance works – see Figs. 5.3-4A to C. This demonstrates that satisfactory service life performance can be achieved and that repairs are not inevitable in properly designed and adequately maintained structures.

A neighbouring pier was built some 30 years later in the 1970s. In this instance the structure was reinforced with cheaper carbon steel – which has a much lower resistance to corrosion than stainless steel in the aggressive marine environment. Today the neighbouring pier is totally destroyed, with the only evidence of this newer neighbouring pier being the founding piles sticking out of the sea (see Fig. 5.3-4C and the yellow dashed ellipse in Fig. 5.3-4A).

This outcome might be summarised by saying that by the application of appropriate knowledge and expertise, the designers of the original pier were able to utilise *low performance concrete* to produce a *high performance concrete structure*. Clearly spending slightly more on the construction of the 1940s pier has paid significant dividends in terms of reducing whole-life costs by minimising expenditure required upon maintenance and repairs. However the neighbouring pier has to be considered to be an example of a *low performance concrete structure*. In this case buying cheap (lower first/direct cost) has resulted in a need to pay much more in the later stages by way of higher through-life operational costs, as the neighbouring pier would need to be rebuilt.

However there are additional factors relating to life cycle costs which may introduce other considerations into an owner's decision making process. For example, the cost of capital, as represented by bank interest or discount rates, may alter the balance between various technical options. Fig. 5.3-5 illustrates the influence of this factor upon service life design strategy and the decision on whether to assume that a repair is undertaken to the concrete structure during its service life. It will be seen that discount rates can have a profound impact upon the technical choices made. Lower discount rates favour a longer term perspective; that is Strategies 2 and 4 which would typically involving higher initial costs (Approach 2 described above). Higher discount rates favour a shorter term perspective; that is Strategies 1 and 3 (Approach 1 described above). The details of the life cycle costing exercise undertaken to demonstrate the observations made in Fig. 5.3-5 are explained below.

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Fig. 5.3-4A: Aerial view of Progreso piers. Original 1940s Progreso Pier still in service, with the remnants of the "modern" 1970s neighbouring pier shown ringed.

Fig. 5.3-4B: Original 1940s Progreso Pier. This is still in service after exposure to the very aggressive marine environment of the Gulf of Mexico for over 60 years.

Fig. 5.3-4C: Original 1940s stainless steel reinforced Progreso Pier, with remnants of the "modern" carbon steel reinforced 1970s neighbouring pier in the foreground. [Knudsen et al. (1999)].

The life cycle costing exercise was undertaken for a 300 m by 10 m concrete pier founded upon concrete piles on the coast of the North Atlantic. Life cycle costs were calculated for the following four service life design strategies:

Strategy 1. A traditional design assuming ordinary Portland cement concrete adopting traditional construction methods and with repairs only being undertaken when extensive corrosion damage causing cracking and spalling of the concrete had been observed by means of regular inspections.

Strategy 2. A design assuming the use of very high performance concrete, large concrete cover values, optimal execution and curing procedures, intensive quality assurance procedures and quality control on site. It is assumed that regular inspections are made, but that only minor local concrete repairs will be needed.

Strategy 3. A traditional structural design using ordinary Portland cement concrete but providing preventive cathodic protection as sacrificial anodes in the water to protect the submerged and tidal zones, and prepared for cathodic protection by ensuring electric continuity of the reinforcement and providing cast-in anode bands in the parts above the water level. The external components of

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the impressed current system would only be provided and energised when local corrosion initiation was observed.

Strategy 4. A traditional structural design assuming ordinary Portland cement concrete and traditional construction methods. However, 50% of the reinforcement is assumed to be replaced with stainless steel reinforcement. Regular inspections are assumed to be carried out.

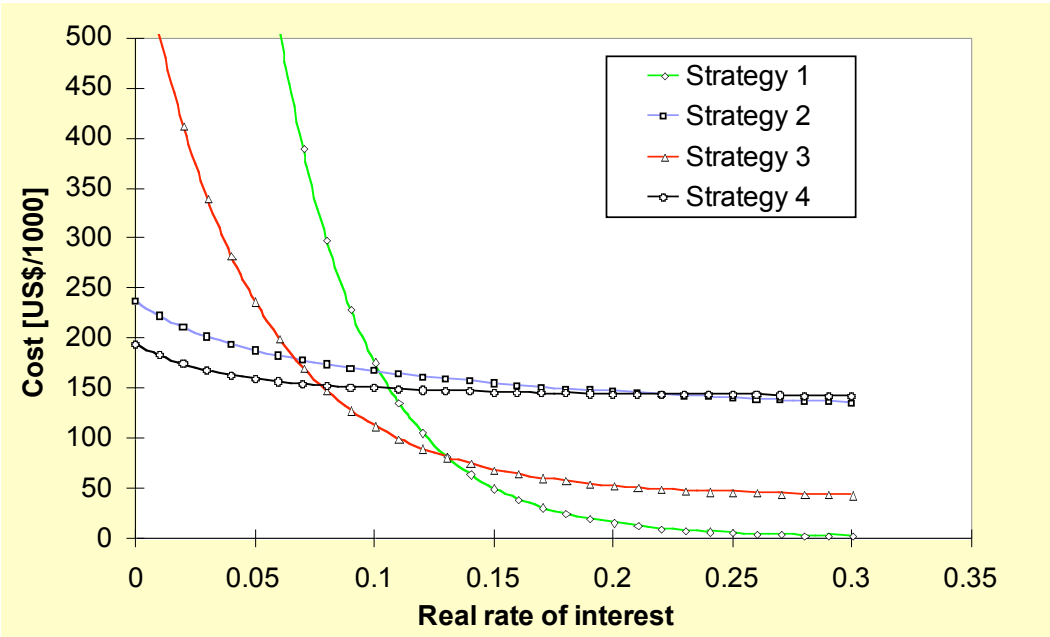

Fig. 5.3-5: Life cycle costs for four different service life design strategies – Influence of financial discount rates upon the choice of service life design strategy for a concrete quay exposed to a marine environment.

Note to Fig. 5.3-5: Summary of associated service life design parameters

Required service life:	*50 years*
Climates:	*10 deg C and 30 deg C*
Design approach:	*Four alternative repair strategies – utilising probabilistic service life performance analysis*

Strategy 1: Traditional concrete grade & carbon steel, allowing for one repair.
Strategy 2: High Performance Concrete and large cover to carbon steel.
Strategy 3: Traditional concrete grade & carbon steel, with the use of cathodic protection later.
Strategy 4: Traditional concrete grade & carbon steel, selective use of stainless steel rebar.

Realistic life cycle costs were taken for all elements of the above strategies, with a probabilistic calculation of the service life costs being made for each strategy taking account of the impact on these costs of the real rate of interest. To illustrate the influence of the real rate of interest, values up to 30% were considered.

The outcome illustrates the two major consequences upon the choice of service life design strategy if this is made solely on the basis of life-cycle cost.

1. The traditional approach (Strategy 1) and the adoption of cathodic prevention (Strategy 3) incur very high relative life cycle costs when low real rates of interest (discount rates). Conversely when discount rates are high it becomes advantageous to postpone costs as long as possible and Strategies 1 and 3 become the most cost effective. However, it also needs to be recognised that in these circumstances

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Strategies 1 and 3 are likely to incur considerably greater environmental impacts than the alternative Strategies 2 and 4. This is because of the additional resources consumed, wastes produced and associated impacts resulting from the need to undertake repair works during the design service life period for Strategies 1 and 3.

2. That the life cycle costs for Strategies 2 and 4 are minimised when real rates of interest (discount rates) are low. Strategies 2 and 4 typically involving higher initial costs, but lower through-life costs, than the alternative strategies considered. The extra construction costs incurred for Strategy 2 arise because of the additional measures taken with respect to design, concrete quality and execution. Strategy 4 solves the critical reinforcement corrosion problem by the use of corrosion resistant stainless steel reinforcement, which incurs higher material cost. In this example, in the case of Strategy 2 uncertainty could remain about the durability that will actually be achieved when the concrete structure is built because of issues such as the availability of concreting materials of adequate quality, plus the experience and workmanship of the local workforce. From a durability point of view Strategy 4 should be less sensitive to factors such as the quality of the concrete materials, the workmanship exercised during construction and the standard of through-life care/maintenance.

Thus it will be appreciated from the comments made above that decisions made and activities undertaken in the earlier phases in the life of a concrete structure can have a profound impact upon its subsequent through-life performance.

There is also concern amongst some experienced engineers and construction personnel that quality management procedures employed during the design and construction of concrete structures do not always achieve satisfactory product quality and hence durability. An aspect of these concerns is that the overall standard of workmanship being achieved during the construction phase (referred to as the execution stage) is not always sufficiently high. However, these problems will often have their roots within earlier stages of the project; such as when the design, detailing or material specification tasks are carried out. Special efforts, primarily in the form of pre-planning construction activities and verification of the processes adopted during execution, can be required to overcome these potential difficulties.

In addition there can be a marked difference between employing what in isolation might be taken to be *high performance materials* and actually achieving a *high performance concrete structure* which has adequate durability, as the example of the Progreso piers illustrates.

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Box 5.3-1: WHOLE-LIFE COST

Whole life costing (WLC) is essentially a tool which can be used to enable owners to appraise projects and assist them in making decisions about different projects or options competing for limited financial resources. It enables expenditure to be discounted over time and normalised to a common base year. The technique is flexible and can, if desired, incorporate many parameters such as:

- initial capital cost;
- financial repayment options;
- revenue streams;
- maintenance costs;
- service life/the operational lifetime of the structure or asset;
- demolition or other end of life costs.

Standard mathematical tools exist and are relatively simple to apply. A choice is made of the parameters most relevant to the decision to be made and an appropriate discount rate for changes in monetary value in the future is decided. The resultant WLC calculated is then compared against other competing expenditure in monetary terms at today's value. Whole life costing techniques can be applied at various stages throughout the construction process.

The key steps in a simple whole life costing analysis are as follows:

1. Determine and assess required service life and what constitutes the end of service life for individual elements and the structure as a whole.

2. For each design option determine the rate of deterioration, the replacement interval and the fraction requiring replacement for each element using appropriate service life forecasting techniques.

3. Determine the level of deterioration at which intervention is required.

4. Ascribe values to initial capital costs.

5. Ascribe values to maintenance, repair and replacement costs including:

   - routine maintenance costs (using data for the frequency from Step 2);
   - costs associated with factors such as traffic management and delays;
   - costs associated with loss of use of a structure during planned maintenance/repair.

6. Use the calculated values to estimate the whole life cost of the structure. Costs of future work can be discounted to net present values using the expression:

$$\text{Present cost} = \frac{\text{Future cost (after t years)}}{(1+r)^t}$$

Box 5.3-1 eqn. 1

where r is the discount rate and t is the time period in years. The discount rate (or cost of capital) is given by the following expression:

$$\text{Discount rate, } r = \frac{(1+\text{interest rate})}{(1+\text{inflation rate})} - 1$$

Box 5.3-1 eqn. 2

Additional aspects of whole-life costing are discussed by De Sitter (1984), Royal Academy of Engineering (1998), Somerville (1998), Clift and Bourke (1999), Edwards et al. (2000), ASTM E 917-02 (2002), Kesäläinen (2003), Indian Roads Congress (2004), Bourke et al. (2005) and ISO 15686-5 (2007).

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5.3.3 Wider societal sustainability perspective

These economic considerations need to be set within the wider framework of sustainability and environment impact related issues which society now expects the construction industry to address. Sustainability is not considered solely in environmental terms and there are a number of interacting and potentially conflicting issues and factors that need to be addressed and balanced during the process of designing a concrete structure. Fig. 5.3-6 illustrates the primary headings under which matters relating to this topic are often broadly grouped. Again these relate to both the construction phase and the processes for through-life management. The *Functional factors* are often referred to as being technical requirements, whereas the other topics concerned with *Economic, Socio-cultural and Environmental factors* are sometimes referred to collectively as being non-technical requirements. Fig. 5.3-6 illustrates one basis for classifying these components.

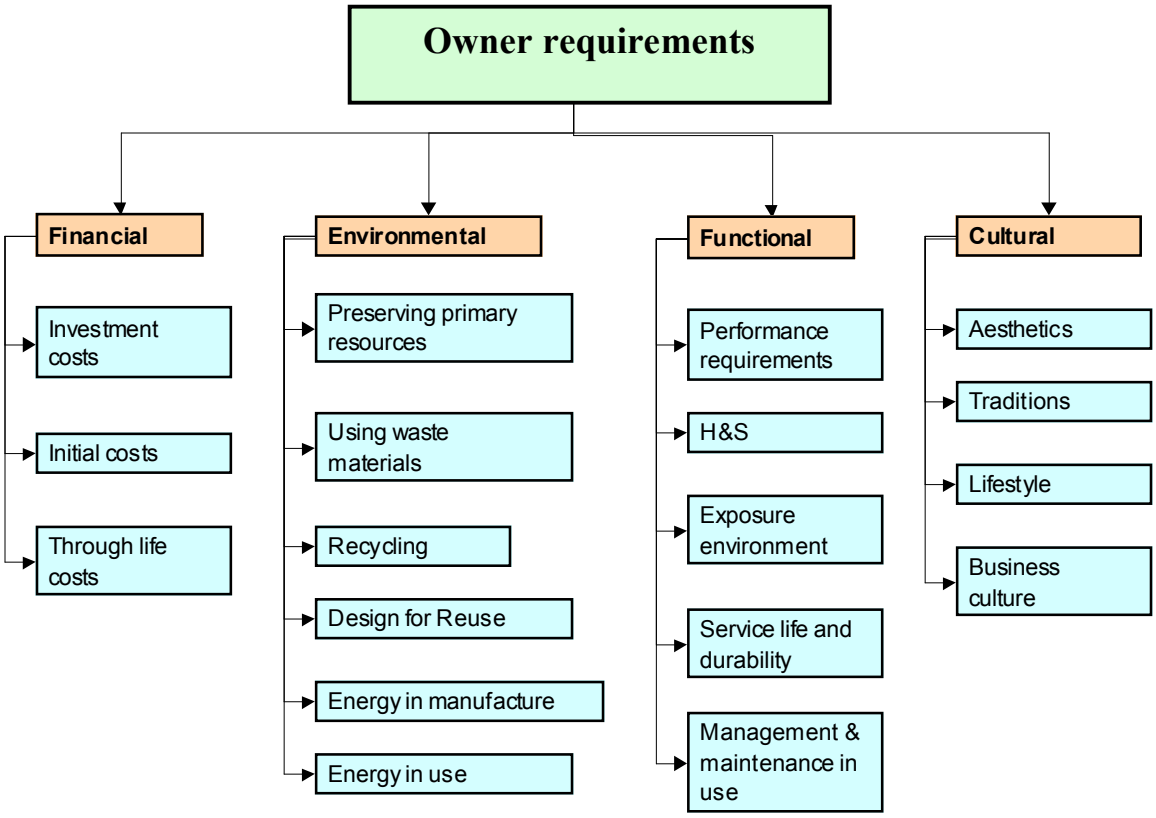

Fig. 5.3-6: A potential breakdown of the components of sustainable construction

There will generally be a number of potentially conflicting issues and factors that need to be addressed and balanced when designing a concrete structure. The weighting of the issues arising may well differ, both in respect of the options available and in relation to owner needs. The flexibility of the potential options to accommodate changing business need may also be a factor. Issues of maintaining functionality, meeting changes of use or business focus may well be higher order requirements than cost. However in some cases the issue of affordability or overriding non-technical factors may necessitate consideration of what might be considered to be sub-optimal technical solutions.

Fig. 5.3-7 presents a schematic showing conceptual the goal of combining the components of sustainable construction, with the objective being to achieve an acceptable balance between all the considered factors. To do this a formal scoring and evaluation system is required which provides a method of weighting the influencing factors and evaluating the potential options. The

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CONREPNET project [Matthews et al. (2007)] and the LIFECON project (see below) examined a number of tools supporting option evaluation and decision making in such circumstances. These tools include various methods for multi-parameter decision making, such as:

- Multi-attribute decision aid (MADA) (Kesäläinen (2003)]
- Quality function deployment (QFD) (Kesäläinen (2003)]
- Quality function deployment (QFD) in combinations with reliability, availability, maintainability and safety (RAMS) evaluation [Söderqvist & Vesikari (2003)]

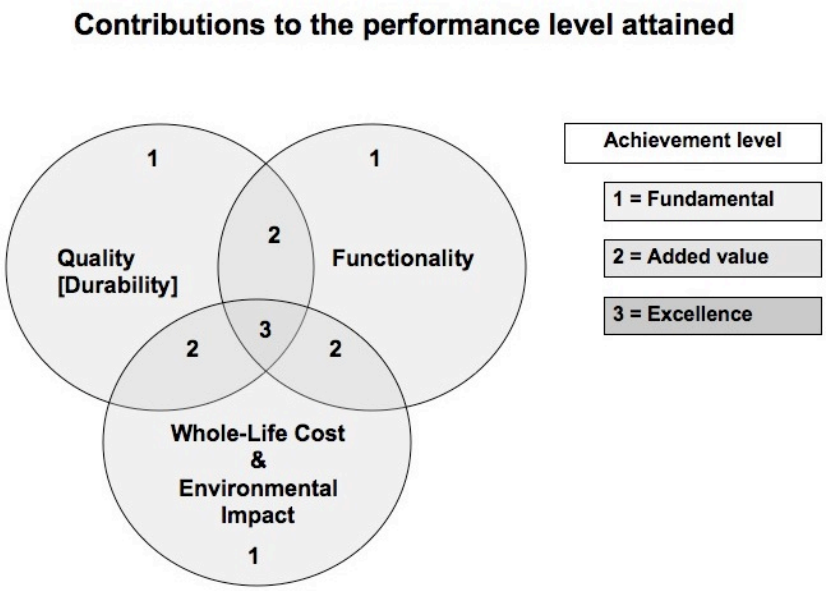

Fig. 5.3-7: Schematic combination of the components of sustainable construction.

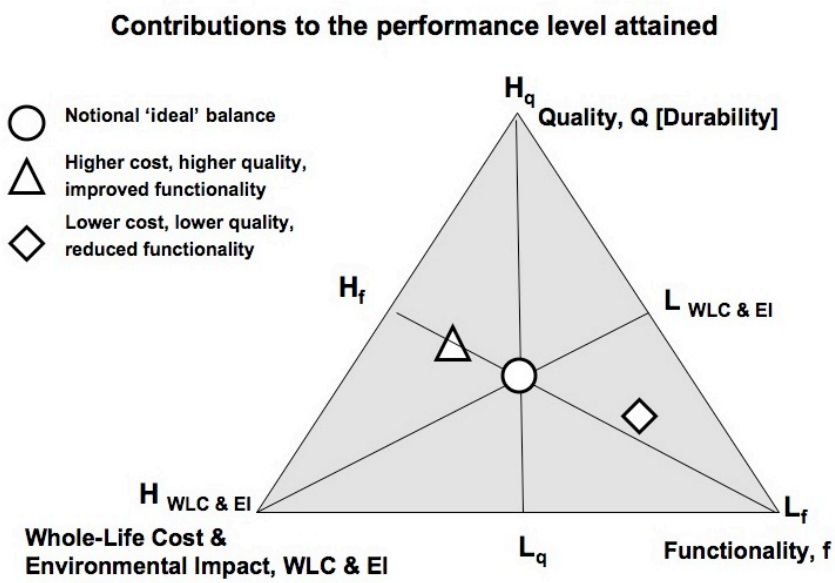

Fig. 5.3-8: Achieving a balance between the components of sustainable construction – Schematic illustration of various compromises in the performance levels attained.

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Fig. 5.3-8 illustrates graphically different balances achieved between the components of sustainable construction, with the schematic illustrating various compromises in the performance levels attained in terms of whole-life cost, environmental impact, functional performance and the quality/durability of the constructed facility.

There are a number of environmental assessment methodologies available that considered these factors within formal scoring systems which give an overall evaluation.

5.3.4 Service life and sustainability considerations

The flexibility in composition and in the engineering properties of concrete – it is effectively an extended family of materials – creates opportunities to use it in environmentally beneficial ways. It can be tailor-made to suit a great variety of uses. It is made all around the world from a diverse range of indigenous materials. The designer needs to play to its strengths; such as its high thermal mass, its good sound insulation properties and the flexibility to form it into almost any shape. For example, these properties allow energy efficiency to be designed into the fabric of a building. Concrete can resist many aggressive chemicals. For many years secondary materials, such as ground granulated blast furnace slag (ggbs) and pulverised-fuel ash (pfa), have been used to enhance the durability of hardened concrete in aggressive environments. Both ggbs and pfa are wastes from other industrial processes; namely steel manufacture and the coal-fired generation of electricity, respectively. Today the ability to consume wastes and by-products from other industries is an increasingly important issue and central to wider environmental and sustainability considerations.

However, there are also significant environmental impacts that have to be tackled. For example, in the UK concrete is the largest component of the construction and demolition waste stream, contributing nearly 53% by volume. Re-use of such material is both necessary and is generally feasible. Most of the cement produced and a high proportion of the aggregate which is quarried, go into concrete. Aggregate extraction is land-hungry and involves much heavy transport.

Portland cement and blended Portland cements are currently the only economically viable binders for concrete which provide the both the performance and durability requirements needed for the range of environmental conditions to which concrete is exposed. However, cement by its nature is an energy and CO_2 intensive material. In the UK, which is an unusually small user of concrete relative to the size of its population and economy, it is estimated that production of concrete accounts for some 2.5% of the UK's total CO_2 emissions [Dti (1997)]. Worldwide it is believed that up to 8% of CO_2 emissions are the result of concrete production [Idorn (1997)], with some 6% of that coming from the production of cement. Clearly the production and use of concrete has a significant environmental impact.

The concepts and processes required to balance the various and potentially conflicting components of sustainable construction have collectively been termed 'lifetime engineering'. Research in this field is leading towards the development of an integrated, holistic approach to the design and management of buildings and structures which takes account of all aspects of design and construction practice, including recognising the benefit of integrated teams and supply chains. This type of approach is seeking to encompass both cost-driven and sustainability-driven initiatives aimed at delivering sustainable construction at an affordable whole life cost.

Service life design concepts (see Boxes 5.1-1 and 5.1-2) can be key facilitators for the delivery of sustainable construction. When used in conjunction with appropriate conceptual and detailed design, in association with properly planned and delivered execution, it can help

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meet the client's needs in terms of the performance of the structure over its lifetime in an economic manner. Somerville (1999) has suggested that the most sustainable construction strategy is one in which the quality of construction is sufficient to enable the structure to meet its envisaged service life requirements. Failure in sustainability terms can arise from both over- and under-design. Somerville (1999) suggests that, for example, over-design might occur where more or higher quality materials have been used than is needed to meet the design requirements. It might also arise through under-design or as a result of poor quality design and workmanship [Jones et al. (1997)] which results in the need for premature repair or replacement. These concepts are illustrated in Fig. 5.3-9. The objective would be to move construction practice closer to what Somerville has termed the 'plateau of sustainable construction'.

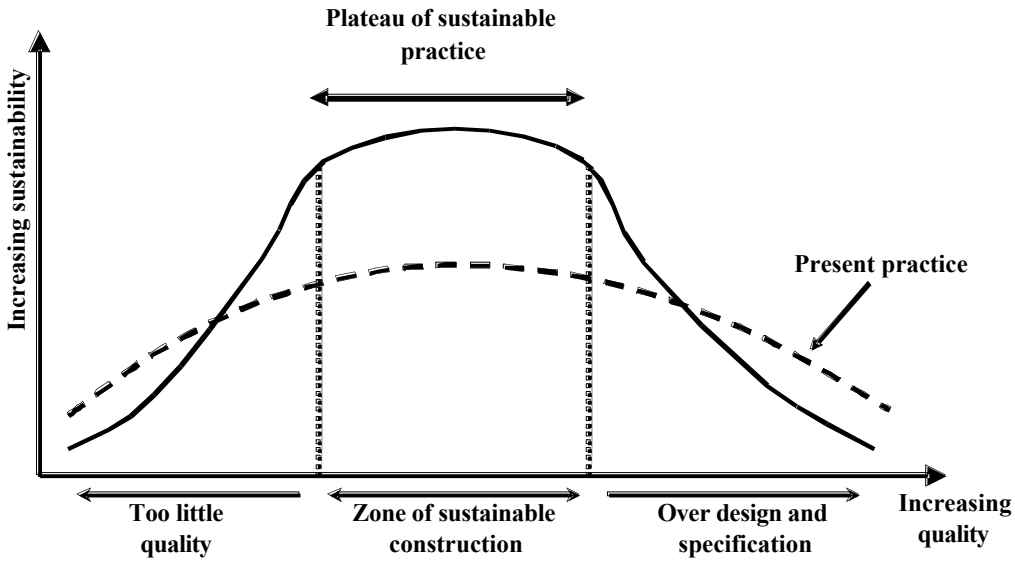

Fig. 5.3-9: The plateau of sustainable construction [Somerville (1999)]

It is felt that significant progress could be made towards these goals and objectives by means of relatively simple, but intelligent, decisions about design and detailing, taking account of the characteristics of the structure's service environment. Overwhelmingly these considerations involve the control of water and limiting the access of moisture to the structure. Thus, it is suspected that much could be achieved, especially in respect of the general population of structures and buildings, without recourse to special materials or sophisticated analyses of failure mechanisms.

A service life design approach provides a basis for addressing many aspects of these issues. The key elements of the service life design system are listed below, reproduced again for the convenience of the reader. These act as a reminder that durability problems can arise as a result of deficiencies which occur in any one of a number of stages throughout the design, specification and construction process:

- client/owner brief;
- assessment of environmental loads;
- definition of required performance under environmental loads;
- development of project specification;
- conceptual and detailed design phases;

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- execution of works;
- through-life care and maintenance.

Fig. 5.3-10 shows a simplified schematic diagram of the service life design process, with construction process sustainability related activities being shown on the right-hand side of the diagram. These are being carried out in parallel or in an iterative manner. The activities would also link with the general structural design process, shown in parallel with the service life design process in Fig. 5.2-1.

Clearly the actual service life design process is somewhat more complicated than the diagrammatic representation portrays. Fig. 5.3-10 puts a strong emphasis on the client brief and conceptual design stages, whilst embracing structural design. Execution, construction quality and through-life care and maintenance in use are crucial if the intended performance is to be achieved in practice. As noted previously, these factors need be considered at the conceptual design stage. The importance of good communication up and down the supply chain and across the service life design process is also highlighted.

Also included in the diagram is reference to the BRE durability assessment procedure, which is an auditing procedure applied through-out the service life design process. This type of additional activity, linked to wider quality assurance goals, would be intended to improve the likelihood of achieving the client's/owner's requirements in terms of durability, sustainability and functional performance for the anticipated whole life cost and environmental impacts.

5.4 Mechanisms that may cause deterioration or damage to concrete structures

5.4.1 Introduction

Whilst most concrete structures provide satisfactory performance over an acceptably long service life period, deterioration processes affect all structures and all materials to varying degrees. As a result through-life structure management and care is needed. This section gives a brief introduction to a number of potential deterioration mechanisms that may affect concrete structures, along with some of the implications of such deterioration.

Deterioration of concrete structures may arise for a number of reasons including:

- poor design, specification, detailing or execution of construction;
- service environment that is more aggressive than anticipated (e.g. chlorides present);
- the structure experiences ageing processes or increased loading;
- lack of funds for through-life care and maintenance;
- poor planning or implementation of maintenance operations.

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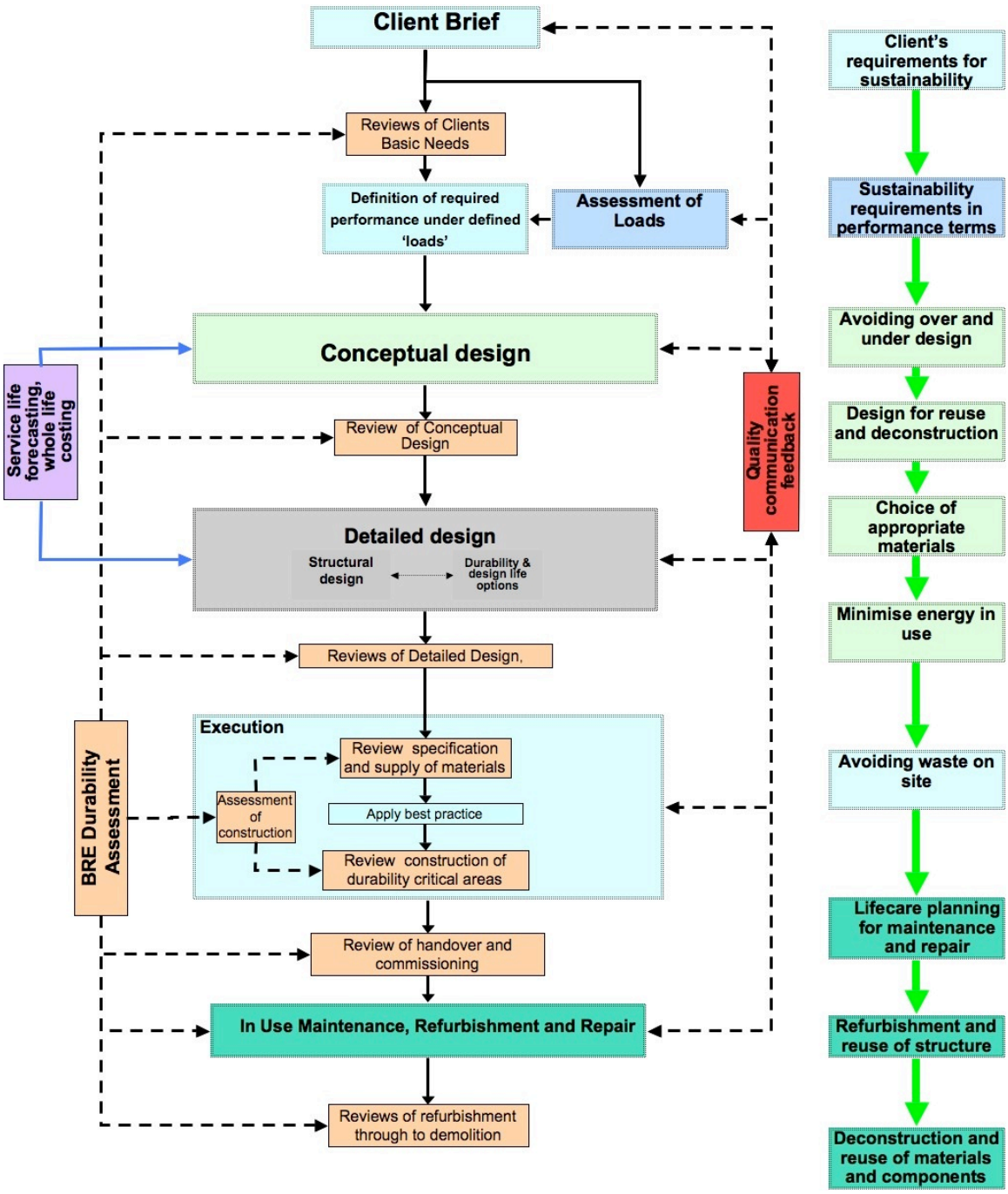

Fig. 5.3-10: Schematic diagram illustrating the BRE service life design process with associated links to sustainability-related initiatives [Nixon (2002)]

Jones et al. [BCA (1997)] carried out a literature survey and showed that 50% of reported cases of concrete deterioration were associated with reinforcement corrosion due to either chlorides or carbonation. The results of this survey are summarised in Fig. 5.4-1.

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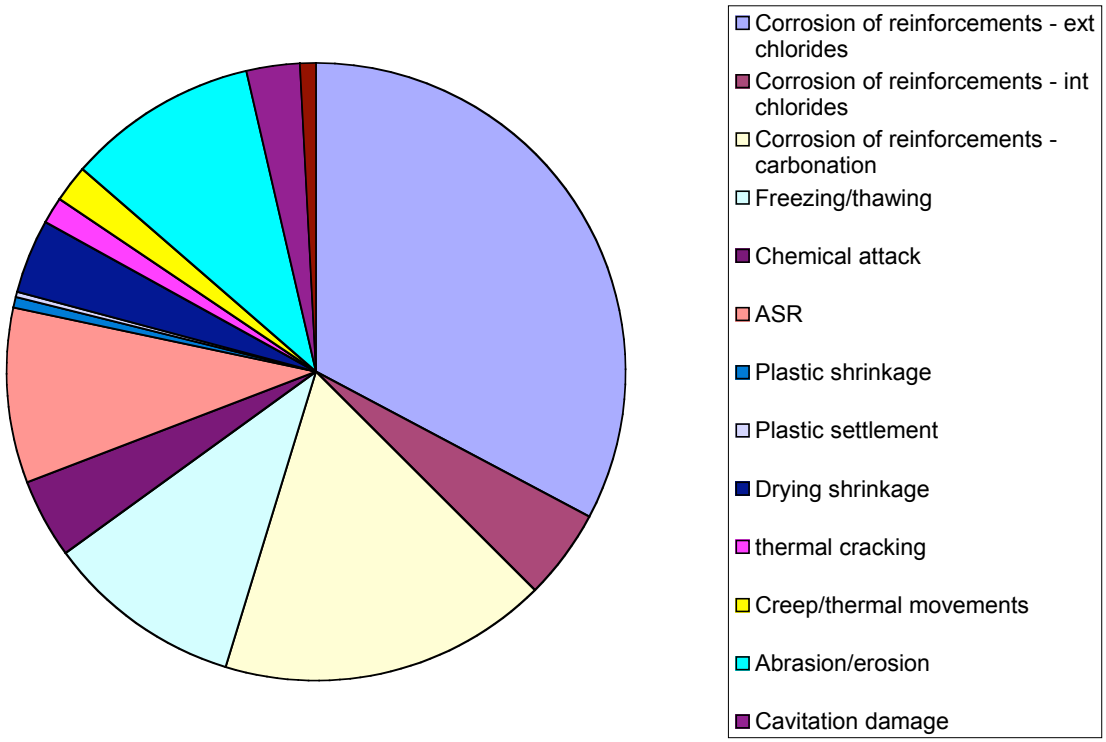

Fig. 5.4-1: Deterioration processes affecting concrete [Jones et al. (1997)]

Fig. 5.4-1 gives an indication of the relative importance of various deterioration processes. Jones et al. [BCA (1997)] also showed that the proportions varied for different types of structure (e.g. buildings, compared with highway structures). They also attempted to identify the root causes of the deterioration and their analysis is shown in Fig. 5.4-2. An inability to resist the effects of an aggressive environment was the biggest single group; combined with contributory design, materials and/or workmanship factors. However, of comparable significance were cases where design, detailing and workmanship caused specific deficiencies.

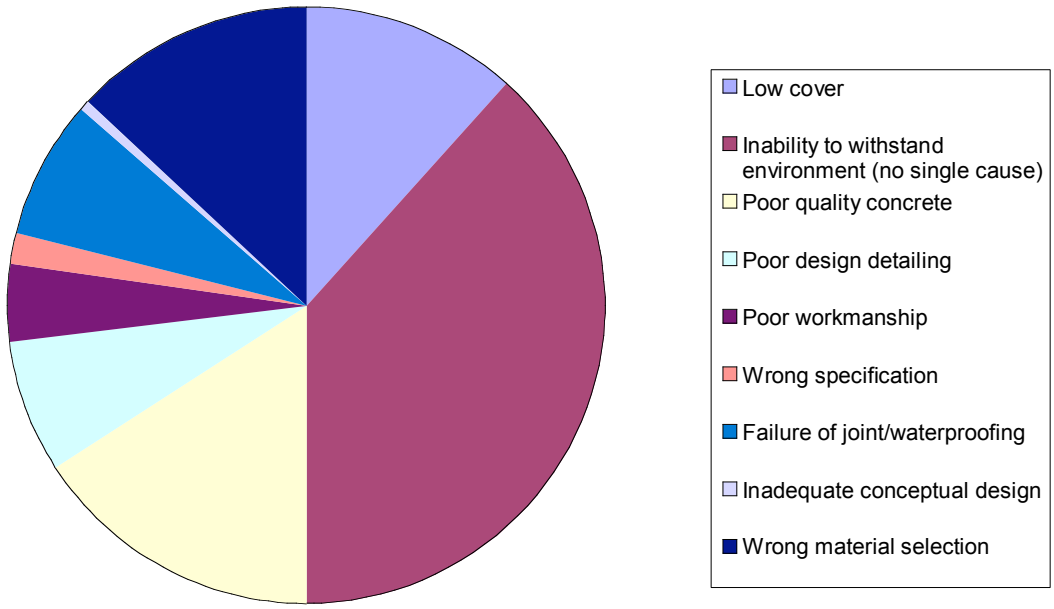

Fig. 5.4-2: Process factors contributing to inadequate durability [Jones et al. (1997)]

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As the chemical reactions associated with deterioration processes require moisture, the rate at which they progress will typically be much less when little moisture is available. The presence of cracks in concrete can facilitate the penetration moisture, potentially 'fuelling' ongoing deterioration processes, along with aggressive chemical species such as chlorides and sulfates.

Reinforced concrete elements subject to loads and external actions are expected to exhibit some degree of cracking. Cracks may also arise from other causes, such as from early age thermal effects and plastic shrinkage. Cracks and crack patterns exhibit different characteristics, as is illustrated in Fig. 5.4-3, depending upon the underlying mechanism causing them. For example, plastic shrinkage cracks tend to form in an irregular pattern, whereas those due to plastic settlement more commonly form over and follow the line of the reinforcing bars.

Fig. 5.4-3 shows seven types of cracking and illustrates their occurrence by means of 14 hypothetical situations (A to N). These examples represent most of the types of cracking which may occur during the lifetime of a concrete structure. The different types of cracks tend to occur at different ages, which help in their differentiation, as noted below [after Fookes (1976)].

Type of 'intrinsic' cracking	***Typical age on appearance***
• Plastic settlement	Up to about 3 hours
• Plastic shrinkage	Up to about 6 hours
• Early thermal contraction	One day to perhaps three weeks
• Long-term drying shrinkage	Some months to several years or longer
• Crazing	One to seven days – perhaps much later
• Corrosion of reinforcement	Several to many years
• Alkali-silica/alkali-aggregate reaction	A few months to several years

Cracking may also arise due to other influences not illustrated in Fig. 5.4-3. Potentially there is an extensive range of circumstances which might cause damage or result in some form of attack on the concrete structure; examples of these include those listed below.

Type of damage or attack	***Typical age on appearance***
• Chemical attack	A few months to several years
• Freeze-thaw damage	First severe winter season
• Damage from structural loading	Depends on timing of application of load
• Damage due to thermal movements	Possibly one year, maybe longer

The implications of cracking, crack width and crack orientation upon reinforcement corrosion, together with the associated implications for the durability of a concrete structure, are discussed in Section 5.5.7.

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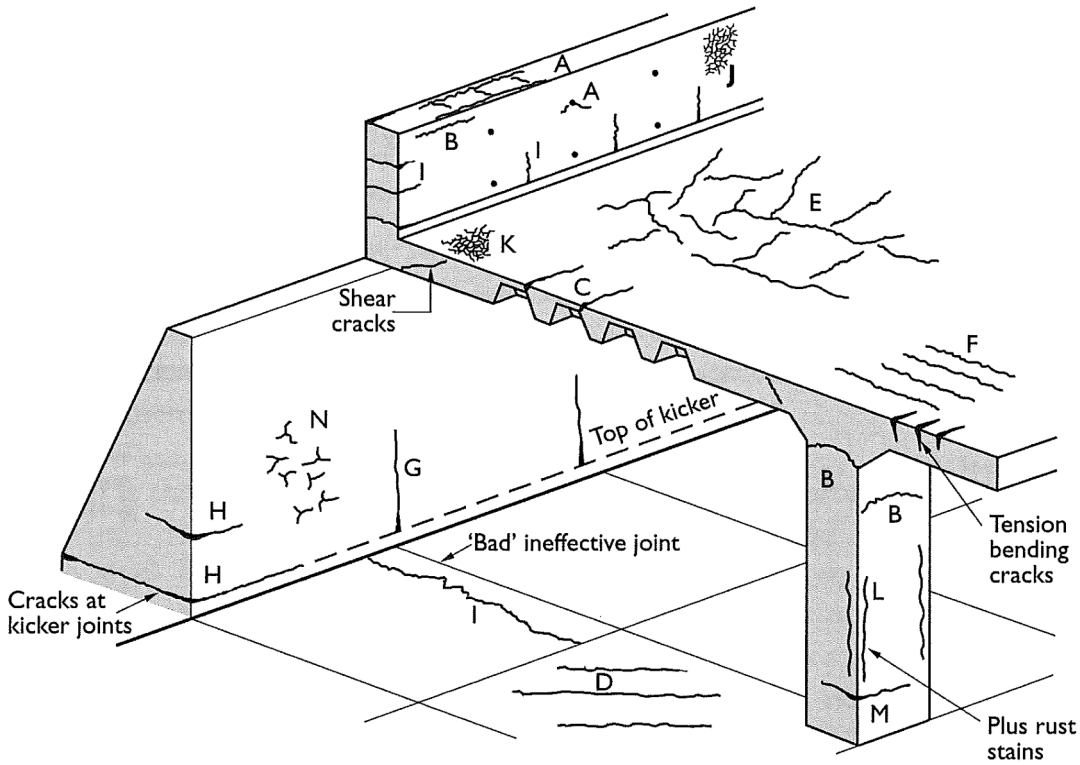

Type of cracking

Plastic settlement	A, B, C	Crazing	J, K
Plastic shrinkage	D, E, F	Corrosion of reinforcement	L, M
Early thermal contraction	G, H	Alkali-silica reaction	N
Long-term drying shrinkage	I		

Fig. 5.4-3: Schematic representation of various types of cracks which can occur in concrete structures [Concrete Society Technical Report 22 (1992)]. Reproduced with permission.

In order to create an adequately durable concrete structure with a satisfactorily long-service, the designer needs to understand the hazards and deterioration mechanisms to which the structure may be subjected. Key to this is to know and understand the transport mechanisms which move liquids and gases into and out of the concrete, but also around within it. Whilst Section 3.1.8 reviews the transport mechanisms and wider durability issues with concrete as a material, this Section is more concerned with the influence that potential deterioration mechanisms have upon the durability of overall concrete structure, especially their potential impact upon reinforcement corrosion.

5.4.2 Overview of deterioration and damage mechanisms, excluding accidental actions

This section provides an overview of deterioration and damage mechanisms, but excludes damage due to accidental actions such as fire and impact. In this section potential deterioration mechanisms causing durability problems in reinforced concrete structures are considered on the basis of:

- physical deterioration and damage processes in concrete;
- chemical deterioration processes in concrete;

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- biological deterioration processes in concrete;
- corrosion of reinforcement.

Examples of some different forms of deterioration and damage to reinforced concrete structures are shown in Fig. 5.4-4, Parts A and B.

Physical deterioration and damage inducing processes causing cracking and other effects in concrete structures can arise from various causes including:

- plastic settlement and shrinkage;
- early age thermal cracking;
- drying shrinkage, including autogenous shrinkage;
- abrasion and erosion;
- freeze and thaw effects (frost attack and surface scaling);
- physical salt crystallisation and scaling;
- thermal effects, movement and cracking;
- weathering, discolouration and soiling of surfaces.

Physical damage in concrete structures can also arise for a variety of additional reasons. These might include overloading and accidental actions such as, fire, impacts and accidents; but also from events such as malicious attacks; and also earthquake/seismic events. Such physical damage is not considered to be a service life design or a durability issue, accordingly it is not addressed here.

Chemical deterioration processes in concrete structures can arise from a variety of causes, these can be grouped into two main forms of attack:

- External chemical attack caused by acids, sulfates, nitrates, soft water (leaching), etc. and includes reactions such as the thaumasite form of sulfate attack (TSA).

  In some eyes attack from biological/bacterial agents might be considered to be an example of external chemical attack, because the attack on the concrete in many instances arises from acids produced by the bacteria. However, in this case this form of attack is presented a separate class (see below).

- Internal deleterious chemical reactions such as leaching, alkali-aggregate reactions (AAR) and alkali-silicate reactions (ASR), delayed ettringite formation (DEF), etc.

Whilst the high pH of concrete generally makes it resistant to biological attack, deterioration from biological processes in concrete structures take two main forms:

- biological growth on concrete structures, which leads to physical and mechanical damage to the concrete;
- bacterial attack through the production of organic or mineral acids that react with the cement paste. These conditions also occasionally occur in sewers and other situations.

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Corrosion of reinforcement can cause cracking of the concrete, spalled concrete and potentially a decrease in load-bearing capacity due to factors such as a reduction in the cross sectional area of the reinforcement and disruption of the bond and anchorage of the reinforcing bars. The severity of the damage caused typically depends upon the length of time corrosion has been taking place. However, the rate at which corrosion occurs varies markedly depending upon circumstances.

The main causes of reinforcement corrosion are:

- Carbonation of the concrete, which is caused by the permeation of atmospheric carbon dioxide into the concrete over a period of years that changes the internal chemistry of the surface zone concrete. The effect is to remove the natural passivity protection afforded to the steel by the concrete, subsequently allowing corrosion to develop. The loss of protection usually takes place slowly over many years, but is dependent upon the quality of the placed concrete.
- The presence of significant levels of chloride ions in the concrete, either incorporated into the concrete when the structure was built or by subsequent ingress into the concrete from the external environment (e.g. through the use of de-icing salts or from the marine environment).

In some circumstances steel reinforcement can be damaged by organic or mineral acids produced by bacteria. These acids initially react with the cement paste, causing damage and deterioration in the concrete. The acid will eventually be able attack exposed steel. These conditions occasionally occur in sewers and other situations. These issues are considered in the section concerned with biological attack on concrete.

The deterioration mechanisms causing durability problems in reinforced concrete structures are also sometimes classified on the following basis:

- corrosion of reinforcement;
- deterioration of concrete itself, due to physical and chemical deterioration processes, etc.

Physical damage to the concrete structure from deterioration and other actions. Table 5.4-1 presents an overview of some of the deterioration mechanisms and their potential occurrence in some types of structure or components of structures. It should be realised that the likelihood of deterioration occurring due to a particular mechanism may be small and is very dependent upon local circumstances in respect of the nature and condition of the structure and the environment to which it is subjected. Accordingly Table 5.4-1 should be taken as only giving a broad indication. It is important that appropriate consideration be given to the respective deterioration mechanisms for each structure and set of circumstances.

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Carbonation induced corrosion of a reinforcing bar

Chloride induced corrosion of reinforcing bar – Note pitting in surface

Marine damage and biological growths

Sulfate/thaumasite form of attack

Cracking: alkali-aggregate reaction

Cracking: freeze-thaw action

Fig. 5.4-4A: Examples of some forms of deterioration affecting concrete structures

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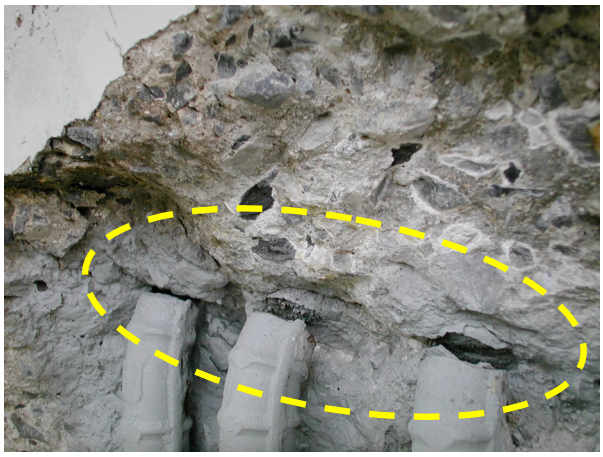
Damage to pier crosshead by water and freeze-thaw action (note icicles)

Carbonation induced corrosion in bridge soffit with inadequate cover to bars



Reinforcing bars snapped by severe ASR expansion (image from Japan)

Damage and spalling of concrete cover on bridge column – induced by fire effects

Fire damage to bridge beams

Severe impact damage to bridge beams

Fig. 5.4-4B: Examples of the effects of some forms of deterioration and also some accidental actions (fire and impact) upon concrete bridges

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Table 5.4-1: Overview of some deterioration mechanisms for some types of structure

Type of structure	**Corrosion of steel**		**Deterioration of concrete[1]**			**Physical damage**		
	CO_2-induced	**Chloride induced**	**Freeze/ thaw**	**External chemicals**	**Internal reactions**	**Impact/ Abrasion**	**Fire[5]**	**Seismic [2]**
Above ground buildings [3]	■			◘	◘		◘	◘
Industrial floors [4]				■		■		
Tunnels	■	◘	In Arctic latitudes	◘	◘		◘	◘
Concrete chimneys	■		◘	■	◘			◘
Sewage plants	■	■		■	◘			◘
Bridges	◘	■			◘	◘		◘
Car parks	◘	■	◘	◘	◘	◘	◘	◘
Swimming pools	◘	◘		◘	◘			◘
Coast marine structures	◘	■			◘	◘		◘
Dams (unreinforced)			■		■	Erosion		■
Foundations				■	◘			◘
Tanks and pipes	◘			■	◘			◘

Key:

■	Commonly affected by deterioration mechanism
◘	Sometimes affected by deterioration mechanism
	Infrequently affected by deterioration mechanism

The availability of moisture to fuel the deterioration reactions is usually the prime factor influencing whether deterioration progresses or not. If the concrete element concerned is in a dry environment, deterioration will generally progress very slowly, if at all. In a wetter environment the deterioration processes will be able to progress more rapidly.

Notes to Table 5.4-1

1 *Refer to Fig. 5.4-4 (Parts A and B) for examples of some forms of deterioration and damage which affect concrete structures.*
2 *Depends on location of building and circumstances, especially seismicity of region. This action is not considered in detail here*
3 *Above ground parts of buildings – this includes most types of buildings and facilities including commercial buildings, hotels, schools, hospitals etc.*
4 *Assumed to be within a building or protected from external environment & frost attack.*
5 *Accidental action – not considered in detail here.*

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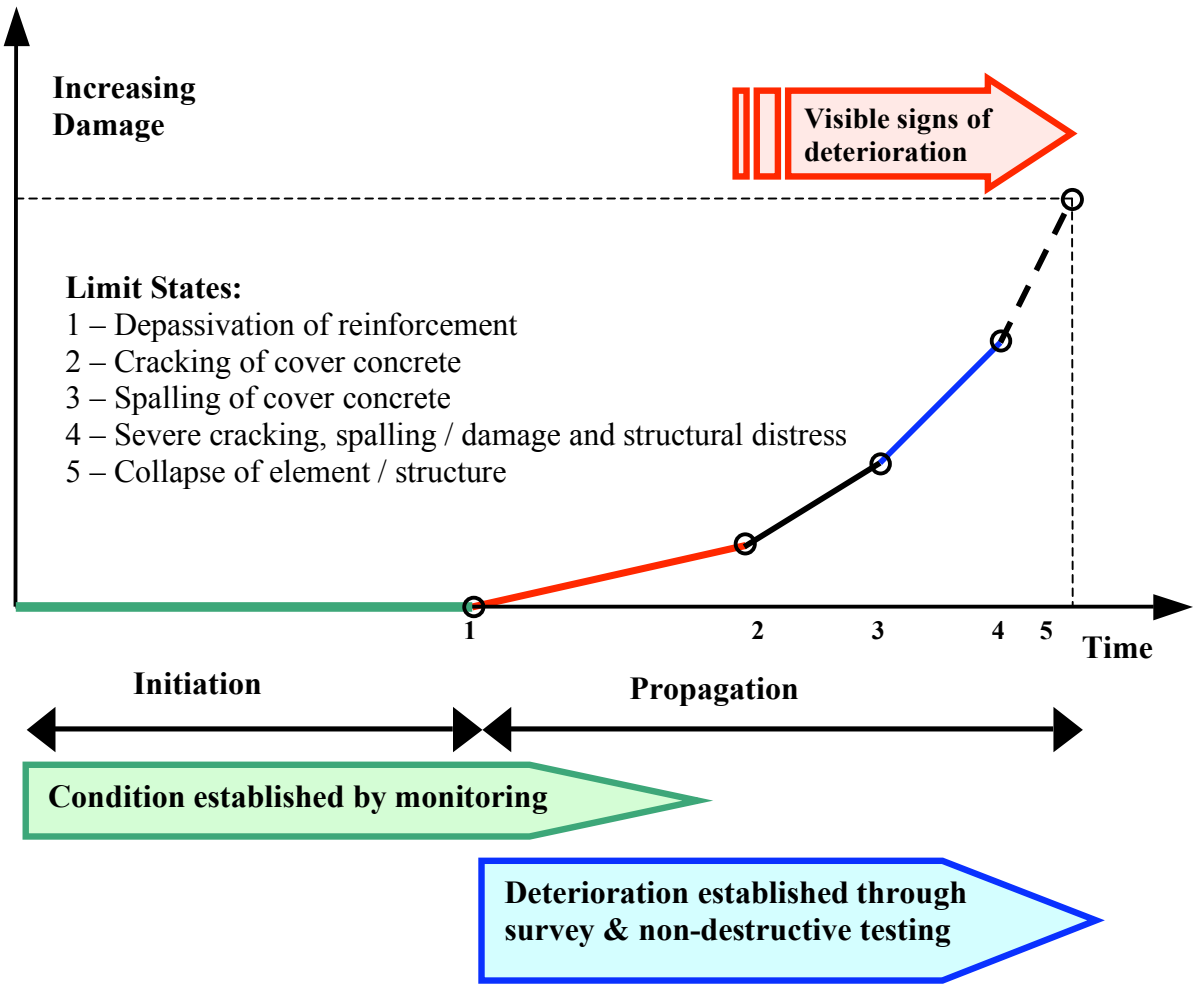

Fig. 5.4-5: Stages of corrosion induced deterioration of a concrete structure

Fig. 5.4-5 shows a number of stages in the development of deterioration of a concrete structure affected by carbonation induced reinforcement corrosion. These are:

Stage 1. Initial depassivation of the embedded steel reinforcement as the internal chemical environment within the concrete is changed by external environment influences. This removes the corrosion protection that the concrete naturally provides to the reinforcement, making it susceptible to corrosion induced damage (green line – resulting in Limit State 1 described in the diagram being reached). At this point there is no observable or actual physical damage.

Stage 2. The development of corrosion products arising from corrosion of the embedded steel reinforcement. As the corrosion products typically occupy a volume significantly larger than that of the original steel, this induces tension in the cover concrete and leads to the development of cracking (red line – resulting in Limit State 2 described in the diagram being reached). At this point there is observable damage (i.e. narrow cracks in concrete) but no significant loss of ultimate load bearing capacity (strength).

Stage 3. As corrosion progresses further and the cracks in the cover concrete develop and widen, this facilitates the penetration of more moisture and aggressive substances into the concrete promoting a faster rate of corrosion (steeper line) eventually leading to spalling of cover concrete (black line – resulting in Limit State 3 being reached). At this point there is observable damage (i.e. cracks and spall-

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ing of concrete) and probably a small reduction in ultimate load bearing capacity (strength) of the affected elements.

Stage 4. The continued development of corrosion an increased rate (steeper line), resulting in further cracking and spalling of cover concrete and the increasing loss of cross-sectional area to the embedded steel reinforcement (blue line – resulting in Limit State 4 being reached). At this point there is observable damage i.e. widespread and severe cracks and spalling of concrete, plus loss of section to reinforcing bars. There will also be a significant reduction in ultimate load bearing capacity (strength) of the affected elements through loss of bond between the reinforcement and the concrete, reinforcement anchorage failure or reduction in the cross-sectional area (strength) of the reinforcing bars affected. This level of corrosion induced damage will cause structural distress, impair the functional performance of the structure and pose a safety hazard to users.

Stage 5. If the process of corrosion is allowed to continue beyond Limit State 4, it is likely to proceed at an ever increasing rate (steeper line). The continued development of corrosion induced deterioration and damage (dashed black line) will exacerbate structural distress. This will eventually lead to the collapse (Limit State 5 being reached) of the affected elements or the entire structure, with potential hazard of injury or worse to structure users and a catastrophic disruption of the functional use of the structure. It might be expected that structures allowed to reach this condition would be classed as hazardous and that this could result in legal or other sanctions being taken against the owner and elements of the professional team involved.

Fig. 5.4-6 shows the changing condition of a deteriorating concrete structure with time and introduces the concept that at a particular time the degree of corrosion and associated damage varies across the structure. It can be seen that initially all the reinforcing steel in the structure is in a passive condition and that no corrosion is taking place. Over time an increasing proportion of the reinforcing steel/structure is affected by varying degrees of corrosion and associated cracking and spalling of the cover concrete. Thus as time goes by various parts of the structures become affected and the overall condition/structural strength of the element deteriorates.

When the deterioration process is a function of the environmental actions acting on the structure, the severity of the deterioration may vary from one area of the structure to another, depending upon a number of factors including the exposure (micro-climate effects), the form and the detailing of the structure. Accordingly it is also likely that the rate at which the deterioration progresses will also vary.

From the client/owners point of view it is of great importance not only to detect any deterioration processes at an early stage, when it remains in the initiation phase, but to determine the cause(s) of such deterioration. This would make it possible for the client/owner to take appropriate decisions and act at the right time.

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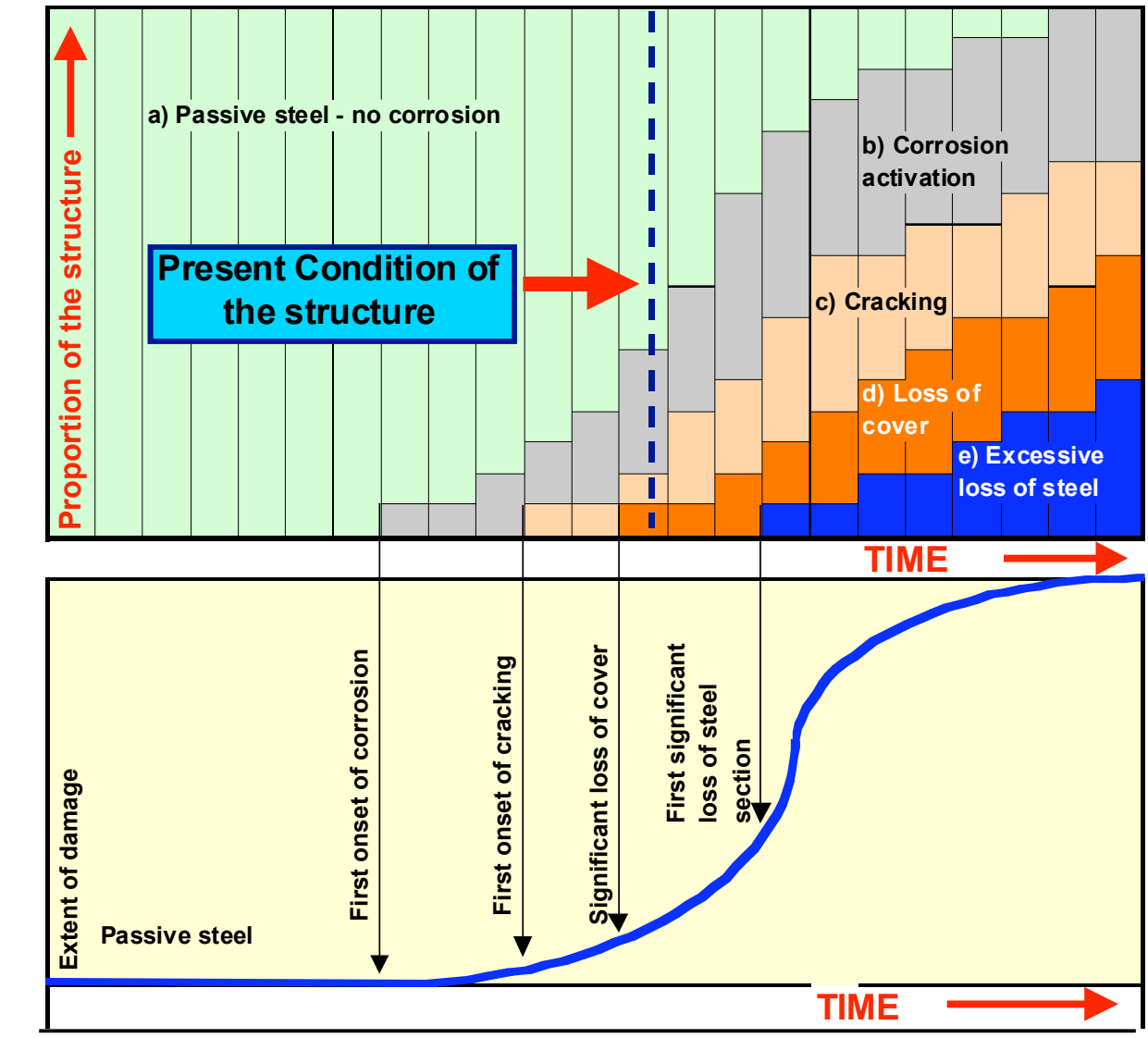

Key:

- Reinforcing steel in passive condition: No corrosion.
- Reinforcing steel in active condition: Corrosion occurring, but no loss of strength.
- Corrosion producing cracking of cover concrete: Some minor loss of strength.
- Corrosion producing cracking and spalling of cover concrete: Some loss of strength.
- Excessive loss to steel reinforcement: Strength of structure dangerously impaired.

Fig. 5.4-6: The changing condition with time of a concrete structure experiencing deterioration due to reinforcement corrosion (after EUROLIFEFORM project)

5.4.3 Role of water and moisture transport mechanisms

The availability of moisture is a critical factor controlling the various deterioration processes. Water may find its way into uncracked concrete by various means, principally by:

- *Diffusion* which controls the penetration into concrete of substances such as carbon dioxide and water vapour into atmospherically exposed concrete and chlorides into saturated concrete. These processes are driven by a pressure gradient mechanism, such as the water vapour pressure gradient.

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- *Capillary suction* or absorption of water or of solutions. These are driven by surface tension effects operating within the large and small pores in the concrete.
- *Permeation* whereby water, and any associated ions dissolved in it, penetrates into concrete under hydraulic pressure; with the degree of penetration being controlled by the water pressure gradient.
- *Combinations* of the above mechanisms.

An important indicator is the net moisture movement over a period of time, which could extend beyond a year for concrete elements with large cross-sectional dimensions. The longer-term variation in moisture content is of considerable significance as this determines whether the concrete is drying out.

Key influences upon water transport by the three transport mechanisms are:

- average relative humidity and temperature inside and at the surface of the structure over the day, the month and the year;
- resistance of the structure to moisture movements, e.g. permeability and diffusivity of the concrete is determined by the water-cement ratio, the presence of cracks, membranes, surface coatings, etc.;
- temperature variation over the day, the month and the year;
- the presence of hydraulic pressure, especially for a water retaining structure.

This information enables the designer to estimate parameters such as the positions of the dew point and the evaporation front, which influence the active deterioration mechanisms.

The transport of water and gases within concrete is determined by pore size and distribution, and by the presence of cracks. Thus these are important factors to control when designing for durability. Hence, amongst other requirements, there is the desire to use a low water-cement ratio and an insistence upon adequate curing of the concrete.

A durable concrete has a high proportion of small radius gel pores. These help maintain a high internal humidity in the concrete, even when the atmospheric moisture level is low. High atmospheric humidity causes the larger pores to fill with water, which reduces the pore space available for the diffusion of gases. Consequently, the gas permeability of the concrete decreases markedly with increasing water content and, in the case of an almost fully saturated concrete, the diffusion of gases (e.g. carbon dioxide and oxygen) becomes almost negligible.

Chloride penetration into concrete: Although this is a slow process, it can happen quickly relative to the planned service life of a concrete structure, resulting in a build up of chlorides within the concrete. The most common transport mechanisms are diffusion, capillary suction and permeation. Diffusion results from chloride gradients within the stationary pore fluid (i.e. not moving through the pore system of the concrete). For diffusion to occur the pore system has to be in a notionally saturated condition. In the case of capillary suction, the chlorides are transported with water which moves into the open (unsaturated) pore system. Permeation involves the penetration of chlorides into the concrete along with water which is driven into the concrete by hydrostatic pressure. All three transport mechanisms may act simultaneously.

Estimates of chloride ingress are typically made using diffusion theory. Calibration against chloride ingress profiles determined on site enables an apparent diffusion coefficient, D_a, to be calculated. This result therefore includes the combined effect of the contributions

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made by the diffusion, capillary suction and permeation mechanisms for the period from the start of exposure up to the time the inspection is made. The apparent diffusion coefficient is used to estimate future chloride penetration and, from this, the remaining service life of the concrete structure.

5.4.4 Physical deterioration and damage processes in concrete

5.4.4.1 Introduction

Physical deterioration and damage inducing processes causing cracking and other effects in concrete structures can arise from various causes including:

- plastic settlement and shrinkage;
- early age thermal cracking;
- drying shrinkage, including autogenous shrinkage;
- abrasion and erosion;
- freeze and thaw effects (frost attack and surface scaling; with or without de-icing salts);
- physical salt crystallisation and scaling;
- thermal effects, movement and cracking;
- weathering, discolouration and soiling of surfaces.

The associated deterioration and damage to the concrete caused by these processes may expose the embedded reinforcement to subsequent deterioration by corrosion or other means.

5.4.4.2 Plastic settlement and shrinkage

For the first few hours after placing concrete is in a 'plastic' condition and has little strength. In this period water is able to move relatively freely within what is still essentially a mixture of cement, aggregate and water; plus other minor but important constituents. Gravity effects encourage the more dense components to settle; whilst the less dense components, particularly air and water, move upwards towards to the top surface of the concrete pour.

Plastic settlement cracking: The downward movement may be restricted by reinforcement or formwork. This causes the still plastic concrete to arch over the locations where its downward movement is restricted. In the case where reinforcement situated in the top of the concrete pour is providing the restraint, cracks may develop above the reinforcing bars and small voids below them. Both features are orientated along the length of the reinforcing bars.

The cracks are known as plastic settlement cracks. Cracks formed when the concrete is in a plastic state are confined to the cement paste and pass around, not through, aggregate particles. Plastic settlement cracks which form in the top surface of a concrete pour are typically 1 mm wide at the surface and usually extend down to the reinforcement. These cracks potentially provide a pathway for the ingress of water and aggressive species, including carbon dioxide and chlorides; which can introduce a risk of reinforcement corrosion.

In situations where there is a dense mat of closely spaced top surface reinforcing bars, the whole concrete cover layer can become 'hung' upon on the reinforcement, while the main body of concrete below (the 'heartcrete' – Fig. 5.1-7) settles. The result is a horizontal discontinuity below the top surface reinforcement, which adversely affects the bond and anchorage

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of these bars and also the corrosion protection which is provided to this reinforcement by the concrete.

Plastic settlement cracks and their consequences can be eliminated (as opposed to being prevented) by the use of re-vibration of the concrete after they have formed (Concrete Society Technical Report 22 (1992)]. Care has to be taken. Re-vibration should not be applied too soon otherwise further plastic settlement could occur, the latest practical time is when a vibrating poker can be removed from the concrete without leaving a significant trace on the concrete surface. It is essential that the re-vibration extends below the upper layers of reinforcement. Re-vibration can improve concrete properties and strength.

Fig. 5.4-3 presents a number of examples of plastic settlement cracks, giving an idea of their characteristics. Plastic settlement cracks may also form on vertical surfaces, where reinforcement has restricted the movement of the concrete. Horizontal plastic settlement cracks may also be found in concrete columns, particularly below splayed column heads where these are cast integrally with the main body of the column. These cracks can be 1 mm wide and pass through the full concrete section of the column.

Plastic shrinkage cracking: The upward movement of water in the plastic concrete mix is termed 'bleeding'. Water on the surface of the concrete pour evaporates. The rate at which this occurs depends upon atmospheric and concrete conditions; increasing at higher temperatures and/or lower humidity, particularly in windy conditions. When water evaporates faster than it bleeds from the concrete, the surface zone experiences a net loss of moisture and an overall reduction in volume. The surface layer attempts to shrink, but is restrained by the main body of concrete which lies below it. This causes tensile strains to develop in the surface zone, potentially leading to the development of cracks. These are known as plastic shrinkage cracks. Plastic shrinkage will be greater where the cement content of the mix is high and the water-cement ratio is low. Plastic shrinkage cracks are most common in slabs, but they also occur in the top faces of walls, etc.

Fig. 5.4-3 presents a number of examples of plastic shrinkage cracks. Plastic shrinkage of concrete can also occur when water is lost by suction to an underlying dry concrete or soil.

Plastic shrinkage cracks tend to be 1-3 mm wide on the surface, but may be wider in some circumstances. They tend to be 300-500 mm long and 20-50 mm deep. However, in some circumstances they may extend through the full concrete section. The cracks expose the reinforcement to the risk of corrosion.

Plastic shrinkage can be avoided by limiting the rate of evaporation of water from the surface of the concrete. A maximum value of 1 kg/m^2/hr has been recommended (ACI 305R-91 (1994)]. The concrete should be protected from sun and wind, placed and finished quickly. Curing measures should be started as soon as possible after placing, particularly when the concrete temperature exceeds ambient.

Plastic shrinkage cracks should not be found in concrete slabs that have been correctly power floated as the action of power floating and trowelling is a form of re-compaction which tends to close plastic shrinkage cracks as soon as they form.

5.4.4.3 Cracking due to early age thermal and restrained movements

The hydration of cement is an exothermic reaction which produces heat, raising the temperature of the concrete mass. As the concrete heats up, it expands. Restraint to this expansion induces compressive stress in the concrete. Typically these stresses are low as young concrete

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has a low elastic modulus, and as such stresses are generally relieved by creep effects they are generally ignored in design procedures. Once the peak temperature has been reached, which may be some 12 to 18 hours after casting but can be considerably longer for massive concrete sections, the concrete will start to cool and to contract. If this contraction is resisted, tensile stresses are created in the concrete which may result in cracking. Restraint to this contraction may arise from internal or external sources.

Internal restraint arises from the temperature differential which occurs between the surface zones and the centre (the core) of the concrete section; and from the restraint created by the presence of reinforcement. During hydration the core of a concrete element will reach a higher temperature than the surface zones. For the same concrete mix, a more massive (thicker) concrete section will reach a higher peak internal temperature than a thinner concrete section. For design calculation purposes, the concrete is assumed to harden across the entire section at the point when the peak temperature is reached.

Initially the surface zones cool faster than the core of the concrete section, which puts the surface zones into tension and the core into compression. The resulting tension and compression forces will be in balance across the concrete section. Cracks may be formed in the surface zones if the temperature difference between the surface and the core becomes excessive. It has been recommended that this temperature differential is limited to a maximum of some 20°C [CIRIA Report 91 (1992), BS 8007 (1987)].

When the concrete element finally cools to ambient, the overall drop in temperature of the core (from the peak temperature) will be greater than that of the surface zones. This means that the core will contract more than the surface zones. This has the effect of inducing tension into the core of the concrete section and putting the surfaces zones into compression. This means that cracks which formed in the surface zones, as a result of internal restraint effects, may potentially close during the cooling phase.

If the core reaches an excessively high temperature during hydration, the tensile stresses subsequently induced may be sufficiently large to crack the core of the concrete element. For this and other reasons, it is recommended that the peak temperature reached be limited. Of course, whilst such cracking of the core would not be visible, it could potentially allow water or other liquids a pathway through the core of the concrete element. However, as tension in the core must be balanced by compression at the surface of the section, it is unlikely that the internal cracks will be coincident with surface cracks.

Various means may be employed when seeking to limit the peak temperature reached. These vary in their effect and cost-effectiveness. The options are essentially to:

- control or reduce the temperature of the constituent materials (mix ingredients);
- reduce the heat gain from the plant, equipment and the environment;
- reduce the temperature of the fresh concrete.

Cooling the mix ingredients is perhaps the most widely practised approach and is generally the most cost-effective method, although there are practical limits as to the temperature reduction which is possible. Reducing the temperature of the fresh concrete, either after production or after placing, is undoubtedly the most effective and controllable. However, it is expensive. The CIRIA *Guide to the construction of reinforced concrete in the Arabian Peninsula* [CIRIA Funders Report CP62 (1999)] provides a detailed review of the various methods available for temperature control.

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In such circumstances the addition of chilled mixing water or flaked ice directly to the concrete in the mixer can prove to be effective. The CIRIA Guide [CIRIA Funders Report CP62 (1999)] suggests that the temperature of the fresh concrete mix can in practice be reduced by 3 or 4 °C using chilled water instead of water at ambient temperature. The use of flaked ice should yield an additional reduction in the temperature in the fresh concrete. It is indicated that 5 or 6 °C is probably the most that can normally be achieved when substituting the maximum amount of ice for chilled water, which is about 70 to 90 kg of ice per cubic metre of concrete. Clearly no masses of ice should remain in the concrete after it is placed and compacted.

External restraint is created by the connection of the concrete element concerned to other structural elements (such as a rigid base, wall elements or an adjacent pour) or possibly from the ground mass, if it is a foundation or ground slab. In these circumstances, restraint may be introduced along the edge or edges of the concrete element concerned, possibly along the underside or perhaps via the ends of the element. In their discussion of restraint factors Alexander and Bamforth (2009) recognise the three forms of external restraint listed below, noting that the type of restraint is influential in determining the way in which cracking develops and the potential width of cracks. Alexander and Bamforth (2009) emphasis that understanding of the nature and the degree restraint is important when seeking to achieve either crack free concrete or to ensure that crack widths are controlled to be within acceptable limits.

Surface restraint – occurs where restraint is provided to the underside of a cast slab element by shear, such as when a layer of fresh concrete is cast onto an older substrate or a slab acts compositely with a steel beam.

Edge restraint – occurs where restraint is provided along the edge or edges of a cast element, such as when a wall is poured onto a footing or a slab is cast against a previous pour.

End restraint – arises in cast elements whose ends are attached to stiff points, such as a slab cast between two cores or a basement floor cast between footings or pile caps.

Alexander and Bamforth (2009) note that restraint factors have been used for some time to estimate the risk of cracking in isolated applications, but that they have not been used consistently to provide a coordinated approach to understanding the behaviour of concrete elements subject to restrained movements. The restraint factor is usually denoted by R and ranges in value from 0 for free movement to 1 for full restraint. R is defined by the expression;

$$R = (\varepsilon_{free} - \varepsilon_m) / \varepsilon_{free} \qquad (5.4\text{-}1)$$

where ε_{free} is the free contraction strain that would occur if no restraint was present and ε_m is the strain that actually occurs. Hence $\varepsilon_{free} - \varepsilon_m$ equals the restrained component of the strain which generates stress. Alexander and Bamforth (2009) offer guidance on the evaluation of restraint factors for the various internal and external restraint conditions described above.

The different forms of restraint create different stress conditions within the concrete element concerned. Fig. 5.4-7 illustrates the pattern of cracking which develops under a commonly occurring condition of continuous edge restraint, that of a wall cast on a stiff foundation.

Alexander and Bamforth (2009) also note that restraint within a wall on a stiff foundation is not uniform, but that this varies with the height above the stiff foundation. Methods for estimating the restraint at the joint with the stiff foundation and its variation with height are

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described in CIRIA C660 (2007), which also provides a spreadsheet calculator. Observations indicate that maximum values of both restrained-strains and crack widths occur at a height above the joint equal to about 10% of the cast length of the wall. Thus, it is suggested that the risk and extent of cracking should be assessed at this height (i.e. 10% of the cast length of the wall), together with the measurement of crack widths for compliance purposes.

Early age thermal cracking typically occurs within a few days in thin sections, but may take several weeks to develop in massive concrete sections. The width of cracks depends on whether the reinforcement in the concrete element concerned has been designed to control early age thermal cracking. The key consideration is whether the reinforcement yields at the crack locations. Satisfactorily designed reinforcement, such as might be provided for water retaining structures, will generally seek to limit crack widths to say 0.2 mm; which should permit autogenous healing to occur. However, wider crack widths can be designed for. If yielding of reinforcement occurs, the width of the cracks will not be controlled and is likely to be many times greater than the above value.

As this type of crack forms after hardening of the concrete but before full strength is achieved, the cracks are generally confined to the cement paste and pass around, not through, aggregate particles.

The likelihood of cracking depends on a number of factors including the temperature rise, the coefficient of thermal expansion of the concrete, the degree of restraint and the ability of the concrete to withstand tensile strains without cracking.

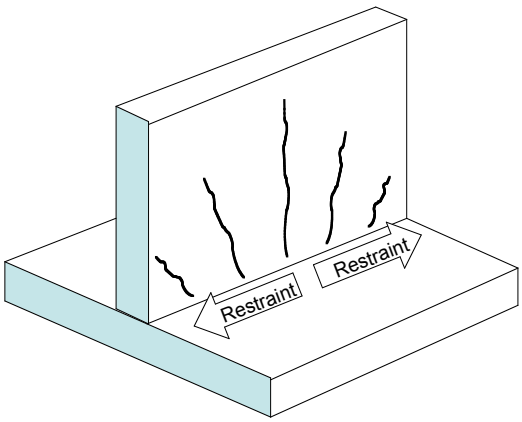

Fig. 5.4-7: Early-age thermal cracking due to continuous edge restraint [Bamforth (2008)]

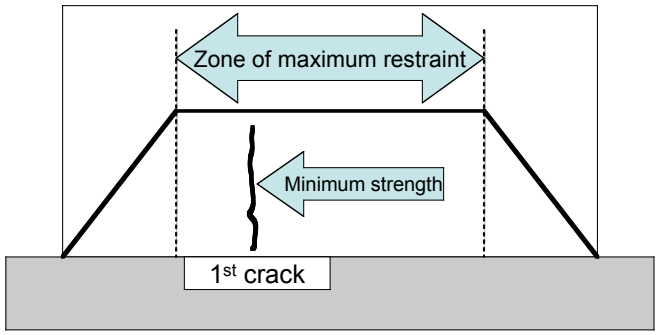

Fig. 5.4-8: Early-age thermal cracking – Location of the first crack on wall elevation [Bamforth (2008)]

There are various options available to reduce the likelihood of cracking although any mix considered must comply with requirements for durability, workability and strength. These options include minimising restraint to thermal movement, reducing the coefficient of thermal expansion of the concrete (through selection of appropriate aggregate), increasing the tensile strain capacity of the concrete and reducing the peak temperature. The temperature rise can be reduced by controlling the cement content, type and composition, using appropriate formwork and insulation, and by limiting the size and geometry of the pour and the initial mix temperature.

For example, replacing some of the cement by ggbs (ground granulated blast-furnace slag) or pfa (pulverised fuel ash or fly ash) is an effective way of reducing the rate of heat generation. This is most effective in thinner concrete sections and when the mix temperature is kept

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as low as practically possible. The use of silica fume to reduce the cement/binder content may also be an effective means of reducing temperature rise.

In terms of the potential rise in temperature, this could be as high as 13°C per 100 kg/m^3 of cement for Portland cement under adiabatic conditions (i.e. in concrete thick sections where little heat is lost from the core of concrete element during the hydration process). For typical structural concretes (35-50 MPa) with cement contents in the order of 330-400 kg/m^3, the notional temperature rise could be over 50°C. However, in practice the actual temperature rise is less. Methods of estimating the temperature drop are given in a number of sources, such as CIRIA Report C660 [Bamforth (2007)].

The free contraction is estimated by multiplying the temperature drop by the coefficient of thermal expansion of concrete. Whilst this is usually taken to be $12 \cdot 10^{-6}$ per °C for normal weight concrete, this value can vary by perhaps 20% depending primarily on the aggregate type. Typical values are given in CIRIA Report C660 [Bamforth (2007)]. The free contraction value is adjusted, according to the level of restraint and relaxation due to creep, to give the restrained strain. This is then compared with the tensile strain capacity of the concrete to determine the likelihood of cracking. Where cracking is predicted to occur, the reinforcement provision may be checked using methods given in CIRIA Report C660 [Bamforth (2007)] to determine whether the crack widths will be acceptable.

Guidance on the control of early age thermal crack in concrete continues to evolve with the introduction of EN 1992 [CEN (2004, 2006)]. However, Bamforth and Denton (2009) suggest that EN 1992 does not fully address all aspects which need to be considered in design for early-age thermal cracking. They note that CIRIA C660 [Bamforth (2007)] has been developed to complement EN 1992, incorporating recommendations that enable the EN 1992 design process to ensure that estimated crack widths reflect more reliably those observed in practice so that a robust design is achieved. Bamforth and Denton (2009) suggest that in the absence of these recommendations, EN 1992 could be interpreted to lead to a significant increase in minimum reinforcement required in sections thicker than about 800 mm. They note, conversely, it could also lead to significantly less reinforcement than that required by BS 8007 (1987) for controlling crack spacing and width. In some cases they suggest the reduction may be greater than 50% and that this could lead to inadequate control of early age thermal cracking.

In this it is therefore important to recognise that the form of external restraint may have a significant effect on crack widths; that is whether the restraint is applied along the edge/edges or the ends of the concrete element. Under conditions of <u>end</u> restraint the crack width is determined by the tensile strength of the concrete and the stress that is transferred to the steel when a crack occurs. In this case, for the same magnitude of contraction, the crack widths may be more than double those occurring under <u>edge</u> restraint conditions [CIRIA and Concrete Society (2008)]. Different expressions apply in the calculation of crack width in these circumstances – see EN 1992-3 [CEN (2006)], CIRIA Report C660 [Bamforth (2007)] and Bamforth and Denton (2009).

Furthermore recent work has been undertaken by Bamforth (2008) and Bamforth et al. (2009) on early-age crack spacing and width in relation to the design of water retaining structures in response to concerns that some assumptions adopted in current guidance, as embodied in EN 1992-3 [CEN (2006)], may be incorrect. The issue concerns early-age thermal cracking in elements subject to continuous edge restraint, e.g. walls on a stiff foundation.

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Bamforth (2008) notes that in these circumstances it is rare to observe a fully developed crack pattern as envisaged by the current theory, such as that portrayed in Fig. 5.4-7. Even when cracks are regularly spaced, the crack spacing is often wider than estimated. In addition it is observed that there are often uncracked zones which may extend over several times the theoretically predicted crack spacing. This is taken to indicate that cracks may not be occurring independently, with the initial cracks relieving the stress and strain over a distance greater than the theoretically predicted crack spacing. Bamforth (2008) postulates that if this is the case, it might be reasonable to expect that individual cracks, which relieve the restrained contraction, would be wider than estimated.

EN 1992-3 [CEN (2006)] assumes that high restraint will provide a worst case with regard to crack width. However, Bamforth (2008) notes that while this may be true in relation to the risk of cracking and the integrated crack width; it may not lead to the worst case with regard to the width of the individual cracks that are of concern with regard to aesthetics and durability.

Along with other investigators such as Beeby and Forth (2005), Bamforth (2008) suggests that the development of cracks occurs progressively, rather than simultaneously, as follows:

1. The restrained strain increases until the tensile stress in the concrete reaches the tensile strength at the weakest point and the first crack forms. This will be at a location where the restraint is highest as shown in Fig. 5.4-8.
2. This crack results in a reduction in the stiffness of the element and a relaxation of stress.
3. With further restrained contraction, the first crack widens and the stress in the concrete increases until a second crack is formed at the next weakest point along the concrete element.
4. This process continues progressively creating further cracks until contraction stops.

Bamforth (2008) and Bamforth et al. (2009) adopt the cracking model for end restraint as a starting point to develop an alternative mechanism for the edge restraint condition. The approach acknowledges that various simplifications are made in the alternative approach. Fig. 5.4-9 shows the relationship between restraint and crack width estimated using the approaches contained in BS 8007 (BSI (1987)], EN 1992-3 [CEN (2006)] and the alternative approach proposed by Bamforth (2008) and Bamforth et al. (2009).

Bamforth et al. (2009) note that BS 8007 [BSI (1987)] and EN 1992-3 [CEN (2006)] assume a default restraint (worst case) value of 0.5 which, when taking into account the effects of creep, represents a true edge restraint (R_{edge}) value 0.77. However, the range of values of R_{edge} estimated for the case studies was 0.49 – 0.60. Bamforth et al. (2009) note that in the specific example presented as Fig. 5.4-9, at this level of restraint the crack widths estimated using the revised approach are very similar to those achieved by assuming the higher (worst case) level of R_{edge} = 0.77 in the methods of BS 8007 and EN 1992-3 (in each case about 0.2 mm).

Bamforth et al. (2009) suggest that it is this over-estimation of restraint that may be why current design methods have, in many situations, provided satisfactory designs. They also observe that in situations in which the axial restraint $R_{ax} < 0.5$ ($R_{edge} < 0.77$), this may also explain why, hitherto, crack widths have often been greater than estimated with no obvious explanation.

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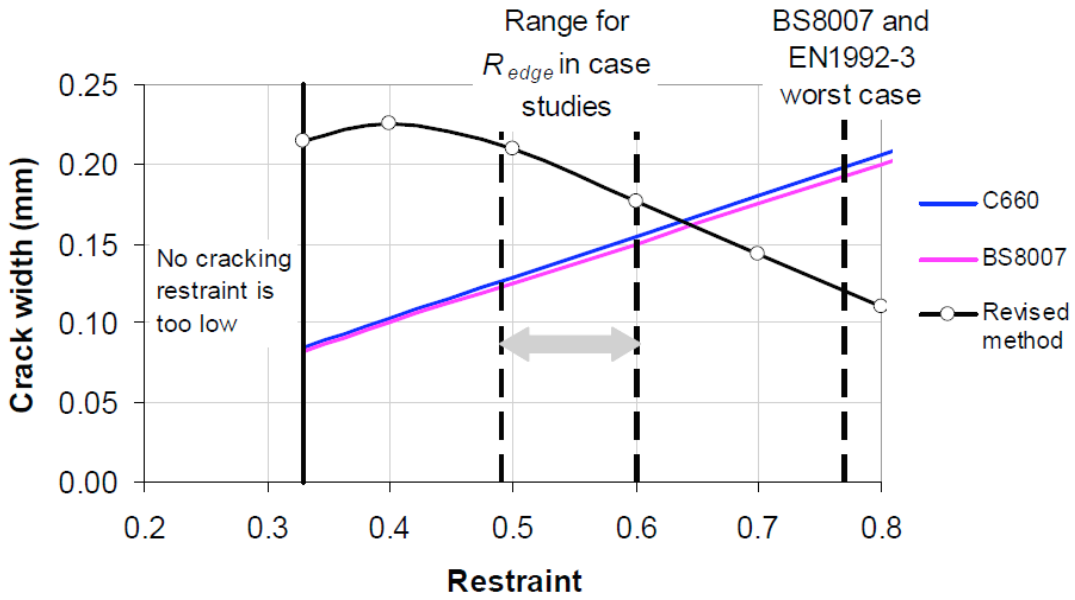

Fig. 5.4-9: Early-age thermal cracking – Comparison of estimated cracks widths using BS 8007, EN 1992-3 and the Bamforth revised method (400 mm wall, C30/37, Portland cement, 20 mm bars at 250 mm) [Bamforth et al. (2009)]

Thus as EN 1992-3 [CEN (2006)] assumes that cracks are influenced only by local stresses and are independent of the formation of other cracks, the estimated crack width increases with restraint. However, using the alternative approach [Bamforth (2008)] and [Bamforth et al. (2009)], in which the influence of restraint on the crack width and distribution is taken into account, the opposite applies; with higher restraint resulting in an increased number of smaller cracks.

Bamforth (2008) notes that assuming that the revised approach more accurately reflects the way in which cracking occurs in concrete subjected to continuous edge restraint, the greatest deviations compared with EN 1992-3 [CEN (2006)] and BS 8007 [BSI (1987)] would be expected to occur for:

- higher strength concrete;
- elements that cool slowly, where a higher strength develops before cracking occurs;
- elements subject to a lower level of restraint, but still sufficient to cause cracking.

Bamforth (2008) and Bamforth et al. (2009) point out that if the revised method is proven to be valid there are significant implications. The following observations are made:

- That the current advice is that reduced restraint will lead to lower crack widths and, if restraint is reduced to a low enough level, then cracking may be avoided. However, the revised model indicates that if cracking is not avoided, then crack widths may be greater at lower levels of restraint, although fewer cracks would be expected.
- That EN 1992-3 [CEN (2006)] assumes high restraint represents a worst design case, but that this may not be a safe assumption.
- It is currently assumed that a slower rate of cooling, which enables the concrete to achieve a higher strength and strain capacity, will be beneficial. However, the revised

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model indicates that, if cracking is avoided, this may be the case. But, if cracking occurs, the crack widths could be greater than currently guidance would suggest.

Bamforth (2008) and Bamforth et al. (2009) recognise that, amongst other things, the revised approach requires further work to validate the method as it takes no account of increasing concrete strength with age and that the influence of other factors, in addition to edge restraint, may need to be considered.

Clearly if crack widths could be significantly wider in certain circumstances than estimated in the current design procedures, this potentially could have implications for the durability of some types of reinforced concrete structures.

5.4.4.4 Drying shrinkage, including autogenous shrinkage

Water added to the concrete mix which is in excess of that required to react with the cementitious content during hydration is held within capillary pores which form within the cement paste. If the hardened concrete exists in an environment where the relative humidity is lower than that within the concrete, moisture will migrate from the concrete to the external environment and drying will take place. The rate at which this occurs depends upon many factors including the nature of the external environment, the type of concrete and its permeability, aggregate type and quantity, as well as the shape and size of the concrete element concerned. The curing regime applied to the concrete can also have an influence the rate of drying shrinkage at an early age, but will have limited influence upon the ultimate drying shrinkage strain. Curing has a significant beneficial effect upon the tensile strain capacity of the concrete and it is primarily for this reason that curing reduces the risk of cracking.

Loss of moisture results in a reduction in volume and the concrete seeks to shrink. The resulting changes in dimensions may be restrained by internal and external factors, in a way somewhat similar to those described previously for early age thermal cracking. Thus as the concrete attempts to shrink, restraint can cause the development of tensile strains and stresses on the surface and through the body of the concrete element concerned. When these restrained stresses exceed the capacity of the concrete to resist them, cracking occurs.

There is no typical pattern of drying shrinkage cracking as cracks can form at any location where there is restraint to the shrinkage induced movement. However, shrinkage cracks are usually orientated approximately perpendicularly to the line between the points/locations of restraint. The width of the shrinkage cracks will depend upon the extent that the concrete has dried out and the length of the concrete element concerned or the distance between points/locations of restraint. Because the cracks form after the concrete has gained full strength, the cracks can pass through weak aggregate particles.

Concrete Society Technical Report 22 [Concrete Society (1992)] provides guidance upon the potential magnitude of these movements. This indicates that for typical UK outdoor exposure conditions the six month drying shrinkage strains for a structural concrete element with a mix water content of 190 litres/m^3 are likely to be in the order of:

- 50με (50 microstrain = a strain of 0.050 mm/m) in a concrete element 150 mm thick
- 25με (25 microstrain = a strain of 0.025 mm/m) in a concrete element 300 mm thick
- 10με (10 microstrain = a strain of 0.010 mm/m) in a concrete element 600 mm thick

These values illustrate the influence of section size upon drying shrinkage development, with shrinkage strains values at an age six months being greater in thinner elements. In thicker

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elements shrinkage strains will be higher in the surface zones than the average value for the overall element.

The long-term (30 year) drying shrinkage values were also given for these conditions. These were expected to be in the order of 80 to 100 με, with the largest value occurring in the 150 mm thick concrete element.

Eurocode 2 [EN 1992-1-1: CEN (2004)].gives values for the nominal unrestrained drying shrinkage (given in με, microstrain) as a function of the relative humidity. This information is reproduced in Table 5.4-2.

Natural aggregates are not, in general, subject to significant shrinkage on drying. However, some dolerites and basalts shrink on drying and can produce concretes with a high initial drying shrinkage. This can lead to cracking and consequently to durability problems. Guidance and recommendations on the use in the of high drying shrinkage aggregates in UK concrete structures are given in BRE Digest 357 [BRE (1991)].

Table 5.4-2: Nominal unrestrained drying shrinkage values (microstrain, με) for different relative humidities, from EN 1992-1-1 [CEN (2004)]

Concrete strength class	**Relative humidity (%)**					
	20	**40**	**60**	**80**	**90**	**100**
20/25	620	580	490	300	170	0
40/50	480	460	380	240	130	0
60/75	380	360	300	190	100	0
80/95	300	280	240	150	80	0
90/105	270	250	210	130	70	0

Note: *1 με is a microstrain or a strain of $1 \cdot 10^{-6}$ = a strain of 0.001 mm/m.*

Autogenous shrinkage (sometimes called chemical shrinkage or self-desiccation) is an external volume change that occurs in concrete during hydration without the transfer of moisture between the concrete and the external environment. It results from the chemical reactions and internal structural changes which are associated with the cement hydration process. As a result it occurs during the first few days after casting.

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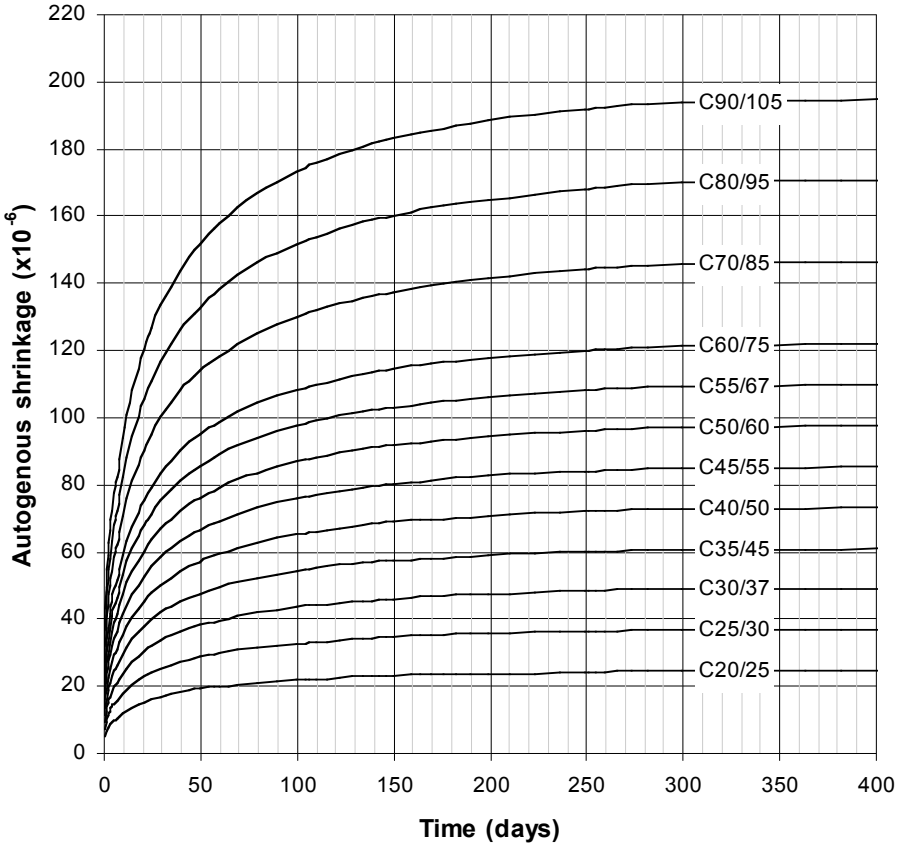

Fig. 5.4-10: Autogenous shrinkage for different strength classes estimated using the method given in EN 1992-2-1 [CIRIA and Concrete Society (2008)]. Reproduced with permission

It is now recognised that autogenous shrinkage occurs in all structural concretes to varying degrees. Autogenous shrinkage is separate from and additional to conventional drying shrinkage, which starts when water curing ceases. There is also some evidence that mineral additions to the concrete mix may have a significant influence on the magnitude of autogenous shrinkage – see Appendix 4 of CIRIA C660 [CIRIA (2007)].

Autogenous shrinkage is usually of concern in high strength or high performance concrete with a low water – cement ratio; that is below about 0.4 or 0.45. It can lead to cracking which may negate the intended benefits of using high performance concrete. It is highly dependent upon cement type. EN 1992-2 [CEN (2005)) provides an expression for calculating the magnitude of autogenous shrinkage and its development with time. Fig. 5.4-10 presents estimates of the magnitude of autogenous shrinkage for different strength classes of concrete. The values portrayed were calculated using the method given in EN 1992-2-1 [CEN (2005)].

Long-term drying shrinkage, including the autogenous shrinkage component, can produce significant shrinkages strains, potentially in the order of 400 to 500 με (400 to 500 microstrain = a strain of 0.4 to 0.5 mm/m). Should uncontrolled cracking occur, it could produce significant crack widths.

5.4.4.5 Abrasion and erosion

Abrasion of concrete surfaces can occur as a result of sliding, scraping or percussive impact. Erosion of concrete in contact with flowing water can also be a significant cause of damage. Erosion through water-borne solids leads to a rounded worn surface in which the

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larger aggregate particles, being harder than the cement matrix, stand proud of the general surface. Where cavitation (the presence of vapour bubbles) occurs, this can rapidly damage the surface of the concrete which can be eroded to into an irregular pitted surface.

Erosion of the concrete surface can reduce the depth of cover to the reinforcement. This can increase the likelihood of corrosion due to the effects of carbonation or ingressed chlorides, which clearly has durability implications. Conversely, in a wet environment where chlorides are not present and the concrete cover zone remains saturated, it is unlikely that the embedded reinforcement will corrode. In this case, the erosion of the surface may not increase the likelihood of reinforcement corrosion until the cover concrete is largely eroded away.

The properties of surface concrete depend, amongst other things, on the finishing operations and curing. These can affect the water-cement ratio and improve compaction at the surface. The use of controlled permeability formwork can help to reduce abrasion. Good curing is very important. EN 206-1 [CEN (2000)] recommends curing for a period of twice as long as normal to achieve good abrasion resistance. It is also suggested that cement content should be limited to about 350 kg/m^3, which is somewhat at odds with the concrete mix recommendations made for achieving durability in other circumstances. Crushed sand and the use of strong hard aggregate are also recognised to improve abrasion resistance. Shrinkage compensating concrete also has improved abrasion resistance. Concrete with large aggregate erodes less than mortar of equal strength. Cavitation resistance can be improved through the use of high strength concrete with a maximum aggregate size near the surface of 20 mm. The use of fibres and coatings may also improve cavitation resistance.

5.4.4.6 Frost attack and surface scaling involving de-icing agents

The transition of water to ice involves an increase in volume of about 9%. When the pores in the concrete are close to saturation, freezing can cause severe cracking and disruption of the concrete. The damage mechanisms involved appear to be somewhat more complicated than the classical explanation, which simply associates concrete damage with tensile stresses induced in the concrete by the increase in volume which occurs when water freezes. This hypothesis postulates that when water freezes in almost saturated capillary pores, the expansive forces cause damage to the concrete when these forces are greater than the strength of the concrete. There have been a number of different and conflicting theories about the damage mechanisms involved. This topic is considered in more detail at the end of this section.

As with other mechanisms which cause cracking and damage to the concrete, the damage increases the ease with which water containing chlorides or other aggressive species penetrates into the concrete and to the reinforcement; increasing the risk of corrosion and other damage.

Two types of frost damage are recognised; internal frost damage and surface scaling.

Internal frost damage is caused by water freezing inside the body of the concrete. The damage is always confined to the parts of the concrete where the water content exceeds a critical value. This value is specific for each concrete mix; it can also be different in different parts of the same structure notionally using the same concrete mix. The damage is manifest by a loss in compressive and tensile strength, a reduction in the elastic modulus and a reduction in bond strength. Internal damage will affect the moment and shear capacity of slabs and beams, plus the compression capacity of columns by lowering the compressive and bond strengths. It can seriously affect the structural capacity of pre-stressed concrete elements by significantly lowering the elastic modulus of the concrete. Internal frost damage also changes

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the moment and force distribution in the structure by changing the stiffness distribution in affected parts of the structure.

Surface scaling is caused by freezing of the concrete surface when it is in contact with weak saline solutions. The damage consists of the removal of small chips or flakes of material. No scaling occurs unless free water was present on the surface of the concrete prior to freezing occurring. The severity of damage increases as the minimum temperature experienced decreases. Numerous partly conflicting theories have been proposed to explain this damage mechanism. These show different degrees of agreement with the observed characteristics of salt scaling. Typically the initial scaling occurs in the cement matrix; with the aggregate particles remain intact and unaffected. The damage is progressive. Gradually the overall depth of material lost due to scaling becomes deeper, such that coarse aggregate particles become undermined and are also lost.

In cold climates, salts are regularly used to de-ice roadways, walkways and types of pavement. Consequently, salt scaling is one of the major durability issues facing cementitious materials in such a climate. While salt scaling alone will not render a structure useless, it results in accelerated ingress of aggressive species, such as chlorides and the propensity for a high degree of saturation. The former renders the concrete element susceptible to corrosion of the reinforcing steel, while the latter results in strength loss due to internal frost action. Both of these effects diminish the service life of concrete structures.

Where concrete is badly affected by surface scaling a large portion of the cover concrete can be eroded. This adversely affects the anchorage and capacity of the reinforcing bars, along with other aspects of structural performance. This problem can also affect the effective capacity of compression and shear reinforcement.

Thus water may freeze in the capillary pores, voids and defects but also in porous aggregate particles either at the concrete surface (leading to scaling/pop-outs) or within the concrete (leading to deterioration of the physical properties of the bulk concrete). The following environmental aspects are critical factors to the development of freeze-thaw damage:

- severity of freezing (minimum temperature);
- rate of fall of air temperature (speed of freezing);
- change of air temperature during periods of freezing;
- number of freeze-thaw cycles;
- the presence of de-icing salts.

Frost damage produces a map pattern of cracking, where the cracks intersect at random angles. The pattern is similar to that produced by alkali-silica reaction. As the freeze-thaw attack progresses, the surface of the concrete becomes rough and material is dislodged.

The application of de-icing agents to a concrete surface covered with ice will cause a substantial drop in temperature at the concrete surface (thermal shock) during thawing of the ice. The difference in temperature between the surface area and the interior of the concrete gives rise to a state of internal stresses which it is postulated may induce cracking near the surface of the concrete (however see discussion below). Frost attack is considered to be more severe when de-icing agents are used. Consequently, to ensure adequate frost resistance under these circumstances, a higher content of entrained air is typically provided.

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Aggregates which are not frost-resistant will commonly absorb water and suffer from expansion effects during freezing. A typical indication of such processes is localised spalling above larger-sized aggregate particles (i.e. causing pop-outs).

Summarising; susceptibility to freeze-thaw damage is affected by the composition of the concrete, its permeability/porosity, moisture content and age, as well as by the exposure environment.

Observations on theoretical explanations of internal frost attack and surface scaling: Scherer and Valenza (2005 & 2006) have suggested that it is a common misconception that freeze-thaw damage is entirely due to hydraulic pressure resulting from the 9% increase in volume as water transforms into ice (Fig. 5.4-11), and that adequate air entrainment is all that is required for complete protection against frost attack on concrete.

Powers and Helmuth (1953) showed that concrete without air entrainment expanded as it was cooled. This behaviour is portrayed by the dashed line in Fig. 5.4-12. The dilation stopped almost immediately during an isothermal hold and began again when cooling was resumed. As there was no relaxation during the hold, it is suggested that the observed expansion effect cannot be due to hydraulic pressure.

In contrast, the concrete sample with adequate air entrainment contracted as it was cooled. This behaviour is portrayed by the solid line in Fig. 5.4-12. When cooling was stopped during an isothermal hold, the rate of contraction diminished slowly – as is shown by the gradual change in gradient of the solid line (curve). This has been interpreted as a relaxation of strain (and hydraulic pressure) as the pore water flowed to the air voids and froze there.

It was originally thought that the small entrained air voids (deliberately introduced into concrete to provide frost resistance) worked by providing sinks for the water displaced by the volume change associated with the transformation of water into ice (Fig. 5.4-13), but it was subsequently realized that the voids actually serve as nucleation sites where ice crystals can grow without constraint (Fig. 5.4-14). During the freezing process ice crystals in the air voids suck liquid from the small pores of the paste, this suction creates negative pressure in the pore liquid which puts the entire solid matrix into compression; thereby inducing compressive stress into the concrete.

Helmuth (1962) concluded that internal frost damage was entirely attributable to crystallization pressure associated with the growth of the ice crystals. In contrast, Fagerlund (1975) considered that internal frost damage results primarily from hydraulic pressure, exacerbated by reductions in permeability as the pores fill with ice.

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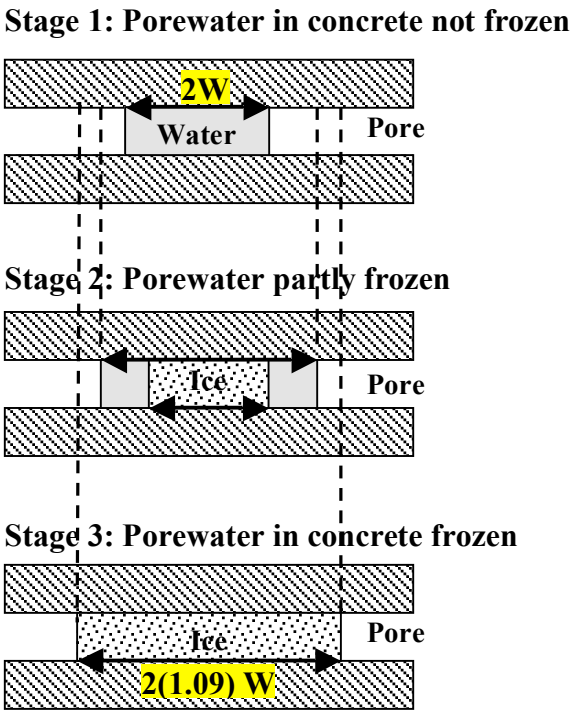

Fig. 5.4-11: Original concept for the development of frost damage in concrete. [Scherer, Sun, Abellan & Valenza (2008)]

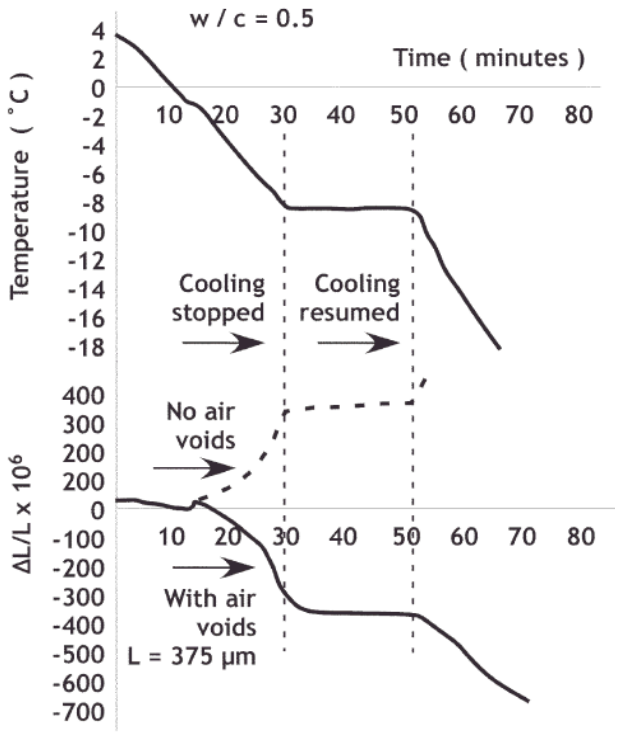

Fig. 5.4-12: Change in dimensions of concrete during cooling and freezing. [Powers and Helmuth (1953)]

***Note to Fig. 5.4-11**: As ice forms in a pore, it pushes unfrozen water away. The frictional resistance to flow along the pore creates hydraulic pressure on the pore walls, creating hydraulic pressure inside the concrete. This was originally thought to be main cause of frost damage to concrete. There is a 9% increase in volume of the pore liquid after it all transforms into ice.*

***Note to Fig. 5.4-12:** The dashed line shows that concrete without air entrainment dilates during cooling below freezing – the expansion damages the concrete. The solid line shows contraction of concrete with air entrainment (air void spacing L = 375 μm) during cooling below freezing – implying that compressive stress is present in the concrete skeleton. Both specimens have 45% porosity.*

Scherer and Valenza (2005 & 2006) suggest that the beneficial effect of air entrainment on both internal frost action and salt scaling led many researchers to conclude that the same mechanisms were responsible for both types of damage. They point out that these two types of damage do not necessarily occur together. Moreover, crystallization pressure does not account for all the characteristics of salt scaling, in particular the issue of the pessimum saline concentration required for salt scaling to occur.

Scherer and Valenza (2005 & 2006) observe that previous explanations of the cause of salt scaling have invoke pore chemistry or attribute the damage to the same causes as internal frost action. However, neither of these explanations adequately accounts for all of the observed characteristics of salt scaling. Instead they propose the glue-spall mechanism and suggest that it is consistent with all of the observed characteristics of salt scaling.

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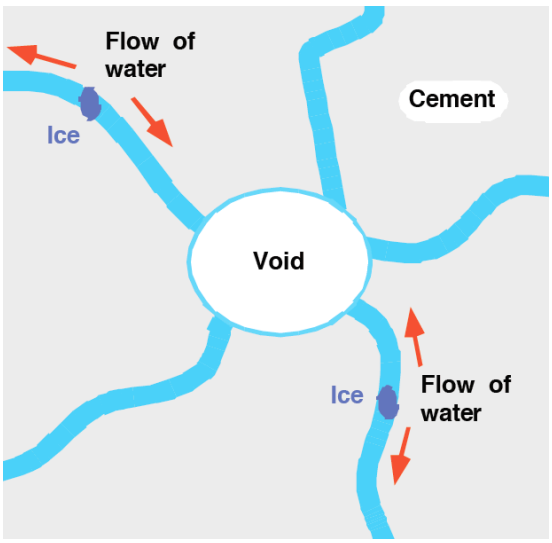

Fig. 5.4-13: Original concept: Air voids prevent excessive pore pressure by providing sinks for displaced water [Scherer, Sun, Abellan and Valenza (2008)]

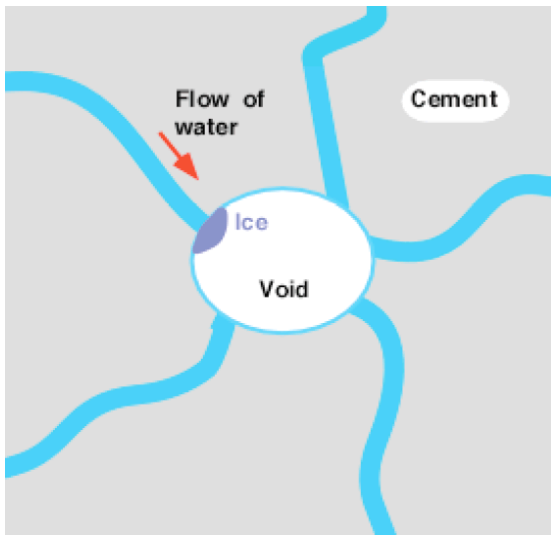

Fig. 5.4-14: Revised concept: Air voids provide site for nucleation and growth of ice crystals drawing in liquid from pores [Scherer, Sun, Abellan and Valenza (2008)]

Scherer and Valenza (2005 & 2006) show that salt scaling is a consequence of the fracture behaviour of the thin layer of ice that forms on the surface of the concrete. The stress arises from a mismatch between the thermal expansion characteristics of ice and concrete, which puts the ice in tension as the temperature drops. They consider the mechanical and viscoelastic properties of ice and they show that this mismatch of properties will not cause pure ice to crack. However, they do show that (brine) ice formed by the freezing of a moderately concentrated solution on the concrete surface, is expected to crack. The cracks in the brine ice penetrate into the concrete substrate, resulting in the type of superficial surface damage associated with salt scaling. At high saline concentrations, the brine ice is not rigid enough structure to cause significant stress in the surface of the concrete when it cracks, so no damage occurs to the surface of the concrete. This is consistent with the observation that a pessimum saline concentration is required for salt scaling to occur.

Summarising [Scherer, Sun, Abellan and Valenza (2008)]:

Internal frost damage: Air entrainment reverses the volume change observed when saturated concrete is frozen. Ordinary concrete expands, owing to crystallization pressure in the pores of the cement paste created as ice crystals are formed. However, air-entrained concrete contracts, owing to the suction created in the pore liquid by ice crystals forming in the air voids. Internal frost damage results from the crystallization pressure created when ice forms in small pores in the concrete. The tensile strength of concrete without entrained air can be exceeded by cooling to just a few degrees below 0°C. Internal frost damage can be avoided by the use of air entrainment.

Surface scaling: This type of damage to the surface of the concrete results from cracking of a thin surface ice layer formed when a weak saline solution freezes. Surface scaling can be prevented by strengthening the concrete in the surface zone.

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5.4.4.7 Physical salt crystallisation and scaling

In concrete the expansive forces due to salt crystallisation near the surface will usually only create minor problems, provided the concrete is not weak and porous. Of more importance is the chemical effect of the increased concentration of aggressive substances near the surface. Fig. 5.4-15 shows what is taken to be a possible combination of frost and de-icing salt induced damage at the base of a concrete plinth carrying a pipeline (in North American maritime environment). The salt has also promoted corrosion of the embedded reinforcement.

Fig. 5.4-15: Frost and de-icing salt induced damage at the base of a concrete plinth supporting a pipeline.

Damage is caused by salts, usually salts of sulfates and chlorides, crystallising in the pores just below the surface of the concrete. This exerts expansive pressure which causes cracking. Typically salt scaling causes flakes of the concrete matrix and possibly also the fine aggregate to become detached from the surface of the affected concrete element. This form of attack is usually limited to a few tens of centimetres above ground level, depending upon the quality and permeability of the concrete. When aggregate particles are more susceptible to this form of attack than the concrete matrix, expansion of aggregate particles may cause localised failures termed ‘pop-outs’. This is only likely to occur when the aggregate particles comprise porous and friable materials such as some sandstones and weathered granites. This form of attack is common in hot dry areas, such as the Gulf States, and may also occur in marine structures.

Where there is the possibility of groundwater containing salts rising up from the foundations into structural elements above ground, protection measures may be required for such elements. Fig. 5.4.16 illustrates the application of tanking of foundations in these circumstances to reduce the risk of reinforcement corrosion just above ground level. In such circumstances, tanking of foundations may also be required to reduce the risk of sulfate attack on foundation concrete.

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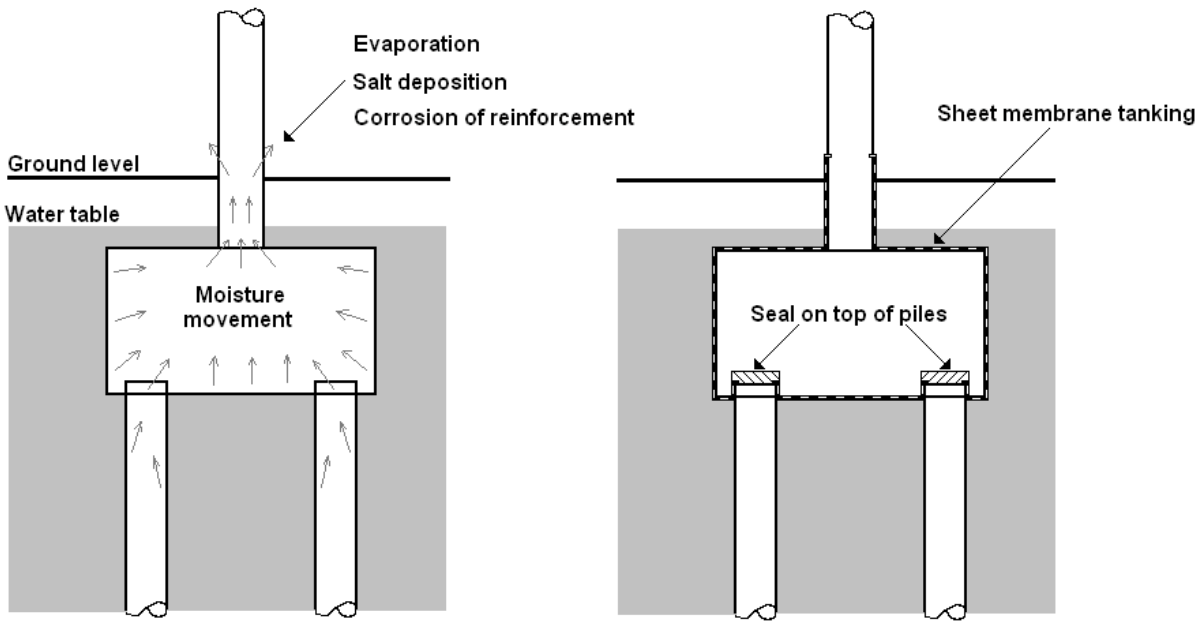

Fig. 5.4.16: Tanking of foundations to reduce the risk of reinforcement corrosion just above ground level [CIRIA and Concrete Society (2008)]. Reproduced with permission.

5.4.4.8 Movement, restraint and associated cracking of concrete

Movements occur in concrete structures for a variety of reasons which may include early-age thermal and moisture related contractions, temperature and humidity effects, creep and long-term drying shrinkage. In addition, there may also be load related effects causing movements and strains in the concrete. Although these movements will rarely determine the ultimate capacity of the structure, they often affect its serviceability and may well impact upon its durability – especially in circumstances where cracking may be an issue.

To establish the risks it is necessary to understand the nature of the movements which may take place; as well as their potential magnitude and the timescale over which the take place. Movements from different sources may combine cumulatively, so that any cracking is often a combination of effects associated with shrinkage, temperature change and early-age behaviours. In addition, there may be contributions from other sources.

Movement in a structure may be of limited consequence if it is not restrained. Restraint may occur in a number of ways. It can arise from within the concrete element concerned (i.e. internally) or from other parts of the structure to which it is connected (i.e. externally). In practice all restraint is partial, that is the restraint 'gives' to some degree in response to the movement of the concrete element concerned. This may occur for various reasons, perhaps due to the deflection of an external restraint or perhaps by beneficial creep effects which alleviate some of the induced stresses. For example, at an early age creep effects can halve the induced stress over a period of a few weeks. So while the forces which might be induced are potentially very large, they are strain-induced. So, if the restraint is reduced or removed, the forces involved will diminish or even disappear.

The magnitude of the movements and the ways that they affect a structure will depend on the materials, environment and construction methods used. It is therefore important to estimate movements at the design stage. It is generally good practice to seek to limit the likely movements to acceptable levels. Thus, one of the objects of design is usually to accommodate movements in the most appropriate way.

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The movements that cannot be accommodated in the design will therefore become restrained movements, and appropriate reinforcement will need to be provided for the forces involved. The consequence of restrained movements is generally cracking, which occurs in concrete when the tensile stress exceeds the tensile strength; or conversely when the induced tensile strain exceeds the tensile strain capacity of the concrete. The accepted design philosophy is usually to produce numerous fine closely spaced cracks, rather than a few wide cracks. These issues are discussed in Section 5.4.4.3 in respect of the provision of reinforcement for early age thermal cracking.

A simple illustration of the effects of movement can be given for a car park structure. In this type of structure storey heights are small and column spacing large, resulting in short stocky columns and long-spanning beams. If the beams are cast in-situ in one pour, when they contract large forces will be generated by the restraint provided by the short columns. This might create tension cracks in the beams or possibly produce cracks in the columns.

To avoid these problems, one approach could be for the beam to be cast leaving out a short section (a pour strip), thus eliminating any restraint to longitudinal movement. The gap could then be concreted after the initial shortening of the beam had occurred. In practice such an approach would have knock-on effects on the construction process and it does not relieve long-term movements. Alternatively the beams and columns could be designed to accommodate the effects of the restrained movements.

Thus in most instances the design philosophy will be to assume that cracking will occur and to control crack widths by providing sufficient correctly detailed reinforcement in critical regions of the structure. Amongst other matters, consideration will need to be given to the following:

Types of movement

A number of types of movement that may affect the structure can be identified, including:

- *deflections and rotations*: movement within the span of a beam or slab or at a continuous support may induce cracking;
- *settlement and heave*: downward or upward movement of a foundation or of a whole building or structure may induce cracking;
- *axial shortening*: vertical downward movement of columns or walls may cause differential movement and rotations which may induce cracking;
- *linear expansion and contraction*: horizontal lengthening or shortening of elements such as beams and slabs, recognising that expansion typically induces compression and contraction typically induces tension.

Sources of movement

A number of sources of movement can be identified, including:

- early age effects such as early thermal contraction and autogenous shrinkage;
- long term effects such as drying shrinkage and creep;
- seasonal effects such as daily/weekly/seasonal temperature variations;
- post-tensioning, which causes both immediate and long-term movements and contractions, whilst inducing a compressive state of stress in the elements concerned;
- dead and imposed loads: causing both immediate and long term movements.

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Restraint to movement

The influence of the four basic types of restraint to movement:

1. *Internal restraint*: such as that provided by the reinforcement or created where temperature differentials exist or perhaps where differential drying shrinkage occurs.

2. *Surface restraint*: This may occur in composite floors because the contraction of the concrete slab is resisted by the supporting steel beams.

3. *Edge restraint*: This may occur when a wall is cast onto a footing or where a slab or wall panel is cast against a previously cast element.

4. *End restraint*: This may occur when a structure links two or more separated cores or shear walls, or where a basement floor is restrained by piles or by ground friction.

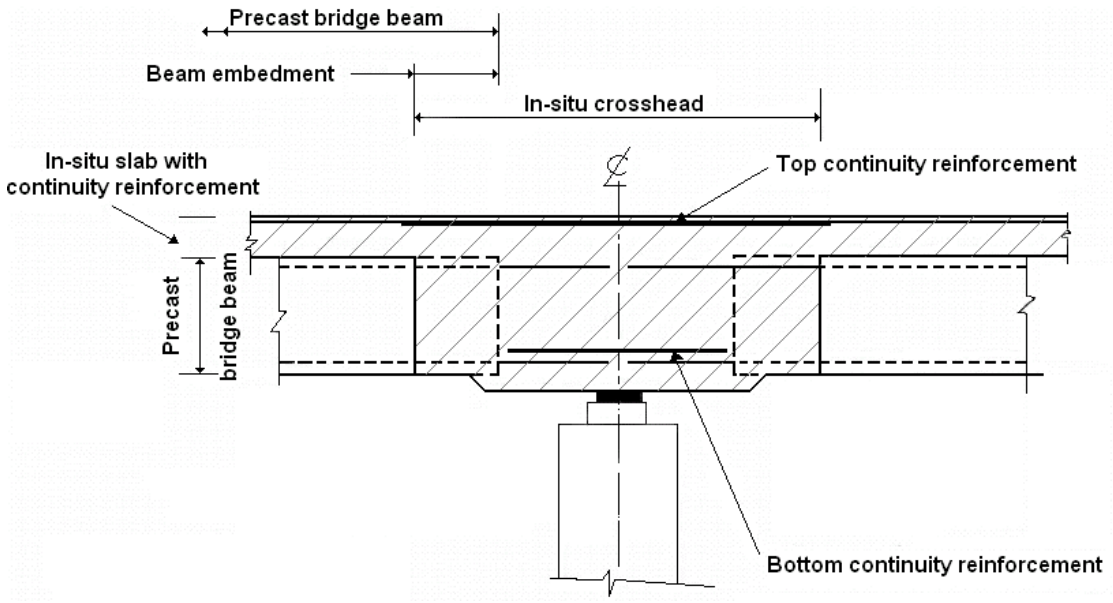

Fig. 5.4-17: Integral crosshead as shown in BA 57/01 [Highways Agency (2001)]

The concept of designing highway bridges with integral crossheads and abutments is an example where the decision was made to remove movement joints in the deck slab, which experience has shown are prone to maintenance problems. The in-service failure of movement joints provides a path for the ingress of water and chlorides into structural joints, deck ends, cross-heads, substructures and support bearings. This ingress promotes deterioration and corrosion. This approach has been adopted by the UK Highways Agency which has produced design guidance for bridges with integral crossheads and abutments in the form of BA57/01, *Design for durability* (Highways Agency (2001)]. This approach is illustrated in Fig. 5.4-17.

5.4.4.9 Weathering, discolouration and soiling of surfaces

Weathering is a natural process and it affects all buildings and structures, it cannot be ignored and therefore it is sensible for this to be recognised and allowance made so that the building or structure concerned can grow old gracefully. To do this successfully design and detailing of facades and concrete surfaces needs to control the flow of water across these surfaces. This needs to achieve a balance between the deposition of dirt from the atmosphere and

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the washing of the surface by rainwater in order to maintain an acceptable appearance in the longer term.

It is generally impractical to design on the assumption that these surfaces are maintained by cleaning, which incurs considerable cost and would commit the owner of the structure to this periodic expenditure for the life of the building or structure. Another problem with such an approach is that the building or structure concerned spends much of its life in a state where it looks as though it is in need of such maintenance work.

Hawes (1986) provides valuable guidance on these issues in his publication entitled "*The weathering of concrete buildings*" which formed part of the useful *Appearance Matters* series produced by the then (UK based) Cement and Concrete Association (C & CA). It is believed that this was prepared with the UK climate in mind, which is relatively wet and windy, and the recommendations made in terms of the detailing of facades took this into account. This suggests that particular consideration needs to be given to the movement of water across a building façade to avoid subjecting some areas to channelled rainwater flow, whilst other areas of the façade are not washed by rainfall at all. Hawes (1986) examines examples of good and poor practice, drawing out the good and problematic aspects of their performance in service. He notes that consideration should be given to a number of issues including:

- The general and local environment conditions.
- The choice of concrete surface; with rough textured surfaces having the advantage of spreading rainwater more evenly over the face and of disguising any localised staining. However, a rough surface texture will hold more dirt.
- Guiding water flow over the façade and at junctions and edges of panels and features.
- Maintaining uniform conditions; avoiding channelled rainwater flow into some areas whilst shielding others. The use of details to shed water from the building (i.e. drip details) clear of the façade and to collect water (i.e. gutters) are important, but seemingly often overlooked, items.

However, in other climates which might be much drier and dustier, other recommendations are likely to be appropriate. For example, in the climate of the Arabian Gulf it is recommended that the use of exposed surface finishes and section profiles which may entrap contaminated atmospheric dust should be avoided. In particular, it is suggested (CIRIA and Concrete Society (2008)] that there should be no elevated exposed horizontal surfaces; and any near horizontal surfaces should be provided with adequate unrestricted falls and drainage to conduct away rainfall and condensation. If a bush-hammered or deeply ribbed finish is chosen, the design and detailing of the reinforcement must allow for this so that adequate cover will be maintained in the finished structure.

5.4.5 Chemical deterioration processes in concrete

5.4.5.1 Introduction

Chemical deterioration arises from two main forms of attack:

- External chemical attack caused by sulfates, acids, nitrates, soft water (leaching), etc. and includes reactions such as the thaumasite form of sulfate attack (TSA).
- Internal deleterious chemical reactions such as leaching, alkali-aggregate reactions (AAR) and alkali-silicate reactions (ASR), delayed ettringite formation (DEF), etc.

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The associated deterioration and damage to the concrete caused by these processes may expose the embedded reinforcement to subsequent deterioration by corrosion or other means.

5.4.5.2 Sulfate attack

Sulfate ions can attack the hydrates in the cement paste, leading to the formation of ettringite and gypsum. The formation of these compounds can cause expansion, cracking and eventual disintegration of the concrete. This expansion is characterised by the presence of internal cracks (lined or filled with gypsum) parallel to the element's surface.

Sulfates of calcium, magnesium, sodium and iron are common in some soils and groundwaters. Fills (made ground) may also contain sulfates. Magnesium and sodium sulfates are particularly soluble and can lead to high concentrations of sulfate in solution. Sulfates can also be produced by the oxidation of sulfides, such as pyrites, through processes such as weathering or construction activities. Sulfates are also widely used in, or produced by, industrial processes. They may consequently be present as a contaminant of the soil and groundwater in the vicinity of former industrial sites. SO_2 gas may be released from various industrial processes and can attack concrete in the presence of water.

The mechanism and severity of sulfate attack depends on the type of sulfate. For example magnesium sulfate is a more aggressive agent than calcium sulfate.

Calcium sulfate reacts with the hydrated calcium aluminates in the cement paste to form ettringite, for example:

$$3CaO{\cdot}Al_2O_3{\cdot}6H_2O + 3CaSO_4{\cdot}2H_2O + 20H_2O = 6CaOAl_2O_3{\cdot}3SO_3{\cdot}32H_2O \qquad (5.4\text{-}2)$$

Sodium sulfate can additionally react with the free calcium hydroxide in the cement paste to form calcium sulfate, which can subsequently react with the calcium aluminate phases:

$$Ca(OH)_2 + Na_2SO_4{\cdot}10H_2O = CaSO_4{\cdot}2H_2O + 2NaOH + 8H_2O \qquad (5.4\text{-}3)$$

In conditions where the supply of sodium sulfate is continually replenished and where sodium hydroxide is removed, such as in flowing waters, this reaction can proceed to completion. However, where sodium hydroxide can accumulate equilibrium will be reached, depending on the concentration of sodium sulfate present.

Magnesium sulfate attack can be more severe due to the formation of (insoluble) magnesium hydroxide or brucite. This can lead to the decomposition of C-S-H gel by driving the following reaction to completion:

$$3CaO{\cdot}2SiO_2{\cdot}xH_2O + 3MgSO_4{\cdot}7H_2O = 3CaSO_4{\cdot}2H_2O + 3Mg(OH)_2 + 2SiO_2{\cdot}yH_2O + zH_2O \qquad (5.4\text{-}4)$$

Ammonium sulfate may be present in brownfield sites subject to industrial use and can be extremely aggressive towards concrete.

The presence of chlorides can reduce the rate of sulfate attack. Sulfate attack also proceeds more rapidly in cold conditions than in warm conditions.

The thaumasite form of sulfate attack (TSA): The presence of sulfates in the groundwater can also lead to the thaumasite form of sulfate attack in concrete where a source of carbonate is also available. Often this is where carbonate-containing aggregates have been used, but the carbonate ions can come from an external source. This process can reduce the cement

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paste matrix to a mushy incohesive white mass. Thaumasite formation in Portland cement concrete generally requires:

- the presence of a mobile groundwater;
- low temperatures (ground temperature may show significant seasonal variations);
- an adequate supply of SO_4^{2-};
- a source of CO_3^{2-} ions (generally from aggregates used in the concrete).

Prior ettringite formation may be required. Otherwise the amount of thaumasite formed where the supply of sulfate and carbonate ions is continuous is limited only by the availability of CaO and SiO_2.

5.4.5.3 Acid attack

Acid attack due to mobile acid groundwater (e.g. that containing sulfuric acid, carbon dioxide or organic acids such as fulvic acid or humic acid) can lead to deterioration of the concrete. The process involves the dissolution of the cement hydrates and of aggregates, such as limestone, leading to erosion of the concrete surface. The extent to which acid groundwaters attack concrete depends on their pH. However, acid groundwaters that are not mobile appear to have little effect on buried concrete.

Organic acids generally produce moderate pH values (>3.5) due to their low solubility whereas inorganic mineral acids can produce lower pH values. In the case of humic acid, reaction products formed on the surface of concrete are mainly insoluble and tend to impede further attack. Inorganic mineral acids are usually therefore more damaging to concrete. Sulfuric acid is the only mineral acid likely to be present in natural groundwater, where it can arise as a result of pyrite oxidation. It may also be found on land contaminated by industrial processes, as may hydrochloric and nitric acids.

Amongst other effects, attack of this type potentially reduces the depth of cover to the reinforcement, which might be of concern if the reinforcing bars could be subject to acid attack or exposed to say chlorides with the attendant risk of corrosion.

In concrete with siliceous gravel, granite or basalt aggregate, the surface attack will produce an ‘exposed aggregate’ appearance. However, in concrete with limestone (carbonate) aggregates, the aggregate may dissolve at a rate similar to that of the cement paste producing a smoother surface.

The rate of attack depends more on the rate of water movement over the surface and on the quality of the concrete than on the type of cement or aggregate. Immobile acidic groundwaters tend have little effect on buried concrete. However, mildly acidic (pH above 5.5) mobile water will attack concrete significantly, but the rate of attack will be generally slow; particularly if the acids are primarily organic in origin. Flowing acidic water may cause rapid deterioration of concrete; therefore high quality concrete is needed.

5.4.5.4 Leaching/soft (pure) water attack

Ground, lake and river waters can have very different chemical compositions depending upon the type of ground through or over which they have flowed. Such waters can be very ‘hard’ (i.e. containing high concentrations of dissolved ions) in areas where the ground is calcareous. Lake and river waters can also be strongly acidic in upland areas where the soil consists of peat and other materials which are rich in organic substances.

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'Pure' or soft waters, which contain low concentrations of dissolved ions, are aggressive to concrete; causing leaching of soluble lime based minerals from the concrete. Such attack can be severe when a concrete surface is subject to a large flow of this type of water. Concrete surfaces that are carbonated are less prone to this form of attack. Lake and river waters fed by large volumes of snow melt water can be very pure and hence aggressive.

Waters used in industrial processes and associated waste waters have highly varying chemical compositions. These might have pH-values below 4.5, which makes them acidic and very aggressive to concrete. However, very pure waters are also produced in industrial processes. Examples of these include condensation water from heat exchangers and cooling towers, etc.

Table 5.4-3 provides a guide to the aggressiveness of water to concrete. This is based upon the amount of aggressive carbon dioxide (CO_2) that is dissolved in the water.

Carbonic acid (H_2CO_3) is a weak acid that forms when carbon dioxide dissolves in water. Rainwater is therefore the common source. As it is readily neutralised by reaction with calcium carbonate in the ground, it will generally only be encountered in relatively pure soft waters such as those flowing from uplands of non-calcareous rock. The carbon dioxide (CO_2) dissolved in water essentially comprises three fractions:

- The part which combines with calcium ions to form highly soluble calcium bicarbonate ($Ca(HCO_3)_2$).
- The part remaining as dissolved carbon dioxide that is needed to stabilise the calcium bicarbonate.
- The remainder, which exists as carbonic acid (H_2CO_3) that can potentially attack concrete. This portion of the dissolved carbon dioxide is termed aggressive carbon dioxide. The carbonic acid reacts with the cement paste matrix or any limestone aggregate. It is this phenomenon which accounts for the formation of solution features (karst) in limestone strata.

Table 5.4-3: The aggressiveness of water to concrete – Ref EN 206-1 [CEN (2000)]

Water type	Aggressive CO_2 in water [mg/l]	Aggressiveness
1	< 15	Not aggressive
2	15 – 40	Slightly aggressive
3	40 – 100	Moderately aggressive
4	> 100	Highly aggressive

Thus aggressive carbon dioxide is usually only present to an appreciable extent in exceptionally pure natural waters since, in most cases, where the water contains dissolved salts, sufficient calcium carbonate is available to combine with the carbon dioxide as harmless calcium bicarbonate. Also, the potential aggressiveness to concrete is only of concern in situations where water is continually flowing over (or seeping through) the concrete. Diversion pipes or culverts around dams retaining moorland waters containing high concentrations of aggressive carbon dioxide can be subject to severe erosion, as can poorly compacted concrete or permeable concrete products (e.g. some aggregate concrete blocks) used in foundations.

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Interestingly the solubility of lime *increases* at lower temperatures. Fagerlund (2000) reports that pure water (no dissolved CaO or CO_2 and pH=7) is able to dissolve the following amounts of calcium hydroxide ($Ca(OH)_2$):

0°C: 1.85 g/litre; 20°C: 1.65 g/litre; 100°C: 0.71 g/litre.

Different component of the cement paste have varying degrees of solubility and are therefore at different risk of being leached out of the concrete (Fagerlund (2000)]. For example, calcium hydroxide ($Ca(OH)_2$) is more soluble than hydrated calcium silicate (CSH), which in turn is more soluble than calcium carbonate in carbonated concrete ($CaCO_3$). Aluminate and ferrite compounds in cement gel appear to be highly insoluble in acidic water. The following gives an idea of amount of soluble lime based minerals that might potentially be leached from the cement paste.

- The total weight fraction of all soluble calcareous substances in the cement paste (all lime based products except those in unhydrated cement) is about 85%.
- The total weight fraction of the very soluble $Ca(OH)_2$ in the cement paste is 25%.
- The total weight fraction of easily soluble reaction products in the cement paste (CSH-gel, exclusive of the reaction products relating to C_3A and C_4AF, but including $Ca(OH)_2$) is 65%.
- The total weight fraction of the more insoluble reaction products relating to C_3A and C_4AF in the cement paste is 20%.

For a water-cement ratio of 0.60 and a degree of hydration of 80% the *volume fraction* of soluble substances in the cement paste is about 62%. If the density of all the hydration products is assumed to be the same (as a first approximation), dissolution of all CSH-gel reduces the solid volume of the cement paste by 47%. Dissolution of all $Ca(OH)_2$ reduces the solid volume of the cement paste by 18%. Thus from this it can be understood that the effect of leaching on the strength and the elastic modulus of the concrete can be substantial; as are the potential implications for the durability of affected concrete structures.

The CONTECVET project team (CONTECVET 3 (2000)] observed that the annual water flow through each metre length of a crack notionally 0.1 mm wide through a 1 m thick wall subjected to a pressure difference of 10 m water head [105 Pa] would be in the order of 260,000 kg. This compares with a water flow through one square metre (1 m^2) of uncracked concrete of only 150 kg (assuming the permeability to be $5 \cdot 10^{-12}$ [s]). They concluded that leaching in the vicinity of cracks is likely to be very rapid. In addition, leaching will effectively widened the crack over time; so the water flow will therefore progressively increase. As the volume of water passed by the crack is proportional to the third power (cube) of its width, a doubling of the effective width of a crack will result in an eight-fold increase in the flow.

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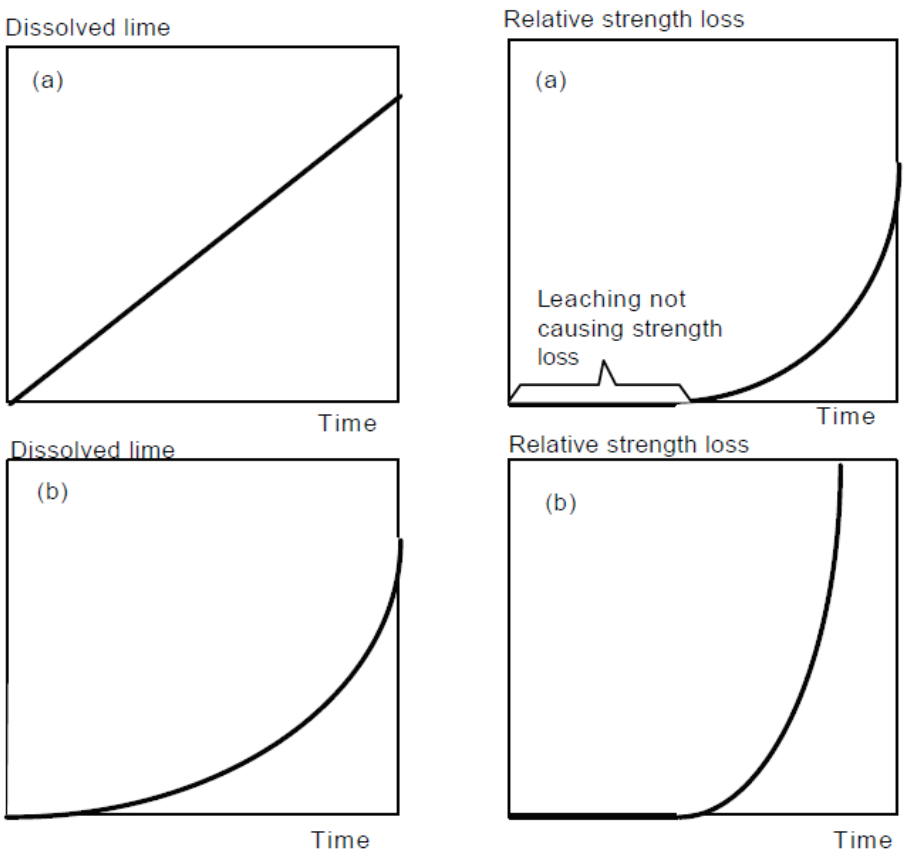

Fig. 5.4-18: Illustrative time relationships for leaching and loss of concrete strength. (a) Permeability of concrete is constant with time. (b) Permeability of concrete increases with increased leaching. [CONTECVET project: Report on leaching CONTECVET 3 (2000)]

Fig. 5.4-18 illustrates the notional time relationships for leaching and loss of concrete strength when (a) the permeability of concrete is constant with time, and (b) the permeability of concrete increases with time in relation to the effects of the greater degree of leaching that has previously occurred. This indicates that when the effect of previous leaching is taken into account, the rate of deterioration of concrete strength is likely to be rapid (i.e. the local reduction which occurs).

In addition to the adverse effects of leaching upon strength and elastic modulus of concrete, as leaching removes calcium hydroxide $Ca(OH)_2$ from the concrete this will ultimately have the effect of reducing the pH of the concrete pore solution. Depending upon circumstances, a reduction in the pH of the concrete (below a value of about 9) will increase the risk of corrosion of the embedded reinforcement. This problem will be exacerbated if the water causing the leaching contains chlorides. As leaching increases the porosity of the concrete, this could result in an increase in the rate at which chlorides penetrate into concrete (an increase in the apparent diffusion coefficient with time).

In cold climates leaching will also increase the risk of freeze-thaw damage: both internal frost damage and also surface scaling, which in turn is likely to further increase the risk of reinforcement corrosion.

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The CONTECVET project [CONTECVET 3 (2000)] considered five leaching scenarios to evaluate the potential implications of leaching upon structural performance. These were:

- Type 1 leaching: Surface leaching without erosion or a hydraulic pressure gradient.
- Type 2 leaching: Surface leaching as in Type 1 in combination with surface erosion.
- Type 3 leaching: Homogeneous internal leaching throughout the entire structure.
- Type 4 leaching: Semi-homogeneous internal leaching throughout the entire structure.
- Type 5 leaching: Selective leaching in defects and cracks within the structure.

Leaching types 3 to 5 involved a hydraulic pressure gradient applied across the section.

For leaching types 1 and 2 it was assumed that all easily dissolvable lime based minerals are removed from the surface zone experiencing leaching, but not from beyond that depth. In type 2 leaching, it was assumed that surface erosion occurred in conjunction with surface leaching.

For leaching types 3 and 4, the easily dissolvable lime based minerals were assumed to be gradually removed with time throughout the entire structure. In type 3 leaching, the removal of lime based minerals was assumed to occur uniformly throughout the full depth of the structure. In type 4 leaching, lime based minerals were assumed to be removed from the upstream part of the structure in preference to the downstream part (i.e. the water was assumed to arrive in the downstream part of the structure saturated with lime and was therefore unable to dissolve more lime). Over time the zone in which lime was being dissolved was assumed to gradually migrate downstream through the structure as all the easily dissolvable lime was removed in the upstream zone.

For type 5 leaching, it was assumed that leaching only occurred close to cracks passing through the element. Over time lime was gradually removed from these localised areas.

The leaching scenarios investigated gave some insight how leaching might potentially influence various aspects of the performance of concrete structures.

5.4.5.5 Alkali silica reaction (ASR)

Alkali silica reaction (ASR) occurs where alkalis in the pore solution react with certain siliceous constituents of the aggregate to form an expansive gel which is able to absorb water and swell. The pressure generated by this process can lead to an overall expansion of the concrete, typically producing a random network of fine cracks (map cracking) in unrestrained concrete. In reinforced concrete the cracking tends to form parallel to the reinforcement. In general ASR expansion does not have a major adverse effect on structural performance although this may depend on the structural context and the location of reinforcement detailing. At low levels of expansion the compressive strength of the concrete is little affected, although the tensile strength and elastic modulus may be reduced. Restraint of expansion can also cause bowing and movement of concrete components.

ASR-induced damage develops fairly slowly and visible cracking on the concrete surface may only be apparent at a late stage. Substantial amounts of gel are usually required before cracking occurs and the presence of the gel is not sufficient evidence to confirm that ASR is the cause of any observed expansion. ASR generally occurs where all of the following conditions are present:

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- The alkali content of cement is high and the alkalis are readily soluble. Soluble alkali components of the aggregate and salts from external sources (such as seawater or de-icing salts) may increase the total alkalinity.
- There is a reactive constituent in the aggregate. For example, in the UK the most reactive forms of silica are crushed greywacke, greywacke-type sandstones, greywacke-type siltstones/mudstones or combinations containing more than 10% of these. These materials are rarely used in concrete but the more commonly used cherts, flints and quartzites also contain reactive silica. The potential alkali reactivity of aggregates can be tested using standard test methods.
- There is a supply of water (although mixing water can, in theory, prove sufficient for ASR, in practice ASR damage is only observed where an external source of water is present).

Damage through ASR can therefore be prevented by limiting the total alkali content of the cement used in the concrete, by avoiding the use of ASR-susceptible aggregates or by ensuring that the concrete remains dry. Guidance for the prevention of ASR-induced damage is given in the Concrete Society report TR30 (1999), BRE IP1/02 (2002) and BRE Digest 330 (2004).

The orientation of cracks in structural elements due to alkali-silica reaction (ASR), such as that in the column shown in Fig. 5.4-19, is influenced by the provision of reinforcement and the state of stress in the element due to permanent loading. In the case of the columns, the expansion effects associated with ASR in the concrete are suppressed along the length of the column by the combined effect of heavy longitudinal reinforcement creating axial restraint and the presence of compressive stresses in the concrete from permanent loading. The horizontal component of the expansion of the concrete associated with ASR in the column has caused the relatively light horizontal stirrup reinforcement to yield, creating the pronounced vertical (longitudinal) crack in the column. Prestressed concrete elements affected by ASR exhibit similar characteristics, with the prestressing force tending to suppress expansion along the direction of the prestress, with cracking occurring transversely.

5.4.5.6 Delayed ettringite formation

The mineral ettringite is commonly formed at early ages in Portland cement concretes that have been cured under ambient conditions. Ettringite formed under these conditions is not damaging. However, there are circumstances where ettringite formation can lead to expansion and cracking of the concrete. Where the concrete has been subject to elevated temperatures during curing, either as a result of externally applied heating to accelerate strength development or in large concrete pours where significant heat generated as a result of cement hydration cannot readily be dissipated, ettringite formation may be ‘delayed’. Its gradual formation in the cooled, set concrete can lead to expansion and cracking in a process known as delayed ettringite formation (DEF).

DEF-induced damage is not a common phenomenon in mass concretes. Where it has occurred the phenomenon has been linked to high peak temperatures and high alkali levels in the cement. However, it can take up to 20 years for cracking to become apparent.

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Fig. 5.4-19: Cracking due to alkali-silica reaction in a column – Cracks form in the direction in which there is least resistance to expansion effects [BRE (2004)]

5.4.6 Biological deterioration processes in concrete

5.4.6.1 Introduction

Organisms such as bacteria, algae, fungi, plants, insects and even some marine molluscs and aquatic invertebrates are reported to affect concrete. Some influences primarily cause changes in the appearance of the concrete; whereas others cause damage and deterioration of the concrete surface, for example through the release of corrosive chemicals produced by metabolic action.

Deterioration in concrete structures from biological processes takes two main forms:

- Bacterial attack through the production of organic or mineral acids that react with the cement paste. These conditions also occasionally occur in sewers and other situations.
- Biological growth on concrete structures which may lead to physical and mechanical damage to the concrete surface.

An apparently rare form of biological attack has been reported by Neville (1995). This occurs in some tropical marine environments where particular types of oysters and sponges attack concrete made using limestone aggregate to obtain lime. They create holes some 10 mm diameter and 150 mm deep; with the reported rate of attack being up to 10 mm per year!

The rough surface of concrete can shield bacteria, making surface cleaning ineffective. However, admixtures introduced into the concrete mix can be use to control the colonising

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organisms. It is necessary to use an admixture that is toxic to the colonising organisms. These may be anti-bacterial, fungicidal or insecticidal in nature. However, their effectiveness tends to lost with time.

5.4.6.2 Microbial contribution to chemical attack

Bacterial attack is strictly a misnomer as the bacteria themselves do not attack the concrete. However, attack can occur through the organic or mineral acids produced by bacteria that react with the cement paste. The alkalinity of the pore water will neutralise the acid, producing a salt and water. The neutralising effect consumes alkali causing erosion of the cement paste, especially where the salt produced is soluble.

For example, under certain anaerobic conditions found in sewers and in sewage treatment works, the deterioration of concrete is caused by bacteria that feed on sulfate in the effluent. The biochemical process is as follows. In the absence of dissolved oxygen in the sewage, anaerobic bacteria split oxygen from the sulfate ion (SO_4^{2-}) generating sulfide (S^{2-}). This immediately reacts with water to produce hydrogen sulphide gas (H_2S), which is released from the effluent into the sewer atmosphere. As moisture is available, the hydrogen sulphide (H_2S) is readily oxidised by aerobic bacteria that live on the moist sewer wall to form sulphuric acid (H_2SO_4). This leads to acid attack upon the cement paste and erosion of the concrete, potentially leading to severe internal deterioration and loss of section in the sewer wall above the waterline. The sulphuric acid will also attack any exposed steel reinforcement.

If a new sewer is proposed in circumstances where hydrogen sulphide may be present, it is very important to obtain a realistic estimate of the likely hydrogen sulphide concentration. Experience [da Silva and Goyns (2009)] has shown that concentration readings taken in a manhole only represent the conditions in the manhole and that these can be very different from those in a sewer. This is because the ratio of effluent volume to atmosphere volume in a manhole is radically different to those in the adjacent sewer. Thus hydrogen sulphide concentration readings taken in a manhole will underestimate those in the adjacent sewer. Measurements taken in an experimental sewer [Goyns (2008)] found that hydrogen sulphide concentrations at a manhole were an order of magnitude lower than those in the adjacent section of sewer.

In another important process, sulfate-oxidising bacteria, such as Thiobacillus ferroxidans, help to oxidise pyrite (FeS_2) in the ground producing both sulfuric acid and sulfates that subsequently lead to sulfate attack of concrete. It is necessary to take pyrite oxidation into account where pyretic soils will be disturbed by construction [BRE (2005)].

Broomfield (2007) observes that bacteria also attack the sulphides (FeS) in steel and that this has been found to produce a black corrosion product where reinforcing steel is located in water saturated conditions in concrete bridge decks at locations where the water proofing membrane has failed. In anaerobic conditions such bacteria can contribute to pitting corrosion.

5.4.6.3 Biological growths on surfaces (aesthetic effects)

Whilst the high pH of concrete generally makes it resistant to biological attack, carbonation of the concrete surface will reduce its pH making it suitable for biological colonisation. A concrete surface is first colonised by micro-organisms; algae, fungi and associated bacteria. These are followed by visible growths of algae and lichens. Accumulations of dirt and dead lichen may then provide a footing for mosses and later even for larger plants.

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Algae require moisture, some traces of mineral salts and sunlight to growth. Green or dark coloured algae will grow on most concrete surfaces that remain damp. Fungi require moisture but get their energy from decaying plant life; so they can grow without sunlight. Lichens are symbiotic combinations of algae and fungi, which readily grow on moist concrete surfaces where the pH has dropped to about 7.5.

Colonisation of a concrete surface by algae and lichen change its appearance; as do inorganic deposits such as lime-bloom, efflorescence and calcium carbonate deposits. The microtexture (roughness) and permeability of the concrete surface play an important part in its biological colonisation. Poorer quality concrete surfaces accumulate more dirt, inorganic deposits and more biological growth than dense impermeable concrete surfaces. Such surfaces may also be wetter and more prone to deterioration by freezing in cold environment conditions.

Biological growths such as moss are reported [CEB (1992)] to promote chemical attack by the production of (humic) acids which attack the cement paste, resulting in erosion of the cement paste.

It is also suggested [CEB (1992)] that biological growths, such as moss and perhaps larger plants, can lead to physical and mechanical damage to concrete surfaces in some instances. This may be associated with plant roots which penetrate into cracks and weak spots in the concrete surface, perhaps at a very small scale, where they create bursting forces promoting further cracking and deterioration of the concrete surface.

5.4.7 Corrosion of reinforcement

5.4.7.1 Introduction

In general, reinforced concrete has proved to be a highly successful material in terms of both structural performance and durability. However, durability problems can arise from the corrosion of reinforcement, particularly where the concrete quality has been poor, where the cover to reinforcement has been inadequate or where chlorides have been present in the concrete. Corrosion can lead to cracking and spalling of concrete and to reductions in structural capacity. Corrosion of reinforcing steel is acknowledged to be the most common factor limiting the durability of reinforced concrete structures [Jones et al. (1997)].

A steel surface consists of grains, the presence of which is sufficient to set up small differences of electric potential over the surface. In the presence of moisture, small electrochemical cells may be established on the surface of the steel, creating anodic and cathodic reactions. An oxide layer is created by these corrosion reactions, forming the familiar rust layer. The oxide layer formed on steel (i.e. the rust) is friable, porous and has only relatively moderate adhesion to the parent metal. For corrosion to be significant and to be ongoing a source of oxygen is needed and there must be moisture on the reinforcement surface and in the adjacent concrete.

Stainless steels are produced by modifying the composition of steel by the addition of less reactive metals such as chromium, nickel and molybdenum. While these additions are expensive (i.e. high cost) they do create steels where the basic resistance to oxidation is increased, due in part to their relative position in the reactivity series of metals.

5.4.7.2 Passivation of steel in concrete

If the alkalinity of the electrolyte surrounding a piece of steel is greater than about pH 10.5 a very thin uniform protective surface oxide layer forms on the surface of the metal. The mi-

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croscopic oxide layer which is formed on the steel surface changes the electrochemical potential of the steel towards a more noble condition. This oxide layer constitutes a "passive" film, which impedes the dissolution of iron. Under these circumstances the rate of corrosion is insignificant.

In uncarbonated Portland cement concretes the pH is maintained at levels of at least 12.6 due to the presence of significant amounts of $Ca(OH)_2$ (the mineral portlandite otherwise known as calcium hydroxide) as well as sodium and/or potassium salts which can increase the pH further, up to say 13.5.

Reinforcement corrosion can occur when the protective oxide layer that is normally formed on the surface of the reinforcement is lost. There are two main causes for this in reinforced concrete:

- A reduction in the pH of the pore solution as a result of leaching in the vicinity of cracks or carbonation of the surface concrete due to the ingress of atmospheric CO_2.
- The ingress of chlorides from an external source or the presence/release of chlorides from an internal source (i.e. chlorides introduced at the time of casting the concrete).

5.4.7.3 Depassivation of steel in concrete by carbonation or chlorides

The reaction between calcium compounds, primarily $Ca(OH)_2$, and atmospheric CO_2 causes a chemical change within the concrete that increases the risk of reinforcement corrosion. This process, known as carbonation of the concrete, causes a decrease in the alkalinity of the pore solution to below pH 9 – see Fig. 5.4-20. The reduction in the alkalinity causes the "passive" surface film on the steel to breakdown which, in the presence of sufficient moisture, allows the reinforcement to start to corrode, as is illustrated in Fig. 5.4-21. Thus the reduction in the alkalinity of the concrete removes its ability to protect the reinforcement against corrosion. Fig. 5.4-22 shows diagrammatically the effect of a crack in concrete resulting in the localised depassivation by carbonation and the associated corrosion of a reinforcing bar in the vicinity of the crack.

The carbonation front within the concrete is usually very pronounced. At a particular location, the change in pH from 13 to 9 generally occurs over just a few millimetres (Fig. 5.4-23). Accordingly, the position of the carbonation front provides a basis for defining when corrosion initiation at the front face of the reinforcing steel will occur, which allows an estimate to be made of the age of the structure when this situation will occur.

Different concrete surfaces, such as the soffit and sides of a concrete beam, may experience different rates of carbonation depending upon local environmental conditions. Concrete elements which are exposed to a dry internal environment which carbonate at a significantly different rate to those exposed to an external environment. The rate of carbonation in an external environment will depend on the characteristics of the microclimate to which a particular surface is exposed, such as whether it is directly wetted by rain or it is in a sheltered situation.

The rate at which the carbonation front progresses from the surface of the concrete slows with time and this relationship is typically portrayed by a simplified relationship involving the square root of time since construction. The modelling of the increase in carbonation depth with time is discussed in Section 5.8.

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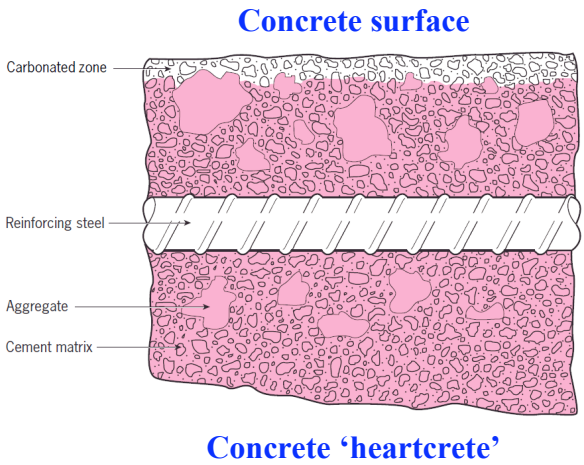

Fig. 5.4-20: Diagrammatic view of steel protected from carbonation-induced corrosion in partially carbonated concrete.

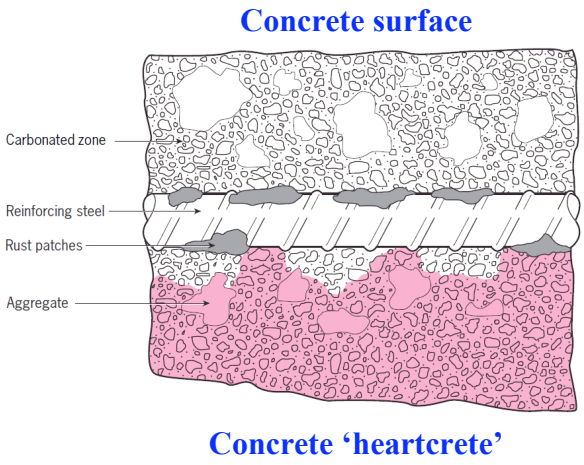

Fig. 5.4-21: Diagrammatic view of steel corroding in carbonated concrete.

Note: *The magenta colouring illustrated in these diagrams is produced by the use of the phenolphthalein test indicator. When it is sprayed onto a freshly cracked concrete surface the magenta colour indicates where the concrete remains in an uncarbonated condition (pH > 9). Carbonated concrete does not react and retains its previous natural colour.*

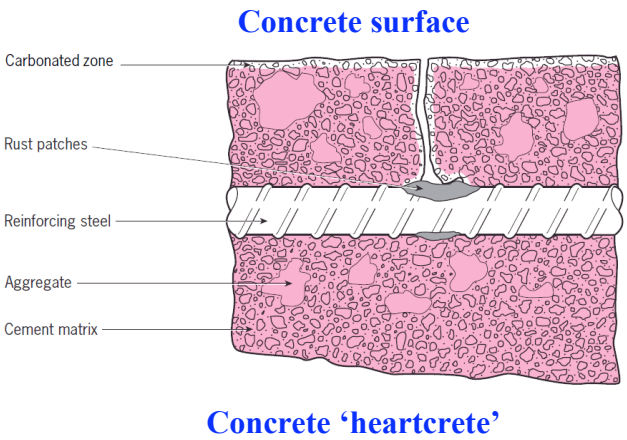

Fig. 5.4-22: Diagrammatic view of steel corroding in cracked concrete.

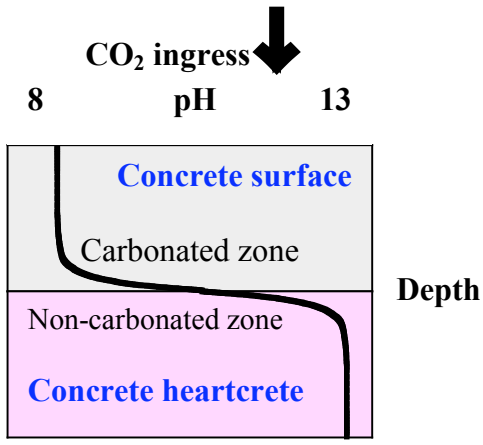

Fig. 5.4-23: Conceptual model showing the typical shape of a carbonation front. The grey shading shows the carbonated zone where the concrete no longer provides corrosion protection to the steel reinforcement. Compare Fig. 5.4-23 with the characteristics of Figs. 5.4-20 and 5.4-21

The ability of the concrete to carbonate is strongly dependent on the porosity (which is dependent on the quality of the concrete), the amount of carbonatable material present in the concrete and the relative humidity. An intermediate RH of between 60% and 80% allows the fastest rate of carbonation [BRE Digest 491, Sergi & Dunster (2004)]. Either of the two extreme conditions, that is very dry or water-saturated, practically eliminates the process of carbonation. Concrete subjected to wet/dry cycling will have a convective zone of a significant

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depth which is determined by the relative lengths of wetting and drying, curing and quality of concrete (Fig. 5.4-29). The range of RH in this zone is likely to be at the intermediate level that would allow significant carbonation to occur. Beyond this depth, carbonation would be very slow as moisture saturation of the pores would exist. The quality of concrete determines the depth of the convective zone.

The presence of oxygen and sufficient quantities of free chloride ions in the pore water of concrete can produce reinforcement corrosion even in highly alkaline conditions. Chlorides may be present in the components used to make the concrete at the time of casting (cast-in chloride) or enter the concrete from the environment after placing (ingressed chlorides). Principal external sources of chloride include de-icing salts, seawater, marine conditions, airborne salt and saline groundwaters.

Chloride ingress is due to either diffusion, taking place in totally or partially water-filled pores, or capillary suction of water containing chlorides. Cement has a certain chemical and physical binding capacity for chloride ions (forming Friedel's salt), depending on the chloride concentration in the pore water. However, not all the chlorides can be bound as equilibrium has to exist between the bound chlorides and the free chloride ions in the pore water.

Only the free chloride ions are relevant to the corrosion of the reinforcement. It is important to note, therefore, that after carbonation of concrete bound chlorides are released, so that the chloride content in the pore water, and consequently the risk of corrosion due to chlorides, will increase considerably. The critical chloride concentration at which corrosion will occur depends on many parameters and cannot be generally fixed. Fig. 5.4-24 shows chloride induced pitting corrosion of reinforcing steel.

Fig. 5.4-24: Chloride induced pitting corrosion of reinforcing steel (Photo courtesy BRE)

Generally where the concrete remains saturated, reinforcement corrosion is unlikely to be significant because of the restricted supply of oxygen. Chloride ingress into dry concrete exposed to chloride solutions is initially likely to be rapid and to occur through absorption, especially in poor quality concrete that is wetted infrequently and where subsequent drying is rapid. The depth to which chloride can penetrate through absorption is limited to the depth at

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which the pores remain dry. In temperate climates up to 20 mm of quality concrete can be penetrated in a few hours (Sergi & Dunster (2004)]. However, in arid conditions (either natural or due to the element concerned being located in internal or sheltered conditions) wetting and drying depths can be in the range 75 to 100 mm. Subsequent evaporation can lead to a build up of chlorides in the concrete. Under these conditions, reinforcement corrosion is likely to be a major concern. Surface chlorides can be removed through the action of rainfall.

5.4.7.4 Corrosion of steel in concrete

The passive film on the surface of the steel may be destroyed locally, or over larger surface areas, by the action of chloride ions or due to carbonation of the concrete. Another depassivating mechanism is the reduction of alkalinity which occurs when alkalis are leaching out of the concrete by aggressive water. In practice, this may happen at the surface of the concrete, but also at 'weak' points in the structure (e.g. leaking construction joints and wide cracks) and where concrete quality is poor (e.g. honeycombed, high placed water-cement-ratio etc). If the pH of concrete at the reinforcement drops below 9, or if the chloride content exceeds a critical value, the passive film and the associated corrosion protection will be lost.

Consequently, corrosion of reinforcement is possible when passivation has been lost and sufficient moisture and oxygen are available. This is certainly the case for structures in the open environment. The principles of passivation and depassivation hold true for both reinforcing and prestressing steels.

A simplified representation of the corrosion process is presented in Fig. 5.4 25. It is separated into two single processes; namely the cathodic process and the anodic process.

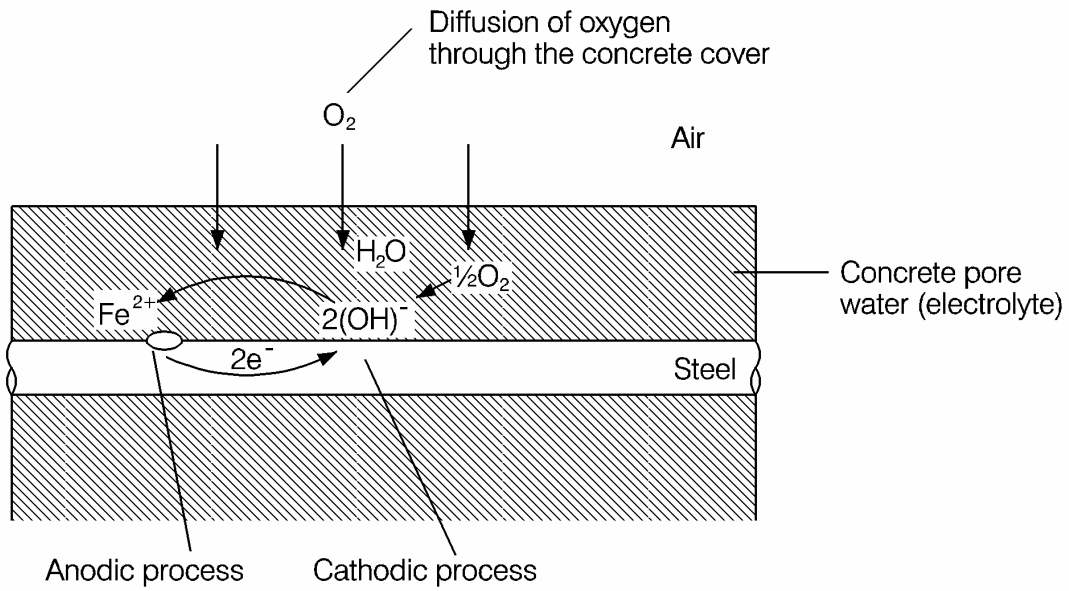

Fig. 5.4-25: Simplified model of the corrosion of reinforcement in concrete.

The anodic process involves the dissolution of iron. Positively charged iron ions (cations) pass into solution:

$$Fe \rightarrow Fe_2+ + 2e- \qquad (5.4\text{-}5)$$

The surplus electrons in the reinforcing steel move to areas where the passive film is intact and there combine with water and oxygen to form hydroxyl ions in the cathodic process:

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$$2e^- + \tfrac{1}{2}O_2 + H_2O \rightarrow 2(OH)^- \qquad (5.4\text{-}6)$$

After some intermediate stages, the iron and hydroxyl ions combine to form unhydrated rust products such as Fe_2O_3 and Fe_3O_4. These have a volume about twice that of the original steel that they replaced (Fig. 5.4-26). When these become hydrated they swell even more and become porous. Overall the increase in volume which occurs at the steel – concrete interface is some six to ten times the original steel that was consumed (Fig. 5.4-26). This change produces very high expansive pressures which, in turn, induce tensile stress in the surrounding concrete; potentially leading to cracking and spalling of the concrete.

This process requires only oxygen to form the initial corrosion products. This oxygen must normally diffuse through the concrete cover to the reinforcement. Water is necessary to enable the electrolytic process to take place and, depending on its availability, to combine with the initial corrosion products to form the hydrated rust that we observe. Comprehensive reviews of reinforcement corrosion are made by various authors including Page et al. (1990), Nürnberger (1996) and by Broomfield (2007).

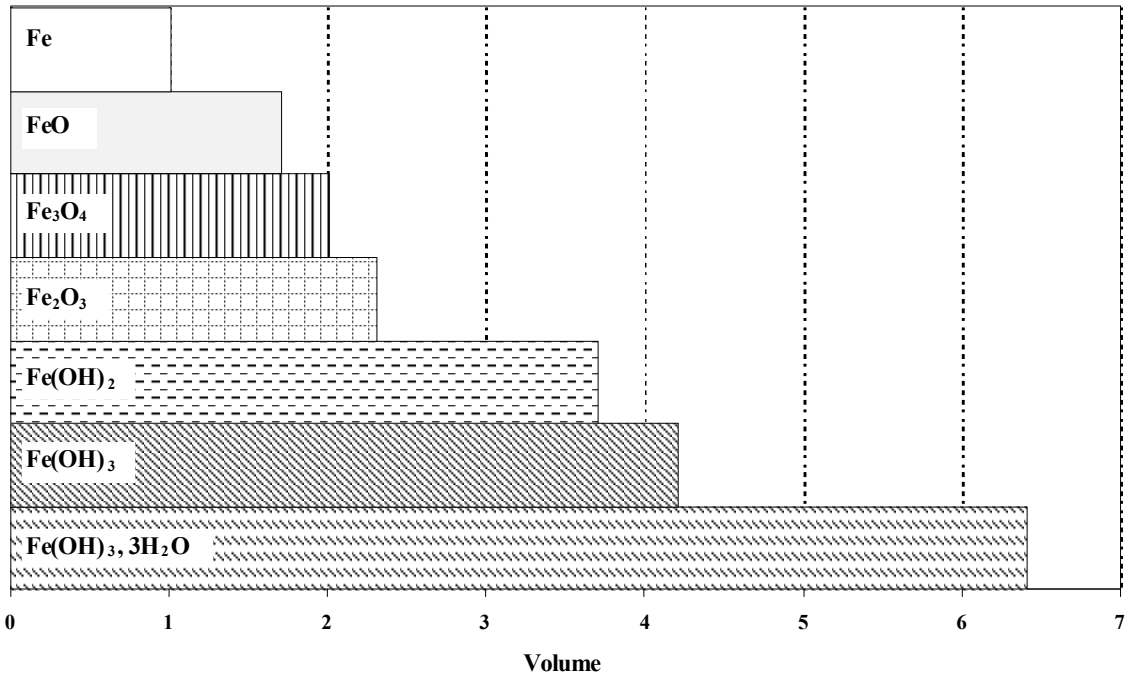

Fig. 5.4-26: Relative volume of iron and of its oxides created during the corrosion.

When corrosion develops in environments with low oxygen availability, the volume of the rust products may only be some 50-100% greater than the volume of the steel consumed in the reaction. Such corrosion processes proceed slowly, and in certain circumstances the corrosion products stay in solution and may diffuse into the voids and pores of the porous concrete without causing cracking and spalling. In such circumstances serious reinforcement corrosion and loss of cross-sectional area of the reinforcement may develop without there being any visible warning and a sudden failure of the element concerned might occur. Broomfield (2007) cites examples where reinforcing bars have experienced very severe damage and loss of section in saturated deoxygenated conditions in bridge deck slabs particularly under membranes or when water is permanently ponded on the surface of the slab.

From the above it follows that for corrosion of reinforcement to occur the following have to occur or exist:

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- Depassivation of the reinforcement must have taken place creating a location for an anodic process to develop.
- The depassivated area must be in metallic contact with a neighbouring or nearby area of the reinforcement which is passivated, in order for a cathodic process to take place.
- There must be oxygen available, primarily at the cathode in order to sustain the cathodic process creating hydroxyl ions, but also at the anode, where the availability of oxygen determines the type and volume of the rust products produced.
- The cathodic area and the anodic area must be connected electrolytically in order that ions can move between the two areas. Thus there must be sufficient moisture available in the concrete surrounding the reinforcement for the ions to migrate between the cathode and anode. This means that the electrical resistivity of the concrete must be sufficiently low for this ion movement to take place at a rate which sustains corrosion as an active deterioration mechanism.

All these conditions must be fulfilled for corrosion to take place. In this it is important to recognise that the rate at which iron is dissolved at the anode is determined by the rate of the cathodic process. In fact, the cathodic process is in most ordinary cases the rate determining (limiting) process. The corrosion process can perhaps be compared to the action of an engine. To operate an engine needs fuel; in this case the fuel is the oxygen diffusing from the surface of the concrete through the concrete cover to the surface of the steel at the cathode.

Chloride induced corrosion is probably the most serious and widespread type of deterioration of reinforced and prestressed concrete structures. Apart from chlorides entering into the concrete from sources such as seawater spray in the marine environment or from de-icing salts (these are commonly referred to as ingressed chlorides); chloride contamination of concrete can occur at the time of its manufacture, and these are referred to as cast-in chlorides. There are various mechanisms by which this can occur; one is for the fine aggregate to contain chlorides and another is for the concrete mix water to contain chlorides.

For example, in regions such as the Gulf countries chloride contaminated sand and water have been used in construction because of lack of alternative sources of construction materials. In some locations this practice no doubt continues for the same reasons. This has led to serious and in some cases, very early, corrosion damage. Fig. 5.4-27 illustrates the nature of the deterioration. In this case cracking of columns had been caused by reinforcement corrosion associated with chloride contaminated aggregates and concrete mix water.

During the construction boom of the 1960´s and early 1970´s calcium chloride based accelerators was used in many industrialised countries, these later led to the development of serious corrosion damage. Fig. 5.4-28 illustrates the nature of the deterioration which can arise from the use of cast-in chlorides. Surprisingly, calcium chloride based accelerators still appear to be marketed in some parts of the world. This is in spite of the known high risk of premature deterioration from reinforcement corrosion.

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Fig. 5.4-27: Cracking of column due to chloride induced corrosion caused by contaminated aggregates and mixing water. Age of structure at first cracking: 3 years (Gulf climate.

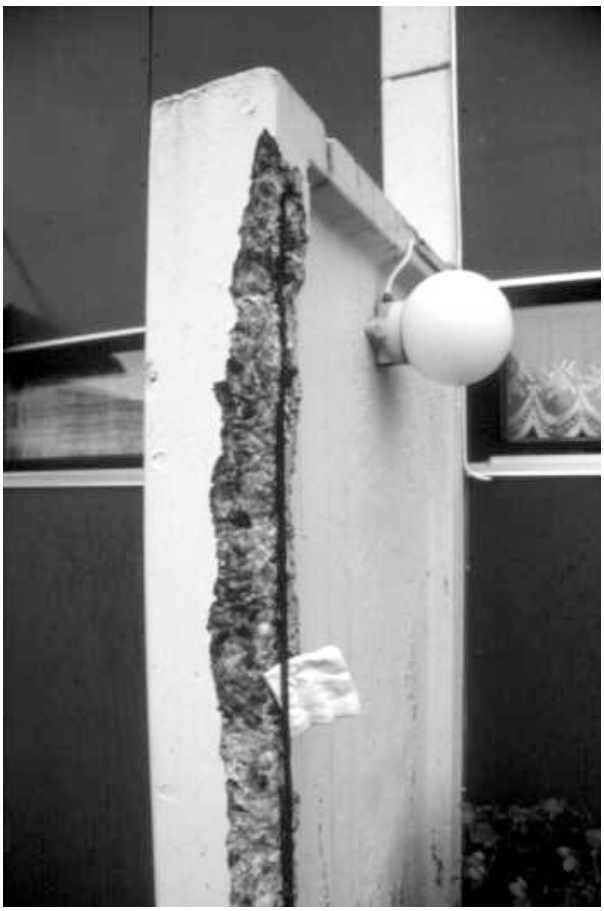

Fig. 5.4-28: Delamination of precast balcony partition wall due to chloride induced corrosion caused by calcium chloride added as mix accelerator. Age of structure: 15 years (North European Climate).

5.4.7.5 The influence of humidity upon the corrosion of steel in concrete

Thus in general significant corrosion will not occur, even if the passivating oxide layer on the surface of the reinforcement has broken down, when:

1. The concrete is in a dry condition; because the electrolytic process is impeded and the electrical resistivity is too high for a significant corrosion current to flow, or
2. When the concrete is water-saturated; because oxygen cannot penetrate to the cathode in a sufficient quantity to support the cathodic reaction.

The highest corrosion rate is likely to occur in a concrete structure exposed to changing conditions where both wetting (which ensures a low resistivity) and drying (allows diffusion of oxygen) frequently occur.

Sergi and Dunster (2004) examine the effect of humidity upon the corrosion of steel in concrete; explaining a number of the difficulties associated with defining the relative humidity (RH) at which significant corrosion activity occurs. The RH within the pore system of the concrete varies with depth into the concrete, approaching 100% deep into the body of the concrete. In a climate where the external RH and temperature are constantly changing, the RH close to the reinforcement some depth into the concrete is likely to be different from both the external ambient RH and the RH in the main body of the concrete. The degree of difference depends on a number of factors, including the degree of drying of the concrete and the depth

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of cover to the reinforcement. During relatively dry periods, the concrete loses moisture by evaporation and vapour transport through the exposed faces. However, the concrete gains moisture mainly by water absorption through the capillaries during wet periods. Vapour transfer can also occur through the exposed faces if the ambient humidity is significantly higher than that within the outer capillary pores. Water absorption is typically rapid and may carry with it soluble agents, such as chlorides.

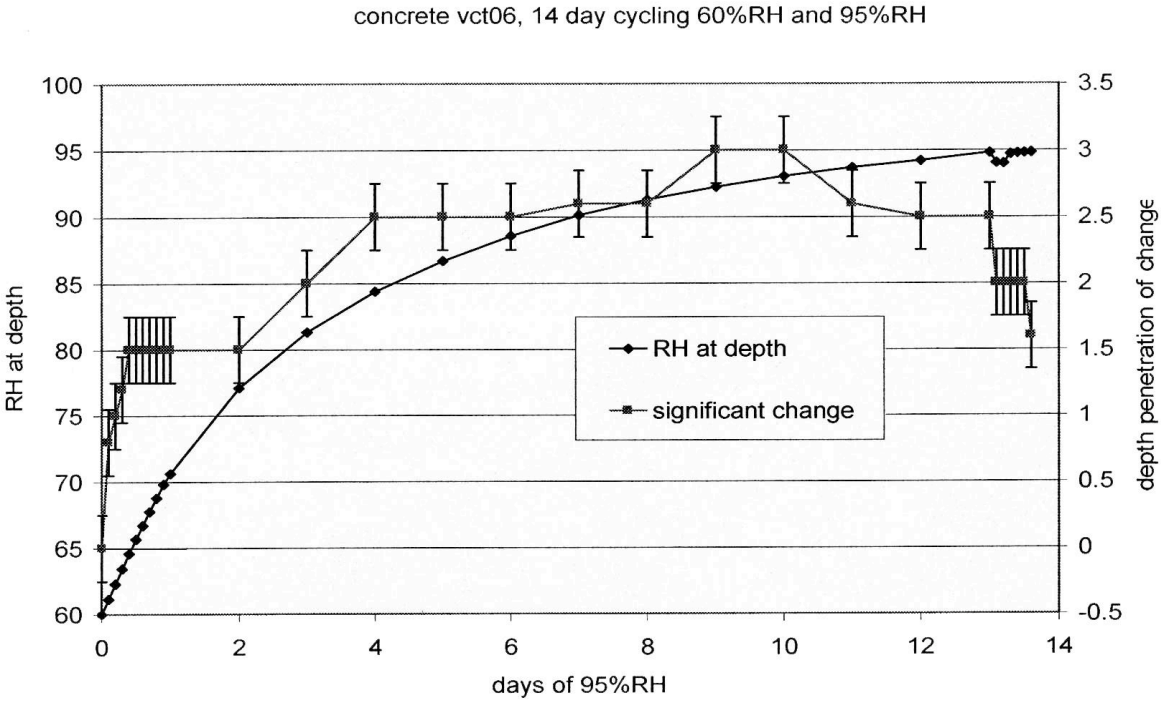

Fig. 5.4-29: Variation of RH at depth and of the convective zone for a concrete with a 0.6 water-cement ratio over a 14-day full cycle between T1 and T2 [from BRE Digest 491: Sergi & Dunster (2004)]

Notes to Figure 5.4-29:

The Jam-P software, developed by the Lund Institute of Technology, was used to model moisture changes in concrete with a 0.6 water-cement ratio, assuming a 14-day cycle of step changes from 60% RH directly to 95% RH. The temperature was assumed to be constant at 20°C.

The equilibrium relative humidity at depth and the penetration of the moisture changes were recorded for 30 combinations of external moisture cycle ranging from 14 days at 60% RH (T1) with 0 days at 95% RH (T2), to 0 days at 60% RH (T1) with 14 days at 95% RH (T2) ambient conditions.

Fig. 5.4-29 shows the results of this investigation where the depth of penetration of cyclic changes in moisture and the mean RH at depth in the concrete have been plotted for each of the 30 external RH combinations.

External RH affects both the mean RH at depth and the depth of penetration of changes (i.e. depth of the convective zone). The following trends can be inferred from obtained data.

1. *A period of at least 2 days of 95% RH exposure in 14 days is required to raise the RH at depth above 80% RH.*
2. *The maximum penetration depth, the convective zone, is 30 mm and this occurs with approximately 10 days at 95% RH and 4 days at 65% RH.*

Thus useful information about the depth of penetration of the ambient air RH fluctuations and about the RH at depth within the concrete can be obtained from the Jam-P simulations.

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Natural exposure studies [Bamforth & Pocock (1990), Bamforth & Chapman-Andrews (1994) and Bamforth & Price (2000)], involving a range of concrete types exposed to sea spray, showed that absorption and capillary suction were the predominant mechanisms of water (and hence chloride) uptake in the first 1–2 cm of concrete. However, the much slower ionic diffusion process was the main long-term migration mechanism further into the concrete. In a separate study on the effect of seawater spray, Salta (1994) found that the first 1-2 cm of concrete was rapidly penetrated by chloride-bearing water. As might be expected, this work provides evidence that the outer 1-2 cm of concrete responds rapidly to external moisture fluctuations, but that deeper into the concrete the RH of the internal environment is more stable and changes only slowly – see Fig. 5.4-29. In most cases, the rate of drying is considerably slower than that of wetting.

Laboratory work conducted by Glass et al. (1991) and Senaviratne et al. (2000) examined how the rate of corrosion relates to the resistivity of the concrete and more directly with the internal RH of the concrete.

Carbonation induced corrosion: The relationship established between corrosion rate and internal RH in carbonated concrete is presented in Fig. 5.4-30 [Sergi and Dunster (2004)]. In addition to the above cited research by Glass et al. (1991) and Senaviratne et al. (2000), the data presented also draws upon work by Parrott (1998) and Bamforth (1999). Although significant scatter is evident, the trend suggests that at RH values of less than about 75–80%, the corrosion rate of the steel reinforcement is lower than 1-2 μm/year cross-sectional loss (0.1-0.2 μA/cm^2 of steel area). This is a rate which is accepted as being insignificant over the design life of most structures. It is therefore possible to restrict the corrosion rate of steel in carbonated concrete to acceptable values by maintaining the concrete's internal RH at values below 75-80%.

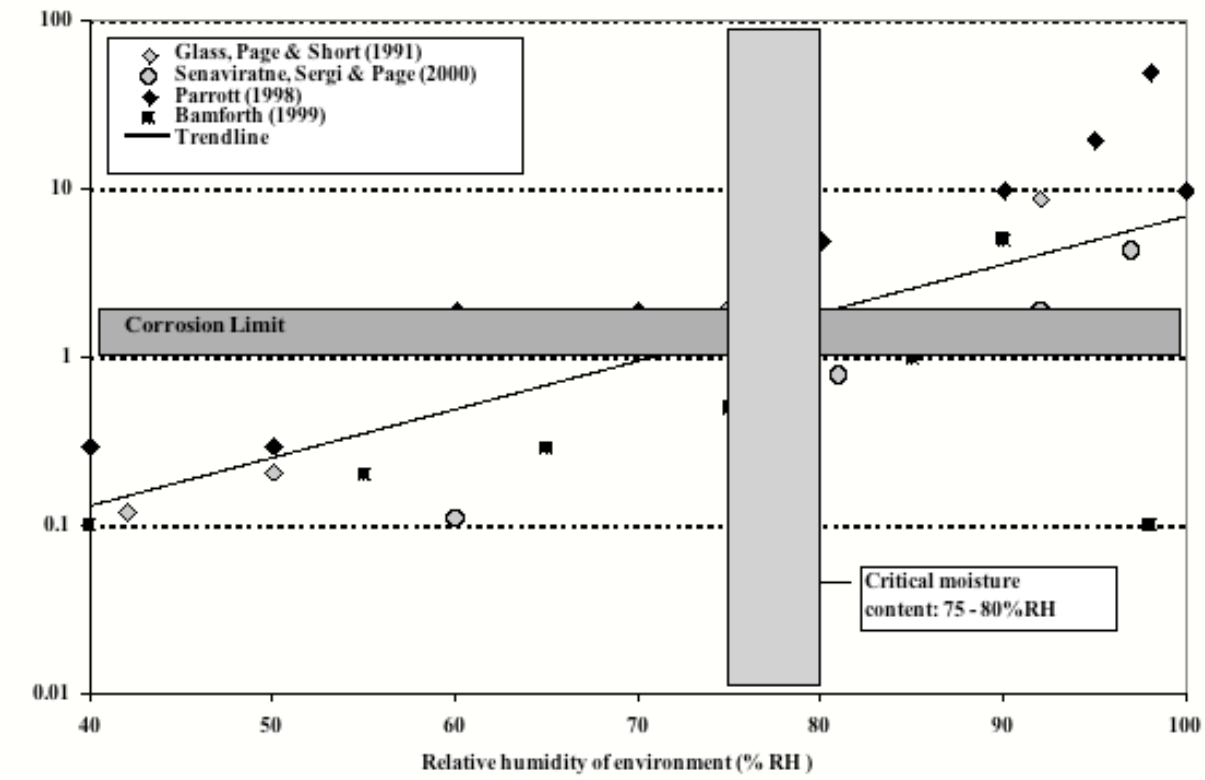

Fig. 5.4-30: Increase in corrosion rate with increase in RH of carbonated concrete [after BRE Digest 491 Sergi & Dunster (2004)]

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Chloride induced corrosion: The effect of internal RH of chloride-bearing concrete upon the rate of corrosion is less well understood, but it is generally accepted that there is a risk of corrosion of the steel reinforcement if significant chlorides are present at the depth of steel, irrespective of the level of moisture. Sergi and Dunster (2004) note that a few studies have been conducted which have shown that, although some control can be exercised on the rate of corrosion by reducing the internal RH of the concrete, it is not totally effective if significant chloride concentrations are present. When the concrete is also carbonated, the risk of corrosion increases further. So even for a low chloride content (~0.4% by weight of cement), the presence of chloride can increase the corrosion rate at relatively low RH values to unacceptable levels (Fig. 5.4-31).

Summarising: the internal RH of carbonated concrete directly determines the rate of corrosion of the steel reinforcement; which can be controlled if the internal humidity is reduced below about 80% RH. In the presence of chlorides, it is unlikely that reinforcement corrosion can be controlled just by keeping the internal humidity of the concrete below a certain (threshold) value. It is necessary to use additional protection measures to control reinforcement corrosion in concrete containing chlorides.

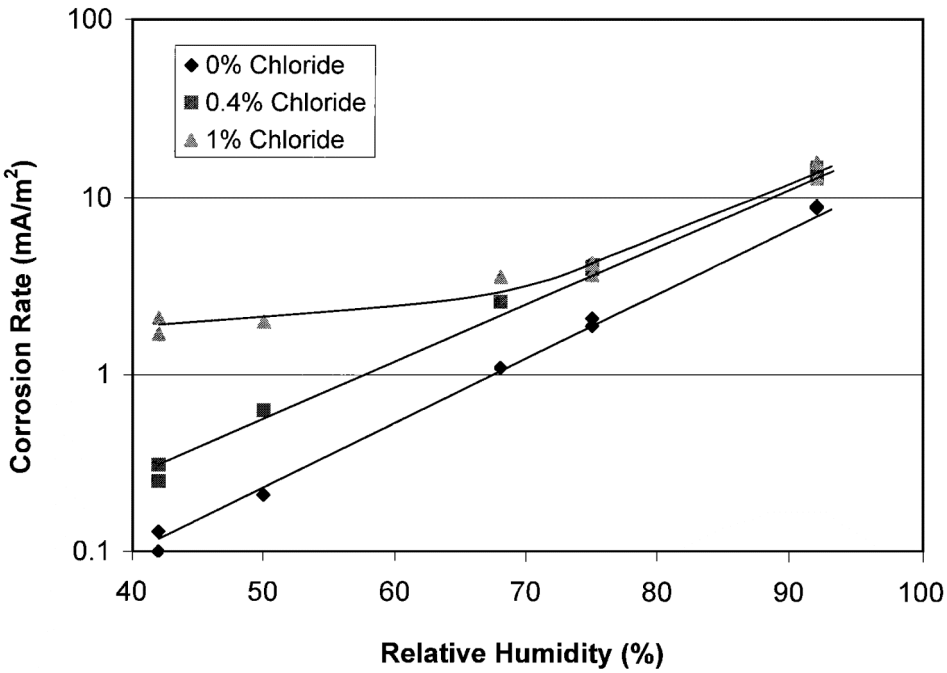

Fig. 5.4-31: Relationship between relative humidity and corrosion rate of steel reinforcement obtained on carbonated mortars dosed with chloride (1 mA/m^2 ≈ 1 µm/year) [from BRE Digest 491, Sergi & Dunster (2004) adapted from Glass et al. (1991)]

5.4.7.6 Critical chloride content (threshold) level to initiate corrosion

Chloride ions present in the pore water of the concrete act to cause the breakdown of the passive film established on the surface of embedded reinforcement, but this action is opposed by hydroxyl ions that act to reinstate the passive film. Accordingly, significant corrosion will only occur when sufficient chloride ions are available to overcome the beneficial effect of the hydroxyl ions. This suggests that, other than at a microscopic level, depassivation is a progressive rather than a sudden process.

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In recognition of this process, the critical chloride content (threshold) level to initiate corrosion of embedded steel reinforcement is commonly defined as a percent weight of the total chloride ion (Cl^-) content to the weight of cement. This is a parameter that can be easily and practically measured.

However, it is often postulated that it is the chloride ion to hydroxyl ion [Cl^-/OH^-] ratio in the concrete pore water that is critical. Thus, for a given total chloride content, those concretes that bind more chlorides would be expected to exhibit higher threshold levels as there would be less chloride available in the pore solution to compete with the hydroxyl ions. It has been recognised for many years that the C_3A content of the cement influences its capacity to bind chloride ions. This in turn influences the rate of chloride migration through concrete. Whilst there is considerable scatter in the results, concretes made from cements with a high C_3A content tend to have lower levels of unbound, or free, chlorides.

The critical chloride content (threshold) level required to initiate corrosion of embedded steel reinforcement has been the subject of a considerable amount of debate over recent years. A value of 0.4% by weight of cement is often used. However, values reported in the literature vary widely. On the basis of this it seems somewhat simplistic to assume that the chloride threshold level can be represented by a single value when initiation of corrosion is dependent on a number of factors, such as:

- the cement type;
- the water-cement ratio;
- the pH of the pore solution;
- the capacity of the cement paste to bind the chloride ions;
- whether the chloride ions are ‘cast-in’ or have ingressed from an external source;
- the exposure conditions;
- measurement difficulties associated with making experimental determinations, especially those for free chloride ions and hydroxide values near the reinforcement;
- local variations in chloride concentration at the reinforcement.

Fig. 5.4-32 shows the postulated relationship between the critical chloride concentration required for the initiation of reinforcement corrosion (i.e. the so-called critical chloride threshold level) and the moisture level in the concrete which was given in the 1992 CEB Design Guide [CEB (1992)]. It is assumed that this relationship was developed with Portland cement concrete in mind and the authors indicate that the represented condition is intended to relate to the state of incipient corrosion of the reinforcement. The relationship is influenced by various parameters as shown.

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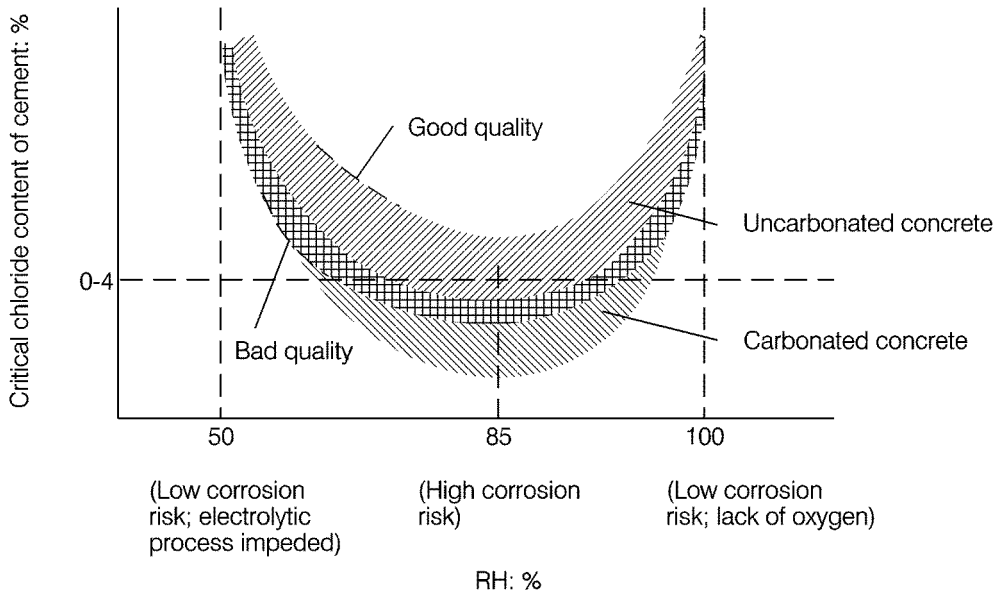

Fig. 5.4-32: Postulated variation of critical chloride concentration for corrosion initiation with moisture level in the concrete [CEB Design Guide (1992)]

The 1992 CEB Design Guide [CEB (1992)] indicated that there was no single value of chloride threshold level generally applicable to all concretes and environmental situations. The plot differentiates between uncarbonated and carbonated concrete. A baseline value of 0.4% chloride ion by mass of cement is adopted, but Fig. 5.4-32 indicates that the chloride threshold level could potentially be higher or lower depending upon environmental circumstances. The 1992 CEB Design Guide [CEB (1992)] also indicated that a lower chloride threshold level of 0.2% chloride ion by mass of cement was to be adopted for prestressed concrete structures.

Bamforth (2004) notes that whilst most data on chloride ingress is presented in terms of a weight of concrete, the critical chloride concentration for corrosion initiation (i.e. the chloride threshold level) is generally defined as a percent weight of the cement/cementitious material. Bamforth (2004) suggests that a conservative default value of 6.7 is used for the conversion when no specific data is available; which assumes a concrete density of 2350 kg/m³ and a cementitious content of 350 kg/m³.

Whilst a considerable body of research work has been carried out in an attempt to define a suitable threshold level, Bamforth (2004) also confirms that it has become increasingly clear over the years that no single value can represent the wide range of concreting materials and exposure conditions encountered. The general view seems to be that it is most appropriate to define chloride threshold level on the basis of perceived corrosion risk. Table 5.4-4 presents threshold level criteria based upon the corrosion risk classification proposed by Browne (1982).

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Table 5.4-4: Corrosion risk classification proposed by Browne (1982)

Chloride (% weight of cement)	Risk of corrosion
< 0.4	Negligible
0.4 to 1.0	Possible
1.0 to 2.0	Probable
>2.0	Certain

Bamforth (2004) notes that the recommendations given in Table 5.4-4 are reasonably consistent with over 450 results were obtained for the UK bridge stock by Vassie (1984). This data is shown in Fig. 5.4-33. This shows that for chloride contents below 0.2% by mass weight of cement, the risk of corrosion was very low.

Bamforth (2004) suggests that the commonly used chloride threshold value of 0.4% by weight of cement appears to be most applicable in conditions of wet/dry cycling of high water-cement ratio concrete mixes (greater than 0.6). Conversely it seems that higher chloride threshold values can be tolerated for concrete mixes with a low water-cement ratio in circumstances where environmental conditions are more stable. Blended cements tend to exhibit chloride threshold levels that are similar to, or lower than, that of Portland cement concrete. Corrosion inhibitors will increase the tolerance to chlorides (i.e. it raises the chloride content level at which corrosion of embedded reinforcing steel is initiated).

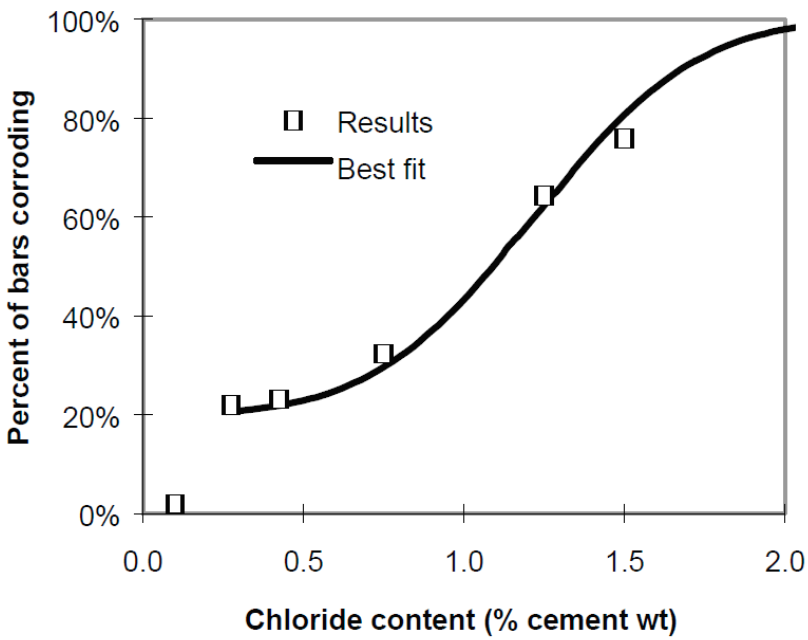

Fig. 5.4-33: The risk of corrosion at different levels of chloride [Vassie (1984)]

Angst and Vennesland (2009) report on a recent state of the art review of the critical chloride threshold above which corrosion of steel in reinforced concrete is reported to be initiated. Their findings are presented graphically in Fig. 5.4-34.

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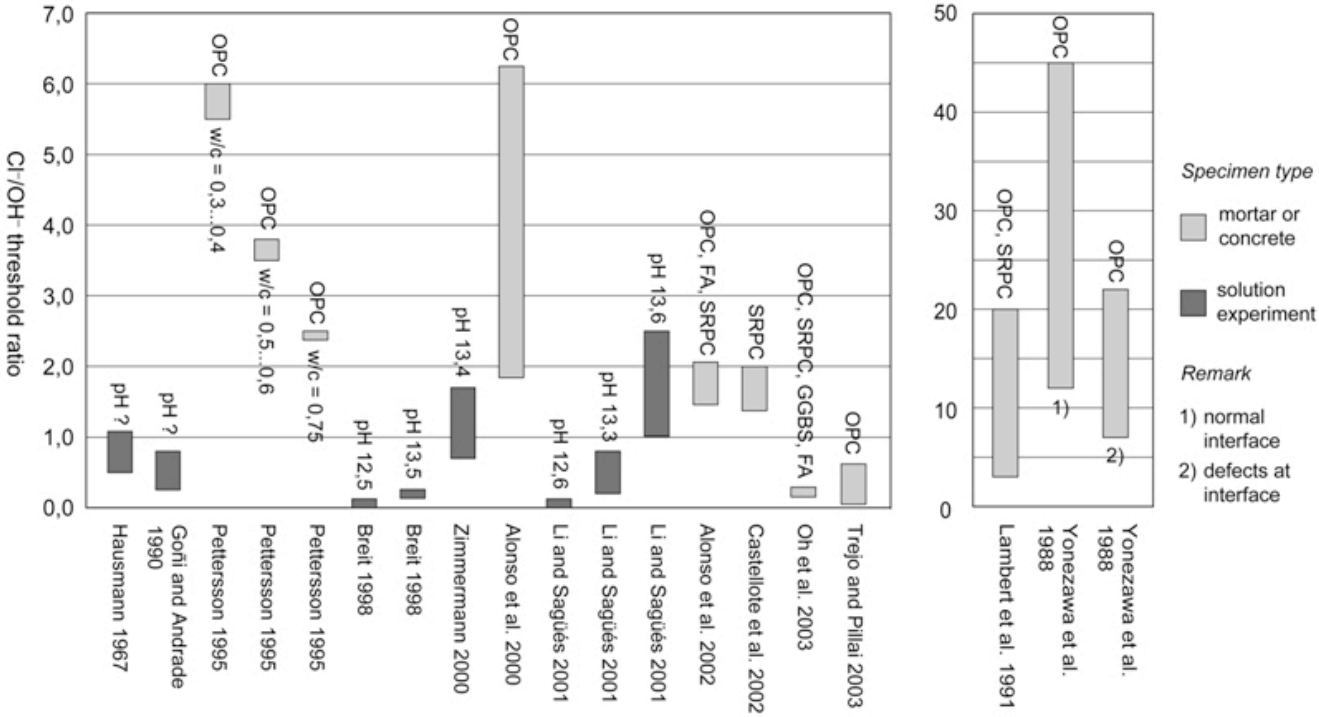

Fig. 5.4-34: Reported chloride threshold values expressed as Cl−/OH− ratios. As the chloride threshold values span a very large range, they are divided into two separate plots with different scaling on the ordinate [Angst and Vennesland (2009)]. Reproduced with permission.

Angst and Vennesland (2009) note that the results available in the literature show considerable scatter, with the reported values span from 0.02 to 3.08% total chloride by weight of cement/binder. This is over two orders of magnitude. The published Cl^-/OH^- threshold ratio values range from 0.03 to 45, which is over three orders of magnitude.

They also conclude that there is no single chloride threshold appropriate for the wide range of concreting materials and exposure conditions that are encountered. Major influencing factors identified by Angst and Vennesland (2009) included the quality of the steel-concrete interface, the pH of the pore solution and the electrochemical potential of the steel.

The quality of the steel-concrete interface depends on compaction achieved (absence/size of voids) and on the presence of a lime rich layer of hydration products. But this later feature has not been quantified. Other influencing factors are the moisture content of the concrete, type of cement, water-cement ratio, temperature, etc.

One of the problems encountered by Angst and Vennesland (2009) was the multiplicity of parameters employed in the experimental studies reviewed. These features include reinforcing bar type (smooth or ribbed), steel surface condition (polished, sandblasted, etc), matrix (cement paste, concrete, solutions, etc), chloride introduction techniques (mixed-in, diffusion, etc), depassivation detection techniques, etc. The diverse range of experimental possibilities created great difficulty in comparing the reported results. Thus the lack of standardisation in the experimental set ups also creates difficulties when seeking to compare the results produced by different authors.

Alonso et al. (2000), Alonso et al. (2002) and Izquierdo et al. (2004) also investigated the chloride threshold value required to cause depassivation and active corrosion of reinforcing bars cast into mortar specimens. They noted (amongst other matters) that the large scatter in the reported results was the number of variables that influence the chloride level required for

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depassivation. A difficulty was the lack of clarity in the definition of the chloride threshold itself and how depassivation is identified. Some workers use electro-chemical effects such as a shift in the measured corrosion potential or corrosion current to establish depassivation, whereas others use approaches such as visual observation to identify the appearance of rust spots on the steel surface. The threshold values determined by the various authors were expressed in terms of the Cl^-/OH^- ratio or by the weight of chloride ion to cement or concrete, with the chloride ion value sometimes being expressed in terms of free Cl^- and on other occasions being expressed in terms of total Cl^-.

Alonso et al. (2000) recognising these difficulties suggest that corrosion rate is the most reliable indicator, but they note that this approach also has its difficulties and uncertainties. Alonso et al. (2000) report chloride thresholds to initiate corrosion in mortar specimens were in the range 1.24% to 3.08% for total chlorides and 0.39% to 1.16% by weight of cement for free chlorides, respectively. Chloride thresholds in the range of 1.17 to 3.98 for Cl^-/OH^- ratios were found for chlorides added to the mixing water. Active corrosion was considered to be occurring when the corrosion rate of the reinforcement was higher than 0.1 $\mu A/cm^2$ over a local area.

Alonso et al. (2002) report on the influence of electrical potential upon the chloride threshold values required to cause depassivation and active corrosion of reinforcing bars cast into mortar specimens. They observed two forms of behaviour, with the dependence of the chloride threshold value on the electrical potential reportedly changing at −200 mV (SCE) [SCE = Saturated calomel electrode: mercurous chloride]. Their reported results were based upon six types of cement and the testing a series of identical samples, which allowed a statistical analysis of the results obtained. Alonso et al. (2002) report different chloride threshold relationships for electrical potentials, as follows.

- For electrical potentials more noble (more positive) than −200 mV (SCE): minimum chloride threshold values of 0.73 ± 0.03% by weight of cement for total chlorides, 0.50 ± 0.03% by weight of cement for free chlorides and a Cl^-/OH^- ratio of 1.76 ± 0.3.

- For electrical potentials more cathodic (more negative) than −200 mV (SCE), the chloride threshold values to initiate corrosion were found to vary linearly with electrical potential. Alonso et al. (2002) provide separate relationships for % Cl^- by weight of cement for total chlorides, free chlorides and as a Cl^-/OH^- ratio.

These differences in observed behaviour were attributed to differences in the composition in the passive oxide layer on the surface of the reinforcing bars at the different electrical potentials and the influence of oxygen reduction reaction producing OH^-.

Izquierdo et al. (2004) report on the influence of electrical potential upon the chloride threshold values required to cause depassivation and active corrosion of reinforcing bars cast into mortar specimens using six types of cement. They tested a series of identical samples, which allowed a statistical analysis of the results obtained. They observed two forms of behaviour, with the dependence of the chloride threshold value on the electrical potential reportedly changing at −200 mV (SCE).

- For electrical potentials more noble (more positive) than −200 mV (SCE): minimum chloride threshold values for a 95% probability of corrosion initiation range from 0.63 ± 0.11% to 0.77 ± 0.35% by weight of cement for total chlorides, and from 0.50 ± 0.13% to 0.57 ± 0.18% by weight of cement for free chlorides. The statistical analysis of the experimental results identified differences in the chloride threshold

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value for different types of cement, largely depending upon the C_3A content of the cement.

- For electrical potentials more cathodic (more negative) than –200 mV (SCE), the chloride threshold values to initiate corrosion were found to fit a log-normal distribution with a mean value for total chloride content of 1.5% and a coefficient of variation of 0.36. Chloride threshold values for various environmental situations and water-cement ratios are given.

For the purposes of service life design modelling, Bamforth (2004) proposes what he considers are conservative values as the critical chloride threshold for the different cementing materials/binders. These comprised Portland cement, pfa, ggbs, silica fume and metakaolin; with the latter four being used as partial replacements for Portland cement. The assumed critical chloride threshold values are given in Table 5.4-5.

Table 5.4-5: Assumed critical chloride concentrations for the initiation of reinforcement corrosion for different binders for service life modelling [Bamforth (2004)]

Binder	**Critical chloride concentration** (% Cl^- by weight of cement)	**Proportion of PC replaced** **Comment**
PC	Ct = 0.4%	
pfa	Ct = 0.4% X_{pfa} X_{pfa} = [1 – 0.010 (%pfa – 10)]	for pfa contents in the range from 10% to 50% Ct = 0.4% • 0.24% (for 50% pfa)
ggbs	Ct = 0.4% X_{ggbs} X_{ggbs} = [1 – 0.005 (%ggbs – 20)]	for ggbs contents in the range from 20% to 80% Ct = 0.4% • 0.28% (for 80% ggbs)
sf	Ct = 0.4% X_{sf} X_{sf} = [1 – 0.025%sf]	for sf contents in the range from 0% to 10% Ct = 0.4% • 0.3% (for 10% sf)
mk	Ct = 0.4% X_{mk} X_{mk} = [1 – 0.010%mk]	for mk contents in the range from 0% to 20% Ct = 0.4% • 0.32% (for 20% mk)

This table presents details given in Section 11.4 of Bamforth (2004), but with correction of various typographic errors contained in the original document.

Where:

PC = Portland cement
pfa = pulverised fuel ash or fly ash
ggbs = ground granulated blast furnace slag
sf = silica fume
mk = metakaolin

For pfa contents of less than 10% and ggbs contents of less than 20%, the threshold value is assumed to be the same as that for Portland cement (PC) concrete.

The critical chloride concentration is the assumed amount of chloride contamination of concrete (expressed in terms of % chloride ion [Cl^-] by weight of cement) required to initiate corrosion of embedded steel reinforcement. This is also referred to as the chloride threshold level.

The above expressions are only appropriate to concrete mixes containing PC with one of the following replacements pfa, ggbs, sf or mk (i.e. applied singularly).

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Bamforth (2004) indicates that the addition of pfa to concrete has the effect of lowering the pH of the concrete and that this is compensated for by a reduction in the free chlorides present in the pore solution; with the magnitude of the effect depending upon the level of pfa addition. However, he indicates that deriving a relative chloride threshold level (compared with PC concrete) is difficult, as the available data are inconsistent. Bamforth (2004) concludes that for service life design modelling purposes, it must be assumed that the addition of more than 10% pfa to concrete results in a reduction in the chloride threshold level (compared with PC concrete).

In the case of the addition of ggbs to concrete, this also has the effect of lowering the pH of the concrete and again this is compensated for by a reduction in the free chlorides. Bamforth (2004) indicates that deriving a relative chloride threshold level (compared with PC concrete) is difficult, as the available data are inconsistent. He concludes that for service life design modelling purposes, it must be assumed that the addition of more than 10% ggbs to concrete results in a reduction in the chloride threshold level (compared with PC concrete).

The addition of silica fume to concrete also has the effect of lowering the pH of the concrete, which for a given chloride content in the concrete has the effect of increasing the Cl^-/OH^- ratio. The available data on the influence of sf upon chloride binding are inconsistent. Bamforth (2004) concludes that for service life design modelling purposes, it must be assumed that the addition of silica fume to concrete results in a reduction in the chloride threshold level (compared with PC concrete).

Bamforth (2004) notes that comparatively few research studies have been undertaken to establish the effect of metakaolin on the chloride level necessary to initiate corrosion of the reinforcing steel. The influence of metakaolin on the chloride threshold level involves various conflicting factors, which makes it difficult to establish the factor which dominates. Bamforth (2004) suggests that for service life design modelling purposes, it be assumed that the addition of metakaolin to concrete results in a reduction in the chloride threshold level (compared with PC concrete).

5.4.7.7 General corrosion

Corrosion of steel reinforcement starts after depassivation of the steel surface. This occurs either as a result of the loss of protection provided by the high pH of concrete after carbonation of the concrete or by the presence of a sufficiently high level of chlorides in the concrete in the vicinity of the reinforcement.

Corrosion of steel reinforcement generally starts with the formation of small corrosion pits. These increase in number, expand in size and join up leading to generalised corrosion of the overall surface of the steel. This is sometimes referred to as microcell corrosion. General corrosion affects the whole surface of the steel and is typically associated with carbonation induced corrosion. The corrosion products (Fig. 5.4-26) typically occupy a considerably larger volume than the original steel. The production of ‘normal’ red rust eventually leads to cracking and spalling of the concrete cover to the affected reinforcing bars. These circumstances typically produce visually observable signs of deterioration before significant loss of reinforcement cross-section takes place.

5.4.7.8 Pitting corrosion

The chemistry of pit formation is quite complex. After local depassivation of the steel surface, corrosion is initiated on the surface of a reinforcing bar. Broomfield (2007) and Glass et al. (2006) provide some explanation about the nature of the pit formation process, noting po-

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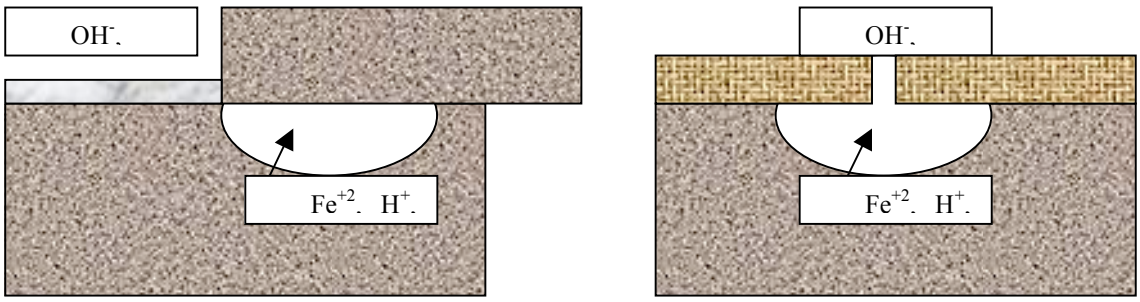
tential contributions from acidification, hydrogen sulphide and the exclusion of oxygen. Iron goes into solution in the pit and the absence of oxygen prevents the formation of a protective (passive) oxide layer. The presence of chloride and these circumstances accelerate corrosion. Severe pitting can result in significant local reduction in the cross-sectional area of affected steel reinforcing bars (see Fig. 5.4-24), potentially without there being any of the visual indications associated with cracking and spalling of concrete. Pitting corrosion can therefore severely weaken reinforcing bars and reduce the load capacity of affected concrete elements.

5.4.7.9 Other mechanisms – crevice corrosion

Crevice corrosion is an electrochemical process which develops within localised volumes of stagnant electrolyte trapped in pockets, corners or beneath some form of shielding layer, such as a surface coating. The gap or crevice can be formed between two metals or between a metal and a non-metallic material. The crevice develops a local chemistry which is very different to that of the bulk electrolyte outside the crevice. Crevice corrosion is a type of localised attack and corrosion damage is normally confined to one of the metals meeting at the junction between the surfaces forming the crevice, the other being passivated by the electrochemical reactions taking place.

Crevice corrosion is extremely dangerous because it is highly localised and can lead to component failure, even though the overall loss of material is minimal. The initiation and progression of crevice corrosion is difficult to detect.



Part A: Crevice between two metals or between a metal and another material

Part B: Crevice between metal and a surface coating

Fig. 5.4-35: Examples of situations potentially leading to crevice corrosion.

Note to Fig. 5.4-35: *Crevice corrosion can occur in the gap, the crevice, which exists between a metal and another material (Part A) or under a surface coating (Part B). Corrosion is initiated because there is no protection to the surfaces inside the 'crevice'. Corrosion continues because of the acidification which occurs inside the 'crevice' or because of the difference in oxygen availability or because of the difference in the concentration of solutes in the 'crevice'.*

Crevice corrosion is initiated by a difference in the concentration of some chemical constituents, which sets up an electrochemical concentration cell. This usually involves a difference in the availability of oxygen. If this is the case, the cell is referred to as a differential aeration cell. Thus outside of the crevice, which forms the cathode of the cell, the oxygen content and the pH of the electrolyte are higher than inside the crevice. Once a crevice has formed, the corrosion propagation mechanism is the same as for pitting corrosion. If chlorides are present, the electrochemical reactions taking place will concentrate them inside the crevice, exacerbating the corrosivity of the crevice environment.

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As oxygen diffusion into the crevice is restricted, the cathodic oxygen reduction reaction cannot be sustained in the crevice area. Accordingly the inside of the crevice becomes anodic. The anodic imbalance associated with the differential aeration cell leads to the creation of highly acidic micro-environment conditions within the crevice. If chlorides are present, the highly acidic environment is likely to be combined with a high chloride ion concentration, which arises because of the migration of chloride ions into the crevice. However, the chloride ions are not consumed in the corrosion process and remain to act as a catalyst, enhancing the corrosion rate.

Crevice corrosion can also occur in metal beneath a coating. The pores that are present in all coatings allow the passage of electrolyte into zones beneath the coating, perhaps where the coating is not well bonded to the metal substrate. Corrosion develops where the metal is not passivated. It usually progresses in an autocatalytic manner as the corrosion process produces an acidic environment in which the hydrogen ions (cations) are able to sustain the process. If there is a large ratio between the areas which are anodic and those which are cathodic promotes, as there is likely to be, this will promote a high rate of corrosion in a localised area.

The mechanism of crevice corrosion can be summarized as involving the following steps:

1. The electrolyte reaches the restricted space in the crevice or under the coating.
2. Corrosion starts where the metal there is not passivated. This corrosion leads to the increase in acidity of the electrolyte in the crevice as metal ions are taken into solution in the electrolyte. The hydrogen ions (cations) are able to maintain the cathodic reduction process inside the crevice.
3. The process is more aggressive if there is an external metallic surface able to maintain this reduction process in a larger surface. When the external surface is a passivated metal the crevice can progress significantly. A differential aeration cell develops separating the anode and the cathode. Due to current flow and mass transport phenomena, chloride ions migrate to, and build up inside the crevice, causing a reduction of pH in the crevice solution (acidification).

Anodic reactions inside the crevice:

$$Fe = Fe^{2+} + 2e^{-} \text{ (the dissolution of iron)} \tag{5.4-7}$$

The electrons given up by the anode flow to the cathode where they are discharged via the cathodic reaction:

$$Fe^{2+} + 2H_2O = Fe(OH)_2 + 2H^{+} \tag{5.4-8}$$

If the cathodic reaction occurs on the external passivated metallic surface, the reaction would be:

$$1/2O_2 + H_2O + 2e^{-} = 2(OH^{-}) \tag{5.4-9}$$

As a result of these reactions the electrolyte enclosed in the crevice gains positive electrical charge in contrast to the electrolyte surrounding the crevice, which becomes negatively charged. The positively charged electrolyte in the crevice attracts negative ions of chlorine Cl^{-} increasing acidity of the electrolyte on the basis of the following reaction:

$$FeCl_2 + 2H_2O = Fe(OH)_2 + 2HCl \tag{5.4-10}$$

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The pH of the electrolyte inside the crevice decreases (i.e. its acidity increases) from 6 to perhaps 2-3, which causes a further acceleration of the crevice corrosion process.

Corrosion products ($Fe(OH)_3$) form at the entrance to the crevice resulting in further separation of the electrolyte inside and outside the crevice.

Thus the mechanism of crevice corrosion is similar to that associated with pitting corrosion. However, as the initiation of the crevice corrosion process is different, it merits separate consideration.

Depending on the environment which develops in the crevice and the nature of the metal involved, the crevice corrosion can take a form of pitting (i.e. formation of pits), intergrannular attack or stress corrosion cracking.

In the case of steel reinforcing bars embedded in concrete, crevice corrosion may occur where bars are lapped or welded. In the case of prestressing wires or tendons, crevice corrosion might potentially develop in various situations such as at anchorages or in the ungrouted parts of postensioned ducts. It might notionally also develop in zones of incomplete compaction where say if an aggregate particle is in contact with a reinforcing bar. However, crevice corrosion has not been mentioned in the literature as a direct cause of failures associated with corrosion nor is it typically reported as one of the types of reinforcement corrosion to be concerned about. This is considered to be justified as pitting corrosion is attributed to be the main cause of reinforcement corrosion in the presence of chlorides or where prestressing strands corrode in ungrouted zones (i.e. where the steel is not protected against corrosion by the alkalinity of grout). Thus, in general, crevice corrosion does not appear to be a common problem in reinforced or prestressed concrete construction.

An important exception is the case of epoxy coated reinforcing bars, which are said to corrode because of crevice corrosion which is known to develop underneath the epoxy coating due to the penetration of the electrolyte through pores in the coating.

5.4.7.10 Other mechanisms – stress corrosion cracking

Stress corrosion cracking (SCC) is a particular case of localised corrosion. It can be a serious potential problem because the fine cracking that it produces can be difficult to find and is not necessarily associated with the more easily visually recognised general corrosion/rusting that engineers are used to looking for during inspections of structures. In some cases SCC can propagate quickly, while in other circumstances propagation can take many years to manifest itself.

There is a lack of awareness about SCC, which is compounded by the fact that there is no easily defined single rule that can be applied to cover all the situations where SCC might occur. However, in general the following combination of circumstances must exist together to lead to its occurrence.

- susceptible material, which can be a pure metal or an alloy;
- conducive aggressive environment involving appropriate chemistry, temperature, etc.;
- the presence of tensile stress – sometimes residual stresses caused by fabrication;
- the presence of a stress raising feature, such as a bolt hole incorporated in the design or the existence of damage.

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SCC can cause, through the development of cracking, sufficient reduction in the cross-section of the element concerned to result in a brittle failure which occurs without warning.

Environmental assisted cracking mechanisms such as SCC can occur in a wide range of situations, with examples including:

- Collapse of the roof of the Berlin Congress Hall in 1980 [Isecke (1983)] and the Ynys-y-Gnas Bridge in Wales in 1985 [Woodward and Williams (1988)]; both of which were due to the hydrogen embrittlement of high-strength carbon-steel prestressing tendons. In both of these cases SCC led to catastrophic structural failure without any visible signs of deterioration prior to failure.
- The failure of dry dock floor in Scaramanga, Greece, in 1977 which resulted from stress corrosion damage to high-strength carbon-steel anchor pile tendons [Martin and Arnold (1992)].

In the case of concrete structures, SCC tends to occur only in prestressing wires. Here the development of SCC is the result of a combination of high stress in the wires, which also have a very high yield stress, and contact with a specific aggressive medium. SCC requires the simultaneously combination of an aggressive electrolyte and a susceptible material stressed to above a certain tension.

However, SCC can also occur in materials such as stainless steel, as the failure of stainless steel roller bearings on the Thelwall Viaduct road-bridge in the UK in 2002 demonstrated [Edwards, Schofield and Burdekin (2009)].

The process of SCC starts by the nucleation of microcracks in the surface of the steel as shown in Fig. 5.4-36. When one of the cracks reaches a critical depth, the crack propagation velocity becomes very high and the wire breaks in a brittle manner due to the reduced cross-sectional area/associated increase in the level of the local tensile stress.

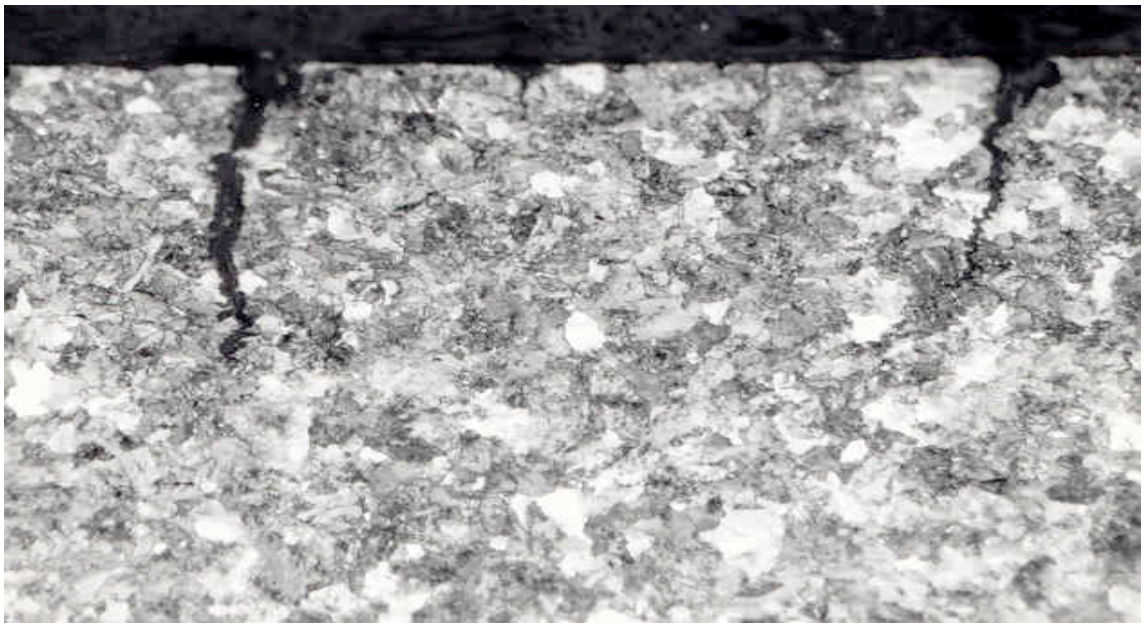

Fig. 5.4-36: Microcracks generated at the metallic surface (courtesy of J. Sanchez, C. Andrade and J. Fullea)

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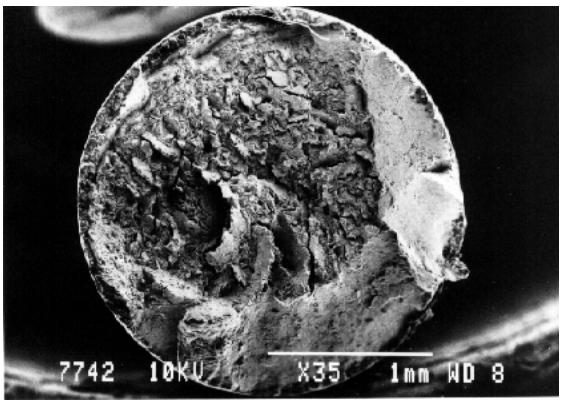

Fig. 5.4-37: A ductile fracture

(courtesy of J. Sanchez, C. Andrade and J. Fullea)

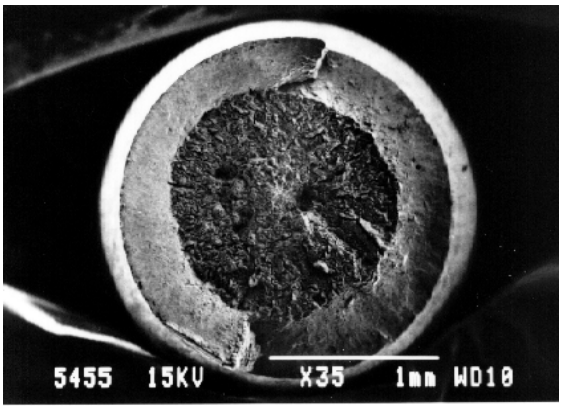

Fig. 5.4-38: A brittle fracture

The mechanisms of nucleation and the progression of SCC are still subjected to controversy. Nucleation can start at a surface inhomogeneity such as a corrosion pit. Crack progression is enhanced by the generation of atomic hydrogen at the bottom of the crack. Several mechanisms have been proposed to explain the process, but the one based in the concept of 'surface mobility' seems the best fit the experimental evidence. The concept of 'surface mobility' assumes that the progression of the crack is not of electrochemical nature, but that it is due to the mobility of atomic vacancies in the interface between the metal and the electrolyte. This implies the generation of atomic vacancies, which is likely to be due to the corrosion of the surface. These vacancies move towards the crack tip because of the stress concentration which exists there, thereby allowing the crack to progress. When the crack reaches a critical depth, failure occurs because of the reduced cross-section.

The only way to diagnose the occurrence of SCC is by the microscopic examination of fractured surfaces in order to identify the brittle fracture. Fig. 5.4-37 shows a ductile fracture of a prestressing wire and Fig. 5.4-38 a brittle one (no striations are produced). Fig. 5.4-39 shows a photograph of a prestressing wire which has experienced SSC in chloride contaminated concrete.

In the SCC phenomenon the metallographic nature and treatment of the steel plays a crucial role. While quenched and tempered steels are very sensitive, cold drawn steels are much less susceptibility. In many countries the use of the former is forbidden for prestressing products.

Specialist advice is required to assess the likely performance of materials and the risk of SCC during the design phase of a project. A major cause of this type of failure, which is not commonplace, is a lack of understanding of designers of the potential for changes in the performance of materials as they interact with their service environment.

The work of *fib* Task Group 9.5 into the occurrence of stress corrosion cracking of high-strength prestressing steels and associated failures in prestressed concrete construction in Germany is reported in *fib* Bulletin 26 [*fib* (2003)]. The influence of materials, together with design and construction issues, upon stress corrosion cracking of high-strength prestressing steels was investigated. Whilst experience has shown that steel tendons in prestressed concrete construction are durable when satisfactorily protected from aggressive environmental influences, *fib* Bulletin 26 (2003) reviews a number of structural failures and situations where steel tendons broke in a brittle manner due to the effects of stress corrosion cracking. The cases reported are from German experience over a period of about 30 years. Analysis of the

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examples considered indicates that the failures occurred as a result of a combination of causes such as poor design and detailing, errors made during construction, as well as the specification/use of unsuitable materials.

Fig. 5.4-39: Stress corrosion cracking in a prestressing wire (courtesy of J. Sanchez, C. Andrade and J. Fullea)

The work of *fib* Task Group 9.5 followed on from earlier activities in this field undertaken by the Fédération Internationale de la Précontrainte (FIP) and reported in the 1st, 2nd and 3rd FIP Symposia held between 1971 and 1981 – refer Bijl et al. (1971), Blekkenhorst (1973) and Elices & Sánchez-Gálvez (1981). The 1st, 2nd and 3rd FIP Symposia were all entitled "*Stress corrosion of prestressing steel*".

Further information on the subject of the durability of post-tensioning tendons is given in *fib* Bulletin 15 (2001). This contains papers presented at an international workshop entitled "*Durability of post-tensioning tendons*" held at Ghent University, Belgium on 15-16 November 2001. The workshop reviewed different aspects of the durability problems encountered, the resulting improvements in execution procedures and the new or enhanced prestressing systems that had been developed to overcome the problems. The invited papers addressed four principal themes: inventory and condition issues, investigation and repair, technical progress and strategies for improvement.

Following the Ghent workshop, further recommendations were developed to give greater certainty in achieving adequate durability of post-tensioning tendons. These were discussed at a second workshop again entitled "*Durability of post-tensioning tendons*" held in Zurich on 11-12 October 2004. The final recommendations were published as *fib* Bulletin 33 [*fib* (2006)].

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5.4.7.11 Other mechanisms – stray electrical current induced corrosion

Stray electrical currents can arise from various sources such as electrically powered railways, metros, trams and other electrically powered equipment. The electric return to the supply generator occurs through the ground (i.e. this is the normal state of affairs). In some instances the return electrical path passes through buried or embedded steelwork or steel reinforcement within concrete elements. This tends to occur when these form the lowest electrical resistance path to ground. In circumstances where such a current is passing from one concrete element to another, the pore water in the concrete can provide the ionic medium connecting the reinforcement cages in the different concrete elements involved. One end of the reinforcing cage or bar involved will become negatively charged (i.e. a cathode) and will not corrode. However, the other end of the reinforcing cage will become positively charged (i.e. an anode) and will corrode. Stray currents cause corrosion at locations where the electrical current leaves the conductor (reinforcing bar/cage/prestressing tendon). High strength steel tendons are more sensitive to stray electrical current corrosion than ordinary reinforcing steel.

Typically the greatest problems have been with elements (commonly buried in the ground) subject to large direct current (DC) from electrically powered railways, metros, trams, etc. However, Broomfield (2007) does note that alternating current (AC) induced stray electrical currents can also be a problem, but suggests that the associated problems tend to less severe than those encountered with DC systems. In some instances stray electrical currents can adversely affect structures protected against corrosion by the application of cathodic protection. In such conditions the AC induced stray electrical currents may shift the electrical potentials on the structure such that active corrosion can occur. Stray electrical currents can also be produced by cathodic protection systems, interacting with adjacent electrically discontinuous steel. For example, cathodic protection applied to buried steel pipelines can interact with other below ground structures and those adjacent or crossing the pipeline.

Stray electrical current induced corrosion in bridges, buildings and other structures is a specialised problem for which relevant expertise is required.

5.4.8 Deterioration mechanisms acting in combination

In most situations one deterioration mechanism will dominate. However, in some circumstances several deterioration mechanisms can act in combination. This is referred to as synergetic deterioration. In such cases the effects of several deterioration mechanisms will need to be considered in combination. This is illustrated in Table 5.4-6 which gives some examples of deterioration mechanisms acting together.

To date there is little experimental verification of possible synergetic effects, but experience suggests that certain combinations of deterioration mechanisms be considered. In these circumstances a risk based approach might be adopted to guide these considerations.

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Table 5.4-6: Some examples of synergetic deterioration, where several deterioration mechanisms act simultaneously [after CONTECVET project (2001)]

Combination of mechanisms	Possible effects
Surface scaling due to frost and corrosion	This may lead to a gradual reduction in reinforcement cover and, hence, to a greater likelihood of corrosion.
Alkali-silica reaction (ASR) and either frost action and/or corrosion	The expansive action of ASR may lead to wide cracks which can fill with water, and which, if frozen, may cause internal mechanical damage. Such cracks may facilitate the penetration of water containing chlorides or other aggressive species into the concrete and to the reinforcement, potentially increasing the severity of the corrosion. Conversely, gel formed by ASR may fill pores, thus densifying the cement matrix.
Leaching and frost action	An influx of water may increase moisture uptake, reducing the resistance of the concrete to internal frost damage.
Leaching and corrosion	The leaching of lime from the concrete cover zone reduces the alkalinity of the concrete and potentially increases the ease with which chlorides penetrate into the concrete. Leaching may also reduce the critical chloride threshold at which reinforcement corrosion is initiated.

5.5 Some factors influencing the durability of concrete structures

5.5.1 Introduction

The previous sections have illustrated the well established deterioration mechanisms which are typically responsible for the deterioration of concrete structures. In reality these deterioration mechanisms adversely affect only a limited number of concrete structures to a significant degree. However, consideration needs to be given at the design stage to the potential interaction between the service environment and the structure and how such deterioration mechanisms might affect the durability of a proposed concrete structure. The response of the structure is influenced by the nature and composition of the materials forming the structure, as well as other factors such as its geometry and the standard of execution achieved during construction.

It is also evident that most deterioration mechanisms involve an aggressive substance penetrating into the outer concrete layer of concrete (the cover) from the surrounding environment, as is illustrated schematically in Fig. 5.5-1. This emphasises how important the concrete cover is in terms of protecting the overall structure against deterioration from external environmental influences.

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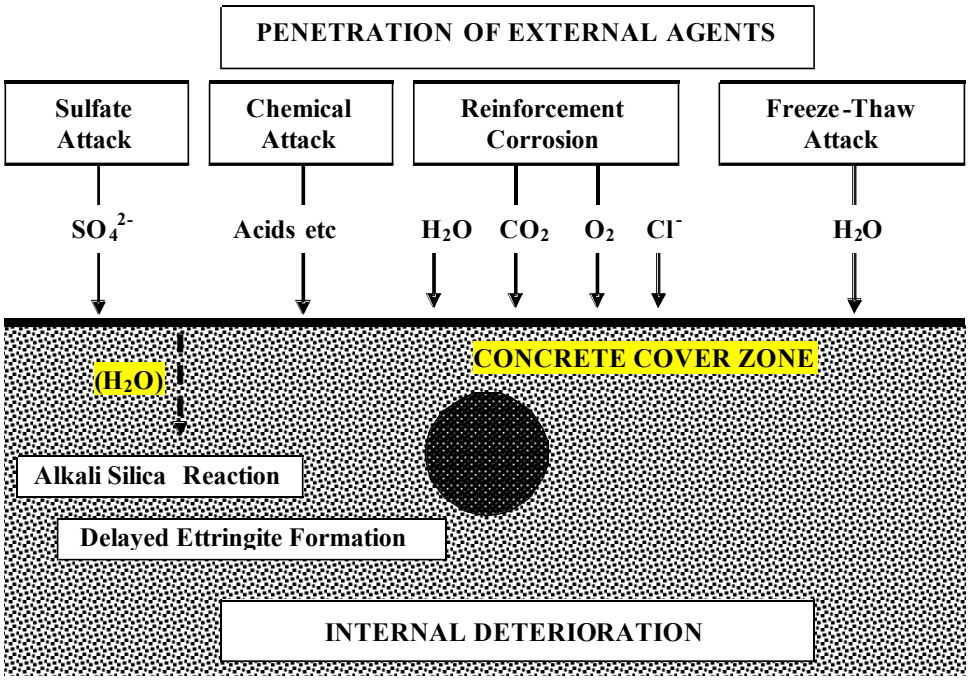

Fig. 5.5-1: Some mechanisms of deterioration that affect concrete structures and associated environmental influences [Price (2000)]

During the service life design process the designer needs to consider the main factors influencing the durability of concrete structures by taking the following actions:

- Identify the specific physical actions and aggressive chemical substances which may be expected in the service environment in question during the foreseen service life, together with the potential transport mechanisms bringing the aggressive substances into contact with the exposed concrete surfaces and into the concrete. Section 5.4 has reviewed the mechanisms that may cause deterioration or damage to concrete structures.

- Differentiate the service environment into appropriate zones for service life design and detailing purposes. This topic is discussed in Section 5.2.3 which recognises differences in environmental influences at the macro-climate, meso-climate and micro-climate scales; with Fig. 5.2-2 identifying some micro-climate conditions on a bridge subject to de-icing salts. The differences which occur at the micro-climate scale are difficult to establish definitively during the early stages of the design process because these variations arise from the interaction between the local environment and the detailing of the structure. Section 5.6 considers environmental aggressivity issues in more detail and presents a classification of various environments by their aggressivity.

- Design the geometrical form of the exposed parts of the structure to enhance resistance to the service environment. For example, in wet environments consideration would need to be given to the movement of water across the structure in order to avoid enhancing loadings from aggressive substance present in the service environment. In drier and dustier environments, the objective could be to avoid exposed surface details which might entrap contaminated atmospheric dust. Design considerations could include matters such as the articulation and jointing of the structure. These matters also greatly influence the weathering of concrete surfaces.

- Select the cement type and concrete properties (strength, water-cement/binder ratio, mix ingredients and proportions, etc) required for the service environment to resist the

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possible deleterious actions identified; assuming the design, execution and planned maintenance activities are undertaken to a satisfactory standard.

- Select the type of reinforcement most appropriate to the chosen concrete and the service environment, plus the detailing arrangements of its layout in the structure.
- Select an appropriate depth of concrete cover and acceptable crack widths for the service environment.

Mathematical modelling of deterioration mechanisms can be used to gain insight into the potential rate and consequences of deterioration under different assumptions. Such activities can be used to inform wider considerations about the service environment, the structural and geometric form to be adopted, the choice of concrete composition and the type and location of the reinforcement, together with the assumed nature and frequency of maintenance activities. On the basis of these assumptions the expected service life of the structure can be estimated.

Later, when the structure is in service, these assumptions can be verified by observations and measurements made on the actual structure in-service. This process allows the service life estimate to be updated on the basis of the actual environmental conditions and the in-service performance of concrete structure in response to these.

5.5.2 Geometrical form and architectural detailing of the structure

Both geometrical form, in conjunction with architectural design and detailing, can have important influences upon the durability and longer-term aesthetic appearance of a building or structure. The nature of these influences depends upon the particular climatic conditions, for example whether the climate has maritime or continental characteristics. An important influence on the local performance of façade elements is whether the environmental conditions are wet or are dry and dusty. Thus consideration needs to be given to these factors in addition to those focussing solely upon material properties and performance.

Good architectural design and detailing can help promote excellent performance of concrete structures with respect to durability and appearance, which are components of a long and satisfactory service-life. Weathering, discolouration and soiling of concrete and other surfaces are natural processes which affect all buildings and structures. They cannot be ignored and therefore it is sensible for this to be recognised and appropriate allowances made at the time of design, so that the building or structure concerned can age gracefully.

In wet climates the design and detailing of facades and other concrete surfaces needs to control the flow of water across these surfaces. This seeks to achieve a satisfactory balance between the deposition of dirt from the atmosphere and the washing of the surface by rainwater so that an acceptable appearance can be maintained in the longer term. In this context ‘dirt’ may potentially include a wide range of substances including for example particulate matter, saline deposits in marine or coastal environments, chemical deposits, as well as various forms of inorganic and organic soiling, etc.

A complex geometrical structural form is likely to increase the sensitivity of the structure to the aggressivity of the service environment, potentially shortening the overall length of the useful service life achieved or increasing the maintenance effort needed to keep the structure in a satisfactory condition. Other factors, such as the quality of the execution achieved during construction and the conditions of use, will also affect the durability of the structure.

Hawes (1986) provides guidance on how a number of these issues might be addressed in his publication entitled “*The weathering of concrete buildings*”. The advice given was devel-

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oped by the then Cement and Concrete Association (C & CA), a UK based organisation, and relates primarily to buildings situated in the relatively wet and windy UK climate. However, this guidance will be generally applicable to buildings and other structures in other temperate wet climates; and with aspects of the underlying principles being applicable to a wider range of climate types. Hawes (1986) discusses the various influencing factors associated with other climate conditions. For example, most of the UK has a moderate amount of rainfall occurring on many 'rain days' spread throughout the year. It is rare to get prolonged periods of dry weather. In the UK climate relative humidity is usually is excess of 60% and frequently rises to more than 95% for part of the day. In common with other maritime climates, condensation can occur on façade elements causing wetting of elements not normally exposed directly to rain. Taken together, the above factors give externally exposed materials little opportunity to dry out. Thus concrete surfaces exposed to these environmental conditions are expected to be frequently wetted and to remain damp for long periods.

Hawes (1986) observes that the presence of moisture on a surface may facilitate the bonding of particulate matter to that surface, even to soffits. Thus light rain, drizzle or condensation may encourage the attachment of dirt. Conditions of long periods of heavy driving rain causing rainwater to flow down a façade can be expected to act as a cleansing process. However, for shorter periods of rainfall, the water flowing down a large expose concrete façade will commonly be absorbed before it reaches the lower levels. This partial washing of a façade can lead to a streaked appearance, potentially forming a conspicuous wavy dirt line at the bottom of the washed zone (Fig. 5.5-2) which is aesthetically unappealing and commonly promotes a strong adverse reaction from the public. In these circumstances these differences in appearance make the building look 'dirty'. This is judged by most observers, professional and public alike, to be an unacceptable outcome. These effects can apply at different scales; both at a large-scale affecting say an entire façade on one elevation of a building and at the smaller-scale of an individual cladding unit.

There are a number of factors that determine whether a surface will be cleaned or not include the quantity of rain, its intensity and its cleanliness and acidity; together with the texture, porosity and cleanliness of the surface, plus the degree of saturation of the façade material. However, when detailing a concrete façade or other surface, it is suggested that the intent should be for water to be kept off the surface unless enough water will be available to flow across and clean the entire surface.

Fig. 5.5-2: Partial washing of concrete surface associated with rainwater runoff from a horizontal surface; with an upper washed zone and a lower soiled concrete surface, and a wavy interface line between the two zones [Hawes (1986)]

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The following suggestions for the control of weathering effects on exposed surfaces relate primarily to field observations on existing concrete structures. The design and architectural detailing of facades and exposed concrete surfaces needs to start with consideration of the siting and overall massing of the building or structure concerned. The potential wind environment around the building or structure is extremely important. The individual facades or elements of the structure need to be broken down into zones in which the conditions of exposure and dirt accumulation are likely to be reasonably consistent. It then has to be decided which parts of these zones can be kept clean and which protected from partial washing effects; taking a longer-term perspective considering the localised conditions that are expected to be created.

Hawes (1986) notes that, in addition to the general and local environment conditions, consideration should be given to a number of issues including:

- The choice of concrete surface; with rough textured surfaces having the advantage of spreading rainwater more evenly over the face and of disguising any localised staining. However, a rough surface texture will hold more dirt.
- Guiding water flow over the façade and at junctions and edges of panels and features.
- Maintaining uniform conditions; avoiding channelled rainwater flow into some areas whilst shielding others. The use of details to shed water from the building (i.e. drip details) clear of the façade and to collect water (i.e. gutters) are important, but seemingly often overlooked, items.

More specific observations made by Hawes (1986) include:

- ***Choice of concrete mix:*** The cement rich skin which occurs next to formed surfaces may be removed by a planned surface treatment at the time of construction or may be weathered away with the passage of time. These processes reveal the fine aggregate particles, with an associated change in the appearance and colour of the concrete surface from that of the cement to that of the fine aggregate. As ever the goal of the mix design is to produce a concrete with a low water-cement ratio which produces a dense well-compacted surface. Once exposed the shape, size, grading, colour and absorption characteristics of the aggregate become important, along with those of the matrix.
- ***Choice of concrete surface:*** Rough textured surfaces have the advantage of spreading rainwater more evenly over the face and of disguising any localised staining. However, a rough surface texture will hold more dirt. Surfaces with exposed aggregate tend to absorb less water than cement-rich formed surfaces. On a concrete surface with spatially varying properties, weathering will lead to a patchy appearance even if the environmental exposure is uniform. Gently sloping concrete surfaces may accumulate more 'dirt' and although water flows will be greater the velocity will typically be low. In these circumstances more water may be absorbed, depending upon the permeability of the concrete surface, and the surface may stay damp for longer thereby promoting organic growths. Generally surfaces which are not washed by rain are likely to look better with a bold surface texture/features and a darker appearance.
- ***Maintaining uniform conditions:*** A lot can be learnt from the study of the performance of the detailing on existing buildings and structures, both examples which are deemed to be performing satisfactorily and those which are not. Concentrated flows of water within unwashed areas produce clean streaks in an otherwise dark or

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soiled area, as noted before. Extended periods of dampness of the concrete substrate, such as where concentrated flows of water occur, can encourage mould and algal growths. The underlying principle for achieving uniform conditions in zones which are to be kept clean is to obtain an adequate controlled flow of water across these zones, but to ensure that this can be removed from the structure in a satisfactory manner to avoid affecting other parts of the façade. It is necessary to make an estimate of the quantity of water which may be involved and the directions from which it will come. With all of these considerations, details which work well in one location may not be suitable for another location. Conditions are most difficult to predict and control on facades parallel to the direction of the prevailing or predominant winds. Useful points to note include:

- The provision of metal sills at the top of precast concrete façade elements to protect the tops of the panels and to throw the water clear of the face of the panel.
- Where exposure conditions and, in particular, the quantity of rain vary from elevation to elevation on a building, consider the benefit of different details and/or materials to be employed on each elevation. It may be appropriate to change architectural details and/or materials employed on different elevations. It may also be appropriate to vary these factors up the height of the building.
- The effect of projecting features on a façade exposed to washing by rain need to be carefully considered as features which occur higher up on the elevation can provide protection, and hence a 'rain shadow', on those lower down on the elevation.
- On walls sheltered from direct (driving) rain, rainfall will have a noticeable cleansing effect on upward facing features, such as mouldings, sills and plinths.
- Bold architectural details are required for concrete surfaces on the same elevation if they are to have a reasonably uniform appearance when subject to different exposure conditions and partial washing effects.
- In situations where the environment is relatively clean and surfaces are washed by rain, biological colouration associated with the growth of algae and mould may be more noticeable than soiling due to the accumulation of dirt.

- ***Getting water onto concrete surfaces:*** Features which provide protection against the weather or rain influence the quantity of water received by an area of façade. These features include details such as projections, overhangs, drips, etc. Conversely factors such as the texture and porosity of the façade materials influence how much of this water then stays on the façade to create a cleansing flow. More water flow is required to wash a porous panel than an impervious one. Too little water flow will result in the formation of a streaky dirt line, as mentioned previously. Above this line the panel is washed and cleansed by the rain, below the line the panel is effectively unwashed and is subject to the accretion of dirt. For a large area of exposed concrete, extending perhaps to several storeys in height, it is preferable to break up the area with horizontal features which either collect or throw off the water at intermediate positions. Useful points to note include:

  - In the case of large and/or tall buildings, variations in airflow across the facades mean that it is difficult, perhaps impossible, to keep facades either completely clean or completely unwashed.
  - Only pyramids will be washed clean on all faces. However, generally this is not a practical geometric form for most building functions or situations.

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- Features projecting from an elevation have a more pronounced shading effect on sheltered elevations.
- As algal and mould growths can occur on surfaces which remain wet for extended periods, to avoid such growths the permeability of the concrete surface needs to be chosen with the worst exposure/wettest conditions in mind.

- ***Shedding water clear of faces:*** Cornices, copings, sills, string courses and other types of projections are architectural details used to protect the top edge of a surface or to shed water from a façade. A decision on the vertical spacing of these features depends upon a number of factors such as the exposure of the elevation and the porosity of the surface used. Generally these horizontal features are made to be continuous, which avoids the creation of streaks where the projecting features finish. Each projecting feature should have a drip groove on its underside to prevent water from flowing back onto the vertical surface. Architectural details required to control water flow and weathering effects need to be bold in nature, rather than subtle.

- ***Collecting water on facades:*** On some occasions it may be better to collect the water running down the façade in channels or gutters, rather than throwing it clear of lower parts of the surface. The collected water can be discharged via pipes, gargoyles or possibly via carefully detailed systems of vertical channels or grooves. Ideally these discharges should be located at close centres, allowing the fall along the length of the gutter to be generous. However, as solutions for collecting and removing water from facades tend to be relatively expensive, there is always the temptation to extend the spacing of the discharge points to reduce costs, which is likely to compromise performance of the façade drainage system.

- ***Guiding water on facades:*** The movement of water on facades dictates their long-term appearance. Bold architectural details are generally more effective at controlling water flow on facades. Rough textured surfaces have the advantage of spreading rainwater more evenly over the face and of disguising any localised staining. Surfaces over which water will flow are better with more pronounced surface texture/features, rather than being flat.

- ***Junctions:*** Traditionally bold architectural features have been used to disguise the change of appearance which occurs as a result of different exposure conditions being experienced on different elevations. Junctions, which form the boundary between zones of different exposure, detailing and possibly also surface characteristics, require careful consideration. As a result they are always going to present the designer with difficulties. For various reasons concentrated water flows tend to occur at external corners, whereas dirt and soiling tends to accumulate at internal corners. Similar characteristics apply to individual arrises and vertically ribbed surfaces.

- ***Edges:*** These relate to the top and bottom edges of facades. The top edge of facades and panels are usually exposed to large amounts of rain, unless they are protected by some higher feature. These surfaces also collect dirt. They can be effectively protected by a capping/flashing detail with a projecting overhang of some 40-50 mm. However, care needs to be taken with the joints between individual pieces of the capping/flashing detail, because badly made joints can lead to leakage and staining/streaky washdown appearance of the facade. Drip grooves need to be provided at the underside of each element to control and guide water flow off of the facade.

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Developments in concrete materials in recent years have included work upon topics such as self-cleaning concrete surfaces. This was presumably inspired by the commercial availability of products such as self-cleaning glass. This patented technology uses a titanium dioxide based coating applied to the surface of the glass which combines photocatalytic and hydrophilic properties to loosen the bond between dirt and the glass, allowing organic dirt to be removed by rainwater. It has been postulated that concrete with a self-cleaning hydrophobic surface would have advantages in preserving the aesthetics of the built environment and of increasing the durability of concrete elements. Improvements in durability would be expected to have significant implications for reducing the maintenance costs and the cost of ownership.

For example, a white cement containing titanium dioxide (TiO_2) has been produced commercially in recent years by the Italcementi Group [Italcementi (2009)]. As titanium dioxide is white in colour, it helps preserve the whiteness of the original concrete finish. The titanium dioxide acts as a photocatalytic agent which, in the presence of air and light, oxidises organic and inorganic airborne substances deposited on the concrete surface. It is reported that experiments were undertaken to observe the performance of white cement concrete specimens naturally aged in the open atmosphere. This work was reportedly supported by accelerated aging tests upon laboratory samples examining the effect of various types of pollutants upon the performance of the white cement concrete. It has been reported that this material was used for the shell-shaped surfaces forming the roof of the Jubilee church in Rome.

The idea of combining titanium dioxide with cement in order to produce photocatalytic concrete or active coatings for concrete was explored by the European PICADA project, which ended in 2005 [PICADA (2009)]. It was reported that a cement binder containing 5% of active titanium dioxide produced a concrete with a photocatalytic surface.

Bartos (2009) has reviewed the above and associated developments, reporting positive results in respect of the appearance of the buildings utilising the technology and for the environment from the oxidation/breaking-up of atmospheric pollutants such as NO_x, SO_x, CO and NH_3 and various other volatile organic compounds such as benzene, toluene, acetaldehyde, formaldehyde and organic chlorides [Cassar (2008)]. Bartos (2009) notes that titanium dioxide in the form of 'anatase' is one form which has the photocatalytic capability, with the oxidising reactions occurring in response to the UV rays present in sunlight. The longest period of performance monitoring of photocatalytic concrete has been on the building of the Cité de la Musique et des Beaux Arts in the centre of the city of Chambéry in France. The multi-storey in-situ concrete structure was completed in 2000, with the exposed external concrete surfaces being monitored periodically ever since. Bartos (2009) reported that the quality and appearance of the concrete surfaces were virtually unchanged when he visited the building in 2008, after eight years in service. Indeed he suggested that the concrete structure looked as though it was only recently built.

However, the findings of some less successful early experimental work undertaken by the Centre Scientifique et Technique de la Construction [CSTC (1977)] should perhaps also be borne in mind. This involved various commercially available proprietary waterproofing products exposed on a building to a heavily polluted city environment over a two year period. The trials also employed control specimen panels and the introduction of metal sill on the top edge of the concrete panels, to throw the water from the top of the panel clear of the face of the panel. The study found that the addition of a metal sill on the top edge of the concrete panels produced a far more acceptable outcome than any of the applied waterproofing products used on their own. It was concluded at that time by Hawes (1986) that architectural detailing of the facade was more effective than surface treatments in ensuring a satisfactory appearance.

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Clearly all the influencing factors discussed above still need to be considered when making decisions about the detailing and materials to be employed on a façade in order to get a satisfactory long-term appearance. On this basis it may be unwise to rely entirely on surface treatments alone. However, perhaps they should be considered as one component of the range of design options available.

The CIRIA and Concrete Society (2008) publication '*Guide to the design of concrete structures in the Arabian peninsula*' also gives guidance on the use of exposed concrete surface finishes and section profiles, suggesting those which may entrap contaminated atmospheric dust should be avoided.

Guidance on issues relating to the geometrical form and architectural detailing of facades has also been provided in Chapters 10 and 11 of the *CEB Design guide for durable concrete structures* [CEB (1992)]. This gives consideration to both dusty and wet environments. The observations made include:

- The benefits of effective drainage of water from concrete surfaces and avoiding discharging water, either intentionally or accidentally from leaking joints or pipes, which may contain deleterious substances (e.g. chlorides) onto concrete surfaces. Excessive water flow across concrete surfaces may cause leaching of the concrete.
- Avoiding standing water, by ensuring adequate falls exist on horizontal concrete surfaces.
- Water should be drained away from critical locations such as joints and seals between units.
- Avoiding splashing of concrete surfaces by water contain deleterious substances (e.g. chlorides), such as bridge piers or parapets, or by providing them with protection against splashing.

Rostam [*fib* Bulletin 3 (1999)] indicates that the geometrical form and architectural detailing of a façade might also be considered in terms of its durability risk factor; so that a design with an inherently higher durability risk could be differentiated from one with a lower risk of deterioration. He suggested that the façade illustrated in Fig. 5.5-3 has high durability risk factor. The building design was based on precast concrete elements the ends of which are externally exposed. Whilst this solution creates large column free areas within the building, it does result in large number of horizontal concrete surfaces which collect dust and dirt, especially in a dusty desert environment. Horizontal surfaces also remain wet for long periods after rainfall, further encouraging the accumulation of dirt and other forms of surface soiling. The architectural detailing of the façade also includes a large number of joints and connections, which are also likely to increase susceptibility to the ingress of deleterious substances.

Another architectural facade design is pictured in Fig. 5.5-4. This is a very smooth surface with no window sills to create rain shadows on the façade. However, this style of architectural detailing of a façade would be expected to cause uncontrolled water run-off patterns. As noted previously, partial washing of parts or all of a façade can lead to a streaked appearance, potentially forming a conspicuous wavy dirt line at the bottom of the washed zone (see Fig. 5.5-2) which is aesthetically unappealing and is an unacceptable long-term appearance.

However, from a durability perspective the drainage of rain from the window areas and the remainder of the façade would be very efficient, allowing quick drying of the concrete surface after rainfall. Rostam suggests that the durability risk factor for this style of facade would be low, based upon the 'simple and clean' geometric form employed.

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Cyclic wetting and drying effects will strongly accelerate both the rate at which deleterious substances dissolved in rainwater can enter the concrete and may be concentrated near the surface by evaporation. In these cases the selection of rounded corners and edges will reduce the concentration effects and thus enhance the durability of the structure.

Near external edges and corners aggressive substances can penetrate into the concrete from more than one side, thus leading to more aggressive local concentrations. In an environment promoting deterioration, this concentration effect could lead to early development of damage at the external corners and along the edges. A so-called 'corner-effect' can develop.

Fig. 5.5-3: A building facade with a high durability risk [Rostam: fib Bulletin 3 (1999)]

Fig. 5.5-4: A building facade with a low durability risk [Rostam: fib Bulletin 3 (1999)]

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5.5.3 Cement type, mix design and concrete quality

Cement is the primary active component in a concrete mix. In most instances this will be Portland cement. The properties of the cement paste, which has a critical role in determining the durability achieved by the concrete mix, can be modified both physically and chemically by the use of different cement types, pozzolanic materials and mineral additions. These materials include:

- pulverised fuel ash (pfa) and fly ash;
- ground granulated blastfurnace slag (ggbs);
- silica fume (sf) or microsilica;
- metakaolin (mk).

The effects associated with the use of these cementitious materials include:

- Modification of the pore structure; increasing the penetration resistance of the concrete by reducing the average pore size, increasing in the proportion of discontinuous pores, and reducing micro-cracking. These changes reduce the permeability of the concrete.
- Modification of the chemistry of the system such that materials are formed which have a higher capacity for binding aggressive species such as chloride ions. This is beneficial as it slows down the rate of penetration of chloride ions into the concrete, thereby delaying the onset of reinforcement corrosion.

Pfa, ggbs and sf can also have beneficial affects upon the properties of fresh concrete; for example, potentially enhancing the stability of the fresh concrete, reducing its tendency to segregate, reducing its viscosity and improving its pumpability.

Section 5.11.2 reviews the selection of cementitious materials as part of the process of considering the available measures to enhance resistance or avoid reinforcement corrosion, which can be avoided if the cover concrete creates an effective barrier between the environment and the embedded steel reinforcement.

Section 5.11.3 reviews various materials have been developed which can be added to concrete to improve the fresh, hardening and hardened properties of the concrete. These materials can enhance the durability of concrete by improving its resistance to the ingress of aggressive species. The following classes of admixture are discussed in Section 5.11.3:

- plasticising admixtures (water-reducing admixtures);
- superplasticising admixtures (high-range water-reducing admixtures);
- integral (hydrophobic) water repellents;
- pore-blocking admixtures.

Typically when the concrete mix is being designed, consideration will be given to whether a pure Portland cement based mix is to be used or one containing some of the different cement types, pozzolanic materials and mineral additions noted above.

The water-cement ratio (w/c ratio) or alternatively the water-binder-ratio (w/b ratio) is the most important parameter to control in practice as it is the dominating factor controlling the permeability and diffusivity of the concrete. The permeability and diffusivity of concrete re-

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duces dramatically as the water-cement ratio/the water-binder-ratio is reduced. Maximum w/c ratio / w/b ratio values as low as 0.35 have been specified. However, such low values can also reduce the workability of the concrete; depending upon the nature and effectiveness of the water-reducing admixtures introduced into the concrete mix. Such low w/c ratio / w/b ratio values may increase the sensitivity of the concrete mix to workmanship issues and increase the risks of plastic shrinkage cracking, as well as increasing the amount and proportion of autogenous shrinkage (otherwise known as chemical shrinkage or self-desiccation), which increases the likelihood of early age cracking occurring. Fig. 5.15-3 (see Section 5.15.5) illustrates diagrammatically some of the 'trade-offs' which have to be considered when choosing the target w/c ratio / w/b ratio value.

In highly aggressive environments it is normally satisfactory to employ a water-cement ratio/water-binder ratio with a maximum value between 0.40 and 0.45 – refer Table 5.7-7 (see Section 5.7.4).

The concrete mix requirements given in codes and standards are usually minimum values which are generally suitable for the vast majority of cases encountered in practice. Accordingly it may be appropriate for the designer to consider the requirement for additional measures when faced with a particularly aggressive environment, a complex or sensitive structure or the requirement for a long service life, extending beyond the service life assumed in the codes and standards. Section 5.11 gives guidance upon a variety of measures which may be adopted to enhance resistance or to avoid reinforcement corrosion.

In addition to the above, general guidance given to ensure a high quality concrete with good long-term resistance against the ingress of deleterious substances and with a slow rate of degradation once the initiation period has passed typically recommend a range of ancillary requirements along the lines of the following:

- Aggregates shall be classed as non-reactive, on the basis of approved testing procedures, and used in conjunction with low-alkali or blended cements in order to further reduce the likelihood of AAR/ASR occurring in the finished structure.
- Aggregates for reinforced or prestressed concrete dredged from the sea or taken from a source contaminated with chloride are to be washed in fresh water so that the level of chloride contamination of the concrete mix is reduced to acceptably low levels, determined on the basis of approved testing procedures.
- No chloride containing additives (e.g. as accelerators) are to be used in the concrete.
- Water containing excessive amounts of chloride, sulfates or deleterious species (i.e. from the sea, ground or brackish water sources) will not be used as mixing water nor as curing water on young concrete. The acceptability of the chloride, sulfate and other mineral content of the water is to be determined on the basis of approved testing procedures.

The importance of the curing of concrete should not be overlooked. The implications of effective curing upon absorption were evaluated in BRE Digest 491 [Sergi and Dunster (2004)]. This noted that various field investigations have supported the observation that absorption can be an important ingress mechanism in concrete structures Novokshchenov (1991), ACI (1995), Schjolberg (1993) and Bashenini et al. (1994)]. BRE Digest 491 [Sergi and Dunster (2004)] also reported that various studies [Aldred (1987), Parrott (1992 and 1994) and Dass et al. (1994)] had showed that absorption was related to the depth of the drying zone and that the initial curing period of the concrete is significant. It was reported that a

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28-day curing period and 1-day curing resulted in 75% and 135% of the water absorption rates found for 3-day curing, respectively; regardless of cement type.

5.5.4 Reinforcement type

In most instances standard design and the execution practices for concrete structures will provide adequate protection to normal and prestressing (carbon steel) reinforcement to avoid premature corrosion and structural deterioration. However in severe environmental conditions standard design and the execution practices may not provide adequate durability.

Section 5.11 gives guidance upon a variety of measures which may be adopted to enhance resistance or to avoid reinforcement corrosion. In particular Section 5.11.5 reviews ways in which the surface of the reinforcing steel can be modified by applying a chemically resistant coating to act as a barrier, by covering the reinforcing bars with a sacrificial coating so that the steel is essentially cathodically protected, or by alloying to make the material electrochemically more noble. Galvanised (zinc coated) reinforcement and epoxy-coated reinforcement are examples of steels treated in this way.

Section 5.11.5 also reviews ways in which the composition or the structure of the reinforcing steel can be modified to provide enhanced tolerance to carbonation and chlorides, such as by the use of stainless steel clad carbon steel reinforcement or by the use of micro-composite multi-structural formable (MMFX) reinforcing steel.

Section 5.11.6 reviews other ways in which reinforcement corrosion can be avoided; these include the design-out of those elements which experience in-service deterioration. The available options include replacing the carbon steel reinforcement with an austenitic stainless steel or a non-metallic reinforcement.

Investigations have also shown that localised defects such as honeycombing, leakage through cracks and joints and unintentionally small cover depths to reinforcement may lead to local deterioration in otherwise well performing structures.

Section 5.14 discusses construction quality issues, looking at the role of the project execution specification. This explains that by "thinking construction" long before the construction process actually starts on site, designers develop a construction-focused approach which potentially leads not only to a reduction in construction problems and costs, but that it helps increase the certainty of that the desired outcomes will be achieved. Furthermore, it also helps to reduced future preventive or remedial intervention costs over the life of the structure by ‘getting it right first time’.

Section 5.15 discusses ways of improving durability, outlining the benefits to be gained from pre-construction planning and trials. This examines how early investment made in pre-construction planning helps to ensure effective construction; thereby contributing to achieving the desired level of through-life performance of the structure, whilst minimising the whole-life cost of ownership of the asset.

5.5.5 Concrete cover

Jones et al. [BCA (1997)] indicated that 50% of reported cases of concrete deterioration were associated with reinforcement corrosion due to either chlorides or carbonation. The results of this survey are summarised in Fig. 5.4-1. They also attempted to identify the root causes of the deterioration and their analysis is shown in Fig. 5.4-2. This identified low achieved cover depth as a significant contributing factor. One of the examples of the effects of

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deterioration presented in Fig. 5.4-4B shows the effect of carbonation induced corrosion occurring on a concrete bridge soffit where there is inadequate cover to the reinforcement.

Studies reported in 1997 by Clark et al. (1997) of concrete structures found that the specified depth of cover to reinforcing bars was not achieved for a significant number of structural elements on the 25 construction sites investigated in the UK. It was judged that responsibility for the problems were about equally shared between management (including designers and suppliers) and operatives. Thus only about half the defects were attributable to poor workmanship by operatives. The study concluded that the problem of lack of concrete cover was extensive and chronic. Perhaps most worryingly was the suggestion that only about 50% of the instances would be detected by standard quality assurance and checking systems.

Thus failure to achieve the specified depth of cover is generally taken as being the greatest single factor influencing the premature corrosion of conventional reinforcement. In environments containing chlorides, durability problems may also be due to the specification of an inadequate cover depth. As the protection offered by concrete to embedded reinforcement is broadly related to the square of the depth of cover, durability and service life are highly sensitive to cover depth. Thus measures taken to consistently achieve an appropriate depth of cover for the service environment may secure more benefit than effort expended in the pursuit of other control parameters. These matters are discussed in Sections 5.14 and 5.15 as noted above.

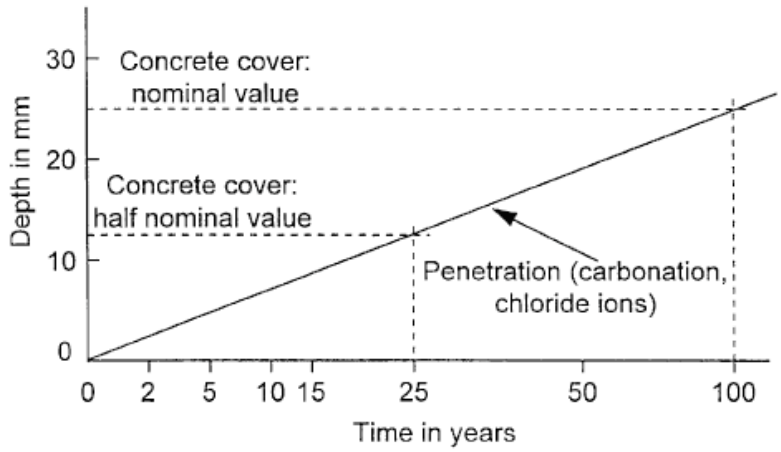

Fig. 5.5-5: Time to corrosion initiation due to carbonation or chloride penetration depending on the actual concrete cover as achieved in the finished structure. The example corresponds to high quality uncracked concrete. [Concrete Society 109 (1996)]

The influence of cover depth on durability and service life are illustrated in Fig. 5.5-5. This shows the influence of concrete cover depth upon the time to corrosion initiation due to carbonation or chloride penetration, depending on the actual concrete cover as achieved in the finished structure relative to the cover depth specified. Halving the cover depth would reduce the time to corrosion initiation by a factor of four, from the intended 100 years to 25 years as shown in Fig. 5.5-5.

It is perhaps interesting to note that the specified tolerances for cover depth do not seem to be based upon the measured cover and therefore the variations in the cover achieved in practice, but rather upon the expectations of the various committees (American, British, Australian, European, etc) responsible for the preparation of the relevant codes and standards [Morgan et al. (1982)].

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Typically the nominal cover depth value, c_{nom}, is equal to the minimum cover depth value, c_{min}, plus the tolerance according to the following rule:

$$c_{nom} = c_{min} + \text{tolerance} \tag{5.5-1}$$

Tolerance should generally be taken as 10 mm for site work, which requires a good standard of workmanship to be achieved, unless in the individual case it can be demonstrated that a lower value can be reliably obtained in practice. Tolerances for precast elements manufactured in a more controlled factory environment may be reduced, perhaps to 5 mm or less. Generally the tolerance value should not be less than 5 mm. The requirements for reinforcement spacers are discussed in Section 5.13.10.

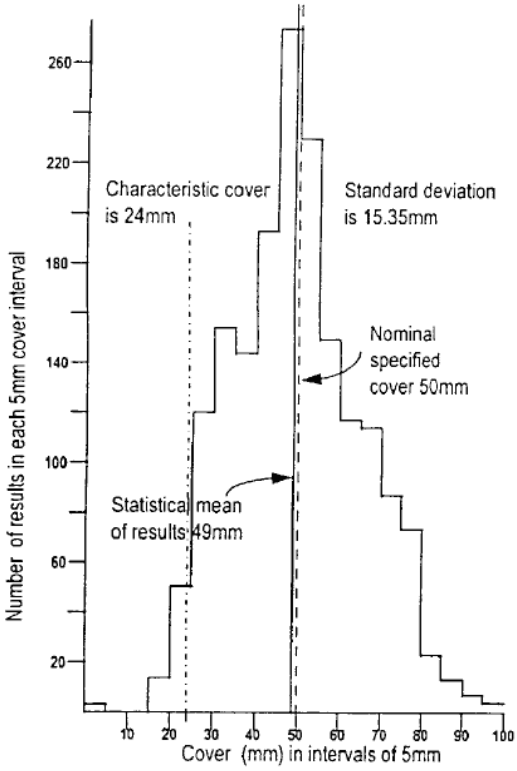

Fig. 5.5-6: Distribution of cover – Analysis of 1600 covermeter readings for a 13 m high retaining wall [Concrete Society 109 (1996)]

Fig. 5.5-6 presents measured values for the distribution of achieved cover depth; it represents an analysis of 1600 covermeter readings on a 13 m high retaining wall. Whilst there may well be some differences between the cover depth values estimated using a covermeter and the actual cover depths, this variable will be assumed to be small for this example. In any actual site survey the degree of uncertainty and error in the covermeter readings can be evaluated by a 'truthing' exercise, where the covermeter estimates are compared with directly measured values achieved by locally exposing a proportion of the reinforcing bars surveyed. In the example presented in Fig. 5.5-6 many of the actual cover values are considerably below the 50 mm nominal cover value specified by the designer. In this case the structure was designed to BS 8110 [BSI (1985)], a previous UK code for the design of concrete structures, which specified a 5 mm tolerance value. Many of the actual cover values in Fig. 5.5-6 are less than minus 5 mm tolerance value, i.e. a cover depth value of 45 mm.

Fig. 5.5-6 also shows that in this case the variation in achieved cover depth generally followed a Gaussian probability distribution, with the mean value (49 mm) being close to the specified 50 mm nominal cover value.

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The required value of cover will depend upon requirements relating to a number of factors including maximum aggregate size, bond, surface crack width, fire resistance and durability/service life considerations. These requirements are checked using the following parameters:

- Maximum aggregate size: characteristic minimum cover depth
- Bond: characteristic minimum cover depth
- Surface crack width: nominal cover depth
- Fire resistance: characteristic minimum cover depth
- Durability/service life: characteristic minimum cover depth
- Structural load capacity design, etc: nominal cover depth

The characteristic minimum cover depth is the value below which 5% of the population of all possible measurements of cover depth are expected to occur.

Where there are local reductions in concrete cover depth, this may result in early corrosion of the reinforcement at these locations.

In addition to the depth of cover achieved, there may be issues associated with the quality of the concrete in the cover zone – referred to elsewhere as the 'covercrete' (see Fig. 5.1-7). The surface layer of concrete, say the outer 5 to 10 mm, typically has a higher permeability caused by an increase in the cement paste content and a locally increased water-cement ratio at surfaces cast against impermeable formwork. Similarly, trowelled or floated horizontal surfaces may be more porous than the bulk of the concrete due to the trowelling or the floating operation, and/or due to bleeding and settlement of the coarse aggregate.

In the case of industrial floors extensive trowelling and powerfloating operations are commonly carried out to reduce the permeability of the surface by 'densifying' it. The surface treatments employed to improve the durability of industrial floors include the use of dry shake finishes. Concrete Society Technical Report TR34 entitled '*Concrete industrial ground floor – A guide to their design and construction*' [Concrete Society (2003)] provides further details of these and other surface treatments for industrial floors.

As part of the process of considering the available measures generally used to enhance resistance or avoid reinforcement corrosion, Section 5.11.4 reviews measures which can be applied to the surface zone of concrete to improve its resistance to the ingress of carbon dioxide, chlorides and other environmental agents, including moisture.

Of course, if the design process has selected a form of reinforcement that is corrosion resistant/notionally does not corrode, the cover depth requirements for durability purposes will no longer be critical. However, there are other factors which are influenced by the depth of cover, as noted above, which is why it is commonly recommended that the nominal depth of cover is not reduced.

Summarising the permeability of the concrete cover, and hence the durability of the concrete structure, is influenced by a number of factors including:

- *The water-cement ratio/water-binder ratio:* The lower the water-cement-ratio, the lower the permeability of the cover concrete and the better the durability of the concrete structure; assuming that adequate compaction can be achieved, particularly around reinforcement.

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- *Concrete mix constituents:* This concerns not only the cement paste, but also the characteristics of the aggregate and other constituents used. However, the properties of the cement paste are generally considered to have the critical influence upon the durability the concrete mix. The properties of the cement paste are changed by the type of cement, pozzolanic materials, mineral additions and admixtures used. Mixes containing materials such as pulverised fuel ash, ground granulated blastfurnace slag, silica fume, metakaolin and superplasticising admixtures typically have a reduced permeability and/or a higher capacity for binding aggressive species such as chloride ions, thereby delaying the onset of reinforcement corrosion compared with similar concretes based on Portland cement alone. However, such 'blended' concrete mixes may also have a reduced resistance to carbonation.

- *Compaction of the concrete:* Compaction seeks to produce a concrete with no-voids and a minimum of accidentally retained air content (generally less than 1% unintended air voids, excluding air bubbles associated with air entrainment). This is critical for the surface 'covercrete' zone where the presence of honeycombing will locally greatly increase the permeability of concrete, to such an extent that the protection of reinforcement may be effectively reduced to zero. In such cases corrosion of reinforcement may commence immediately, depending upon the aggressivity of the service environment.

- *The quality of curing:* Curing seeks to control the availability of moisture for continued hydration, the development of strength and the control of temperature regimes and thermal gradients; particularly of the surface layer of concrete after hardening. This determines the quality of the surface layer of concrete and influences the occurrence of associated effects such as plastic shrinkage cracking and early age thermal cracking.

5.5.6 Cracking, crack width and crack orientation

Reinforced concrete elements subject to loads and actions are expected to exhibit some degree of cracking. Cracks may also arise from other causes, such as from early age thermal effects and plastic shrinkage. A cracked concrete structure can perform satisfactorily. It can be serviceable, have a satisfactory visual appearance and be adequately durable for its service environment; provided that the width and frequency of the cracks are within acceptable limits.

It is intended that the width of cracks associated with loading effects, typically flexural cracks, should be controlled by the design process. Similarly it is intended that some types of cracks which arise during construction, such as early age thermal cracks, should be controlled by the design and specification processes.

Other potential cracks associated with construction, such as plastic shrinkage and settlement cracks, should be controlled by pre-planned construction procedures – such as instigating effective curing, use of membranes, controlling thermal effects and limiting the temperatures reached during hydration of the concrete, etc. Other considerations may include the addition of a small amount of polypropylene fibres, at a dosage rate of say 0.5 kg/m^3, to the concrete mix to reduce the risk of plastic shrinkage cracking. In some instances it might be appropriate to use steel fibre reinforcement to control thermal cracking, but other measures could be more cost effective, e.g. use of an appropriate choice of cement, concrete mix, etc.

An important factor in controlling the width of cracks is to ensure that the reinforcement does not yield. This means that the strength of the reinforcement provided must exceed the tensile strength of the concrete at the time of cracking. It should be recognised that if the contractor uses a higher strength concrete than that specified in the design, for example to permit early striking of formwork or stressing of post-tensioning, this may increase the steel provi-

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sion required. In some situations it might be appropriate to consider limiting the maximum strength of the concrete to ensure that cracking is controlled.

Acceptability of crack widths in 'normal' structures – Code recommendations: Design procedures and codes seek to limit the width of cracks, depending upon the exposure class the structure is designed for. Smaller (narrower) crack widths are specified for more aggressive environments or demanding circumstances. For example, the crack widths recommended in Eurocode 2 "*Design of concrete structures*" [EN 1992-1-1: CEN (2004)] for 'normal' structures are as follows:

- For reinforced elements and for prestressed elements with unbonded tendons, a maximum crack width of 0.3 mm under the quasi-permanent load combination is recommended.
- For prestressed elements with bonded tendons, a maximum crack width of 0.2 mm under the frequent load combination is recommended. The smaller crack width is recommended because prestressing tendons are more likely to be adversely affected by corrosion than reinforcing bars, which typically have a larger cross-sectional area and are therefore more tolerant of corrosion damage (induced loss of cross-section).

Code design formulae are intended to result in crack widths which have an acceptably small chance of exceeding the recommended crack width value. Accordingly, the occasional wider crack is to be expected. Furthermore, crack formation is a fairly random process because of the inherent variability which occurs in material and circumstances, which means that an absolute maximum crack width cannot be predicted. Clearly crack width can only be assessed visually at the surface of the concrete.

The *fib* Model Code for Service Life Design (MC-SLD) [*fib* (2006)] gives the following observations upon the influence of cracks upon the corrosion of reinforcement. The *fib* MC-SLD suggests that corrosion rates in the region where cracks cross reinforcement are extremely dependent on the micro-climatic conditions at the concrete surface and the orientation of the concrete surface. It indicates that the most severe exposure conditions occur for upward-facing horizontal concrete surfaces, where upper surface cracks allow chloride ingress. Where service lives of more than 10 years are required and frequent chloride exposure occurs (e.g. parking decks in regions where de-icing salts are used), special protective measures are necessary to avoid the rapid penetration of chlorides to the reinforcement (e.g. crack-bridging coatings).

In case of vertical surfaces and soffits subject to chloride spray from the underside, but with the stipulation that chloride-rich water does not leak through cracks from above, the *fib* MC-SLD indicates that high quality concrete cover should be sufficient to ensure a long service life (over 50 years) without extra protection measures being necessary. In this case high quality concrete cover is indicated to be achieved by using a low permeability concrete with a water-cement ratio of less than 0.5, a cover thickness greater than 50 mm and a (ordinary) crack width limit of 0.3 mm.

In case of carbonation induced corrosion of reinforcement, the *fib* MC-SLD indicates that adequate quality of concrete cover and ordinary crack width limitation should ensure a long service life (over 50 years) without extra protection measures being necessary.

There may be situations where it is desirable to increase the cover depth to more than the code recommended value. As the width of the crack at the surface is related to the depth of cover to the reinforcement, an increase in the cover will increase the surface crack width.

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However, durability considerations relate to the width of a crack at the level of the reinforcement, which cannot be easily assessed. Thus in a situation where durability is the governing criterion, it may be appropriate to estimate the crack width by calculation using the code recommended cover depth (rather than for the actual cover). However, it may also be necessary to check that the crack width at the surface will be acceptable aesthetically.

Campbell-Allen (1979) investigated the relationship between the acceptability of crack width and the distance from the viewer for nine categories of structure ranging from 'little used buildings' to 'monumental buildings'. His conclusions were adopted by the Concrete Society in their Technical Report 22: *Non-structural cracks in concrete* and again more recently in Technical Report 67: *Movement, restraint and cracking in concrete structures.* Campbell-Allen's findings are reproduced here as Fig. 5.5-7.

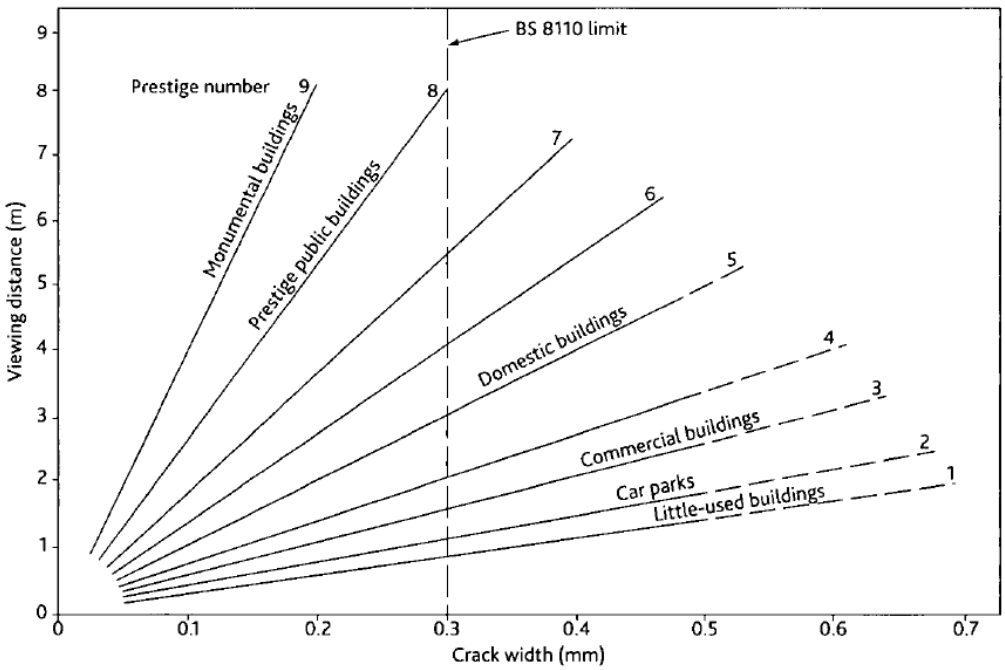

Fig. 5.5-7: Aesthetically acceptable crack widths [Concrete Society TR 67 (2008) adapted from Campbell-Allen (1979)]. Reproduced with permission.

Thus the 0.3 mm crack width limit may be applied to achieve acceptable appearance. Although this criterion is quite subjective, it is generally accepted that under normal lighting and associated conditions a 0.3 mm crack will be visible to the naked eye at a distance of about 3 m. Thus if the viewing distance and circumstances are significantly different, an adjustment may need to be made to this crack width criterion, depending upon the expectations of the 'quality' of the finished surface.

It should also be recognised that cracks are more visible (for a short period of time) on surfaces which are drying after being wet, as the cracks retain moisture for longer. The 'visibility' of cracks can also be greater when they are contaminated with dirt. If the viewing distance is significantly greater than 3 m, on the basis of aesthetic considerations only, it may be possible to tolerate wider cracks depending upon circumstances.

Influence of cracking: The effect of cracking on carbonation and on the corrosion of embedded steel appears to depend upon circumstance. There is a range of evidence and opinion as to what the effects are in different situations and about their significance.

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In an environment where the concrete is experiencing only carbonation, cracking will often initiate localised early corrosion, but typically the corrosion does not then progress significantly (see Fig. 5.4-22). Greater depths of cover to reinforcement appear to help inhibit the further development of this form of localised corrosion.

Bamforth, Price and Emerson (1997) and, more recently, Otieno, Alexander and Beushausen (2009) have reviewed the research undertaken on the influence of cracking on corrosion initiation and its propagation. Although the body of research is quite extensive, there are significant differences in the findings; some of which are potentially conflicting. Otieno, Alexander and Beushausen (2009) suggest that opinions on these matters can be grouped into two fairly broad camps, as follows.

A. Those researchers who indicate that there is no direct relationship between crack width and corrosion rate. This group suggest that cracks may accelerate corrosion initiation, but not its subsequent propagation. The factors they consider to be important include crack properties (e.g. whether the crack is active or dormant) and exposure conditions (i.e. the nature of the service environment).

B. Other researchers indicate that cracks accelerate both corrosion initiation and propagation, but these are dependent on the concrete quality (water-cement ratio and the type of binder employed) and cover depth.

   For example, Houston et al. (1972) concluded that although flexural cracking of concrete was found to promote corrosion of reinforcement at the crack location, the severity of the long-term corrosion damage to the bars was primarily dependent on the depth of concrete cover. They also noted that wide cracks, usually found in conjunction with a large cover depth, promoted early corrosion at the crack locations, but that the further development of the corrosion and longitudinal cracking of the cover concrete was inhibited for the larger covers.

   Arya and Ofori-Darko (1996) have reported that crack width is less important to corrosion risk than crack frequency, cover depth and concrete quality. Figs. 5.5-8 and 5.5-9 illustrate the influence of crack frequency upon cumulative corrosion loss as found by their research work.

   Ohno et al. (1996) also reported reduced corrosion with increasing cover. The studied the effects of crack width and water-cement ratio and concluded that the most significant factor was the water-cement ratio.

Otieno, Alexander and Beushausen (2009) note that some of the factors which appear to have an influence upon corrosion initiation and its propagation in cracked concrete include:

- The crack width, although there is a lack of clarity in some of the research work as to whether the findings refer to the surface crack width or to the crack width at the level of the reinforcing steel.
- Crack spacing, which is variously referred to as crack density and crack frequency. There appears to be uncertainty as to whether this factor is more important than crack width.
- The depth of the crack and the fundamental question of whether the crack reaches the reinforcing steel.
- The shape of the crack and whether the crack is wedge-shaped or parallel-sided.

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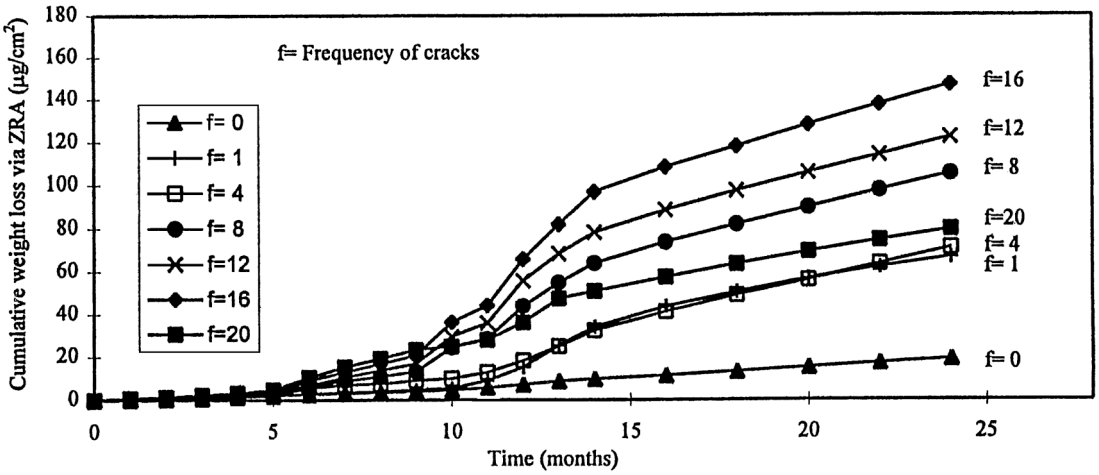

Fig. 5.5-8: Effect of crack frequency on cumulative weight loss due to corrosion – As estimated using galvanic current (ZRA) measurements [Arya and Ofori-Darko (1996)]

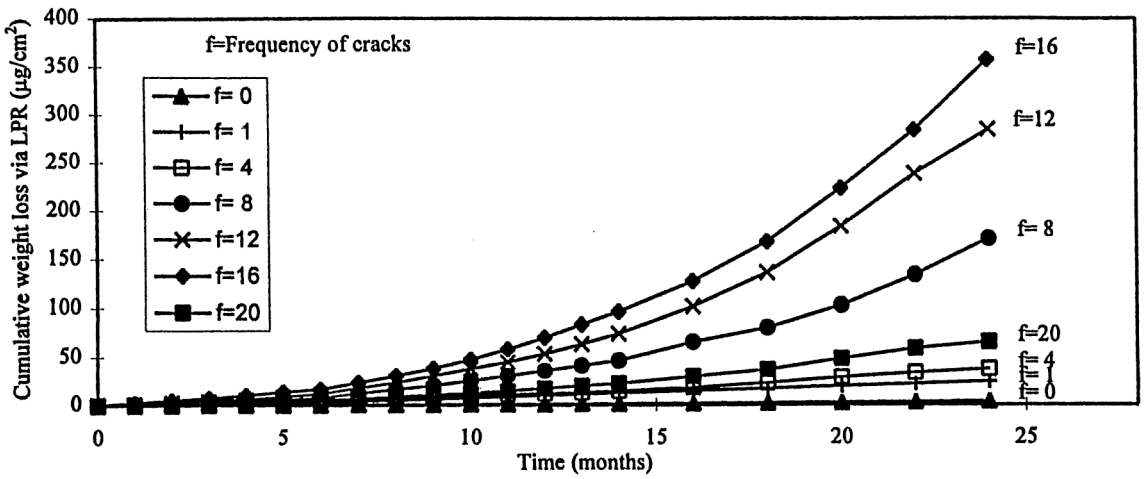

Fig. 5.5-9: Effect of crack frequency on cumulative weight loss due to corrosion – By linear polarisation resistance (LPR) measurement [Arya and Ofori-Darko (1996)]

Note to Figs. 5.5-8 and -9: *The weight loss estimated by LPR measurements (Fig. 5.5-9) is significantly greater than those estimated using galvanic current measurements (Fig. 5.5-8). Arya and Ofori-Darko (1996) suggest that this could be due to the fact that the LPR technique measures the total corrosion activity, whereas the galvanic technique measures just the current flowing between the electrodes.*

- The orientation of the crack orientation, that is whether it is coincident with the reinforcement (aligned with the longitudinal axis of the reinforcing steel) or intersects the reinforcement transverse – see Fig. 5.5-10.
- Whether the cracks will self-heal (i.e. autogenous healing), what is the maximum width of crack that can undergo self-healing and the period of time that this may take.
- Whether the crack is active or dormant.

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The different results obtained by various researchers mainly arise from the varied combinations of the above parameters studied and employed in laboratory experimental set-ups. As a result it is very difficult to compare the outcomes of the different studies and this can only be done to a very limited extent.

Studies undertaken at the University of Cape Town in South Africa [Scott (2004), Scott. and Alexander (2007), Otieno, Alexander and Beushausen (2009) and Otieno, Alexander and Beushausen (2009)] have revealed the following with respect to the influence of cracking on corrosion:

- Corrosion initiation is almost instantaneous in the presence of cracks, especially for reinforced concrete elements and structures that are exposed directly to corrosion-inducing species such as chlorides. Available service life models may not recognise this and therefore these may tend to overestimate the service life of a structure.
- For a constant depth of cover, the corrosion rate increases with the width of the crack. Thus the wider the crack, the higher the corrosion rate. However, the effect of cracks on corrosion rate is modified by concrete quality (water-cement ratio and the type of binder employed) and this must therefore be taken into account.
- For a constant surface crack width, corrosion rate increases with decreasing cover.

Other related conclusions from the studies at the University of Cape Town are that:

- Corrosion in cracked (and uncracked) concrete can be reduced by the use of higher quality concrete. That is by the use of a lower water-cement ratio and especially by the use of pfa and ggbs in conjunction with Portland cement – this topic is discussed in Section 5.11.2.
- Concretes made using pfa and ggbs in conjunction with Portland cement are less sensitive to the effects of cracking than concretes made solely with Portland cement. The blended cement concretes have an inherently higher concrete resistivity, which typically reduces the corrosion rate.

The UK Concrete Society also investigated the relevance of cracking in concrete to the corrosion of reinforcement, reviewing the available research findings [Concrete Society Technical Report 44 (1995)]. The purpose of this work was to try and resolve if the presence of cracks significantly reduced the service life of concrete structures or whether their overall effect on the rate of corrosion was minimal. Whilst the authors concluded that cracks coincident with a reinforcing bar might reduce the service life (see below); they suggested that in the case of transverse (intersecting) cracks the rate of corrosion propagation depended upon (a) the properties of the crack, (b) the properties of the concrete and steel, and (c) the aggressivity of the service environment. No definitive conclusions appeared to be reached in the case of transverse (intersecting) cracks and an assessment procedure was suggested to determine the risk of crack-induced corrosion in existing structures.

Audenaert et al. (2009) investigated the influence of cracks on chloride penetration into laboratory concrete specimens containing artificially formed cracks 0.2 mm and 0.3 mm wide, for crack depths ranging between 5 mm and 20 mm deep. The specimens were made using two different grades of concrete. Taken in conjunction with earlier currently unpublished work, the results indicate that crack width has no influence on penetration of chloride until the crack width reduces to less than 0.1 mm. As expected, the induced cracks permitted rapid penetration of chloride into the concrete and locally increased the chloride content at the depth of the reinforcement.

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The question of a limiting or threshold crack width: Previous studies have proposed a crack width of 0.4 mm as a limiting crack width, below which corrosion would be expected to proceed in a manner similar to that which occurs in uncracked concrete. Otieno, Alexander and Beushausen (2009) report that matters are more complex than envisaged and that it is not possible to obtain a universal limiting/threshold crack width for all concrete types. They suggest that the setting of limiting crack widths needs to take into consideration a number of factors such as the concrete quality and cover depth. The situation is similar to that encountered with respect to establishing a chloride threshold level which initiates corrosion discussed in Section 5.4.7.6.

Bamforth, Price and Emerson (1997) suggest that the adoption of a crack width limit may be acceptable for static (dormant) cracks, provided their width can be maintained below the limit for self-healing (see section below concerned with the autogenous healing of cracks. They suggest that in the case of live (active) cracks, whose width varies with time, it may be more appropriate to adopt the approach proposed by Arya and Ofori-Darko (1996) of limiting the crack frequency with increasing depth of cover.

Influence of crack orientation: Cracks coincident with a reinforcing bar (Fig. 5.5-10), that is along the length of the reinforcement, will tend to expose a greater proportion of the bar to moisture and oxygen. In this situation the cathodic and anodic areas of the reinforcing bar are likely to be reasonably equal in size. Where the concrete is experiencing only carbonation, this would typically be expected to lead to generalised corrosion; with the expansive forces potentially promoting further cracking depending on the availability of moisture.

The development of cracks along the length of the reinforcement could also significantly reduce the bond between the reinforcement and the concrete. As a consequence of this, properties such as the bending and shear strength of the concrete element concerned may also be adversely affected.

In the case of cracks intersecting reinforcing bars, the anodic areas are likely to be considerably smaller than the cathodic areas (see Fig. 5.4-22). Whilst this could notionally lead to severe localised corrosion; it typically seems not to be such a problem when the concrete is only subject to the effects of carbonation.

When chloride penetrates the concrete from an external source, corrosion can be very severe where reinforcing bars intersect cracks because of the relatively small size of the anodic zones compared with the cathodic zone. In these situations intense localised (pitting) corrosion of affected reinforcing bars can cause very large losses of cross-sectional area (see Fig. 5.4-24). In extreme cases even total local loss of a bar has been known over a length of a few centimetres in the vicinity of a crack. Just outside this zone, perhaps just a centimetre or two away, the reinforcing bars concerned may be in pristine condition. Clearly the loss of a large proportion of the cross-sectional area of adjacent reinforcing bars (each intersected by the crack) could seriously impair the load carrying capacity of that part of a structure.

The risk of coincident cracking (aligned with the longitudinal axis of the reinforcing bars) is particularly high for transverse reinforcement such as stirrups in beams, especially where it has a low concrete cover.

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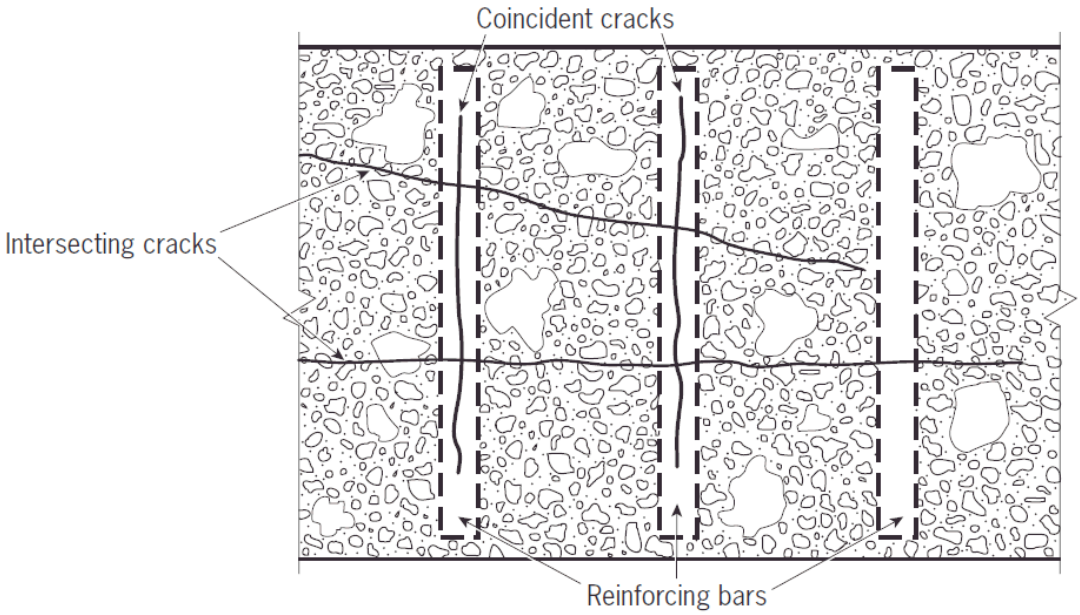

Fig. 5.5-10: Alignment of cracks relative to the longitudinal axis of reinforcing bars.

Strategies to limit cracking and crack widths: Prestressing the concrete structure may be adopted as a means of controlling and limiting the width of load- and action-induced cracks. The technique can also be used to limit the number of joints in a structure. Often the requirement to avoid "tension" in the cross-section is sufficient to ensure that cracks do not appear. However, this simple design criterion may not be adequate for more complicated cross-sections and structures like box girders where pronounced temperature gradients (e.g. from solar insolation [1]) and other effects may arise.

Perversely specifying small allowable crack widths can have adverse effects on durability. The design formulae used to calculate crack widths during design tend to lead the designer to consider:

- using many small diameter reinforcing bars with a small bar spacing;
- using the minimum cover allowed by the relevant codes and specification.

These measures may increase the difficulty of achieving an adequate depth of cover, full placement of concrete in the critical concrete cover zone (i.e. no voids) and ensuring good compaction of this concrete. In addition, smaller reinforcing bars are more susceptible than larger diameter bars to the loss of cross-sectional area due to corrosion (i.e. the loss of a given area of metal forms a larger percentage of a smaller diameter reinforcing bar).

In addition, the design of a structure may require large amount of reinforcement in sections with locally high bending moments and forces, such as at the corners of frames and above intermediate supports. Again this may well cause difficulty in achieving good quality low permeability concrete in the cover zone, which would be counterproductive to ensuring a long service life and a low through-life maintenance cost.

In these circumstances a greater depth of good quality concrete forming the cover zone, in conjunction with fewer but somewhat larger diameter reinforcing bars, should be more effec-

[1] Solar insolation is a measure of solar radiation energy received on a given surface area in a given time.

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tive at producing a durable concrete structure. Whilst both of these measures notionally contribute to increased crack widths at the concrete surface, overall they are expected to be more effective at limiting reinforcement corrosion and achieving a long service life for the concrete structure.

The survey of the in-service performance of North Sea oil installations reported by Helland et al. (2008) has shown the importance of achieving good quality concrete cover in producing durable high performance concrete structures in aggressive environments. Helland et al. (2008) note that the critical factor in achieving this was to avoid the type of problems described above during the concreting process. Thus it is not sufficient just to devise an appropriate concrete mix, but it is also necessary to consider the effectiveness of the overall process for the execution of the works on site. These issues are discussed further in Sections 5.14 and 5.15.

Autogenous healing of cracks: The formation of cracks in a concrete structure will create paths which may allow the passage of water. The extent to which they may self-heal (i.e. autogenous healing) depends on a variety of factors including the crack width, its length, the hydraulic gradient, etc. Practical experience has demonstrated that cracks often have the ability to heal themselves, e.g. the water flow reduces with time. Depending upon circumstances, cracks may seal completely with time (i.e. the water flow stops). Clear (1985) demonstrated that cracks narrower than 0.2 mm wide have the potential to self-heal even under continuous water flow.

Cracks may also be 'dormant' or 'active'. A crack is classified as 'dormant' if its width does not vary appreciably with time. Dormant cracks are often caused by physical processes (plastic or drying shrinkage, shrinkable aggregates, long term loading etc). They may self-heal through on-going cement hydration, precipitation of calcium carbonate or diffusion of calcium hydroxide from the bulk of the concrete (i.e. autogenous healing). Cracks having width changes with time are referred to as being 'live' cracks. They are typically caused by on-going processes such as chemical (alkali aggregate reaction, sulfate attack etc), structural and thermal actions. Typically 'active' cracks are less likely to heal autogenously.

The processes of autogenous crack healing and the influencing parameters have been investigated using both theoretical studies and verifying experiments, with the work being reported by Edvardsen (1996). The work was summarised in Edvardsen (1999). Fig. 5.5-11 shows the flow of water through an active crack and a dormant crack. The active crack was subjected to a daily movement cycle. The width of the dormant crack was equal to the minimum width of the active crack. The water pressure applied to both cracks was the same. The result demonstrates that autogenous healing can potentially occur in active cracks.

For water retaining structures it has traditionally been assumed that cracks 0.2 mm or less in width will generally seal themselves through the process of autogenous healing discussed above. However, more specific guidance is now available in Eurocode 2 Part 3 [EN 1992-3, CEN (2006)] for the design of liquid retaining concrete structures. This defines acceptable limiting crack widths as part of the provisions for achieving different (liquid) tightness classes, relating the limiting crack width to the pressure gradient across the section, see Fig. 5.5-12. It is stated that for the limiting crack width values, the cracks should seal within a relatively short time.

Fig. 5.5-12 implies that for crack width of less than 0.05 mm autogenous healing will occur even when there is a pressure gradient in excess of 35 (pressure gradient = pressure head, hp/wall thickness, h). However, conversely, the Fig. 5.5-12 suggests that when crack widths exceed 0.2 mm, autogenous healing will not necessarily occur. This seems to be at odds with

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the findings reported by Edvardsen (1996). However, successful experience with water retaining structures designed using BS 8007 [BSI (1987)], which sought to limit surface crack widths to 0.2 mm, would suggest that this may not be a critical issue.

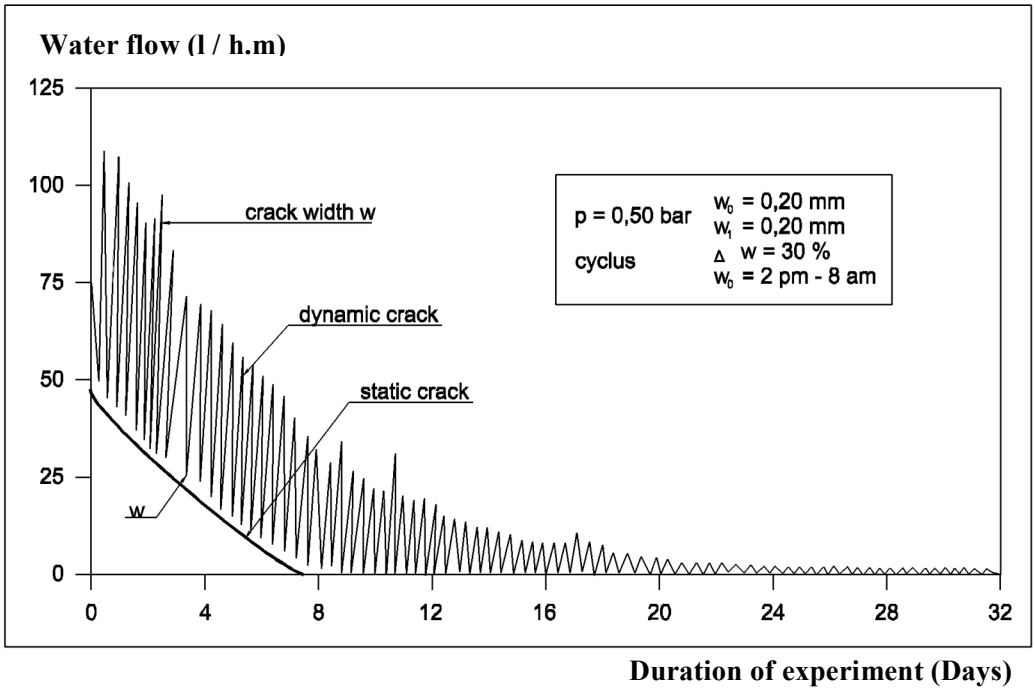

Fig. 5.5-11: Self-healing of a dormant crack and an active crack [Edvardsen (1996)]

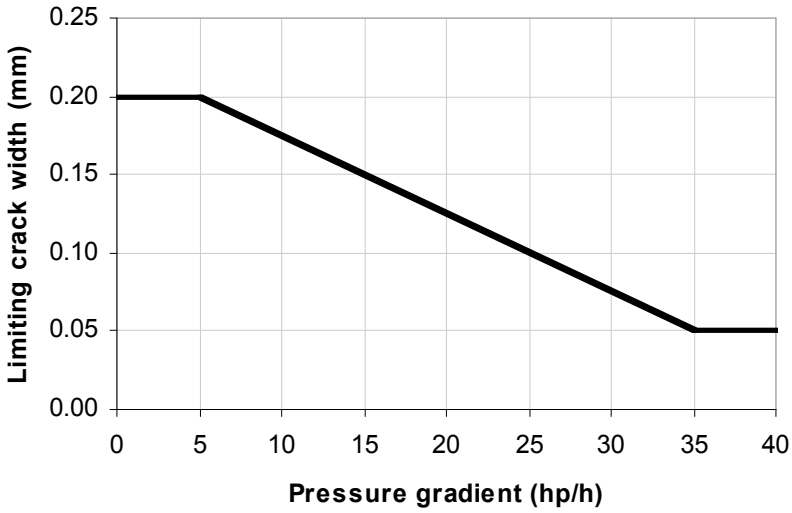

Fig. 5.5-12: Limiting crack width for self-healing of crack in a concrete wall section related to pressure gradient across the section [CIRIA Report 660 (2007)]

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5.6 Environmental aggressivity

5.6.1 Introduction

A critical aspect of designing for durability is to establish the exposure conditions for which each element of a structure shall be designed. This involves consideration of:

- *The macro-climate:* This is general environmental conditions of the area in which the structure is situated.
- *The meso-climate:* The specific location and orientation of the concrete surface being considered and its exposure to prevailing winds, rainfall, etc.
- *The micro-climate:* Localised conditions (micro-climate) such as surface ponding, exposure to surface run-off and spray, aggressive agents, regular wetting, condensation, etc. These aspects include those arising as a consequence of the interaction between the structure and the environment (e.g. cladding to keep the structure dry or ponding due to poor detailing, etc.). It is essential that the interaction between the structure and its environment is addressed when seeking to design durable structures.

Fig. 5.2-2 illustrates some possible micro-climate conditions for a bridge subjected to de-icing salts. The associated influencing factors are discussed in Section 5.2.3.

Another example is the division of a marine structure into submerged, tidal, splash and the atmospherically exposed zones. In addition, there may also be different geographic orientations (north/south/east/west or seaward/landward orientation) which may warrant different categorisations with respect to exposure class. At the next level of differentiation, consideration might, for example, be given to whether the concrete surface being considered was orientated vertically or horizontally and whether it was, say, upward facing (risk of ponding) or downward facing (protected against wetting but at risk of an increased rate of carbonation).

Clearly, it is not easy to decide on the conditions of exposure of particular elements in particular environments. However, the factors listed below are known to have a dominant influence upon the aggressivity of a particular environment. These are grouped by transport mechanism; that is via water or moisture and the atmosphere, and thermal effects.

5.6.1.1 Water or moisture driven deterioration processes

Aggressive substances which may be present in moisture or interact with water include:

(a) carbon dioxide – associated with the carbonation process;

(b) oxygen – necessary for corrosion, it supports the cathodic reaction;

(c) chlorides – promote corrosion of steel and reinforcement;

(d) acids – dissolve the cement paste;

(e) sulfates – may produce an expansive reaction with cement;

(f) alkalis – may produce an expansive reaction with aggregate.

5.6.1.2 Atmospheric induced deterioration

Aggressive acidic substances which may be present in the atmosphere include:

(a) carbon dioxide and carbonic acid – associated with carbonation and acid rain;

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(b) sulfur – attack reduces concrete surface alkalinity (pH), dissolves concrete surface.

5.6.1.3 Temperature induced effects

Aggressive effects of temperature include the following:

(a) freezing conditions – frost is a main influence determining deterioration rate;

(b) with frost – the number and severity of temperature variations influences damage;

(c) high temperature – affects speed of chemical attack/deterioration/creep effects.

5.6.2 Moisture driven deterioration processes

As all deterioration processes, apart from most types of physical deterioration, require the availability of water to driven the associated chemical reactions; it is the moisture condition in the concrete which is the decisive factor influencing the durability of the structure. Under steady state conditions moisture content within the concrete will be stable (constant). However, under varying external moisture conditions, concrete takes water up from the surrounding environment more rapidly than it loses it. As a result the internal average humidity within the concrete tends to be higher than the average ambient (atmospheric) humidity. This principle holds true where elements are subject to wetting and drying. Frequent wetting, as in tidal zones, can maintain concrete in a saturated condition.

Section 5.4.3 discusses the role of water as a transport mechanism and factor controlling various deterioration processes. Section 5.4.7.5 examines the influence of humidity upon the corrosion of steel in concrete.

Table 5.6-1A indicates the influence of the effective relative humidity on various processes related to durability as given by CEB in its publication *Durable concrete structures: Design guide* [CEB (1992)]. Table 5.6-1A provides only a broad indication of the inter-relationships, but it highlights the fact that there are no chemical or corrosion problems in very dry concrete, and that there are generally no serious problems of corrosion of steel reinforcement in water saturated concrete (Risk rating ‘1’). The main problems associated with the corrosion of steel reinforcement occur when the relative humidity inside the concrete is within the range of 75 – 98%RH (Risk ratings of ‘2’ and ‘3’).

From Table 5.6-1A it will be seen that chemical attack of the concrete and frost damage is more likely to be a problem when the concrete is wetter, and may be at high risk when the concrete is saturated (Risk rating ‘3’).

On the basis of the CEB publication “*Durable concrete structures: Design guide”* [CEB (1992)], the highest risk for carbonation of concrete is at "Low relative humidity" (45-65%RH), which commonly exists in many indoor situations. However, at a low relative humidity the risk of corrosion is only slight, so corrosion would progress at a slow rate in such circumstances. Should the relative humidity increases after the concrete has become carbonated to the depth of the embedded steel reinforcement, the risk of corrosion increases greatly. Thus in these circumstances, should the relative humidity change to "Medium" (65-85%RH) the risk of corrosion becomes high (Risk rating ‘3’).

However, there is some disagreement with other published evidence and guidance about the RH values promoting the fastest rates of carbonation and corrosion of steel embedded in concrete. For example, Sergi and Dunster (2004) report that an RH of between 60% and 80% allows the fastest rate of carbonation. If the concrete is either very dry or water-saturated, the process of carbonation practically stops. Concrete subjected to wet/dry cycling will have a

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convective zone of a significant depth which is determined by the relative lengths of wetting and drying, curing and quality of concrete (see Fig. 5.4-29). The range of RH in this zone is likely to be at the intermediate level that would allow significant carbonation to occur. Fig. 5.6 1 illustrates the rate of carbonation of concrete as a function of relative humidity.

A relationship established between corrosion rate and internal RH in carbonated concrete is presented in Fig. 5.4-30 [Sergi and Dunster (2004)]. This draws upon work by Glass et al. (1991), Parrott (1998), Bamforth (1999) and Senaviratne et al. (2000). Although significant scatter is evident, the trend suggests that at RH values of less than about 75–80%, the corrosion rate of the steel reinforcement in carbonated concrete is lower than 1–2 μm/year cross-sectional loss (0.1–0.2 μA/cm^2 of steel area). This is a rate which is accepted as being insignificant over the design life of most structures. Therefore if it is possible to keep the concrete's internal RH at values below 75–80%, the corrosion rate of steel in carbonated concrete should be acceptable small. Table 5.6-1B presents a revised indication of the influence of moisture condition upon deterioration processes.

Table 5.6-1A: General indication of the influence of moisture condition upon deterioration processes – from Durable concrete structures: Design guide [CEB (1992)]

Effective relative humidity (RH)	**Process**				
	Carbonation	**Corrosion of Steel**		**Frost Attack**	**Chemical Attack**
		In carbon-ated concrete	**In chloride contaminated concrete**		
Very low (<45%)	1	0	0	0	0
Low (45-65%)	3	1	1	0	0
Medium (65-85%)	2	3	3	0	0
High (85-98%)	1	2	3	2	1
Saturated (>98%)	0	1	1	3	3

Key: 0 = insignificant risk; 1 = slight risk; 2 = medium risk; 3 = high risk.

Table 5.6-1B: Revised general indication of the influence of moisture condition upon deterioration processes – adapted from Durable concrete structures: Design guide [CEB (1992)] using BRE Digest 491: Corrosion of steel in concrete: a review of the effect of humidity [Sergi and Dunster (2004)].

Effective relative humidity (RH)	**Process**				
	Carbonation	**Corrosion of Steel**		**Frost Attack**	**Chemical Attack**
		In carbon-ated concrete	**In chloride contaminated concrete**		
Very low (<45%)	1	0	0	0	0
Low (45-65%)	2 rising to 3	1	1	0	0
Medium (65-85%)	3	2	3	0	0
High (85-98%)	2	3	3	2	1
Saturated (>98%)	0	1	1	3	3

Key: 0 = insignificant risk; 1 = slight risk; 2 = medium risk; 3 = high risk.

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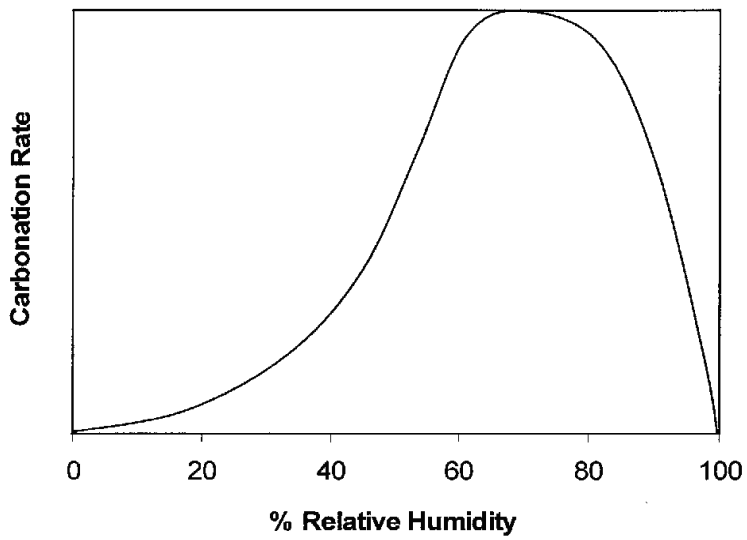

Fig. 5.6-1: Rate of carbonation of concrete as a function of relative humidity. From BRE Digest 491: Corrosion of steel in concrete: a review of the effect of humidity [Sergi and Dunster (2004)].

The complexities of the above issues, coupled with the variability of many climates, make it difficult to establish realistic and detailed environmental loads for individual structures. The environment within and around each structure is unique, depending on its location, orientation and local topography. Most structures are subjected to alternate wetting and drying which creates varying moisture conditions within the structure. In these circumstances, the relative humidity within the concrete will generally be higher than that of the surrounding atmosphere.

BS EN 8500-1 [BSI (2006)] defines the following exposure environments for concrete:

- dry or permanently wet;
- wet, rarely dry;
- moderate humidity or cyclic wet and dry.

The moisture condition relates to that in the concrete cover where reinforcement or other embedded metal tends to be located. However, in many cases, conditions in the concrete cover can be taken as reflecting that in the surrounding atmospheric environment. In these cases classification of the surrounding atmospheric environment may be adequate (unless there is a barrier between the concrete and its environment). It is important to note again that the environment will be influenced by the structure itself. For example, concrete inside a building, in the absence of leaks and condensation, is likely to be dry. Sheltered concrete will not be subjected to wetting and drying cycles, unless exposed to water through leaks or splashing, etc. However, external sheltered concrete may be subject to wetting by condensation in certain circumstances.

5.6.3 Atmospheric induced deterioration

Carbon dioxide (CO_2) is present in atmosphere at a concentration of about 0.3%, although it may be higher locally in the vicinity of industrial plants and processes as a result of their emissions to atmosphere. Carbon dioxide does react with the hydrated cement, through the process of carbonation as discussed in Section 5.4.7.3; potentially leading to reinforcement corrosion under certain conditions.

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Gases such as SO_2 and NO_x are also released to the atmosphere from various industrial processes. They are water-soluble and, in the presence of water or moisture, will form acids that can attack concrete and other susceptible materials. Other contaminants such as particulates and micro-droplets of acid or alkaline solutions or low volatility hydrocarbons or oils can arise in industrial areas. The effects of these on construction materials may need to be considered.

Although acid rain may, at times, attain a very low pH value, it typically has a negligible ability to dissolve concrete. This is because of the large buffering capacity of the alkaline pore fluid in the concrete, which neutralises the acidity of the rainwater. This also holds true for carbonated concrete. Only in cases of very porous or lean concrete are the effects of acid rain likely to become visible after a long period of exposure, which might be manifest as exposure of the aggregate particles after the removal of the concrete matrix from the exposed surface.

For structures close to the sea or estuaries, and those in the vicinity of roads where de-icing salts may be used, airborne chlorides may well be a factor influencing durability. In this case consideration will need to be given to factors such as the distance from the source of the chlorides, the direction and strength of any prevailing wind, together with the moisture content of the concrete.

For a given atmospheric environment, the internal water content of a concrete element will be higher when chlorides are present, due to their hygroscopic effect. This accounts, for example, for the heavy corrosion of reinforcement which occurs with chloride-contaminated concrete specimens situated indoors in permanently air-conditioned rooms where temperatures average (20°C) and atmospheric humidity is low (50-60%RH). This effect will be more pronounced in hot, humid and saline environments.

5.6.4 Temperature induced effects

For temperature above freezing, chemical and electro-chemical reactions are accelerated by increases in temperature. A simple rule-of-thumb which is commonly used is that an increase in temperature of 10°C will cause a doubling in the rate of a chemical reaction. This factor alone makes hot and tropical environments considerably more aggressive than temperate climates.

For example, in a situation where the average yearly temperature is approximately 30°C higher, this would lead to an 8-fold faster deterioration (2 raised to the power 3). Thus, all other factors being equal, the higher temperature in the Arabian Peninsular/Gulf area would produce a rate of deterioration that is likely to be some 8-times than that in say the European Nordic countries. This has important implications on design strategies and protective measures which might be adopted under different climatic conditions, irrespective of the nature of the aggressive agents present in the environment.

Fig. 5.6-2 shows a reinforced concrete jetty in the Arabian Gulf which is exhibiting extensive chloride induced corrosion damage after being in service for only 2 or 3 years. This structure reached the stage of failure and collapse after being in service for less than 10 years.

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Fig. 5.6-2: Reinforced concrete jetty in the Gulf exhibiting extensive chloride induced corrosion damage after only 2-3 years.

When service temperatures are below freezing for an appreciable period of the year, three effects need to be considered:

- Long-term temperatures below freezing may freeze the pore water through the bulk of the concrete. If the moisture content exceeds the critical degree of saturation for the specific concrete, the freezing may cause cracking and disintegration of the bulk of the concrete.
- The number and frequency of freeze-thaw cycles may determine whether the concrete suffers frost damage or not. If the concrete has a chance of some degree of drying out during thawing, it may resist freeze-thaw cycles for a longer period of time.
- Freezing conditions combined with the regular application of de-icing chemicals is the most severe type of frost attack. The spraying of de-icing chemicals results in a lowering of temperature of the concrete surface, causing a degree of thermal shock to the outermost layer of the exposed concrete. These conditions may lead to progressive spalling of the concrete surface.

The mechanisms of frost attack and surface scaling involving de-icing agents are discussed in Section 5.4.4.6.

In all cases air entrainment of the concrete can provide adequate resistance against frost damage, which extends the useful service life of the structure. However, it shall be noted that with dense and impermeable concretes, such as high performance concretes, it may be difficult to entrain sufficiently air to provide adequate protection. Accordingly, particular care needs to be taken in each individual case to ensure and verify that such concretes have adequate frost resistance.

5.6.5 Classification of environmental exposure

When defining exposure classes it is important to relate the exposure to the type and severity of deterioration that may result from the exposure. Hence, detailed knowledge of the governing deterioration mechanisms is needed in order to define exposure classes. In this respect differentiation is needed between mechanisms causing deterioration of the concrete and mechanisms leading to reinforcement corrosion (refer Section 5.4).

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The designer may find valuable help in defining the design basis for the exposure of different elements of his structure from the classifications presented in different national or international codes or design guidelines.

One such guideline for exposure classes is European Standard EN 206 [CEN (2000)]. This is an updated version of the exposure classes originally presented in CEB-FIP Model Code 90 [CEB (1992)]. This standard specifically differentiates between exposures leading to reinforcement corrosion and exposures leading to deterioration of the concrete. This classification is presented as Table 5.6-2.

The various exposure classes for concrete structures defined in EN 206-1 [CEN (2000)] are as follows:

X0: No risk of corrosion or attack

XC: Corrosion of reinforcement induced by carbonation

XD: Corrosion of reinforcement induced by chlorides other than from seawater

XS: Corrosion of reinforcement induced by chlorides from seawater

XF: Freeze-thaw attack upon concrete

XA: Chemical attack upon concrete

Table 5.6-2: Exposure classes for concrete structures according to EN 206-1 [CEN (2000)]

Class designation	**Description of the environment**	**Informative examples where exposure classes may occur**
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: All exposures except where there is freeze/thaw, abrasion or chemical attack	Concrete surfaces inside structures, buried in non-aggressive soil, permanently submerged in non-aggressive water. Concrete in cyclic wet and dry conditions not subject to abrasion, freezing or chemical attack.
	For concrete with reinforcement or embedded metal: Very dry	Concrete inside buildings with very low air humidity.
2 Corrosion induced by carbonation		
Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified ad follows: NOTE: The moisture condition relates to that in the concrete cover to the reinforcement or other embedded metal. In many cases conditions in the concrete cover can be taken as reflecting that in the surrounding environment. In these cases classification of the surrounding environment may be adequate. This may not be the case where there is a barrier between the concrete and its environment.		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations

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Table 5.6-2: Exposure classes for concrete structures according to EN 206-1 [CEN (2000)]

Class designation	**Description of the environment**	**Informative examples where exposure classes may occur**
XC3	Moderate humidity	Concrete inside building with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides other than from seawater		
Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from seawater, the exposure shall be classified as follows:		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from seawater		
Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from seawater or air carrying salt originating from seawater, the exposure shall be classified as follows:		
XS1	Exposed to airborne salt but not in direct contact with seawater	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5 Freeze-thaw attack		
Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet, the exposure shall be classified as follows:		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agent	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation, with de-icing agent or seawater	Road and bridge decks exposed
Class designation	Description of the environment	Informative examples where exposure classes may occur

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Table 5.6-2: Exposure classes for concrete structures according to EN 206-1 [CEN (2000)]

Class designation	Description of the environment	Informative examples where exposure classes may occur
6 Chemical attack		
Where concrete is exposed to chemical attack, which occurs in natural soils and groundwater as given in Table 5.6-3, the exposure shall be classified as given below. The classification of seawater depends on the geographical location; the classification valid in the place of use of the concrete applies. **Notes:** A special study may be needed to establish the relevant exposure condition where there are: • limits outside the range of Table 5.6-3; • other aggressive chemical; • chemical pollution of ground or water; • high water velocity in combination with the chemicals in Table 5.6-3.		
XA1	Slightly aggressive chemical environment according to Table 5.6-3	
XA2	Moderately aggressive chemical environment according to Table 5.6-3 or exposure to seawater	
XA3	Highly aggressive chemical environment according to Table 5.6-3	

Thus there are a number of types of external chemical attack as identified in Table 5.6-2. For groundwaters these environments are classified in terms of ranges of concentrations for sulfate ions, pH, aggressive CO_2 and ammonium ions and magnesium ions. Soils are classified in terms of sulfate content and acidity. Table 5.6-3 details the exposure classes for chemical attack from natural soil and groundwater, as classified in EN 206-1.

However, in some other countries a wider range of environmental actions is considered than is covered by EN 206-1 [CEN (2000)]. For example, the UK guidance on this matter is given in BS EN 8500-1 [BSI (2006), which uses the approach given in BRE Special Digest 1: *Concrete in aggressive ground* [BRE SD1 (2005)]. This uses a system for the classification of the aggressivity of ground conditions to concrete known as the *Aggressive Chemical Environment for Concrete Class* (ACEC Class) which takes account of a number of factors including the soluble sulfate and magnesium, the potential sulfate (e.g. from oxidation of pyrite), the type of site (i.e. natural or brownfield) and the mobility and pH of the groundwater.

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Table 5.6-3: Limiting values for exposure classes for chemical attack from natural soil and groundwater, as classified in Table 5.6-2 (CEN)

The aggressive chemical environments classified below are based on natural soil and groundwater at water/soil temperatures between 5 ˚C and 25 ˚C and for a water velocity sufficiently slow to approximate to static conditions.

The most onerous value for any single chemical characteristic determines the environmental classification/exposure class to be adopted for design.

When two or more aggressive chemical characteristics lead to the same exposure class; the severity of the environmental classification adopted shall be increased to the next higher exposure class, unless a study of the specific circumstances establishes that it is not necessary.

Chemical characteristic	**Reference test method**	**XA1**	**XA2**	**XA3**
Groundwater				
SO_4^{2-} mg/l	EN 196-2	≥ 200 and ≤ 600	> 600 and ≤ 3000	> 3000 and ≤ 6000
pH	ISO 4316	≤ 6.5 and ≥ 5.5	< 5.5 and ≥ 4.5	< 4.5 and ≥ 4.0
CO_2 mg/l aggressive	prEn 13577	≥ 15 and ≤ 40	> 40 and ≤ 100	> 100 up to saturation
NH_4^+ mg/l	ISO 7150-1 or ISO 7150-2	≥ 15 and ≤ 30	> 30 and ≤ 60	> 60 and ≤ 100
Mg^{2+} mg/l	ISO 7980	≥ 300 and ≤ 1000	> 1000 and ≤ 3000	> 3000 up to saturation
Soil				
SO_4^{2-} mg/kg (1) total	EN 196-2 (2)	≥ 2000 and ≤ 3000 (3)	> 3000 3) and ≤ 12000	< 12000 and ≤ 24000
Acidity ml/kg	DIN 4030-2	> 200 Baumann Gully	Not encountered in practice	

(1) Clay soils with a permeability below 10^{-5} m/s may be moved into a lower class.
(2) The test method prescribes the extraction of SO_4^{2-} by hydrochloric acid. Alternatively, the water extraction method may be used, if appropriate experience is available in its use.
(3) The 3000 mg/kg limit is reduced to 2000 mg/kg, where there is a risk of an accumulation of sulfate ions in the concrete as a result of drying and wetting cycles or capillary suction effects.

In the case of contaminated land, special attention is required where building materials are placed in contaminated soil and groundwater. Contamination may be present in solid form (e.g. mining wastes), as a liquid (e.g. solvents, oils and acids) or as gases (e.g. methane). There are a number of possible deterioration mechanisms affecting building materials under such circumstances. Contaminated sites can include or have been used for:

- landfill sites – including the surrounding areas;
- breakers yards, timber treatment works, railway sidings;

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- works employing metal finishing processes;
- industrial sites (e.g. chemical and manufacturing plants of various types);
- sewage treatment plants;
- fuel storage facilities;
- areas of filled or made ground;
- farm land;
- former mining sites.

Sulfate and acid attack are the most common factors affecting concrete in contaminated land, although the potential effects of any other contaminants on concrete would need to be assessed. Some of the activities which have caused contamination include:

- deposit or burial of industrial and domestic waste;
- spills or leaks of noxious liquids at the surface, or from underground tanks, pipes and drains;
- demolition of industrial structures and dispersal or burial of contaminated rubble and other materials;
- contaminated fill material imported to the site;
- spraying of agricultural chemicals using inappropriate procedures;
- stockpiling materials such as road salt, mining waste, etc.;
- burial of animal carcasses using inappropriate procedures.

5.7 Recommendations made in some standards and codes of practice

5.7.1 Introduction

This section considers the recommendations made in a number of standards and codes of practice for the design and construction of concrete structures. These are:

- CEB-FIP Model Code 1990;
- *fib* Model Code for Service Life Design 2006;
- EN 1992: Concrete Structures (Eurocode 2) and associated material product standards;
- new *fib* Model Code – publication expected in 2010.

5.7.2 CEB-FIP Model Code 1990

The CEB-FIP Model Code 1990 (MC90) was the first code of practice which defined specific service life requirements for new structure designs, but also indicated how these could be achieved in the design process. The importance of the level of workmanship during execution, the integration of anticipated future maintenance procedures and the need for quality assurance schemes throughout the process of designing, constructing and operating structures were recognised as integral elements of modern service life design.

MC90 expressed the basic requirements for service life design as follows:

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Concrete structures shall be designed, constructed and operated in such a way that, under the expected environmental influences, they maintain their safety, serviceability and acceptable appearance during an explicit or implicit period of time without requiring unforeseen high costs for maintenance and repair.

MC90 introduced three new features into the design process, compared with contemporary national codes of practice of the time. These were recognition of:

- The influence of the period of use as a basis for the service life to be achieved.
- The need to maintain an acceptable appearance, implying that structures should 'grow old gracefully'; thereby enhancing the wider public reputation of concrete structures.
- The cost for maintaining the structure in use at a satisfactory level of performance; which lead to a service life design process taking account of issues such as access for inspection and maintenance activities, ensuring that repairs could be undertaken or elements of the structure with a service life less than that required could be replaced.

To achieve an appropriate level of durability, MC-90 recognised that the measures to be adopted to achieve the required service life needed to be chosen on the basis of both the environmental conditions and on the significance of the structure. MC90 recognised that achieving the required service life of a concrete structure depended upon the successful performance of both structural and non-structural elements; both facets were to be considered as aspects of the design, construction and use of the structure.

MC90 recognised that non-structural elements such as drainage, joints, bearings, installations, etc. might require special attention to ensure the overall durability of the structure. It was also recognised that particular structural components such as anchorages, couplers and deviators for prestressing tendons might require special attention.

The MC90 clauses on design, execution and maintenance were generally intended to lead to a service life of the structures in excess of 50 years. However, it was also recognised that some structures would require a substantially longer service life, say 100 years or more, and other structures might require a considerably shorter service life, of perhaps less than 25 years.

5.7.3 *fib* Model Code for Service Life Design 2006

5.7.3.1 Introduction

The *fib* Model Code for Service Life Design (MC-SLD) was published as *fib* Bulletin 34 [*fib* (2006)]. The basic concept for service life design as presented in the MC-SLD is the adoption of design approach for the environmental loads associated with environmental actions which is comparable to the structural load design philosophy used in contemporary design codes for concrete structures, such as Eurocode 2 (EC2) [EN 1992: Parts 1-3: *Design of concrete structures*: CEN (2004-2006)]. Thus this service design approach seeks to minimise the chance of deterioration caused by environmental action using a similar philosophical approach employed for structural design to minimise the chance of structural failure.

This means the adoption of quantifiable models on the load side (these are the environmental actions) and on the resistance side; which is the resistance of the concrete against the considered environmental actions. The service life design approach is illustrated by examples

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given for design against (a) reinforcement corrosion caused by carbonation of concrete and (b) for chloride induced corrosion; assuming that there are no load or restraint induced cracks within the concrete. However, the MC-SLD also discusses the application of these concepts to service life design for freeze-thaw attack on concrete for situations where (a) de-icing agents are not used and (b) they are used.

Approaches to the modelling of deterioration processes are reviewed in Section 5.8.

The first step in the service life design approach defined in the MC-SLD is to quantify the deterioration mechanism being considered with realistic models describing the process physically and/or chemically with sufficient accuracy for prediction purposes. In the case of reinforcement corrosion caused by carbonation of concrete, this requires models to represent the progress of carbonation into the concrete taking account of the environment influences and the relevant concrete quality parameters. Such a model for the progress of carbonation is given in the MC-SLD. Sufficient accuracy means that the model should be validated by realistic laboratory experiments and by in-practice observations, so that mean values and scatter of the material resistance parameters are known and can be considered in the model. Of course the gathering of such data is a time consuming process which requires planning and may need a considerable lead time, as such the application of this approach may be restricted to major projects.

In the same way models for the environmental actions need to exist with statistically quantified environmental parameters (e.g. temperature, relative humidity, splashing rain events etc).

The second step is the definition of limit states for which the structure is to be designed. The following are suggested in the MC-SLD as appropriate limit states:

- depassivation of reinforcement caused by carbonation of the concrete cover;
- cracking of the concrete cover due to reinforcement corrosion;
- spalling of concrete cover due to reinforcement corrosion;
- collapse due to loss of cross-sectional area of the reinforcement, etc.

The third step is the calculation of the probability of occurrence of the limit states defined above (i.e. determination of the probability of occurrence). This is done by applying the models described in the first step (above). Nowadays it is common practice for the safety of structures to be expressed in terms of reliability (reliability index β). Depending on the type of limit state and the consequences of a failure, appropriate values for the reliability index β are chosen. Appropriate values for β are given in what is commonly known as Eurocode 0 (EC0) [EN 1990: *Basis of design*: CEN (2002)] and also in ISO 2394: '*General principles of reliability for structures*' [ISO (1998)].

The fourth step is the definition of the type of limit state described in the second step (above). The types of limit state considered are typically either related to the serviceability limit state (SLS) or to the ultimate limit state (ULS). Normally depassivation of the reinforcement will be classified as a SLS, as there is no immediate consequence to structural safety if the reinforcement is depassivated. Accordingly, β-values in the range of $\beta = 1.0$ to 1.5 may be appropriate for depassivation. However, the owner may require the adoption of higher β-values in certain circumstances. For example, this might occur if the owner indicates

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that it is particularly important to ensure there is a high probability that the aesthetic appearance of the structure is not impaired. This might be relevant to a prestige building.

For the limit states of cracking and spalling the designer has to decide which type of limit state is appropriate. If, for example, cracking and spalling occurs in the anchorage zones of a structural member with inadequate transverse reinforcement, spalling might potentially lead to collapse. In this case cracking and spalling would need to be defined as an ULS. In other cases if cracking and spalling does not influence the load bearing capacity of the structural element, cracking and spalling may be defined as a SLS.

The service life design approach in the MC-SLD recognises two basic strategies, namely:

- Strategy 1: Provision of resistance to deterioration.
- Strategy 2: Avoidance of deterioration; the so-called '*design-out*' approach.

At the time of writing work is underway in ISO Technical Committee TC-71/SC-3/WG-4 to develop an ISO standard entitled "Service life design of concrete structures". This is based upon the *fib* MC-SLD. It is expected that work on the ISO standard will have progressed sufficiently to have a Committee Draft prepared by the end of 2010.

5.7.3.2 Contents of *fib* MC-SLD

The *fib* Model Code for Service Life Design (MC-SLD) is divided into five chapters and contains four informative annexes.

The five chapters comprise:

1. Introduction/general comments
2. Basis of design
3. Verification of service life design
4. Execution and its quality management
5. Maintenance and condition control

The four informative annexes comprise:

Annex A: Management of reliability for service life design of concrete structures

Annex B: Full probabilistic design methods

Annex C: Partial factor methods

Annex R: Reliability management – from SLS to ULS

5.7.3.3 Design options and approaches given in *fib* MC-SLD

The flow chart in Fig. 5.7-1 illustrates the flow of decisions and the design activities needed in a rational service life design process with a chosen level of reliability. The MC-SLD identifies four different levels of sophistication in the performance-based design approach:

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Option 1 A *full probabilistic design*, corresponding to *Design Strategy B* (see Section 5.1.10). It will seldom be possible to employ this approach for new structures due to lack of statistical data. The approach might be adopted for special long-life or monumental type structures. The approach may be applicable to the assessment of existing structures. This approach is designated as '*Level 1 design*'.

Option 2 A *partial factor design* (semi-probabilistic), also corresponding to *Design Strategy B* (see Section 5.1.10), but with factors calibrated with Level 1 approach above. This approach is based upon design values for loads, capacities and geometrical characteristics. This is a deterministic approach where the probabilistic nature of the problem (i.e. the scatter in material resistance properties and environmental load parameters) is taken into account by partial safety factors. This approach is designated as '*Level 2 design*'.

Option 3 A *deem-to-satisfy design* corresponding to methods in current codes, but also corresponding to *Design Strategy B* (see Section 5.1.10) and with requirements calibrated, to the extent possible, with Level 1 above. This approach is applied using tabulated numbers for w/c-ratio, reinforcing bar cover, crack width, etc. This type of approach is comparable to the approach generally used in contemporary standards. However, the descriptive rules used in contemporary standards are commonly not based on physically and chemically correct models, but more on practical experience and represent what is, by consensus, deemed to be appropriate. However, it should also be recognised that sometimes the experience is not always appropriate to the full range of circumstances which may be encountered. In the future such deemed-to-satisfy rules are expected to be calibrated against the full probabilistic approach, assuming appropriate data is available. This approach is designated as '*Level 3 design*'.

Option 4 *Avoidance of deterioration*, corresponding to *Design Strategy A* (see Section 5.1.10). This approach involves avoiding circumstances which can lead to deterioration or the use of non-reactive materials. Examples are avoiding the saturation concrete to minimise the risk of frost attack, the selective use of stainless steel to avoid corrosion induced deterioration, the use of non-reactive aggregates to minimise the risk of alkali-aggregate reactions, etc.

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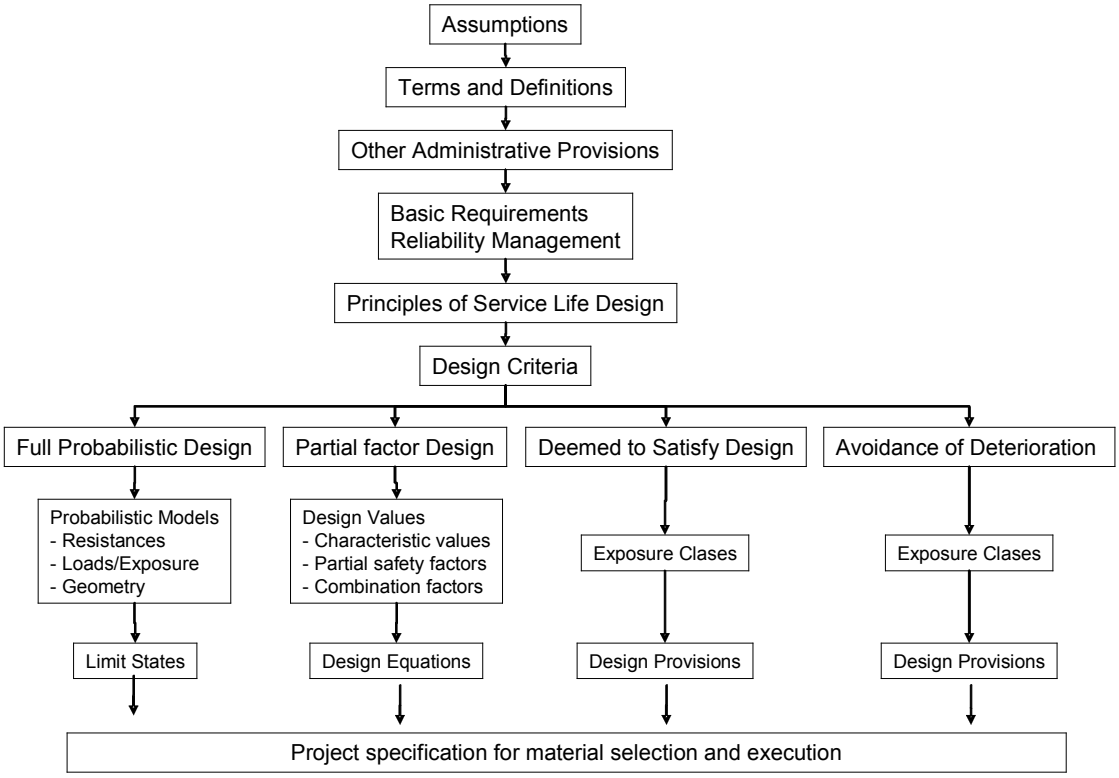

Fig. 5.7-1: Flow chart for the Model Code for Service Life Design (MC-SLD)

The MC SLD gives:

- "principles" that have to be obeyed;
- "application rules" as informative examples of sufficiently verified methods.

The MC-SLD defines various ***limit states***: states beyond which the structure no longer fulfils the relevant design criteria. These include:

- *serviceability limit states (SLS)*: states that correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met;
- *ultimate limit state (ULS)*: states associated with collapse or with other similar forms of structural failure.

The ***design service life*** is the assumed period for which a structure or any part of it is to be used for its intended purpose with anticipated maintenance, but without major repair being necessary.

The design service life is defined by:

- a definition of the relevant limit state;
- a number of years;
- a required level of reliability of not passing the given limit state during this period.

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The MC-SLD gives guidance upon ***reliability management*** of the concrete structure concerned. Annex A of MC-SLD gives guidance upon reliability differentiation. It defines a number of general classifications including the following:

- Consequence class CC3, CC2 and CC1. (Refer Table 5.7-1)
- Reliability class RC3, RC2 and RC1. (Refer Table 5.7-2)
- Robustness class ROC3, ROC2 and ROC1. (Refer Table 5.7-3)
- Design supervision level DSL3, DSL2 and DSL1. (Refer Table 5.7-4)
- Execution class EXC3, EXC2 and EXC1. (Refer Table 5.7-5)

In addition Annex A of the MC-SLD defines 3 levels of condition control to be applied during the service life of the concrete structure concerned:

- CCL3, CCL2, CCL1 and CCL0. (Refer Table 5.7-6)

The MC-SLD also sets down a series of assumptions which have been made for its use. These are in addition to the general assumptions made in EN 1990: *Basis of design*: CEN (2002). The MC-SLD assumptions are:

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided in factories, in plants and on site.
- Construction is carried out by personnel having the appropriate skill and experience.
- The construction materials and products are used as specified in the relevant material or product specifications.
- The structure will be adequately maintained according to the options given in this document.
- The structure will be used in accordance with the design brief.
- The minimum requirements for execution and workmanship given in EN 13670 are complied with.

Table 5.7-1: Definition of consequence classes (CC) (Ref: Table A2-1: MC-SLD)

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

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Table 5.7-2: Recommended minimum values for reliability index β for use in service life design for the intended design life time (Ref: Table A2-2: MC-SLD)

Exposure class – Eurocode 2	Description	Reliability class	Reliability value β (Probability value, p_f)	
			SLS[1] Depassivation[2,3]	ULS Collapse
XC[3]	Carbonation	RC1	1.3 ($p_f \approx 10^{-1}$)	3.7 ($p_f \approx 10^{-4}$)
		RC2	1.3 ($p_f \approx 10^{-1}$)	4.2 ($p_f \approx 10^{-5}$)
		RC3	1.3 ($p_f \approx 10^{-1}$)	4.4 ($p_f \approx 10^{-6}$)
XD[3]	De-icing salt	RC1	1.3 ($p_f \approx 10^{-1}$)	3.7 ($p_f \approx 10^{-4}$)
		RC2	1.3 ($p_f \approx 10^{-1}$)	4.2 ($p_f \approx 10^{-5}$)
		RC3	1.3 ($p_f \approx 10^{-1}$)	4.4 ($p_f \approx 10^{-6}$)
XS[3]	Seawater	RC1	1.3 ($p_f \approx 10^{-1}$)	3.7 ($p_f \approx 10^{-4}$)
		RC2	1.3 ($p_f \approx 10^{-1}$)	4.2 ($p_f \approx 10^{-5}$)
		RC3	1.3 ($p_f \approx 10^{-1}$)	4.4 ($p_f \approx 10^{-6}$)

Key to Table 5.7-2:

1 A SLS reliability level of β = 1.3 in the consequence class could lead to lower ULS reliability level than is usually required by structural design codes, e.g. ISO 2394. This implies that for very aggressive environments, higher values for β_{SLS} are likely to be required. Annex R of the fib MC-SLD gives guidance on how to fulfil the ULS requirements.

2 Depassivation of the surface of the reinforcement in the area exposed to the design environmental load.

3 In situations where there is sufficient availability of oxygen and moisture to support corrosion.

Notes to Table 5.7-2:

A The reliability classes (RC) may be defined by the β reliability index concept.

B Three reliability classes RC1, RC2 and RC3 may be associated with the three consequences classes CC1, CC2 and CC3 as defined in Table 5.7-1.

C Table 5.7-2 gives recommended minimum values for the reliability index associated with the defined reliability classes.

Table 5.7-3: Robustness classes (ROC) (Ref: Table A3-1: MC-SLD)

Robustness class	Characteristics	Postulated characteristic loss of cross section (preliminary estimate) ΔAs [%]
ROC 3	Bending reinforcement outside of anchorage and laps	25
ROC 2	Shear reinforcement, anchorage zones with confinement	15
ROC 1	Anchorage zones without confinement	5

Notes to Table 5.7-3:

A The service life of a structure susceptible to reinforcement corrosion depends on the lengths of the initiation and propagation periods. This means the structural ULS reliability can be achieved either by excluding corrosion so there is no loss of cross-section or deterioration of the bond or anchorage of the reinforcement, or by adding extra reinforcement (i.e. sacrificial cross-section). Accordingly, the higher the reliability with regard to depassivation, the lower is the potential need for extra reinforcement.

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B Structural failure caused by corrosion of reinforcement may be due to loss of cross-section of the bars or due to the influence of the spalling of concrete cover and the associated potentially adverse effects upon bond and anchorage (i.e. reductions).

C Spalling of concrete cover in anchorage zones without confinement may lead to sudden failure.

D Reliable data for limit values upon the amount of corrosion required to cause spalling is not currently available. Limit values are expected to depend upon arrange of factors such as bar diameter, bar spacing, concrete quality, cover depth and upon other factors such as the volume of rust products which can be accommodated before cracking/spalling occurs. Environmental conditions, such as whether chlorides are present or not, could well also influence the limit values. The values in Table 5.7-3 are preliminary estimates which need to be confirmed by further research. The MC-SLD suggests that the preliminary estimates reproduced in Table 5.7-3 can be taken as mean values providing the element cross-section contains more than three single bars.

Table 5.7-4: Design supervision levels (DSL) (Ref: Table A4-1: MC-SLD)

Design supervision level	**Characteristics**	**Minimum recommended requirements for the checking of calculations, drawings and specifications**
DSL3: Relating to RC3	Extended supervision	Third party checking: Checking performed by an organisation different to that which has performed the design
DSL2: Relating to RC2	Normal supervision	Checking by different persons to those originally responsible and in accordance with the procedures of the organisation
DSL1: Relating to RC1	Normal supervision	Self-checking: Checking performed by the person who has prepared the design

Notes to Table 5.7-4:

A Design supervision differentiation consists of various organisational quality control measures, which can be used together. For example, the differentiation of design supervision level may be used together with other measures such as classification of designers and checking authorities depending upon their competence and experience, as well as their internal organisation for the relevant to the type of construction works being designed.

B The design supervision levels may be linked to the reliability class selected or chosen according to the importance of the structure, which may be made in accordance with national requirements or the design brief, and implemented through appropriate quality management measures.

C Minimum levels for the quality management regimes may be given in national legislation.

Table 5.7-5: Execution classes (EXC) (Ref: Table A5-1: MC-SLD)

Execution class	**Characteristics**	**Requirements**
EXC3: Relating to RC3	Extended inspection	Third party inspection
EXC2: Relating to RC2	Normal inspection	Inspection in accordance with the procedures of the organisation
EXC1: Relating to RC1	Normal inspection	Self inspection

Notes to Table 5.7-5:

A The execution classes may be linked to the quality management classes selected and implemented through appropriate quality management measures.

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B Minimum levels for the quality management regimes may be given in national legislation.

C CEN ENV 13670-1 refers to "inspection classes".

D "Inspection" is defined by ISO 9000 as "Conformity evaluation by observation and judgment accompanied as appropriate by measurement, testing or gauging".

E The "Execution classes" might also include other elements of the quality management regime at the construction site.

Table 5.7-6: Condition control levels (CCL) (Ref: Table A6-1: MC-SLD)

Condition control level	Characteristics	Requirements
CCL3	Extended inspection	Systematic inspection and monitoring of parameters relevant to the deterioration process(es) that is (are) critical in the SLD
CCL2	Normal inspection	Regular visual inspection by qualified personnel
CCL1	Normal inspection	No systematic monitoring nor inspection
CCL0	No inspection	No possible inspection, for instance due to lack of access

Notes to Table 5.7-6:

A A proper inspection during the service life of a structure will give the owner the possibility of apply protective measures should the expectations for the service life design not be achieved. In this way the consequences of unacceptable performance in service are therefore reduced. This opens the possibility of employing a more liberal reliability class and associated β – value in the design.

5.7.4 EN 1992: Concrete structures (Eurocode 2) and associated product standards

The cornerstone of the system of European standards is the Construction Products Directive (CPD) and the European Committee for Standardisation (CEN: Comité Européen de Normalisation) is mandated by the by the European Commission for the development of harmonised European technical standards for construction products. The objective of the CPD was to break down technical barriers to trade in construction products and services between member states in the European Union, and somewhat more widely, the European Economic Area. This is achieved by using European Standards as a means to resolve technical differences between member states. Particular attention has been given over recent years to the development of the structural Eurocodes as they are the European Standards whose use is crucial to the standardisation of structural design calculations and the concepts of service life planning across the different European countries.

The CPD defines provisions, including requirements, relating not only to building safety, but also to health, durability, energy economy, protection of the environment and other aspects important to the public interest. The CPD requirements '*must, subject to normal maintenance, be satisfied for an economically reasonable working life*'. Thus a concrete structure having adequate durability is one which satisfies this stipulation.

Design for durability is not fully covered in EN 1992-1-1 [CEN (2004)] and reference needs to be made to a number of other standards. The relationship between the various European standards for the design and execution of concrete structures, together with related standards for constituent materials and associated material test standards, is presented in Fig. 5.7-2. Further information upon the components of the system of European standards is given in Fig. 5.14-1.

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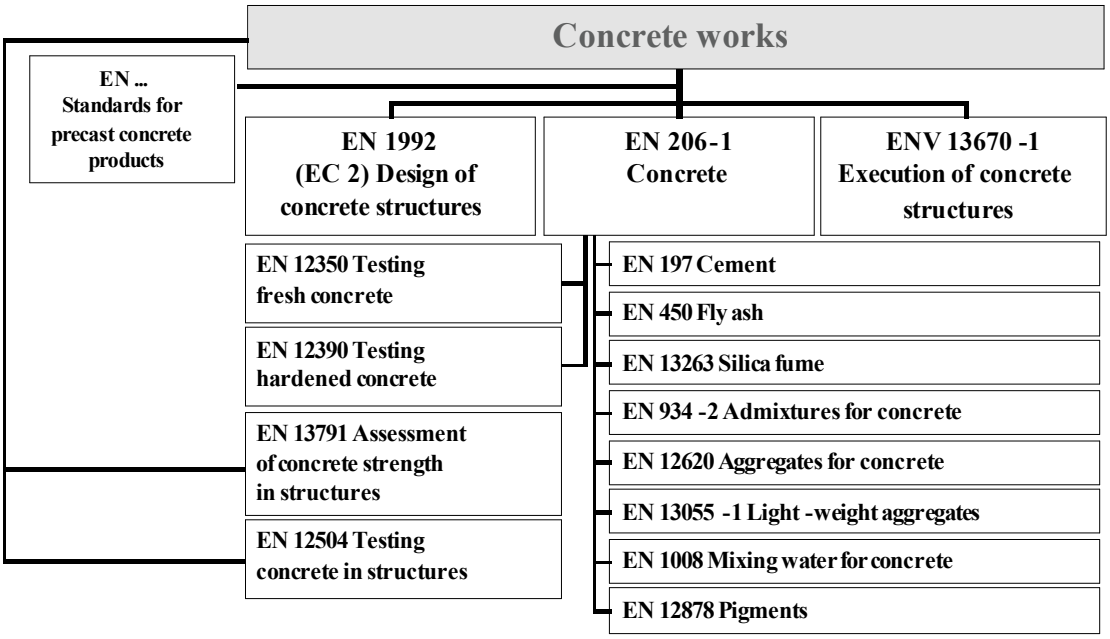

Fig. 5.7-2: Relationship between EN 206-1 and other CEN standards for the design and execution of concrete structures, including standards for constituent materials and associated test standards – as defined in Figure 1, EN 206-1 [CEN (2000)].

European Standard EN 206-1 [CEN (2000)] applies to concrete for structures cast in situ, precast structures, as well as structural precast products for buildings and civil engineering structures. EN 206-1 [CEN (2000)] applies to:

- concrete mixed on site, ready-mixed concrete and precast concrete;
- normal-weight, heavy-weight and light-weight concrete;
- high-strength concrete.

However, EN 206-1 [CEN (2000)] does not apply to aerated concrete, foamed concrete, concrete with an open structure or concrete with a density less than 800 kg/m^3.

Fig. 5.7-3 portrays the components of the EN 206-1 [CEN (2000)] concept for the durability for concrete structures, namely:

- the specification of the exposure conditions to which the structure will be subjected;
- the requirements for the composition of the concrete;
- the specification of the minimum depth of cover to reinforcement;
- adequate curing of the concrete structure whilst the freshly cast concrete hardens.

Guidance upon exposure classes is given in European Standard EN 206-1 [CEN (2000)]. This standard specifically differentiates between exposures leading to reinforcement corrosion and exposures leading to deterioration of the concrete as follows:

X0: no risk of corrosion or attack

XC: corrosion of reinforcement induced by carbonation

XD: corrosion of reinforcement induced by chlorides other than from seawater

XS: corrosion of reinforcement induced by chlorides from seawater

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XF: freeze-thaw attack upon concrete

XA: chemical attack upon concrete

The complete classification is presented as Table 5.6-2.

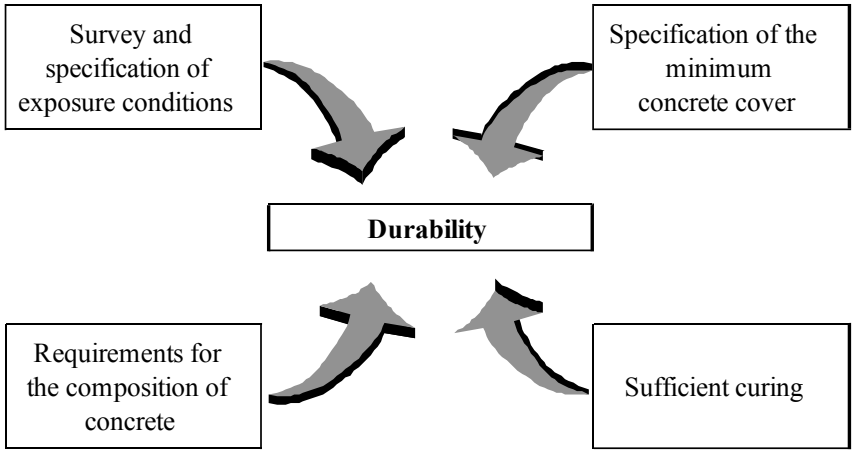

Fig. 5.7-3: The EN 206 durability concept for concrete structures [after Thielen (2009)]

Fig. 5.7-4 illustrates the exposure conditions and environmental actions associated with the corrosion of reinforcement in concrete. This shows the principal influencing variables recognised in EN 206-1 [CEN (2000)].

Fig. 5.7-5 illustrates the exposure conditions and environmental actions associated with deterioration of the concrete. Again this shows the principal influencing variables recognised in EN 206-1 [CEN (2000)].

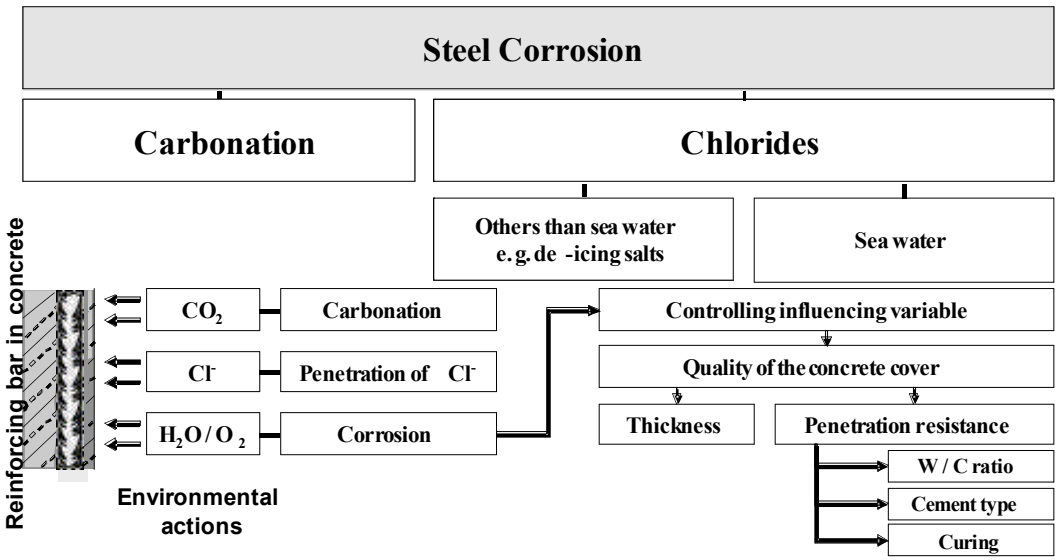

Fig. 5.7-4: EN 206: Exposure conditions and environmental actions associated with the corrosion of reinforcement in concrete. Illustration of influencing variable [after Thielen (2009)]

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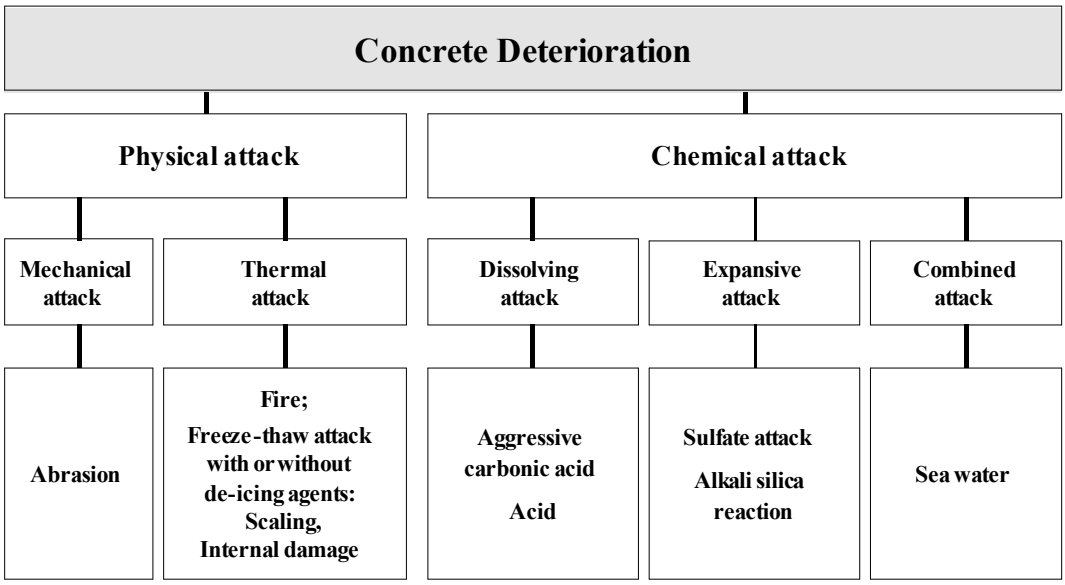

Fig. 5.7-5: EN 206: Exposure conditions and environmental actions associated with deterioration of the concrete [after Thielen (2009)]

Table 5.7-7 details the recommended minimum values for composition and properties of concrete in accordance with EN 206-1 [CEN (2000)].

Table 5.7-8 provides some further observations with regard to nationally determined parameters associated with EN 206-1 for exposure classes XC (corrosion due to carbonation) for an intended working life of at least 50 years.

Although EN 206 -1 [CEN (2000)] does not provide direct guidance on the depth of cover to be adopted for different service environments and different concrete mix designs; BS EN 8500-1 [BSI (2006)] does. The BS EN 8500-1 [BSI (2006)] guidance is given in a series of tables giving durability recommendations for reinforced or prestressed concrete elements with various intended working lives ranging from at least 50 years to at least 100 years for normal weight concrete with a maximum aggregate size of 20 mm. The parameters considered in the durability recommendations are aggressivity of the service environments (classes XC, XD and XS), the compressive strength class of the concrete, maximum water-cement ratio, minimum cement or combination of binders content and cover depth, including a design allowance for the deviation of the cover depth to the reinforcement. A separate guidance table is given for buildings for an intended working lives of at least 50 years.

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Table 5.7-7: Recommended minimum values for composition and properties of concrete as defined by EN 206-1 [CEN (2000)]

	Exposure Classes																	
	No risk of corrosion or attack	**Carbonation-induced corrosion**				**Chloride-induced corrosion**						**Freeze-thaw attack**				**Aggressive chemical environment**		
						Seawater			**Chloride other than seawater**									
Class	X0	XC 1	XC 2	XC 3	XC 4	XS 1	XS 2	XS 3	XD 1	XD 2	XD 3	XF 1	XF 2	XF 3	XF 4	XA 1	XA 2	XA 3
Maximum w/c	---	0.65	0.60	0.55	0.50	0.50	0.45	0.45	0.55	0.55	0.45	0.55	0.55	0.50	0.45	0.55	0.50	0.45
Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
Minimum cement content (kg/m^3)	---	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
Minimum air content (%)	---	---	---	---	---	---	---	---	---	---	---	---	4.0 (1)	4.0 (1)	4.0 (1)	---	---	---
Other require-ments												Freeze-thaw resisting aggregates in accordance with the recom-mendations in EN 12620					Sulfate-resisting cement (2)	

1) Where the concrete is not air entrained, the performance of concrete should be tested in accordance with an appropriate test method and comparison made with a concrete whose freeze-thaw resistance for the relevant exposure class is proven.

2) When SO_4^{2-} leads to exposure classes XA2 and XA3, it is essential to use sulfate-resisting cement. Where cement is classified with respect to sulfate resistance, moderate or high sulfate-resisting cement should be used in exposure class XA2 (and in exposure class XA1 when applicable) and high sulfate-resisting cement should be used in exposure class XA3.

Note to Table 5.7-7

1 C20/25 indicates the characteristic compressive strength of the concrete; where C20 represents the characteristic cylinder strength; and C25 the characteristic cube strength.

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Table 5.7-8: Some observations with regard to NDPs to EN 206-1 for exposure classes XC for an intended working life of at least 50 years

Exposure class	Minimum cover, mm		Compressive strength class			Max. water-cement ratio			Minimum cement content		
	min	max	min	max	mfu	min	max	mfu.	min	max	mfu
XC1	10	20	NR	C25/30	C20/25	0,60	NR	0.65	NR	300	260
XC2	15	35	NR	C28/35	C25/30	0,55	NR	0.60	150	300	280
XC3	10	35	NR	C32/40	C25/30	0,45	NR	0.55	150	340	280
XC4	15	40	NR	C32/40	NR	0,45	NR	0.50	150	340	300

NDP = nationally determined parameter
mfu = most frequently used value
NR = not required

It is understood that CEN TC 104: '*Concrete and related products'* is working on the preparation of the next revision of EN 206-1 [CEN (2000)] which, amongst other matters, is expected to involve changes in the provisions concerning durability, use of additions and conformity evaluation.

The changes envisaged in EN 206-1 [CEN (2000)] relating to durability are reported to potentially involve [Thielen (2009)]:

- Revisions to the classification of exposure conditions; with the intention of reducing the number of environmental classes employed for typical concrete works and possibly including additional information upon alkali-silica reaction in an annex to the standard.
- Verification of resistance to environmental actions; with further consideration being given to matters such as:
- The prescriptive values given for Portland cement (CEM I) and blended Portland cement (CEM II) concretes in respect of the maximum water-cement ratio, the minimum characteristic compressive strength (f_{ck}), the minimum cement content and the minimum air content:
  - equivalent concrete performance based upon reference testing;
  - methods of durability design (in co-operation with CEN TC 250/SC2).

5.7.5 *fib* Model Code 2010

Currently *fib* is working on a full revision of the CEB-FIP Model Code 1990 (MC90) and a draft of this new document is expected to be available for the third *fib* Congress in Washington in June 2010. Whilst MC90 deals mainly with structural design of concrete structures, the new *fib* Model Code 2010 (MC2010) has taken a holistic through-life perspective upon design, construction and in-service care and management of concrete structures. Accordingly the revised *fib* code also addresses issues concerned with matters such as sustainability, through-life care, impacts and management, as well as providing guidance upon matters of structural

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design. These broader issues are reflected in a consistent manner in the recommendations made concerning execution, materials and conservation, as well as in service life design concepts as described here.

As currently proposed at the time of writing (2009), MC2010 is divided into five parts and ten chapters. The overall layout of the proposed revised *fib* code is shown in Table 5.7-9.

Table 5.7-9: Main components of MC2010

Part	**Chapter**
Part I	1. Scope
	2. Terminology
	3. Basic principles
Part II: Design Input Data	4. Design principles
	5. Materials
	6. Interface characteristics
Part III: Design	7. Design
Part IV: Construction	8. Construction
Part V: Conservation	9. Conservation
	10. Dismantlement

On matters relating to issues of durability and service life design of concrete structures, MC2010 largely adopts the concepts and principles defined in the *fib Model Code for Service Life Design* (MC-SLD) as discussed in Section 5.7.3. This emphasises the adoption of a through-life perspective upon design, construction and in-service care and management of concrete structures.

However, there are some differences in the terminology used in the two documents. Perhaps the most notable in respect of durability and service life design issues concerns the terms '*conservation*' and '*condition control*'. Although MC2010 uses both of these terms, it primarily employs the term '*conservation*' to describe activities for the through-life management of a concrete structure; whereas the term '*condition control*' was adopted in the *fib* MC-SLD. These matters are noted in Section 5.17, which uses the term '*condition control*' on the basis that the *fib* MC-SLD is currently (2009) the published reference document.

As noted previously, it is necessary to quantify the deterioration mechanism being considered with realistic models describing the process physically and/or chemically with sufficient accuracy for prediction purposes. These potentially need to model the effects of deterioration processes over time to establish change in structural condition and to set margins against an *unacceptable* state being reached. Fig. 5.7-6 illustrates schematically an engineering representation of this. The concepts are set out in terms of two probability distributions; one for the resistance (strength) function R(t) and the other for the loading function S(t). The result is a probability of failure (non-compliance) against time. However that said, the limitations of currently available models, or indeed the absence of such models, for a number of deterioration processes also need to be recognised.

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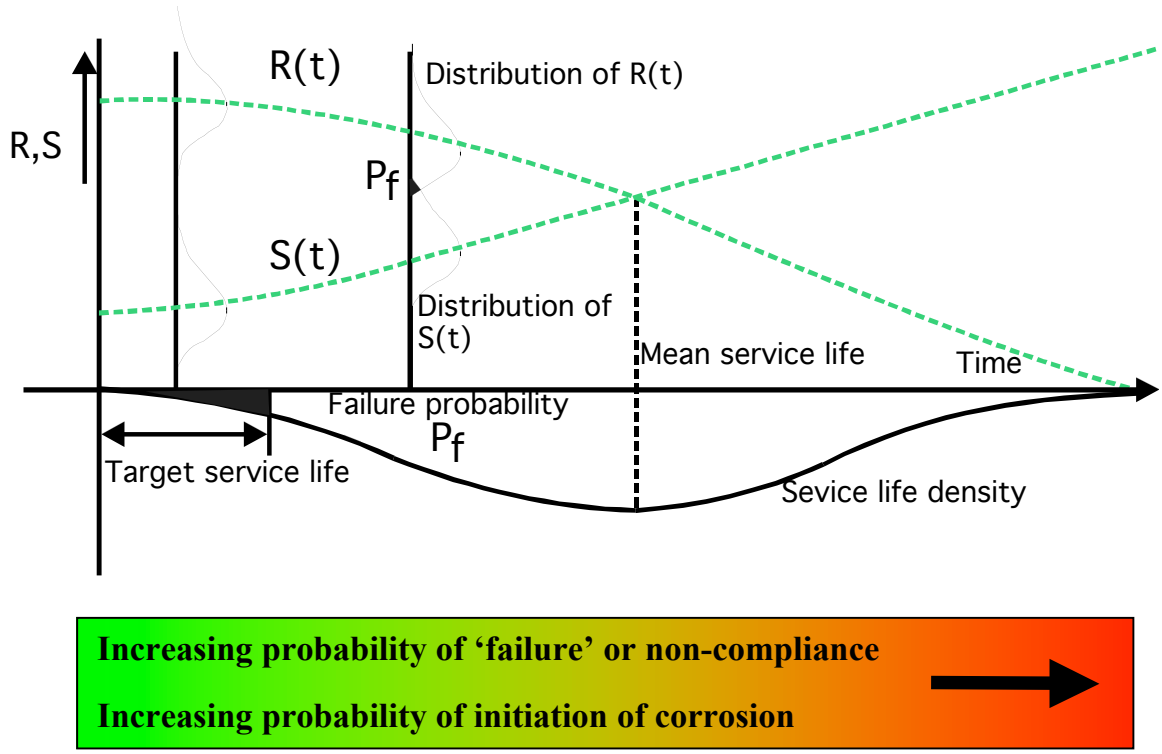

where:

R(t) is some form of time dependent resistance function. In the case of the structural performance this might be the ultimate load bearing capacity (e.g. strength) of the element which, if affected by progressive deterioration, would be expected to decrease with time, as shown above.

S(t) is some form of time dependent loading function. For structural performance this might be the applied loading. In the case of an environmental load (e.g. carbonation or chloride effects) the burden would be expected to increase with time, as shown above.

Fig. 5.7-6: Schematic illustrating the concepts of probabilistic service life

5.8 Overview of modelling of deterioration processes

5.8.1 Introduction

The importance of modelling follows from the fact that no prediction of future developments can be made without some form of model, no matter how crude. Models for predicting time to durability 'failure', that is non-compliance with a specified durability criterion, for concrete structures are increasingly being used as tools for service life design and prediction. The corrosion of reinforcement, induced by carbonation or by chloride ingress, is considered to be the main durability problem affecting concrete structures. These two degradation processes, along with abrasion, have proved to be the most amenable for service life modelling.

A number of other concrete deterioration processes have been modelled, but for various reasons their development has progressed to a lesser degree than the models for reinforcement corrosion. In many instances the design approach adopted for these deterioration processes has been to seek to avoid the problem by using a 'design-out' strategy (Strategy A – see Section 5.1.10), which does not require the use of models to estimate the time over which resistance to deterioration is expected to be achieved.

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Reinforced concrete deterioration is generally assumed to follow a two-stage process, with a corrosion initiation stage, in which the reinforcement is protected by the cover concrete, and a propagation stage, in which corrosion proceeds. Fig. 5.1-8 illustrates this concept.

The use of models in performance based service life design enables, amongst other things, an estimate to be made of the point at which a particular performance/functional requirement or fundamental property criterion is (or is not) being achieved. In practice there are various service life criteria which may be used, depending upon the aspect of performance being considered. Perhaps the three most common service life criteria are technical service life, functional service life and economic service life; which are discussed in Section 5.1.9.

Currently in service life design the end of technical service life in a structure subject to deterioration caused by reinforcement corrosion is commonly defined by the point at which corrosion is initiated. The discussion of the models presented in this Section stops this point. Other criteria for the end of technical service life (refer Section 5.1.9) more appropriate to the assessment of the performance of existing concrete structures, such as the first appearance of cracking visible to the naked eye or the first spalling, are considered in Chapter 8.

Chloride ingress in saturated concrete is generally modelled using Fick's 2nd law of diffusion, although other processes such as wick action and convection are important where the concrete is periodically dry. The inputs for these models can either be estimated from measurements on actual structures or predicted using empirical relationships derived from laboratory and field concretes. Carbonation leads to a reduction in the pH of the concrete pore solution. Carbonation depth generally shows a square root relationship with time. The rate depends on the physical properties of the concrete and its pH buffering capacity, which is in turn dependent on the cement content. Corrosion is initiated when the carbonation front reaches the reinforcement.

One of the main problems with the development and application of models for reinforced concrete deterioration is the large number of variables involved. These include:

- the composition of the concrete and its components;
- the physical properties of the concrete;
- the curing conditions employed;
- the range of exposure conditions;
- the use of protective measures such as coatings and corrosion inhibitors.

There is a considerable amount of data, primarily from laboratory samples and field trials, which can be used in calibrating and validating models. However, the large number of variables involved means that the data are often very limited in important areas. The reliability of models based on the data is consequently limited.

However, the reliability of these models is likely to improve over time as more data (from real structures, field trials and laboratory experiments) for calibration become available, and as the models themselves are further developed. Models based on single value inputs for variables will consequently produce a single value output; for example, the time taken for the chloride ion concentration at a given cover depth to exceed a representative threshold level. Provided reliable input data are available, this approach is relatively straightforward to use. However, it does not indicate the uncertainty inherent in the models or in the performance of structures in their environment. It has been suggested, therefore, that durability design might

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be better undertaken using probabilistic methods similar to those employed for structural design purposes; the approach would recognise the variability in the chemical and physical properties of the concrete as well as in its environment, and would present the effect of changes in specification and design in terms of changes in risk of failure.

Such probabilistic reliability based treatment of the models and parameters governing the durability performance and service life of concrete structures exposed to different types of environmental aggressivity was developed in the European DuraCrete research project. The results are presented in a series of reports which give general guidelines for the probabilistic performance based durability design of concrete structures [DuraCrete (1999)].

Further information on modelling deterioration mechanisms is given in various publications including: Helland et al. (1995), HETEK Reports 1, 6 & 9 (1997), CEB (1997), Engelund (1998), DuraCrete (1998), Quillin in BRE Report 434 (2001) and BRE Digest 455 (2001), as well as Bamforth in Concrete Society Technical Report No 61 (2004).

Table 5.8-1: Overview of availability of service life models for various forms of deterioration and their relationship to structural limit states [Buenfeld et al. (2008)]

Degradation mechanism	**Serviceability limit state**	**Ultimate limit state**
Reinforcement corrosion	Corrosion initiation	Flexural failure
	Cover cracking	Shear failure
	Spalling	Anchorage failure
	Excessive crack widths	
	Excessive deflections	
Alkali aggregate reaction	Cover cracking	Degradation of mechanical properties
	Spalling	
Sulfate attack	Cover cracking	Degradation of mechanical properties
	Spalling	
Freeze-thaw attack	Cover cracking	Degradation of mechanical properties
	Spalling	
Leaching	Loss of pH at reinforcing bar	Degradation of mechanical properties
Acid attack	Loss of pH at reinforcing bar	Degradation of mechanical properties
Abrasion	Loss of cover	

Key to Table 5.8-1: Availability of service life models:

Widely available	**Limited modelling**	**No models**

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Buenfeld et al. (2008) have conveniently produced an overview of the availability of service life models for various forms of deterioration and these are classified as to whether the models relate to the ultimate or serviceability limit states. This overview is reproduced as Table 5.8-1. Buenfeld et al. (2008) note that the majority of service life models developed attempt to predict the duration of the initiation phase of the deterioration process concerned, but that much less work has been carried out in respect of analysing potential structural behaviour during the propagation phase.

5.8.2 Carbonation induced corrosion of reinforcement in uncracked concrete

The carbonation process is commonly modelled by assuming that the carbonation front (i.e. the leading edge of the band of carbonated concrete) does not advance until all the calcium hydroxide closest to the surface of the concrete has reacted with atmospheric carbon dioxide (CO_2) which has ingressed into the concrete. The carbonation rate coefficient (K) will depend mainly upon the rate of diffusion of atmospheric carbon dioxide through the already carbonated surface layer of concrete and upon the amount of calcium hydroxide ($Ca(OH)_2$) present to react with the carbon dioxide. Thus the carbonated material provides protection to the uncarbonated concrete beneath it, producing a slowing of the advance of the carbonation front into the concrete with time. The sharpness of the carbonation front and the rate at which it advances is controlled by the chemical buffering capacity of the concrete.

The rate at which the carbonation front advances into the concrete is generally described by a power function, which in its simplest form is as follows:

$$D = A + K \cdot t^{n} \qquad (5.8\text{-}1)$$

where:

D is the depth of carbonation

K is the carbonation rate coefficient (an environment & concrete related constant)

t is the time of exposure to carbon dioxide

A and n are constants related to the environment and to properties of the concrete

In its most simplified form this relationship is described as being a function of the square root of time (i.e. A = 0 and n = 0.5). In these circumstances the above relationship becomes:

$$D = K \cdot \sqrt{t} \qquad (5.8\text{-}2)$$

Fig. 5.8-1 shows this relationship graphically, illustrating the influence of concrete quality upon the depth of carbonation with time for notionally low, high and very high grades of concrete. It assumes that all three types of concrete are exposed to the same environmental conditions. On the basis of the above relationship it is possible to estimate the time to depassivation of the reinforcing bars and therefore the time to the initiation of corrosion. The permeability of the concrete is reduced if there are fewer and smaller capillary pores in the cement paste matrix. This can be achieved with concrete mixes with a low water-cement ratio and high cement content, but also which are properly cured to facilitate hydration. Such concrete mixes carbonate more slowly and have a smaller value for the carbonation rate coefficient, K. Data on the performance of real structures indicates that the constant ‘A’ may have a value as high as 4 mm and that it is common for the constant ‘n’ to have a value of less than 0.5.

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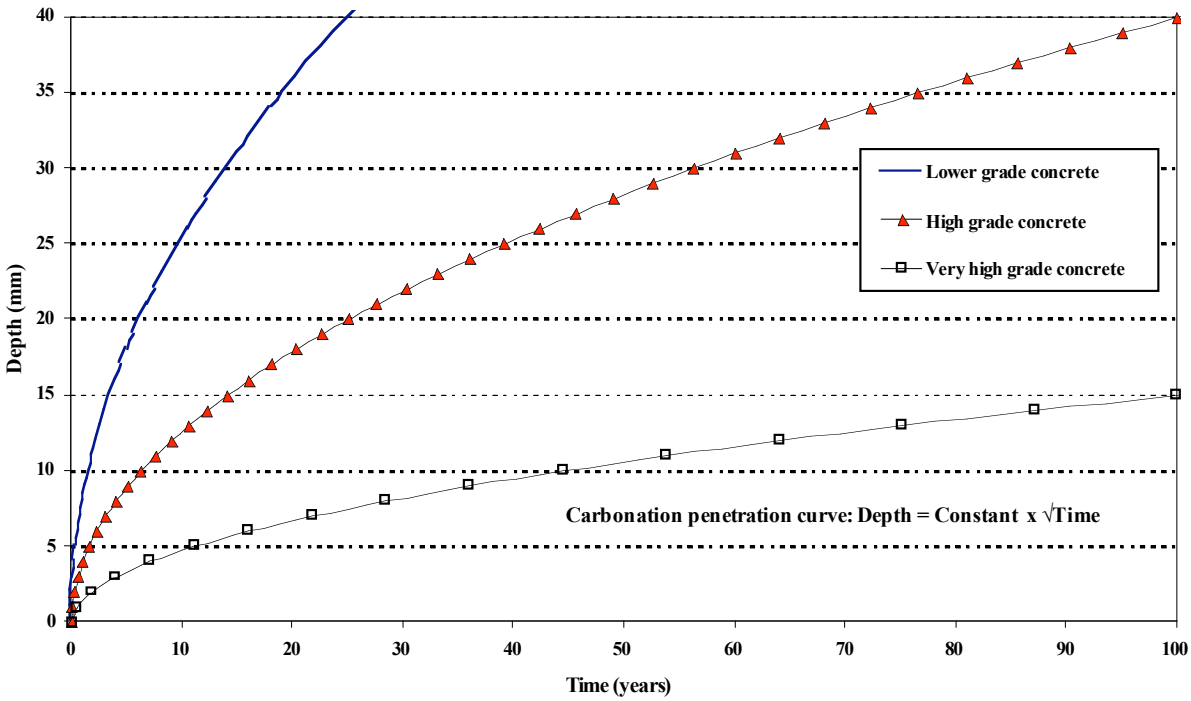

Fig. 5.8-1: Increase of carbonation depth with time – illustrating the influence of concrete quality

The *fib* Model Code for Service Life Design (MC-SLD) [*fib* Bulletin 34 (2006)] includes models for the verification of service life design for carbonation induced corrosion of reinforcement occurring in initially uncracked concrete.

The *fib* MC-SLD [*fib* Bulletin 34 (2006)] also presents models for a partial factor method for carbonation induced corrosion of reinforcement, with the treatment again assuming that the induced damage originates in initially uncracked concrete.

Additional information upon carbonation induced corrosion of reinforcement and some models to represent this process are given in the CONTECVET user manual for assessing residual service life of concrete structures affected by corrosion [CONTECVET (2000)].

5.8.3 Chloride induced corrosion of reinforcement in uncracked concrete

Chloride ingress into concrete occurs by a complex interaction of processes. Chloride solutions, arising from exposure to a marine environment, airborne salts or the application of deicing salts, can enter the concrete through the following mechanisms:

- diffusion;
- capillary suction (where the concrete is not fully saturated);
- hydrostatic pressure;
- ingress through cracks and joints.

Diffusion through the pore water will be the main mechanism for chloride ingress in situations where the concrete can be assumed to be fully saturated. Although some of the chlorides become chemically combined by the cement hydrates; free chlorides continue to migrate through the pore water. Over time the chloride concentration profile, that is the change of

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chloride concentration with depth into the concrete, takes up a shape that is consistent with diffusion theory (Figs. 5.8-5 and 5.8-6).

Chloride ingress through diffusion is commonly modelled using Fick's second law. Choice of appropriate boundary conditions gives the following:

$$C_x - C_b = (C_s - C_b) \cdot [\ 1 - \mathrm{erf}\,(x\ /\ \{2 \cdot (D \cdot t)^{0.5}\}\ )\ ] \qquad (5.8\text{-}3)$$

where:

C_x is the chloride at depth x.

C_s is the chloride content at surface of the concrete.

C_b is the background chloride content (from the mix ingredients).

D is the apparent chloride diffusion coefficient.

t is the exposure period.

The apparent diffusion coefficient, D, is the only unknown in theory. However, the surface chloride level, C_s, is difficult to measure and may vary with time. The usual approach is therefore to fit a curve to measured data with C_s and D as the independent variables.

Thus Eq. 5.8-3 can be used to estimate D for an existing structure by calculating a best fit curve for measured chloride concentration data and extrapolating back to the surface to obtain C_s. These values can then be used in residual service life calculations. For new structures the diffusion coefficient and surface chloride level need to be related to the mix design, curing conditions and exposure condition. A significant amount of work has been done on this using data for laboratory concretes and real structures. Measurements using trial mixes and then upon the actual structure could verify the values obtained.

Apparent diffusion coefficients may fall with time due to the ongoing hydration of the cementitious component. There is very little change beyond the first 12 months for Portland cement concretes, but there can be a marked drop for pfa and ggbs concretes. The time dependence of the apparent diffusion coefficient can be modelled using a relationship of the form:

$$D = D_0 \cdot (\ t\ /\ t_0\ )^n \qquad (5.8\text{-}4)$$

where:

D is the apparent diffusion coefficient after time, t.

D_0 is the apparent diffusion coefficient after a reference period time, t_0.

n is the ageing factor. It represents how rapidly the diffusion coefficient of a particular concrete mix reduces with time due to the effects of hydration and related effects, such as ion exchange which occurs in seawater. This parameter is dependent on the cement type.

For Portland cement concretes the value of 'n' is typically about -0.25. Respective values for pfa and ggbs concretes are about -0.7 to -0.6, although these may depend on the actual composition of the cement. Apparent diffusion coefficients, and consequently rates of chloride ingress, for blended cements will therefore fall more rapidly with time than will occur for Portland cement concretes. The effect of the ageing factor 'n' on service life calculations is

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considerable, even for deterministic modelling. Larssen (1999) showed how a change in the value of 'n' dramatically altered the required depth of concrete cover, with all other parameters remaining constant, to achieve the required service life of 50 years. This example indicated the following variations:

Taking n = -0.6 gave a required depth of cover of 37 mm.

Taking n = -0.4 gave a required depth of cover of 70 mm.

Taking n = 0.0 gave a required depth of cover of 244 mm.

Markeset (2000) showed that the effect of a change in the value of 'n' on the chloride ingress profile was even more pronounced when the uncertainty in the influencing parameters is taken into account. Using the same parameters as in the above analysis by Larssen (1999), Markeset (2000) showed that assuming coefficients of variation (CoV) of 10% and 15% resulted in the following required depths of concrete cover:

Taking n = -0.6 & CoV = 10% gave a required depth of cover of 48 mm (up from 37 mm).

Taking n = -0.6 & CoV = 15% gave a required depth of cover of 56 mm (up from 37 mm).

Taking n = -0.4 & CoV = 10% gave a required depth of cover of 84 mm (up from 70 mm).

Taking n = -0.4 & CoV = 15% gave a required depth of cover of 92 mm (up from 70 mm).

Where CoV = the standard deviation of the probability distribution/its mean value.

Corrosion activation is assumed to have occurred when the chloride content, C_x, at the cover depth exceeds a threshold level, C_{th}. Thus a concrete with the greatest resistance to reinforcing bar corrosion will have a combination of a high resistance to chloride migration, a high capacity for chloride binding and a high corrosion initiation threshold level, C_{th}.

The required depth of cover necessary to achieve a specified service life is sensitive to the corrosion initiation threshold level, C_{th}. A concrete which increases the chloride content required at the concrete-reinforcing bar interface to initiate corrosion (i.e. a high threshold level, C_{th}) will, all other matters being equal, need a smaller depth of cover to achieve the same service life as a concrete with a lower threshold level, C_{th}.

In addition, the concrete itself can also influence the surface chloride level, C_s. Accordingly, concrete mixes that help to reduce C_s are beneficial.

In practice, the chloride problem is further complicated by the fact that values of C_s and D may be time-dependent, and D is not uniform with depth. Furthermore, the most rapid chloride penetration has been observed in conditions where the concrete is not fully saturated, and ingress through capillary suction is also influential. As it is recognised that the conditions prevailing in practice are such that the use of the error function equation is not scientifically correct; the equation is used empirically.

Values of apparent diffusion coefficient thus represent an integrated value over the period of exposure prior to the profile analysis, and include effects of chloride binding and spatial and temporal changes. Many investigations have addressed these difficulties, allowing ways to be developed to enable prediction of chlorides ingress rates for design purposes.

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The following graphs presented in Figs. 5.8-2 and 5.8-3 give examples of predicted chloride ingress over time for nominal Portland cement and pfa concretes where the effects of the time dependency of the diffusion coefficient are illustrated.

Surface chloride level: The concentration in the concrete surface may be greater than that in the surrounding environment. Wetting and drying cycles can lead to a build-up of salts at or near the surface of the concrete as can physical and chemical absorption of chlorides and the inhomogeneous nature of the concrete. The surface chloride level may be higher for pfa and ggbs concretes due to enhanced binder capacity and higher binder volume. For structures affected by de-icing salts the surface chloride level will depend on the rate and frequency of application, and the location of the structure in relation to the source of the chlorides.

According to Bamforth (1999) the surface level remains approximately constant over time for concretes in marine spray and splash zones with the steady state value being attained relatively early in the lifetime of the concrete. However, there are significant variations in the steady state value over the surface of the concrete. Concrete type also has a significant effect. The surface chloride level is generally higher for pfa and ggbs concretes due to enhanced chloride binding capacity and higher cement volume.

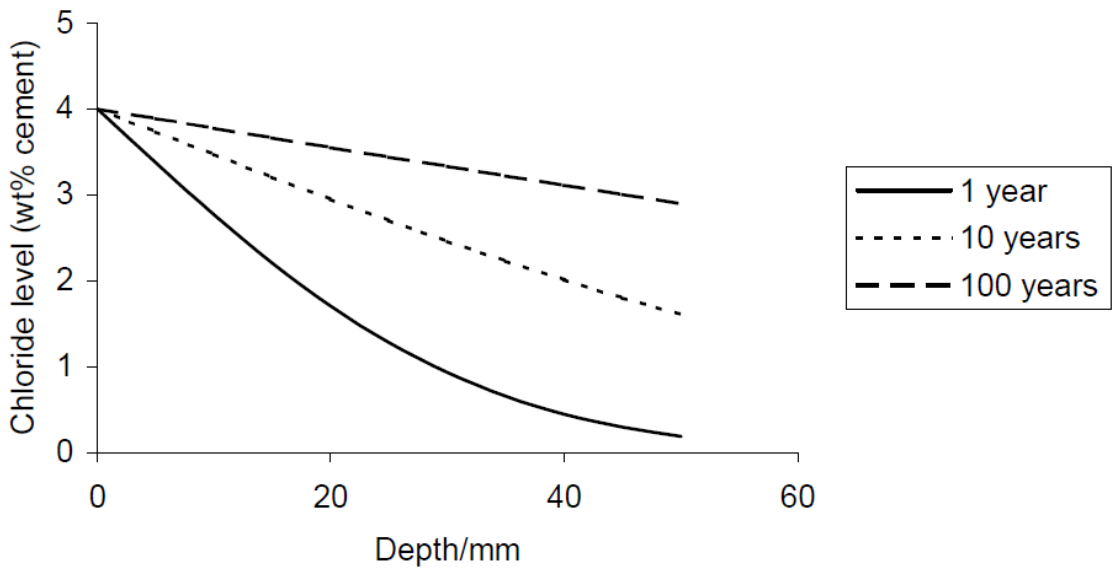

Fig. 5.8-2: Predicted chloride profiles for a C30 Portland cement concrete assuming a diffusion coefficient (D) after 1 year of 10^{-11} m^2/s and n = -0.26 [from Quillin BRE Report 434 (2001).

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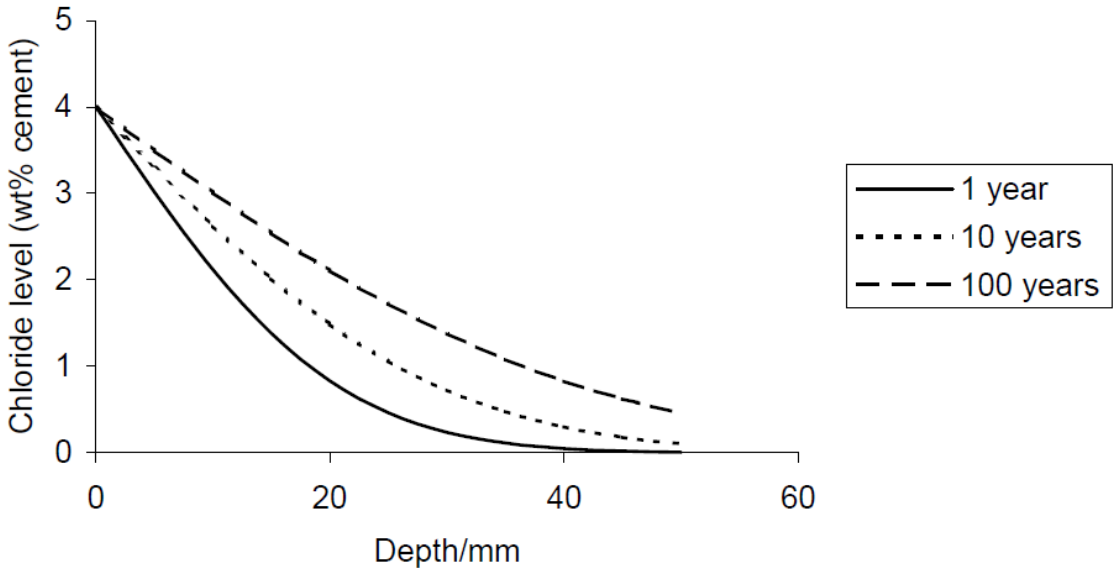

Fig. 5.8-3: Predicted chloride profiles for a C30 15%pfa concrete assuming a diffusion (D) coefficient after 1 year of 4×10^{-12} m^2/s and n = -0.7 [from Quillin BRE Report 434 (2001)].

The HETEK programme (1997) gives the following information for concrete exposed to chlorides:

For sheltered concrete:

- Surface chloride levels increase with time and can reach 0.5% by mass of concrete.
- The surface chloride level will decrease linearly with height above road level and logarithmically with distance from the road.
- The surface chloride level will have a maximum value on the leeward face.
- The apparent diffusion coefficient will be significantly lower than in wet concrete. It has been suggested that assuming a constant value for (D · t) should be used for modelling purposes.

For exposed concrete:

- The surface chloride level can reach 2% by mass of concrete and a steady state is likely to have been reached after a few seasons.
- The surface chloride level will decrease linearly with height above road level and logarithmically with distance from the road.

The *fib* Model Code for Service Life Design (MC-SLD) [*fib* Bulletin 34 (2006)] includes models for the verification of service life design for chloride induced corrosion of reinforcement occurring in initially uncracked concrete.

The *fib* MC-SLD [*fib* Bulletin 34 (2006)] also presents models for a partial factor method for chloride induced corrosion of reinforcement, with the treatment again assuming that the induced damage originates in initially uncracked concrete.

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Additional information upon chloride induced corrosion of reinforcement and some models to represent this process are given in the CONTECVET user manual for assessing residual service life of concrete structures affected by corrosion [CONTECVET (2000)].

5.8.4 Other mechanisms – frost attack

The mechanisms and processes responsible for frost attack are not well understood. There are a number of different theories as to how the different forms of freeze – thaw damage occur (refer Section 5.4.4.6). Susceptibility to damage is affected by the composition, porosity, moisture content and age of the concrete, together with the environment of exposure (including factors such as temperature cycles, wet-dry cycles and the presence of salt).

A two-stage process has been suggested for internal freeze-thaw-induced deterioration. The initiation period is the time taken for a critical degree of saturation to be reached. This is a factor which can be determined experimentally. Concrete is assumed to be highly resistant to frost action where its degree of capillary saturation (S_{cap}) remains below the critical level (S_{cr}). S_{cap} depends on the properties of the concrete and the environment of exposure.

Where S_{cap} exceeds the critical value, the risk of damage is substantial and is likely to occur following the first freeze-thaw cycle.

The following model has been suggested [Duracrete (1998)]:

$$F = S_{cr} - S_{cap} = S_{cr} - d - e \cdot \log (t \cdot . Y) \qquad (5.8\text{-}5)$$

Where ‘t’ is elapsed time, with ‘d’ and ‘e’ being constants that can, in principle, be derived from laboratory experiments. ‘Y’ is a correction factor to allow for intermittent wetness in practice. Thus, initiation is likely to occur when F takes on a negative value.

The following model has been suggested [Duracrete (1998)] as a basis for predicting the time for the development of a certain level of frost damage.

$$t_{cr} = (W_{cr} \cdot n) / (k_1 \cdot k_2 \cdot W_n \cdot x \cdot n^{k3}) \qquad (5.8\text{-}6)$$

where:

t_{cr} is the time taken to reach a the critical level;

W_{cr} is the critical degree of scaling;

n is the number of freeze-thaw cycles in a laboratory test;

W_n is the actual degree of scaling per cycle;

x is the number of freeze thaw cycles in a year for the service environment.

k_1, k_2 and k_3 are correction factors to take account of the fact that the relationship between the amount of scaling and the number of freeze-thaw cycles is affected by various concrete and environmental parameters and need not be linear.

The following general equation has been proposed [RILEM Report 14 (1996)] as a model for the strength reduction at the surface of a concrete element due to frost attack:

$$f_{ck}(d) = f_{ck} \{ 1 - [1 - (d / H)^n] \} \qquad (5.8\text{-}7)$$

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where:

f_{ck} is the characteristic compressive strength of undamaged concrete;

$f_{ck}(d)$ is the characteristic compressive strength of concrete at depth, d;

d is the depth;

H is the depth of influence;

n is an index related to the number of freeze-thaw cycles or time.

For structural design the loss in strength at the edge could be treated as a decrease in the design strength or as a reduction in the cross sectional area of the concrete. A number of other models for freeze-thaw induced damage are summarised in the Duracrete reports (1998).

The *fib* Model Code for Service Life Design (MC-SLD) [*fib* Bulletin 34 (2006)] includes models for the verification of service life design for (a) frost induced internal damage originating in uncracked concrete and (b) frost induced salt surface-scaling originating in uncracked concrete. The terminology used in the *fib* MC-SLD for these two conditions is somewhat inconsistent; with these two forms of deterioration also being referred to as (a) freeze-thaw attack without de-icing salts (i.e. de-icing salts are not present) and (b) freeze-thaw attack with de-icing salts (i.e. de-icing salts are present).

The *fib* MC-SLD [*fib* Bulletin 34 (2006)] also presents models for a partial factor method for frost induced damage, with the treatment again assuming that the induced damage originates in initially uncracked concrete.

Additional information upon freeze-thaw deterioration and some possible models to represent this process are given in the CONTECVET user manual for assessing residual service life of concrete structures affected by frost [CONTECVET (2001)].

5.8.5 Other mechanisms – sulfate attack

Atkinson and Hearne (1990) developed a model to predict the effects of sulfate-containing groundwaters on the service life of concrete. In this model cracking and delamination is assumed to occur as a result of stresses induced by sulfate ingress (through diffusion) and subsequent expansive reaction (leading predominantly to the formation of the mineral ettringite) with aluminate-containing cement phases. As the concrete cracks and delaminates new surfaces are exposed. The depth of degradation is assumed to increase linearly with time and depends on the concentrations of sulfate ions and aluminates, the rate of diffusion and reaction and the fracture energy of the concrete. Some of the values require for the model have to be determined by laboratory experiments. Others are 'typical' values for specific concrete types and mix designs, but these cannot be readily derived. Atkinson and Hearne (1990) used this model to predict degradation rates for Portland cement and sulfate-resisting Portland cement concretes in 0.01 M sulfate solutions.

Time to failure models for the thaumasite form of sulfate attack (TSA) and for delayed ettringite formation (DEF) and do not appear to have been developed to any degree. TSA is an external chemical attack (refer Section 5.4.5.2), whereas DEF is an internal deleterious chemical reaction (refer Section 5.4.5.6).

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5.8.6 Other mechanisms – alkali-aggregate reaction (AAR)

Notionally the two-stage modelling concept of initiation and propagation can be applied to alkali-aggregate reaction (AAR) which occurs where alkalis in the pore water solution react with siliceous constituents of the aggregate to form an expansive gel which, on imbibition of water, can lead to cracking of the concrete (refer Section 5.4.5.5). However, because of the complexity of the process and the number of contributory factors, suitable models have not yet been developed or verified for this deterioration mechanism. Typically the approach taken during durability design is to seek avoid deterioration processes such as AAR, and the associated problems, by adopting the 'design-out' strategy.

Additional information upon alkali-silica reaction (ASR) deterioration and some possible models to represent aspects of this process are given in the CONTECVET user manual for assessing residual service life of concrete structures affected by ASR [CONTECVET (2000)].

5.8.7 Other mechanisms – leaching

The mechanisms and processes responsible for leaching are described in Section 5.4.5.4.

Additional information upon leaching deterioration and some possible models to represent a number of aspects related to this behaviour are given in the CONTECVET user manual for assessing residual service life of concrete structures affected by leaching [CONTECVET (2000)].

RILEM 14 briefly discusses modelling surface deterioration by leaching [RILEM (1996)].

5.8.8 Other mechanisms – surface weathering and abrasion by ice

Procedures for the modelling surface deterioration due to weathering and the abrasion of concrete by ice are briefly discussed in RILEM Report 14 [RILEM (1996)].

5.8.9 Application of a deterministic model

The requirement for performance during the service life can be written using the following simplified limit state function:

$$R(t) - S(t) \geq 0 \qquad (5.8\text{-}8)$$

where the resistance, R, must be greater than or equal to the load, S, at all times up to the required or target service life, t_{req}. In the case of structural design the resistance is assumed to be constant and the load, although fluctuating, is often assumed to be characterised by a single value. Partial safety factors are then used to take account of variability and uncertainties, leading to the design values. The limit states can refer to structural safety, serviceability, functionality, comfort, aesthetics, etc. The nature of R and S will depend on the performance or serviceability limit state being considered.

In the case of service life design for durability for carbonation induced reinforcement corrosion; the resistance parameter (R) is the depth of cover concrete and the load parameter (S) is the depth of carbonation, with this assumed to be increasing with time on the basis of Eq. (5.8-1) or Eq. (5.8-2).

In the case of service life design for durability for chloride based corrosion; the resistance parameter (R) is the chloride level initiating corrosion (referred to as the chloride threshold

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level – refer Section 5.4.7.6) and the load parameter (S) is the chloride concentration at the depth of the reinforcement.

Service life predictions can be made using single or mean values for the resistance parameter (R) and the load parameter (S). In reality both of these input parameters will have an associated scatter, which may be represented by probability distributions. For example if the depth of carbonation and the depth of cover concrete are represented by single values; use of a mean value implies that 50% of the respective parameter values will be larger than the quoted mean value and 50% will be smaller.

Thus, for simplicity if it is assumed that the depth of carbonation at a particular moment in time could be represented by a single (maximum) value; when the depth of carbonation equals the mean depth of cover of the outer layer of reinforcement, 50% of this reinforcement would notionally already be depassivated. Accordingly such reinforcement would be at risk of corrosion as the depth of carbonation would exceed the depth of cover.

Fig. 5.8-4 illustrates the general version of this case; showing the overlap between the depth of carbonation, represented by a single (maximum) value, at a particular moment in time and the depth of cover to the reinforcement. If the mean values for the depth of carbonation and cover are used for such an evaluation, 50% of the reinforcement will notionally already be depassivated when this state is reached.

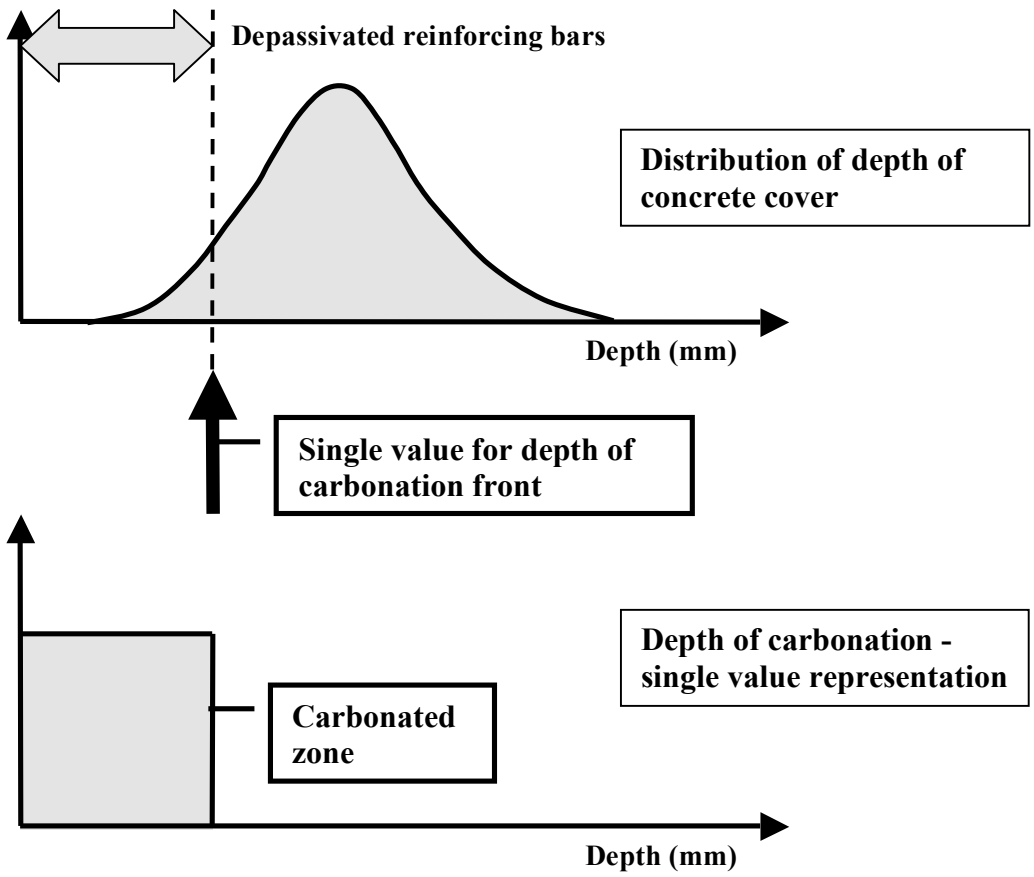

Fig. 5.8-4: Overlap between the depth of carbonation, represented by a single (maximum) value, at a particular time and the depth of cover to the reinforcement.

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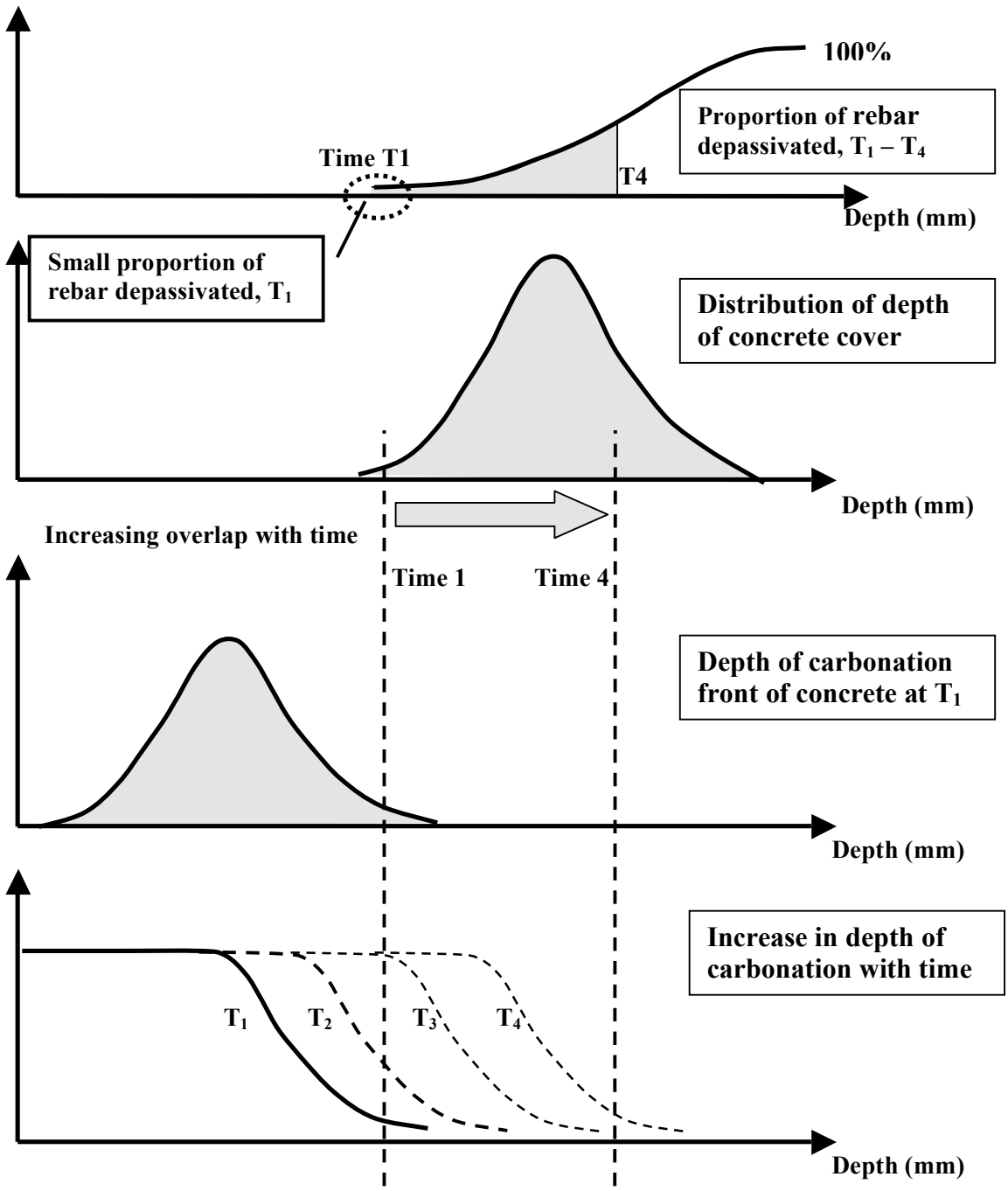

Fig. 5.8-5: Increase in overlap between the depth of carbonation and the depth of cover to reinforcement with time, leading to depassivation of more reinforcement.

The rate at which corrosion was actually progressing in the depassivated reinforcement would depend upon a variety of environmental factors, but primarily upon the amount of moisture available and the time of wetness. If the internal relative humidity of the concrete was low, as would usually be expected in an indoor environment, very little corrosion would be taking place even though the steel reinforcement was in a depassivated condition.

Fig. 5.8-5 portrays the increase in overlap between the depth of the zone of carbonated concrete (S) and the depth of cover to reinforcement (R) with time, leading to the depassivation of a greater proportion of the reinforcement with time.

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If the service life durability design was based upon 5% characteristic values for the loads (S) and resistance (R) properties; that is where for loads (S) 5% of the values for the depth of carbonation would be greater than the value used in the calculations and for resistance (R) 5% of the values for depth of cover would be less than the value used in the calculations; the overall probability of failure or non-compliance would be in the order of 0.25%. This criterion is too conservative for durability design and would lead to uneconomic design.

Accordingly it would be desirable to employ a different characteristic value for durability design purposes. If considered simplistically, this deterministic approach would give a probability of durability 'failure' or non-compliance in the order of those portrayed in Table 5.8-2. Use of a percentile value of 30% would give an overall probability of durability non-compliance in the order of 9%; which is likely to be acceptable for a service life condition not having wider implications relating to structural safety. If the durability 'failure' or non-compliance relates to depassivation/the initiation of reinforcement corrosion as the criterion defining the end of the technical service life, there would be a subsequent period whilst corrosion propagation occurred before any significant structural deterioration was caused.

Table 5.8-2: Illustration of the notional change in the overall probability of failure or non-compliance as the overlapping characteristic values for the load (S) and resistance (R) properties change from a 5% percentile value to a 30% percentile value.

Case	Percentile %: Load, S	Percentile %: Resistance, R	Notional probability of 'durability failure' or non-compliance	Standard deviation
Case 1	5	5	0.25%	SD = 1.64
Case 2	10	10	1%	SD = 1.28
Case 3	30	30	9%	SD = 0.52

Note to Table 5.8-1: *The standard deviation parameters quoted assumes that a normal Gaussian probability distribution applies to the depth of the carbonation front in the concrete (S) and to the depth of cover to reinforcement (R). The two factors are assumed to be independent variables.*

In these circumstances it may be appropriate to incorporate a 'lifetime' safety factor in the durability estimates made to ensure that there is an adequate 'time margin' associated with the envisaged design service life (refer Box 5.1-2, Section 5.1). The design service life, t_d, would then be given by:

$$t_d = \gamma_t \cdot t_g \tag{5.8-9}$$

where t_g is the required or target service life and γ_t is the lifetime safety factor.

The safety factor, and consequently the design service life, will increase as the probability of failure that is considered to be acceptable decreases. These issues are discussed in detail in RILEM 14: *Durability design of concrete structures* [RILEM (1996)].

5.8.10 Application of a probabilistic model

Whilst service life predictions can be made using single or mean values for input parameters (as noted above), factors such as environment and the physical properties of the materials can vary greatly. The resistance, load and service life are therefore unlikely to be known with certainty and are better represented by distributions rather than just by using single, mean or other deterministically derived values. In principle service life can then be predicted in prob-

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abilistic (or stochastic) terms. These concepts are illustrated by Fig. 5.8-6. Alternatively appropriate safety factors, or a safety margin, would need to be introduced to take account of variability and uncertainties in the calculation.

The top half of Fig. 5.8-6 shows the load, S(t), increasing and the resistance, R(t), constant over time. Both are, at any given time, represented by distributions. The dashed lines represent their mean values over time. The mean service life is then given by the intersection between these two mean value lines. However, there is a finite probability of durability failure at earlier ages represented by the overlap between the two distributions. The probability of failure increases with time as shown in the lower half of Fig. 5.8-6. The acceptable probability of failure occurring within the target service life will depend on the nature of the structure and the client's needs. Where the potential consequences of durability failure are large, and the accepted failure probability correspondingly small, the mean service life could be substantially greater than the target service life. However, this will depend on the nature of the probability distributions for R and S.

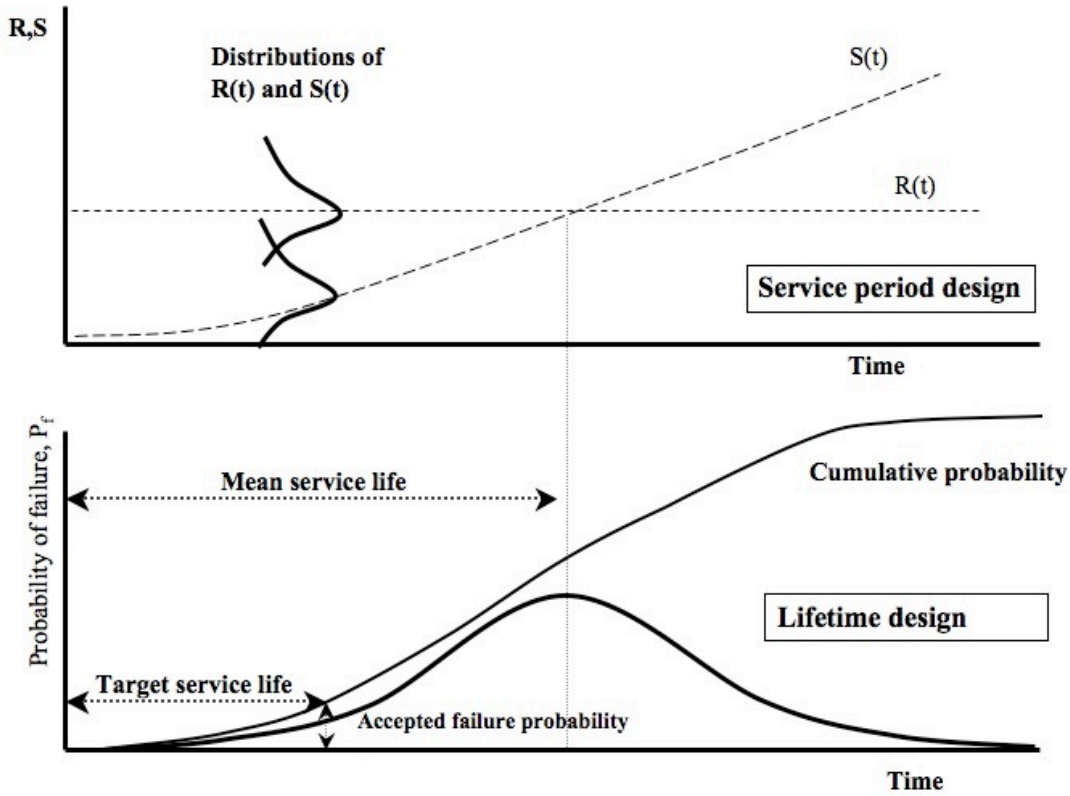

Fig. 5.8-6: Service period design and lifetime design [from Quillin (2001), adapted from Siemes & Rostam (1996)]

Where a probabilistic approach is used assumptions must be made about the form of distributions. Service life is often assumed to follow a log-normal distribution, i.e. the service life is distributed normally on a logarithmic time scale, with a peak in the probability of failure in a given year, followed by a gradual decrease towards zero as the service life increases. The resistance and the load may be treated as being normally distributed. Cover to reinforcement can be treated as having an exponential distribution.

Durability design can be presented in two (theoretically equivalent) ways:

- intended service period design;
- lifetime design.

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These approaches use the same information and will lead to the same result.

Examples of the use of probabilistic methods in durability design are given in RILEM 14: *Durability design of concrete structures* [RILEM (1996)], in the suite of publications produced by the Duracrete project [Duracrete 1 to 7 (1997 to 1999)] and in the *fib* Model Code for Service Life Design (MC-SLD) [*fib* Bulletin 34 (2006)]. Further details of the theory and application of reliability theory are given in the standard texts on the subject such as Thoft-Christensen and Baker (1982), Ang and Tang (1984) and Melchers (1999).

5.8.10.1 Intended service period design approach

In this approach it is not permissible for the limit state to be reached within the target service life. This approach is comparable with that used in conventional structural design and is summarised in the top part of Fig. 5.8-6. The concept can be represented by the formula:

$$P_{f,t} = P\{R(t) - S(t) < 0\}_t \quad \leq \; P_{target} = \Phi(-\beta) \qquad (5.8\text{-}10)$$

where:

$P_{f,t}$ is the probability of failure of the structure within the target service life, t_g.

P_{target} is the accepted maximum value of the probability of durability failure.

Φ is the standard normal distribution function

β is the reliability index. It is the number of standard deviations from the mean of a normal distribution outside which the area under the curve represents the probability of failure or non-compliance. The β value adopted is dependent upon the consequences of failure and this will be high if the potential consequences of failure or non-compliance are large.

If the onset of corrosion is used as the determining factor in establishing the service life, this becomes the time taken for (a) the carbonation front to reach the depth of the reinforcement or (b) the chloride ion concentration at the reinforcement to exceed a minimum threshold value. In these cases the following expressions can be used:

For carbonation induced corrosion:

$$P_{f,t} = \{X_c(t) - X_f < 0\}_t \quad \leq \; 0.1 \qquad (5.8\text{-}11)$$

For chloride induced corrosion:

$$P_{f,t} = \{C_x(t) - C_{th} < 0\}_t \quad \leq \; 0.01 \qquad (5.8\text{-}12)$$

where:

C_x is the chloride level at the reinforcement.

C_{th} is the threshold level for corrosion.

X_c is the carbonation depth.

X_f is the depth of cover to the reinforcement.

$P_{f,t}$ is the probability of corrosion initiation within the target service life, t_g.

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In relation to carbonation induced corrosion of reinforcement a value of $P_{f,t} = 0.1$ has been used in Eq. 5.8-11. This corresponds to a β value of 1.3. Whilst this β value is relatively low, with an associated high probability of occurrence, the depassivation of the reinforcement by carbonation is considered to be a serviceability limit state. Subsequent corrosion propagation would generally be expected to take many years before significant deterioration was caused affecting the structural load capacity (and therefore the ultimate limit state behaviour) of the members concerned. However, it has been suggested that in other circumstances other β values might be more appropriate. The suggestions made include:

$\beta = 1.0$ giving a $P_{f,t} = 0.160$; for an XC1 environment (refer Table 5.6-2).

$\beta = 1.5$ giving a $P_{f,t} = 0.067$; for XC2 and XC3 environments.

$\beta = 2.0$ giving a $P_{f,t} = 0.023$; for an XC4 environment.

In relation to chloride induced corrosion a value of $P_{f,t} = 0.01$ has been used in Eq. 5.8-12. This corresponds to a β value of 2.3. A higher β value has been used than the case for carbonation induced corrosion because of the fact that chloride induced corrosion may result in faster localised pitting corrosion, which can be far more structurally damaging. This type of corrosion may potentially cause a rapid deterioration of the structural load capacity of the members concerned.

However, the *fib* Model Code for Service Life Design (MC-SLD) [*fib* Bulletin 34 (2006)] suggests the use of a β value of 1.3 for reinforcement depassivation in all exposure environments; that is for exposure environments XC (carbonation induced corrosion), XS (chloride induced corrosion associated with seawater) and XD (chloride induced corrosion associated with chlorides other than in seawater).

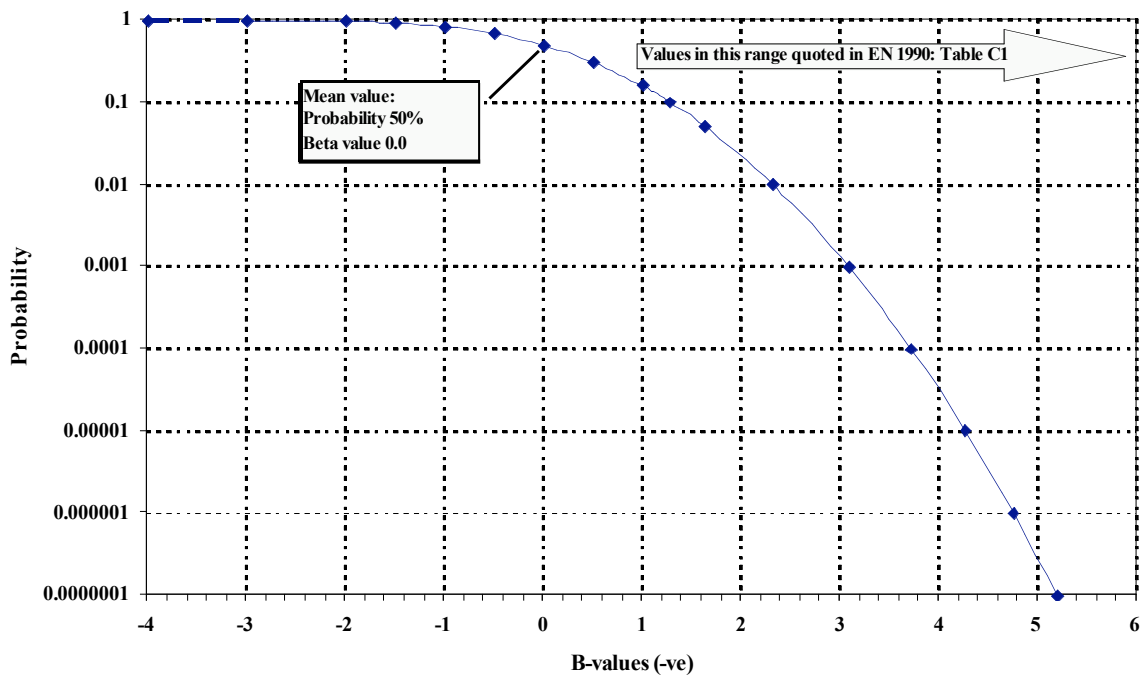

Fig. 5.8-7: Relationship between probability P_f and β-values

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Fig. 5.8-7 illustrates the relationship between probability of occurrence, P_f, and β-values (reliability index) assuming a normal Gaussian distribution and drawing upon information presented in Table C1 of EN 1990 (CEN).

In the case of service life design for durability for carbonation induced reinforcement corrosion; the resistance parameter (R) is the depth of cover concrete and the load parameter (S) is the depth of carbonation, with this assumed to be increasing with time on the basis of Eq. 5.8-1 or Eq. 5.8-2.

In the case of service life design for durability for chloride based corrosion; the resistance parameter (R) is the chloride level initiating corrosion (referred to as the chloride threshold level – refer Section 5.4.7.6) and the load parameter (S) is the chloride concentration at the depth of the reinforcement.

The *fib* Model Code for Service Life Design (MC-SLD) [*fib* Bulletin 34 (2006)] also presents models for a probabilistic service life design method for a number of forms of deterioration of concrete structures. These are:

- Carbonation induced corrosion of reinforcement; with the treatment assuming that the induced damage originates in initially uncracked concrete (refer Chapter 3.1 and Annex B1 of the MC-SLD).
- Chloride induced corrosion of reinforcement; with the treatment assuming that the induced damage originates in initially uncracked concrete (refer Chapter 3.2 and Annex B2 of the MC-SLD).
- Frost induced internal damage to concrete; with the treatment assuming that the induced damage originates in initially uncracked concrete (refer Chapter 3.5 and Annex B3 of the MC-SLD).
- Frost induced salt surface-scaling; with the treatment assuming that the induced damage originates in initially uncracked concrete (refer Chapter 3.6 and Annex B4 of the MC-SLD).

5.8.10.2 Lifetime design approach

In this approach (portrayed in the bottom half of Fig. 5.8-6) the reliability of the structure is related to the probability that the target service life will be exceeded. In this case the probability of failure is given by the expression:

$$P_f = P\ \{L < t_g\} \quad < P_{target} = \Phi(-\beta) \qquad (5.8\text{-}13)$$

Thus the actual service life, L, must be greater than or equal to the target service life, t_g

where:

$P_{f,t}$ is the probability of failure of the structure within the target service life. For the ultimate limit state (collapse) Eurocode 1 gives a probability, P_{target}, of 7 x 10^{-5} for a service life of 50 years. However, much higher levels of risk may be tolerated where the consequences of failure are less severe.

P_{target} is the accepted maximum value of the probability of failure.

Φ is the standard normal distribution function.

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β is the reliability index. It is the number of standard deviations from the mean of a normal distribution outside which the area under the curve represents the probability of failure (this is dependent on the consequences of failure and will be high if the potential consequences of failure are large).

5.8.11 Application of the partial factor method

The *fib* MC-SLD [*fib* Bulletin 34 (2006)] also presents models for a partial factor method for a number of forms of deterioration of concrete structures. These are:

- Carbonation induced corrosion of reinforcement, with the treatment assuming that the induced damage originates in initially uncracked concrete (refer Annex C1 of the MC-SLD). An example of this approach is included. The partial factor method is based on the full probabilistic design approach presented in Annex B1 of the MC-SLD.
- Frost induced damage to concrete, with the treatment assuming that the induced damage originates in initially uncracked concrete (refer Annex C2 of the MC-SLD).

5.8.12 Reinforcement corrosion – Deterministic versus probabilistic service life design models

The merits of the probabilistic approach to durability design are illustrated by the following example of a marine structure. Two different environments are considered, representing yearly average temperatures of 10 °C (Northern Europe) and 30 °C (Middle East), respectively. The requirement is for a design service life of 50 years, being taken as the length of the initiation period, i.e. the time to corrosion initiation due to chloride ingress.

Fig. 5.8-8 depicts the required concrete covers for the two environments, based on a traditional deterministic approach. The values are 34 mm for the Northern Europe climate and 68 mm for the Middle Eastern climate.

Fig. 5.8-9 highlights the fact that the deterministic approach only provides a 50% probability of achieving the required 50 year service life without reinforcing bar corrosion being initiated. This fact is often overlooked in the standard approaches to durability design. If, say only a 10% risk of having corrosion initiated before 50 years is considered acceptable, then much larger covers are required, as seen from Fig. 5.8-9. The values are:

- 49 mm for the Northern Europe climate (up from 34 mm as per the deterministic model);
- 91 mm for the Middle Eastern climate (up from 68 mm as per the deterministic model).

The deterministic approach used here is based on mean values of the governing parameters. In the probabilistic approach the mean values and their known or assumed standard deviations are used together with the relevant distribution functions. This latter approach makes it possible to relate cover thickness to probability of corrosion, and also to quantify the consequences of choosing different levels of corrosion risk. These consequences relate not only to concrete quality and cover thickness, but more importantly to the economic consequences.

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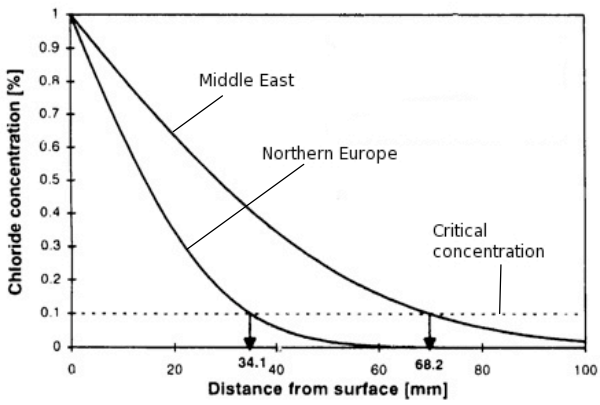

Fig. 5.8-8: Deterministic approach [Rostam: fib Bulletin 3 (1999)]

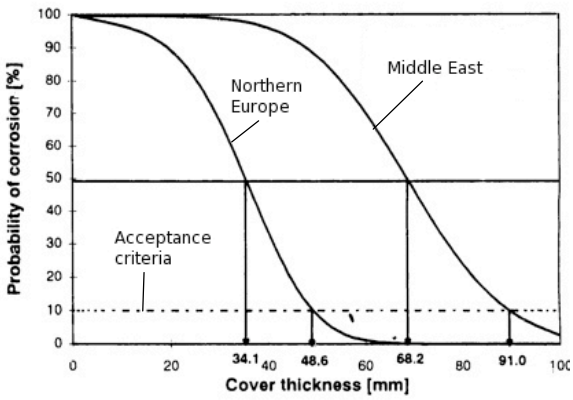

Fig. 5.8-9: Probabilistic approach [Rostam: fib Bulletin 3 (1999)]

Note to Fig. 5.8-8: *The above defines the concrete cover required to ensure a 50 year service life, assuming a chloride threshold value of 0.1% by weight of concrete.*

Note to Fig. 5.8-9: *The deterministic approach based upon the mean depth of cover results in a 50% probability of corrosion initiation after a service life of 50 years. Specifying that the probability of corrosion initiation after 50 years should be limited to 10% requires a considerably larger cover.*

5.9 Factorial approach to estimating service life

5.9.1 Introduction

A number of techniques can be used to provide a service life forecast. For example, service life can be predicted based on models which represent the physical deterioration processes which affect concrete in different environments – as has been described in Section 5.8. However, an alternative and relatively simple alternative is the factorial approach to service life forecasting described by ISO 15686-1 [ISO (2000)]. This approach is applied generally to buildings and constructed assets and is therefore applied to a wide range of construction materials, manufactured products and building components.

The description of the application of the factorial approach to service life forecasting and design of reinforced concrete structures that is set down in this Section has been abridged from the BRE publication: *Reinforced concrete service life design: Service life forecasting and enhancement* [Nolan et al., BRE (2006)].

5.9.2 Factor method for predicting service life

This approach considers a standard ('reference') service life for components with assumptions made for its quality and its incorporation within the structure. Adjustment coefficients (which can be either positive or negative) are used where these assumptions are not met. The concept is summarised by the following equation (taken from ISO 15686-1):

$$ESLC = RSLC \cdot A \cdot B \cdot C \cdot D \cdot E \cdot F \qquad (5.9\text{-}1)$$

ESLC = Estimated service life. It is the expected life of the particular component it its specific environment allowing for the factors (A to F) listed below.

RSLC = Reference service life. The 'reference service life' here refers to the period of time that the component or assembly can normally be expected to last under specified service

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conditions. It may be derived from modelling, experience, accelerated testing, data from the manufacturer or occasionally from product standards.

Eq. 5.9-1 provides a simple summary of the factors that can affect the service life of materials or components in circumstances where they do not meet the levels specified in manufacturers' recommendations or in the appropriate codes and standards. These factors are represented by A to F in equation 5.9-1 as follows:

A = quality of materials factor,

B = design level factor,

C = work execution level factor,

D = environmental conditions factor

E = in-use conditions factor,

F = maintenance conditions factor.

Appropriate values for these factors need to be selected by the designer based on the particular circumstances of the project and previous experience with the materials in the envisaged service life environment. The designer will also need to take account of the available information upon the design concept, structural detailing, service environment(s), workmanship and proposed maintenance regime in order to make a judgement of the effects of these aspects of the design and construction process upon service life. Consequently the approach needs to be applied by suitably experienced designers.

In using Eq. 5.9-1, factors A to G should be applied by the designer based on the particular circumstances of the project, previous experience and available information. The factorial system allows the designer to identify deviations from the norm or factors leading to a specific concern. It is a guide to help select appropriate approaches to meeting the required service life. The designer should use experience to inform decisions and give 'best' rather than conservative estimates. If in doubt, expert assistance should be sought.

Generally speaking, factors should be left at 1 when the designer has no information that the design or conditions are contrary to that assumed in the codes and standards used. Table 5.9-1 provides additional guidance on the use of these factors in the context of reinforced concrete construction. An example of using the factorial approach for a hypothetical concrete element is given in Box 5.9-1.

The iterative nature of service life design means that it cannot readily be illustrated in a simple schematic diagram by which a designer can ensure that the required service life design is met. However, by using the approach outlined here, it is expected that designers will come to understand better the choices they face and will have to make fewer or no iterations.

Where the ESLC falls below that required, strategies for increasing service life can be used to assist the designer in selecting or modifying the approach used in protecting the structure. There are a number of basic technical strategies to choose from when seeking to increase service life. For the purposes of this example these have been categorised as follows:

- design-out;
- enhance basic material resistance or provide additional resistance;
- improved site practice and quality control during construction;
- planned maintenance;

Combinations of these options can also be considered.

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5.9.3 Combining additional protective measures to extend service life

Table 5.9-2 shows how additional resistance measures can be combined to enhance service life. The measures considered are categorised into three types:

1. basic material resistance type (e.g. water-cement ratio, cover, binder type, etc.);
2. measures which increase the threshold for corrosion (e.g. corrosion inhibitors, etc.);
3. measures which give the surface of the concrete added resistance (e.g. coatings, etc.).

When combining measures to enhance service life it is most effective to choose measures of different types, rather than choosing measures of the same type. The overall effect of a number of different factors is assessed by taking their product, as shown below:

$$Fm = Fm1 \cdot Fm2 \cdot \ldots \cdot Fmn \qquad (5.9\text{-}2)$$

Table 5.9-1: Factors that can affect service life [Ref: BRE IP3/06 (2006)]

Factor	Summary of BS ISO 15686-1 definition [1]	Application to reinforced concrete
A Quality of materials	This factor represents the component as supplied to site. It is a measure of the quality of the design of the component itself.	Likely to be equal to 1 where reinforcing bar and the concrete quality management regimes are used. For demanding concretes (including self compacting and lightweight), or demanding placement conditions (pumping, slip forming), extra quality measures may have to be introduced. Values in the range 0.8 to 1.05 suggested.
B Design levels	This factor relates to the nature of the protection from aggressive agents in the environment that the component will receive when installed in the structure	This allows the designer to further subdivide the chosen exposure classification, where there is evidence that the actual conditions depart from the norm suggested in the classification. Values in the range 0.8 to 1.1 suggested.
C Work execution levels	This represents the level of skill and control likely in site work. It is based on whether the site work is likely to be in accordance with manufacturers' recommendations and appropriately supervised. The assessment should be based on the likelihood of achieving the designed level of workmanship, including issues such as storage, protection during installation, number of trades required for each activity and site applied coatings.	Less than 1 where there are concerns about the quality of execution. If the BRE service life design system recommendations on execution are implemented a value of 1 may be appropriate. Theoretically this factor could be as low as 0 [2]. If assessed as less than 0.8, measures should be taken to improve construction Values in the range 0.8 to 1 suggested

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Table 5.9-1: Factors that can affect service life [Ref: BRE IP3/06 (2006)]

Factor	Summary of BS ISO 15686-1 definition [1]	Application to reinforced concrete
D Indoor environmental conditions	This factor indicates the environmental conditions assessment, the exposure to agents of degradation and their severity. The general use of the building should be taken into account within this factor, together with relevant local aspects (e.g. locations subject to wetting such as kitchens and bathrooms). Once again, it is the deviation from assumed conditions in input data that is measured by the factor.	Values of less than 1 should be considered where there are concerns that the environment is particularly aggressive or not well characterised. Deviations from conditions assumed in the chosen exposure factor should be assessed. Choose either factor D or E but not both (the unused factor should be 1). Values in the range 0.8 to 1.2 suggested. Most likely exposure conditions are XC and XA in EN 206.
E Outdoor environmental conditions	A general designation may be adequate (e.g. coastal) for this factor. However, when considering specific components at the detailed design stage the micro-environment should be taken into account (e.g. southern elevation, wind suction or uplift at high level and salt spray zone). A combination of the agents can have a critical effect (e.g. a combination of wetting and freezing). Note also that components may be exposed both to external weathering and aggressive ground.	Values of less than 1 should be considered where general factors, e.g. distance from the coast, height above sea level and orientation of elevation may make the environmental classification more severe than the conditions assumed. Choose either factor D or E, but not both (the unused factor should equal 1). Values in the range 0.8 to 1.2 suggested.
F In use conditions	This factor represents the effects on service life arising from the use of the building. The specific use of the space where the component is installed or the assembly constructed is likely to be relevant (e.g. delivery areas will be subject to mechanical impacts by vehicles). Deviation from the assumed conditions is measured.	Where the use of the building impacts on the likely service life. Generally 1, except where impact or abrasion feature in the deterioration mechanism. Values of 0.8 to 1 suggested.
G Maintenance conditions	This factor addresses the likely level of maintenance for a specific component. This will depend on the client's maintenance requirements, accessibility, etc. Cleaning should also be taken into account as this may introduce agents that themselves can lead to deterioration.	The likelihood that any specified maintenance will be carried out should be considered. The factor could be based on motivation to maintain – i.e. a client/user is more likely to maintain a building than a user who is renting. Generally set to 1 unless the design specifically requires maintenance. Values in the range 0.8 to 1 suggested.

Notes

1. BS ISO 15686-1 also allows factors A to G to exceed 1 in certain cases. Designers should only use values in excess of 1 if satisfied that they have information which allows them to responsibly relax the guidance provided by the basic design codes. The decision to use a factor in excess of 1 should be based on expert knowledge and relevant experience.
2. For example, an element where reinforcement showed through the cast surface in a project where initiation of corrosion was to be taken to represent failure.
3. Adopted from BRE publication BRE IP3-06: *Reinforced concrete service life design: Service life forecasting and enhancement* [Nolan et al.: BRE (2006)].

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Box 5.9-1: ESTIMATING SERVICE LIFE: WORKED EXAMPLE

Estimated service life using the reference standard life factors- Ref: BRE IP3/06 (2006)

ESLC = RSLC · A · B · C · D · E · F · G (Eq. 1: Box 5.9-1)

The building structure is designed for service life of 50 years (EN 206-1) and is likely to experience durability attack in the form of carbonation. Therefore the RSLC of a particular element will be 50 years.

A Quality of materials: Taken as 1 in this instance as the site is not isolated and is served by several ready mixed concrete plants which participate in an established quality management scheme for concrete suppliers. The project specification stipulates that reinforcing steel is to be obtained from a supplier which participates in an established quality management scheme for reinforcement suppliers.

B Design levels: Taken as 0.9 as the client/architect requires minimum slab thicknesses supported on irregularly positioned columns, fitting in with internal wall positions of the building. The design is judged to lack overall redundancy.

C Work execution levels: Taken as 0.9 as the concrete frame contract is expected to be let on the basis of lowest cost. Using the relationship between diffusion and cover depth, 0.9 represents an average achieved cover depth of 32.4 mm compared to a specified 40 mm cover depth, making no allowances for achieved cover concrete quality.

D Indoor environmental conditions: Not relevant, taken as 1

E Outdoor environmental conditions: Taken as 1 as carbonation risk in this case is not increased due to its geographic location/orientation.

F In use conditions: The building is expected to be used appropriately during its service life. Taken as 1.

G Maintenance conditions: Since the client will be the user, G is taken as 1.

ESLC = 50 years ·1 · 0.9 · 0.9 ·1 ·1 · 1 = **40.5 years** (estimated service life)

Looking at the above example, the ESLC could be increased to 45 years if the recommendations in the BRE service life execution recommendations [1] were implemented and C is taken as 1 as a result.

Note:

1. Adopted from BRE publication BRE IP3-06: *Reinforced concrete service life design: Service life forecasting and enhancement* [Nolan et al.: BRE (2006)]

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Table 5.9-2: Effectiveness of combinations of measures to enhance service life Ref: BRE IP3-06: Reinforced concrete service life design: Service life forecasting and enhancement [Nolan et al.: BRE (2006)

	Basic resistance					Surface effects						Corrosion threshold effects			
	Decrease w/c	Increase Cement Content	Ground Granulated Blast-furnace Slag	Pulverised Fuel Ash	Silica Fume	Surface Coating	Hydrophobes *Ingressed*	Anti-carbonation Coating	Controlled Permeability Formwork	Permanent Formwork	Sacrificial Layer	Admixed Corrosion Inhibitors	Stainless Steel	Galvanised Steel	Cathodic Protection
Decrease w/c ratio	X	B	B	B	B	B	DV	B	DV	B	B	B	B	B	B
Increased cement		X	DV	DV	DV	B	DV	B	DV	B	B	B	DV	DV	B
ggbs			X	RU	DV	B	DV	B	DV	B	B	B	B	B	DV
pfa				X	DV	B	DV	B	DV	B	B	B	B	B	DV
sf					X	B	DV	B	DV	B	B	B	B	B	DV
Surface coating						X	I	X	B	I	B	B	B	B	I
Hydrophobes							X	I	DV	I	I	B	B	B	B
Anti carb. coating								X	DV	I	B	B	B	B	B
CPF									X	I	I	B	B	B	B
Perm. formwork										X	I	B	B	B	B
Sacrificial layer											X	B	B	B	DV
Corr. inhibitors												X	RU	RU	RU
Stainless steel													X	I	RU
Galvanised steel.														X	RU
Cathodic protectio															X

Key to Table 5.9-2

B	Beneficial: Factors derived from Table 6 of BRE IP3-06 (Nolan et al. (2006)] can be summed without modification.
DV	Diminishing value with increase in level: Factors derived from Table 6 of BRE IP3-06 (Nolan et al. (2006)] should not be used in combination without reduction of total effect.
RU	Rarely used.
I	Incompatible.

Note to Table 5.9-2: *The above is presented to introduce the concept of combinations of measures to enhance service life. Reference would need to be made to BRE Information Paper 3-06 [Nolan et al. (2006)] to understand how to apply the approach developed within the BRE service life design system.*

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Table 5.9-3 shows the types of measure which could be employed to increase the service life of the structure after a number of years in service. It also gives an indication of the additional service life which could arise from their use. Table 5.9-3 should be used to support the development of the maintenance strategy during the initial design. However, it does not represent a guaranteed service life extension in every situation. It should not be used to justify maintenance actions currently being considered on an existing structure. Freeze thaw and chemical attack exposure conditions are not represented as reliable data are not available.

Table 5.9-3: Estimated additional service life (in years) from use of additional protection measures for conventional Portland cement concrete structure. Ref BRE IP3-06: Reinforced concrete service life design: Service life forecasting and enhancement [Nolan et al.: BRE (2006)]

	Exposure class from BS EN 8500-1									
		Carbonation-induced corrosion			Chloride-induced corrosion: De-icing salt exposure			Chloride-induced corrosion: Marine exposure		
	X0	**XC1**	**XC2**	**XC3/4**	**XD1**	**XD2**	**XD3**	**XS1**	**XS2**	**XS3**
Remedial Measure	No risk of corrosion	Dry or permanently wet	Wet, rarely dry	Moderate humidity or cyclic wet and dry	Moderate humidity	Wet, rarely dry	Cyclic wet and dry	Exposed to airborne salt but not in direct contact with seawater	Permanently submerged	Tidal, splash and spray zones
Patch repair (conventional)			10	10	5	3	3	5	2	2
Electrochemical (chloride extraction)			-	-	5	5	5	5	-	5
Realkalisation			10	10	-	-	-	-	-	-
Repair with sacrificial anodes			#	#	20	20	20	20	#	10
Crack-bridging elastomeric coatings			10	10	5	-	-	-	-	-
Sacrificial layers			20	20	-	-	-	-	-	-
Ingressed corrosion inhibitors				#	#	#	#	#		

Key to Table 5.9-3

#	Some service life benefit likely but no sufficient supporting evidence exists at present
	Not applicable
	Consider these measures first.
	These measures may be alternatives in some circumstances.
	These measures may be considered in special circumstances.

Note to Table 5.9-3: *The values given in Table 5.9-3 are only illustrative; they should not be used outside the context of this example of the design of new concrete structures*

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Enhanced service life by improved site practice and quality control activities: The factorial approach indicates that a longer service life can be justified if it is possible to increase the ratings of some of the factors A to G above their normal estimated levels (Box 5.9-1). Factors A (quality of materials), B (design level) and C (work execution level) could potentially be increased, although this would need to be assessed on a case by case basis. The execution strategy, as discussed in Sections 5.14 and 5.15, also presents the designer and construction team with opportunities to improve the consistency with which the required cover depth and cover concrete quality are achieved on site. Associated issues include the need for buildability, effective curing of the cast concrete, increased onsite supervision, a specification for fresh wet properties, etc. Taken together these should allow the goal of meeting the requirements for depth of cover and concrete quality to be achieved.

5.10 Service life design process and considerations

5.10.1 Introduction

At the design stage consideration needs to be given to a wide range of issues which directly or indirectly have an influence upon the durability of the structure, its through-life care, its environmental impact and the whole-life cost of ownership. The goal would typically be to seek to minimise environmental impact and the whole-life cost. The processes of structural and durability design are interlinked. National codes and associated standards will usually specify the Ultimate Limit State (ULS) requirements and the Serviceability Limit State (SLS) requirements which need to be met. The traditional structural design procedure will typically satisfy these requirements. In many cases the deemed-to-satisfy approaches commonly given in the national codes and associated standards will produce a satisfactory durability design for many service environments; albeit that the level of reliability offered by these solutions is generally not known or has yet to be determined by calibration studies.

However, in more aggressive service environments or where durability performance requirements are more demanding, service life design procedures will assist in attaining the above goals by providing a rational methodology. When seeking to create durable concrete structures there is a need to take a holistic view upon through-life performance, whole-life cost and sustainability perspectives. Clearly mechanisms that may cause deterioration or damage to concrete structures in the envisaged service environment need to be taken into account, as well as factors which may influence their durability. These considerations will need to give recognition to the recommendations made in appropriate standards and codes of practice. These matters are considered, along with a range of other issues, in earlier Sections of this document – refer, for example, Sections 5.1.5, 5.1.8, 5.3.3, 5.3.4, 5.4, 5.5, 5.6 and 5.7.

It is also important during the durability design that consideration is given to matters relating to the envisaged execution (construction procedures and associated quality management – refer Section 5.14 and 5.15) and through-life care issues in order to achieve a suitable balance of these factors in the design; and to ensure the structure is easy to inspect and to maintain.

In short, the structure needs to be designed so that it is: constructible (buildable), accessible, inspectable, maintainable and repairable.

In recognition of the current focus on all aspects of environmental impact, these matters need to also include consideration of making elements replaceable where appropriate. End-of-life issues include the adaption of the structures concerned to new applications, dismantling and the re-use of structural elements where possible, together with demolition and the recycling of the associated materials, where direct re-use of the elements is not practical. It is anticipated that there will be an increasing future focus on environmental impact issues.

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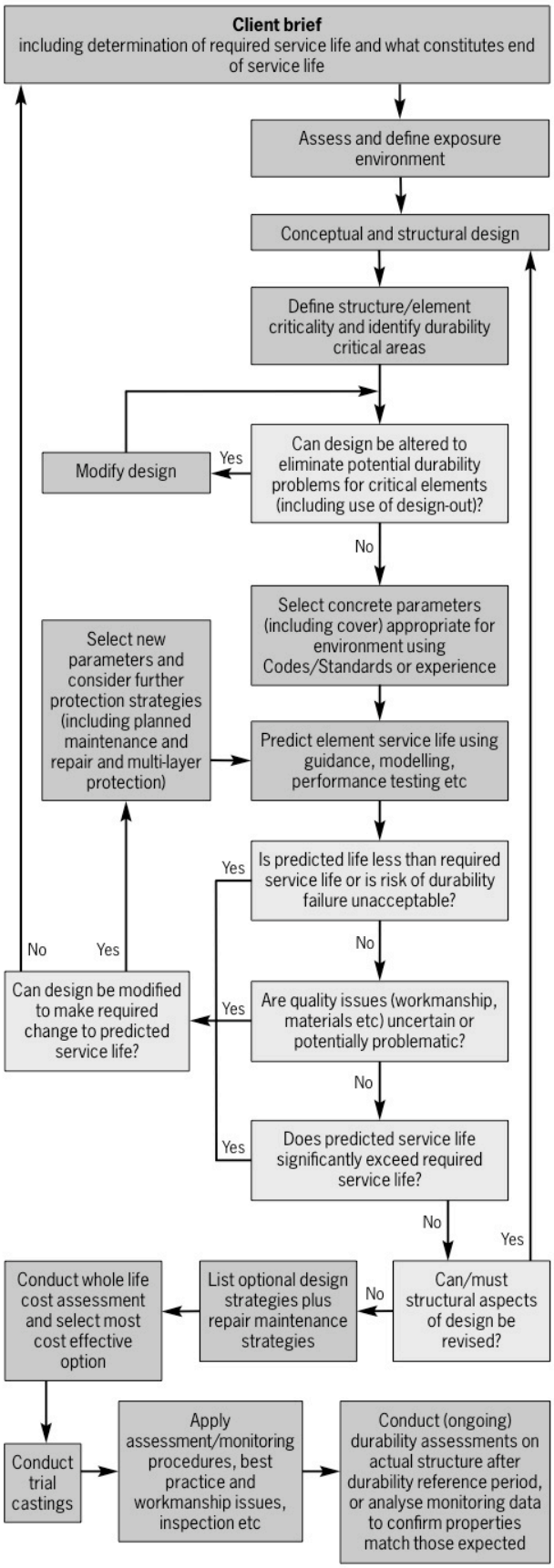

Service life design options: The main procedural steps associated with service life design option selection typically involve:

1. Specifying the required (target) service life.
2. Defining the exposure environment – refer Section 5.6.
3. Establishing the minimum requirements defined in standards or codes.
4. Chose preliminary concrete mix design/protection measures.
5. Making preliminary predictions of the anticipated service life on the basis of the defined environmental conditions and various concrete mix or protection combinations.
6. Comparing the predicted service life with that required.
7. Making any design modifications required/define any additional protection measures required to achieve the target service life.
8. Establishing a 'short list' of additional measures options, if this is appropriate, considering cost & environmental impact implications.
9. Selecting a suitable combination of measures for the required (target) service life.

Fig. 5.10-1: The main steps in a service life design procedure for concrete structures.

Note to Fig. 5.10-1: *The flowchart shown here is simplified. The service life design procedure is likely to be iterative, or with steps conducted in parallel [Quillin: BRE Digest 455 (2001)].*

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5.10.2 Main steps in a service life design procedure

The main stages of a service life design procedure are illustrated and simplified in Fig. 5.10-1. The actual process will be more complex with various steps conducted in parallel or iteratively.

When the required target service life has been fixed, the particular durability related design issues which need to be considered include the following:

- structural layout and geometric form;
- structural system;
- location of expansion joints and construction joints;
- environmental aggressivity at macro, meso and micro-climate scales;
- critical deterioration mechanisms;
- selection of materials and concrete mix;
- concrete cover;
- maximum design crack widths;
- special protective measures required;
- execution methods.

When taking a through-life perspective, whole life costing is an important consideration. The service life design process can be used as a basis for a whole life costing assessment. Using this process, technically equivalent options (i.e. those with a high probability of achieving the client's requirements) can be compared and evaluated over the life of the structure. Notionally a similar approach might be adopted in respect of environmental impacts.

Whole life costing is essentially a tool which can be used to enable owners to appraise projects, and assist them in making decisions about different projects competing for limited financial resources. In financial terms it enables expenditure to be discounted over time and normalised to a common base year. The technique is flexible and can incorporate the influence of many parameters, such as:

- initial capital cost;
- financial repayment options;
- revenue streams;
- maintenance costs;
- lifetime of the structure;
- demolition costs.

A choice is made of the parameters most relevant to the decision and an appropriate discount rate for changes in monetary value in the future is adopted (see Fig. 5.3-5). The resultant whole life cost is then calculated. Whole life costing allows an informed decision to be made between different design options. For example, the differences between a minimum capital cost scheme with future high maintenance or renewal costs against one with a higher capital cost but lower future costs.

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5.10.3 Approaches to detailed service life design

As noted previously, service life design approaches may be divided into two strategies:

Service life design strategy A — Avoiding deterioration by the design-out approach.

Service life design strategy B — Providing resistance to the deterioration mechanisms active in the service environment.

Fig. 5.10-2 portrays a hierarchy of levels of service life design related to the degree of correctness of the representation of physical processes associated with deterioration, the aggressivity of the service exposure environment, the resistance of the structure to deterioration and related factors. However, these aspects have to be balanced against the difficulty and effort involved in the use of the different approaches for service life/durability design, where simplicity is an important consideration. It is important to get an appropriate balance in these aspects in the service life/durability design for a particular structure; which, amongst other things, will depend upon the required service life and the importance of that structure.

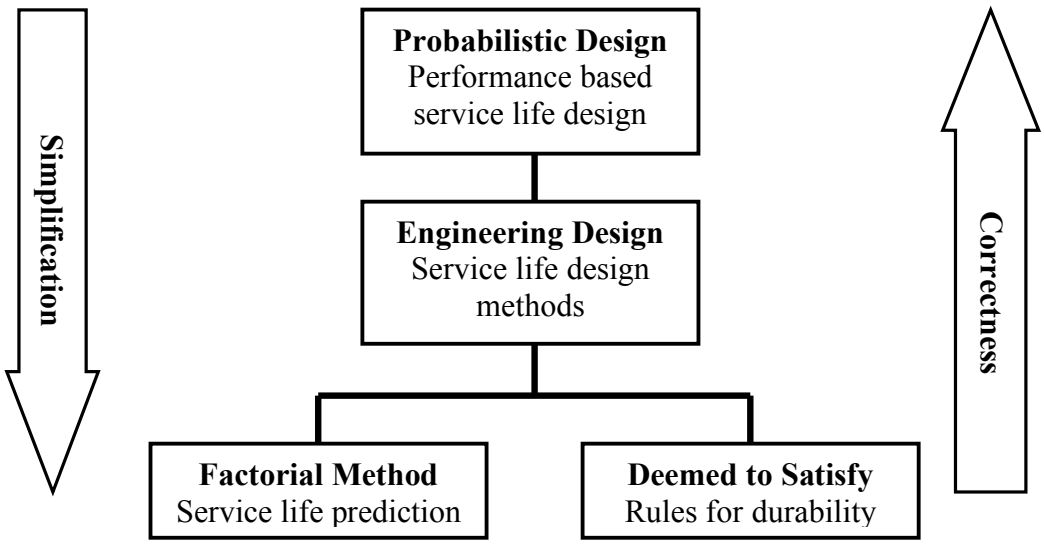

Fig. 5.10-2: Levels of service life design: Hierarchy of simplifications versus degree of correctness of representation of physical processes [after Siemes [undated)]

The two principal service life design (SLD) strategies (A and B) can be sub-divided into a number of different methodologies for achieving the service life design objectives of the particular design strategy (see next page). The overall list of service life design strategies and associated methodologies is given below. There is some overlap between the different methodologies, for example SLD strategy B3 potentially overlaps with activities in B1, B2 and B4.

Strategy A. Avoiding deterioration by the design-out approach on the basis of:

A1. Changing the service environment to remove the deterioration mechanisms.

A2. Using non-reactive materials to avoid the deterioration reactions.

A3. Inhibiting the deterioration reactions.

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Strategy B. Providing resistance to the deterioration mechanisms active in the service environment on the basis of adopting:

B1. Deemed to satisfy provisions.

B2. A single or multi-stage (barrier) protection strategy.

B2.1 Basic resistance using a single protection strategy.

B2.2 A multi-stage protection strategy.

B3. The factorial method (adapted from ISO 15686: Part 1).

B4. A reliability-based methodology:

B4.1 Full probabilistic design.

B4.2 Partial-factor design (semi- probabilistic design).

The alternative approaches to service life design are discussed in detail in Section 5.1.10. Section 5.2 provides an overview of the service life design, construction (execution) and through-life care process.

Strategy B represents different types of design provisions to achieve resistance to the service environment. For example, corrosion protection could be achieved by selecting an appropriate depth of cover and a suitably dense concrete mix. In addition, the structure can be made more resistant to an aggressive environment by appropriate detailing which might include minimising the area of concrete surface exposed to the aggressive environment, by the use of rounded rather than sharp corners and by adequate falls on surfaces and associated drainage to minimise the time that surfaces are wet.

Modelling of deterioration processes is only applicable for Strategy B. An outline procedure for Strategy B could be as follows:

- Definition of the performance criteria related to the environmental conditions to be expected in service for different parts of the structure.
- Preliminary definition of the concrete mixes which might be used in various environmental conditions or particular circumstances, together with an initial evaluation of their properties and resistance characteristics to the envisaged environmental actions.
- Realistic modelling of the environmental actions and the material resistance to these actions, giving a preliminary indication of anticipated service life for the various environmental conditions and concrete mix or protection combinations.
- Performance tests, based upon the performance criteria, are indispensable for quality control purposes. The performance tests must be suitable both to check the potential quality of the material under laboratory conditions and, even more importantly, to assess the potential quality of the material as placed in situ in the final structure.
- Based upon the outcome of these elements, the service life design procedure can be established.

Section 5.11 provides guidance upon a variety of measures which may be taken to enhance resistance or avoid reinforcement corrosion where exposure conditions are very severe.

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The additional measures which may be required to achieve sufficient protection against reinforcement corrosion are considered under five principal headings. Namely:

A. Selection of cementitious materials (Refer Section 5.11.2)

B. Use of admixtures and fibres (Refer Section 5.11.3)

C. Enhanced resistance of the surface of the concrete (Refer Section 5.11.4)

D. Enhanced tolerance to carbonation and chlorides (Refer Section 5.11.5)

E. Avoidance approach (Design-out approach) (Refer Section 5.11.6)

Categories A and B are concerned with improving the resistance of the cover concrete to aggressive actions associated with carbonation and chlorides.

Category C is concerned with measures for improving the resistance of the near surface and involves the use of coatings, surface penetrants, controlled permeability formwork and permanent formwork.

Category D is concerned with measures for improving the tolerance of the reinforcing steel to carbonation and chlorides. These measures include the use of corrosion inhibitors, epoxy coated reinforcing bars and galvanised reinforcing bars.

Category E is concerned with measures that avoid corrosion considering stainless steel reinforcement, non-metallic reinforcement and the use of cathodic prevention/protection.

5.10.4 Target service life

In order to formulate durability requirements it is necessary to identify the duration for which the structure is intended to perform satisfactorily without unforeseen high costs for inspection, maintenance and repair. This duration, known as the target service life or target working life (as in ISO 2394), may be given in existing codes or other requirements. If there are no such recommendations the target service life will need to be agreed between the owner and the designer, with the owner taking the final decision. Section 5.2.2 discusses issues relating to the client brief and the definition of performance expectations for the structure concerned.

The end of the target service life for structures and their principal structural components is usually defined as the end of the deterioration initiation period and the onset of the identified critical deterioration mechanism(s). However, in some instances it may be possible to include part of the propagation period within the target service life. This would be appropriate when safety and performance is not jeopardised by some initial deterioration. Examples of this could be for the more robust parts of harbour and marine structures where some initial corrosion of reinforcement, cracking and maybe spalling would not materially affect their functionality nor pose a danger to people.

For non-structural elements the end of their service life may be taken to be when their renewal or replacement is necessary before their reduced performance or malfunction could create secondary damage to the primary structure. Among others, this might involve activities such as renewal of coatings, replacement of drainage systems, bearings, joints or wearing courses; or possibly work upon secondary (replaceable) structural components.

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Examples of suggested service lives for structures and main structural elements include:

- 35 years: Offshore structures.
- 50 years: Structures designed to MC90, Eurocode 2, national codes or similar.
- 100 years: Bridges, tunnels, harbours, etc.
- 200 years: Storm surge barriers, monumental buildings, etc.

Examples of possible service lives of non-structural and secondary (replaceable) structural components are:

- 10-15 years: Surface coatings, sealants, waterproofing, etc.
- 25-30 years: Bearings, expansion joints, membranes, wearing courses, etc.
- 50-60 years: Drainage systems, parapets, etc.

5.10.5 Environmental aggressivity

The first and foremost task is to identify and quantify to the extent possible the type and aggressivity of the environment in which the structure is to perform during its service life. This will then clarify which types of deterioration could be a threat to the future performance and service life of the structure or its component parts.

Section 5.6 provides an overview of environmental aggressivity, indicating the influencing factors and how environmental aggressivity may be classified.

Environmental aggressivity issues are considered at three levels relating to the macro-climate, the meso-climate and micro-climate effects; which in turn are grouped by transport mechanism. These mechanisms either involve water or moisture driven deterioration processes, the atmospheric induced deterioration or thermally induced deterioration effects (refer Sections 5.6.1 to 5.6.4).

The approach presented in this document uses the exposure classes for concrete structures defined by EN 206-1, which forms part of the European system of standards for the construction of concrete structures. However, some supplementary information is provided concerning chemical attack upon concrete and some situations in which this may arise.

Table 5.6-2 details the exposure classes for concrete structures according to EN 206-1 [CEN (2000)]. The European standard specifically differentiates between exposures leading to reinforcement corrosion and exposures leading to deterioration of the concrete.

5.10.6 Deemed-to-satisfy approach

The information provided in this document in respect of the deemed to satisfy approach also relates to the recommendations made within the European system of construction standards. The European system contains standards for the design and execution of concrete structures, together with related standards for the constituent materials and associated material test standards.

As noted above, guidance upon exposure classes is given in European Standard EN 206-1 [CEN (2000)]. The information upon exposure classes given in Table 5.6-2 is then used in conjunction with Table 5.7-7 to give recommended minimum values for composition and

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properties of concrete, as defined by EN 206-1 [CEN (2000)]. This is an example of deemed-to-satisfy prescriptive design rules. This approach is the one most generally used at present, where requirements are presented as prescriptive values in the form of single values, tables or graphs. Examples include the specified requirements for minimum cover, cement type, minimum cement content, maximum water-cement ratio and curing time. This approach gives no basis for estimating the expected service life that will be achieved in circumstances other than those specified.

Eurocode 2 (EN 1992-1) for concrete structures has an a priori assumption that a 50 year service life will be achieved for structures designed in accordance with its recommendations and exposed to environments which have been classified in accordance with the exposure classes identified in EN 206-1 [CEN (2000)]. Further details European system of standards for the design and execution of concrete structures are given in Section 5.7.4.

If a service life longer than 50 years is required, or a more aggressive environment is to be considered, then a more in-depth service life design is required.

Table A5 of BS EN 8500-1 [BSI (2006)] provides deemed-to-satisfy recommendations for reinforced or prestressed concrete elements with an intended working life of at least 100 years for exposure classes XC, XD and XS as given in EN 206-1 [CEN (2000)].

5.10.7 Avoidance of deterioration

This approach corresponds to *Design Strategy A* – refer Sections 5.1.10 and 5.12. This approach involves avoiding circumstances which can lead to deterioration or the use of non-reactive materials. Examples are avoiding the saturation concrete to minimise the risk of frost attack, the selective use of stainless steel to avoid corrosion induced deterioration, the use of non-reactive aggregates to minimise the risk of alkali-aggregate reactions, etc.

5.10.8 Factorial method for service life prediction

These matters are discussed in Section 5.9.

5.10.9 Types of modelling for service life design

When designing the structure for its intended target service life the detailing of the modelling required in each case should be decided, as this will govern the whole design as well as the possibilities of later updating of the service life design, [CEB (1997)]. Different levels of service life modelling are discussed in Section 5.7.3.3. These are:

A full probabilistic design: This corresponds to *Design Strategy B* (see Section 5.1.10). It will seldom be possible to employ this approach for new structures due to lack of statistical data. The approach might be adopted for special long-life or monumental type structures. The approach may be applicable to the assessment of existing structures. This approach is designated as '*Level 1 design*'.

A partial factor (semi-probabilistic) design: This corresponds to *Design Strategy B* (see Section 5.1.10), but with factors calibrated with Level 1 approach above. This approach is based upon design values for loads, capacities and geometrical characteristics. This is a deterministic approach where the probabilistic nature of the problem (i.e. the scatter in material resistance properties and environmental load parameters) is taken into account by partial safety factors. This approach is designated as '*Level 2 design*'.

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5.10.10 Designing for resistance – Using models in service life design

The modelling approaches described above allow updating of predictions of service life using data obtained from the structure and the concrete after the construction has been finished, e.g. as part of the Birth Certificate provided at the time of handing over the structure to the owner. This facility also allows the owner to update the service life forecast at will.

Rostam (1999) observed that a service design undertaken in accordance with Strategy B: *Designing for resistance*, refer Section 5.1.10, should contain the following:

A. Models describing the deterioration processes, quantified as far as possible.

B. Prescriptive requirements to ensure durability, such as maximum water-cement ratio, minimum concrete cover (including tolerance), curing procedures (moisture and temperature control), etc.

C. Quality control procedures covering all steps of the production and placement of concrete materials and the associated elements of the construction processes. Of particular importance are interfaces where the responsibility for the quality of the product changes (e.g. handing over the ready mixed concrete at the site).

D. Definition of a "Reference Durability Period", e.g. 5 years. After that period relevant durability parameters such as porosity, permeability, diffusivity, carbonation, chloride penetration, etc. can be determined to verify the achieved quality and resistance of the structure under the given environment. On completion of construction, surveys of some of the durability parameters such as cover, porosity, permeability and diffusivity could be carried out and entered into the "Birth Certificate", refer Section 5.10.16. Procedures equivalent to this are already in normal use in some countries.

E. Using the models (item A above) and the durability parameters measured (item D above), the future performance of the structure can realistically be estimated and an updating of the achievable service life can be made, refer Section 5.10.17.

Whilst improved construction specifications may be needed, which include long-term material performance properties; they will not by themselves improve in-situ quality without a changed attitude towards on site production and towards the interaction between design and production. The competence and experience of the work force is crucial. These matters are discussed in Sections 5.16 and 5.17. The owner should both ask for quality and he, or his representatives, must also check that the relevant quality is achieved in the actual structure. If these steps were taken, quality, consistency and durability would undoubtedly improve considerably.

5.10.11 Adoption of a multi-layer protection approach

The concept of a multi-layer protection approach for durability design recognises the difficulty of achieving perfect protection against deterioration by the use of a single method. Notionally each layer is able to provide a proportion of the total protection required. With the provision of a number of complementary protective measures the integrity of the overall protection will be maintained, even if another one of the layers of protection becomes partially ineffective. The measures chosen need to be applicable to the envisaged mechanisms of deterioration and the environment in question. Sometimes this type of approach is referred to as a ‘multi-stage protection strategy’. Measures to enhance resistance or avoid reinforcement corrosion are discussed in Section 5.11; and other forms of deterioration in Section 5.12.

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Thus a set of appropriate measures (one or more) can be combined to ensure that there is a sufficiently high probability of achieving the required service life. The protective measures available may influence either the duration of the initiation phase or the rate of propagation of the active deterioration. The protection measures may involve various actions such as:

- the selection of a suitable structural form;
- the choice of concrete composition, including special additions or admixtures;
- the reinforcement detailing including concrete cover;
- enhancing quality of the concrete surface;
- limiting or avoiding crack development and crack widths, e.g. by prestressing;
- providing additional protective measures such as tanking, membranes or coatings, including coating applied to the reinforcement;
- specified inspection and maintenance procedures during the in-service operation of the structure, including monitoring procedures;
- active protective measures such as cathodic protection.

The different protection measures achieve different levels of performance and reliability in terms of their ability to protect the structure. The level of reliability achieved may depend heavily upon the quality assurance scheme associated with implementing the particular protection measure and upon burden of its future maintenance.

5.10.12 Multi-layer protection of pre-stressing tendons

fib Bulletin 33 (2006) and Concrete Society Technical Report 47 (2010) are concerned with the risk of depassivation and corrosion of pre-stressing tendons in various circumstances, but particularly in relation to bridges and similar structures. They are concerned with the provision of corrosion protection to post-tensioned prestressing tendons. In recognition of the difficulty of achieving perfect protection by any single means, a multi-layer protection approach to the durability design of post-tensioned construction is recommended. Notionally each layer is able to provide a proportion of the total protection required. With the provision of a number of complementary protective measures the integrity of the overall protection will be maintained, even if another one of the layers of protection becomes partially ineffective. This approach has also been used for the provision of corrosion protection to ground anchors.

There are a large number of potential layers of protection available to the post-tensioned prestressing tendons. In this context a layer of protection is anything whose purpose is to ensure the overall long-term integrity of the tendons. Hence methods of inspection, and subsequent maintenance, could ultimately also be considered as layers of protection.

A prime concern is also to minimise the contact of water with the concrete members. This involves a combination of conceptual design, structural detailing and attention to finishes to the structure including items such as drainage, waterproofing and surfacing.

Fig. 5.10-3 illustrates a range of potential hazard scenarios relating to prestressing steel in a notional box girder bridge. This indicates a number of potential ‘weak points’ where water, possibly contaminated with chlorides, might gain access to the post-tensioned prestressing tendons and subsequently result in their corrosion.

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Other considerations relevant to achieving durable post-tensioned concrete construction include:

- the importance of the quality of construction, plus the confidence that can be given through contractor qualification, specific competence and workmanship certification;
- the availability of products and the quality of the materials to be used;
- the benefits of an appropriate inspection and maintenance programme;
- the possible need to undertake replacement of post-tensioned prestressing tendons.

The objective of the durability design strategy developed in *fib* Bulletin 33 (2006) is to select an appropriate Protection Level (PL) for the post-tensioning tendons based on:

- the aggressivity of the service environment;
- the exposure of the structure or element;
- the number of protection layers inherently provided by the form and construction of the structure.

It is the combination of the tendon Protection Level (PL) provided to the post-tensioning tendons and the Protection Layers provided by the structure which together provide the resistance against the aggressivity of the environment and the particular exposure condition of the structural element. Thus, if overall a high level of protection is required to the post-tensioning tendons because of the combination of the exposure conditions and the aggressivity of the environment; this could be supplied by either:

- a high level contribution from the Protection Layers inherently provided by the structure and a low level of tendon Protection Level (PL1); or
- a low level contribution from the Protection Layers inherently provided by the structure and a high level of tendon Protection Level (PL3); or
- any combination of the two parameters.

The durability design strategy developed in *fib* Bulletin 33 (2006) is summarised in Table 5.10-1. The entry points to the table are as follows:

- the aggressivity of the environment/exposure of the structure or structural element concerned (Low, Medium and High);
- the Protection Layers inherently provided by the structure (Low, Medium and High).

Taken together, these permit the selection of an appropriate Protection Level (PL1 to PL3) for the post-tensioning tendons. Thus, Table 5.10-1 is defining the level of additional protection required by the tendons to supplement that inherently provided by the structure; that is which of Protection Levels PL1 to PL3 is required. PL1 provides the lowest level of additional protection, whilst PL3 provides the highest level of additional protection.

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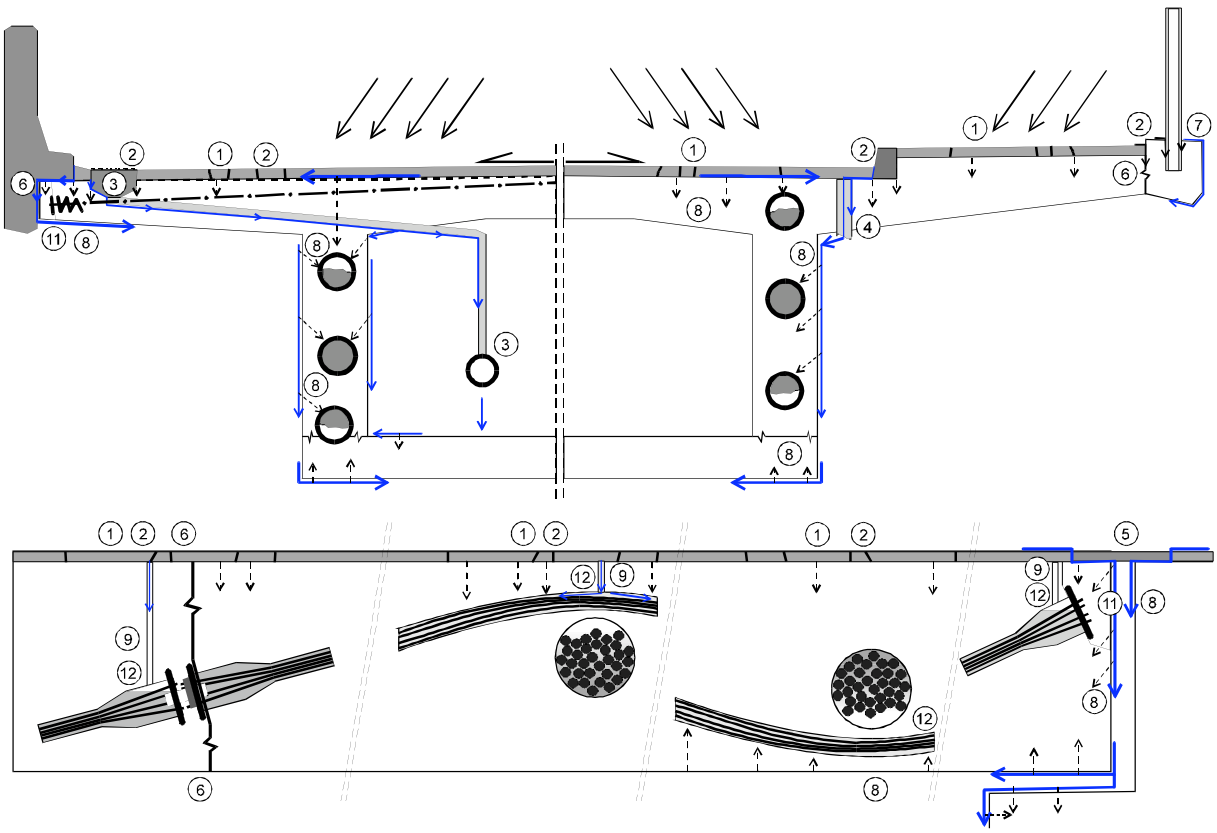

Fig. 5.10-3: Illustrative hazard scenarios for prestressing steel in a notional box girder bridge [fib Bulletin 33 (2006)]

Notes to Fig. 5.10-3:

Hazard scenarios for prestressing steel in a typical box girder bridge indicating potentially 'weak points' where water, possibly contaminated with chlorides, might gain access to the prestressing tendons and subsequently result in their corrosion. [from fib Bulletin 33 (2006) after Matt (2000)].

The following detail a series of potential threats to the prestressing steel tendons:

Failure of external barriers:

1 *Defective wearing course (e.g. cracks).*

2. *Missing or defective waterproofing membrane incl. edge areas.*

3 *Defective drainage intakes and pipes.*

4. *Wrongly placed outlets for the drainage of wearing course and waterproofing.*

5 *Leaking expansion joints.*

6. *Cracked and leaking construction or element joints.*

7. *Inserts (e.g. for electricity).*

8. *Defective concrete cover.*

Failure of tendon corrosion protection system:

9. *Partly or fully open grouting inlet and outlets vents.*

10. *Leaking or metallic ducts damaged mechanically or by corrosion (case not shown on figure).*

11. *Cracked and porous pocket concrete.*

12. *Grout voids at tendon high and low points.*

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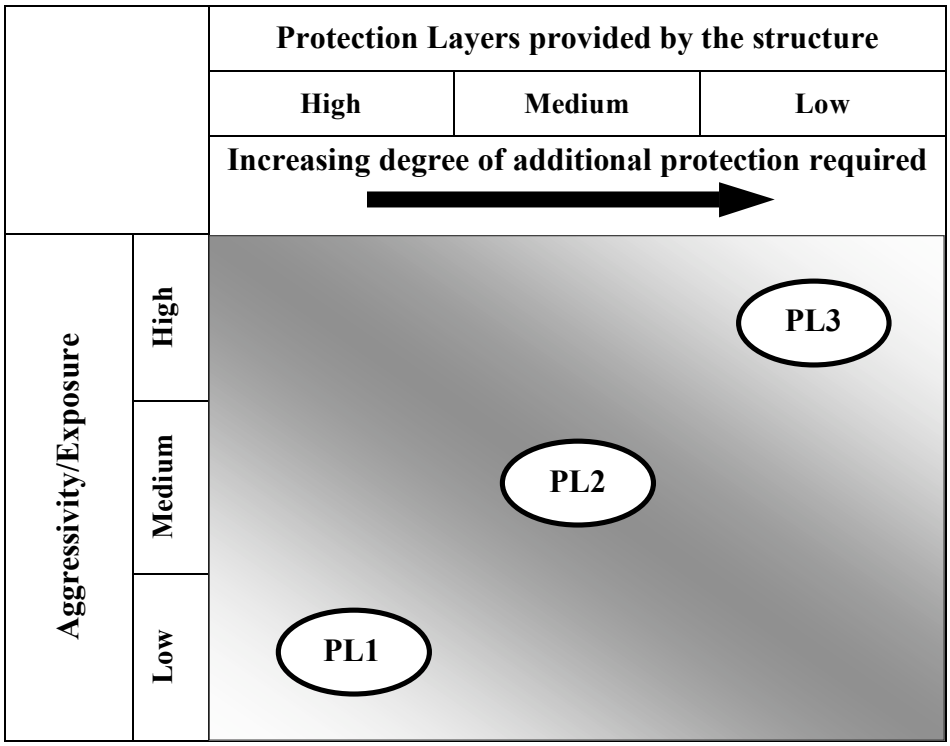
Table 5.10-1: Protection Levels (PL) required for post-tensioning tendons based on aggressivity/exposure versus structural protection layers present [fib Bulletin 33 (2006)]



Three levels of post-tensioning tendon Protection Level (PL1 to PL3) are portrayed in Figs. 5.10-4 to 5.10-6 [refer *fib* Bulletin 33 (2006)].

The outline definitions that *fib* Bulletin 33 (2006) gives for the three levels of post-tensioning tendon Protection Level (PL1 to PL3) are as follows:

Protection Level PL1: A duct filled with a material giving durable corrosion protection.

Protection Level PL2: PL1 plus an envelope, enclosing the tensile element bundle over its full length, and providing a permanent leak tight barrier.

Protection Level PL3: PL2 plus integrity of tendon or encapsulation capable of being monitored or inspected at any time.

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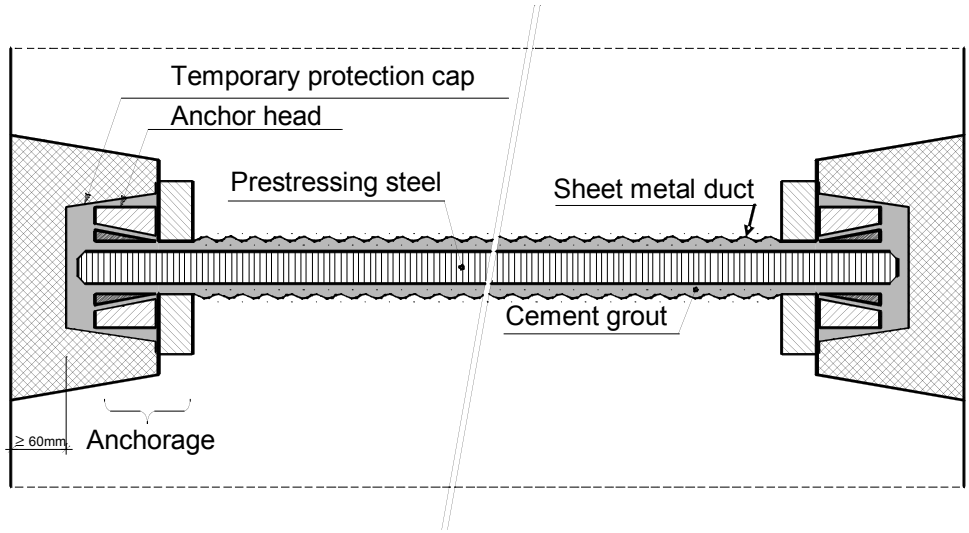

Fig. 5.10-4: Tendon system for Protection Level 1 – PL1 [fib Bulletin 33 (2006)]

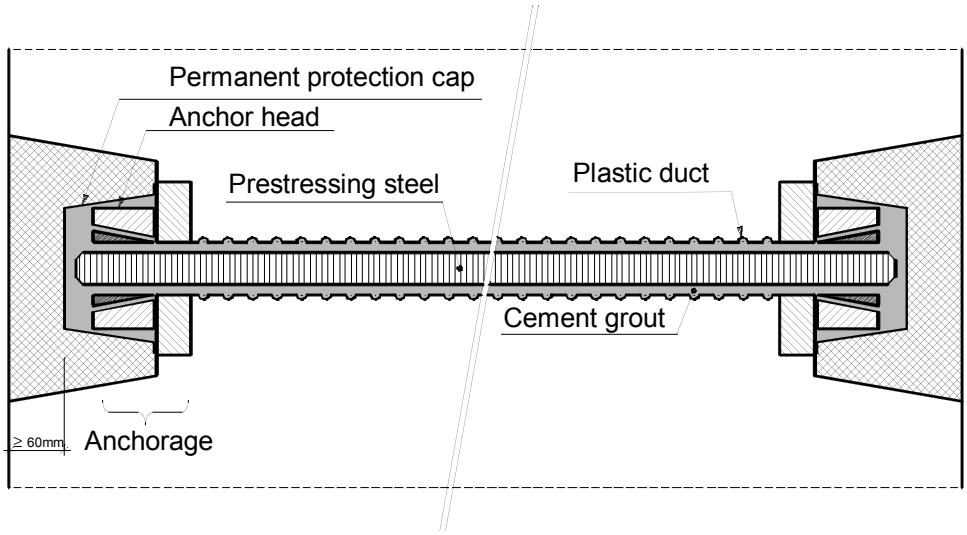

Fig. 5.10-5: Tendon system for Protection Level 2 – PL2 [fib Bulletin 33 (2006)]

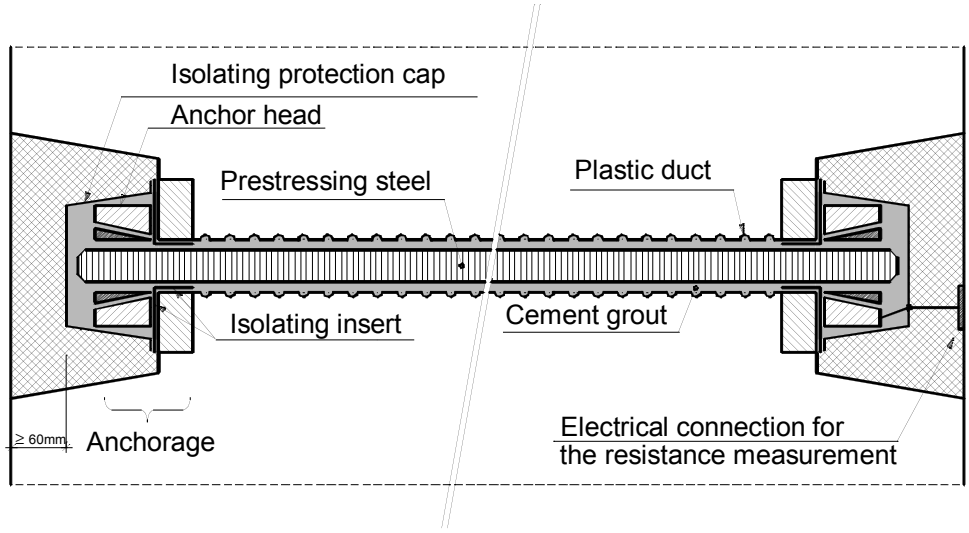

Fig. 5.10-6: Tendon system for Protection Level 3 – PL3 [fib Bulletin 33 (2006)]

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fib Bulletin 33 (2006) provides detailed specifications for the three levels of post-tensioning tendon Protection Level (PL1 to PL3). It also presents procedures for defining (a) the aggressivity of the environment/exposure of the structure and (b) the Protection Layers inherently provided by the structure.

fib Bulletin 33 (2006) also notes that for the finished works to be durable it is necessary:

- for good design practice to be matched with good construction practice;
- to implement the procedures set down in international quality assurance (qa) and quality control (qc) standards;
- for expertise and craftsmanship to be recognised and required in project specifications in order to assure that best practices are followed;
- that verification of the protection levels (pl) be established by performance testing rather than by prescriptive solutions.

Further information on the development of the above concepts for ensuring the durability of post-tensioning tendons is given in Section 5.4.7.10 and in *fib* Bulletin 15 (2001). This document contains papers presented at an international workshop entitled "*Durability of post-tensioning tendons*" held at Ghent University, Belgium on 15-16 November 2001.

5.10.13 Observations for environments where de-icing salt is applied

De-icing salts are widely used in most countries which experience freezing winter conditions. The de-icing salts commonly adversely affect concrete bridges and structures in the vicinity of the road surface as a result of direct application or spray caused by vehicles using the road; where they promote reinforcement corrosion. De-icing salts can also affect structures such as car parks where saline water can run off of vehicles.

The environment faced by concrete elements in or around a bridge or road will depend on their specific location in relation to the road surface and the design and condition of the structure. There are three main sources of chlorides in a road environment are leakage of chlorides through joints, spray from passing traffic and spray from traffic passing under a bridge. A number of factors may influence the impact that de-icing salts have on a concrete structure. These include the:

- Moisture state of the concrete; as saline water can be drawn into the concrete through absorption and capillary suction mechanisms.
- Exposure to rainwater, which can wash surfaces free of chlorides and can remove chlorides from within the surface zone of the concrete. Chlorides may consequently be moved to other parts of the structure, such as drains, and they can also penetrate cracks and joints in the concrete structure.
- Evaporation of water from the surface zone of the concrete increases the chloride ion concentration near the surface.
- Variation in exposure conditions with height above, and distance from, a road. These dimensions influence the amount of chloride reaching the concrete surface from the splash and spray produced by the vehicles using the road. It has been suggested that most of the salt is deposited within 50 m of the road; with airborne salt levels likely to be close to the background level for distances over 100 m from the road. It is indicated

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that about 20% of the salt will be deposited within 3 m of the road and about 25% within 7 m. However, these figures will depend on the speed and size of the vehicles using the road, as well as on the local environmental conditions.

- Frequency of application of de-icing slats. This is an important consideration. For example, in Nordic countries de-icing salts may be applied between 30 and 60 times a year.

Although the micro-environment of concrete elements forming bridges will depend on the design of the structure, they can be divided into two principal types:

- ***The wet zone*** which is directly exposed to rain and often has a humidity of over 90% RH. As the road surface will be wet in the vicinity of these parts, exposure to splash from rain water and saline water will occur. It is also likely to be exposed to solar radiation, potentially leading to significant temperature variations.
- ***The dry zone*** which is not exposed to direct rain and has a lower relative humidity. However, exposure to de-icing salts can occur through spray and splash from vehicles.

For sheltered concrete concentration of chloride at the surface will increase over time. For exposed concrete it is likely to reach a steady state after a few seasons. It has been suggested that the chloride concentration at the concrete surface will decrease linearly with height above road level and logarithmically with distance from the road. It has also been found to have a maximum value on the leeward face [HETEK (1997)].

5.10.14 Some observations upon structures in a marine environment

There is a wide range of distinct, and potentially damaging, environments that may be faced by reinforced concrete in marine conditions. The risk of reinforcement corrosion occurring will depend on the actual environment and the location of the concrete in relation to sea level.

Fig. 5.10-7 shows a number of scenarios for chloride ingress into concrete in a marine environment. Where the concrete is submerged and fully saturated diffusion is likely to be the main mechanism for chloride ingress. Under these conditions reinforcement corrosion is unlikely to occur due to the restricted supply of oxygen. In the upper tidal and splash/spray zones the concrete will be infrequently wetted and processes such as absorption and capillary suction will occur. The concrete is likely to be partially saturated and subject to wetting and drying cycles. This may lead to rapid chloride ingress in some instances. Subsequent evaporation can lead to a build up of chlorides in the surface zone concrete (although chlorides can be removed through the action of rainfall). Under these conditions reinforcement corrosion is a major concern.

Fig. 5.10-7 also shows a suggested severity rating on a scale of 1-12 for different climatic zones. Four climate types are considered:

- cold climate, which is subject to freezing conditions;
- temperate climate;
- hot wet climate;
- hot dry climate.

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The severity ratings are based on the relative rates of chloride-induced corrosion for the same reinforced element exposed to different marine environments corresponding to the XS2 to XS3 classification [refer EN 206: CEN (2000)]. The severity rating within parts of the structure that would be classified as XS3 under BS 8500-1 [BSI (2006)] varies from 2, for concrete in the lower tidal zone in cool/temperate conditions, to 12 for infrequently wetted concrete in hot/dry conditions. In structures subject to tidal action the microclimate depends on the relationship of each element to the fluctuating seawater level and, in particular, the amount of time it remains dry after inundation by seawater or spray. The zones most at risk from corrosion are those which are subject to an unbalanced wetting and drying cycle such as:

- locations subject to annual changes in mean sea level and tidal range due to seasonal, barometric or other reasons;
- locations such as dry docks, locks and slipways;
- areas adjacent to bollards, mooring winches and salt water hydrants;
- areas at, or above, high water level which are subject to splash or irregular inundation;
- concrete exposed to seawater in bridge piers, underground structures and tunnels (e.g. through leakage or moisture transport through joints or the concrete);
- arid conditions (natural or manmade) which exacerbate all the previous conditions

Important factors in assessing the environment of a maritime structure include:

- water chemistry – in particular chloride and sulfate levels.
- water level and fluctuations due to tides and waves;
- river flow in estuaries;
- wind;
- scour effects;
- speed and direction of water movement;
- the effects of long-term sea level rise also need to be considered;
- ice movement in polar latitudes or locations where sea ice forms;
- geomorphologic development can occur and may be the most significant feature of the ground conditions affecting the works.

The maritime environment can also be very severe to the concrete itself in terms of physical weathering, abrasion and chemical attack.

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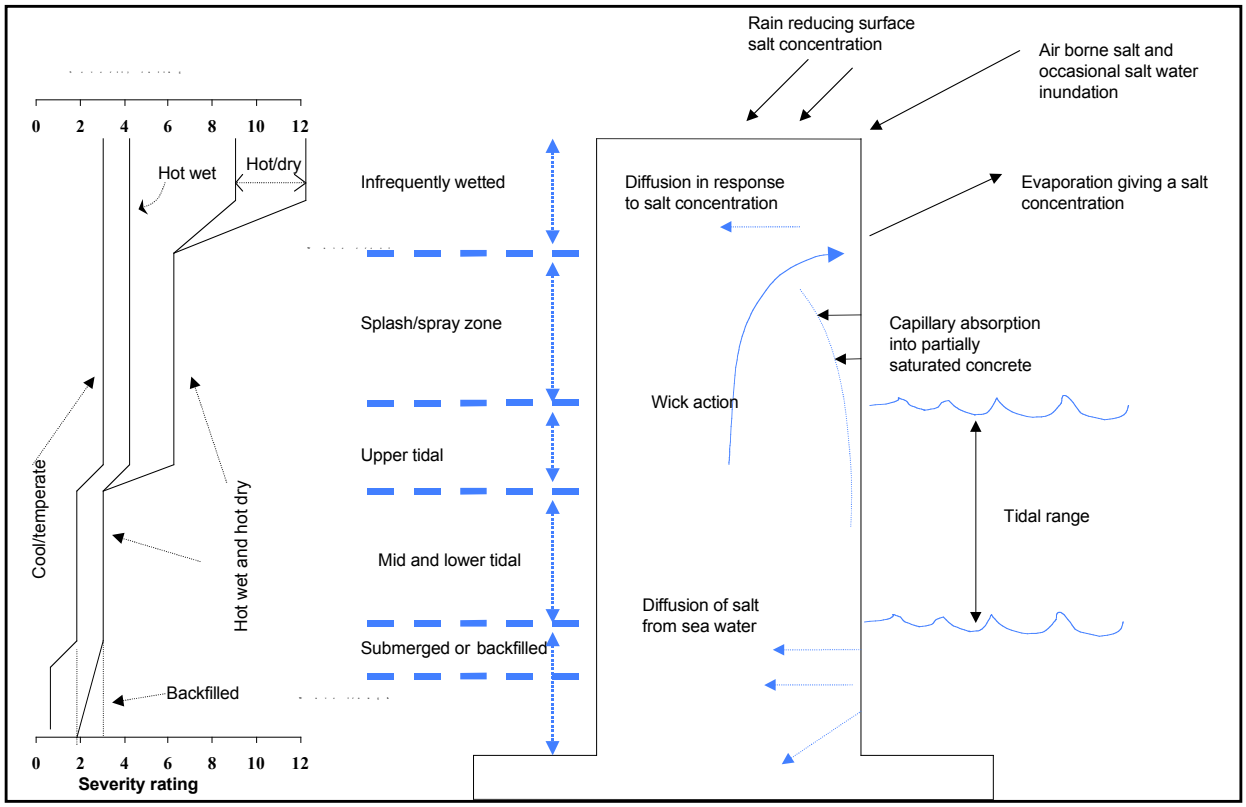

Fig. 5.10-7: Transport processes active in a sea wall, with the influence of different climatic conditions

Note to Fig. 5.10-7: *This is derived from Slater & Sharp (1998), with reference to Concrete Society CS109 (1996).*

5.10.15 Execution and quality management issues

These matters are discussed in Section 5.2.9 and in more detail in Sections 5.15 and 5.16.

5.10.16 'Birth certificate' documentation

The following definitions are given in *fib* Bulletin 44 (2008) for the concept of Birth and Re-birth certificate documentation.

Birth certificate: *A document, report or technical file (depending on the size and complexity of the structure concerned) containing engineering information formally defining the form and the condition of the structure after construction. The document/report should provide specific details on parameters important to the durability and service life of the structure concerned (e.g. cover to reinforcement, concrete permeability, environmental conditions, quality of workmanship achieved etc) and the basis upon which future knowledge of through-life performance should be recorded. This framework should provide a means of comparing actual behaviour/performance with that anticipated at the time of design of the structure. The document/report should facilitate ongoing (through-life) evaluation of the service life which is likely to be achieved by the structure.*

Re-birth certificate: *Similar to the 'Birth certificate' for a structure, but relates to the information and circumstances associated with a project for the repair/remediation/refurbishment of the structure or a part thereof to extend its anticipated service life.*

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Thus a Birth or Re-birth certificate would form part of the quality assurance documentation for design and execution of a concrete structure, but extended to include information gathered during the operation and use of the structure [Rostam (1999 and 2005)]. Thus the intention would be for a Birth certificate to document how the design specifications have been fulfilled and to record information to verify the subsequent performance of the structure, or alternatively where these goals were not being met. Accordingly a Birth certificate would be an operation and maintenance manual specific to each structure. It would be prepared by the designer and would include all information from the structural design and the construction processes relevant to the future inspection operation and maintenance of the structure. It would also include recommendations regarding the type and frequency of future inspections and should highlight possible sensitive or critical parts of the structure which are envisaged to need particular care and attention during use.

The intention would be for the Birth certificate documentation to be updated as events occurred during use, after inspections were made and works were undertaken. Thus the Birth certificate would therefore be a 'live' evolving document.

In their paper entitled *Life cycle management of concrete structures Part I: Birth certificate* Mayer and Schiessl (2008) have also presented ideas for a 'Birth certificate' as a basic element of a predictive life cycle management system for concrete structures. They indicate that the process would provide improved knowledge of current and future condition states of the structure, as well as a means of assessing structural resistance for known or anticipated environmental stresses by the use of full-probabilistic deterioration models. The evaluations would be undertaken in combination with (a) material testing during the compliance stage verification of concrete properties and (b) non-destructive test methods during acceptance testing after completion of the structure.

Fig. 5.10-8 illustrates the envisaged stages in the development of Mayer and Schiessl's Birth certificate. The approach would allow optimisation of structural resistance during design, the review/improvement of the condition prognosis for the structure based upon Bayesian updating using material and performance data gathered from the final structure, as well as the use of focussed remedial interventions when necessary.

The concept of Birth and Re-birth certificates is currently (2009) being advanced by a Task Group working within *fib* Commission 5 'Service life aspects'. The *fib* Commission 5 Task Group (*fib* Com5: TG10: *Birth and re-birth certificates & through- life management aspects*) is examining and further developing the concept and application of Birth and Re-birth certificates.

fib Com5: TG10 has suggested that the concept of Birth and Re-birth certificates should form a key component of an overall structure inventory management system, and would be particularly valuable to owners of large portfolios of concrete structures. It could provide a framework for establishing and documenting the design decisions, quality of workmanship during construction, and scheduled (and unscheduled) maintenance activities that determine the initial target *Design Service Life* and the *Remaining Working Life* of structures in an owner's inventory. The Birth or Re-birth certificates for each structure in an inventory would then form the basic building blocks for a thorough structure inventory management system.

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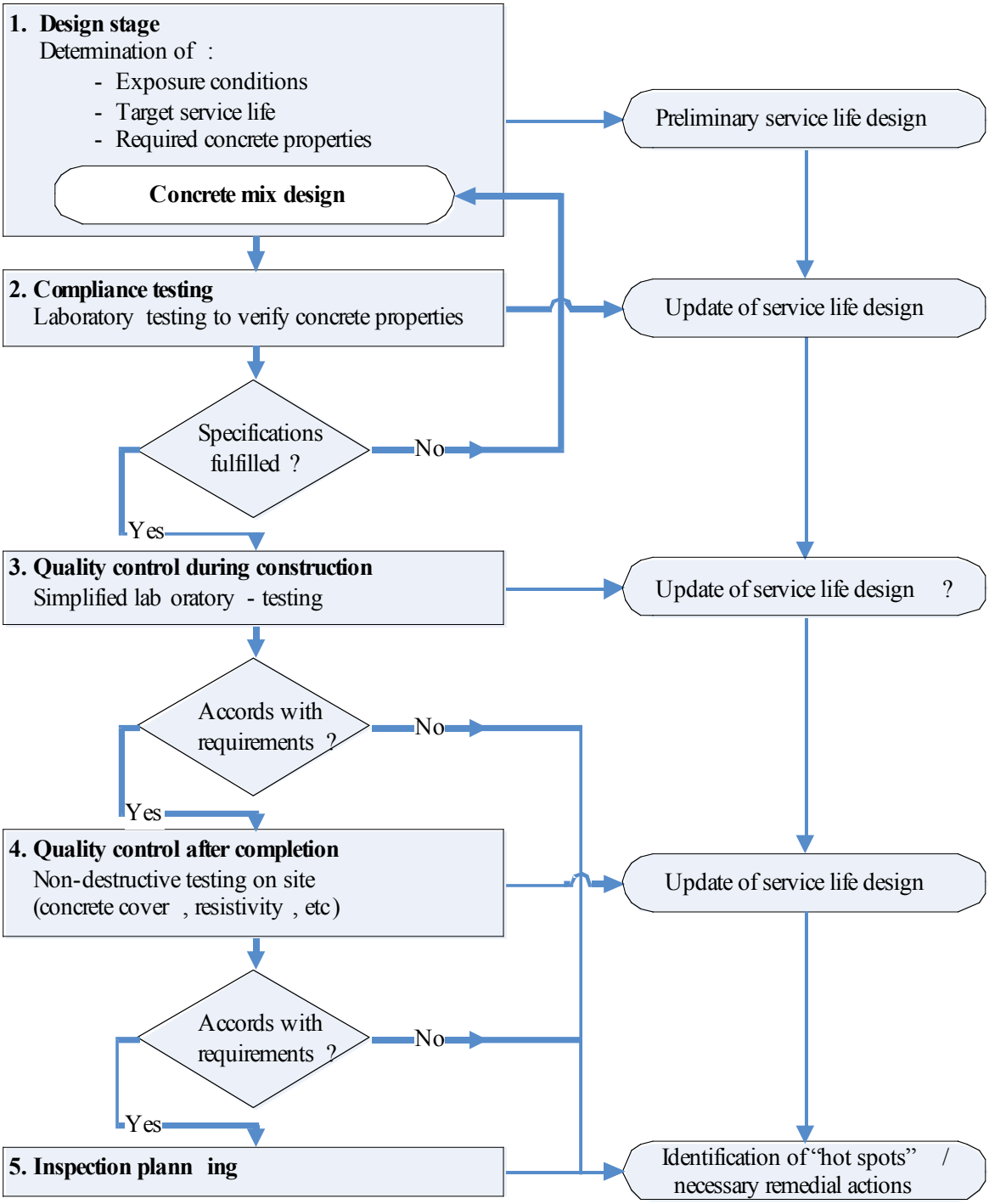

Fig. 5.10-8: Stages in developing a Birth Certificate [after Mayer & Schiessl (2008)]

fib Com5: TG10 has suggested that Birth and Re-birth certificates for a structure might potentially contain seven separate sections as follows:

- description and identification of the asset;
- environmental exposure conditions;
- deterioration mechanisms and models;
- summary of individual major structural components;

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- summary of ancillary (replaceable) components;
- in-service inspection plan;
- dismantling plan.

fib Commission 5/TG10 is currently (2009) working on developing suggested formats for the individual sections and are preparing an application example for a highway bridge.

Summarising, Birth and Re-birth certificate documentation has an important role to play in the life-cycle management of concrete structures. Potentially Birth and Re-birth certificates could provide a valuable means of communication between the various stakeholders during the design and construction phases (to help ensure an appropriately durable structure was created) and then as a basic element of a predictive life-cycle management system for concrete structures.

5.10.17 Reliability updating

A benefit of adopting the probabilistic service life design method is that the expected uncertainties can be taken into account at the initial design stage, when the concrete and the construction and maintenance conditions are not known. Subsequently, when these important values can be determined by testing, it is then possible to update the service life predictions on the basis of information about the actual concrete structure. If the quality of the concrete in the actual structure is less than that anticipated during the design, the revised estimate of the service life may indicates that the predicted service life is likely to be less than the desired target life. In this case it may be necessary to make a preventive or remedial intervention to extend the service that will be achieved.

It is desirable that such testing on the actual structure be undertaken relatively early in the life of the concrete structure, but this should not be until the concrete has reached a reasonable state of maturity. Early testing allows time to undertake any necessary precautions or preventive interventions.

Thus adopting a probabilistic service life design approach allows the service life and the economic consequences of any decisions taken (whether they be savings or additional costs) to be readily updated when testing is undertaken and additional information becomes available. For example, if the depths of carbonation are measured in a structure when it is say 5 years old, it should be possible to make an initial estimate of the time when carbonation is likely to reach the reinforcement. Depending upon the rate of carbonation and the age at which the measurements are made, it should be possible for such predictions to be made with reasonable reliability. Similarly, the time to initiation of chloride induced corrosion can be estimated using chloride transport models, if the calculation is based on real chloride profiles determined after the structure has been in service for some years.

5.10.17.1 Bayesian updating

Increasingly Bayesian analysis is taking a more prominent role in the reliability assessment of concrete structures. Bayesian statistics provides a framework within which data obtained from routine site investigations and continuous real-time monitoring of structural response to the external environment and applied loads can be combined rationally with analytical predictions of structural performance at the design stage to update and refine initial estimates of service life. Such an approach has significant advantages over conventional probabilistic analysis where no systematic methodology is available to re-evaluate structural reli-

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ability in the light of any new information obtained regarding the in-situ behaviour of the structure.

Information of this type may be used to update the reliability estimate of the structure. The reliability is updated on the basis of Bayes rule:

$$P(\text{failure} \mid \text{observation}) = \frac{P(\text{failure} \cap \text{observation})}{P(\text{observation})} \tag{5.10.1}$$

Bayesian updating will also improve the accuracy of service life predictions by reducing the uncertainty associated with the input parameters required by the structural analysis models. During the design phase, decisions regarding the input values of model parameters are often made based on engineering judgement and past experiences gained from constructing structures in similar environmental conditions. This introduces a degree of uncertainty into the analysis which is reflected in the high coefficients of variation assigned to the basic random variables when assessing long-term reliability performance using probabilistic methods. The acquisition of additional data on the actual in-service characteristics of the structure such as material properties, applied loading and quality of workmanship allows the statistical estimators describing the model parameters to be recalibrated, reducing uncertainty and hence improving the accuracy of the model prediction. This can be best achieved within the framework of Bayesian statistics.

Karimi and Buenfeld (2007) indicate that Bayesian analysis is becoming more widely applied in the engineering field to estimate the future reliability of structural components based on data gathered from site inspections. Enright and Frangopol (1999) applied Bayesian techniques to investigate the influence of inspection updating on the time-variant reliability of a reinforced concrete bridge at risk of damage from chloride-induced reinforcement corrosion.

Karimi and Buenfeld (2007) also observe that these studies, along with many others published in the literature, are concerned with updating structural reliability predictions based on the results of on-site inspections carried at certain specified time intervals. The use of data obtained on a continuous basis from sensors embedded in structures has received scant attention in the research field. Karimi and Buenfeld (2007) suggest that this is surprising since real-time sensor data on the in-situ performance of a structure is ideal for use in the context of Bayesian statistics for the determination of structural reliability.

5.10.17.2 Application example

Fig. 5.10-9 shows the estimated probability of corrosion versus time; illustrating the change in perceived corrosion risk resulting from an ability to update the prediction based upon performance measurements undertaken upon the actual structure. Fig. 5.10-9 shows:

- The initial prediction of corrosion risk, which was made on the basis of measurements of chloride ingress performed upon laboratory test specimens of concrete. This gave the result marked as Fick's second law of diffusion in Fig. 5.10-9.
- The updating of prediction of corrosion risk based upon field measurements of chloride ingress on the actual structure after a pre-determined period of exposure. This gave the result marked as the Duracrete model in Fig. 5.10-9. This shows a substantial reduction in the risk of reinforcing bar corrosion associated with the maturing of the concrete with time, reducing the apparent chloride diffusion coefficient with an increase in the period of exposure (see Fig. 5.11-6).

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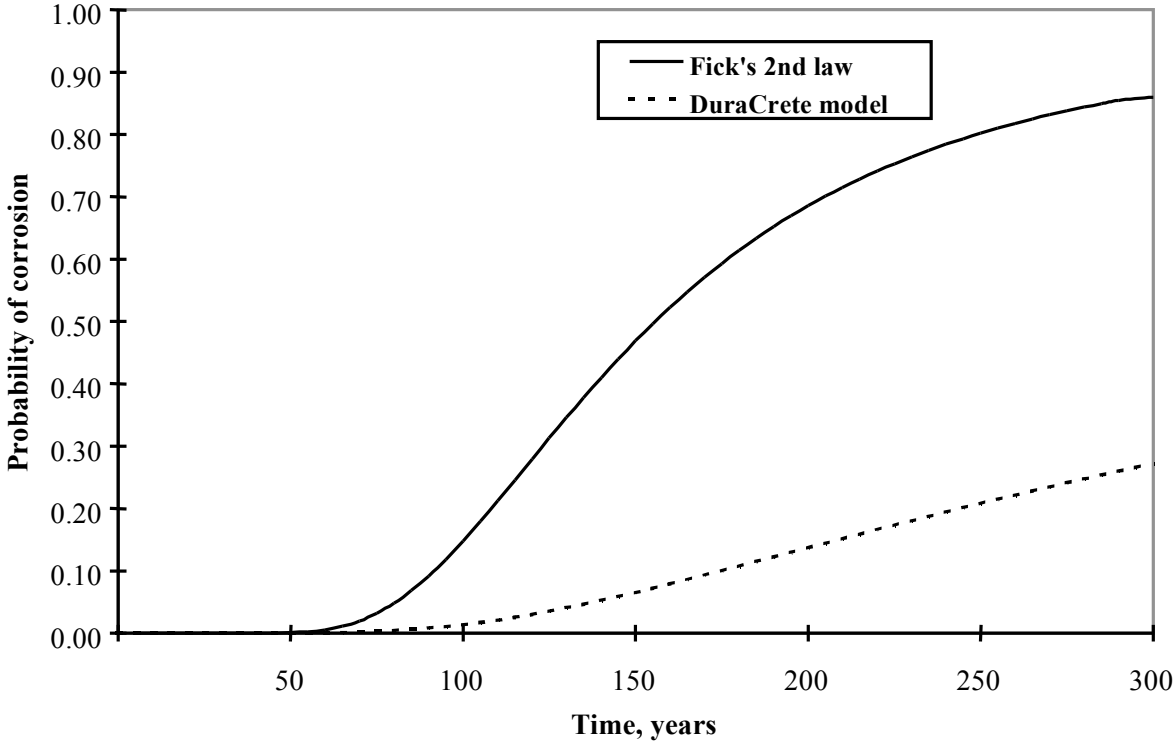

Fig. 5.10-9: Probability of corrosion versus time [Rostam (1999)].

Note to Fig. 5.10-9: *Updating of the service life estimate may be based on experience and in-situ testing of the ageing effect of the concrete (i.e. the continued maturing of the concrete) with respect to chloride penetration. The DuraCrete model result shown is based on an ageing factor of 0.3 – see [DuraCrete (1999)].*

5.10.17.3 Applying Bayesian updating in combination with intelligent monitoring

Karimi and Buenfeld (2007) discuss the application of Bayesian updating in combination with intelligent monitoring. They report that data obtained from embedded sensors provides an invaluable source of information with regards to the in-situ performance of structural components. The installation of sensors to monitor variations in concrete characteristics such as the internal moisture and temperature distributions and applied tensile and compressive stresses is gaining further acceptance in industry although adequate guidance on how to best utilize the data collated in the reliability assessment process is currently lacking.

Karimi and Buenfeld (2007) indicate that sensor data can be most effectively employed in combination with suitably developed deterioration models to predict service lives within the framework of Bayesian statistics – refer Section 5.17.8. However they note the application of such a multidisciplinary approach to durability assessment is not widely reported in the literature. Nevertheless they indicate that there is the potential for significant improvements in the accuracy of service life predictions through the use of sensor data in probabilistic deterioration models, which would be a key reason for installing such intelligent management and monitoring systems.

The application of just such an assessment procedure is proposed by Rafiq et al. (2004) for performance updating of a deteriorating reinforced concrete bridge exposed to de-icing salts. In this study a methodology is developed for utilizing data obtained from sensors embedded in slabs or beams to update the predicted probabilities of chloride-induced reinforcement corrosion at the level of the reinforcing bars with time.

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5.10.18 Maintenance, condition control and through-life care

These matters are discussed in Section 5.16.

5.10.19 Through-life monitoring and management of the structure

These matters are discussed in Section 5.17.

5.11 Measures to enhance resistance or avoid reinforcement corrosion

5.11.1 Introduction

Reinforcement corrosion is acknowledged to be the most common cause of premature deterioration of reinforced concrete structures around the world. Such deterioration arises for a variety of reasons, which have been discussed earlier in this document. It is clear that good specification, design, detailing and construction practices are essential to achieve durable concrete structures. In many circumstances well-designed and constructed Portland cement concrete structures will be entirely adequate. However, in situations where exposure conditions are very severe additional measures can be required to achieve sufficient protection against reinforcement corrosion.

This Section provides an overview of these additional measures, with a variety of options for enhancing durability and reducing the risk of reinforcement corrosion being considered under the following headings. This synopsis draws upon various sources, but particularly on the detailed guidance given in Concrete Society Technical Report No 61 entitled "*Enhancing reinforced concrete durability*" [Bamforth (2004)].

A. Selection of cementitious materials (Refer Section 5.11.2)

B. Use of admixtures and fibres (Refer Section 5.11.3)

C. Enhanced resistance of the surface of the concrete (Refer Section 5.11.4)

D. Enhanced tolerance to carbonation and chlorides (Refer Section 5.11.5)

E. Avoidance approach (Design-out approach) (Refer Section 5.11.6)

Categories A and B are concerned with improving the resistance of the cover concrete to aggressive actions associated with carbonation and chlorides.

Category C is concerned with measures for improving the resistance of the near surface and involves the use of coatings, surface penetrants, controlled permeability formwork and permanent formwork.

Category D is concerned with measures for improving the tolerance of the reinforcing steel to carbonation and chlorides. These measures include the use of corrosion inhibitors, epoxy coated reinforcing bars and galvanised reinforcing bars.

Category E is concerned with measures that avoid corrosion considering stainless steel reinforcement, non-metallic reinforcement and the use of cathodic prevention/protection.

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5.11.2 Selection of cementitious materials

5.11.2.1 Introduction

Reinforcement corrosion can be avoided if the cover concrete creates an effective barrier between the environment and the reinforcement such that it prevents both the carbonation front and chlorides, in quantities exceeding the threshold level, from reaching the embedded steel bars. The properties of the cement paste, which has a critical role in determining the durability afforded by the cover concrete, can be modified both physically and chemically by the use of different cement types, pozzolanic materials and mineral additions. These include:

- pulverised fuel ash (pfa) and fly ash
- ground granulated blastfurnace slag (ggbs)
- silica fume (sf) or microsilica
- metakaolin (mk).

The effects of changes associated with these cementitious materials include:

- Modification of the pore structure; increasing the penetration resistance of the concrete by reducing the average pore size, increasing in the proportion of discontinuous pores, and reducing micro-cracking. These changes reduce the transport coefficient (e.g. permeability or diffusion) of the concrete.
- Modification of the chemistry of the system such that (in relation to reinforcement corrosion) materials are formed which have a higher binding capacity for chloride ions. This is beneficial as it slows down the rate of penetration of chloride ions into the concrete.

Details of the material standards developed under the European system (CEN) are given in Section 5.21: References and bibliography. Fig. 5.7-2 shows the relationship between EN 206-1 [CEN (2000)] and other CEN standards for the design and execution of concrete structures, including standards for constituent materials and associated test standards – as defined in Fig. 1, EN 206-1 [CEN (2000)].

5.11.2.2 Pulverised fuel ash (pfa) and fly ash

Pfa is a by-product of the dry combustion coal-fired furnaces used for electricity generation. It is also referred to as fly ash in European Standards and in many countries. However, the term fly ash is also used for ashes produced from other furnaces, e.g. incinerator ash. Such fly ash may have significantly differing properties and may not offer the same performance advantages of pfa which are discussed below.

Pfa is commonly added to concrete as a separate material at the mixer, and typically constitutes between 20% and 40% (by weight) of the total cementitious material content, although lower and higher dosage levels have been used. However, pfa may also be interground with Portland cement clinker or factory blended with Portland cement.

There are national standards for pfa and fly ash in many countries. However, a significant issue is that two ashes, both meeting a specific standard, can give very different performance in concrete. Therefore, strict procedures are required for the specification and evaluation of pfa and fly ash for concrete.

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As a Portland cement-pfa mix hydrates more slowly than Portland cement alone; pfa concretes require a lower water-cement ratio to achieve the same grade (i.e. 28-day standard-cured cube strength). This is partly achieved by the reduction in water demand associated with the use of pfa. However, a higher overall powder content, of perhaps 10%, may be required.

Whilst pfa/fly ash is generally beneficial in providing long-term strength and impermeability to hardened concrete, the resulting slow rate of hydration means that both short-term strength and chloride resistance of the concrete also develop relatively slowly. It should be borne in mind that the slow development of these properties might be significant if the structure in question is exposed to an aggressive (severe chloride) environment early in its life, perhaps after only a few days. This could adversely affect the longer term performance of the concrete. This emphasises the importance of proper curing of the concrete. Such considerations would apply to other concrete mixes, such as those containing ggbs, which also hydrate slowly and develop their performance over a longer period of time.

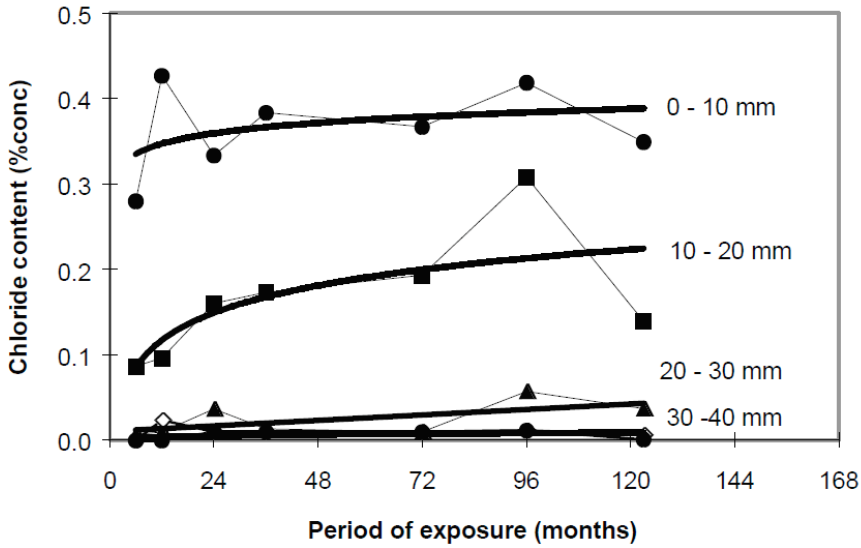

Fig. 5.11.1: Penetration over a 10-year period of chloride ions into a 30% pfa concrete with a water-cement ratio of 0.48 [Ref Thomas (1996)]. Reproduced with permission.

Resistance to chloride ingress: The influence that the addition of pfa has upon the resistance of concrete to chloride ingress is well documented in both laboratory tests and field data [see Chapter 16 of Bamforth (2004)]. On the basis of measurements obtained from coastal and marine structures, a pfa concrete provides an enhancement of at least 3 times the performance of a Portland cement concrete alone, when mature concretes of the same water-cement ratio are compared. This enhancement can increase to more than 10 times when concretes of equal strength classes are compared. Thus the addition of pfa leads to a substantial reduction in the rate of chloride penetration, which is achieved by a combination of enhanced pore structure (increase in the proportion of discontinuous pores) and chloride binding. However, this benefit may not always be apparent in short-term tests on young concrete because of the slower rate of hydration in pfa concretes, where the full benefit of the more refined pore structure in a pfa concrete may not be achieved for an extended period after casting. Thus the initial rate of chloride ingress into the near-surface zone may be rapid, but this reduces markedly over the long term – Fig. 5.11-1.

The influence of pfa content upon the relative apparent diffusion coefficient is illustrated in Fig. 5.11-2, with a lower value representing an increase in the relative resistance of pfa

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concrete to chloride ingress. Fig. 5.11-2 also highlights the importance of evaluating the performance of pfa over a sufficiently long period. The two sets of data represent pfa concretes that are (a) one year old and (b) aged two years or more. The aging effect is pronounced. Fig. 5.11-3 also shows that the apparent chloride diffusion coefficient, D_{ca}, reduces as the time of exposure increases. The slope of the best fit line gives the age factor. The results presented in Fig. 5.11-3 represent a wide range of pfa concretes, made using materials from a variety of sources and different combinations of Portland cement and pfa.

Resistance to carbonation: The rate of carbonation in a pfa concrete is more rapid than in Portland cement concrete, when mature concretes of the same water-cement ratio are compared. However, when compared on the basis of equal strength grade, the difference is insignificant. For practical design purposes, pfa concretes can be treated as being the same as Portland cement concrete of the same strength grade unless very high proportions of pfa (over 40%) are used.

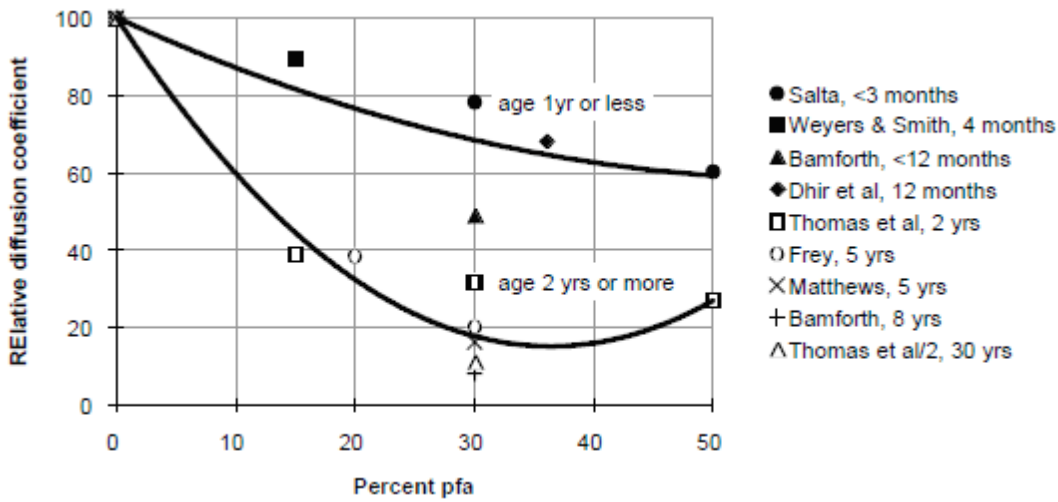

Fig. 5.11-2: Relationship between pfa content and relative apparent diffusion coefficient [Ref Bamforth (2004)]. Reproduced with permission.

Note to Fig. 5.11-2: *Bamforth (2004) indicates that the data presented in this figure comes from the following references: Salta (1994), Weyers & Smith (1989), Bamforth & Chapman-Andrews (1994), Dhir & Byars (1993), Thomas et al. (1990), Frey et al. (1994) and Matthews (1994).*

Prevention of reinforcement corrosion: Pfa also influences pore solution chemistry. Whilst some studies have suggested that the chloride threshold level initiating corrosion may be lower in pfa concrete compared with Portland cement concrete, in others no significant difference was observed. For design purposes it suggested [Bamforth (2004)] that it may be appropriate to assume a modest reduction in the threshold level when using pfa concrete.

The enhanced chloride binding capacity, which limits the extent of wash-out, and the much lower chloride diffusion coefficient achieved with pfa concrete, produces an increase in the surface chloride level. Thus once chlorides have been captured in the surface zone, they are less able to diffuse out of the pfa concrete again.

However, it is the change in the apparent chloride diffusion coefficient, D_{ca}, with time which dominates the overall performance of pfa concretes in chloride-contaminated environments (Fig. 5.11-3). This is much more significant than the effects associated with threshold level and surface chloride level discussed above.

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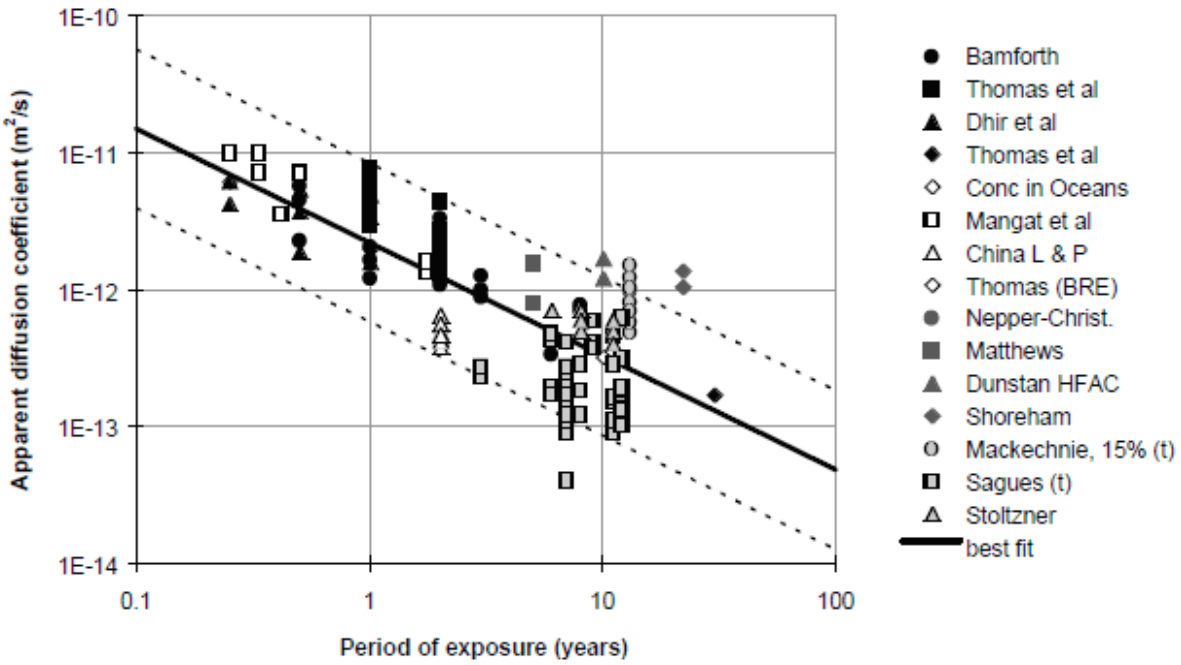

Fig. 5.11-3: Relationship between apparent diffusion coefficient and period of exposure for pfa concretes exposed to regular wetting [Ref Bamforth (2004)]. Reproduced with permission.

Note to Fig. 5.11-3: *Bamforth (2004) indicates that the data presented in this figure comes from the following references: Bamforth & Price (1993), Bamforth & Chapman-Andrews (1994), Thomas (1990), Thomas et al. (1990), Matthews (1994), Taylor Woodrow (1984 & 1986), Bamforth (1999), Dhir & Byars (1993), Dunstan et al. (1993), Mangat & Gurusamy (1987), Thomas & Matthews (1991), Stoltzner (1995), China Light & Power (1992), MacKechnie (1996) and Sagues (1994). Note references for the data attributed to Nepper-Christ and Shoreham were not given by Bamforth (2004).*

Chapter 16 of Bamforth (2004) contains a detailed discussion of the effect of pfa upon the three principal parameters affecting reinforcement corrosion, namely the effect of:

- the surface level of chlorides,
- the rate of migration of chlorides from the surface to the reinforcing steel, and
- the threshold level of chlorides at which corrosion can be initiated.

A summary of values for use in design is given in Table 5.11-1.

Other influences on durability: Concretes containing pfa have a high resistance to sulfate attack. However, the benefit of using pfa for sulfate resistance diminishes when it is used in combination with a Portland cement concrete which has a good sulfate resistance.

The use of pfa also minimises the risk of deleterious expansions due to alkali–aggregate reaction (due to the associated reduction in alkalinity of the concrete), but again the benefit becomes less apparent when the Portland cement concrete is considered to be at low risk.

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Table 5.11-1: Comparison of material factors influence time to onset of corrosion.

Cement type used in concrete	Apparent diffusion coefficient (m^2/s) [1]	Age factor	Surface chloride level (% concrete) [2]	Chloride threshold level (% cement) [3]
Portland cement	9.4×10^{-13}	-0.26	0.51	0.40
Pulverised fuel ash (30%)	2.2×10^{-13}	-0.70	0.62	0.32
Ground granulated blast-furnace slag (65%)	2.1×10^{-13}	-0.62	0.70	0.32
Silica fume (10%)	3.9×10^{-13}	-0.54	0.67	0.30
Metakaolin (15%)	3.6×10^{-13}	-0.26	0.67	0.34

[1] For 0.45 w/c, 400 kg/m3 cement at 20 years.
[2] At the 80% level (i.e. 20% chance of this value being exceeded).
[3] Based on a low risk of corrosion.

5.11.2.3 Ground granulated blastfurnace slag (ggbs)

Blastfurnace slag, the residue from iron smelting, is granulated by quenching molten slag in water. It is then ground to produce ggbs, having a fineness similar to that of Portland cement. Its primary constituents are similar to those of Portland cement, that is calcium, silica and alumina, but occurring in different proportions. Ggbs has latent hydraulicity and will therefore react on its own with water, but only very slowly. When mixed with Portland cement the reaction is accelerated by the presence of alkalis.

Ggbs is commonly added to concrete as a separate material at the mixer, and typically constitutes between 40% and 70% (by weight) of the total cementitious material content, although lower and higher dosage levels have been used. However, ggbs may also be interground with Portland cement clinker or factory blended with Portland cement.

The hydration of ggbs is slower than that of Portland cement, which reduces early strength in ggbs concretes, particularly for high ggbs contents. Ggbs and Portland cement concretes achieve comparable 28 day strengths when the proportion of slag is less than 50%, with the long term strengths of the ggbs concrete exceeding that of the Portland cement concrete. However, at higher ggbs contents strength parity may not be achieved for two months or more.

It should be borne in mind that the slow rate of hydration means that both short-term strength and chloride resistance of the concrete also develop relatively slowly. The slow development of these properties might be significant if the structure in question is exposed to an aggressive (severe chloride) environment early in its life, perhaps after only a few days. This could adversely affect the longer term performance of the concrete. Again this emphasises the importance of proper curing of the concrete.

Resistance to chloride ingress: The influence that the addition of ggbs has upon the resistance of concrete to chloride ingress is well documented in both laboratory tests and field data [refer Chapter 17 of Bamforth (2004)]. Relatively low slag contents of 30% achieve reductions in the apparent chloride diffusion coefficient, D_{ca}, of some 70%; with slag contents of about 65% achieving maximum reductions in the order of about 90% – Fig. 5.11-4. Increasing the proportion of slag beyond about 65% tends to result in a small reduction in resistance to chloride ingress (i.e. a higher D_{ca} value).

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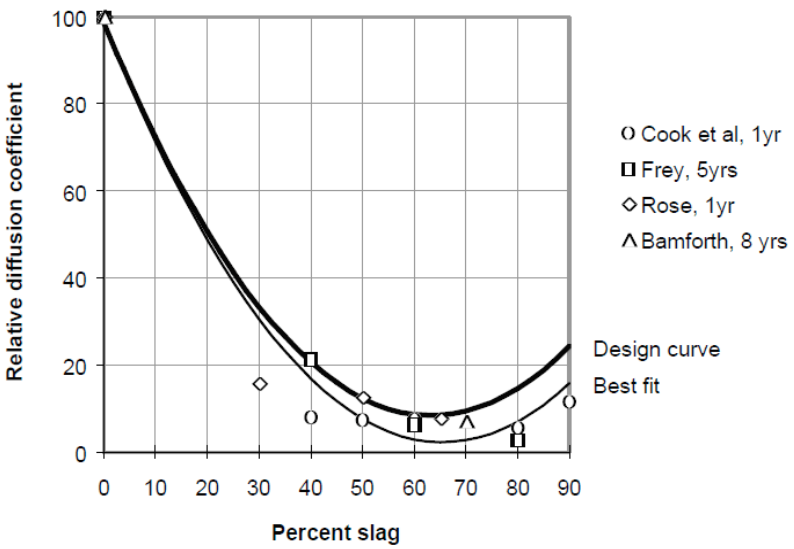

Fig. 5.11-4: Relationship between ggbs content and relative apparent diffusion coefficient [Ref Bamforth (2004)]. Reproduced with permission.

Note to Fig. 5.11-4: *Bamforth (2004) indicates that the data presented in this figure comes from the following references: Cook et al. (1989), Frey et al. (1994), Rose (1987) and Bamforth & Chapman-Andrews (1994).*

The ability of ggbs concretes to resist chloride ingress is relatively insensitive to changes in strength grade. With a slag content of about 40% or more, very low values of apparent chloride diffusion coefficient, D_{ca}, are achieved; which means that it is not necessary to use high strength concrete mixes to achieve a high degree of resistance to chloride ingress. This implies that ggbs concretes are more tolerant of variations in water-cement ratio.

On the basis of measurements obtained from coastal and marine structures, a ggbs concrete provides an enhancement of at least 3 times the performance of a Portland cement concrete alone, when mature concretes of the same water-cement ratio are compared. This enhancement can increase to more than 10 times when concretes of equal strength classes are compared.

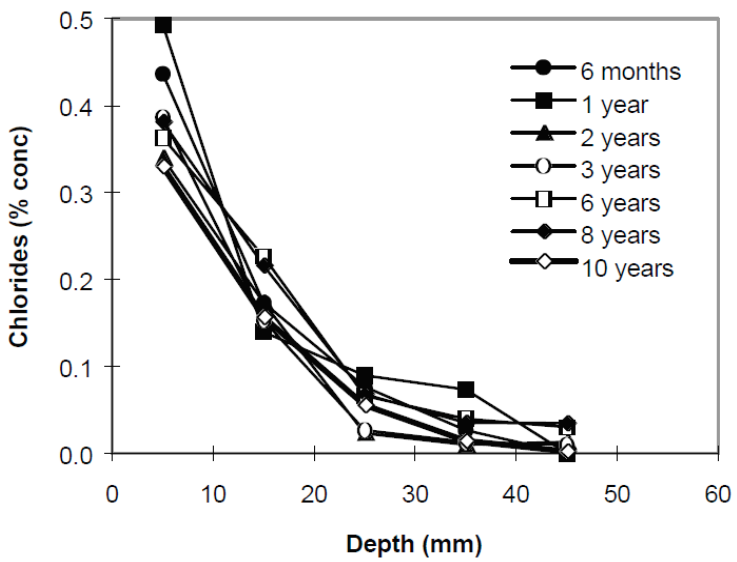

Fig. 5.11-5: Chloride profiles recorded over a 10-year period of splash zone exposure for 70% ggbs concrete with a water-cement ratio of 0.48 [Ref Bamforth (2004)]. Reproduced with permission.

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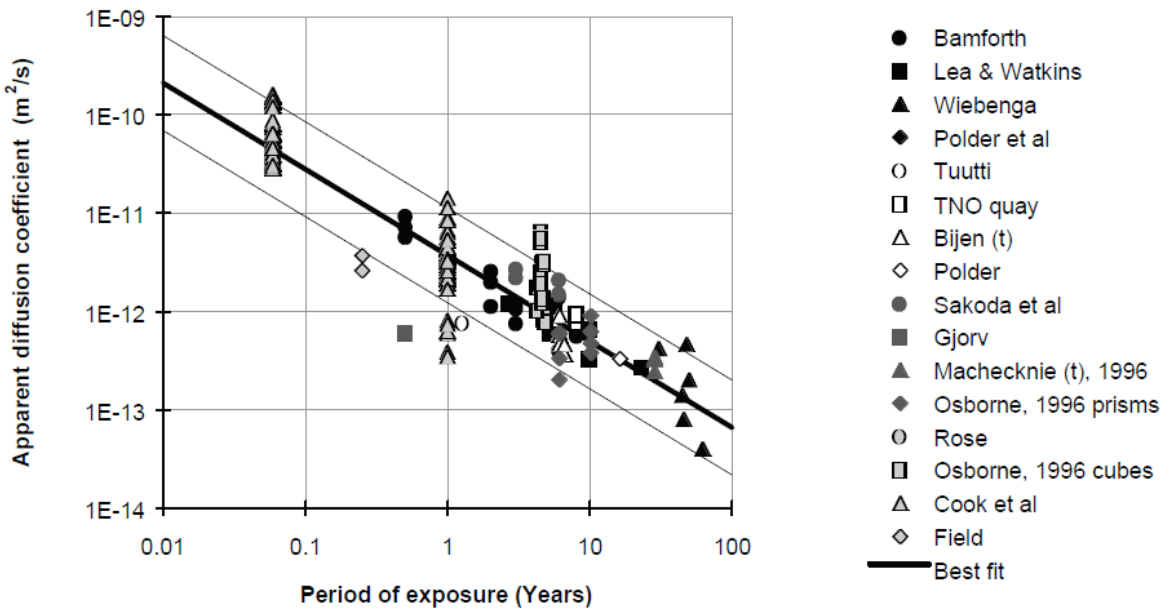

Fig. 5.11-6: Relationship between apparent diffusion coefficient and period of exposure for ggbs concretes exposed to regular wetting [Ref Bamforth (2004)]. Reproduced with permission.

Note to Fig. 5.11-6: *Bamforth (2004) indicates that the data presented in this figure comes from the following references: Bamforth & yChapman-Andrews (1994), Bamforth & Price (1993), Lea & Watkins (1960), Wiebenga (1980), Polder et al. (1997), Tuutti (1982), Bijen (1992), Polder & Larbi (1993), Sakoda et al. (1992), Gjorv & Vennesland (1979), MacKechnie (1996), Osborne (1996), Rose (1987), Cook et al. (1989), Field (1985). Note a reference for the data attributed to TNO quay was not given by Bamforth (2004).*

The enhanced chloride binding capacity of ggbs concrete leads to a rapid building up of chloride in the near surface zone resulting in an average surface chloride content value of about 0.5% by weight of concrete. However, further chloride ingress occurs much more slowly with little change in the initial chloride ingress profile over many years – see Fig. 5.11-5. This characteristic tends to suggest that the initial chloride penetration into the near surface zone is an absorption effect, whereas deeper penetration of chloride is controlled by diffusion processes. Thus short-term tests on young concrete may fail to reflect the potential performance that will be achieved by the ggbs concrete over the life of the structure.

The aging effect in ggbs concretes is pronounced. Fig. 5.11-6 also shows that the apparent chloride diffusion coefficient, D_{ca}, reduces as the time of exposure increases. The slope of the best fit line defines the age factor. The results presented in Fig. 5.11-6 represent a wide range of ggbs concretes, made using materials from a variety of sources and different combinations of Portland cement and ggbs.

Resistance to carbonation: The rate of carbonation of ggbs concrete can be more rapid than in Portland cement concrete. However, for strengths of C25/30 or greater and for slag contents of say 50%, the difference in the rate of carbonation for equal strength grades is likely to be modest. Ggbs concretes are likely to experience a significant higher rate of carbonation for lower strength grades (C16/20 or less) with a higher (>70%) slag content.

Carbonation may increase the porosity of ggbs concrete, potentially increasing it susceptibility to other deleterious actions. However, if surface moisture is available, hydration will occur thereby reducing the surface porosity below that of its uncarbonated state.

Prevention of reinforcement corrosion: Ggbs also influences pore solution chemistry. Evidence of the chloride threshold level required in ggbs concrete to initiate corrosion relative

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to that in Portland cement concrete has been conflicting. For design purposes it suggested [Bamforth (2004)] that ggbs is assumed to have no effect on the total chloride content required to initiate corrosion (i.e. the threshold level).

The enhanced chloride binding capacity, which limits the extent of wash-out, and the much lower chloride diffusion coefficient achieved with ggbs concrete, produces an increase in the surface chloride level. Thus once chlorides have been captured in the surface zone, they are less able to diffuse out of the ggbs concrete again.

However, it is the change in the apparent chloride diffusion coefficient, D_{ca}, with time which dominates the overall performance of ggbs concretes in chloride-contaminated environments. This is much more significant than the effects associated with threshold level and surface chloride level discussed above.

Chapter 17 of Bamforth (2004) contains a detailed discussion of the effect of ggbs upon the three principal parameters affecting reinforcement corrosion (refer to the discussion of pfa concrete above). A summary of values for use in design is given in Table 5.11-1.

Other influences on durability: Concretes containing ggbs have a high resistance to sulfate. The use of ggbs also minimises the risk of deleterious expansions due to alkali–aggregate reaction, with there being no evidence of deleterious expansion in structures containing more than 35% slag. Ggbs concrete can be more susceptible to freeze-thaw scaling than Portland cement concrete.

5.11.2.4 Silica fume (sf) or microsilica

Typically silica fume (sf) is over 90% SiO_2 and is exceptionally fine, being about 50 -100 times finer than Portland cement. Sf does not react with water, but does react with calcium hydroxide to produce calcium silicate hydrates. Sf is available in four forms: undensified, densified, micro-pelletised and slurry. The densified and slurry forms of sf are most commonly used in concrete. Densified sf comprises loose agglomerates that break down in high-shear mixers. The slurry form of sf is the most easily used in concrete. Dunster (2009) provides an overview and case studies of the use sf concrete, summarises its properties and the enhancement given over conventional Portland cement concretes in terms of early strength development, resistance to deterioration, particularly abrasion, impact and chemical attack.

Sf is usually added to concrete in relatively small quantities equivalent to 5–10% sf powder by weight of cement. The use of sf increases the water demand of the concrete markedly, partly due to its fineness and partly due to the rapid acceleration of the hydration processes it promotes. Accordingly, sf is normally used in conjunction with a plasticising or superplasticising admixture. The addition of sf can produce a significant reduction in the porosity of concrete, with the greatest benefits being realised when sf is added to a concrete mix that already has a low water-cement ratio. For a given water-cement ratio, a sf concrete will typically achieve a higher strength than a comparable Portland cement concrete.

BRE Information Paper IP 5/09 [Dunster (2009)] gives further information about the use of silica fume in concrete. It describes the nature and manufacture of silica fume, its uses and the benefits which may be obtained from the use of sf in concrete, its track record, giving details of the available European standards and specifications for the material. The advice given in IP 5/09 [Dunster (2009)] relates to what is known as Class A sf (in accordance with BS EN 13263-1 [CEN (2005))] which comprises at least 85% non-crystalline SiO_2. Class B sf, which has an SiO_2 limit of 75%, is outside the scope of the guidance.

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IP 5/09 [Dunster (2009)] also discusses practical considerations such as the approaches to concrete mix design, batching and mix issues, placing and pumping, curing; as well as the potential properties of concretes containing sf such as compressive and tensile strength, durability, abrasion and impact resistance, chemical resistance, freeze-thaw resistance, the suppression of alkali-silica reaction, permeability and the protection of reinforcement. IP 5/09 [Dunster (2009)] also considers economics and the environmental aspects associated with the use of sf in concrete, reviewing the material's embodied environmental impact and its carbon footprint. The above aspects are illustrated by a series of case studies and responses to frequently asked questions.

Resistance to chloride ingress: The resistance of silica fume concrete to chloride ingress is very sensitive to the water-cement ratio. Fig. 5.11-7 shows that the apparent chloride diffusion coefficient, D_{ca}, reduces as the time of exposure increases, but it also demonstrates the sensitivity to the water-cement ratio. The slope of the best fit line defines the age factor. At high water-cement ratios the influence of sf is modest in concretes of the same strength grade, and only a relatively weak age factor is observed. However, when sf is used in concretes with a low water-cement ratio, there is a much more pronounced aging factor and substantial benefits are gained from long-term exposure which causes a significant reduction in the apparent chloride diffusion coefficient with time. Considerably enhanced resistance to chloride ingress can be obtain for sf concretes where the sf content is higher (sf > 15%), but to date such mixes have only been used in special applications (e.g. proprietary products such as Densit).

Thus the aging effect in sf concretes can be pronounced. The results presented in Fig. 5.11-7 represent a wide range of sf concretes, made using materials from a variety of sources and different combinations of Portland cement and sf. Thus short-term tests on young concrete may fail to reflect the potential performance that will be achieved by the sf concrete over the life of the structure.

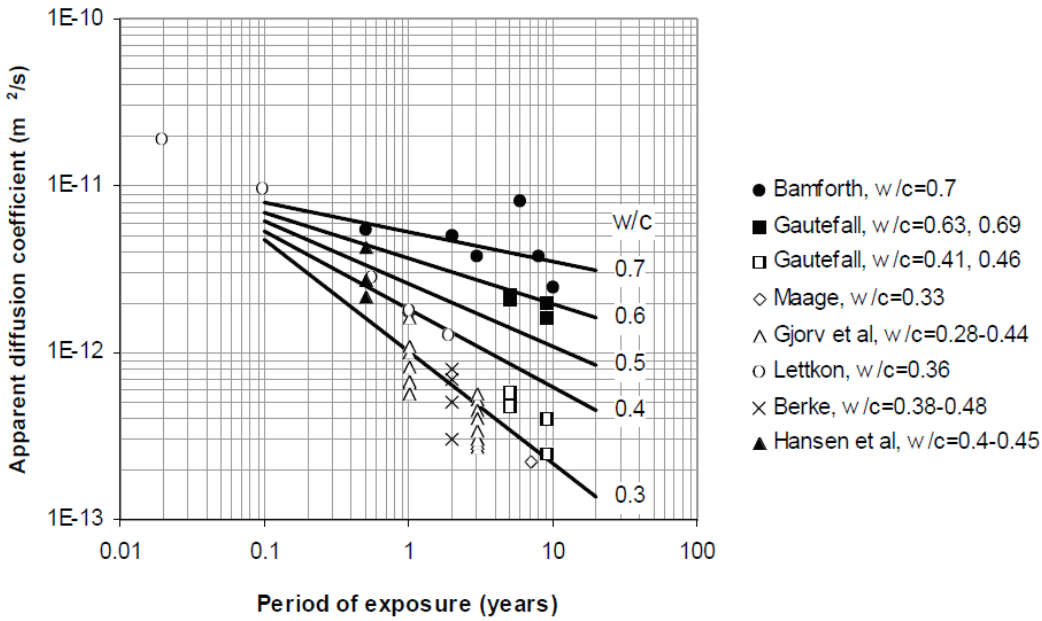

Fig. 5.11-7: Relationship between apparent diffusion coefficient and period of exposure for sf concretes exposed to regular wetting, as affected by water-cement ratio [Ref Bamforth (2004)]. Reproduced with permission.

Note to Fig. 5.11-7: *Bamforth (2004) indicates that the data presented in this figure comes from the following references: Bamforth & Chapman-Andrews (1994), Bamforth & Price (1993), Bamforth (1999), Gautefall (1993), Maage & Helland (1993), Gjorv et al. (1994), Maage et al. (1998), Berke (1989), Hansen et al. (1993).*

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An average surface chloride content value of about 0.5% by weight of concrete has been obtained for sf concretes which have a water-cement ratio of 0.5 or less.

Resistance to carbonation: Studies of the influence of sf upon the rate of carbonation of concrete have produced inconsistent results. Sf reduces the alkalinity of concrete. It seems that higher levels of sf addition may marginally increase carbonation rates and depths relative to a Portland cement concrete of the same strength grade. However, the difference is more marked for lower grade concretes such as C20/25, where carbonation rates in a sf concrete may be increased by perhaps 30% relative to the same strength Portland cement concrete.

Prevention of reinforcement corrosion: Sf reduces the alkalinity of concrete, this effectively increases the chloride/hydroxyl ratio $[Cl^-]/[OH^-]$. The outcomes of studies into the influence of chloride binding upon the threshold level required in sf concrete to initiate corrosion relative have been conflicting. Bamforth (2004) suggests that in view of the uncertainty associated with threshold levels in sf concretes, design should be based on preventing chlorides from reaching the reinforcing bars at levels in excess of about 30% of the value for Portland cement concrete under similar conditions.

Chapter 18 of Bamforth (2004) contains a detailed discussion of the effect of sf upon the three principal parameters affecting reinforcement corrosion (refer to the discussion of pfa concrete above). A summary of values for use in design is given in Table 5.11-1.

Other influences on durability: *S*f concrete has enhanced resistance to sulfates when exposed to sodium sulfate, but not when exposed to magnesium sulfate. It has been reported that concrete with 15% sf performs as well as a sulfate resisting Portland cement (SRPC).

The use of sf can also prevent deleterious expansions associated with alkali–aggregate reaction, but this is dependent upon the type of siliceous aggregate involved. However, it is imperative that the sf is well dispersed to avoid the potential for alkali-silica reaction (ASR).

5.11.2.5 Metakaolin (mk)

Mk is manufactured by feeding clay containing more than 90% kaolinite into a kiln, where it is heated to some 700–900°C. This process thermally activates the clay, which changes form into an amorphous material (metakaolin). The calcined material is then ground and graded. The primary constituents of mk are similar to those of Portland cement, that is calcium, silica and alumina, but with these occurring in different proportions. Mk is commonly supplied as a white powder of almost 100% purity.

Mk reacts only very slowly with pure water. However, it reacts rapidly when mixed into concrete as a result of its activation by the release of calcium hydroxide, alkalis and calcium sulfate during the hydration of the Portland cement. Mk is used either as a partial replacement for, or as an addition to, Portland cement in concrete at levels up to 25% (by weight), depending on application. It is typically used to replace 8–15% of the Portland cement to improve general concrete durability and to reduce the risk of reinforcing bar corrosion.

So far there is only limited field experience of the performance of mk concrete. The higher degree of reactivity and uniformity of mk manufactured in recent years means that the protective qualities of the modified concrete are achieved at a comparatively early age, and in this respect mk might be considered to have similar effects to those of silica fume.

Resistance to chloride ingress and reinforcement corrosion: Laboratory and field data demonstrate that metakaolin greatly reduces the rate of chloride ingress into concrete; with the performance of mk concrete being at least comparable with that of pfa, ggbs and sf concretes.

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For design purposes Bamforth (2004) suggests it may be assumed that the early-age apparent chloride diffusion coefficients will be similar to those achieved with sf concretes at the same water-cement ratio. However, in the long term, he suggest that the diffusion coefficient should conservatively be assumed to decrease at the same rate as Portland cement concrete (i.e. the age factor, $n = -0.26$) due to the lack of long-term performance data for mk concretes. Bamforth (2004) also indicates that account must also be taken of the reduced pH by use of a lower threshold value. He suggests that until more data becomes available a reduction in threshold value to 80% of that of Portland cement concrete should be assumed.

Fig. 5.11-8 also shows the effect of increasing the metakaolin replacement level from 10% to 20%, resulting in about a six fold reduction in the concentration of chloride penetrating to a depth of 20 mm. Bamforth (2004) conservatively estimates, using the error function solution of Fick's second diffusion law, that this corresponds to a more than four-fold reduction in the apparent diffusion coefficient, D_{ca}.

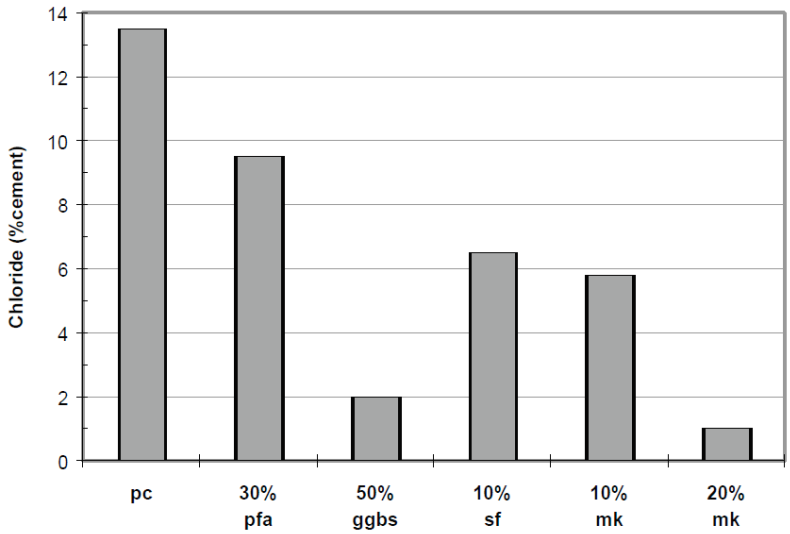

Fig. 5.11-8: Detected chloride levels at 20 mm depth in various types of concrete after 12 months' wetting and drying. [Ref Bamforth (2004)]. Reproduced with permission.

Note to Fig. 5.11-8: *After initial moist curing, the test specimens were submerged in water baths for 14 days. The specimens were then dried at 40°C for 14 days before being subjected to 48 wetting & drying cycles. Each cycle consisted of 1-day ponding with a 0.55 M NaCl solution, followed by drying in a 55% RH and 20°C room for a further 6 days.*

Chapter 19 of Bamforth (2004) contains a discussion of the limited information available upon the effect of mk upon the three principal parameters affecting reinforcement corrosion (refer to the discussion of pfa concrete above). A summary of values for use in design is given in Table 5.11-1.

Resistance to carbonation: There appears to be only limited data available on the carbonation rate of mk concrete under natural exposure conditions. Some data originates from unpublished studies performed by BRE using low- to medium-quality concretes, which indicated that, under indoor exposure conditions, concrete with mk concretes carbonates at a higher rate than a corresponding plain Portland cement concrete of the same water-cement ratio. The increase ranged from 5% to 30%.

Fig. 5.11-9 shows the results of a BRE study involving a control Portland cement concrete tested together with two mk concretes having 10% and 15% mk replacement levels. All the

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concretes were proportioned to have the same water-cement ratio of about 0.55. The test specimens were cured at 20°C and 100% RH for 1, 3 and 7 days; and were subsequently stored indoors under natural carbonation conditions (65% RH and 20°C). The results are presented as a percentage carbonation depth relative to that of the control concrete with 1-day moist curing. There is a general trend of increased carbonation rate with higher metakaolin replacement level. However, it would appear that a longer period of moist curing reduces the negative impact of mk addition upon the rate of carbonation. For design purposes it is suggested that mk concretes be assumed to perform in a similar manner to sf concretes.

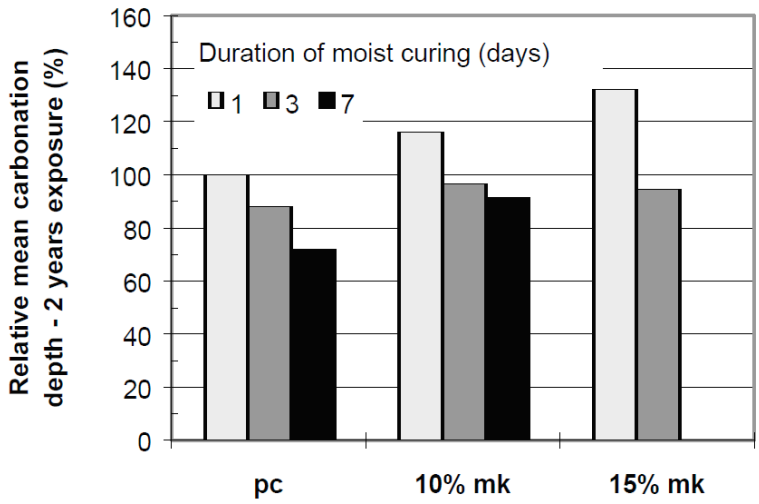

Fig. 5.11-9: Relative mean carbonation depth for concretes with and without mk added, the concretes were cured for different periods then stored indoors under natural carbonation exposure conditions. [Ref Bamforth (2004)]. Reproduced with permission.

Other influences on durability: Concretes prepared with mk have shown improved resistance to deterioration by acid and sulfate solutions. Long-term data have demonstrated the effect of mk in eliminating expansions due to alkali–silica reaction.

5.11.2.6 Use of ternary or quaternary cementitious powder blends

The use of ternary or quaternary powder blends in concrete offers various performance advantages in relation to the properties and performance of both the fresh and hardened concrete. The terms such as ternary cementitious blend, ternary powder blend, triple powder blend and ternary cementitious system are variously applied to concretes containing three cementitious powders. The term quaternary powder blends, and other potential variants on this terminology as noted above, are similarly used for concretes containing four cementitious powders. The specification and use of such concrete mixes are rather specialised and as such are not yet addressed in current concrete standards.

The term ternary cementitious blend is commonly applied to a concrete mix using a combination of Portland cement-pfa/fly ash-sf. Plasticising and superplasticising admixtures will typically be employed with this type of powder blend. However, it is possible that other types of admixture may also be employed, such as air-entraining agents. Ternary blends comprising Portland cement-pfa/fly ash-sf seem to offer significant advantages over binary (two powder) blends and even greater enhancements over the performance of straight Portland cement concrete. In some cases, price differences between the individual (powder) components may allow the ternary blends to compete with Portland cement on the basis of material costs alone.

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It is noted that some workers also appear to use the ternary blend terminology for concrete mixes containing Portland cement-pfa/fly ash-ggbs.

The term quaternary (four) powder blend is used for a blend of Portland cement-pfa-ggbs-sf, again typically being used in conjunction with various admixtures.

One reason for the improved properties and performance relates to the enhanced particle packing that is achieved. Whilst pfa/fly ash particles are often finer than those forming the cement, silica fume/microsilica consist mainly of spherical particles or microspheres which are very much finer (by about two orders of magnitude) than the cement particles. Thus the resulting combination of particle sizes achieves better particle packing since the intermediate particle spaces, which are slightly smaller than the cement particles, are filled by pfa/fly ash particles. The chemical binding of chlorides achieved by pfa/fly ash, which is associated with its aluminium content, works together with the finer pore structure produced by the silica fume to produce a concrete which achieves a high resistance to chloride ingress.

With its lower reaction rate, pfa/fly ash has often been used in high performance concrete (HPC) to reduce the heat of hydration and thereby the peak internal temperature reached during hydration. Although the pfa/fly ash addition is generally beneficial in providing long-term strength and impermeability to hardened concrete, the resulting slow rate of hydration means that both short-term strength and chloride resistance of the concrete develops relatively slowly. In a triple powder blend, the sf improves the early age performance and properties of the concrete, while the pfa/fly ash adds its contribution at a later stage.

The pfa/fly ash-sf combination also has beneficial affects upon the properties of the fresh concrete; for example, potentially enhancing the stability of the fresh concrete, lowering its tendency to segregate and also reduces its viscosity.

The combined use of Portland cement-pfa/fly ash-sf has been investigated by various workers, which includes studies by Sakar et al. (1991), Jones et al. (1997), McGrath & Hooton (1997), Thomas et al. (1999), Lynsdale & Khan (2000), Nehdi (2001), Ramakrishnan V & Sigl (2001), Bouzoubaâ et al. (2002), Nassif et al. (2003), Pandey et al. (2003), Jha et al. (2008) and Gesoglu et al. (2009).

Lynsdale and Khan (2000) studied the chloride and oxygen permeability of concretes made with a triple blend powder combination. Their main conclusion was that the ternary blends allowed minimal chloride transport, even at early ages. In their tests the lowest chloride transport occurred in a low or water-binder ratio for a mix containing 10% sf and 15-20% pfa/fly ash.

In their study of triple powder blend concretes Ramakrishnan and Sigl (2001) noted that the addition of pfa/fly ash and sf reduced the chloride permeability of concrete significantly, while increasing the compressive strength. Based on the analysis of results obtained, they selected a mix as achieving the best combination of properties required for a high performance concrete bridge deck. Another high strength concrete mix was selected for the bridge girders to satisfy the strength requirements for the early stressing of the post-tensioning. The study indicated that the overall cost of the triple powder blend concrete bridges and the standard SDDOT (South Dakota Department of Transport) designed bridges would be about the same. However, the life-cycle cost of triple powder blend concrete bridges was expected to be lower (cheaper) because of their anticipated longer service life and reduced maintenance costs.

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The laboratory studies undertaken by Bouzoubaâ et al. (2002) concluded that the triple powder blend concretes outperformed the binary blend concretes. The addition of sf helped reduce the sensitivity of pfa/fly ash concretes to the effects of inadequate curing. The results showed that the combined use of fly ash and silica fume in concrete is more advantageous in terms of the following parameters: the dosage of superplasticiser, plastic shrinkage, chloride-ion penetrability and the drying shrinkage. Overall various concrete properties were either improved or remained the same.

Nassif et al. (2003) in their study of the development of high-performance concrete for transportation structures in New Jersey concluded that the combination of sf and pfa/fly ash enhances the durability and mechanical properties of concrete. They recommend that a minimum of 5% sf be added to pfa/fly ash concrete to improve its durability. They also observed that the ductility of triple powder blend concrete was enhanced.

The studies undertaken by Gesoglu et al. (2009) gave insight into the use of ternary, and especially quaternary, cementitious blends of mineral admixtures in self-compacting concretes (SCCs). An experimental study was conducted using Portland cement as a control mixture, while the remaining mixtures incorporated binary, ternary, and quaternary cementitious blends of Portland cement, fly ash, ggbs and sf. On the basis of the durability properties of the concretes, the results indicated that the ternary use of ggbs and sf provided the best performance.

Major construction projects which have employed triple powder blend concretes include:

- the Great Belt fixed link, Denmark (discussed further in Section 5.18.1);
- the Øresund crossing between Denmark and Sweden;
- the Confederation Bridge, Canada;
- the Tsing Ma Bridge, Hong Kong.

5.11.3 Use of admixtures and fibres

5.11.3.1 Introduction

Various materials have been developed which can be added to concrete to improve the fresh, hardening and hardened properties of the concrete. In respect of enhancing the durability of concrete by improving its resistance to carbonation and chloride ingress, the following classes of admixture are discussed in this section:

- plasticising admixtures (water-reducing admixtures);
- superplasticising admixtures (high-range water-reducing admixtures);
- integral (hydrophobic) water repellents;
- pore-blocking admixtures.

The addition of fibres to fresh concrete has been used for many years to improve crack control and the ductility of concrete. The following types of fibre are used in concrete:

- steel fibres;
- micro-synthetic fibres – that is plastic fibres, typically polypropylene fibres;

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- macro-synthetic fibres – that is glass and other fibres contributing to the structural properties and strength of the concrete.

The effects of including admixtures and fibres are considered for each type of material in the following sections

5.11.3.2 Plasticising admixtures

Plasticisers are widely used in concrete construction. They are usually based on salts of lignosulfonic acid (lignosulfonates) or polycarboxylic acids. Plasticising admixtures act to disperse the cement particles in the cement–water suspension of the fresh concrete mix. This reduces inter-particle friction. Plasticisers can be used in combination with retarding or accelerating admixtures. Dosage rates vary with specific products, but typically in the range 100–400 ml per 100 kg of cementitious material. Plasticising admixtures act to:

- Reduce the water content (i.e. water-cement ratio) of concrete at a defined workability. Water reductions of up to 10–15% are possible.
- Improve the workability of concrete for a fixed water content (i.e. water-cement ratio).
- Reduce the water and cement content at a fixed water-cement ratio so to achieve a more economic (lower cost) concrete mix. This is perhaps the most common use of these products.

Resistance to chloride ingress: As plasticising admixtures have the ability to reduce the overall water-cement ratio without reducing workability, they act to reduce the capillary porosity of the cement paste thereby increasing its resistance to chloride ingress. As plasticising admixtures can reduce the water-cement ratio by 5–10%, such a reduction may result in a reduction in the chloride diffusion coefficient of some 10 to 25%.

Resistance to carbonation: As plasticising admixtures act to reduce the capillary porosity of the cement paste, they tend to increase its resistance to carbonation.

Other influences on durability: As plasticising admixtures act to reduce the capillary porosity of the cement paste, they tend to decrease the ingress of environmental agents and thereby increase the overall durability of concrete. As plasticising admixtures also act to improve the workability of concrete, they improve the ease of compaction, reduce the risk of honeycombing occurring and help overcome some of the problems encountered during execution works.

5.11.3.3 Superplasticising admixtures

Superplasticisers are also known as high-range water-reducing admixtures (HRWRA). They are usually based on polymeric sulfonated salts of melamine or naphthalene, but products based on other chemicals (i.e. co-polymers) are also available. They work in a similar manner to plasticising admixtures but are considerably more effective, which means that they enable a greater water reduction to be made for a given level of workability in the concrete. Dosage rates vary between specific products, but typically in the order of 1 litre per 100 kg of cementitious material (thus dosage rates are significantly higher than those for plasticising admixtures). Water reductions of 15–25% are possible. Most commercially available products are compatible with all the cementitious materials used in construction.

Resistance to chloride ingress: As superplasticising admixtures have the ability to reduce the overall water-cement ratio without reducing workability, they act to reduce the capillary

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porosity of the cement paste thereby increasing its resistance to chloride ingress. For example, Bamforth (2004) quotes the use of a superplasticiser to reduce the water-cement ratio of a concrete from 0.55 to 0.44 (while maintaining a 75 mm slump), thereby reducing the measured chloride diffusion coefficient from $5.1 \cdot 10^{-12}$ m^2/s to $2.2 \cdot 10^{-12}$ m^2/s. As superplasticisers also act to improve the workability of concrete, they improve the ease of compaction and reduce the risk of honeycombing defects.

As superplasticising admixtures can reduce the water-cement ratio by up to about 30%, this can produce a reduction in the apparent chloride diffusion coefficient, D_{ca}, in the order of 50%. McCurrich (1986) recorded a reduction in the apparent chloride diffusion coefficient, D_{ca}, of 55% with the use of a superplasticising admixture to reduce the water-cement ratio of a 3:1 sand : cement mortar from 0.4 to 0.34.

Resistance to carbonation: As superplasticisers act to reduce the capillary porosity of the cement paste, they tend to increase its resistance to carbonation. On the basis that the use of a superplasticiser could reduce the water-cement ratio of a concrete from 0.55 to 0.44; the relative carbonation depth might be reduced by 80%.

Other influences on durability: Use of superplasticisers act to reduce the capillary porosity of the cement paste, they tend to decrease the ingress of environmental agents and thereby increase the overall durability of concrete.

5.11.3.4 Integral water repellents or 'waterproofers'

Integral water repellents act to increase the contact angle between water and the concrete surface, which increases the pressure required to force water into the concrete. Integral water repellents are based on salts or esters of longchain fatty acids, which react with the calcium ions in cement pastes to precipitate insoluble hydrophobic calcium salts on the walls of the capillary pores. Typical integral water repellents include stearates and oleates. These materials are considered to be more effective in high water-cement ratio concretes than in higher strength grade concrete mixes. These admixtures tend to entrain air in the concrete, but the effect depends upon the type of the admixture and on the dosage rate.

Integral water repellents are commonly added to concrete as powders pre-dispersed in the mixing water or as a liquid using a conventional admixture dispenser. Proper dispersion is essential, as poorly dispersed water repellents create areas which are less water repellent than others. This can lead to a 'patchy' appearance of the hardened concrete.

Resistance to chloride ingress: Integral repellents line the capillary pores with a hydrophobic material, thereby reducing the wetting of concrete by water. However, hydrostatic pressure can overcome this effect, such that the use of these materials is best restricted to concrete exposed to splash zone conditions where only relatively low hydrostatic pressures occur. As integral water repellents act to reduce capillary suction, they can also reduce the ingress of solutions containing chloride ions. This has the effect of reducing surface chloride levels, but there is less influence on the apparent chloride diffusion coefficient, D_{ca}.

If the water-cement ratio of the concrete is reduced to offset any strength loss due to entrainment of air, this will contribute towards a reduction in the chloride diffusion coefficient.

There is concern that some integral water repellents may not maintain their effectiveness in the long term, possibly as a result of leaching, and particularly under conditions where wick action occurs.

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Resistance to carbonation: Integral repellents are not expected to reduce the rate of carbonation of concrete.

Other influences on durability: As integral water repellents reduce the wetting of concrete, they can improve the general durability of concrete exposed to damp conditions or external weathering. The lower water content of the concrete can discourage or reduce algae growth. The combination of reduced wetting of the concrete and the presence of entrained air can also reduce the risk of freeze-thaw damage.

5.11.3.5 Pore blocking admixtures

Pore blocking admixtures are reported to consist of a combination of a water-repellent admixture with a bituminous emulsion. They are normally used together with a superplasticiser. The principle of operation is that the pores are lined with a hydrophobic material, the same manner as with the integral water repellents, in combination with bituminous particles that are compressed into the pores by any external pressure. Pore blocking admixtures are reported to be more effective against externally applied water pressure than the integral water repellents discussed above. Thus pore blocking admixtures seek to make the concrete water resistant against both capillary suction and water under external pressure. From the limited test results available, pore blocking admixtures appear to be most effective in enhancing the performance of lower-grade concrete. Proper dispersion is essential, as poorly dispersed water repellents create areas which are less water repellent than others. This can lead to a 'patchy' appearance of the hardened concrete.

In addition to their pozzolanic activity, silica fume and metakaolin also act as pore blockers, owing to their very fine particle size.

Resistance to chloride ingress: As the water-repellent action of this type of admixture reduces capillary suction, it should also help reduce the build-up of surface chlorides in the splash zone. The bituminous pore-blocking component could contribute to a reduction in the penetration of chlorides in solution under pressure. However, where chlorides were entering the concrete via a diffusion-controlled process, the use of a pore blocking admixtures would not be expected to reduce chloride diffusion coefficient.

Resistance to carbonation: Pore blocking admixtures are not expected to reduce the rate of carbonation of concrete.

Other influences on durability: Pore blocking admixtures should reduce the penetration of water into the concrete, thereby helping improve the general durability of concrete and potentially reducing the risk of freeze-thaw damage.

5.11.3.6 Steel fibres

Steel fibres are produced in a number of different types, including:

- round steel fibres (wire);
- flat steel fibres;
- crimped or deformed steel fibres;
- melt-extract fibres (having an irregular cross-section).

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The length-diameter ratio of typical steel fibres used in concrete is in the range 50–100. Most fibres are mild steel, but stainless steel or zinc-coated fibres are also produced. They are generally added to fresh concrete at volume percentages of up to 1%, but higher volumes are used in some circumstances. Some specialist proprietary high-strength cementitious products can use fibre volumes in the order of 6%. It is essential that the steel fibres are properly dispersed throughout the mix, and that 'balling of fibres' is avoided.

Concrete Society Technical Report 63 (2007) provides guidance on the design of steel-fibre-reinforced concrete.

Resistance to chloride ingress: The addition of steel fibres to concrete does not have any direct effect on chloride ingress into the concrete. However, some aspects of fibre concrete performance do benefit concrete durability in terms of a reduction in bleed of the fresh concrete, control of cracking and a general reduction in crack widths. The combination of smaller crack widths with freedom from excessive bleeding acts to limit chloride ingress by improving the surface quality of the concrete.

Resistance to carbonation: The addition of steel fibres to concrete does not have any direct effect on carbonation. The influence of steel fibres upon improving the quality of the surface concrete is noted above.

Other influences on durability: The addition of steel fibres to concrete improves resistance to erosion and cavitation damage in hydraulic or marine structures, and may improve the resistance in freezing and thawing conditions.

5.11.3.7 Micro-synthetic fibres

Polypropylene fibres are the most common form of plastic fibre introduced into concrete, although other fibres formed from other types of plastic have occasionally been used. Polypropylene fibres are produced either in a fibrillated or in a monofilament form. Fibrillated fibres typically have a diameter in the range 20–200 μm and may be produced in a variety of different lengths for different applications. Polypropylene has an elastic modulus which is substantially lower than that of concrete and is typically in the order of 1-2 GPa. Fibres can be added to concrete during batching or added to the mixer truck on site, but need to be efficiently dispersed throughout the body of the concrete. Additional mixing is usually required to distribute the fibres throughout the mix. Normal concrete curing procedures are adopted.

Polypropylene fibres are incorporated in concrete at dosage rates typically below 0.5% by volume, with a typical addition rate being in the order of 0.1% by volume. For specialised applications much higher dosage rates may be employed, but special manufacturing procedures are adopted in such circumstances.

The beneficial effects of polypropylene fibres typically include a reduction in concrete bleeding, together with better control and reduction of plastic cracking and settlement. This leads to an improvement in the quality of the surface concrete, especially in horizontal surfaces. These effects can reduce water absorption and the ingress of environmental agents.

Monofilament propylene fibres are also frequently added to higher strength concretes to improve their performance under fire conditions, helping to control spalling effects. Whilst the recommended dosage rate for monofilament propylene fibres is 2 kg/m³ or more, there are many examples where polypropylene fibres have been employed to enhance the fire resistance of structures at levels considerably below the 2 kg/m³ minimum specified.

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Concrete Society Technical Report 65 (2007) provides guidance on the use of macro-synthetic-fibre-reinforced concrete.

Resistance to chloride ingress: The addition of polypropylene fibres to concrete has only an indirect effect on chloride ingress. The improved quality of the surface concrete, especially in horizontal surfaces, can reduce water absorption and chloride ingress. However, no long-term performance data is available to quantify these benefits.

Resistance to carbonation: The effects of polypropylene fibres on carbonation are confined to improved quality of the surface concrete due to reduced bleed and the control of plastic cracking.

Other influences on durability: Polypropylene fibres can reduce impact damage and increase the abrasion resistance of the surface concrete.

5.11.3.8 Macro-synthetic fibres

The first macro-synthetic fibres successfully introduced into concrete were glass fibres manufactured from an alkali-resistant glass (Cem-Fil), a process used since the early 1970s. Normal glass fibres are attacked by the alkaline matrix in concrete, causing embrittlement and resulted in premature failure. The fibres used are made by the continuous filament process and usually have filament diameters greater than 10 μm.

A growing range of proprietary macro-synthetic fibres manufactured from various polymers is becoming available from an increasing number of suppliers. These fibres are reported to have an elastic modulus in the range 2-10 GPa, which is substantially lower than that of concrete. The quoted strengths of macro-synthetic fibres are in the range 100 – 650 MPa, which is of the same order as steel fibres. Typically at practicable fibre dosage rates macro-synthetic fibres do not alter significantly the elastic response of the manufactured item relative to that of the parent material, nor do they appreciably change the stress at which cracking first occurs in the hardened material.

The fibres are used in two ways to produce durable concrete structures, either introduced directly into the concrete or, in the case of glass fibres, in the production of glass fibre-reinforced concrete (GRC) sheet materials for use as permanent formwork. Direct introduction into the concrete is intended primarily as a crack-control measure, whereas the use of GRC permanent formwork can improve the long-term durability of a concrete structure.

Influence on durability: The influence of macro-synthetic (glass) fibres on all aspects of durability is envisaged to be similar to that of micro-synthetic (plastic) fibres. That is it is expected to arise primarily through the reduction in bleed in the fresh concrete and the control of cracking in the surface zone of the hardened concrete.

5.11.4 Enhanced resistance of the surface of the concrete

5.11.4.1 Introduction

This Section reviews the measures which can be applied to the surface of concrete to improve its resistance to the ingress of carbon dioxide, chlorides and other environmental agents, including moisture. The bulk of the concrete remains largely unaffected by these measures. In the context of our considerations the primary objective of these measures is to extend the corrosion initiation period and, in some instances, possibly to slow the rate of propagation of corrosion of the embedded reinforcing bars.

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The measures considered are divided into two broad categories, namely coatings/impregnants and special formwork. Coatings and impregnants are applied to the concrete after it has hardened. Special formwork includes controlled permeability formwork (CPF) and permanent formwork made of glass fibre reinforced concrete (GRC). CPF is a specially manufactured formwork liner that is normally used in conjunction with conventional formwork. The whole system is usually removed after a sufficient period of hardening of the concrete. GRC is used to replace conventional formwork directly and is left in place as part of the permanent works of the structure.

Coatings are also frequently used to limit or control the ingress/contact of water and other liquids or moisture vapour/gases – applications that do not necessarily have a protective element. The general mechanism by which coatings/surface treatments protect concrete can be classified according to the film thickness provided on typical surfaces. The generally recognised categories are shown in Fig. 5.11-10.

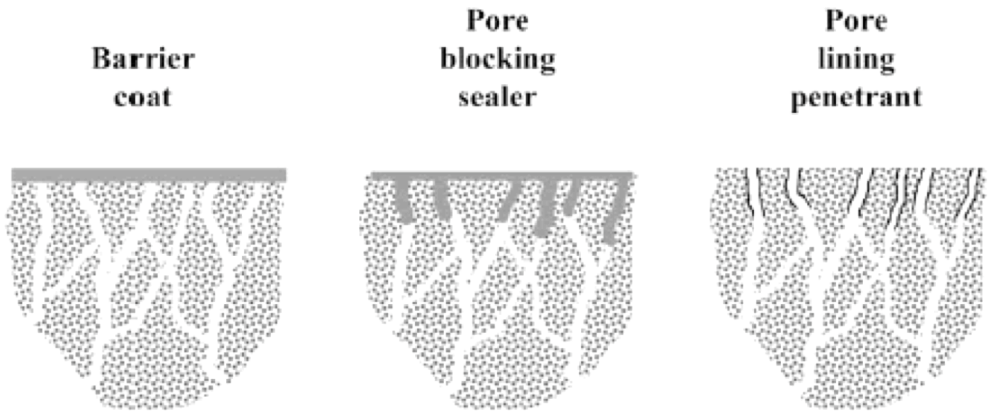

Fig. 5.11-10: Schematic illustration of how surface coatings and impregnants work on concrete [Ref Bamforth (2004)]. Reproduced with permission.

These categories comprise:

- ***Barrier coatings*** which depend on the formation of a continuous bonded film, thus shielding the concrete surface completely.
- ***Pore blocking sealers*** which fill and block the pores and are thus able to resist the ingress of liquids under a hydrostatic pressure. If applied to excess, a visible surface film is formed.
- ***Pore lining penetrants*** which become attached to the pore walls and prevent the type of ingress (e.g. by water) that occurs predominantly by absorption or capillary action.

A very wide range of coatings and related treatments is available for use on concrete surfaces. Each product is different, and its detailed composition is usually known only to the supplier. However, some order can be imposed upon this category of products by the use of a classification system based on the chemical composition of the main component. A generic classification of this type is given in Concrete Society Technical Report 50 (1997) which is reproduced in Fig. 5.11-11.

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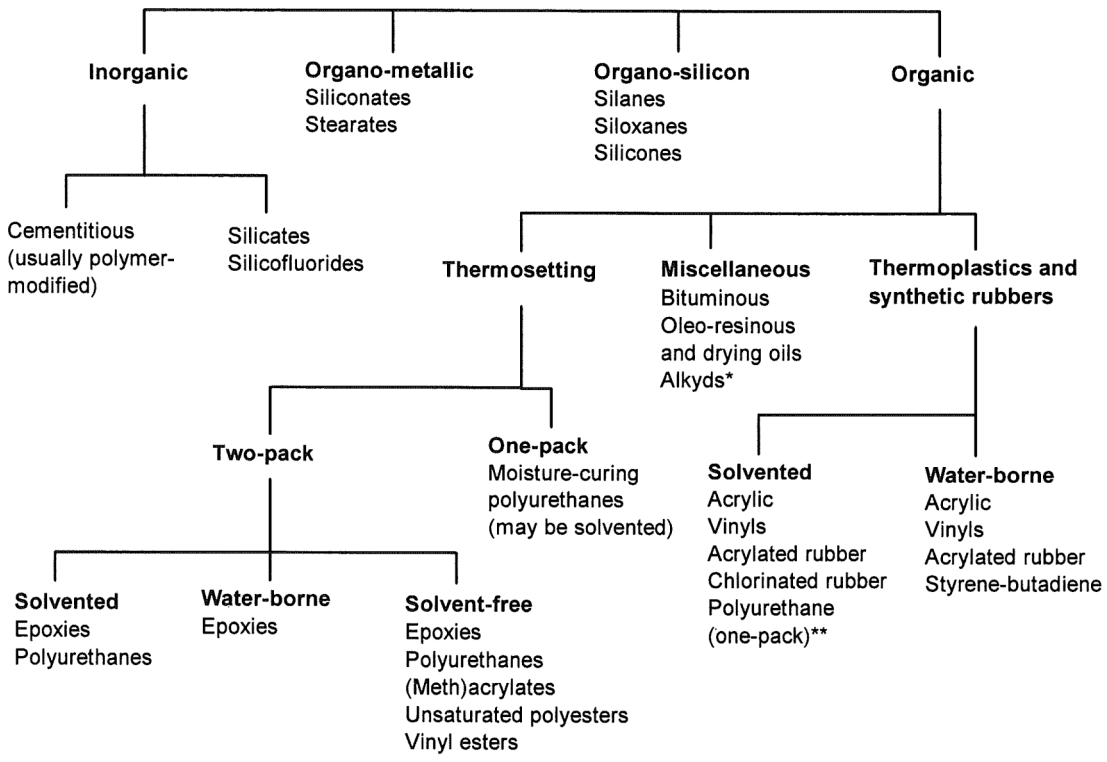

Fig. 5.11-11: A classification of coatings and surface treatments [Concrete Society Technical Report 50 (1997)]. Reproduced with permission.

Surface coatings have been successfully used for very many years on concrete to provide both a decorative and general weatherproofing function. Their application for the protection of concrete, often in aggressive service conditions, is also both widespread and well established. The protective requirement might be subdivided as follows, that is to prevent:

- direct deterioration due to chemical or physical action, e.g. acid attack or abrasion
- indirect deterioration due to reinforcement corrosion following carbonation of the concrete and/or chloride ingress.

5.11.4.2 Surface barrier coatings

The types of coating commonly applied to reinforced concrete (Fig. 5.11-11) include:

- proprietary cementitious systems;
- sodium/potassium silicate-based products;
- one-pack and two-pack thermosetting products;
- solvented and water-borne thermoplastics and synthetic rubbers;
- bituminous systems;

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- products based upon oleo-resinous and alkyd components.

Resistance to chloride ingress: Surface coatings provide the greatest benefits when applied to uncontaminated surfaces, either from new or to concrete after electrochemical chloride removal. Surface coatings are most effective where ingress is driven by any one or more of the following mechanisms: capillary absorption, diffusion or ingress under a hydraulic gradient and wetting/drying cycles. Significant benefits are unlikely to be obtained when surface coatings are applied to chloride-contaminated components, although good barriers may retard further chloride diffusion.

However, the effectiveness of a surface coating is sensitive to workmanship and the adequacy of its application. It has been demonstrated that extensive pinholing in surface coatings greatly reduces their protective ability to just a fraction of that achieved by a pinhole-free coating. It also needs to be recognised that it is difficult to make pinhole-free coatings on concrete, even after applying a surface dressing/fairing coat of a smooth polymer-modified cementitious material to reduce the roughness of the surface of the concrete and to fill small surface holes. Surface coatings are exposed to degrading effects from the environment and from ultraviolet light. A surface coating therefore needs to be maintained, perhaps at intervals of 10–15 years depending on the type of material and exposure. In some circumstances a significantly longer period between overcoating/maintenance works can be achieved.

Resistance to carbonation: Extensive laboratory data and field experience has been obtained which demonstrates that the use of surface coatings can effectively arrest the progression of the carbonation front into the concrete.

5.11.4.3 Surface impregnants

The materials referred to hydrophobic penetrants chemically modify the walls of the concrete pores/capillaries so they become water-repellent (hydrophobic) whilst continuing to allow the transmission of moisture vapour. The appearance and pore structure of the concrete is virtually unchanged. Typically there is a maximum dosage rate beyond which there is limited benefit from additional application of the penetrant.

In contrast, organic surface sealers depend on physical blocking the pores/capillaries. These materials are sometimes known as ‘pore blockers’. The initial application may only partially block the pores/capillaries and further application of the sealer will lead to complete blocking and possibly to the formation of a thin surface film. Organic sealers generally lead to some reduction in the rate of transmission of moisture vapour.

The following materials, supplied as colourless single-pack products, are commonly used as hydrophobic penetrants – sometimes known as ‘pore liners’:

- silanes, siloxanes, siliconates and silicones, and
- metal stearates.

The following materials are commonly available as surface sealers (‘pore blockers’):

- Aqueous solutions of sodium/potassium silicate and magnesium/zinc silicofluorides. These products generate pore-blocking solids by reaction with the concrete. This is in contrast to the systems below, which rely on simple physical blocking of the pores/capillaries following evaporation of the carrier solvent.
- Solvented single-pack (non-reactive) acrylics and polyurethanes.

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- Solvented single-pack moisture-curing polyurethanes.
- Solvented two-pack (reactive) epoxies and polyurethanes.

In some circumstances, water-borne polymer dispersions may be used as surface sealers, such as. SBRs (styrene-butadiene rubbers), acrylics and ethylene-vinyl acetate.

Resistance to chloride ingress: Field experience shows that hydrophobic penetrants (pore lining penetrants) based upon silanes/siloxanes can provide concrete with additional resistance to chloride ingress on surfaces where no hydraulic gradient is present. This qualification can include ponding on horizontal surfaces and, in marine environments, may include vertical surfaces in the splash zone.

Resistance to carbonation: Hydrophobic penetrants (pore-lining penetrants) and the stearate/silicate/silicofluoride types of pore blockers have no significant benefit in reducing the rate of carbonation. Pore-blocking sealers based on polymers will reduce the rate of carbonation although, typically, they are much less effective than an anti-carbonation coating.

5.11.4.4 Controlled permeability formwork

Controlled permeability formwork (CPF) is a manufactured formwork system that allows the passage of water and entrapped air from the fresh concrete and through the formwork, while retaining cement particles and other solids. Principal elements of a controlled permeability formwork system are shown in Fig. 5.11-12. These elements comprise:

- A filter (usually a fabric) that will allow the passage of air and water from the fresh concrete but whose sizing is such as to retain the cementitious particles and other fine solids.
- A drainage system that will transfer the air and water removed from the fresh concrete to outside the formwork.
- Structural support for the filter and drainage elements, which also maintains the required formwork profile and resists the ‘fluid’ pressure from the fresh concrete.

The performance of any given CPF system is dependent on the satisfactory operation of each of these elements, both individually and in combination. The pore size of the filter and the efficiency of the drainage layer are critical to the performance of the overall CPF system.

There are various types of proprietary CPF system which operate in slightly different ways. Price (2000) has described a general classification system for CPF where the different systems are classed as either being Type I, II and III. He also reviewed in detail the CPF systems which were commercially available around 2000. In addition there are a number of ad hoc systems that operate in a broadly similar manner to the proprietary CPF systems. The special permeable formwork liner can take two forms. It can be produced either as a flexible and tissue-like material or be manufactured as a stiff plastic board. In practice, a specially manufactured filter fabric (with a closely controlled pore size) is generally fixed over conventional formwork panels (which may or may not have drainage holes drilled in them).

The use of CPF is restricted to vertical and sloping surfaces, together with soffits. CPF has not been used successfully on the upper horizontal surfaces of concrete elements.

The driving force to remove water and air from the fresh concrete into the CPF system comes from various means including the pressures generated from applying vibration to com-

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pact the concrete, hydrostatic pressure created by the fresh concrete and also from the suction associated with absorption by the formwork materials and system.

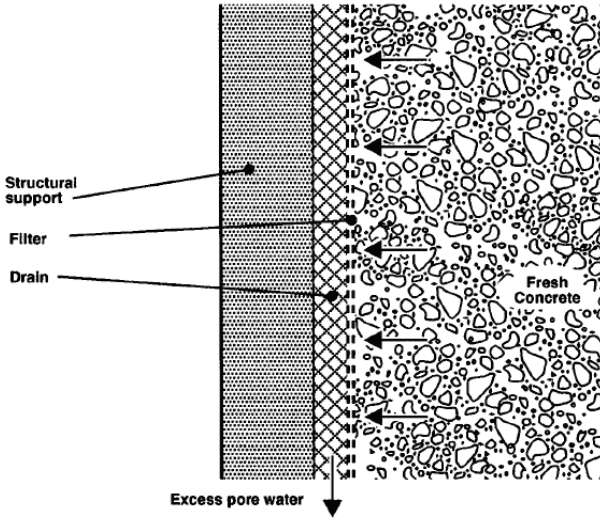

Fig. 5.11-12: Principal elements of a controlled permeability formwork system [Price (2000)]. Reproduced with permission.

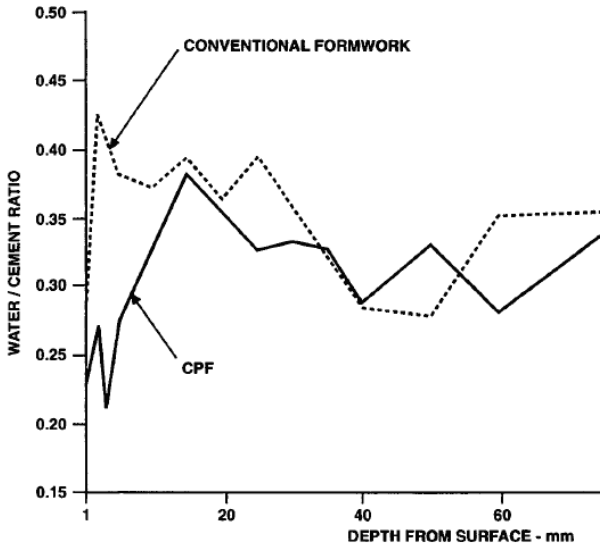

Fig. 5.11-13: Variations in water-cement ratio with distance from the formed surface [Price (2000)]. Reproduced with permission.

The liners in a CPF system are able to improve the quality of the outer zone of the concrete cover by reducing the water-cement ratio and by subsequently enhancing the curing in this thin outer zone. Fig. 5.11-13 compares the variations in water-cement ratio with distance from the formed surface achieved with both conventional formwork and with a CPF system. This shows how in the case of conventional formwork the water-cement ratio increases close to the formed surface. Conversely with a CPF system the water-cement ratio decreases in the vicinity of the formed surface.

When hardening of the concrete subsequently takes place the hydrating cement in the outer zone of the concrete cover adjacent the CPF can draw upon water temporarily absorbed by the liner, providing additional water to support the hydration process. This process provides the water conventionally supplied by wet curing procedures. Thus this helps further improve the quality of the cover concrete, which is critical to enhancing the durability of the concrete structure as the surface skin of the concrete is its first line of defence against penetration of aggressive species from the environment.

Clearly care needs to be taken when constructing a CPF system, as poor or variable workmanship could adversely affect the appearance of the finished concrete structure and potentially the quality of the concrete cover zone.

Influence on durability: The main benefit of casting concrete against CPF is that the water-cement ratio of the concrete close to the formed surface is reduced as a result of the filter that permits water to pass through the formwork while retaining the cementitious particles. The depth of influence of a CPF system is typically about 15 mm (Fig. 5.11-13). CPF has been shown to have this effect on all grades and types of concrete. Skjolsvold (1992a & 1992b) has shown that a 0.65 water-cement ratio concrete cast against CPF has durability characteristics equivalent to a 0.45 water-cement ratio concrete mix cast conventionally. The

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effect of CPF alone is reported to be greater than the benefits which can be achieved solely by efficient wet curing procedures.

Resistance to chloride ingress: Bamforth reports (2004) that comparative tests have shown that the use of CPF will typically reduce the chloride diffusion coefficient to between 40% and 70% of that for the same concrete cast against conventional formwork. The result appears to be applicable to a range of different cement types. In salt spray tests the depth of chloride penetration was reduced to about 20% of the value observed for concrete cast against conventional formwork. Thus CPF acts to improve the quality of the surface zone of the concrete and this provides a more effective barrier to the penetration of chlorides and other environmental agents into the concrete.

Resistance to carbonation: As the CPF significantly reduces the near-surface water-cement ratio and effectively increases the cement content of the concrete an increase in the resistance to carbonation is to be expected. This is compatible with the general correlation found between water-cement ratio and the depth of carbonation of concrete. In accelerated tests, the carbonation depth of a range of concretes cast against CPF was reduced to about 50–70% of that of similar concretes cast against convention formwork. Bamforth (2004) reports natural exposure tests over a 2.5-year period show similar improvements.

The reduction in carbonation depth is particularly noticeable for concretes containing pfa, ggbs or microsilica; concrete mixes which are perceived as being more susceptible to carbonation.

Other influences on durability: Bamforth (2004) reports that the casting concrete against CPF improves a range of concrete surface properties which are primarily associated with the reduction in the water-cement ratio in the surface zone adjacent the formed surface. These improve properties include:

- increased surface tensile strength;
- reduced surface porosity;
- virtual elimination of blowholes;
- increased resistance to weathering, graffiti and algal growth;
- increased abrasion resistance;
- reduced water absorption and capillary suction; and
- increased resistance to frost attack.

Overall the use of CPF enhances the resistance of concrete to penetration of aggressive species from the environment.

5.11.4.5 Permanent formwork (glass reinforced cement, stainless steel etc)

Various forms of permanent formwork, such as glass reinforced cement (GRC) and stainless steel, have been used to provide surface protection to concrete structures located in severe environmental conditions. For example, GRC permanent formwork has been used for more than 20 years, mainly for bridge deck construction. GRC permanent formwork elements are usually produced as flat or corrugated sheets made by the spray process. Rostam reports in Chapter 5 of *fib* Bulletin 3 (1999) the use of thin stainless steel sheet to create permanent formwork to concrete columns located in a severe chloride environment where the structure

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had been exposed to heavy application of de-icing salts for some 30 winters without signs of deterioration.

Let us now consider the properties and attributes of GRC permanent formwork in more detail. Glass fibre-reinforced concrete (GRC) is made from a cement-sand mortar reinforced with 4–5 weight % of alkali-resistant glass fibres. The cement content is normally more than 800 kg/m³, with a water-cement ratio which is normally less than 0.4. GRC permanent formwork elements are usually thin, being some 6–15 mm thick.

As a permanent part of the finished structure, the GRC element forms a protective external skin to the concrete. This skin has a high cement content and a low water-cement ratio. The bond strength between the GRC and in-situ concrete is reported to be at least as good as the inter-laminar strength of the concrete.

In addition to bridge applications, GRC is also used for other formwork applications. For instance, it can be used with an exposed aggregate face to provide a factory-made external finish to in-situ concrete. GRC has also been used for the construction of waffle slab floors.

CIRIA Report C558 gives general guidance on the use of permanent formwork in construction [CIRIA (2001)]. Issues associated with the use of permanent formwork in bridge applications are addressed in BA 36 by the Highways Agency (1990). The Glassfibre Reinforced Concrete Association has prepared guidance on the specification for the manufacture, curing and testing of GRC products [GRCA (2000)].

Resistance to chloride ingress: The apparent chloride diffusion coefficient, D_{ca}, decreases for low water-cement ratios, which reduces the ingress of chlorides and other environmental influence into the concrete. As the typical water-cement ratio of GRC permanent formwork is below 0.40, its apparent chloride diffusion coefficient can be expected to be low. In addition the GRC formwork sheet is manufactured to a minimum thickness and will therefore provide minimum cover to the reinforcement in the permanent structure. Brown (1987) indicated that a combination of GRC and cast concrete has a lower corrosion current than a conventionally cast concrete structure with the same total concrete cover to the reinforcing bars.

Resistance to carbonation: The low water-cement ratio and very high cement content of GRC permanent formwork should result in very low carbonation rates, perhaps less than 10% of the value of a conventional concrete.

Other influences on durability: The low permeability of GRC permanent formwork gives it good resistance to a number of deterioration mechanisms including sulfate attack, water penetration, abrasion and exposure to ultraviolet light.

5.11.4.6 Sacrificial surface layer

Fig. 5.11-14 illustrates the relative benefits to be gained from increasing the concrete cover depth. It shows how the time required for the carbonation front to reach a given depth changes with cover depth. In this example for the model adopted, the nominal cover depth of 40 mm (refer Point A in Fig. 5.11-14) is assumed to give a design service life of 64 years; that is the time required for the carbonation front to reach the embedded reinforcement in uncracked concrete. An increase in the cover depth to 50 mm (Point B in Fig. 5.11-14) increases the time for the carbonation front to reach the embedded reinforcement to 100 years. Thus an increase in the cover depth from 40 mm to 50 mm (1.25 times) increases the time required for the carbonation front to reach the embedded reinforcement by 1.56 times (i.e. 1.25 squared).

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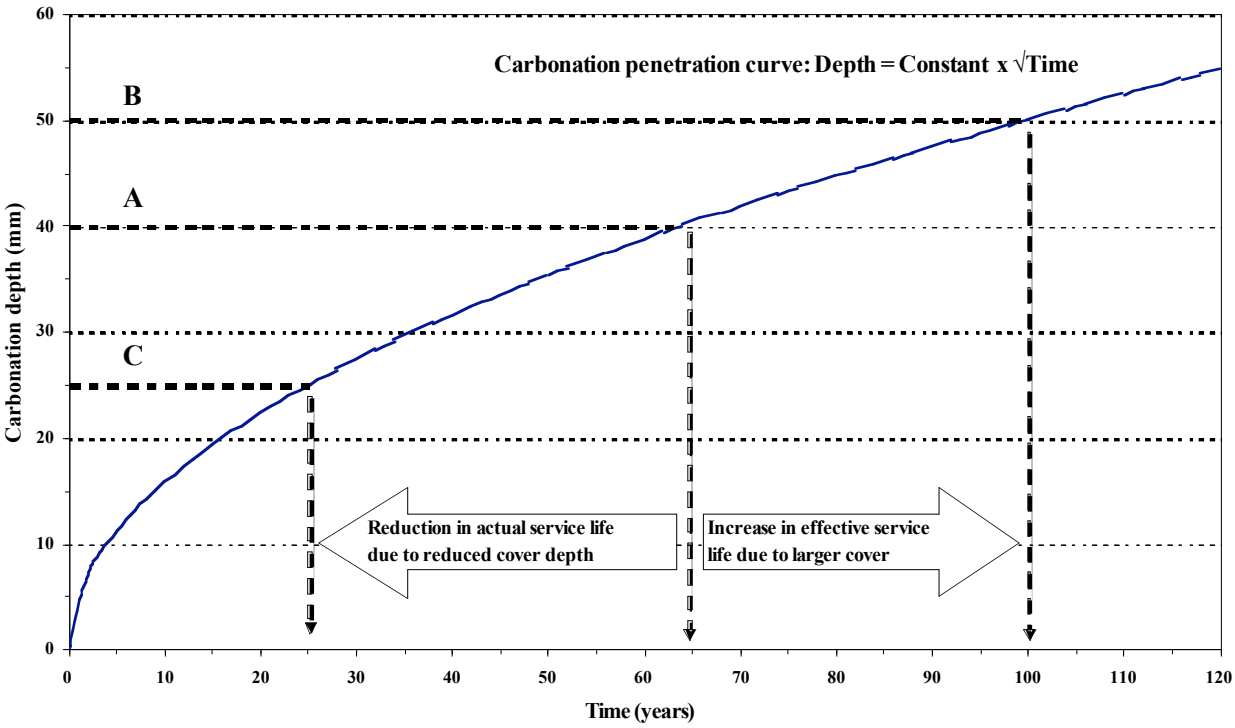

Fig. 5.11-14: Carbonation depth versus time and implications of different concrete cover depths upon the notional service life of a concrete structure.

Thus the provision of extra concrete cover depth, which might be considered as a sacrificial surface layer, can extend the service life of the structure by delaying the depassivation of the reinforcement and therefore the onset of corrosion. Similar benefits can be obtained in environments containing chloride by extending the time taken for ingressing chlorides to reach the threshold level in the concrete (at the depth of the reinforcing bars) which initiates reinforcement corrosion. However, the use of a sacrificial layer will increase the dead weight of the structure.

The provision of additional concrete cover or a sacrificial layer on a concrete structure is used in a variety of situations, perhaps where loss of the concrete surface is expected through erosion by soft water, acid attack, abrasion or other means. For example, additional concrete cover commonly is provided to concrete elements in aggressive ground and a sacrificial topping layer of steel fibre reinforced concrete is frequently provided to concrete deck slabs exposed to freeze-thaw action and used by vehicles fitted with metal-studded winter tyres.

Fig. 5.11-14 also illustrates the implications of decreasing the concrete cover depth. In this example for the model adopted, a reduction in the achieved cover depth to 25 mm (Point C in Fig. 5.11-14) would decrease the time for the carbonation front to reach the embedded reinforcement in uncracked concrete to 25 years. Thus a decrease in the cover depth from 40 mm to 25 mm decreases the time required for the carbonation front to reach the embedded reinforcement to 25 years, down from the original value of 64 years. Therefore a reduction in the cover depth by 37% reduces the time for the carbonation front to reach the embedded reinforcement by 61%.

Similar disbenefits would be obtained in environments containing chlorides, in terms of a reduction in the time taken for the ingressing chlorides to reach the threshold level in the concrete (at the depth of the reinforcing bars) which initiates reinforcement corrosion.

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5.11.5 Enhanced tolerance to carbonation and chlorides

5.11.5.1 Introduction

Modification of the nature and properties of the interfacial zone between the reinforcing steel and the concrete can have a marked impact on the initiation and propagation of corrosion of steel embedded in concrete. The related protective measures are placed into two broad categories, according to whether they act upon the concrete or upon the reinforcing steel.

1. Modification of the concrete by chemical admixtures: These comprise corrosion inhibitors which are incorporated into the concrete at the time of mixing or are subsequently introduced into the hardened concrete from the surface.

2. Modification of the surface of the reinforcing steel by applying a chemically resistant coating to act as a barrier, by covering the reinforcing bars with a sacrificial coating so that the steel is essentially cathodically protected, or by alloying to make the material electrochemically more noble. Galvanised (zinc coated) reinforcement and epoxy-coated reinforcement are examples of steels treated in this way.

3. Modification of the composition or the structure of the reinforcing steel.

5.11.5.2 Chemical admixtures – corrosion inhibitors

Although corrosion inhibitors have been used extensively and successfully in various industries for about 100 years, their application to concrete structures has been limited. Up until the mid-1990's work focussed almost entirely on calcium nitrite; being used extensively in laboratory and field studies, as well as being the centre of large-scale commercial exploitation activities. Recent technical developments have included water-based admixtures of amines and esters, which are commonly known as organic corrosion inhibitors. These are now commercially available, and results from laboratory testing are promising, but to date little experience exists regarding their use in real structures. This section therefore focuses on the use of calcium nitrite.

Calcium nitrite is normally added to the concrete as a separate material at the mixer at a dosage rate of between 10 and 30 litres per cubic metre of concrete, depending on the amount of chlorides predicted to accumulate in the vicinity of the reinforcing steel during the intended design life of the structure. At these dose rates calcium nitrite has little effect upon concrete properties, although it is generally observed to accelerate the setting of concrete and its early age strength development. Calcium nitrite-based corrosion inhibitors formulated with a suitable retarder to offset this action are commercially available. These are recommended for concreting in hot weather, or when it is necessary to minimise the risk of thermal cracking.

Calcium nitrite improves durability by:

- delaying the onset of corrosion (i.e. by raising the chloride threshold level), and
- after corrosion starts, it reduces the corrosion rate of the embedded reinforcement.

After 20 years experience, evidence from existing structures indicates that calcium nitrite performs satisfactorily providing effective corrosion inhibition. Studies suggest that calcium nitrite is most effective when used in high-quality concrete, with adequate cover being provided to the reinforcing steel. The use of calcium nitrite should be viewed as an additional protective measure. On the basis of limited information, the inclusion of calcium nitrite in

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concrete appears to have no significant effect on the rate of ingress of chloride towards the reinforcing steel.

Limited laboratory studies on the effect of calcium nitrite addition on resistance to freeze-thaw attack, along with 20 years of field experience in North America, indicate that the material has no unfavourable effects in properly air-entrained concrete.

Fig. 5.11-15 presents results from laboratory tests reported by Berke (1987) comparing the corrosion behaviour of steel in plain and calcium nitrite-treated concrete under severe chloride exposure conditions. It is apparent that the calcium nitrite delays the onset of corrosion considerably and, after corrosion starts, substantially reduces the corrosion rate. This effect is more pronounced at higher rates of calcium nitrite addition.

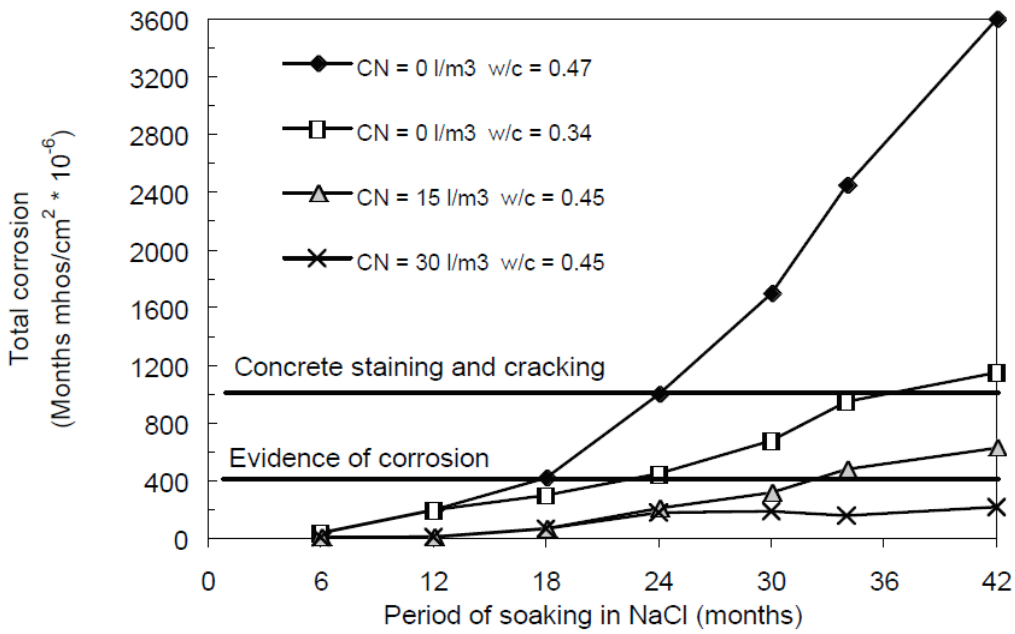

Fig. 5.11-15: Total corrosion versus time in 3% NaCl solution as a function of water-cement ratio and calcium nitrite level (Berke (1987)]. Reproduced with permission.

The penetration of chlorides into concrete from the external environment eventually leads to a situation where the threshold concentration of chloride to nitrite is eventually exceeded. At this point the reinforcement begins to corrode. The nitrite is gradually consumed in this process which explains why calcium nitrite has been found to impart greater levels of protection the higher the dosage rate, the better the quality of the concrete and the greater the cover depth.

Bamforth (2004) reports that analysis of the extensive data upon chloride contents, nitrite levels, corrosion state of reinforcing steel, etcetera has allowed the estimation of the threshold chloride to nitrite ratio. This information is expressed in Table 5.11-2 in terms of the addition rate of calcium nitrite in litres of 30% solution per cubic metre of concrete, which can protect against a particular amount of acid-soluble chloride. The lowest calcium nitrite dosage rate (10 litres per m^3) relates to a chloride content of about 1.2% by weight of cement.

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Table 5.11-2: Recommended calcium nitrite dosage rate (using a 30% solution) as a function of the anticipated acid-soluble chloride level in concrete.

Calcium nitrite solution (l/m^3)	Chloride content (kg/m^3 concrete)
10	3.6
15	5.9
20	7.7
25	8.9
30	9.5

5.11.5.3 Epoxy-coated reinforcement

Fusion-bonded epoxy-coated reinforcement (FBECR or e-bar) was developed in the USA in the 1960s as one approach seeking to prevent the corrosion of steel reinforcement in concrete bridge decks exposed to de-icing salts. The steel reinforcement is produced in a conventional steel mill and is then passed through an additional process by which the epoxy coating is applied by the fusion bonding technique to give an epoxy film thickness of 200 ± 50 µm. E-bar has been used widely in North America from the mid-1970's.

The epoxy coating is intended to prevent aggressive species, such as chloride ions, from reaching the surface of the steel and promoting corrosion. For this objective to be achieved it is necessary to minimise the defects which occur in the coating. Thus it is necessary for the coating to form a film free from holes, voids, cracks, contamination, damaged areas and holidays. The reinforcing bars need to be uniformly coated, with only small deviations occurring in the coating thickness over ridges and valleys in the bar surface. The coating requires good adhesion to the steel surface otherwise rusting may occur under the coating. Epoxy resins normally exhibit good resistance against solvents, chemicals and water. The long-term durability of most epoxy-coatings in concrete is good and they do not soften or deteriorate in the highly alkaline environment.

In order to minimise the defects which occur in the coating, e-bar must be handled, worked and transported with more care than uncoated reinforcing bars. For example, epoxy coated bars need to be lifted with slings rather than chains and should be stored in suitably sized and buffered racks, rather than directly on the ground. It is necessary that each e-bar be inspected so that defects in the coating can be identified and repaired.

If the e-bars are coated before bending, subsequent working of the e-bar may damage the epoxy coating. Bending of coated bars on site is possible provided that nylon-faced mandrels are used and that the bend radius is kept within the strain capacity of the coating (bend radius of three times the nominal diameter of the bar).

E-bar fixing practices also need to minimise the possibility of damage to the epoxy coating which can lead to corrosion problems and it is recommended that conventionally fixed reinforcement is not used to support e-bar cages. A higher degree of access and supervision is necessary with e-bar assemblies than is required for those manufactured from uncoated bar. It is also recommended that plastic-coated tie wire and support chairs/spacers be used when fixing e-bar in-situ.

The allowable extent of defects in the epoxy coating varies according to the country of use and local practice. However, repair of defects on site is commonly permitted up to a certain level of defect, usually expressed as a percentage of the total coating area. For the UK this is four defects/bar length and less than 10 mm^2 total defect area per metre length of bar. Repairs

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are made using a brush-applied liquid form of the epoxy coating. Damaged areas of coating are not always easy to find in zones of congested reinforcement and such patch repairs did not always achieve the level of protection provided by the original (undamaged) epoxy coating.

Placing of concrete around the e-bar has to be undertaken with great care as concrete placement and compaction activities can produce significant levels of damage to the epoxy coating to the bars. To minimise this risk it is important that the concrete is deposited as close as possible to its point of use and that the operator does not strike the e-bars with the vibrator whilst compacting the concrete.

Epoxy coating significantly reduces mechanical bond resistance and increasing slippage between reinforcing bars and the surrounding concrete, relative to the performance of uncoated bars. The thicker the coating, the greater this effect is. The bond strength is greatly reduced for e-bars with a coating film thickness larger than 500 μm (0.5 mm). Where the coating thickness is less than 250 μm, a bond strength comparable with that of uncoated bars is achieved. Although, e-bars tend to experience a higher slip at lower loads than comparable uncoated bars, the total slip at ultimate load for e-bars is comparable with that obtained with uncoated reinforcing bars. The width and spacing of cracks in concrete is increased by epoxy-coating the reinforcement.

One side effect of the use of e-bars is that it is not practicable to control reinforcement corrosion by means of cathodic protection because the e-bars are electrically isolated from one another.

There are a number of mechanisms by which epoxy coating failures lead to corrosion of the embedded e-bars.

- Localised coating defects allow aggressive agents such as chlorides to reach the reinforcing steel, which results in some localised corrosion and loss of reinforcing bar cross-section. Generally the consequences of these small defects are limited.
- Local defects in the epoxy coating allow a process similar to crevice corrosion to progress, with the corrosion products lifting the epoxy coating in the area around the coating defect. However, the bond between the epoxy coating and the steel is normally too good for a large area of under-coating corrosion to develop. At these locations oxygen availability is limited beneath the coating and corrosion tends to progress at a slow rate.

In some circumstances macro-cell corrosion can develop in areas where the epoxy coating has been damaged. Fig. 5.11-16 illustrates such a situation. In this case the coating defects occur on the top surface reinforcement which is located in a wet chloride contaminated zone at the top of a slab. This becomes a zone which has a high electrical conductivity. Conversely the bottom face reinforcement is uncoated in this example, being located in relatively dry and well aerated/oxygenated concrete. In these circumstances the bottom face reinforcement acts as a large cathode and the small areas of the top steel exposed by the coating defects act as anodes. If there is an electrical connection between the coated top face steel and the uncoated bottom face steel, electrical current will be able to flow promoting corrosion. If a sufficiently high difference in electrical potential develops between the coated top face steel and the uncoated bottom face steel, large corrosion currents are able to flow. In these circumstances high current densities can develop at the localised anodes created at the defects in the epoxy coating to top face steel, which will result in rapid corrosion loss of the top face reinforcement at these locations.

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To counteract this type of problem, both the top face and bottom face deck slab reinforcement would now generally be coated.

Similar circumstances can occur in other situations, such as in concrete storage tanks exposed to aggressive species such as chlorides contained in the stored liquor. In these situations the inner face of the concrete tank wall would be effectively be saturated, whereas the atmospherically exposed outer face would tend to be dry and would mean that the outer face reinforcement would be located in well oxygenated concrete.

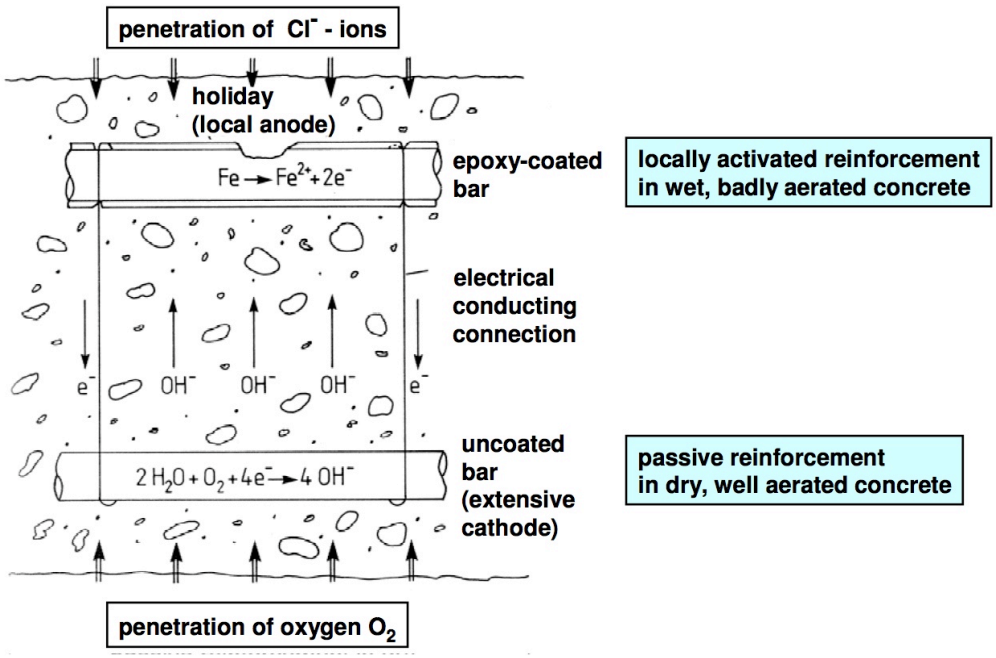

Fig. 5.11-16: Corrosion cell between epoxy-coated reinforcement (e-bar) & uncoated reinforcing bars, such as might occur in a carpark deck slab [Nürnberger (1995)]

For some applications it is feasible to prefabricate the reinforcement cage and then coat the assembled reinforcement. This approach is particularly suitable for precast elements where there is a high degree of repetition. If the reinforcing bars are connected by welding (before coating) this ensures the electrically continuity of the reinforcement cage, which would allow the subsequent installation of cathodic protection to control reinforcement corrosion should this be required. It is suggested that more durable epoxy coating formulations can also be used when the strict requirements for flexibility of the coating, required for the post-coating bending of the reinforcement, are no longer needed.

A number of studies of long-term durability performance have reported that e-bar has performed satisfactorily in bridge deck applications. These studies suggested that in general e-bar would be expected to perform significantly better than uncoated reinforcement. A large-scale investigation concerning the performance of epoxy-coatings to reinforcing steel bars was carried out by the US Federal Highway Administration with particular reference to bridges which were up to 20 years old at the time of investigation. The general result reported by Clear & Virmani (1983), Isecke (1988) and Smith & Virmani (1996) was that a noticeable reduction in corrosion damage to reinforcement had taken place over the period in which epoxy-coating has been in use. The study investigated 92 bridge decks.

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Another extensive investigation into the performance of epoxy-coating reinforcement in bridges in the USA [Weyers et al. (1997)] came to a very critical view on the behaviour of e-bars under severe environmental conditions. The study found that the epoxy coating of e-bars in wet concrete had experienced extensive loss of adhesion (from on the steel substrate) under maritime conditions in some bridge decks and piles. The scope of the investigations were extended to a further 18 bridge decks which were from 3 to 20 years old [Pyc et al. (2000)]. The study revealed that, although the requirements of the then current standards for e-bar had been satisfied, there were numerous instances of the epoxy coating debonding around coating imperfections. Perhaps of greater concern was that this outcome appeared to be independent of the presence of chlorides in the vicinity if the reinforcement. No correlation was found between the age of the structures concerned and the tendency for there to be corrosion loss of the reinforcing steel or for the breakdown of the epoxy coating bond to the steel substrate.

One notable failure in the performance of e-bars which has received considerable attention concerns the five bridges built in the late 1970's and early 1980's linking the Florida Keys, USA [refer Read (1989), Sagüés et al. (1990), Sagüés et al. (1994) and Sagüés (1995)]. In a period of less than ten years following construction these bridges exhibited severe corrosion of the substructure, primarily in the splash and evaporation zones. High temperatures and humidity, as well as extremely high chloride content of the surrounding seawater, created a very harsh environment for these reinforced concrete structures. The principal factors which promoted severe deterioration of the e-bars in these bridges were reported to be:

- Poor workmanship in the manufacture of the epoxy coated bars. This concerned both poor quality of the original coating and poor substrate preparation; where chloride contaminations of the substrate existed behind the coating from the beginning. Testing showed serious coating fracture and debonding when it was subjected to either the standard bend test or fabrication bending.
- That corrosion was initiated in fabricated bars at the bends, subsequently progressing into the straight lengths of the bars.
- That the cover to the bars were less than 25 mm, and in all cases below the specified cover depth. The damage which occurred included extensive metal loss, with additional localised pitting, and spalling of the concrete cover. The deterioration could not be traced to any particular deviation from the construction design or the materials specification. The corrosion appeared to develop at imperfections in the coating, especially where debonding of the epoxy coating had taken place.

Sagüés (1995), who reported on various structures built using e-bar outside of the Florida Keys, found that they were generally corrosion-free. However, many had experienced extensive debonding of the epoxy coating from the steel substrate, irrespective of whether significant chloride contamination existed in the vicinity of the reinforcing bars, or not. Sagüés (1995) postulated that the corrosion-free service life of these structures was likely to primarily result from good concrete quality and thick cover, rather than to be due to the use of epoxy-coated reinforcing bars.

Resistance to chloride ingress: Although the ability of e-bar to withstand exposure to various levels of chloride ion in concrete has been extensively researched, the evidence is not conclusive. What is clear is that the presence of defects in the epoxy coating, coupled with chloride ions and water, will lead to the corrosion of the affected reinforcing bars.

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Without defects in the epoxy coating, chlorides take a long time to reach the reinforcing steel because they have to diffuse through the coating. Results show that this process is extremely slow, and thus the steel is effectively protected from corrosion in these circumstances.

Bamforth (2004) reports that in North America it is thought that slight damage to the epoxy coating can be tolerated without significantly reducing the overall performance of the e-bar and the element that it is situated within. North American practice has been to provide epoxy-coated reinforcing steel for the top mat of reinforcement in slabs exposed to severe chloride attack, while leaving the lower mat of reinforcing steel uncoated. As a result small defects in the epoxy coating become sites of intense anodic activity, driven by the large cathodic area of the uncoated lower mat of reinforcing steel. Although the majority of bars will be passive, it is thought that the high level of localised corrosion can lead to very rapid section loss at locations where the epoxy-coating of the top mat reinforcing steel is damaged.

In Europe a more critical view is taken of this North American practice because of the imbalance in the size of the anodic and cathodic areas.

Resistance to carbonation: The epoxy coating acts as an impermeable barrier to the water and oxygen necessary for corrosion. In addition, the coating is an electrical insulator and therefore will interrupt the electrical continuity of the corrosion cell. As a result the epoxy coating will generally prevent carbonation-induced corrosion of the reinforcement. Furthermore carbonated concrete has a lower pH than uncarbonated concrete, which should improve the durability of the epoxy coating as epoxies are more durable in neutral environmental conditions. Therefore it seems reasonable to infer that e-bar should enhance durability in a carbonated concrete environment.

Other influences on durability: Epoxies are good insulators and therefore e-bar may be useful in situations where the reinforcement could be subjected to stray-current corrosion; such as in structures in the vicinity of railways using direct current traction systems.

5.11.5.4 Galvanised reinforcement

Galvanising reinforcement protects the steel by two mechanisms. In the first it forms a physical barrier between aggressive agents, corrosion reactants and the steel; and in the second by corroding sacrificially. The fact that the protective zinc oxide layer corrodes sacrificially has led some workers to conclude that the protection offered by galvanising is finite. However, others believe that the rate of loss of zinc is very low, which would provide a significant improvement in service life achieved by reinforced concrete structures.

Sacrificial protection to steel can be given by various coating metals, but for reinforcing bars the main activity has been concentrated on the use of zinc. Zinc can be applied in several ways: hot-dip galvanising, zinc spraying, sherardising and electrolytic coating. Of these methods, hot-dip galvanising has been the most widely used for protecting reinforcing bars.

Hot-dip galvanising involves dipping the steel to be protected into a bath of molten zinc. When the steel bars are submerged in the zinc bath, the two metals react producing a coating which has several layers of iron-zinc alloys metallurgically linked to the base steel, and an external layer of nearly pure zinc. The sequence of layers from the steel substrate to the exterior surface of the coating is shown in Fig. 5.11-17.

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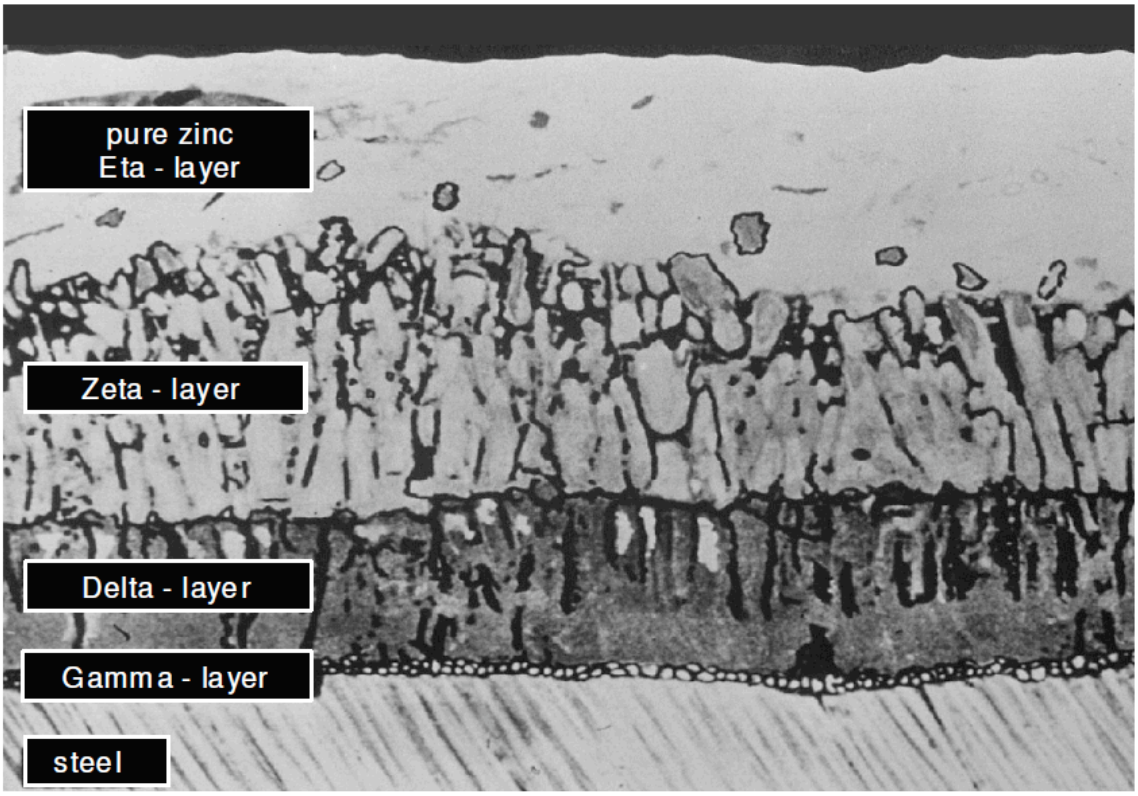

Fig. 5.11-17: Structure of the coating produced by hot dip galvanising of steel [fib Bulletin 49 (2009)]

The efficiency of hot-dip galvanising at preventing corrosion depends on the composition of the steel. An increase in the silicon content of the steel results in an increase in the overall thickness of the iron-zinc layer. Fig. 5.11-17 illustrates the various layers forming the iron-zinc coating. The delta and zeta layers are very hard and relatively brittle. These are located below the outer eta layer, which comprises almost pure zinc. It is suggested that the iron-zinc layer is much more resistant to corrosion than the pure zinc layer. For the galvanising coating not to be too thick, the silicon content of the reinforcing steel should be between 0.15% and 0.25% by weight. To provide adequate adhesion and to avoid cracking or flaking if impacted or bent on site, the thickness of the iron-zinc layers should not be exceed 80 to 120 µm. However, to achieve good protection in concrete, the coating should be as thick as possible. A maximum limit of 200 µm (0.2 mm) is often recommended.

Care has to be taken when hot-dip galvanising cold deformed reinforcing steels, as there is the possibility that the zinc bath may causing liquid metal embrittlement (LME) of the reinforcement [Feldmann et al. (2006)]. Such steels are no longer in common use.

Hot-dipped galvanised reinforcing bars have been used for about fifty years. Galvanised reinforcement is widely used in the USA, Australia, Bermuda, Netherlands, Italy, UK and Germany. In the USA about 2% of the reinforcement used is galvanised, whereas in Europe the figure is about 1%. Galvanised reinforcement is mainly used for concrete bridge and highway construction exposed to high levels of chlorides arising from the application of de-icing salts or from marine environments. It is also used for industrial facilities such as cooling towers, chimneys, paper mills and chemical processing plants. Practical results have generally proved that galvanised reinforcing bars are effective, whereas laboratory tests have tended to indicate that the technology would provide no significant long-term benefits over uncoated steel reinforcement, particularly in high-chloride environments. Of course, there are some structures where galvanised reinforcement has not performed well and which have necessitated programmes of costly maintenance works to be undertaken.

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In comparison with other coating methods for reinforcement, galvanised steel is relatively cheap. This has meant that galvanised reinforcement has been used in the precast industry to reduce cover depths. In turn, this has contributed to a reduction in section size and the weight of precast elements such as cladding panels.

Resistance to chloride ingress: Extensive research has been conducted in an attempt to establish the performance of galvanised steel in chloride-contaminated environments. Although there is some contradictory evidence in the literature (mainly in the results of laboratory studies compared with surveys of field performance), it seems that galvanised reinforcement will perform satisfactorily provided that the chloride level is not excessively high. A safe threshold value of 1% chloride by weight of cement is suggested for design purposes. It seems that beyond this level the zinc coating ceases to be effective, and the underlying steel would then suffer intense corrosion at a rate comparable to that of uncoated steel reinforcing bars.

Fig. 5.11-18 shows the results of marine exposure tests upon normal-weight concrete beams reinforced with normal and galvanised reinforcing bars reported by Nürnberger& Beul (1991). The loss of zinc from the galvanised reinforcing bars increases greatly when the chloride content increases above about 1.5% by weight of cement. The marine exposure tests demonstrated that the chloride concentration required to depassivate galvanised reinforcing bars was significantly greater than the level required to initiate corrosion of uncoated steel reinforcing bars. However, the test results indicated that galvanised reinforcing bars are unsuitable for use as corrosion resistant reinforcement in concrete which is very heavily contaminated with chlorides. If the chloride content exceeds about 1.5% by weight of cement, the corrosion rate increases rapidly.

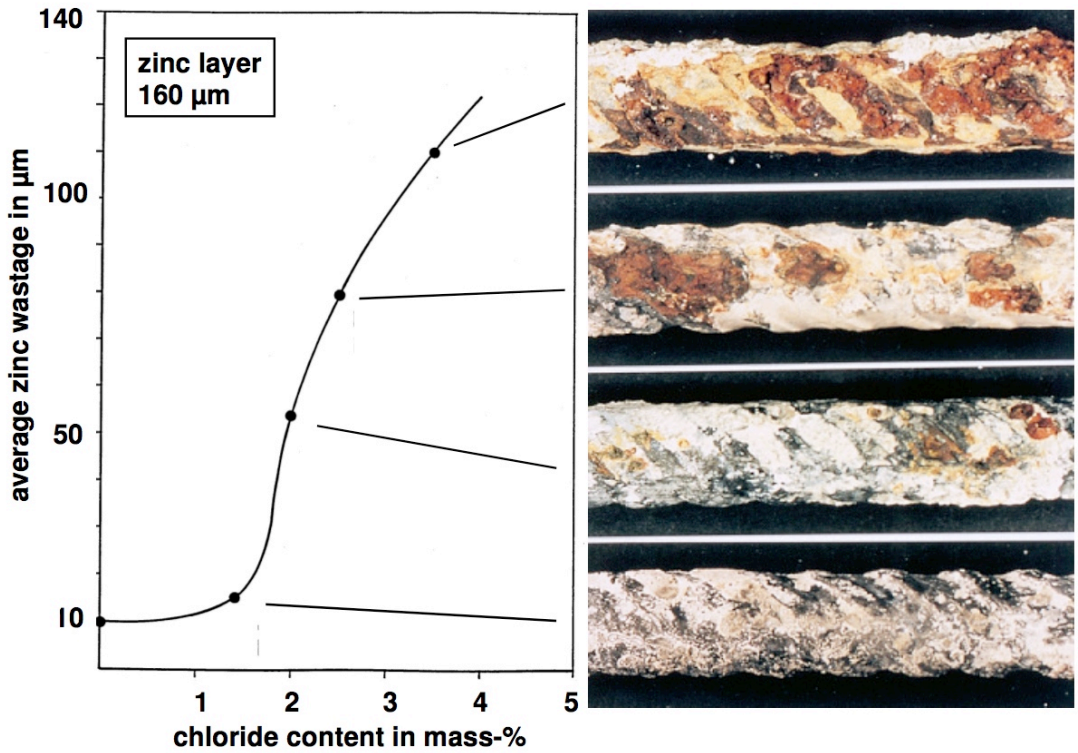

Fig. 5.11-18: Loss of zinc from galvanised reinforcing bars embedded in normal-weight concrete as a function of chloride content after seawater exposure of 2.5 years – for concrete cover depths of 15 and 25 mm [fib Bulletin 49 (2009)]

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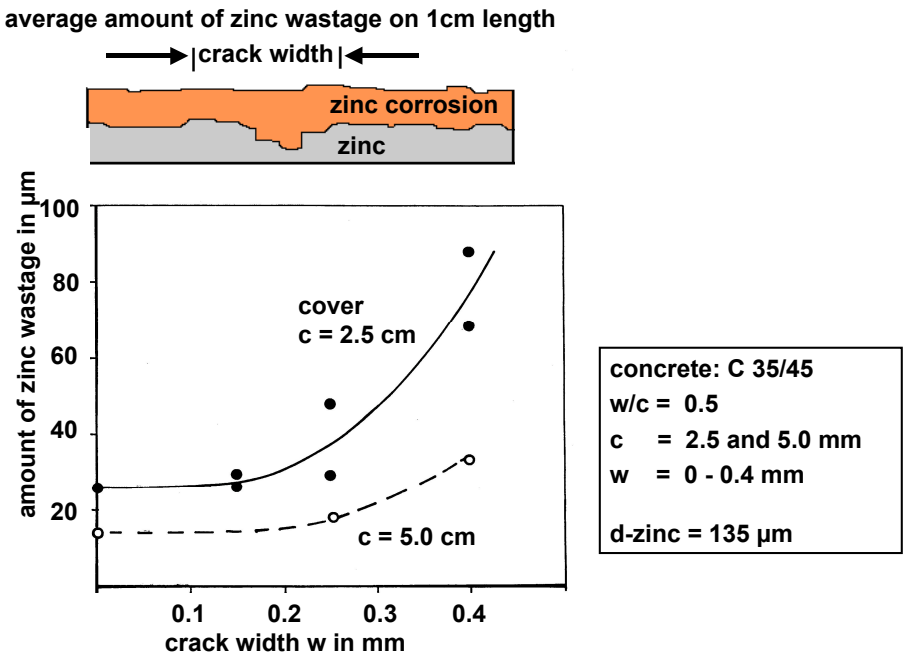

Fig. 5.11-19: The influence of crack width on the corrosion of galvanised reinforcing bars under seawater splash zone exposure conditions [fib Bulletin 49 (2009)]

Fig. 5.11-19 demonstrates that crack width has an influence upon the corrosion rate experienced by galvanised reinforcing bars exposed to seawater splash zone conditions. It would seem that the width of the crack is of minor importance. However, other parameters as concrete quality or cover thickness are expected to have a considerable influence on whether a particular crack will lead to localised corrosion damage, further cracking and concrete spalling. Clearly cracks can contribute to the corrosion process, with the sides of cracks quickly carbonating, and in chloride contaminated environments, they represent a quick way for the chlorides to reach the reinforcement.

A conceptual model for corrosion of galvanised steel in concrete has been proposed by Yeomans (1994). This is illustrated in Fig. 5.11-20, along with the traditional corrosion model for normal carbon steel. An additional stage has been included in the galvanised steel model to describe a period of protection during which the galvanised coating reacts and dissolves and, in so doing, protects the reinforcing bar steel substrate against corrosion. Though the zinc coating is being corroded, the concrete usually remains intact with little evidence of cracking or delamination.

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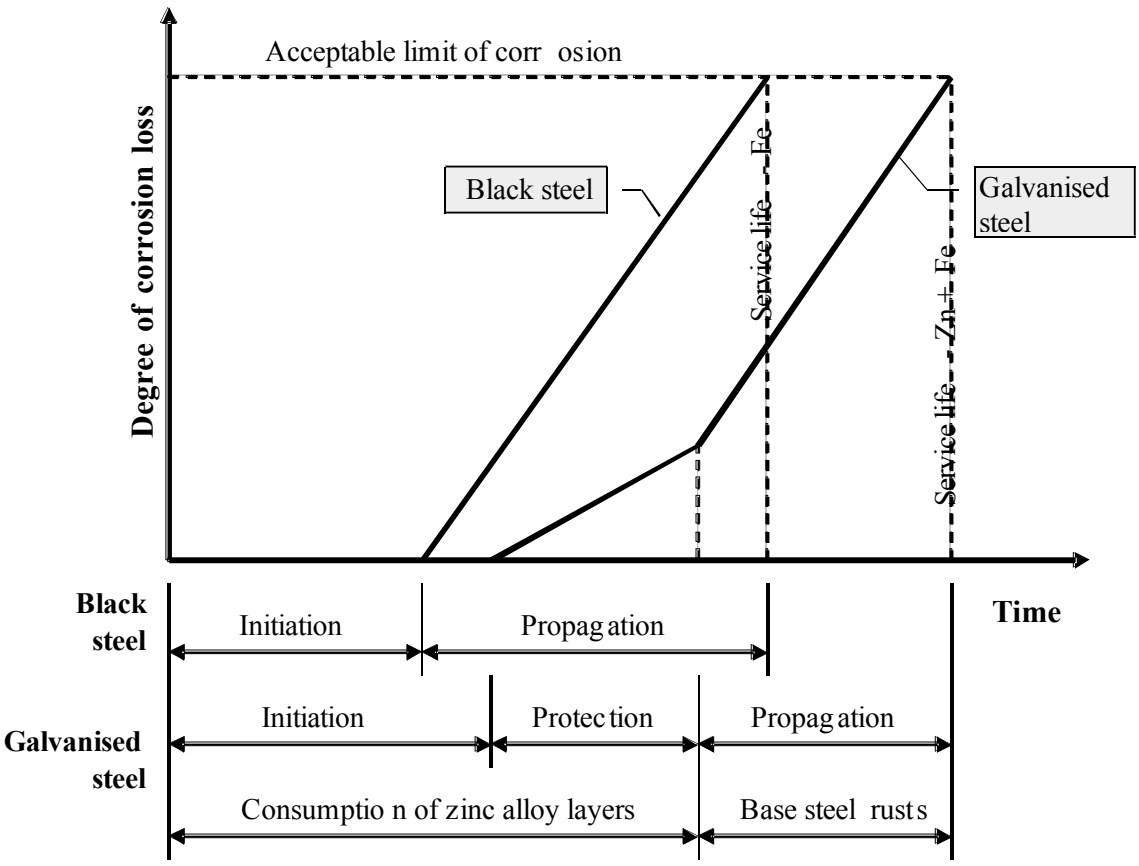

Fig. 5.11-20: A conceptual model for the corrosion behaviour of normal 'black/carbon' steel reinforcement and galvanised reinforcement in concrete. [after Bamforth (2004)].

The zinc corrosion products are thought to be less voluminous than the iron corrosion products, so develop lower swelling pressures and smaller tensile stresses in the surrounding concrete. Yeomans (1994) has postulated that, even when the zinc coating in a local area has been completely dissolved and the underlying reinforcing steel is exposed, the steel in close proximity to the remaining zinc is cathodically protected by the sacrificial nature of zinc.

Similar considerations apply to sites along the reinforcing bars where the zinc layer has been damaged. Once the envelope protection of the zinc coating and the sacrificial cathodic protection of exposed steel by adjacent zinc have been exhausted, corrosion of the steel substrate will commence. However it is suggested that by the time this stage is reached, the concentration of chloride ions at the depth of the reinforcement will be far in excess of that required to initiate corrosion of the normal (black) steel. Accordingly intense corrosion will ensue.

Evidence from real structures about the performance of galvanised reinforcing bars has generally been favourable. Surveys of bridge decks and marine structures show satisfactory in-service performance of galvanised bars for periods of up to 24 years. The measured chloride concentrations were as high at 1-2% by weight of cement. This is well in excess of the generally accepted chloride threshold level for the initiation of corrosion on normal steel bars. In spite of this there was no evidence of corrosion of the underlying steel bars and the surrounding concrete was sound. In most of the cases studied the reinforcing steel had adequate cover (greater than 40 mm) of good-quality concrete (water-cement ratio of 0.5 or lower).

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Resistance to carbonation: There is wide consensus that galvanised steel provides effective protection against carbonation-induced corrosion. This behaviour is illustrated by Fig. 5.11-21 which shows that in carbonated concrete the corrosion rate of galvanised steel is significantly lower than that of non-galvanised steel, but is somewhat higher than that experienced in more alkaline uncarbonated concrete. Thus the zinc coating remains passive in completely carbonated concrete. An important characteristic of zinc corrosion in carbonated concrete is the uniformity of the attack on the galvanising coating and the very low corrosion rate which results. These aspects make galvanised steel reinforcement durable in carbonated concrete.

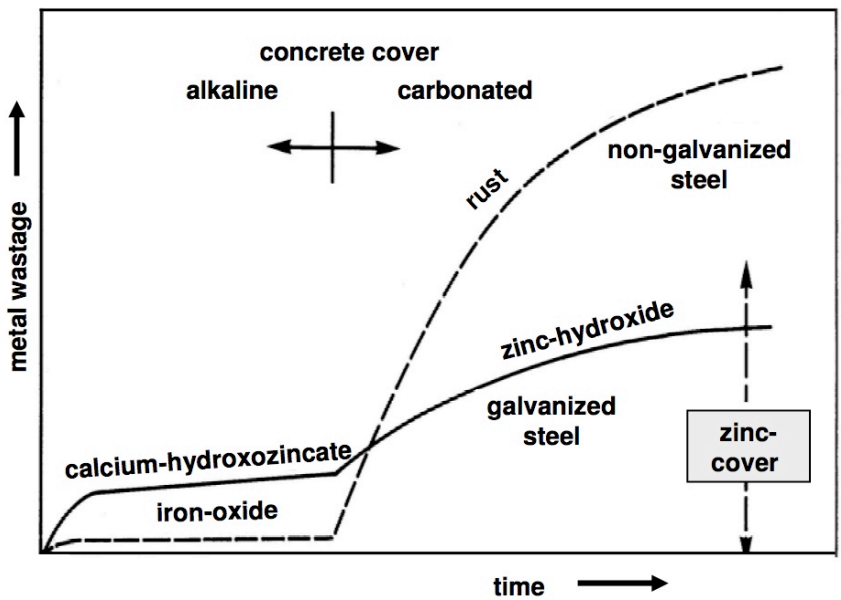

Fig. 5.11-21: Corrosion performance of ungalvanised and galvanised reinforcing steel in concrete [fib Bulletin 49 (2009)]

Other influences on durability: Alkalinity of the concrete pore fluid may pose problems in terms of durability, owing to the reaction of calcium hydroxide with the zinc. In normal concrete, which has a pH of 12.4–12.5, the corrosion rate of the zinc coating is minimal. However, relatively small changes in the pH of the pore fluid either side of this 'window' can result in significant dissolution of the zinc coating. As galvanised steel only receives protection as long as the zinc coating remains intact, this can have a dramatic influence on corrosion resistance.

In fresh concrete the zinc coating dissolves in the highly alkaline pore water, which results in a uniform loss of zinc thickness of about 5 to 10 µm. A layer of calcium hydroxyzincate is formed passivating the zinc surface, which inhibits further corrosion and reduces zinc loss to a very low rate. The time during the setting process to reach passive conditions depends on the alkalinity of the cement. In concrete made with cement enriched with soluble alkali, the formation of the calcium hydroxyzincate film is temporarily inhibited during the setting period and the active corrosion of zinc, with the evolution of hydrogen, persists for longer. In hardened concrete, the corrosion rates of galvanised reinforcing bars tend to equalise irrespective of the cement used with the value depending upon the concrete parameters and environmental conditions (e.g. water-cement ratio and humidity).

It is reported by that if the pH of the fresh or hardened concrete is particularly high, the zinc corrosion products appears as isolated crystals which do not seal the surface of the zinc.

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Consequently the zinc is not sufficiently passivated and zinc dissolution continues at a high rate. The corrosion rate can remain high enough to lead to the total dissolution of a galvanised coating in a few years. At a pH of 13.2 the zinc coating is in an active state and quickly dissolves, such that the zinc coating may totally disappear in a short period of time.

The relation between alkali content and corrosion rate might help to explain the controversy regarding the durability and performance of galvanised reinforcement. Most North American cements have low alkali contents, which could explain why the use of galvanised reinforcement has been most successful there.

A way of reducing the alkalinity of the concrete is to add silica fume to the mix, which makes it more compatible with the use of galvanised reinforcement. Silica fume not only lowers the alkalinity of the mix but it also improves the quality of the concrete, as has been discussed previously.

5.11.5.5 Stainless steel clad carbon steel reinforcement

Stainless steel clad carbon steel reinforcement has been developed over recent years. The outer stainless steel layer provides the required corrosion resistance and the inner carbon steel core gives the reinforcing bar it physical and mechanical properties. The outer stainless steel layer can be made from different grades of stainless steel.

The composite nature of this product can be seen in the cross-section of the reinforcing bar as the outer skin of stainless steel appears brighter than the carbon steel core. The cladding itself is of uniform thickness varying with an annular dimension of between 0.5 – 1.0 mm. Core composition and finishing rolling temperatures can be adjusted to achieve the tensile properties required. The stress strain curve for these bars is understood to be essentially the same as an equivalent carbon steel reinforcing bar.

There are a number of points to note with this type of reinforcement including that:

- It is produced in short lengths, usually of 6 m and by special order 11.6 m.
- It cannot be produced in coils, which would have simplified on site automatic production of prescribed lengths and bending of stirrups and links.
- The cut ends expose the carbon steel core. The producer recommends that bar ends are covered with a stainless steel cap. For this reason the bars need to be cut by a means that does not deform the ends, which means that the scissor type cutting usually employed cannot be used. Potentially this could be a major practical drawback for this technology;
- The largest diameter of reinforcing bar currently available is 20 mm.

5.11.5.6 Low carbon, chromium alloyed steel reinforcing bars

Micro-composite multi-structural formable (MMFX) steel is manufactured to a patented chemical composition and proprietary steel microstructure that is formed by controlled rolling and cooling of the steel during the production process. MMFX steel has a low carbon content (< 0.15%) and contains around 8 to 10% chromium. It is suggested that these characteristics should minimise corrosion initiation by reducing the formation of micro-galvanic cells in the steel structure.

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It appears that more information is required about the characteristics and mechanical properties of MMFX steel reinforcement. It is suggested that grade 60 MMFX exceeds the strength requirements of the ASTM A 615 standard by about a factor 2. MMFX is also reported to be available in strengths suitable for prestressing operations.

The threshold value for initiation of chloride-induced corrosion is claimed to be a factor of nine times greater than that of carbon reinforcing steel. However, comparison of the published data with European experience suggests that initiation of chloride-induced corrosion threshold value would only be about 2 to 4 times higher than that of normal carbon reinforcing steel (taking the critical threshold to be a chloride content of 0.05-0.10% total by weight of concrete). These comparisons also suggest that once corrosion has been initiated MMFX steel experiences a corrosion rate about 3 to 5 less than that of normal carbon reinforcing steel.

Marketed under the trade name MMFX 2, the material has been used in North America for structures such as bridge decks and marine/coastal structures.

Various corrosion research studies into the performance of MMFX have been undertaken in North America involving both accelerated and long-term tests in chloride exposed concrete [Hartt et al. (2007), Yu et al. (2007), Jianxin et al. (2005), Clemeña and Virmani (2004)]. This work has suggested that the critical chloride threshold to initiate corrosion is between 1.8 and 6 times those for carbon reinforcing steel.

With regard to corrosion rate, Jianxin et al. (2005) have indicated that MMFX steel exhibits higher corrosion resistance than normal carbon steel, with corrosion losses being between 16% and 66% those of normal carbon reinforcing steel. Mancio et al. (2007) report that the corrosion rate of MMFX dowel bars in jointed concrete sections was only about 3% of that of carbon reinforcing steel.

Zia (2003) suggests that MMFX reinforcing steel might potentially provide an economical alternative to conventional carbon reinforcing steel whilst providing greater corrosion resistance. He suggests that long-term field study of actual performance is needed, so that the corrosion performance of MMFX in actual structures can be ascertained. Such studies are currently being undertaken in North America [Kahl (2007)].

In addition to improving the corrosion resistance, the microstructure of MMFX steel produces an increase in the yield strength of the steel. The physical properties of low carbon steel (but not its corrosion resistance) are covered by ASTM A 1035, *Standard specification for deformed and plain, low-carbon, chromium steel bars for concrete reinforcement* [ASTM (2007)]. This specifies two minimum yield strengths, namely 690 MPa and 830 MPa. The ASTM Standard notes that these values are in excess of the maximum design strengths for unstressed reinforcement typically found in Codes.

It has been suggested that the cost of MMFX 2 reinforcement will be in the order of 1.5 to 2 times that of conventional carbon steel reinforcement. In some cases, a proportion of this increase will be off-set by the higher strength of the MMFX reinforcement.

5.11.6 Avoidance approach (Design-out approach)

5.11.6.1 Introduction

Other ways of avoiding reinforcement corrosion is to remove (design-out) those elements which experience in-service deterioration or to take action to suppress the electro-chemical reactions associated with the corrosion process. Practical measures for doing this include re-

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placing the carbon steel reinforcement with an austenitic stainless steel or a non-metallic reinforcement, or by introducing a cathodic prevention system.

Use of austenitic stainless steel of the appropriate grade (type) is an effective means of avoiding corrosion. Such stainless steels can tolerate very high chloride levels potentially considerably higher than those which are likely to occur within a concrete structure. One benefit of using stainless steel is that existing structural design procedures can be used with only limited adaptation.

Alternatively consideration can be given to the use of various types of non-metallic reinforcement instead of normal carbon steel. In such circumstances the main durability concern would be whether such materials would experience progressive degradation in the moist highly alkaline environment of the concrete. However, existing structural design procedures would need to be adapted to take account of the different mechanical (and other) properties of the non-metallic reinforcement and also the way they interact with the concrete (e.g. bond). Changes in construction practices may also be needed.

Cathodic prevention is an electro-chemical technique whose principle of operation is identical to the impressed current cathodic protection. However, it is installed at the time of construction and applies a much lower current density. The system may be applied throughout the life of the structure or left dormant until required. Protection is achieved by enhancing the alkalinity at the steel-concrete interface and by preventing the chloride ions from reaching the negatively charged reinforcing bars. Thus the system can be used in a practical way to prevent corrosion initiation.

5.11.6.2 Stainless steel reinforcing bars

Stainless steel is not a specific grade of steel but is a family of steels with a wide variety of physical and mechanical properties and resistance to corrosive environments. The term stainless steel is used for a group of steels which have a minimum 12% chromium content.

Stainless steels are produced by the addition of alloying elements to iron during manufacture which, in addition to chromium, include nickel, manganese, molybdenum and titanium. The alloying elements affect the microstructure of the steel, which in turn affects its mechanical properties and corrosion resistance. Corrosion resistance and other useful properties are enhanced by increasing the nitrogen, chromium, nickel, and molybdenum contents, and by decreasing the carbon content. The level of carbon in the steel is controlled during the manufacture process.

The presence of chromium results in a thin layer of stable oxide forming on the surface of the stainless steel. The oxide layer is passive and is highly resistant to atmospheric corrosion. It has the property of being self-healing in an oxidising environment, accordingly surface damage such as cracks and defects are repaired and do not effect the corrosion resistance. Stainless steels remain passive in concrete at lower pH levels and high chloride concentrations, with the extent of tolerance of these environments being dependant upon the grade (composition) of the stainless steel used.

There are four main groups of stainless steel, known as martensitic, ferritic, austenitic and duplex. A number of different grades (types) are defined within each range of stainless steel. Only the austenitic and duplex ranges are sufficiently resistant to chloride attack to be regarded as effective options for *avoiding* the corrosion processes associated with steel reinforcement embedded in concrete. Table 5.11-3 illustrates their characteristic compositions. Duplex steels are characterised by a high content of chromium and low nickel content.

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Table 5.11-3: Some stainless steel types and their characteristic compositions

Steel type : microstructure	% chromium	% nickel	% molybdenum
Austenitic	18-26	8-21	2-4
Austenitic-ferritic (duplex)	21-28	4-6	1.5-6

Stainless steels are subject to four types of corrosion related attack, namely general corrosion, intergranular corrosion, pitting corrosion and stress corrosion cracking. General corrosion takes place only if the medium is sufficiently acid. The lower the pH value and the higher the temperature, the more difficult it is to achieve passivation of the metal. Intergranular corrosion occurs as a result of changes caused by welding, but it is understood that all weldable stainless steels are now specially alloyed to avoid this problem.

Stress corrosion cracking requires a number of factors involving parameters such as material, environment and stress level. As the tendency to stress corrosion typically increases with increasing chloride content and temperature and decreasing pH, this form of attack is unlikely to be a problem for stainless steel embedded in concrete. It is more likely to occur under atmospheric conditions, so could occur under the appropriate conditions in ancillary items and fittings employed on a concrete structure. Stress corrosion cracking is more likely to occur in welded stainless steel elements, when these are situated in carbonated concrete with very high chloride content and exposed to high temperature.

The corrosion resistance required for the use of stainless steel reinforcing bars is primarily in respect of resistance to localised corrosion (i.e. pitting and crevice corrosion) in a chloride environment. This resistance is indicated by the pitting resistance equivalent number (PREN), which allows the corrosion resistance of different formulations of stainless steel to be compared. The PREN value is based upon the stainless steel's content of alloying elements chromium (Cr), molybdenum (Mo) and nickel (Ni), and is calculated using various formulae, of which the following is perhaps the most popular:

PREN = % chromium + 3.3 ·x % molybdenum + 16 x % nickel.

Examples of commercially available grades of stainless steel which are used in concrete and their PREN value are given in Table 5.11-4. Thus the higher the PREN value, the greater is the corrosion resistance of the grade of stainless steel.

Table 5.11-4: Some stainless steels used as reinforcement [fib Bulletin 49 (2009)]

Steel Grade	Composition	Steel type : microstructure	PREN
1.4301	X5CrNi 18 – 10	Austenitic	18
1.4541	X6CrNiTi 18 – 10	Austenitic	18
1.4401	X5CrNi Mo 17 – 12 – 2	Austenitic	24
1.4571	X6CrNiMoTi 17 – 12 – 2	Austenitic	24
1.4462	X2CrNiMoN 22 – 5 – 3	Austenitic-ferritic (duplex)	34
1.4539	X1NiCrMoCu 25 – 20 – 5	Austenitic	38
1.4571	X1NiCrMoCuN 25 – 20 – 7	Super austenitic	42

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Austenitic stainless steels: These have a good resistance to general corrosion, pitting corrosion and crevice corrosion, but are sensitive to stress corrosion cracking when the nickel content is in the order of 10%. Their resistance to pitting corrosion, crevice corrosion and stress corrosion is improved by higher contents of chromium, nickel and molybdenum. Austenitic steels are not ferromagnetic. Their strength can be increased by cold-forming, without incurring an unacceptable reduction in their deformability. However, cold working of austenitic stainless steels may reduce their corrosion resistance to some degree.

The ductility of austenitic stainless steel reinforcing bars is greater than that of conventional reinforcing bars. In addition, they have high toughness and good ductility properties at low temperatures and, as a result, austenitic steels are commonly used in seismic resistant structures for their energy absorption characteristics.

Austenitic stainless steels retain their strength at elevated temperatures better than normal carbon steels and duplex stainless steel. At 500°C there is negligible reduction in the 0.2% proof stress values. This suggests that the performance of austenitic stainless steel reinforcing bars will behave better under fire conditions than normal carbon steel reinforcing bars.

Summarising austenitic stainless steels have the following properties:

- good general corrosion resistance;
- strength characteristics that can be improved by hot or cold working;
- good toughness and ductility properties at low temperatures;
- low magnetic properties.

Duplex (austenitic/ferritic) stainless steels: These combine the good properties of ferritic steels (high yield strength) and austenitic steels (good ductility and improved corrosion resistance), thereby providing a satisfactory combination of properties for use as reinforcing bars. Summarising duplex stainless steels have the following properties:

- a corrosion resistance that is generally greater than that of most austenitic steels, which means that in most cases effectively they will not corrode;
- they have superior mechanical properties;
- they are magnetic.

Characteristics and mechanical properties: Indicative characteristics and mechanical properties of austenitic and duplex stainless steels are given in Table 5.11-5.

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Table 5.11-5: Indicative characteristics and mechanical properties for austenitic and duplex stainless steels [fib Bulletin 49 (2009) and BS EN 10088-Part 1 (1995)].

Property	**Units**	**Type of stainless steel**	
		Austenitic [5]	**Duplex** [5]
Proof stress (0.2%) [3] – ribbed bars	MPa[1]	500-650	500-650
Proof stress (0.2%) [3] – plain bars	MPa[1]	250	NA[4]
Modulus of elasticity at 20°C	GPa[2]	200	200
Coefficient of thermal expansion	$.°C^{-1}$	17×10^{-6}	13×10^{-6}
Density	kg/m^3	7900-8000	7800
Thermal conductivity	W/m.°C	12-15	20
Magnetic	None	No	Yes
Specific heat	J/g.°C	0.44	0.44

Notes: [1] MPa = MegaPascall = 1 N/mm^2.

[2] GPa = GigaPascall = 1,000 N/mm^2 = 1 kN/mm^2.

[3] The proof stress (0.2%) is taken to be the characteristic strength, the value of which depends upon the diameter of the reinforcing bars employed. The bars are divided into groups 3 – 5 mm diameter, 6 – 16 mm diameter and 20 – 40 mm diameter. Typically the proof stress (0.2%) / characteristic strength is lower for larger diameter reinforcing bars.

[4] NA = reinforcing bars of this type are probably not available in this material.

[5] The characteristics and mechanical properties for austenitic and duplex stainless steels depend upon the grade and source of supply – specific information should be obtained from the manufacturer for detailed design purposes.

Stainless steels differ from carbon steels as they do not have a well defined tensile yield point. For design with stainless steel, proof strengths are used in place of yield strengths and are determined as the stress at which plastic extension equals a specified strain, usually 0.2%. *fib* Bulletin 49 (2009) offers guidance on the specification of the mechanical properties of stainless steels in different countries (UK, Germany, USA, Denmark, Italy, and France). It also mentions the draft European standard for corrosion resistant (stainless) reinforcing steels which defines strength grades with 0.2% proof stress values of 235, 500, 650 and 800 MPa.

All stainless steels can be welded, either to materials of the same type or to carbon steel, provided that the necessary precautions are taken. If cold working has been used to improve the mechanical properties of the stainless steel, care needs to be taken to control the heat input during welding otherwise there may be a reduction in the strength of the parent material in the zone of the weld. Welding of stainless steel forms surface oxide layers which have an adverse effect upon its corrosion resistance. Welding can reduce the electrical pitting potential (i.e. it becomes more negative) of some stainless steels relative to that of the parent (not welded) material, indicating that in environments containing chlorides the corrosion resistance of the welded material is reduced.

In order that the full corrosion resistance of the stainless steel is preserved, the surface of the bars must be free from mill scale or oxides. The presence of scale and oxides on the sur-

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face of stainless steel reinforcement creates a risk of galvanic corrosion, as do particles of 'foreign' rust, swarf or iron deposits from tools used in handling or during the working of the stainless steel. These particles may act as local cathodes which impairs the corrosion performance of the stainless steel. It is normal practice for hot-rolled and warm-deformed stainless steels to be delivered in a de-scaled condition.

Resistance to chloride ingress: Research and experience in use demonstrate conclusively that austenitic stainless steel is an effective measure for avoiding reinforcement corrosion associated with chloride ingress. Austenitic stainless steels containing molybdenum (type 1.4436) are recommended for use in high chloride environments, where the risk of reinforcement pitting corrosion in unprotected concrete can be high. In environments where the chloride exposure is less severe, it may be possible to use stainless steel type 1.4301. Although this grade of stainless steel is less corrosion-resistant, it is less expensive and is more economical to use.

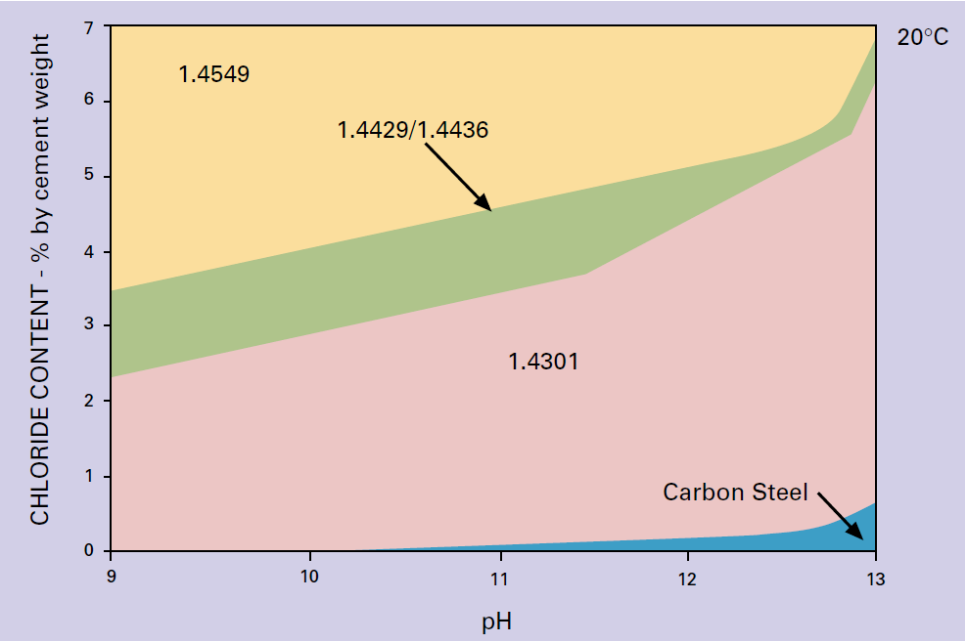

Fig. 5.11-22: Passivity of carbon and stainless steel reinforcement grades in chloride contaminated concrete [UK CARES: Guide to Reinforcing Steels: Part 7 (undated)] Reproduced with permission.

Long-term natural exposure testing carried out in the Arabian Gulf region has also demonstrated the effectiveness of stainless steel type 1.4436 [Summers and Olsen (1996)]. The Arabian Gulf region is characterised by very aggressive conditions involving high ambient and ground salinity, in conjunction with high temperature and humidity. The stainless steel reinforcing showed no significant corrosion after 12 years of exposure in low-quality concrete (220 kg/m^3 cement) that was poorly cured and contained 3.2% chloride (by weight of cement) mixed in at the time of casting. This performance is greatly superior to that of conventional carbon steel reinforcement, which in these conditions has been known to start corroding in only 2 years, even with high-quality concrete.

Stainless steels remain passive in concrete at low pH levels and high chloride concentrations. Fig. 5.11-22 illustrates the application of different grades of stainless steel reinforcement in a chloride contaminated concrete.

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Resistance to carbonation attack: Research and practical experience has shown that austenitic stainless steels remain essentially passive in carbonated concrete, and hence do not corrode to any significant extent.

Combination of stainless steel with carbon steel: The selective use of stainless steel reinforcement in sections of a structure exposed to severe environment conditions (e.g. external locations experiencing carbonation or exposed to chlorides) with carbon steel reinforcement used in sections in a benign environment (e.g. interior locations) can provide a cost effective approach to the design of durable concrete structures.

When both metals are in a passive state in concrete (i.e. not corroding) their electrical potentials will be similar, which means that no significant corrosion current will pass between them. Thus galvanic coupling of the stainless steel with carbon steel in such conditions does not produce significant effects.

In circumstances where the carbon steel in concrete is corroding (i.e. in an active state) and the stainless steel is not (i.e. in a passive state), galvanic coupling will give rise to some degree of accelerated corrosion of the carbon steel.

However, this situation is typically less problematic than the situation where active carbon steel in concrete (i.e. corroding) is in electrical contact with passive carbon steel (i.e. not corroding). This creates a macrocell in which corrosion currents can be very large leading to rapid rates of corrosion. Such circumstances occur where an area of corroding reinforcement is surrounded by, and is in electrical contact with, passive reinforcement, which is common.

Macrocell corrosion can also occur in various other circumstances, such as in bridge deck slabs where the top surface is exposed to chlorides from de-icing salts or in the walls of concrete storage tanks holding liquors containing aggressive species such as chlorides. In these situations one face of the concrete element is effectively saturated and the embedded reinforcement acts as the anode of the corrosion macrocell. The other face of the concrete element is atmospherically exposed, relatively dry and this reinforcement located in well oxygenated concrete and therefore acts as the cathode of the corrosion macrocell.

In these circumstances where the anodic carbon steel reinforcement is replaced by stainless steel reinforcement, the macrocell corrosion current is likely to be smaller by an order of magnitude or more [Knudsen et al. (1998), *fib* Bulletin 49 (2009)]. This means that the increase in corrosion rate due to the galvanic coupling of carbon steel with stainless steel is significantly smaller than that associated with coupling to passive carbon steel. This phenomenon is explained by Nürnberger (1996) and in *fib* Bulletin 49 (2009) as arising from the fact that stainless steels act as poor cathodes, which limits the corrosion current produced. Thus stainless steel reinforcement can generally be coupled with carbon steel reinforcement without problems, assuming that the surface of the stainless steel is free of mill scale and oxide deposits, and overall this is less of a risk than coupling with areas of passive carbon steel.

The guidance issued by the UK Highways Agency entitled "*The use of stainless steel reinforcement in highway structures"* [Highways Agency BA 84 (2002)] also indicates that in normal use and combinations, the risk of galvanic (bi-metallic) corrosion is small and likely to be lower than that of a combination of different carbon steels.

However, if the surface of the stainless steel is contaminated with mill scale or oxide deposits from welding, corrosion currents can increase by an order of magnitude or even more, which are likely to be higher than those associated with coupling with areas of passive carbon steel [*fib* Bulletin 49 (2009)].

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Reduction in concrete cover depth: With the greater corrosion resistance of stainless steel it may be possible to reduce the concrete cover to reinforcing bars. In very aggressive environments this should only be considered where a good quality concrete will be produced, especially in the cover zone. There may be considerations which need to be taken into account in addition to corrosion resistance, such as performance in fire. The UK Concrete Society's Technical Report 51, "*Guidance on the use of stainless steel reinforcement*" [Concrete Society (1998)] recommends that the minimum cover should still be at least 40 mm. The option to reduce cover may allow the use of smaller diameter bars or a reduction in element section size. A reduction in the depth of cover could produce more effective crack control where sections are subject to tensile stress. In some cases a reduction in element section size might lead to a worthwhile saving in the weight of parts of the superstructure, possibly giving a reduction in the size of supporting elements or in foundation loading.

Cost issues: The cost of stainless steel has been a barrier to its use. Over recent years there have been changes in the production processes for stainless steel which have brought the purchase price (first cost) down from about eight to ten times the cost of carbon steel to perhaps nearer six times now. Use of an appropriate stainless steel greatly increases resistance to corrosion which, in practical terms, effectively eliminate the problem of reinforcement corrosion. Thus although stainless steels are more expensive than carbon steels, this is offset by longer term benefits and improved through-life performance of concrete structures in aggressive environments.

A comparative cost study was carried out for the Schaffhausen Bridge built over the Rhine in Switzerland between 1993 and 1995 [Concrete Society (1998)]. Stainless steel reinforcement was used in critical exposed parts of the structure to minimise future repair and disruption costs. These areas included the beams, which are exposed to water contaminated by de-icing salts, and the bottom 7.6 m of the pylons, that would be subject to splashing by the same. The stainless steel reinforcement was used in conjunction with carbon steel reinforcement. It was judged that there was little risk of significant galvanic (bi-metallic) corrosion of the carbon steel reinforcement as its surface area was large comparable with that of the stainless steel. In this case the selective use of stainless steel increased the initial cost by only 0.5% (i.e. the increase in the capital cost of the bridge). This action provided clear benefits in terms of a reduction in the estimated through-life costs for the bridge of some 13% (i.e. a saving).

Life-cycle costs were reported by [Knudsen et al. (1998)] for two Danish case studies:

A. the reinforced concrete columns supporting a series of bridges over a highway;

B. the splash zone of the reinforced concrete columns of a coastal bridge.

As before the studies showed that the initial cost (i.e. the capital cost) was higher for the use of stainless steel, but that the resulting through-life costs for the bridges were significantly less. Interestingly [Knudsen et al. (1998)] also examined the effect of changes in the discount rate upon the net present values calculated and the impact that this had upon the relative ranking of a number of different through-life management and repair options for the two bridges. At low discount rates the selective use of stainless steel reinforcement produced the lowest net present value for an assumed 50 year remaining life. For case study A, other shorter-term through-life management and repair options gave the lowest net present value when the discount rate increased above about 7%. For case study B, this transition occurred when the discount rate increased above about 5%. This effect of the use of a higher discount rate, which might be considered to be an anomalous outcome especially if consideration is given to the associated sustainability implications, is discussed in more detail in Section 5.3.

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Lean duplex: The high costs of the traditional stainless steels have led steel producers to develop new products with much reduced nickel content. These are the so-called 'low nickel duplex' or 'lean duplex'. They are reported to have sufficient nickel content to maintain high corrosion resistance and are apparently marketed as being equivalent to 1.4301 in terms of both corrosion resistance and strength. Their price is reported to be around 4–5 times that of carbon steel reinforcement.

5.11.6.3 Non-metallic reinforcement

Non-metallic fibre reinforced polymer (FRP) bars first became available as reinforcement in about the mid-1980's and their use has been increasing slowly and consistently since then. FRP materials form a family of materials which have a diverse range of properties and characteristics that can be altered and adjusted by suitable design at the manufacturing stage. FRP materials are used for both reinforcing and prestressing concrete structures. However, the current extent of their use still remains limited.

FRP reinforcement has been used in situations where the durability of normal carbon steel reinforcement would have been a concern and where their use has proved to be more cost-effective than stainless or coated steels. However, there have been other drivers for the use of FRP reinforcement. These include exploiting some of the other properties of FRP reinforcing bars, which include their high strength, low weight and electromagnetic neutrality (FRPs are non-magnetic and do not cause electromagnetic interference with magnetic systems). In addition, FRP reinforcement is easily cut by mechanical tools and has been used in temporary construction applications, such as to form 'soft-eye' zones in diaphragm walls of launching and reception pits for tunnel boring machines. These zones enable the tunnel boring machine to leave the launching pit or enter the reception pit by cutting through the diaphragm wall without damaging its shield or its cutting tools.

FRP reinforcement is a manufactured composite material consisting of fibres embedded in a polymer matrix. As a structural material, it has advantages over both the unreinforced polymer and the fibres used alone. FRP reinforcement utilises glass, aramid or carbon fibres. The fibre composites are designated as follows: GFRP for glass fibre reinforced polymer, CFRP for carbon fibre and AFRP for aramid fibre. For each fibre type, a range of properties is available.

Although many polymer types are suitable for forming the manufactured composite material, thermosetting resins are most widely used. These are substantially infusible and insoluble after being cured by chemical reaction at ambient or elevated temperatures. The following generic types are most widely used for FRP reinforcement:

- polyesters and vinyl esters, which have similar curing characteristics, and
- epoxies.

Variations in the properties of these materials can be achieved by modifying the formulation of the resin system.

In addition to the nature of the constituents, the fibre content and orientation and the manufacturing process also significantly influence the properties of the FRP reinforcement.

The most common FRP reinforcement is glass reinforced vinyl esters. This is manufactured by the pultrusion process where strands of continuous fibres (rovings) are impregnated with the resin and are then pulled through a heated die, which consolidates the material and partially cures the resin to produce a reinforcing bar. The pultrusion process is able to incor-

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porate a high proportion of fibres in the cross-section of the bar, ensuring that it has relatively high strength and stiffness. The process produces high-quality components with consistent reproducible properties. A textured surface is added to the reinforcing bar using a secondary process (e.g. overwinding with additional fibres or bonding a sand layer to the surface of the bar) in order to increase its bond to the concrete.

One significant limitation of the use of thermosetting resins in the manufacture of FRP reinforcement is that, once formed, the bars cannot be bent into the range of reinforcement shapes currently used by the concrete industry. Hence other manufacturing processes have to be employed to produce two- and three-dimensional composite grids, spiral reinforcement or closed shapes such as shear links. A further limitation is the limited number of suppliers of FRP reinforcement.

The properties and characteristics of FRP materials and FRP reinforcement are still unfamiliar to most practising engineers. Bamforth (2004) suggests that the currently limited extent of the use of FRP reinforcement may, in part, be attributed to the absence of agreed design codes and comprehensive design data, although this is gradually changing [*fib* Bulletin 40 (2007)]. The lack of standardisation of materials and test procedures are also likely to be contributing factors; as is the limited documentation of the durability of composites in the highly alkaline concrete environment.

The durability of FRP materials is generally established and tends to be taken for granted as these materials have been used in aggressive environments for more than fifty years. However, there is considerably less experience of their use in the highly alkaline and chemically complex environment of concrete. The durability issues for FRP materials used as prestressing tendons are potentially different to those of FRP reinforcement because, in principle, the prestressing tendons are not directly exposed to the alkaline environment of concrete.

The durability of embedded FRP reinforcement is a function of the resin and fibre types, the fibre- matrix interface and the manufacturing process. Some resins and certain types of glass are degraded in a highly alkaline environment. Other factors which can potentially influence the durability of FRP materials include moisture, chlorides, alkalinity, stress, temperature, ultraviolet radiation, carbonation and acids; as well as the condition (e.g. damage, cut ends etc) and the quality (e.g. surface finish, presence of voids, resin homogeneity, degree of resin cure etc) of the FRP material.

Assessment of the durability of FRP materials has been investigated predominantly by accelerated laboratory test regimes. The results of these studies have been extrapolated to provide estimates of the likely long-term performance of FRP materials on actual structures. The accelerated test regimes use harsher and more aggressive environments than will be encountered on actual structures in service. Changes in mechanical properties of FRP materials such as tensile strength, elastic modulus, interlaminar shear and bond strength are deemed to be the best durability indicators, although these results may be complemented by the results of a variety of other tests. In many instances the outcome of the research work to date appears to be unclear because of various difficulties, including insufficient data and a lack of agreement in the test results of different researchers. This often means that no consensus view yet exists. These difficulties appear to be compounded by commercial sensitivities which can constrain the release of technical information.

The following provides a brief summary of some influences upon the durability of FRP materials and the suggested courses of action [*fib* Bulletin 40 (2007)]:

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- ***Moisture:*** Absorption of water by the resin can disrupt the polymer chains and cause cracking of the polymer matrix leading to changes in the elastic modulus, strength, failure strain, toughness and fibre-matrix de-bonding. In moist service environment conditions above 40°C, it is suggested that the performance of the specific FRP material should be evaluated by laboratory testing using more aggressive conditions than those of the anticipated service environment.

- ***Chloride:*** There is insufficient test data to draw definite conclusions. Again it is suggested that the performance of the specific FRP material should be evaluated for the chloride environment taking into account the effects of moisture and alkali conditions.

- ***Alkali:*** The performance of FRP materials in an alkaline environment varies with resin and fibre type. FRP materials appear to deteriorate much faster during accelerated tests in an alkaline solution than in concrete. Thus FRP materials would be expected to perform better in service within concrete than indicated during accelerated tests performed using a strong alkaline solution:

  - GFRP rods can experience severe deterioration after exposure to alkaline solutions at high temperature, but they perform better (less degradation) in concrete.

  - AFRP rods affected less by alkali than GFRP rods, but a combination of high stress (75% of ultimate) and an alkali conditions may cause deterioration of AFRP rods. However, it seems unlikely that such a high stress level would be present under normal service conditions.

  - CFRP rods experience no significant alkali induced deterioration with an appropriate resin system.

  - Vinyl esters have better alkali resistance than polyester resins.

- ***Sustained stress:*** When subjected to sustained load FRP materials fail at a lower stress than given by the short-term ultimate test. This effect is termed stress-rupture, creep rupture or static fatigue. GFRP and AFRP can fail at a relatively low sustained stress, while CFRP is affected to a smaller degree. The stress-rupture failure mechanism is also affected by environmental conditions (wet/dry, presence of acids or alkalis). The values quoted by different researchers vary, but as an illustration Yamaguchi et al. (1997) quotes stress-rupture values of the following order: GFRP of 30%, AFRP of 47% and CFRP of 91% after 50 years. For the same period of time, Ando et al. (1997) found the critical stress-rupture values to be AFRP of 66% and CFRP of 79%. Conversely, Alwis and Burgoyne (2006) found the critical stress-rupture value for AFRP to be 45%.

- ***Ultraviolet radiation:*** For FRP materials embedded in concrete ultraviolet radiation poses no problem. FRP materials should be protected against sunlight whilst in storage. FRP materials applied externally need to be protected against sunlight by proprietary protection systems.

- ***Thermal actions:*** The literature suggests that temperatures over 60°C may cause significant problems for FRP materials, but the available information is not conclusive enough to draw specific conclusions or make recommendations.

- ***Carbonation:*** The reduction in concrete pH associated with the carbonation of concrete should increase the service life of FRP materials embedded in concrete.

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- ***Acid attack:*** There is little published information upon the effects of acid upon FRP materials, however under acidic conditions the deterioration of the concrete is likely to be of greater concern than the deterioration of the FRP materials.

A significant problem is also to establish what the relationship is between an accelerated laboratory test regime and the long-term performance of the material when used on an actual structure. In many cases it is not yet clear what this relationship is.

Agreement on the durability of FRP materials is being constrained by a number of issues including a lack of international agreement on test methods, variations in FRP production methods and material types [*fib* Bulletin 40 (2007)].

Thus although some resins and certain types of glass are degraded in a highly alkaline environment, such problems should be avoidable by careful selection of the appropriate materials [*fib* Bulletin 40 (2007)]. Furthermore as FRP technology is developing rapidly, the situation in respect of durability may change as new developments take place in resins, fibres, manufacturing techniques and use of FRP materials.

In terms of the practical application of FRP reinforcement recognition also needs to be given to issues such as that FRP reinforcing bars have limited strength when loaded transversely to the longitudinal orientation of the fibres. This is a consideration for the connection between main reinforcing bars and stirrups. Some types of FRP bars are also sensitive to impacts. For example, carbon fibre bars may fracture if hit by a falling object such as a hammer or a dropped vibrator. Also during casting of concrete it is recommended that the concrete workers do not walk on the FRP reinforcement and that temporary work bridges be provided. In addition, the very low weight of FRP reinforcing bars means that they must be anchored to the formwork to prevent them floating (due to their buoyancy) when they are surrounded by fresh concrete. Currently the precast industry appears to present the greatest potential for the use of FRP reinforcement.

A number of demonstration bridges have been built in North America, northern Europe and Japan. Initially FRP was used for the prestressing tendons in conjunction with conventional unstressed steel reinforcement, but later FRP was used for both the prestressing tendons and for the reinforcement as well. It is reported that a number of major bridges in North America have been reconstructed creating 'steel-free' bridge decks, in which concrete deck arches between the main girders, with ties provided to take the horizontal thrusts. Whilst tension reinforcement was not required in this application, some nominal 'crack-control' reinforcement was provided by the inclusion of FRP bars.

FRP reinforcement has also been used in a number of different marine and coastal applications where the durability of normal carbon steel reinforcement would have been a concern. Reported applications include the reconstruction of a wharf in Nova Scotia [Newhook et al. (2001)], replacement fender support beams for a jetty in Qatar [Grostad et al. (undated)] and repairs to various marine structures including a sea wall in Florida and a dry dock at Pearl Harbour, Hawaii.

There are also various other applications where FRP reinforcement has been used in order to exploit some of the other properties of FRP reinforcing bars, which include their high strength, low weight and electromagnetic neutrality. Examples of such applications exist in Europe, North America, Japan and elsewhere; with those in the telecommunications and defence industries being of particular note [*fib* Bulletin 40 (2007)]. For example, GFRP reinforcing bars act as an insulator and do not conducting electricity. This has allowed them to be used in situations where stray electrical currents (such as from electrical transmission systems,

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electric railways, trams and metro systems, etc) would be a problem in carbon steel reinforcement.

5.11.6.4 Cathodic prevention/protection

Cathodic prevention is an electrochemical technique which is intended to provide corrosion resistance to new reinforced concrete structures. The technique is a variation on the well established method of cathodic protection applied to existing reinforced concrete structures. Cathodic prevention is installed during the construction phase.

In some instances a cathodic protection system may be installed upon a new structure in order that the system will be available for use at a future time when corrosion of carbon steel reinforcement would otherwise be occurring.

There are two methods of cathodically protecting reinforcing steel: impressed current and sacrificial anode. If the reinforcing steel is connected directly to an external galvanic or sacrificial anode, formed from a material such as zinc which is more reactive than steel, the embedded steel can be protected from corrosion. No external power supply is required. Protection of the embedded steel is achieved by the preferential corrosion of the external anode.

The impressed current method requires an external power supply and the provision of an inert anode, which acts as the cathode of the electrical system, to protect the embedded carbon steel. This approach is conventionally used for atmospherically exposed reinforced concrete structures, and is the method used for cathodic prevention.

The use of cathodic prevention was pioneered by Autostrada on motorway bridges in the mountainous Dolomites region of Northern Italy [Pedeferri (1992)]. The bridges were post-tensioned segmental construction. Previous experience had shown that it had proved to be very difficult to prevent chloride ingress into concrete. Cathodic prevention anodes were built into the deck structure or applied as part of the construction process. Cathodic prevention was applied at a very low level (of applied potential) to avoid the possibility of hydrogen evolution at the reinforcement. As chlorides had not penetrated the concrete when the cathodic prevention system was energised, the goal was that the small applied potential would polarise the reinforcement so that it would effectively repelled any chloride ions which did penetrate the concrete. The systems were installed in the late 1980's and early 1990's and it was anticipated that they would have a service of many years, as the anodes were to be used at a very low rating; assuming and system components were properly maintained. So far the use of cathodic prevention on newly built projects has been limited.

In terms of its principle of operation the cathodic prevention technique is very similar to other electrochemical repair systems such as realkalisation and desalination (electrochemical chloride extraction), except that the system is applied throughout the life of the structure using much lower current densities, typically less than 2 mA/m^2. The low current densities employed eliminated the risk of hydrogen embrittlement associated with cathodic protection and other electro-chemical techniques.

Experience has shown that the electrical connection leads get damaged or are not in the correct locations for future use. It is recommended that when cathodic prevention schemes are to be installed from new, then the system should be fully commissioned and energised before handover to ensure that the system is fully operational and appropriately documented.

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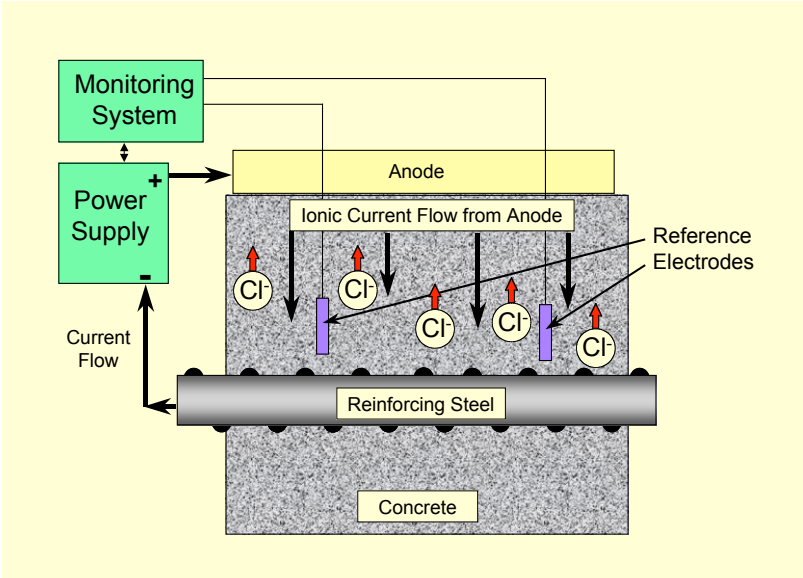

Fig. 5.11-23: Schematic illustration of the principle of operation of cathodic prevention. [Bamforth (2004)] Reproduced with permission.

Fig. 5.11-23 shows a schematic illustration of the principle of operation of a cathodic prevention system. The corrosion process at the reinforcing steel is prevented by passing a small direct current from an anode to the embedded reinforcement. The applied current is sufficient to cause the cathodic reaction to be the only one to occur at the steel. In time the current density may have to be increased if the environment becomes progressively more conducive to corrosion, or if there is current leakage. In some cases the resistivity of the concrete may be such that the system reaches its maximum voltage. In these instances corrosion is not generally a problem as it is controlled by the cathodic reaction and the resistivity of the concrete.

Certain siliceous aggregates are susceptible to reaction with alkalis in the concrete pore fluid, forming silica gel. The silica gel absorbs water, expands and ultimately leads to characteristic map cracking of the concrete. Polarising the reinforcement notionally creates conditions favourable for alkali–silica reactions. However, no electrochemically induced cases of alkali–silica reaction are reported to have been found in practice.

5.11.7 Overview of measures to enhance resistance or avoid reinforcement corrosion

There are a number of measures that can be used to enhance resistance or avoid reinforcement corrosion, thereby seeking to increase the service life of a concrete element in a given environment. Table 5.11-6 briefly summarises the potential durability benefits arising from the use of these measures.

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Table 5.11-6: Some measures to enhance resistance or avoid reinforcement corrosion

Technique	Potential beneficial effects on durability
High quality concrete	• Reduced permeability reduces rate of chloride and CO_2 ingress. High cement content provides pH buffer against carbonation.
Use of pfa (Needs to be well cured) (Ref: Section 5.11.2.2)	• Substantially reduced chloride diffusion coefficient in the long term (initial rate of chloride ingress may be comparatively rapid). May reduce chloride threshold level. • May lead to increased rate of carbonation when compared with PC concrete based on equal w/c ratio. Insignificant difference when comparison based on equal strength grade.
Use of ggbs (Needs to be well cured) (Ref: Section 5.11.2.3)	• Substantially reduced chloride diffusion coefficient in the long term (initial rate of chloride ingress may be comparatively rapid). • May lead to increased rate of carbonation when compared with PC concrete.
Silica fume (sf) (Needs to be well cured) (Ref: Section 5.11.2.4)	• At low w/c ratios substantially reduced chloride diffusion coefficient in the long term (initial rate of chloride ingress may be comparatively rapid). May reduce chloride corrosion threshold level. • No significant effect on the rate of carbonation for addition levels up to 10% when compared with PC concrete of equivalent grade. Higher carbonation rates at higher addition levels, as well as a reduction in pore-solution pH to below 12.5, when addition levels approach 30%.
Metakaolin (mk) (Needs to be well cured) (Ref: Section 5.11.2.5)	• Substantially reduced chloride diffusion coefficient. • Higher rate of carbonation than PC concrete with the same w/c ratio.
Ternary blends (Needs to be well cured) (Ref: Section 5.11.2.6)	• The combined use of Portland cement-pfa/fly ash-sf (triple powder blend) in concrete reduces its chloride permeability significantly, while increasing its compressive strength. Triple powder blend concretes reportedly outperform binary blend (Portland cement-pfa/fly ash/ggbs) concretes and have reduced sensitivity to the effects of inadequate curing.
Plasticising admixtures (Ref: Section 5.11.3.2)	• Reduction in w/c ratio reduces all environmental penetration including chloride ingress and carbonation.
Superplasticisers (Ref: Section 5.11.3.3)	• Benefits are as plasticisers, but more effective.
Integral waterproofers (Ref: Section 5.11.3.4)	• Can reduce capillary suction and hence chloride ingress. No significant effects on diffusion expected, but likely to result in reduced overall chloride concentration. Best used where hydrostatic pressures are low. No significant effect on carbonation.
Pore blockers (Ref: Section 5.11.3.5)	• Reduced build up of surface chloride. Reduced chloride penetration under pressure. No effect on chloride diffusion, but likely to result in reduced overall chloride concentration. No significant effect on carbonation.
Fibres (Ref: Sections 5.11.3.6-8)	• Lead to indirect improvements in durability through improved quality of surface zone concrete.

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Technique	Potential beneficial effects on durability
Surface barrier coatings (Ref: Section 5.11.4.2)	• Depending on the nature of the coating protection of new structures against chloride ingress and carbonation can be effective for long periods. Coatings must be applied correctly and regularly maintained. Coatings may increase the rate of chloride ingress into the concrete once the protective effect has been lost by maintaining the concrete in a relatively wet state. The apparent diffusion coefficient under these conditions is higher than in an equivalent dry concrete.
Surface impregnants (Ref: Section 5.11.4.3)	• Hydrophobic penetrants (silanes, siloxanes, siliconates, silicones and metal stearates) chemically modify the walls of the concrete pores/capillaries so they become water-repellent (hydrophobic) whilst continuing to allow the transmission of moisture vapour. • These treatments can provide concrete with additional resistance to chloride ingress on surfaces where no hydraulic gradient is present. This qualification can include ponding on horizontal surfaces and, in marine environments, may include vertical surfaces in the splash zone. • These treatments have no significant benefit in reducing the rate of carbonation.
Surface sealers/Pore blockers (Ref: Section 5.11.4.3)	• These products generate pore-blocking solids by reaction with the concrete. They can reduce the rate of carbonation although, typically, they are much less effective than an anti-carbonation coating.
Controlled permeability formwork (Ref: Section 5.11.4.4)	• Chloride diffusion coefficients reduced to between 40% and 70% of concrete cast using conventional formwork. • Increased resistance to carbonation due to improved quality at surface of the concrete.
Permanent formwork (Ref: Section 5.11.4.5)	• Permanent formwork can be used to provide a durable outer skin for the structure. Improved chloride and carbonation resistance (depending on nature of materials used) as structural concrete is sheltered from environment.
Sacrificial concrete layer (Ref: Section 5.11.4.6)	• Provision of additional depth of cover provides extra protection to reinforcement in environments containing chlorides. • Recommended as an additional protective measure to concrete in aggressive ground. The quality of the additional concrete should be equal to, or greater than that of the inner concrete.
Corrosion inhibitors (Admixed) (Ref: Section 5.11.5.2)	• No effect on chloride ingress but effectively raise chloride corrosion threshold level, increasing the time to the onset of corrosion and therefore increasing the durability/service life of reinforced concrete in chloride-rich environments. • No known effect on carbonation-induced corrosion.
Coated steel • Epoxy coated bars (Ref: Section 5.11.5.3) • Galvanised bars (Ref: Section 5.11.5.4)	• Where the coating remains intact epoxy coatings can provide good resistance to chloride induced corrosion. However, where the coating is scratched or damaged corrosion can occur; anodic activity can be intense where uncoated reinforcing bars are also present to act as the cathode. • Epoxy coatings act as an effective barrier to carbonation-induced corrosion.

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Technique	Potential beneficial effects on durability
	• Where chloride levels are not excessive (up to 1% by weight of cement) and where the pH is below 13.3 galvanising helps provide an effective barrier to chloride-induced corrosion. Also provides effective protection against carbonation-induced corrosion.
Stainless steel clad mild steel (Ref: Sections 5.11.5.5-6)	• Effective if of a high quality and of adequate and constant cladding thickness. Cheaper option than stainless steel, but in very severe conditions may suffer from perforation to the mild steel substrate. Some potential problems with capping of ends.
Stainless steel (Ref: Section 5.11.6.2)	• Highly effective means of preventing corrosion in most conditions and where high initial costs can be justified.
Non-ferrous reinforcement (Ref: Section 5.11.6.3)	• Not prone to the corrosion experienced with metallic bars. Some resins may not be durable in alkaline conditions.
Impressed current cathodic protection/prevention (Ref: Section 5.11.6.4)	• Prevents chloride-induced corrosion. Maintains alkalinity around reinforcement and the stability of the passive oxide film.
Sacrificial anodes (Ref: Section 5.11.6.4)	• As impressed current technique, but has a defined limit in terms of the length of service life.

Notes to Table 5.11-6:

PC = Portland cement concrete

w/c = Water-cement ratio or water-cementitious binder ratio

Chloride threshold level = amount of chloride contamination of concrete required to initiate corrosion of embedded reinforcement (refer Section 5.4.7.6).

5.12 Some measures to enhance resistance or avoid other forms of deterioration

The avoidance of deterioration is the approach most commonly used to preserve the integrity of the concrete itself. This approach aligns with service life design Strategy A – "*Avoiding deterioration by the design-out approach*", as described in Section 5.1.10. In reality Strategy A does not guarantee complete protection against deterioration. In most instances the Strategy A options seek to minimise the risk of deterioration occurring under a range of envisaged circumstances.

A basic principle for avoiding attack on the concrete by an external agent is to ensure that the concrete is dense and impermeable, so that the deleterious chemicals cannot penetrate the concrete. Clearly this requirement is particularly important for the surface zone of the concrete.

For example, the main compositional factors that determine the resistance of concrete to aggressive ground conditions are its water-cement ratio and the type of cement or combination used. Recent research [BRE Special Digest 1 (2005)] has shown that resistance to sulfate attack is not a function of cement content. Concretes made with the same materials and the

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same water-cement ratio, but different cement/combination contents, have similar sulfate resistance providing there is sufficient fine material to give a dense and impermeable microstructure. However, as there is not yet any agreed method for verifying that a concrete has a dense and impermeable structure, it is still common to recommend a minimum cement/combination content for a concrete. Whilst a compressive strength requirement has never formed part of many recommendations for sulfate resistance, it should be recognised that some specifications may include a compressive strength class requirement for structural and durability purposes.

There has been considerable recent research [BRE Special Digest 1 (2005)] which has focused on determining what constitutes an adequate concrete specification and performance of different cement types in aggressive ground conditions. This has shown the excellent performance of concrete incorporating ground granulated blastfurnace slag (ggbs) cements, which has been recognised allowing some relaxation of the requirements for concretes made with these cements in aggressive ground conditions. However, the work also revealed the mixed performance of concrete made with sulfate resisting Portland cement in sulfate conditions conducive to the thaumasite form of sulfate attack (TSA) – refer Section 5.4.5.2 – has led to tightening of requirements. In the case of concretes incorporating pfa and fly ash cements it was considered that there was insufficient information upon their performance in aggressive ground conditions and these remain under investigation.

Where options for the selection of the cementitious materials, mix additions and admixtures to be incorporated into the concrete may not be able to provide adequate resistance, additional protection measures may be employed. Possible approaches include:

- Use of controlled permeability formwork to enhance the quality of the concrete achieved at formed concrete surfaces – refer to Section 5.11.4.4.
- Provision of surface protection in the form of surface barrier coatings and water resisting barriers – refer to Section 5.11.4.2.
- Provision of a sacrificial layer of surface concrete.

The following examples illustrate other facets of the ‘design-out’ approach, with avoidance measures being described for a range of potential threats to the concrete:

- Air entrainment is adopted to avoid freeze-thaw cycling causing internal frost damage to concrete. This is a very well established and reliable mitigating measure, provided the intended air bubble distribution is achieved in the hardened concrete. There have been instances where the entrained air content has been adversely affected by excess pumping and vibration of the concrete during placement and compaction activities.
- Conversely it is suggested that surface scaling of concrete, which typically occurs on slabs and structural elements exposed to de-icing salts, can be prevented by increasing the strength of the concrete in the surface zone.
- Non-reactive aggregates are selected to avoid or minimise the risk of harmful alkali-aggregate reactions. This is a well established and reliable mitigating measure. If some uncertainty remains with a particular aggregate, the use of low-alkali cement and possibly also the addition of a pozzolan, such as pfa or a limited amount of silica fume, may further enhance resistance to the deleterious reactions. Alternatively concretes incorporating ggbs cement may be used.

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- Low C_3A cements are selected to avoid harmful sulfate attack. The addition of a pozzolan, such as pfa or a limited amount of silica fume, may enhance resistance to sulfate reactions. Alternatively concretes incorporating ggbs cement might be used.
- Surface coatings may be used to avoid disintegration of the concrete due to acid attack. However, such coatings need maintenance and a full reapplication of the coating at regular intervals, which is an issue needing detailed consideration during the initial design. Particular consideration would need to be given to the whole life cost implications of employing such a coating. Blended cements or pozzolanic additions may also contribute to an increased resistance against acid attack.
- Control of the peak temperature during hydration, together with the length of time that the concrete remains at or close to this temperature, to minimise the risk of expansion and cracking associated with delayed ettringite formation (DEF). Similar measures may be employed to reduce the risk or early age thermally induced cracking of the concrete.
- The risk of expansion and cracking associated with delayed ettringite formation (DEF) may also be reduced by the use of pfa, ggbs or low alkali sulfate resisting cement. Silica fume may also be effective at preventing DEF. However, limited research has been undertaken on this matter and it is possible that expansion may be delayed, rather than prevented, due to the reduced rate of moisture ingress resulting from the low permeability of concretes made with pfa, ggbs and silica fume.

5.13 Influence of some design, execution and workmanship issues upon durability

5.13.1 Introduction

High quality durable concrete structures can be created, but this is not always achieved because of problems which can arise with the concreting processes. In some unfortunate circumstances initial indications of durability problems, such as reinforcement corrosion, may in exceptional circumstances occur after perhaps as little as 10 years. To achieve greater certainty, owners and members of their professional team need to pay more attention to certain aspects of what is being created and exert more influence over the processes being employed.

Thus the execution stage constitutes the most vital step in achieving the required quality of concrete structures. From a durability point of view the essential part of the execution process is to ensure the provision of the correct concrete quality and thickness of the concrete cover, consistently in every location on the structure exposed to an aggressive service environment. Two key aspects to this are:

- Compaction, in order to ensure uniform, dense and strong cover concrete.
- The type and duration of curing, in order to control moisture levels and temperature differences thereby seeking to limit or avoid early age cracking.

This Section is concerned with examining the influence of some design, execution and workmanship issues upon durability. Sections 5.14 and 5.15 also contribute to this general topic. Section 5.14 is concerned with construction quality issues and the potential role that the project execution specification can play in achieving improved durability of concrete structures, especially ones with a long design service life. Section 5.15 is also concerned with ways

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of improving durability and looks into the benefits to be gained from effective pre-construction planning and trials undertaken in preparation for the construction phase.

Thus the implication is that deficiencies in the construction process need to be avoided. Essentially there is only one opportunity to "*get things right first time*", which is before and during the time the concrete for the structure is cast. Fig. 5.15-3 illustrates the challenges that may be faced and the potential "durability problems" that may be encountered and need to be overcome. It should also be recognised that deficiencies include situations where geometrical variations on cover to the reinforcement exceed the allowable geometrical tolerance values (i.e. the nominal cover value less the permitted tolerance – see Section 5.5.5).

EN 13670: *Execution of concrete structures* [CEN (expected 2009)] also gives guidance upon many aspects of these matters.

5.13.2 Influence of locally available concrete materials and labour

At the design stage it is important to bear in mind and adapt requirements to local conditions, recognising restrictions which may be imposed by the quality of locally available concrete materials and the expertise of local construction workers. It is therefore essential to adapt the requirements for the concrete mix, casting, compaction and curing procedures to what can realistically be achieved at the particular location. This needs to consider the available concrete components, local workmanship and prevailing climatic conditions. This is particularly important in remotely located structures and in geographic regions with few alternatives to the local cement, aggregates and water, and only local, maybe unskilled, labour.

One such situation might be hot, arid and saline environments with unavoidable high chloride content in the available fine aggregate and in the mixing water. The difficult curing conditions and the unavoidable high initial chloride content in the concrete could then warrant the application of stainless steel reinforcement selectively in the most critically exposed parts of the structure. Such an approach could 'at a stroke' minimise or eliminate the problem of reinforcement corrosion for a relatively modest initial cost increase; which usually will be marginal compared with the overall cost for construction and furnishing of the building. In the long-term this approach could provide considerable cost savings and would extend the service life of the structure many times compared to previous experience in such environments with conventional construction using standard carbon steel reinforcement. The service life in such cases would be determined by the durability of the concrete itself, or by the structure becoming functionally obsolete at some time in the future.

5.13.3 Dimensioning of structural elements

The concept of minimalist structures has developed into a challenging engineering and architectural discipline. By selecting combinations of structural form and materials, which minimise the total consumption of materials, very light structures can be achieved.

This full utilisation of the material strength throughout the structure assumes a priori that no deterioration takes place while the structure performs its long-term load carrying duty under the influence of the prevailing environment. However, if local weakening or deterioration should occur in a fully optimised structure, there are limited alternative load paths which enhance the resilience of overall structure and help maintain a satisfactory safety level. Such structures, without reserves or redundancies, are likely to have limited robustness. Accordingly they are likely to impose higher demands – and costs – for the inspection and maintenance of the structure [Rostam (1998)].

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The engineering attractiveness of optimising concrete structures continues to gain momentum, among others through the continuous development of ever stronger high strength/high performance concretes. These developments are valuable and attractive so long as the overall through-life performance of the structure in its service environment is foreseen and adequately taken into account during design and construction.

The designer can ease the execution phase, and thus increase the probability of achieving good quality concrete in the final structure, by using dimensions of the individual elements which facilitates the positioning of reinforcement and the placing and compacting of concrete. Tight dimensions, small element sizes and poor accessibility during casting are among the factors that lead to inadequate quality in the placed concrete cast into the final structure. This is often in spite of a high quality concrete mix being produced during batching. The durability issues commonly arise from problems during placing and compaction.

Abrupt changes in geometry should be avoided if possible. The local concrete quality along edges and at corners is usually lower than in the bulk of the concrete and along smooth formed faces. In addition, the concrete along edges and at external corners is exposed to the aggressive environment from two or three sides simultaneously. This is often reflected by the so-called 'corner effect', where cracks due to deterioration develop first along the exposed edges. Hence, edge zones and corners are more prone to premature deterioration than the other parts of the exposed concrete surface. Rounded edges and corners can help minimise deterioration of this type.

5.13.4 Reinforcement detailing and congestion

Reinforcement congestion can have an adverse effect upon the durability of a concrete structure where it prevents the concrete being placed and compacted effectively, especially in the concrete cover zone. Accordingly the reinforcement, whether it be conventional (non-prestressed) reinforcement or prestressed, should be adequately spaced and distributed within the cross-section of the concrete element so that such problems do not arise.

Where possible the reinforcement should be detailed to form a closed 3-dimensional reinforcing cage. Ideally the geometric form of links or stirrups should also be a closed loop which contributes to the 3-dimensional closed cage concept. In case of some unexpected deterioration taking place which would crack or split the concrete, such as bulk internal frost damage or alkali-aggregate reactions, then the 3- dimensional of the reinforcement cages would tend to hold the cracked concrete together. If the deterioration is associated with an internal expansive reaction, such as takes place with alkali-aggregate reactions, the reinforcement effectively becomes prestressed and tightly holds the cracked concrete together by virtue of the compressive stress induced in the damaged concrete. There are examples of concrete elements in which the concrete has been cracked into a myriad of pieces by alkali-aggregate reactions, so that it effectively reduces the concrete to a 3 dimensional "puzzle" of pieces, but which maintain a full structural load-carrying capacity because of the confining effect of the reinforcement cage. The high stresses which can be generated in the enclosing reinforcement cage are illustrated in one of the photographs presented in Fig. 5.4-4B, where the link reinforcing bars have been snapped by the expansion force created by a severe case of alkali-silica reaction (ASR).

Guidance on a standard method for detailing structural concrete is given in a manual for best practice produced by the UK Concrete Society in collaboration with the Institution of Structural Engineers [IStructE (2006)].

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5.13.5 Compaction of concrete

Adequate compaction of the concrete in the cover zone may be difficult to achieve where the reinforcement is congested. The movement of the concrete into the cover zone may cause "sieving" of the concrete if the spacing of the reinforcing bars is too small or the concrete is not fluid enough/too stiff. Section 5.15.2 discusses these issues and Fig. 5.15-1 illustrates one example where reinforcement is very congested, with further details being given in publications such as HETEK 3 & 5 (1997).

The compaction of concrete by vibrators needs to be achieved by vibrating the concrete inside the reinforcement cage. Experience has shown that vibrating directly in the cover concrete zone may lead to inferior compaction of the cover concrete. The problem requires particular attention in the case of dense high performance concretes with mineral admixtures because of the thixotropic properties of such low water-cement ratio concretes incorporating large quantities of plasticiser and superplasticiser admixtures.

The use of external form vibrators to compact the concrete, and in particular the cover concrete, is also a questionable procedure. The reason is that the entrapped air voids in the concrete tend to move towards the location of the vibrator and accumulate to form zones of porous concrete. With the correct use of poker vibrators such air bubbles are extracted from the concrete when slowly lifting the vibrator head out of the concrete.

5.13.6 Curing of concrete

Curing is the name given to the procedures used for the promoting the hydration of the cement and other binding agents introduced into the concrete mix to facilitate, amongst other parameters, the development of its strength and the reduction in its porosity and permeability (i.e. as the continuity of the capillary pores is reduced). This is achieved by the control of the temperature of the concrete and the movement of moisture into or out of the immature concrete. In the case of fresh concrete this may involve measures to minimise the risk of plastic shrinkage. The required actions may also involve protecting the concrete from freezing conditions or excessive heat generation, together with potentially harmful vibration, impact or other damage. Conventionally measures to minimise the risk of plastic settlement have been considered to be part of the concrete placing and compaction operations, rather than an aspect of the concrete curing activities.

In most instances the primary concern is with moisture escaping from the immature concrete, reducing the degree of hydration achieved, particularly of its surface zone. Typically the objective of curing is to keep the concrete saturated or as close to this state as is possible, until the originally water-filled space within cement paste has been filled to the extent desired by the products of the hydration of the cement and the associated binding agents introduced into the concrete mix. This might be achieved by ensuring that moisture losses, such as evaporation from exposed concrete surfaces, are kept low or that such surfaces are kept permanently wet. Neville (1995) notes that in the case of site concrete, active curing almost always stops long before the maximum degree of hydration has occurred.

Many years ago Powers (1947) showed that hydration proceeds at a maximum rate under conditions of saturation. He also established that the degree of hydration is greatly reduced when the relative humidity within the capillary pores drops below about 80%RH, which was also confirmed by Pattel et al. (1988). Thus for hydration to continue the relative humidity within the concrete has to be maintained above about 80%RH. Where the relative humidity of the external atmosphere is at this level, or higher, there should be little movement of moisture between the concrete and the external atmosphere. Accordingly, no active curing would no-

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tionally be required to ensure continued hydration. However, matters are never that simple. Moisture loss may be caused by other associated factors. For example, there are bound to be temperature differences between the concrete and the external atmosphere, which will no doubt experience fluctuations in temperature and relative humidity, as well as influences from wind and solar insolation.

Guidance upon curing of concrete is given in various publications, including that from the UK Concrete Society entitled '*Concrete Practice – Guidance on the practical aspects of concreting'* [Concrete Society – CS 164 (2008)]. Amongst other matters, this document gives guidance upon the use of curing membranes, the treatment of horizontal and vertical surfaces, as well as issues associated with exposed concrete and achieving uniformity of colour. This recognises that concrete maintained at a high temperature will hydrate more quickly and that the minimum curing period for the concrete depends on a number of factors including the:

- type of cement or combination of cementitious binders in the concrete mix;
- cement content and the water-cement or water-binder ratio;
- temperature of the surface layer;
- ambient atmospheric and environmental conditions;
- intended use of the concrete.

Other documents giving guidance upon curing include those by RILEM (1994) and HETEK 4, 7 & 8 (1997).

EN 13670: *Execution of concrete structures* [CEN (expected 2009)] also gives guidance on these matters relating to curing. It describes four curing classes which are defined by curing period or the percentage of the specified 28 day characteristic compressive strength achieved. For curing classes 2 to 4 the minimum curing period in days is given in a series of tables (refer Annex F, EN 13670) based on the surface temperature (t) °C of the concrete and the ratio (r) of the mean compressive strength after two days to the mean compressive strength after 28 days, determined from initial testing or based on known performance of concrete of comparable composition. The ratio will fall into one of three ranges 'rapid', 'medium' and 'slow' corresponding to the rate of strength gain. Accordingly, a concrete mix with a high ggbs content will be 'slow' to cure compared with a pure Portland cement concrete.

Thus curing of the concrete is an essential part of the hardening process. General observations which can be made in respect of curing include that:

- The formwork should not be stripped until the concrete has developed sufficient strength to accommodate the loads and/or strains which may be applied to it.
- Drying-out of the newly-cast concrete surface should be avoided, as this may lead to large plastic shrinkage cracks.
- Premature exposure to freezing conditions may cause severe damage to concrete.
- Some concrete mixes exhibit increased sensitivity to early drying out, particularly those using composite and blended cements and those containing significant amounts of chemical and mineral admixtures.
- It is necessary to minimise early age thermal differences across a concrete section or across construction joints between hardened and newly-cast concretes to avoid crack-

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ing. Experience with Portland cement concretes indicates that in general the following early age temperature difference limits should be complied with:

- A maximum of a 20°C difference over the element cross-section, that is between the core and surface zone.
- A maximum of 10-15°C difference across construction joints and for structures with greatly varying cross-sectional dimensions.

Concretes with very low water-cement ratios, such as high performance concretes, may be particularly sensitive to heat and temperature control during the hardening process. Accordingly, values for temperature difference limits should be lower – and at times considerably lower – than indicated above for Portland cement concretes.

In short, good curing is needed to get the best durability from a good concrete mix. Bad curing destroys the performance of an otherwise good concrete mix. However, good curing cannot compensate for a poor concrete mix. All efforts to ensure an optimal curing regime are likely to be in vain if the initial quality of the concrete mix is inferior.

5.13.7 Controlled permeability formwork (CPF)

Recognition that the quality of the outer skin of the concrete has a dominant influence upon the durability of a concrete structure has lead to the development of controlled permeability formwork (CPF). The outer concrete skin protects the concrete against the penetration of aggressive substances, in respect to both the concrete and the reinforcement.

CPF is a manufactured formwork system that allows the passage of water and entrapped air from the fresh concrete and through the formwork, while retaining cement particles and other solids. The principal elements of a controlled permeability formwork system are shown in Fig. 5.11-12. The application of CPF as a measure to enhance resistance to reinforcement corrosion is discussed in Section 5.11.4.4. There are various types of commercially available proprietary CPF systems and a number of ad hoc systems that operate in a broadly similar manner. Price (2000) describes these and a general classification system for CPF where the different systems are classed as either being Type I, II and III.

CPF is able to improve the quality of the outer couple of centimetres of the concrete cover by locally controlling the water-cement ratio and by then enhancing the curing in this thin outer zone. Excess water in the surface layer is temporarily absorbed by the CPF liner, thus reducing the water-cement ratio of this zone. However, during hardening when the hydrating cement requires additional water for hydration, this water is then readily available from the CPF liner. Thus, optimal moist curing takes place in the outer layer, improving the quality of the cover and the overall durability of the concrete structure.

CPF is available either as a flexible and tissue-like material or be manufactured as a stiff plastic board. The tissue-like liner material leaves a characteristic patterning on the surface of the concrete. This requires that care be taken when placing the liner on the formwork, particularly to avoid folds or wrinkles as these become a permanent feature visible on the formed concrete surface.

Numerous tests and practical experience have demonstrated the beneficial effects of CPF. From a technical point of view some observers have suggested that this technology may be one of the most durability enhancing developments within the past couple of decades. The

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main drawbacks of this system are the difficulties in fixing the lining to the formwork so attractive appearance of the concrete surface is achieved, and the relative high cost of CPF.

Whilst CPF can improve the quality of the surface zone of the concrete cover, it cannot compensate for local poor compaction or honeycombing.

5.13.8 Self-compacting concrete (SCC)

Concretes which are durable in an aggressive environment typically have an optimised mix composition and a low water-cement ratio. Such concrete is difficult to compact and there is a higher risk of honeycombing, particularly in the cover concrete. In addition, the quality of compaction achieved is extremely dependent upon the competence of the operator of the vibrator. Thus the conundrum arises that the better the concrete mix is from a durability point of view (e.g. low water-cement ratio, etc), the greater becomes the risk of poor compaction being achieved at some locations; which in turn reduces the potential durability of the final structure. This conundrum arises because of the dominant influence of the execution process upon the final quality and performance of the structure, regardless of the quality of the initial concrete mix. Fig. 5.15-3 illustrates some of the challenges of concrete mix design.

Ideally the development of a concrete mix where placing and compaction has minimal dependence on the standard of workmanship available on a particular site should improve the true quality of the concrete in the final structure, and hence its durability. This was an important driving force behind the development of self-compacting concrete (SCC). With the aid of a range of chemical admixtures and optimal grading of the aggregates, concrete with low water-cement ratios can be made to flow through complicated formwork and around complex reinforcement, without segregation. The formwork can be filled and a uniform compaction achieved, without any honeycombing, with no or only minimal additional contribution to the compaction and levelling of the concrete from the workforce on site. This is particularly important for the cover zone concrete.

The reduction in the compaction effort required means that there is less noise on site because of the greatly reduced use of concrete vibrators. This also means that the workforce is less exposed to the potentially damaging effects of vibration, which in the past have caused medical problems such as ‘white-finger’.

However, there are a number of potential drawbacks with SCC. One is that it exerts larger loads on the formwork because of its greater fluidity, which need to be taken into account when designing the formwork. Also the formwork joints need to be tighter (less liable to leak) than for other types of concrete. The use of SCC increases the level of expertise and experience required by the staff in the concrete mixing plant.

The main drawback with the current SCC technology is the potential sensitivity of such concretes to the precise dosing, mixing and transport conditions; together with their dependence on the weather conditions while casting takes place. Under adverse conditions concrete mixes with less than ideal rheological properties may suddenly lose the ability to flow. In addition, the increased first-cost of SCC is appreciable. Further consideration is required to establish the ‘whole-life cost of SCC. Perhaps perversely, some contractors using SCC report that to ensure success with the material they have to employ their most experienced and skilful operators because of the sensitivity of concretes whose rheological performance is highly optimised and may potentially be unstable in some circumstances.

One situation where SCC technology has perhaps been taken up most widely and effectively is in the precast concrete industry sector, where it has allowed environmental improve-

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ments and production efficiency gains to be realised in the factory environment. In this particular situation both the costs and the beneficial effects are borne and gained by the same party (i.e. the precaster), which is not generally the case in the wider construction industry. This is very important. SCC technology has delivered distinct advantages in the production of precast concrete elements.

Further guidance on SCC is given in a number of publications. These include the UK Concrete Society's Technical Report 62 entitled '*Self-compacting concrete – A review*' [Concrete Society (2005)] and the Concrete Bridge Development Group Technical Guide 7 entitled '*Self-compacting concrete in bridge construction: Guide for design and construction*' [Bartos (2005)]. The latter document gives information upon current limitations and potential pitfalls. Interestingly Bartos (2005) notes that '*the basic fact that a mix is self-compacting does not mean it is suitable for a given application*'. The Concrete Society's Technical Report 62 document probably gives more technical information on the overall process of procuring and using SCC, including matters such as specification and conformity testing.

5.13.9 High performance concrete (HPC)

The demand for increased strength and improved durability and performance of concrete structures led to the development of high performance concrete (HPC). This development has been very successful in many respects, with advanced HPC products meeting a variety of complex and demanding structural challenges. It had three main objectives:

- Protecting the reinforcement against corrosion; in particular by creating dense impermeable concrete in the cover zone, with a high resistance to the penetration of aggressive ions such as chlorides and CO_2.
- Resisting deterioration of the concrete itself in aggressive environments containing sulfates, seawater and other chemicals; as well as resisting freeze-thaw attack.
- Achieving compressive strengths high enough to meet evolving structural requirements.

HPC will usually have a relatively high cementitious binder content (often referred to as blended cements), a low water-cement ratio, typically in the range of 0.35-0.40, and a high content of plasticiser and superplasticiser. Such concretes can conveniently be used for bridges, marine works, offshore structures, high rise buildings, etc. where the strength requirements are usually within the range 50-80 MPa.

One drawback has been that these more optimised concrete mixes are more sensitive in their handling during transport, placing and compaction. Their successful use places high demands on the competence and skill of the workforce. To varying degrees HPC differs from the previous 'conventional' types of structural concrete in the following ways:

- The dosing and mixing processes become more complicated and sensitive in HPC, particularly with respect to the timing and sequence of adding the various mix ingredients.
- HPC mixes are more thixotropic and 'sticky' such that their placing requires special methods and routines.
- The compaction process is more demanding as more vibration energy is required for HPC; accordingly the vibration needs to be applied more intensively.

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- The denseness of HPC, when correctly placed and compacted, eliminates bleed water. As a result the available water is so limited that protection against evaporation needs to be introduced promptly following levelling and trowelling of horizontal surfaces exposed to drying, in order to avoid plastic shrinkage cracking.
- The high content of cementitious material used in HPC will commonly generate more heat than 'conventional' types of structural concrete; however, this depends on the cementitious materials used in the individual mixes.
- Autogenous (chemical) shrinkage is more pronounced due to the low water-cement ratio, which imposes more rigorous demands on the control of temperature differences arising in the hardening concrete if the risk of early age thermal cracking is to be minimised. The limiting temperature differences that are stated in Section 5.13.6 might have to be halved. In extreme cases these may need to be reduced by perhaps a factor of three.
- HPC requires air entrainment to be frost resistant on the basis of the generally adopted, but rather severe, freeze-thaw testing regime. However, a concrete mix with a high content of superplasticiser is usually difficult to entrain air in. Furthermore, the entrained air may disappear during the compaction process because of the greater amount of vibration energy often required by HPC mixes. As a result HPC may be more sensitive to freeze-thaw attack and therefore it can be less frost resistant than conventional types of structural concrete.

Whilst the above list of HPC sensitivities are unlikely to be relevant to all types and uses of such concretes, it is sensible to bear these potential problems in mind when designing structures based on the application of HPC. These are perhaps particularly important when seeking to take account of the influence of locally available concrete materials and labour (refer Section 5.13.2).

One issue which may influence the adoption of HPC is related to the increased availability and cost competitiveness of corrosion resistant reinforcement such as stainless steel reinforcing bars and allied products including galvanised reinforcement and non-metallic fibre reinforced polymer (FRP) bars made from either glass, aramid or carbon fibres. These materials are reviewed in Section 5.11.6.

Further guidance on HPC is given in a number of publications. These include the UK Concrete Society's Technical Report 49 entitled '*Design guidance for high strength concrete'* [Concrete Society (1998)].

5.13.10 Spacers and chairs to support reinforcement

The minimum concrete cover specified, c_{min}, is usually the value adopted in the service life design based on assumptions regarding the penetration of depassivating agents (e.g. chlorides and CO_2) into the concrete. Accordingly it is necessary to achieve this minimum value, c_{min}, in the final structure by taking the relevant tolerances into account in the selection of the type, dimension and distribution of spacers on the basis of the following rule:

$c_{nom} = c_{min}$ + tolerance.

The tolerance value should generally be taken as 10 mm for site work, but this value may be changed for other circumstances. Further details on matters relating to concrete cover are given in Section 5.5.5.

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The distribution of spacers and chairs to support reinforcement shall be adequate to ensure the required cover is achieved taking account of the stiffness of the reinforcement cage or mat and the loadings which will be applied to these elements during the placing and compaction of concrete. The geometry of spacers, chairs and fastening shall ensure accurate and stable positioning of the reinforcement such that it will not be displaced during operations associated with placing and compaction of concrete.

The material from which spacers, chairs and fastening are manufactured shall achieve good bond with the concrete. In this respect plastic spacers are not compatible with the surrounding concrete in that they have no direct adhesion with the concrete and that they have a different coefficient of thermal expansion (by a factor ten or so) to the concrete. In addition, plastic spacers undergo ageing under exposure to the sun and in a marine environment. There have been examples of reinforcement corrosion being initiated by aggressive substances which penetrated at plastic spacer locations.

In aggressive environments high quality concrete spacers are the preferred option. It is important to ensure that the spacers are made from the same high quality concrete required for the actual structure.

5.13.11 Inserts and fixtures

Inserts and fixtures constitute elements which are especially prone to early deterioration. Often deterioration starts from such elements corroding followed by cracking and local spalling, which develops into more widespread deterioration and damage to the structure. The interaction between inserts and the cast-in reinforcement can also cause rapid corrosion. Special precautions need to be taken to counteract such deleterious reactions.

When selecting the type of inserts and fixtures, the composition of the material used and the way of embedding these elements in the concrete and/or fixing them to the reinforcement need to be considered. Factors to be considered include the following:

- Exposure conditions: whether chlorides are present, whether the zone will be submerged, in the splash zone or in the atmospheric zone.
- Contact to the reinforcement and the electro-chemical potentials might be developed.
- Material composition: whether normal carbon (black) steel, galvanised steel or stainless steel is involved.

There is a risk of macro-cell corrosion (see Fig. 5.11-6) when inserts are in electric contact with the reinforcement and exposed to a corrosive environment at the concrete surface. In such cases a reliable long-term solution would be to use stainless steel for the cast-in part of the insert. For the surface element fixed to the cast-in part of the insert, it is possible to use either hot dip galvanised steel or, if no maintenance is expected to be carried out, stainless steel.

5.13.12 Stainless steel reinforcement

Over recent years stainless steel reinforcement has become commercially available in dimensions, strengths and alloy types which are compatible with normal structural requirements for reinforced concrete structures, and at more competitive prices. Such stainless steel reinforcement is corrosion resistant even in highly chloride-contaminated environments. Used selectively in the most exposed zones of the structure the higher cost of stainless steel rein-

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forcement compared with the costs of conventional carbon steel will have marginal effect on the overall initial construction cost for the structure. In addition the through-life costs will be reduced due to savings in future repair and maintenance [Concrete Society (1998)].

From a practical point of view this technology is particularly interesting because it "only" solves the corrosion problem. All other techniques and technologies within design, production and execution can remain essentially unchanged, a fact that is very attractive to the generally very conservative construction industry.

The adoption of the selective use of stainless steel reinforcement potentially allows the use of locally available materials, even where they have a degree of chloride contamination. Furthermore, the durability of the final structure is also less sensitive to the expertise and knowledge of the local workforce. The selective use of stainless steel reinforcement permits the construction of highly durable long lasting reinforced concrete structures. Examples where this approach is of particular value are the extremely aggressive environments in the hot, arid and saline environments as found in places such as the Middle East, particularly in the Gulf and Red Sea environments, and in similar regions in the Far East and Central America.

A convincing demonstration of the performance of stainless steel reinforcement in highly chloride contaminated concrete is given by the 60 year old 2 km long concrete pier which projects out into the Mexican Gulf at Progreso in Mexico. This is reinforced with stainless steel. No appreciable corrosion has taken place within the structure, despite the harsh environment and poor quality materials used in the construction. The chloride levels at the surface of the reinforcement are between 10 and 20 times the traditionally assumed corrosion threshold level [Knudsen et al. (1999)].

A neighbouring concrete pier was built in the 1970's using ordinary black steel reinforcement. This experienced very severe reinforcement corrosion and has been demolished, see Fig. 5.13-1.

Fig. 5.13-1: The 60 year old stainless steel reinforced pier at Progreso in Mexico

The original 1940's stainless steel reinforced pier at Progreso in Mexico, which is shown still intact after 60 years in service without any maintenance. Whereas in the foreground are the remains of a 1970's pier reinforced with ordinary carbon (black) steel after being in service for only about 30 years [Knudsen et al. (1999)]

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The selective use of stainless steel reinforcement can also be extremely beneficial in the industrialised regions suffering severe winters in which extensive quantities of de-icing salts are used. These environmental conditions occur in northern, central and eastern Europe, North America and northern Japan, etc.

Stainless steel reinforcement can be combined with conventional steel reinforcement cast into concrete without risk of galvanic corrosion due to bi-metallic corrosion because the two types of steel reach nearly the same electro-chemical potential when cast into concrete. This is an important condition for general application of stainless steel reinforcement to be used selectively in the parts of the structure where this enhanced corrosion resistance is required [Knudsen et al. (1999)] and [Pedeferri et al. (1998)].

The Krk Bridge, Croatia which connects the mainland and the island of Krk was built between July 1976 and July 1980 provides an interesting case study which illustrates the potential benefits which might be obtained from the selective use of stainless steel reinforcement. It consists of two arches and incorporates the small island of Sv. Marko within the 1310 m overall length of the bridge (Fig. 5.13-2A). The main arch is one of the largest reinforced concrete arches in the world, having a span of 390 m (Fig. 5.13-2B). The length of road over the island of Sv. Marko is 96 m.

The Krk Bridge has been in service for more than 25 years, with the bridge being exposed to strong salt laden winds (gales and the Sirocco wind). The high salinity of the seawater, combined with the wind, accelerated the penetration of chlorides into the bridge elements, columns and the arches. This has provoked corrosion of the embedded reinforcement, as well as micro-cracking, spalling and deterioration of the cover concrete (Fig. 5.13-2C).

As a result both arches have been under intensive repair for a number of years. The associated maintenance and repair works are detailed in Fig. 5.13-3 which shows graphically the through-life remedial intervention costs incurred and projected for the Krk Bridge. In a period of less than 30 years service, these costs amounted to some 25% of the cost of construction of the structure.

Fig. 5.13-3 also shows the estimated additional cost of selective replacement of (a) 30% and (b) 50% of the overall weight of normal carbon steel with stainless steel during construction. It is estimated that this would have added between about 10% and 15% to the cost of construction). The cost estimates were prepared assuming that stainless steel reinforcing bars were ten times more expensive per tonne than normal carbon steel reinforcing bars.

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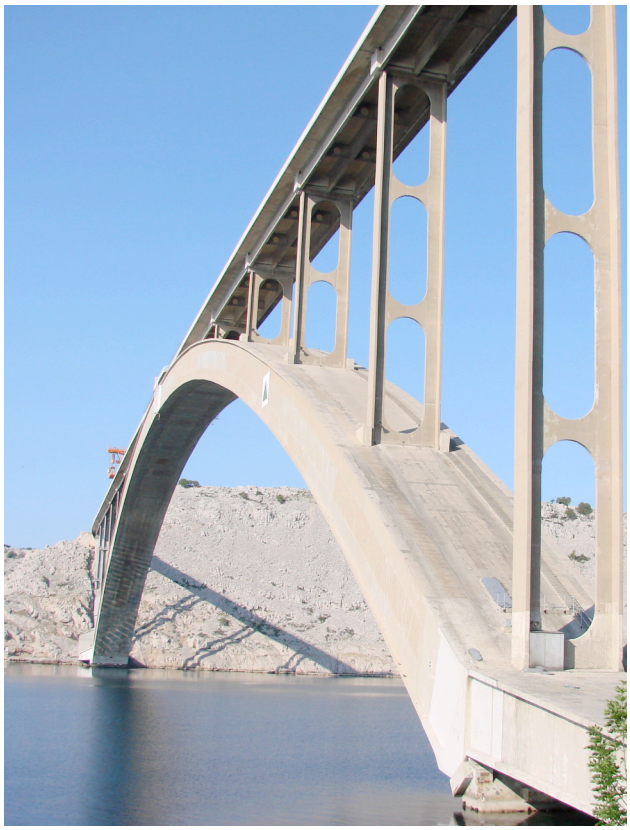
A: View of the Krk Bridge from the island of Krk to the mainland



B: Main span of the Krk Bridge

C: Deterioration of part of the main arch

Fig. 5.13-2: Krk Bridge, Croatia: General views and illustration of deterioration [fib Bulletin 44 (2008)]

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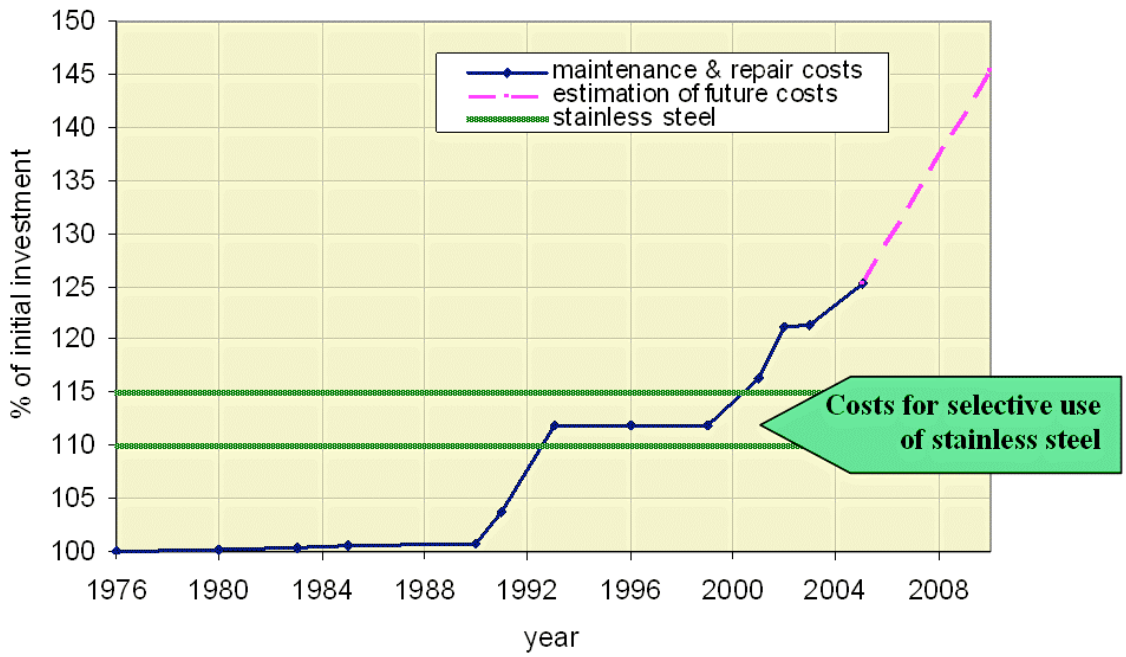

Fig. 5.13-3: Life-cycle costs for the Krk Bridge

Note to Fig. 5.13-3: *The green lines are estimates of the cost of the selective use of stainless steel as a proportion of the bridge construction costs, assuming that (a) 30% and (b) 50% of the overall weight of normal carbon steel was replaced with stainless steel. It was estimated that the replacement of 30% of the carbon steel reinforcement by stainless steel would have resulted in an increase in the cost of construction of about 9%; with a replacement rate of 50% estimated to increase in the cost of construction by about 14%. [fib Bulletin 44 (2008)]*

5.13.13 Quality assurance and quality control

Quality assurance (QA) and quality control (QC) procedures need to be followed in order to enhance the prospect of achieving the construction quality and the associated durability envisaged during the service life design. It is necessary to document the actual quality obtained in the structure by the construction process. This needs to include factual documentation of durability related parameters achieved by the construction process. This would be based on selected tests being made on the finished structure and included in the Birth Certificate (refer Section 5.10.16). Such information would form part of the future operation and maintenance manual to be handed over to the owner upon completion of construction and commissioning activities.

Sections 5.14 and 5.15 also contribute to this general topic. Section 5.14 is concerned with construction quality issues and the potential role that the project execution specification can play in achieving improved durability of concrete structures, especially ones with a long design service life. Section 5.15 is also concerned with ways of improving durability and looks into the benefits to be gained from effective pre-construction planning and trials undertaken in preparation for the construction phase.

5.13.14 Aesthetics and appearance

The aesthetics and visual appearance of a structure are very important. If the visual appearance deteriorates to a level where the structure becomes very unsightly, users may lose confidence in the structure and consider it unfit for continued use. Accordingly it is necessary to avoid the structure appearing to deteriorate prematurely.

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Section 5.5.2 considers issues associated with the geometric form and architectural detailing of a structure or building which affects its appearance. This includes provisions to ensure satisfactory weathering and ageing of exposed surfaces thus allowing structures and buildings to grow old gracefully without excessive maintenance being required. An appropriate selection of geometric form should be made during the conceptual design stage of the project.

Unsightly or dirty structures project an image of apparent poor quality and bad design, which can reflect adversely upon the organisations owning or operating it. Such structures also degrade the wider reputation of concrete structures. This perhaps has lead to the standing of concrete being somewhat lower than this versatile material deserves.

A pleasant visual appearance and attractive ageing properties will help contribute positively to the reputation of concrete as a desirable structural material. All those involved in the creation of concrete structures and buildings should bear this in mind; that is architects, engineers, contractors and owners.

5.14 Construction quality issues – the role of the project execution specification

5.14.1 Introduction

The service life of a structure and its associated through-life management costs and environmental impacts are largely determined by the achievement of appropriately specified durability related requirements (or conversely the failure to define and/or to achieve them) set down in the project specification for the design and construction of a concrete structure. The project specification, incorporating the execution specification and project-specific quality plan requirements, provides the owner and his professional team with a means of defining and achieving the standard of performance they require.

It is critical that appropriate information is included in the tender and contract documents which provide the legal and financial foundation for all aspects of procuring the desired concrete structure. Thus owners and their professional teams need to know how to draw up appropriate tender and contract documents to ensure that problems do not occur which could diminish the actual durability of the structure below the level that is required. The centrepiece of these documents is the project specification. The goal is that these documents should incorporate the necessary preventive action requirements and effective control procedures and that they should not simply create a “forms for forms sake” based approach to quality management.

These issues relate particularly to the execution of construction and to quality plan requirements when the structure is being built. Many of the considerations also potentially apply directly to certain types of structural concrete repair works, particularly those which involve the recasting of part or all of a concrete component. The philosophical points which underlie these considerations also apply more generally to other forms of intervention works upon structural concrete components.

The term execution is used to refer to all the physical activities undertaken for the physical completion (construction) of the works. These activities include procurement, scaffolding, falsework, formwork, reinforcement, concreting, curing etcetera; as well as the related inspection and documentation of those activities. The usage of this term is consistent with the convention adopted in European and international technical construction standards.

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5.14.2 The benefits of "Thinking construction and whole-life performance"

By "thinking construction" long before the construction process actually starts on site, designers develop a construction-focused approach which potentially leads not only to a reduction in construction problems and costs, improvements in health and safety for those involved in the construction, which helps increase the certainty of that the desired outcomes will be achieved, but also to reduced future preventive or remedial intervention costs over the life of the structure.

If a "thinking construction" culture can be developed from the bottom-up, designers are more likely to appreciate the practical problems faced by the workers involved in preparing formwork and falsework, fixing reinforcement and placing concrete. This is important if these workers are to be encouraged to use their initiative and to recognise and eliminate conditions adverse to quality, which could result in nonconforming work that does not meet the longer-term durability related goals for the structure.

To be successful and gain the maximum benefit, the "thinking construction" approach also needs to be actively and practically promoted from the top level of management in all the organisations involved.

This approach should enable the owner and his professional team not only to establish the standard of execution necessary and that the owner is prepared to pay for, but also the definition of the execution requirements to be incorporated in the tender documents. This should ensure that clearly specified project related requirements are in place and used; thereby reducing the possibility that inappropriate assumptions may be made by the constructor about the level of project management, site organisation and workmanship that will be necessary.

As a result, both at tender stage and during the works, contractors and repair specialists will know better what they have to provide and what the owner/his professional team can be expected to provide. As a minimum these actions would provide a basis for negotiation if disputes should arise about inspection and related execution requirements.

For every proposed execution process and for each step in that process, the details of activities involved need to be carefully thought through at a level of detail commensurate with subsequent execution. This should start with the constraints imposed on the process by the planning and design decisions which have been made. This methodology could provide an effective basis for a risk-based approach to managing the construction processes so as to eliminate or minimise non-conformances. In short – effective preventive procedures require a thorough understanding of the proposed execution processes.

Picturing Processes and Potential Problems is a
Primary Prerequisite for effective
Preparation utilising Pre-Planned Procedures that
Put Preventive Practice into the Processes of Production

5.14.3 European construction standards and the CEN Execution Standard

Europe has developed an integrated suite of technical standards for the principal construction materials. In the current context we are only concerned with those relating to the design and construction of concrete structures. Fig. 5.14-1 shows the main modules and the hierarchy of the system of European technical construction standards which provide a basis for the design, execution and selection of materials for concrete structures.

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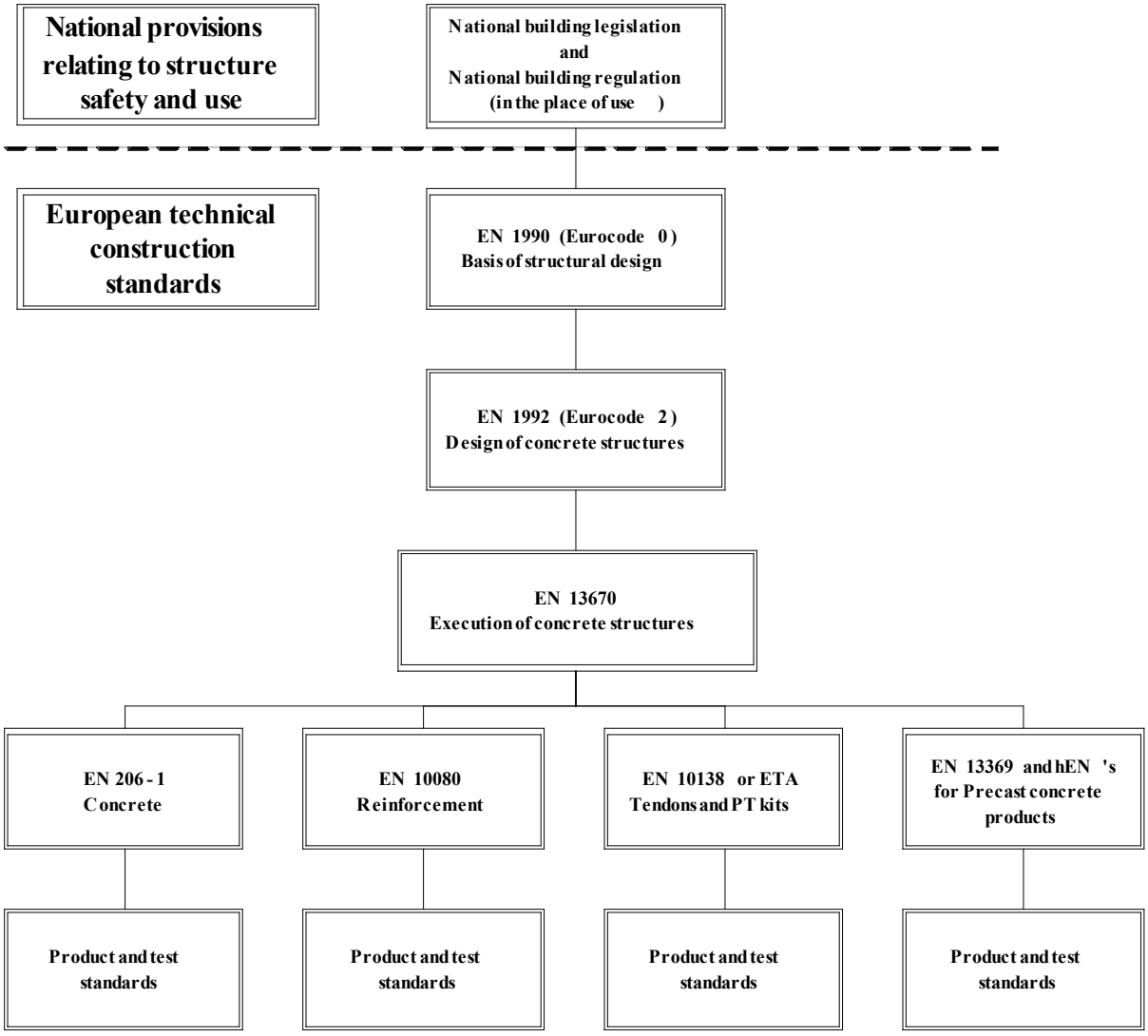

Fig. 5.14-1: Main modules of the system of European standards which provide a basis for the design, execution and selection of materials for concrete structures [fib Bulletin 44 (2008)]

***Notes to Fig. 5.14-1**:*

1. *ETA is a European Technical Approval*
2. *hEN is a harmonised European Norm (European technical standard)*
3. *PT kit relates to the kit of parts and associated operations required for the post-tensioning of a concrete structure*

The European and international approach is used to illustrate philosophical points which are expected to apply more widely and in other parts of the world using different systems for technical standards. In this context it is also relevant to note that the International Organization for Standardization document for the execution of concrete structures (ISO/DIS 22966) is very similar to EN 13670, the applicable European technical construction standard. Thus the principles explained and recommended in this appendix should be applicable to either document, giving a wider potential impact. Both documents due to become available for use in 2009.

The European technical construction standards sit beneath the national building legislation and regulations applicable to the country in which the structure is to be situated. EN 13670

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applies to the execution of concrete structures to achieve the intended level of safety and serviceability during the design service life, as defined by EN 1990 – Basis for structural design and EN 1992 – Design of concrete structures (using the Nationally Determined Parameters which relate to issues of safety as applicable in the country of use).

The Execution Standard [EN 13670: CEN (2009)] has three functions, namely:

- to transfer the requirements set during design to the construction (i.e. to be the link between design and construction);
- to give a standardised set of technical requirements for the execution when ordering a concrete structure;
- to serve as a check list for the designer to ensure that the constructor is provided with all the technical information for the execution of the structure (as given in Appendix A of the Execution Standard).

EN 13670 [CEN (2009)] notes that in order to achieve these objectives the design shall result in a set of documents and drawings giving all information required for the execution of the work. EN 13670 refers to this set of documents as the "execution specification". EN 13670 leaves a number of items open to be decided in the execution specification and it is these items which are of particularly relevant to the recommendations made in this Section.

In the Execution Standard (EN 13670: CEN (2009)] the requirements for quality management are specified in terms of three Execution Classes (1, 2 and 3), for which the required "strictness" increases from Class 1 to Class 3. This gives the option to specify the required level of quality management based upon the importance of the structure/component and the criticality of the execution to its ability to fulfil its intended function. The three execution classes are intended to correspond to the three levels of reliability differentiation given in EN 1990 Annex B [CEN (2002)].

This implies that the achieved quality of construction should increase from Class 1 to Class 3, but no explicit performance related definition of this achievement is given. Instead the Execution Standard [EN 13670: CEN (2009)] relates the three execution classes to the type of inspection to be carried out, who should carry out the inspection and to what standard, the extent of the inspection and whether an inspection report is needed.

5.14.4 Quality management standards – ISO 9000 series

Consideration needs to be given to the contribution which can be made by quality management processes. Whilst ISO 9001 (1987-2000) is accepted world-wide as the model approach for quality management, its implementation for the construction of concrete structures on site can present various difficulties.

There was a change in the focus of the third (2000) edition of ISO 9001 (2000). This adopted the concept of the *"continual improvement"* of an organisation's management system in order to improve overall performance and customer satisfaction. Whilst this is suited to the continued evolution and improvement of an organisation, it did represent a move away from the stated "primary aim" of the earlier (1987 and 1994 editions of ISO 9001 – which were concerned with *"preventing nonconformity at all stages from design through to servicing"*.

It might be argued that the process control philosophy of the earlier editions of ISO 9001 (1987, 1994) was more compatible with the needs of assuring quality in the "one-off" circumstances associated with the site placement of concrete in a particular structure or component.

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Typically the site environment is very different to that of a "factory" involved in the production of standard items using a continuously repeated manufacturing process. The "factory" circumstances are well suited to a process of continual improvement. However, it should also be recognised that the concept of *"continual improvement"* can make a major contribution to establishing an appropriate ethos in the organisation undertaking the construction of concrete structures on site. Both aspects have a role to play.

Quality assurance (QA) is a term which has strong associations with the sampling and testing/inspection systems developed by suppliers to guarantee the quality of their product. An example of this is the premixed concrete industry. However, for concrete structures and other major concrete works, the emphasis placed on the product testing/inspection aspects of quality assurance needs to be accompanied by a greater emphasis on the process control and monitoring aspects of the (site) execution works.

It is proposed that there is a need for the standard of site/process supervision to be related to the circumstances appertaining. In light of this, many contractors are adopting a risk-based approach and related methodology to quality management. This could provide a way forward.

In essence a risk management plan and a quality management plan have the same objective. Both seek to identify risks and eliminate potential problems.

5.14.5 Project quality plan – Illustration of contents

The preparation of a project quality plan, including subsequent review and adjustment during the progress of the works, will assist in the overall process of risk management and achievement of owner requirements. The project quality plan needs input from all parties, including the owner, the professional team and the constructor/repair specialist depending upon the nature of the contractual relationships and the works to be undertaken. The items that a project quality plan would usually address in the design and construction phase are detailed in Appendix G of *fib* Bulletin 44 [*fib* (2008)].

5.14.6 The project specification – Greater certainty of achieving durable concrete structures

Most concreting processes should be considered to be *"special processes"* because process deficiencies may not be evident initially and will become apparent only after the product (i.e. the concrete structure) is in use. In the context of the current discussion the product is typically the cover concrete of the structural element concerned and the potential deficiency will be in the durability and/or the length of time that the structural element performs satisfactorily without some form of preventive or remedial intervention.

The term *"special processes"* was originally defined in ISO 9001: 1987 and this definition is still appropriate in the context of the construction of concrete structures. It was defined as follows:

"These are processes, the results of which cannot be fully verified by subsequent inspection and testing of the product and where, for example, processing deficiencies may become apparent only after the product is in use. Accordingly, continuous monitoring and/or compliance with the documented procedures is required to ensure that the specified requirements are met."

For example, deficiencies in the construction process may compromise the quality of the cover concrete (e.g. lack of compaction or excessive permeability for whatever reason, but

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perhaps due to inadequate curing) or its thickness (i.e. distance over the reinforcing bars). In such circumstances the cover concrete may then not provide adequate protection to the reinforcing bars for the environment in which the structure is situated.

Thus the implication is that deficiencies in the construction process need to be avoided. Essentially there is only one opportunity to "*get things right first time*", which is before and during the time the concrete for the structure is cast.

It is possible to meet the objective of ensuring the durability of the structure by defining inspection clauses in a way that goes beyond the minimum requirements of the Execution Standard. It is for the owner and his professional team to take advantage of the provisions incorporated in the Execution Standard to improve the certainty of achieving the desired quality/standard of execution required.

Essentially the Execution Standard [EN 13670: CEN (2009)] comprises a generic set of construction requirements from which it is expected that the specifier will define requirements which are appropriate for a specific project or element type. It is also expected that the specifier will do this through appropriate clauses introduced into the execution specification, as required to address the specific needs of a particular project or element type.

Annex A of the Execution Standard [EN 13670: CEN (2009)] provides a convenient mechanism for identifying the clauses (in the Execution Standard) which can be used to produce the required project execution specification. This is done through Table A1 of the Execution Standard, which is a checklist of the clauses identifying parameters to be defined to suit project specific requirements. This may involve providing project specific requirements for only a few, but possibly many, of the standard clauses. An example is given in Section 7.2 of *fib* Bulletin 44 (2008) to illustrate the application of Annex A and Table A1 of the Execution Standard [EN 13670: CEN (2009)].

This mechanism would enable designers to go beyond the assumptions currently made during the development of their designs that adequate levels of workmanship, site management and project supervision will be achieved during construction in all circumstances, even in aggressive environments. In such demanding circumstances the commonly made assumptions have frequently been found to be inadequate. More needs to be done to provide assurance that the quality, and hence durability, required for satisfactory through-life performance will be achieved.

5.14.7 Execution management and the requirement for supporting plans

There may be a need to insert additional clauses (requirements) into the execution specification in order to enhance the certainty that the desired durability will be achieved. These clauses will be additional to those in the Execution Standard.

Clause 4 of the Execution Standard [EN 13670: CEN (2009)], entitled *"Execution management",* provides the opportunity for suitable execution requirements to be specified. These requirements might be defined under the following broad headings:

- Inspection Plan, and a
- Concreting Plan, which in turn might be supported by other subsidiary plans such as:
    - Reinforcement Plan,
    - Formwork Plan,

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- Falsework Plan, and
- Post-tensioning Plan.

Some suggestions for, and examples of, these additional clauses are given in Appendix G of *fib* Bulletin 44 (2008). This sets out steps for the development of the above, together with associated supplementary documents to support the preparation of the project specification.

Figs. 5.14-2 and 5.14-3 set out in cartoon format some of the activities involved in preparing to concrete and in the placing, compacting and caring of concrete.

5.14.8 Summary – the project execution specification as a means of enhancing durability

If the above steps are followed during the development of the project execution specification, this should result in an effective working document which will:

1) Comply with the stated and implied requirements of applicable codes and standards.

2) Ensure that all construction processes are effectively analysed with a view to identifying significant potential problems and risk of non-conformance including assessing the level of risk involved in each case.

3) Quality Assurance Actions: Ensure that effective preventive actions [2] are incorporated in relevant process control procedures at all stages of design and construction so as to minimise the identified problems and/or reduce the level of risk to an acceptable value.

   The process control procedures – along with the preventive actions incorporated in them – must be documented so that it is quite clear to the responsible operators and supervisors not only <u>what</u> is to be done during the course of each process, but also <u>how</u> the process is to be undertaken.

   These procedures will define QC roles and responsibilities. The documentation must be such that quality auditors who are knowledgeable and experienced in the relevant processes can verify that the process control procedures are effective.

4) Quality Control Actions: Ensure that verification procedures[3] are prepared and implemented which define how, when, where and by whom each process will be monitored. The observations need to be recorded by appropriately qualified and experienced (independent) staff dedicated to these surveillance activities, with adequate time and resources to conduct the required monitoring role. In addition to the conventional contractor QA roles, the requirements and responsibilities of independent verifiers will also be defined.

---

[2] Quality Assurance (QA) is a process forming part of quality management system focused upon providing confidence that a specified set of quality requirements <u>will be/have an acceptable probability of being</u> fulfilled by a manufacturing process. This is achieved by undertaking verification (eg. inspections and assessments) of the control processes put in place to achieve the specified set of quality requirements.

[3] Quality Control (QC) is a process forming part of quality management system for verifying that a specified set of quality requirements <u>have been</u> satisfied by a manufacturing process. This is achieved by undertaking testing / inspection of an appropriate sample of the output.

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These verification procedures will define all inspection roles and responsibilities, as provided by the Execution Standard [EN 13670: CEN (2009)]. The documentation must be such that quality auditors who are knowledgeable and experienced in the relevant processes so that verification procedures are effective.

Ensure that verification procedures will be developed in full recognition of the "special process" nature of most major construction processes which determine the long-term durability of a concrete structure, or critical parts thereof. These special processes also include welding and galvanising, as well as the basic concreting and associated processes which include fixing reinforcement and erection of formwork. Together such activities form the fundamental elements of concrete construction and repair operations .

The concrete verification procedures should include all necessary testing and validation procedures required to augment the standard compliance tests used to characterise the properties of the hardened concrete within the structure.

The project execution specification should provide for all necessary pre-construction investigation work necessary to ensure that the short-term verification tests on the hardened concrete will provide a reliable assessment of the long-term durability of the concrete in the structure.

5) Ensure that the schedule of items and amounts accompanying the tender documents includes appropriate items for developing and implementing the necessary preventive actions. These should also include separate sub-items for each process to provide for the estimated costs of the contractor providing independent appropriately qualified and suitably experienced QA staff dedicated to these surveillance activities as required to comply with the specified verification (QA) requirements.

   The tender documents also need to ensure that these additional scheduled sub-items are properly evaluated and assessed, with these matters being taken into account in the tender evaluation process.

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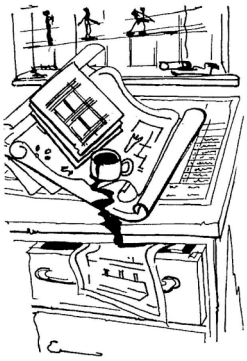
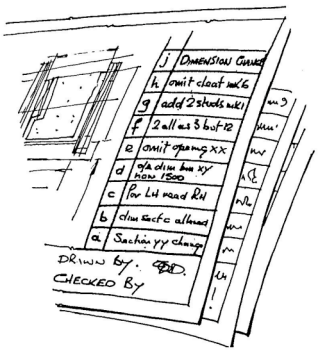
Preparing to concrete

First of all let's ensure that those involved understand the problems and the likely result of omissions.

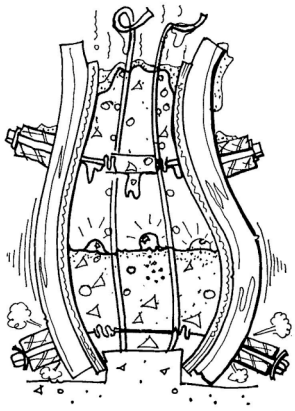


Take a look at the drawings and schedules – are they the latest? (and are they legible?)



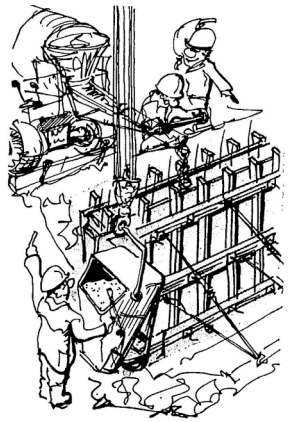
Be warned, there is a traditional game of "Snakes and Ladders" which goes on at the bottom corner of many drawings! – don't be caught out.

What has the formwork designer got in mind, maybe we can help with ideas and local detail.

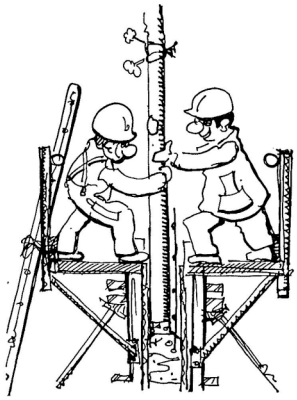


The concrete technologist has a contribution too – he can advise on the effects of admixtures and their likely impact on the rheology of the mix!

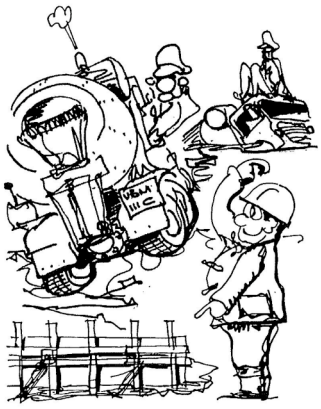
No doubt the weather will have some influence on operations too!



Planning techniques (and unplanned changes in method) can vitally alter rate of fill AND THUS FORM REQUIREMENTS.



The concrete pump is a potent tool and demands good access for distribution. Full head conditions can otherwise develop in minutes.



Good access for plant and concrete supply is critical to the concreting operation. If we can get these trucks sorted out by next month we will take a look at the pre-concreting check!

Fig. 5.14-2: Preparing to concrete by John G. Richardson [Cartoon reproduced with permission from page 60 of Richardson (2002)]

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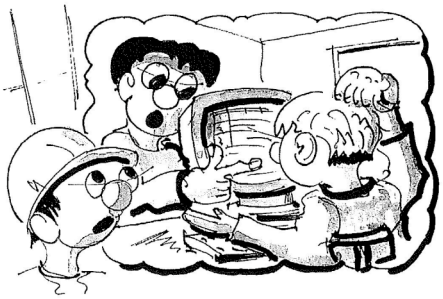
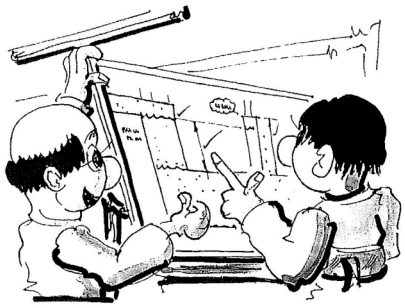
Placing, compacting and caring

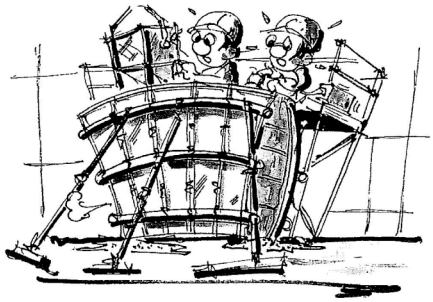


It dawned upon our man that achievement of sound compaction begins not on site but during the design, specification and contract planning...

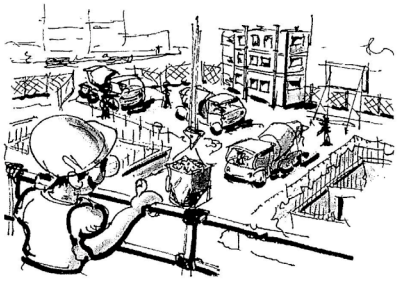


... as agreement is reached about permissible lift heights, construction joint location and formation, and also...

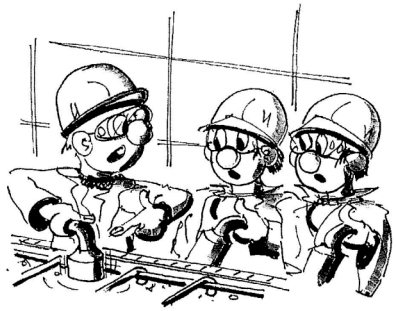
... about sensible reinforcement selection, design and detailing. Such an agreement is needed to allow positive fixing and location as well as access for vibratory gear and to resist displacement during ensuing operations.*



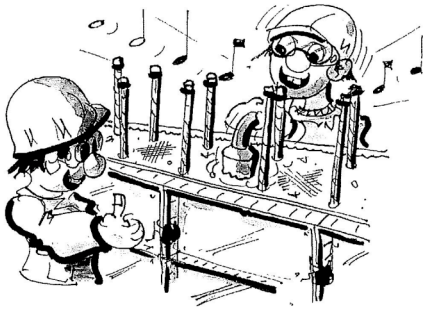
He realised that the formwork has to be robust, grout-tight and resistant to the pressures developing as concrete is placed.



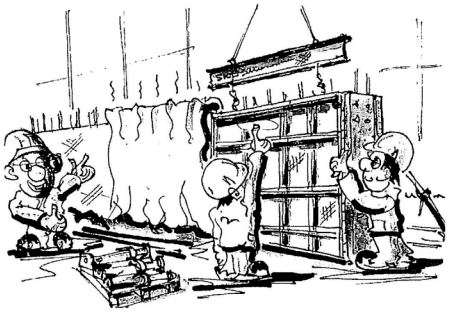
Provided with the essential, steady supply of concrete, designed for the component under construction and delivered to the point of placing...



... a crew trained in the safe use of appropriate equipment and systematic placing techniques, with good access to all parts of the form...



... and able to recognise when good compaction is being achieved (such as cessation of air bubbles, glazing of the surface, a line of paste at the concrete/form interface and possibly a change in the sound of the equipment)...



... and assuming care and attention in stripping the forms and the curing process, the result will be concrete exhibiting all the signs of good compaction. In other words, a job to be proud of!

** Towards rationalising reinforcement for concrete structures. Concrete Society Technical Report 53, 1999.*

Fig. 5.14-3: Placing, compacting and caring for cast concrete by John G. Richardson [Cartoon reproduced with permission from page 79 of Richardson (2002)]

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5.15 Improving durability – Benefits of pre-construction planning and trials

5.15.1 Introduction

Early investment made in pre-construction planning helps to ensure effective construction, thereby contributing to achieving the desired level of through-life performance of the structure, whilst minimising the whole-life cost of ownership of the asset. This involves committing to significant pre-construction funding in order to ensure the acquisition of adequate knowledge about the potential through-life performance and to ensure that the construction processes are controlled and verified in an effective manner. This is necessary so that the (post-construction) behaviour of the structure during operation and use will conform to the required performance levels, and will minimise the whole-life cost of ownership.

The potential activities required are examined under the following six steps:

Step 1: investigation of potential concrete supply problems

Step 2: research into test criteria and procedures for verifying durability

Step 3: trial concrete mixes

Step 4: investigation of potential problems during placement of concrete

Step 5: finalising construction requirements for the project specification

Step 6: provision of adequate resources for quality management – particularly monitoring personnel.

The following sections have been prepared assuming that the owner either has the ability to undertake the various tasks and physical trials or has achieved early contractor involvement in the process of developing and finalising the project specification for the concrete works.

The set of questions presented below should not be considered to be all encompassing or applicable to all circumstances. They are more indicative of the considerations that need to be made. Also they should be sufficient to generate whatever further questions are necessary to ensure that the project quality plan procedures for mix design and control of concrete properties will produce concrete which conforms to the specified requirements.

5.15.2 Some potential problems in producing durable concrete structures

There is concern amongst some experienced engineers and construction personnel that quality assurance procedures employed during the design and construction of concrete structures do not always achieve satisfactory product quality; and this potentially impacts upon through-life performance and the whole-life cost of ownership. An aspect of these concerns is the overall standard of workmanship being achieved during construction.

Durability, however it is specified or evaluated, is generally a property of the cover concrete. Unfortunately this is the part of a reinforced concrete structure which is most vulnerable to the various aspects of poor workmanship, particularly when the structure is built in an aggressive environment. Just a glance at the reinforcement cage shown in Fig. 5.15-1: Part A should convince anyone that the quality of the concrete cover depends greatly upon the planning and execution of the concreting operations. These need to be compatible with the design layout of the reinforcement system and the processes by which it will be assembled and

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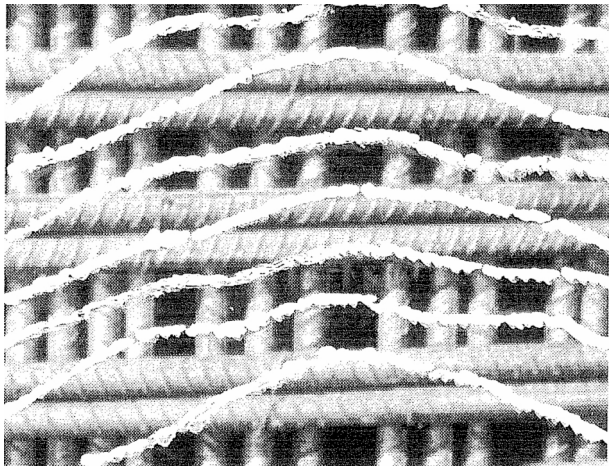
placed. All these matters need to be considered adequately during the planning and design of the structure.

There are a wide range of factors affecting the quality of the cover concrete and the assurance of that quality. Unfortunately these factors interact in a complex, variable, difficult-to-control, difficult-to-observe, difficult-to-specify and often poorly-understood process by which the cover concrete is placed, compacted and subsequently cured.

To help visualise this concreting process Fig. 5.15-1: Part B simulates the movement of the concrete into the cover zone, as it is forced outwards through the available openings in the cage "wall" to work its way along and up the (say) 50 mm wide cavity between the near-solid wall of bars and the formwork. This may help understanding of some of the challenges and difficulties encountered in achieving the specified durability properties in the cover zone concrete.



Part B *(above): Notional progressive level of cover concrete as it flows outward through the larger gaps in the "wall" of reinforcing bars.*

Part A *(left): Congested reinforcement in bridge plinth – note limited gaps between bars available for the "flow" of concrete into the cover zone during concrete placement.*

Fig. 5.15-1: Congested reinforcement in a bridge plinth (courtesy of Storebaelt Authority)

Fig. 5.15-2 is typical of concreting failures which can occur on projects both large and small. Problems were encountered in the compaction of the concrete in the near surface zone, which resulted in a lack of compaction and the presence of "honeycombed" concrete. Whilst obvious honeycombing in concrete will be chopped out and repaired, it may be pertinent to reflect upon how many incidents might have occurred where less than ideal compaction is not recognised, and therefore will not be properly treated or repaired. Such incidents could have a bearing upon the potential durability of the structure and the level of through-life maintenance and the associated cost involved.

Although the non-conformance shown in Fig. 5.15-2 would have been treated to address the obvious performance and durability problems, it must be recognised that repairs are not easy to do well and there is always the potential doubt about their long-term effectiveness.

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Clearly, if the problems can be anticipated, the processes can be planned accordingly and work methods put into place which avoids these difficulties, so much the better. Some aspects of these are discussed below.

Fig. 5.15-2: Example of a potential "durability problem" in base of bridge pylon wall

Note to Fig. 5.15-2: *Extensive honeycombing was revealed after removal of the formwork. Poorly compacted concrete in front of reinforcing bars and behind was removed by cutting back to "sound" concrete by water jetting. It was necessary to cut back behind reinforcing bars to provide adequate anchorage for the repair concrete (courtesy of the Storebaelt Authority).*

5.15.3 Step 1: Investigation of potential concrete supply problems

This step might consume perhaps 10% of additional effort invested in pre-planning.

Matters concerned with the source of aggregates, cement, water, etc. might be identified by posing questions such as:

Q1: Have all potential deterioration problems due to aggregate reactivity been adequately addressed – for all potential aggregate sources?

Q2: Have the relevant properties of the proposed/approved cement, along with other cementitious materials, been adequately researched and tested?

Q3: Have any potential durability problems associated with the proposed water supply been adequately investigated?

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Matters concerned with concrete batching plants which are being seriously assessed to supply concrete for the project, including the associated levels of control and the mix designs required might be identified by posing questions such as:

Q4: How accurately can the total water content of each batch be measured?

Q5: What are the causes of significant variations in water content of the concrete and how difficult are these to control?

Q6: How can these variations be monitored and minimised?

Q7: What is the range of variation from the nominal or nominated water content which can reasonably be expected between batches delivered to site and used in the "final works" of the project?

For each proposed mix design for elements where the durability requirements are critical:

Q8: What are the critical durability criteria – should these include parameters such as shrinkage, the tendency to crack, resistance to chloride penetration and others ? What is the rate at which deterioration might occur?

Q9: Can a reliably consistent, quantitative relationship be established between the standard acceptance criterion for concrete delivered to the site and the critical durability criteria?

5.15.4 Step 2: Research into verification of durability

This step concerns research into test criteria and procedures for verifying durability and might consume perhaps 10% of additional effort invested in pre-planning.

Matters concerned with concrete durability criteria might be identified by posing questions such as:

Q10: Does the specified test procedure accurately represent the actual concrete deterioration process (such as the depassivation process) which is expected to occur in the structure? Does the test concerned provide an understanding of the rate at which deterioration might occur?

Q11: Does the test have a satisfactory level of accuracy over the full range of water/cement ratios at which the concrete will be supplied? Does the test concerned provide an understanding of the rate at which deterioration might occur?

Q12: Can the test criteria for accepting the mix design be accurately correlated with the test criteria for routine acceptance of batches of concrete delivered to the site?

Matters concerned with concrete sampling and testing procedures might be identified by posing questions such as:

Q13: Is the specified sampling procedure such that the test samples are always a true representation of the concrete in the cover zone?

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Q14: Do the specified requirements for preparation and curing of the test specimens ensure that the acceptance test procedure will accurately represent the method of curing which will be used on the concrete in the structure?

5.15.5 Step 3: Trial concrete mixes

This step might consume perhaps 20% of additional effort invested in pre-planning.

Having established preliminary answers to the above questions, the mix design process can now proceed in accordance with the following steps. During this process it may be discovered that the preliminary answers to the above questions (Q1 to Q14) may not be adequate. In this case further research and investigation may be necessary.

a) Determine the allowable range of water-cement ratio.

b) Determine the corresponding range in the chosen durability criteria.

c) Determine the experimental relationship between "durability" and the water-cement ratio over the allowable range. Durability issues cannot be properly assessed until trial concrete mixes are developed and evaluated.

d) Repeat a, b and c if necessary.

e) Confirm the nominal mix and allowable range of water-cement ratio.

During the mix design process the principles embodied in Fig. 5.15-3 need to be understood by all personnel in the contractor's and concrete supplier's organization, as well as by all process verification (QA) personnel.

In particular, they should be aware that the lower level of water content must be accurately controlled and recorded, as this affects placing and compaction of the concrete. It is essential that the water content be kept above a predetermined lower limit to prevent shrinkage cracking. All responsible personnel should be aware of the vulnerability that concrete slabs have to surface cracking when the concrete is placed at low water-cement ratios, particularly under conditions of low ambient humidity.

The impact of the upper level of water content must also be accurately controlled and recorded, as this influences the potential durability of the concrete in other ways. Fig. 5.15-3 identifies the conundrum experienced in relation to the prediction of the service life of a concrete structure and determining which value of water-cement ratio should be employed in these evaluations?

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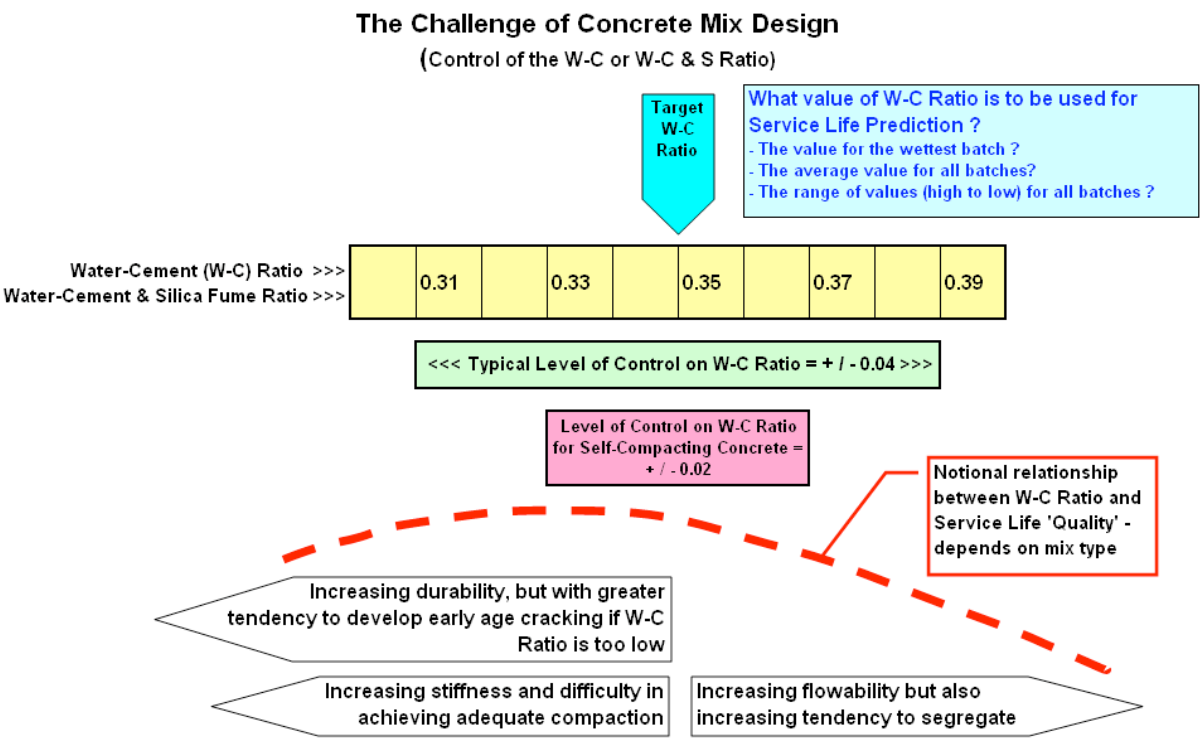

Figure 5.15-3: The challenge of concrete mix design [fib Bulletin 44 (2008)]

5.15.6 Step 4: Investigation of potential placement problems

This step investigating potential problems during placement of the concrete might consume perhaps 20% of additional effort invested in pre-planning.

These issues might be approached by undertaking the following steps:

a) Preparation of small scale (half-size to full-size) detailed drawings, or even three-dimensional computer or physical models. These would be of critical reinforcement configurations that would control access for concreting operations, such as for vibrators, pump hoses and workers. Consideration needs also to be given to illumination in the cover zone, so that the flow of concrete into this critical zone can be visually monitored and thereby controlled. There will be a need to fine-tune these details and proposed work methods until all access and concrete delivery & placement problems are satisfactorily overcome.

b) Construct mock-ups of worst-case scenarios and if necessary carry out trial pours, including batches at the lower and upper limits of the allowable range of water-cement ratio (Fig. 5.15-3).

c) Confirm under field conditions that the trial mix (Fig. 5.15-3) complies with:

- the durability criteria, and does not segregate, at the maximum permitted water-cement ratio; and
- all flowability and compaction requirements at the proposed minimum water-cement ratio.

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d) Compile a comprehensive list of all significant potential concreting and concrete supply problems, together with appropriate preventive actions to minimise the risk of each identified problem occurring.

These activities will require the input of experienced construction engineers, foremen and operators to liaise with the designer and concrete supplier to ensure that avoidable concreting problems are not incorporated into the design of the final works and the construction processes.

5.15.7 Step 5: Finalise construction requirements in project specification

This step seeks to finalise the construction requirements for the project specification, it might consume perhaps 20% of additional effort invested in pre-planning.

These issues might be approached by undertaking the following steps:

a) Prepare necessary supplements to the:

- Concrete production standard(s)
- Execution standard for concrete structures
- Standards (ISO 9001) for quality planning and verification, including development of the associated requirements for procurement and process control, and relevant site management and interface issues, to ensure that the project specification addresses all identified potential concreting and concrete supply problems.

b) Prepare guide project plan procedures, including monitoring procedures, to address all concreting problems identified above.

c) Prepare schedule of items and quantities for insertion into the project specification.

These activities are crucial. If they are not done effectively and comprehensively, the necessary control and verification of the concreting processes will not happen. As a result the durability of the structure will not be assured.

5.15.8 Step 6: Provision of adequate resources for quality management

This step addresses the provision of adequate resources for quality management, particularly the personnel needed for monitoring construction processes, and might consume perhaps 20% of additional effort invested in pre-planning.

These issues might be approached by undertaking the following steps to ensure:

a) All "special processes" are verified by appropriate physical inspection/visual monitoring/surveillance by appropriately qualified and experienced independent staff dedicated to these activities.

b) Quality records are to include the nature, location and extent of all significant observed non-conformances. These are to include suspected non-conformances which may be recorded as potential non-conformity notifications. The records must also include corrective action reports by which the non-conformance was rectified and the approved preventive actions which were implemented to prevent future non-conformances of this nature (corrective/preventive action reports).

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Where effective repairs are not possible this will require commitment from the contractor to provide ongoing maintenance/protection of the non-conforming component.

c) Effective, practical auditing of the contractor's quality system, particularly the procedures for process control and verification of critical concreting operations.

Step 6 is an appeal for the return of effective process monitoring, which is widely recognised as a vital missing element in many of today's construction projects. However, for this to happen, in the end it has to be paid for by the owner. It would be preferable that the owner accepts to pay for these activities at the outset, rather than chase what may prove to be a false economy. Furthermore, potential contractors and sub-contractors must be assured that if they price for the specified quality requirements in their tender, they will not be undercut by competitors who do not.

5.16 Condition control – Planned through-life structure management and care

5.16.1 Introduction

This Section provide a response to the concepts introduced earlier in the service life design process, which requires the structure/its component parts to be allocated to a condition control category at the time of design. Different categories are defined depending on factors such as the importance of the structure, its function, design service life, impact on third parties, environmental conditions, ease of maintenance and cost, etc.

The following is based upon the approach being developed as part of the work undertaken for the new *fib* Model Code (MC2010). On matters relating to issues of durability and service life design of concrete structures, MC2010 largely adopts the concepts and principles defined in the *fib Model Code for Service Life Design* (MC-SLD), which was published as *fib* Bulletin 34 [*fib* (2006)]. The *fib* MC-SLD is discussed in Section 5.7.3.

At the time of writing (2009) MC2010 uses two terms, '*conservation*' and '*condition control*'. However, it primarily employs the term '*conservation*' to describe activities for the through-life management of a concrete structure. Definitions of these terms are given below. Conversely the term '*condition control*' was adopted in the *fib* MC-SLD. In this Section, whilst the term '*condition control*' will generally be used, on the basis that the *fib* MC-SLD is currently (2009) the published reference document, the opportunity has been taken to illustrate the envisaged developments in this subject area.

Conservation *concerns the activities and measures taken which seek to ensure that the condition of a structure remains within satisfactory bounds to meet the performance requirements for a defined time; namely structural safety and functional performance requirements, which may include considerations such as aesthetics. Conservation activities may involve restoring the current condition of a structure to a satisfactory state, or include preventive measures which seek to ensure that the future condition of a structure remains within satisfactory bounds, or improvements to meet revised performance requirements. In this consideration may need to be given to the effects of potential future deterioration.*

Condition control *is the overall through-life process for conserving the condition of a structure. It involves various activities including condition survey, condition assessment,*

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condition evaluation, decision-making and the execution of any necessary interventions; performed as a part of the conservation or maintenance process for the structure.

Condition control activities required to enable a concrete structure to meet its intended service life shall be planned when designing the structure. These plans shall be developed on the basis of the design assumptions and a prognosis for the behaviour under the envisaged environmental and loading conditions over its intended service life, considering the performance requirements that need to be met. The goal will usually be to minimise the life cycle costs for the structure.

The selection of condition control activities will depend on various factors including the:

- Consequences of a potential failure; which are related to the social and economic importance of the structure, the impact on third parties, the function of the structure or components thereof, the design, etc.).
- Feasibility of evaluating the condition of the structure; that is how easy it will be to undertake a condition survey and assessment (which is related to the ease of inspections, measurements, the nature of the design, etc.).
- Predictability of the service life of the structure or its components; which is related to factors such as the design service life, environmental conditions, etc.
- Recording and quantification of the actions during the service life of the structure (e.g. mechanical loads, physical, chemical and biological actions on the structure).
- Feasibility of preventative or remedial interventions (e.g. repairs, replacements, etc.).
- Cost of condition control activities; which are related to matters such as the direct cost of the works; plus the frequency of inspections, repairs, replacements, indirect costs, etc. when a through-life perspective is being taken.

5.16.2 Classes of condition control

During the service life design process a structure/its component parts shall be allocated to a category of condition control. Different categories are defined depending on factors such as the importance of the structure, its function, design service life, impact on third parties, environmental conditions, ease of maintenance and cost. Requirements for condition control should be defined at the time of design. The categories defined in the new *fib* Model Code 20xx [*fib* (expected 2010)] are as follows:

Category A: Structures which are to be managed by planned condition control activities.

- Structures where deterioration would be technically unacceptable or must not be seen.
- Monumental, important or sensitive buildings & structures.

Category B: Structures or parts thereof which are managed by reactive activities.

- Structures where remedial measures can be taken after deterioration becomes visible.
- Buildings and other common structures.

Category C: Structures or parts thereof for which condition control is not practical.

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- Structures where it would be difficult economically and/or technically for preventative or remedial measures to be taken, such as foundations.

Linked to the above classes of condition control; four levels of condition control/inspection regime are defined – see Section 5.16.4.

The necessity for condition control, conservation and maintenance works varies widely depending on factors such as the social and economic importance of the structure, the nature of its design and that of its components, potential threats posed to third parties and the intended (planned) service life of the structure. Account needs to be taken of the potential difficulties of undertaking inspections and associated condition control, conservation and maintenance works, along with prediction of deterioration and execution of intervention works.

5.16.3 Through-life management of a structure – overview of activities

To achieve effective and efficient life cycle management, the condition control process needs to be adequately managed and documented. In particular, the condition control planning shall be prepared in a manner consistent with the selected condition control classification for the structure or its component parts chosen at the service life design stage. In general, the through-life condition control process for a structure involves the types and sequence of activities detailed below. The general flow of the conservation process procedures is shown in Fig. 5.16-1.

- Condition survey, which involves gathering information on the current condition and performance, and related to the previous development of that condition. This might involve activities such as inspections, surveys, testing and monitoring. An initial inspection, the "Birth Certificate" for the structure, should be conducted as the first step in collecting information on the performance of the structure throughout its life.
- Condition assessment, which involves assessing the available information to obtain an indication of current performance and to make a prognosis of future performance for management of the asset, including identification of deterioration mechanisms and prediction of future damage.
- Condition evaluation and decision making activities are concerned with establishing the implications for the conservation of an asset, evaluating potential conservation options, the selection of an appropriate intervention option and the timing of such works.
- Execution of preventive intervention or remedial works.
- Undertaking through-life condition survey and/or monitoring including recording of the information required for life cycle management.

These activities provide feedback to the service life design and updating of the associated theoretical model employed, facilitating assessment of compliance with the original intent or re-design as appropriate. Elements of these activities are survey and monitoring, which are defined as follows:

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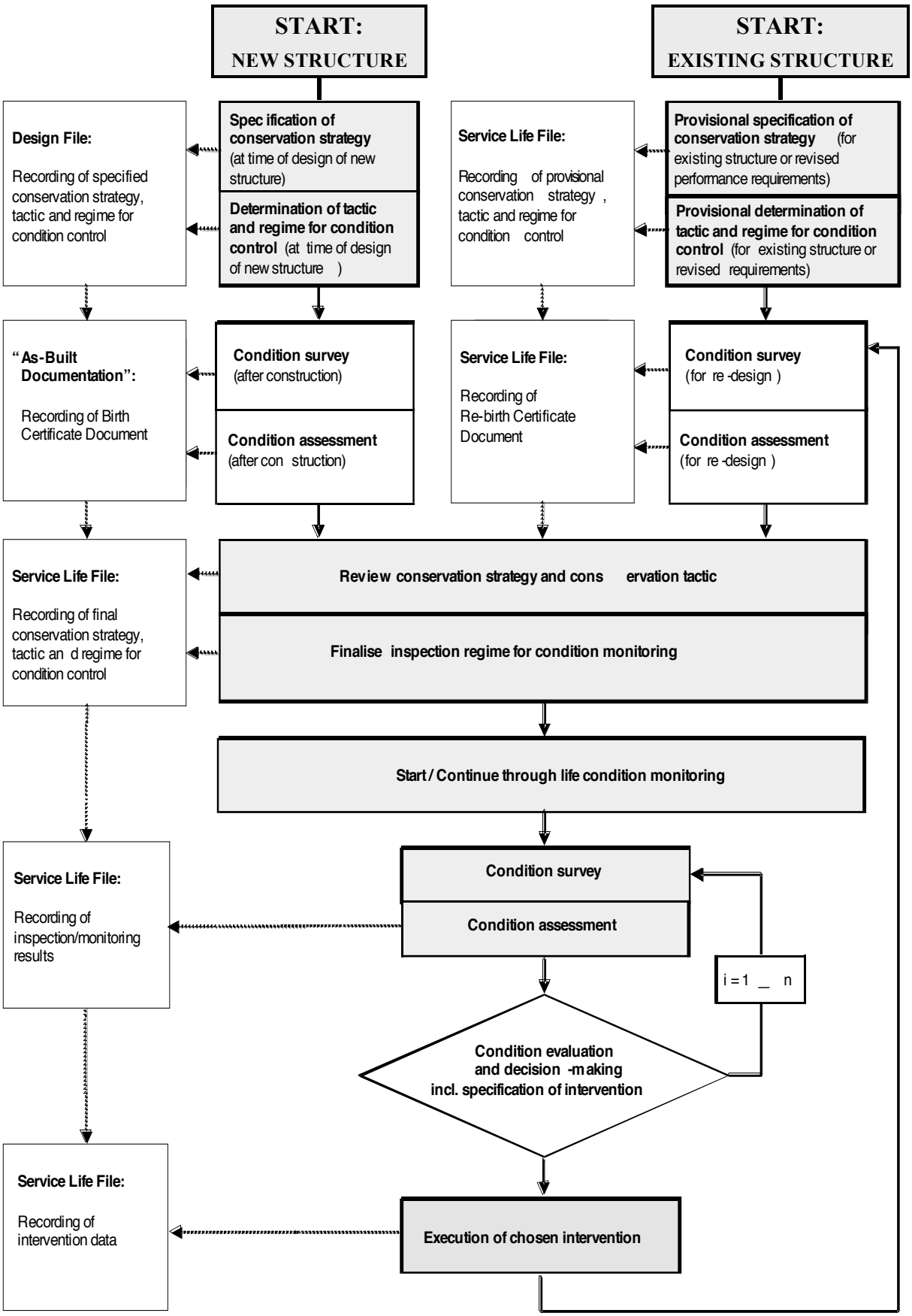

Fig. 5.16-1: Conservation management: Overview of the through-life condition control process and recording of information (from current draft of MC2010)

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'Survey' is taken to mean the range of activities used to evaluate conformity with the design data for actions and/or material and/or product properties used in the service life design (SLD) on a periodic basis during the service life of the structure. Survey activities would be expected to include a visual inspection undertaken in conjunction with various forms of localised condition testing and measurement (e.g. measurement of depth of cover to reinforcement).

'Monitoring' is taken to involve similar activities, but with measurements being undertaken on an ongoing and possibly quasi-continuous basis. Monitoring could involve installed instrumentation. If so, this will introduce different forms of measurement and data gathering to that used for a survey. In some circumstances, these activities might possibly include various forms of local/global response measurement or testing.

5.16.4 Condition control levels/inspection regimes

Four levels of condition control/inspection regime are defined in MC2010 , namely:

CCL3 is a proactive approach utilising a regular inspection, testing and monitoring regime, as appropriate.

CCL2 is a reactive approach which utilises a planned inspection regime but this is only visual in nature.

CCL1 is a reactive approach utilising an ad-hoc inspection and testing/investigation regime.

CCL0 involves no physical inspection, testing or monitoring. In such circumstances it maybe possible to gather indirect indications of performance through ancillary behaviours. For example, the performance of foundations might be inferred from related indirect performance parameters (e.g. structural movements).

Inspections of structures are likely to be carried out primarily visually. However, appropriate testing and monitoring methods/equipment will be required to gather information upon other aspects of the performance of the structure or parts thereof. The extent to which monitoring methods/equipment are employed will depend on the importance of the structure, the category of condition control level, and the expected mechanism(s) and rate of deterioration. These factors will also influence the intervals at which inspections/surveys are made and data is gathered. All the foregoing need to be compatible with the condition control category selected for the structure and its component parts, if these differ.

5.16.5 Strategy using proactive condition control measures

A proactive approach to the conservation of concrete structures is desirable as it should enable early identification of problems and possible risk issues affecting the condition of the structure, potentially enabling early preventive action to be taken to minimise the overall cost of ownership. A proactive conservation strategy should be applied to structures or parts thereof where it is necessary or desirable that the performance be kept above a specified minimum performance requirement.

A proactive condition control strategy is based on preventive measures and interventions aimed at avoiding and minimising future deterioration by applying some form of treatment or taking action prior to damage becoming visible. Tactics for delivering a proactive condition control strategy may involve various approaches including the following:

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- condition based conservation;
- time based conservation

5.16.5.1 Condition based conservation

With this approach the condition of the structure or its components is determined regularly and the evaluated condition shall be registered, together with changes over time. The frequency at which the data is gathered and the associated evaluations are made may vary with time, potentially being related to the perceived criticality of the condition of the structure. Condition based conservation may also be referred to as state dependent conservation.

This approach requires that an appropriate parameter (or parameters) exists which can be evaluated and used to reflect the condition of the structure or its components. The condition based tactic involves norms related to limit states. Exceeding the pre-determined limit states invokes a preventative intervention to extend the service life of the structure.

Condition based conservation might involve consideration of factors such as:

- the number of load or action cycles applied;
- the development of cracking and/or spalling;
- loss of structural strength or stiffness;
- aesthetic effects (i.e. unacceptable deterioration in appearance).

5.16.5.2 Time dependent conservation

This approach involves the use of a time based norm to define when a preventative intervention should be undertaken to extend the service life of a structure.

This involves estimating an appropriate timing of preventive interventions or the time period between such interventions. The goal is to estimate a time period (life-span) corresponding to a sufficiently low probability of failure/non-compliance for the actions being considered, typically taking them as a random variable over the period of time in question.

Criteria for judgement/triggering time dependent preventative interventions could be related to issues such as:

- load or other action effects;
- the development of cracking and/or spalling;
- aesthetic effects (i.e. unacceptable deterioration in appearance).

5.16.6 Strategy using reactive condition control measures

A reactive condition control strategy is based on remedial (or corrective) measures and interventions, aimed at arresting currently active processes which are causing deterioration or damage. Typically remedial interventions involve some form of treatment or the taking of measures after damage has become apparent, presumably involving visual indications (e.g. cracking or spalling of concrete). The tactics for delivering a reactive condition control strategy may include waiting for a specified failure/non-compliant condition to develop.

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A reactive condition control strategy might be applied to structures or components thereof when apparent deterioration does not compromise meeting critical performance requirements. A reactive approach may be appropriate where direct condition control by regular inspections and interventions is not practical (e.g. because it is not feasible, too expensive and/or technically too difficult).

A reactive condition control strategy relates to the following assumed condition control level/inspection regimes. For a new structure these requirements would be defined during the service life design process:

- CCL2 is a reactive approach which utilises a planned inspection but this is likely to be primarily visual in nature.
- CCL1 is a reactive approach utilising an ad-hoc inspection and testing regime.

The failure dependent condition control tactic involves a pre-determined failure norm/non-compliant performance condition. Thus on this basis a remedial intervention, perhaps involving a repair or replacement, to a structural component would not be undertaken until it no longer meets the specified performance requirements. It is suggested that a failure dependent condition control tactic might potentially be applied to those parts of the structure that do not contribute to the probability of failure of the structural system as a whole. However, use of this tactic would not be acceptable if potentially large consequences could arise from the chosen failure norm/non-compliant performance condition.

5.16.7 Situations where condition control measures are not feasible

There are parts of structures where direct condition control, inspection and the application of conservation measures is very difficult or not practical, or where it would be difficult economically and/or technically for preventive or remedial measures to be undertaken. However, it may be feasible to obtain indirect indications of condition or performance of these elements via activities such as land surveying, structure or ground movement measurement, or other indirect but related activities, such as leakage detection.

Before this approach is adopted a judgment should be made about the potential consequences of failure and the feasibility of condition control. Should significant damage or deterioration occur and be detected, a potentially difficult and expensive remedial intervention could be required. These issues and risks might be managed during the service life design process by adding barriers or sacrificial material to provide protection

5.16.8 Condition survey and monitoring activities

Condition survey includes all activities performed to gather information regarding the form and nature of the structure, current or potential deterioration mechanisms and/or change in performance of the structure or service conditions which are needed to evaluate conformity with the design (data) for actions and/or material and/or product properties. Various types of inspection, testing and monitoring regime are specified depending upon the condition control level/inspection regime (see Section 5) adopted for the structure.

Fig. 5.16-2 presents a flow diagram illustrating the decision making process for inspections/surveys and interventions; with particular attention being given to circumstances where judgments need to be made. Further details on condition survey and monitoring activities are given in Section 5.17.

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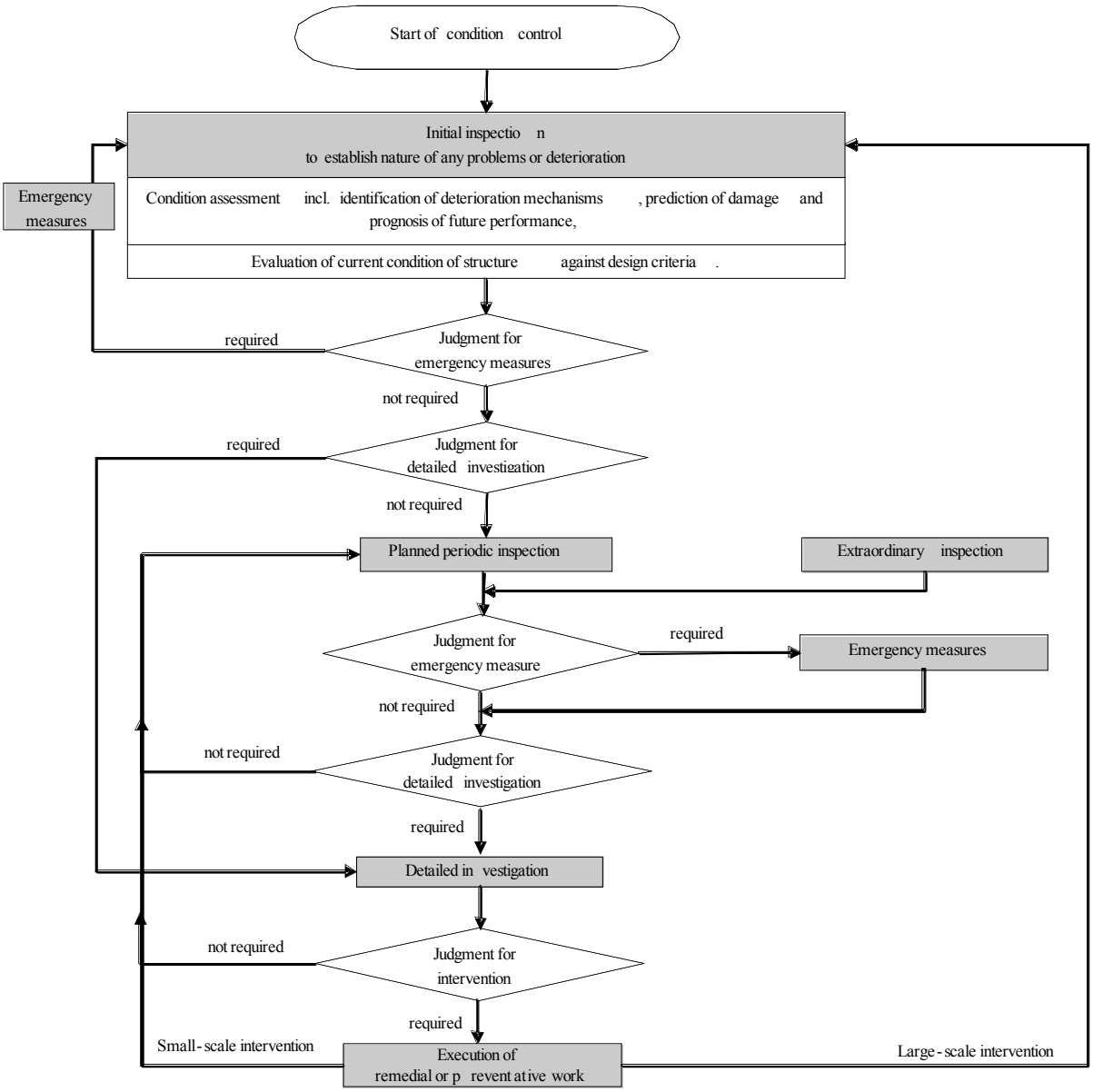

Fig. 5.16-2: General flow of condition survey for structures taken into condition control following construction (from current draft of MC2010)

5.16.9 Condition assessment

The deterioration mechanisms, present deterioration level and deterioration rate of materials and/or structural performance shall be determined using appropriate models on the basis of the information obtained during from inspection, testing and monitoring activities, from the design and construction records, information upon previous interventions and the environmental conditions. This process would be expected to draw upon information obtained for and detailed in the *Birth Certificate*, and potentially also in the *Re-Birth Certificate* assuming that these were available.

Factors influencing deterioration may be classified into the following:

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- Actions upon the structure (external factors) such as temperature, humidity and any other characteristics of the environment, including physical loading,
- Factors related to the nature of the structure (internal factors) such as designed parameters (e.g. material characteristics, concrete cover for the reinforcing bars, geometry of sections, etc.) and quality management during construction.

Consideration may need to be given to the combined (synergetic) effects of deterioration.

5.16.10 Condition evaluation and decision-making

Evaluation of the extent of material/structural performance deterioration shall consider:

- Records in the Design File and the As-Built Documentation (see Fig. 5.16-1).
- Results of inspection and monitoring outlined above.
- Assessment of the nature, severity, significance and rate of deterioration.

Depending on how the evaluation compares with the minimum required performance level, together with prognoses for the future management of the structure or its components; it may be necessary to make preventative (proactive) or remedial (reactive) interventions.

5.16.11 Interventions

Interventions are one facet of the overall process of condition control and through-life management of a structure. They involve time, disruption, as well as cost; all factors to be considered when evaluating possible options. In future issues such as the carbon burden/the environmental impacts will also need to be considered, together with their first-cost and their whole-life cost. Interventions arise in various guises such as those for preventive, maintenance, remedial and strengthening works

On the basis of their planning, potential interventions may be classified as either:

- Planned interventions: i.e. maintenance, preventive and remedial works which are planned at the time of designing new structures or repairs to existing structures.
- Unplanned interventions: i.e. remedial or strengthening interventions not planned at the time of design, but become necessary due to damage or revised requirements.

5.16.12 Recording of life cycle information

To allow effective and efficient life cycle management, all data and conservation records shall be kept and preserved for future reference i.e. in the Service-Life File (refer Fig. 5.16-1).

5.17 Monitoring of durability and performance

5.17.1 Introduction

Designing for a long service life needs some form of verification during the period the structure is in use. Ideally this is a natural part of the operation and maintenance activities. The service life design is based on assumptions regarding the aggressivity of the service environment and the potential deterioration mechanisms. Because of the potential differences which may arise between the assumptions made at the design stage and the actual in-service

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environmental conditions experienced by the finished structure, it is necessary to gather data to verify the service life design assumptions and, if necessary, revise the service life forecast on the basis of new information gathered.

The information presented in this Section compliments the details given in Section 5.16 in respect of condition control processes and, more generally, with planned through-life structure management and care activities.

5.17.2 Locations for surveys, testing and monitoring activities

Locations where inspection, testing and condition monitoring activities are to be undertaken need to be carefully selected so that the desired information about the deterioration of materials and/or structural performance can be obtained, keeping in mind factors such as:

- The environmental conditions.
- The likely mechanism(s) and rate of deterioration.
- The condition control strategy and tactics, and the inspection testing and monitoring regimes defined at the time of design.

5.17.3 Condition survey and monitoring activities

Condition survey includes all activities performed to gather information regarding the form and nature of the structure, current or potential deterioration mechanisms and/or change in performance of the structure or service conditions which are needed to evaluate conformity with the design (data) for actions and/or material and/or product properties. Condition survey is a part of condition control process discussed in Section 5.16. Fig. 5.16-2 presents an overall flow chart of condition survey processes as applied to structures taken into condition control following construction.

Monitoring is concerned with activities that keep watch over a concrete structure, recording progress and changes in materials and/or structural properties with time. Monitoring could potentially also involve the control of an associated process such as operating warning alarms based upon parameters such as applied load, element deflection or some aspect of structural condition or response.

Accordingly, condition surveying is an organised examination by a range of appropriate techniques and procedures performed on a periodic basis during the service life of the structure; while condition monitoring is understood to mean the process of observing such parameters on a quasi-continuous basis.

Significant changes in the data relating to the condition of the structure may be indicative of a developing failure. However, such deductions can only be established through an adequate process of data assessment and evaluation, which enables an appropriate context-based engineering interpretation to be made of the indications obtained.

It is important that surveys, inspections, testing and condition monitoring activities shall be carried out from an early stage in the service life of the structure. Regimes for survey and/or monitoring the condition shall be devised on the basis of the requirements of the condition control strategies selected for the structures or their components. Potential approaches are defined in Table 5.17-1.

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Table 5.17-1: Inspection, testing and monitoring regimes for different classes of condition control

Condition control strategy: **Class of condition control**	**Inspection, testing and monitoring regime** **(Associated Condition Control Level)**
Proactive conservation measures: Category A structures or structural elements – refer Section 5.16	Planned periodic inspection and systematic monitoring of parameters relevant to the design, especially the deterioration processes that are critical for the verification of the limit states associated to durability. (Condition Control Level: CCL3 – refer Section 5.16.4)
Reactive conservation measures: Category B structures or structural elements – refer Section 5.16	Planned periodic inspection (i.e. visual inspection by qualified staff). No systematic testing or monitoring. (Condition Control Level: CCL2 – refer Section 5.16.4)
	Ad-hoc inspection and testing/investigation. No systematic inspection, testing or monitoring. (Condition Control Level: CCL1 – refer Section 5.16.4)
No conservation measures: Category C structures or structural elements – refer Section 5.16	No direct inspection, testing or monitoring. (Condition Control Level: CCL0 – refer Section 5.16.4)

5.17.4 Tools and techniques for surveys and monitoring

Information acquired by inspection, testing and monitoring the condition of the structure can greatly improve the accuracy of performance prediction by more accurately assessing the variability of the input parameters, which are typically assumed to be random variables. This can help reduce the uncertainty associated with the input parameters and aid characterisation of their variability.

Tools and techniques for inspection, testing and monitoring could involve a wide range of procedures. Typically they are likely to include a combination of visual observations, material sampling and possibly selected non-destructive and non-invasive testing methods.

Gathering the data necessary to establish the form and current condition of a structure may involve numerous techniques including inspections, measurements, sampling, as well as various forms of local condition and global response testing. The approaches chosen shall be appropriate to:

- The required reliability of the information to be obtained.
- The conservation strategy and tactics, together with the inspection testing and monitoring regimes defined at the time of design or re-design.

For more information regarding specific tools and techniques for inspection, testing and monitoring reference may be made to *fib* Bulletins 17 [*fib* (2000)] and 22 [*fib* (2003)], as well as to Buenfeld et al. (2008) and Hoult et al. (2009). For more information regarding regimes for inspection, testing and monitoring reference may be made to Moss and Matthews (1995), Matthews (2000) and *fib* Bulletin 44 [*fib* (2008)].

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5.17.5 Gathering data for condition control purposes

Data may be gathered from the structure for condition control purposes by a variety of techniques which are used for undertaking inspections, measurements, testing and monitoring activities.

Inspection, surveys and investigations can be classified on the basis of their nature and the methods to be used, their frequency, the planning and timing of inspections/surveys, as shown in Table 5.17-2.

Table 5.17-2: Gathering data for condition control purposes for various conservation strategies

Data gathering procedure (Inspection / survey type)	Proactive condition control measures	Reactive condition control measures	No condition control measures
Initial inspection or survey	Planned	Planned or Ad-hoc	None
Routine (or regular) inspection or survey	Planned	Planned or Ad-hoc	None
Detailed investigation	Planned	Planned or Ad-hoc	None
Extraordinary inspection or survey	Planned	Planned or Ad-hoc	None

An inspection is a primarily visual examination, often at close range, of a structure or its components with the objective of gathering information about their form, current condition, service environment and general circumstances.

A survey is a process, often involving visual examination but which may utilise various forms of sampling and localised condition testing (e.g. measurement of depth of cover to reinforcement), whereby information is gathered about the form and current condition of a structure or its components. The testing undertaken is usually non-invasive or non-destructive in nature.

An investigation is a systematic process of inquiry undertaken to establish the specific details of a situation without restriction upon the methods used. For example, it may involve inspections, measurements, sampling, as well as various forms of local condition and global response testing to establish the form and current condition of a structure. The process may also provide data for a prognosis of future condition and behaviour of the structure.

5.17.6 General flow of condition survey process

Fig. 5.16-2 illustrates the decision making process for inspections/surveys and interventions; showing where judgments need to be made. When making inspections/surveys or investigations of a deteriorated structure, consideration should be given to whether the structure poses an immediate threat to the environment, is users, any third party, or the inspection/investigation team. In such a situation, suitable emergency measures and temporary works may be required to be taken immediately in order to minimise risks and to attempt to avoid potential collapse, etc.

Depending upon the circumstances, it may be appropriate to take a number of immediate actions to minimise the risk of injury or worse. These measures might include closing the structure to further use, evacuating the structure and the surrounding area, setting up an exclu-

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sion zone to control access to the structure and the area which might be affected by any collapse and/or the release of stored/contained materials, shutting-off of services such as piped gas which might be damaged and create secondary hazards, etc. Detail investigations may be required once the structure has been 'stabilised' and put into a 'safe' condition.

5.17.7 Automated monitoring of concrete structures

The need for adjustment, and the benefits of obtaining more and more reliable forecasts of the service life to be expected, leads naturally to automatic monitoring of the durability performance of the structure in service. Figs. 15.17-1 and 15.17-2 show the River Tees Barrage, Middlesbrough, UK where instrumentation was installed in the concrete structure to monitor in-service performance, in particular the risk of corrosion of the embedded reinforcing steel in the semi-marine service environment.

Detailed information about the specific devices and techniques employed for the automated monitoring of concrete structures reference is given in various publications including *fib* Bulletin 22 [*fib* (2003)], Buenfeld et al. (2008) and Hoult et al. (2009).

Buenfeld et al. (2008) have conveniently produced an overview of the feasibility of automated monitoring of various properties and behaviours in a concrete structure. This overview is reproduced as Table 5.17-3. Their assessment is supported by detailed reviews of the available instrumentation and technologies for automated monitoring of concrete structures, which are linked to reviews of deterioration mechanisms and models for representing these, the application of these models to the prediction of remaining service life using monitoring data, including the use of such data with statistical techniques as required for probabilistic evaluations and the updating of service life estimates. Taken together this represents a large body of work illustrating the current status and potential role of 'intelligent' monitoring in the through-life management of concrete structures.

Fig. 5.17-1: River Tees Barrage, UK during construction – instrumentation was installed for monitoring the in-service performance of the concrete structure (Photo BRE)

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Fig. 5.17-2: River Tees Barrage, UK – illustrating the in-service conditions (Photo BRE)

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Table 5.17-3: Feasibility of automated monitoring of various properties and behaviours in a concrete structure [after Buenfeld et al. (2008)]

Element	**Property or behaviour**			
	Structural change	**Reinforcing bar corrosion**	**Moisture & Temp**	**Chemistry**
Reinforced concrete/prestressed concrete section	Section strength			
	Deflection			
	Vibration			
Joints & cracks	Displacement			
Concrete	Concrete stress	Concrete resistivity	Temperature	Total chloride
	Concrete strain	Cl threshold level	Moisture state	Free chloride
	Cracking	See chemistry column ref: chlorides, pH and nitrite		pH
	Young's modulus			Sulfate, nitrite..
				Alkali silica gel
Reinforcing bar/prestressing steel	Reinforcing bar stress	Half-cell potential	Temperature	
	Reinforcing bar strain	Galvanic current		
	Prestressing wire breaks	Polarisation resistance		
		Cumulative corrosion		
		Electrochemical noise		
External environment	Earthquake	Soil stray currents	Soil/water temp, pressure	Water pH, Cl, SO_4
			Air temp, RH, rainfall	Air composition

Key to Table 5.17-3: Feasibility of automated monitoring

Possible	**Debatable**	**Not currently possible**

5.17.8 Automated monitoring and updating of service life prediction

Fig. 5.17-3 portrays the role of 'intelligent' monitoring in the through-life management of concrete structures and the various activities involved described by Buenfeld et al. (2008).

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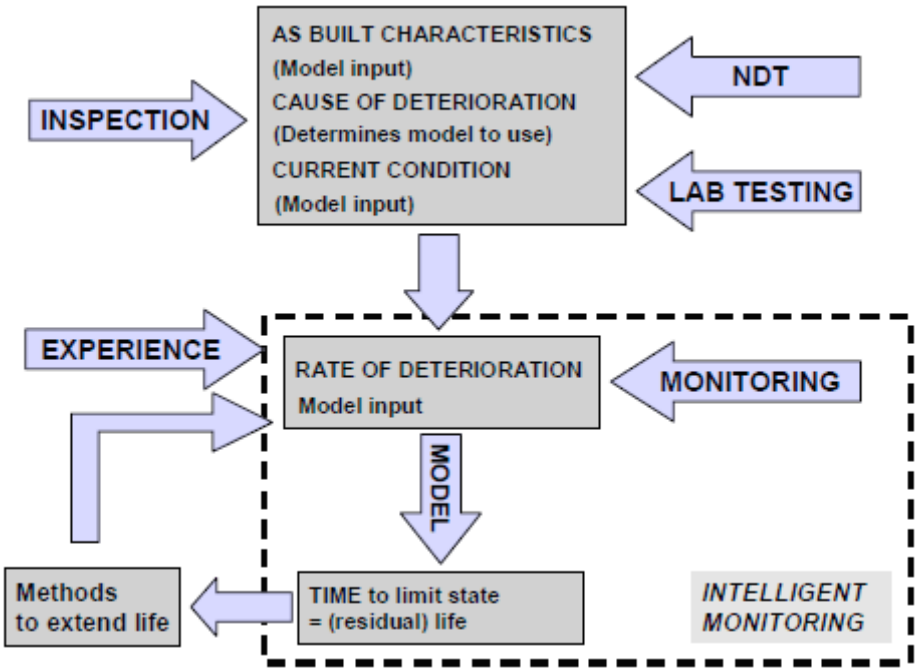

Fig. 5.17-3: The role of monitoring in the management of concrete structures [Buenfeld et al. (2008)]

Buenfeld et al. (2008) review the suitability of service life models for use with automated monitoring systems; they classify service life models into four categories as per Table 5.17-4.

Table 5.17-4: Suitability of service life models for use with automated monitoring

Type of model	Comment on suitability for use with automated monitoring
Empirical models: SL Model type A	***Not suitable:*** Empirical models do not describe the actual chemical or physical behaviours causing deterioration, typically they are simple equations derived from regression analysis of laboratory or field exposure data.
Analytical models: SL Model type B	***Limited suitability:*** Analytical models are derived by solving governing partial differential equations based on simplifying assumptions, with input parameters derived from laboratory or field tests. However, as the models may not realistically describe the underlying chemical or physical behaviours causing deterioration their applicability to atypical circumstances may be questionable.
Numerical models: SL Model type C	***Potentially suitable:*** Numerical models seek to represent the underlying chemical or physical behaviours causing deterioration, with the governing partial differential equations being solved using numerical methods such as finite difference or finite element techniques. However, they commonly require a large number of input parameters, not all of which can be readily obtained. However, where these models prove to be practicable, monitoring data can be used as it becomes available to update the predictions.
Measurable parameters: SL Model type D	***Suitable:*** These models are based upon detectable changes in the chemical or physical behaviour of concrete occurring directly as a result of the deterioration mechanism concerned. Generally this type of model requires few input parameters and these can be measured using sensors embedded in the structure at appropriate locations and depths. Furthermore, these models do not employ complicated mathematical models.

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Buenfeld et al. (2008) review the suitability of some carbonation and chloride ingress models for use with automated monitoring systems; with their overview being reproduced as Table 5.17-5. The models considered correspond to service life models type B and C in Table 5.17-4.

Table 5.17-5: Feasibility of the use of some carbonation and chloride ingress models with automated monitoring [Buenfeld et al. (2008)]

Deterioration mechanism	**Analytical models**	**Numerical models**		
Carbonation	Duracrete (2000)	Saetta et al. (1993)	Isgor & Razaqpur (2004)	Khunthongkeaw & Tangtermsirikul (2005)
Chloride ingress	Duracrete (2000)	Clinconc [Tang (1996)]	HETEK [Nilsson et al. (1997)]	Life 365 [Thomas & Bentz (2000)]

Key to Table 5.17-5: Feasibility of using model with automated monitoring

Possible	**Debatable**	**Not currently possible**	

Buenfeld et al. (2008) also review a number of service life models for reinforcement corrosion which are considered most suited for use with automated monitoring; these correspond to service life models type D in Table 5.17-4. The approaches considered involve:

- Corrosion front monitoring and prediction: based upon the anode-ladder monitoring technique developed by Raupach and Schiessl (2001).
- Direct monitoring of contaminants/aggressive species or other effects in the concrete, such as chloride ions and pH. Monitoring of such parameters is apparently limited by currently available sensor technology. However, in the case of pH change due to carbonation effects, this is not such a significant limitation as measurements of carbonation depth and the prediction of future reinforcement deterioration due to carbonation effects is relatively straightforward.

Buenfeld et al. (2008) also review service life models for the deterioration of concrete due various forms of attack including sulfate attack, freeze-thaw attack, alkali-silica reaction, leaching, acid attack and abrasion of concrete. They suggest that only numerical models for sulfate attack [Marchand et al. (2002)] and freeze-thaw attack [Penttala and Al-Neshawy (2002)] provide limited possibilities of use of such models with automated monitoring systems.

5.18 Examples from practice

5.18.1 Great Belt Fixed Link, Denmark

The Great Belt Fixed Link (*Danish: Storebæltsforbindelsen*) is the fixed link between the Danish islands of Zealand and Funen across the Great Belt. It consists of a road suspension bridge and railway tunnel between Zealand and the island Sprogø, as well as a box girder bridge between Sprogø and Funen. The decision to construct the link was made in 1986; the link was opened to rail traffic in 1997 and road traffic in 1998. At an estimated cost of DKK

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21.4 billion (1988 prices), the link is the largest construction project in Danish history. Its operation and maintenance are performed by A/S Storebælt under Sund & Bælt. Its construction and maintenance are financed by tolls on vehicles and trains. The construction of the Great Belt Fixed Link and the Øresund Bridge have, together, enabled one to drive from mainland Europe to Sweden and the rest of Scandinavia.

The Great Belt Link comprises:

- The East Tunnel: A twin bored 7.9 km long railway tunnel [Storebælt (1997)].
- The West Bridge: A 6.6 km long low level combined road and railway concrete bridge [Storebælt (1998-1)].
- The East Bridge: A 6.8 km long high level road bridge with concrete pylons rising 254°m above sea level supporting a suspension span of 1624 m. [Storebælt (1998-2)].

The West and East Bridges are Europe's two longest bridges and the main span of the East Bridge is the third longest span in the world (Fig. 5.18-1).

Fig. 5.18-1: Great Belt East Bridge: main span of 1624 m; 100 year service life design (photo: Henrik Sendelbach, Wikimedia Commons)

The Great Belt Link was designed for a service life of 100 years, operating in severe marine environmental conditions. This means that the concrete structures were designed and constructed to operate under these severe environmental conditions for period of 100 years without incurring high (unforeseen) costs for maintenance and repair.

The durability requirements needed a smooth, reasonably crack-free and impermeable concrete surface able to resist chloride and sulfate attack from the seawater, freezing and thawing conditions and to avoid alkali-silica reactions. In the case of the bored tunnel, the concrete needs to resist a hydrostatic water pressure of 8 bars.

A multistage protection approach was adopted to provide successive barriers against ingressing chlorides and sulfates in order to ensure the 100 year service life requirements in the

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most aggressive environments [Rostam (1993)]. These are the splash zones of the pier shafts, the tunnel lining segments and the edge beams of the road girders.

The most important factor in this multi-stage approach was the use of high quality concrete. A three powder (ternary) mix incorporating Portland cement, fly-ash and microsilica was used, see Fig. 5.18-2 [Storebælt (1999)].

Fig. 5.18-2: Proportions of a high performance concrete mix.

Note to Fig. 5.18-2: *The water-powder ratio is 0.33. In addition to coarse and fine aggregates, the mix contains three powders (Portland cement, pulverised fuel-ash and silica fume/microsilica), plasticiser, superplasticiser, air entraining agent – and a little water. (Photo: Idorn Consult)*

In view of the marine environment, precautions were adopted to avoid alkali-silica reactions, freeze-thaw damage and chloride induced corrosion. The main provisions to avoid these deterioration mechanisms were the following.

- Selecting aggregates with a very low level of reactivity (i.e. adopting the non-reactive/design-out strategy) and by using pozzolanic additions to the cement (microsilica and fly ash) to further reduce the risk of alkali-silica reactions to negligible levels.
- Using air entrainment to reduce, and hopefully eliminate, the risk of frost damage in the concrete. A very dense and impermeable concrete was achieved by the chosen concrete mix which used a three powder mix and a very low water-powder ratio which had a maximum value of 0.35. The concrete mix was expected to permit only a very slow ingress of chlorides. In addition, should chlorides in sufficient quantities to depassivate the reinforcement reach the depth of the reinforcing bars, the high electric resistivity of the concrete is expected to result in a very limited and slow rate of corrosion.
- Finally, special efforts were made to ensure a high degree of electrical continuity of the reinforcement which would facilitate the future introduction of a cathodic protection system should the need ever arise.

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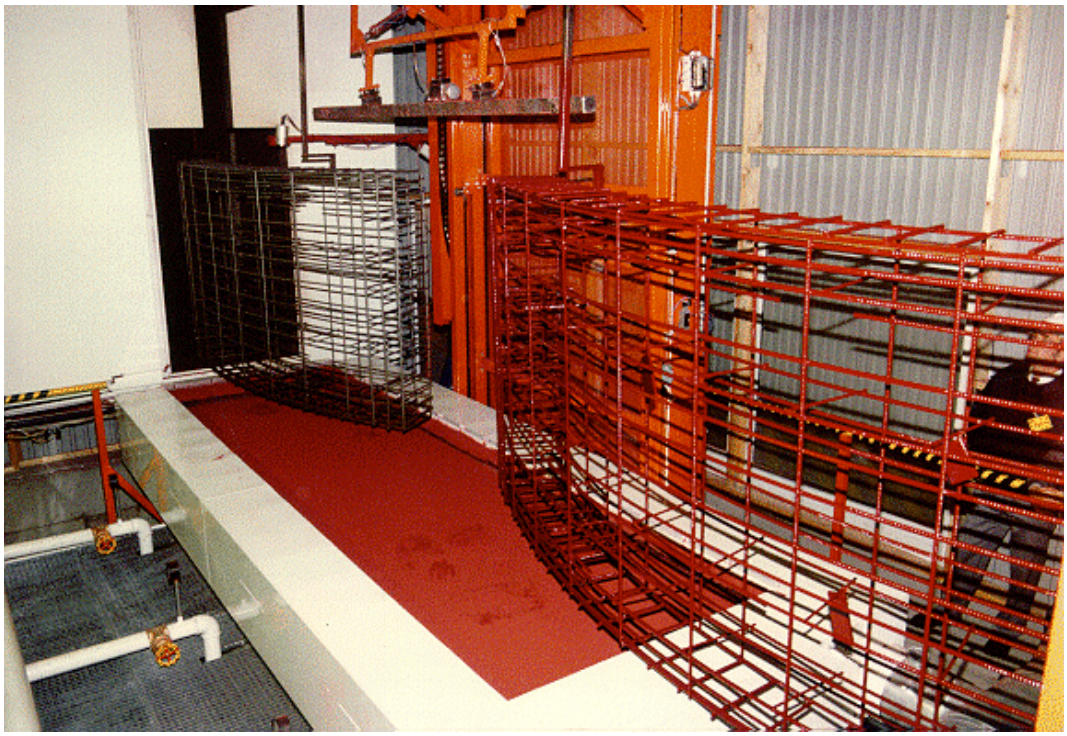

Fig. 5.18-3: Great Belt Link project – Epoxy coating of reinforcement cage

Note to Fig. 5.18-3: *The fluidised-bed dipping technique was used to epoxy coat 62,000 fully welded 3-dimentional reinforcement cages for the precast tunnel segmentsproduced for the East Tunnel of the Great Belt Link project.*

The East Tunnel is the most critical structure in the Great Belt Link from a durability point of view. This is because if serious deterioration should develop, it would be extremely difficult to undertake repairs. This is not only due to restricted access in the tunnel, but primarily because the segments forming the tunnel are critical elements which cannot be replaced without very substantial sealing operations. This would have to be in the form of grouting or freezing of the soil surrounding the damaged zone or establishing a temporary artificial island and shaft down to the tunnel from above sea level.

The multi stage approach to protect the tunnel against incoming chlorides and sulfates is provided by the following consecutive barriers:

- Annular grouting of the void between the lining and the surrounding soil mass, with the grout formulated so it has a high chemical and physical binding capacity in respect of chloride and sulfate ions.
- Dense, low permeability and low diffusion concrete as described above.
- Fully welded 3-dimensional reinforcement cages fully welded and cleaned by grit blasting, heated and dipped in fluidised epoxy powder providing a very reliable epoxy coating without having to cut and bend bars after coating; thereby minimising the potential for damage to the epoxy coating during the construction process. This was a novel technique and damage to the coating was reduced to close to nil by the avoidance of the cutting and bending of the epoxy coated bars. It was also possible to use more durable coating formulations as there were no extra requirements for coating flexibility to allow for post-coating bending. Fig. 5.18-3 illustrates the fluidised bed dipping technique used.

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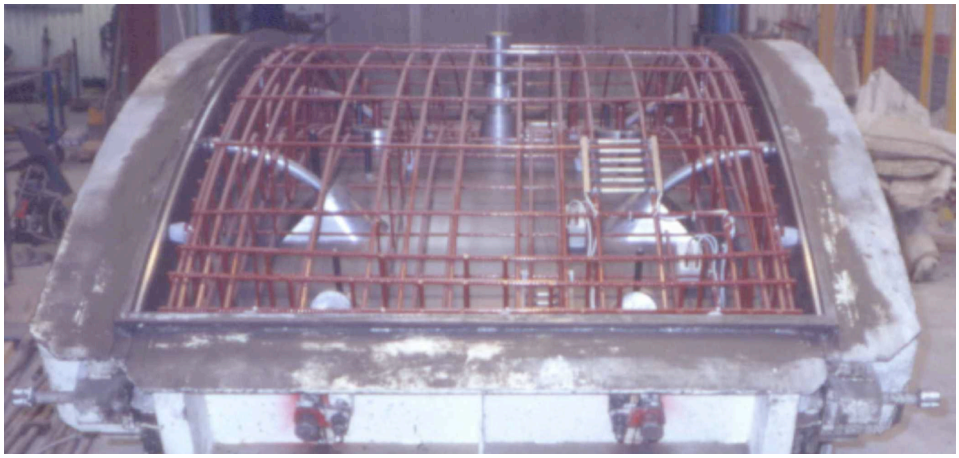
- Creating the possibility of installing cathodic protection some time in the future, if the other durability related provisions prove insufficient at some future time. The fluidised bed dipping technique for epoxy coating also allows for the installation of cathodic protection, if needed. This is not the case with the traditional method of epoxy coating of single bars, which are then electrically isolated from each other when tied together and cast into the concrete.

As a final measure, a number of rings along the length of the tunnel are provided with corrosion sensors as illustrated in Fig. 5.18-4. These should provide early warning of chloride ingress. However, even then the chlorides would have to penetrate the epoxy coating to the reinforcement before corrosion could be initiated. The sensors were wired so they can be linked up to the reinforcement, thus allowing some measurement of the actual corrosion currents flowing should the reinforcement begin to corrode. In such a situation the sensors would provide valuable information to the personnel operating and maintaining the East Tunnel.



Part A

Part B

Fig. 5.18-4: Great Belt Link project – Corrosion sensor installed in a tunnel segment

Note to Fig. 5.18-4: *A total of 450 corrosion sensors similar to the one shown above to introduced to monitor the performance of the reinforcement of a tunnel segment were installed in the three Great Belt Link projects to facilitate the future inspection and maintenance of the structures [Schiessl and Raupach (1992)]*

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The concrete structures of the West and East Bridges are of a more conventional design and are more easily accessible for maintenance and repair works. As a result slightly simpler protective measures were selected.

The concrete cover for the permanently submerged caissons is 50 mm. The cover in the splash zones is 75 mm throughout. For the superstructure of the West Bridge the outer concrete cover is 50 mm and inside the box girders the cover is 35 mm. All these values are absolute minimum values and, due to the high level of quality control performed by the contractor, the addition for tolerance was +5 mm throughout.

In addition efforts were made during binding the reinforcement to ensure electric continuity of the reinforcement. This was to facilitate future easy installation of cathodic protection if found necessary. The electric continuity was checked after casting the critical elements and found to be about 95% for the main reinforcement and about 80% for the links and tie bars. Whilst this was considered to be satisfactory in the present case, better simple measures need to be found in the future to ensure full electrical continuity without having to weld or clamp the reinforcing bars.

For the superstructure of the West Railway Bridge full electrical continuity was achieved by the welding measures introduced to provide full earthing of the reinforcement needed for safety reasons avoiding accidents due to short circuits associated with the railway.

A special protective measure was introduced for the caissons of the West Bridge. Slipforming of the caissons introduced extensive micro-cracking of the concrete. Investigations revealed this to be systematic for most slipformed structures, but to be a more pronounced potential risk when slipforming using high performance concrete. To compensate for this slightly less impermeable concrete and to assist in early passivation of the reinforcement, it was decided to install cathodic protection on all caissons using simple sacrificial aluminium anodes placed on the bottom slab of the caissons, see Fig. 5.18-5. The anodes were designed for 7 years temporary performance, after which the possible continuation of this protective system was to be re-evaluated.

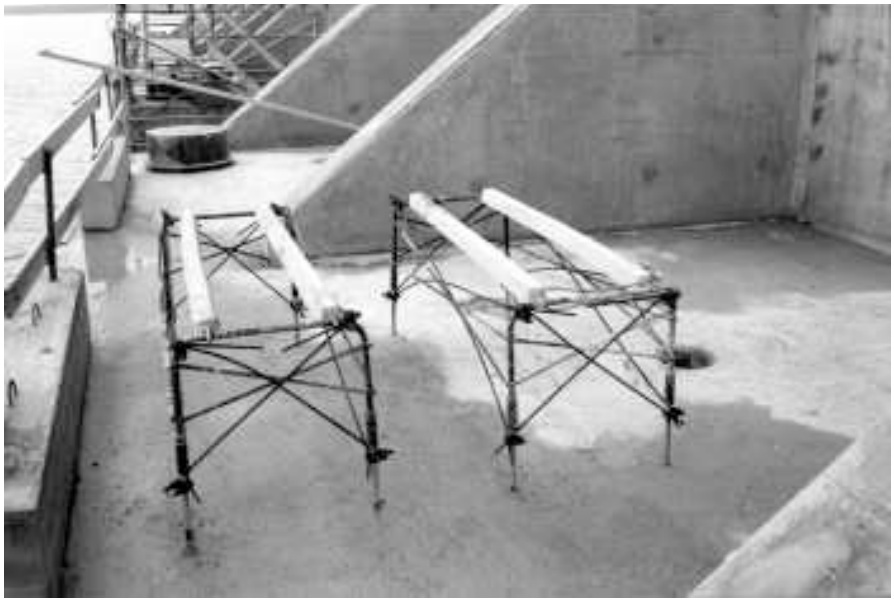

Fig. 5.18-5: Great Belt Link project – Sacrificial anode installation: sacrificial anodes placed on the bottom slab of prefabricated caissons to be placed permanently submerged in the sea.

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Finally, it is appropriate to mention an alternative protective measure which was considered during the design process and was evaluated as part of the tender stage. This considered the adoption of a very large depth of concrete cover for the piers of the Great Belt Link, say 100 mm. However, it was envisaged that this could result in some individual cracks with excessive large crack widths in the large unreinforced cover zone. To control this cracking, a stainless steel reinforcement mesh with a cover of only 35 mm was specified. However, the contractor envisaged that there would be difficulties in casting and compacting concrete in the presence of so much reinforcement. As a result, and because of the novelty of this approach, the indicative costs received at tender stage were very high which made this solution prohibitively expensive.

Nevertheless, the concept of relying mainly on the concrete cover to provide a very long lasting protection of the reinforcement against chloride corrosion is still an option. Recently, new Norwegian specifications for coastal bridges have specified a concrete cover of 120 mm in the splash zone.

5.18.2 Western Scheldt Tunnel, the Netherlands

The Western Scheldt Tunnel (*Dutch: Westerscheldetunnel*) is an 11.3 m diameter 6.6 km long tunnel in the southwest part of the Netherlands on highway N62. It passes under the Western Scheldt estuary between Ellewoutsdijk and Terneuzen. It is the longest tunnel for highway traffic in the Netherlands. The tunnel was opened on 14 March 2003. It is a bored tunnel and consists of two tubes excavated by tunnel boring machine. The tunnel reaches its deepest point under the Pas van Terneuzen, 60 metres below water level.

The Western Scheldt Tunnel represents the first practical application of the DuraCrete design approach [Breitenbücher et al. (1999)]. The Ministry of Transportation, the owner, required a service life for the bored reinforced concrete tunnel of at least 100 years.

The bored tunnel was designed against various forms of reinforcing bar corrosion. The bored tunnel is located in chloride contaminated soil and therefore is subject to chloride attack. The walls inside of the tunnel are not only ventilated, and therefore subject to carbonation, but also exposed to a road traffic induced chloride contaminated salt fog and splash environment. Eventually leaking joints are also expected to lead to chloride attack within the joints, as well to some zones of the inside face of the tunnels, especially at the deep points of the tunnels.

The decision about the thickness of concrete cover within the joints on the Western Scheldt Tunnel project provides a useful case study of the application of the DuraCrete durability design method. Due to structural design constraints the concrete cover within the joints had to be minimised. Accordingly the joints, with their restricted depth of cover, were identified as the most critical durability detail. In the case of permanent leakage the concrete is subjected to chlorides coming from the chloride contaminated soil. As constantly humid conditions are assumed, carbonation can be neglected. The most significant deterioration mechanism identified was chloride induced corrosion. This meant that it was necessary to establish the material's resistance to chloride penetration, so that the depth of concrete cover necessary to fulfil the owner's requirements could be estimated.

The desired service life was 100 years, with the end of service life being defined as the onset of reinforcement corrosion. As this is a serviceability limit state, a minimum service life reliability index of $\beta > 1.50 - 1.80$ was specified. The main steps in the durability design to determine the necessary combination of concrete quality and cover depth were as follows [Gehlen and Schiessl (1999)]:

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- Selection of an appropriate deterioration model: In the case of chloride induced reinforcement corrosion, deterioration models are required which describe the initiation period together with the propagation period. However, in this case it was only necessary to consider the initiation period.
- Establish appropriate input data for the probabilistic analysis: Examples of the types of data needed to perform a probabilistic service life design for these tunnel segments are given below:
  - depth of cover;
  - chloride migration coefficient;
  - critical chloride content;
  - age factor;
  - environment factor;
  - execution factor;
  - surface chloride concentration factor;
  - reference time period.
- Obtain results of the probabilistic durability design: It was then possible to calculate whether the stochastic variables representing the design depth and quality of cover concrete was sufficient to verify the serviceability limit state for a minimum service life period of 100 years. The result of the calculations proved that, for the finally chosen concrete mix and the planned concrete cover, a durable structure would be obtained with a reliability index of $\beta = 1.50$ in respect of chloride induced steel corrosion. A reliability index of $\beta = 1.50$ gives a probability of failure/non-compliance of 0.068. This was in agreement with the specified design criteria, which demanded a minimum service life reliability index of $\beta = 1.50$.

5.19 Future look – Some potential developments influencing service life design

5.19.1 Introduction

There are a number of developments which potentially may have an influence in the coming years upon durability issues, service life design and related matters. Whilst it is not possible to give a comprehensive overview of these, the topics include items such as:

- the *fib* new Model Code (MC2010);
- improved scientific understanding of deterioration processes;
- developments in cementitious materials;
- the wider use of probabilistic performance based service life design concepts;
- life cycle cost optimisation to reduce whole life cost;
- life cycle environmental impacts and sustainability considerations;
- process changes, such as more emphasis on factory production of concrete elements.

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A number of these topics are examined in more detail below to illustrate some aspects of these potential influences.

5.19.2 The proposed *fib* new Model Code (MC2010)

The *fib* Model Code (MC2010) takes a holistic through-life perspective upon design, construction and in-service care and management of concrete structures. As a result MC2010 also addresses issues concerned with matters such as sustainability, through-life care, impacts and management, as well as providing guidance upon matters of structural design. These broader issues are reflected in a consistent manner in the recommendations made concerning execution, materials and conservation, as well as in the service life design concepts as described here. Accordingly, MC2010 is expected to have an influence on the engineering profession over the coming years encouraging a greater focus upon service life design concepts, durability issues and the adoption of a through-life perspective for concrete structures.

5.19.3 Improved scientific understanding of deterioration processes

There is a need to improve understanding of the effect of the nature and variability of processes which occur at the small scale. In respect of the corrosion of steel reinforcement, these mechanisms result in either passivation or corrosion of embedded steel reinforcement; with associated implications for service life design. For example, the potential issues relating to the corrosion of steel reinforcement include:

- The formation and behaviour of the passive surface layer which forms on embedded reinforcement and protects it from deterioration.
- Variability in reinforcing steel composition, which is a re-cycled product, and the influence that this has upon the mechanisms of corrosion.
- The criteria for durability limit states, such as the threshold level for chloride contamination for particular cement types/mix compositions that induce corrosion of embedded reinforcement and the influence of cracks and crack width upon corrosion activity in aggressive environmental circumstances.
- The sequence of corrosion propagation/deterioration which occurs after depassivation of the reinforcement and the implication that this might have for the selection of the most appropriate criteria during (a) design and (b) the assessment of an existing structure, or indeed whether some other condition or criteria should be considered for the end of service life.
- There is also a need to gather more data from actual structures so that simpler service life models can be properly calibrated for wider use. However, this is a long-term process as deterioration is unlikely to develop for many years in actual structures (unless there are durability problems), albeit that changes occur within the cover concrete zone over a shorter period.

5.19.4 Developments in cementitious materials

Various new cementitious binders and changes in concrete materials are in the process of being investigated. Whilst these are expected to have a lower CO_2/carbon footprint than traditional Portland cement, their long-term durability and through-life performance have not yet been established. These materials include calcium sulfoaluminate (CSA) based cements and geopolymers. The development of new materials for large-scale use is an inherently long-term

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process and much still remains to be done before such materials could be accepted for widespread use. The quantities of CO_2 involved are significant. Based upon an emission of 0.83 tonne of CO_2 per tonne of cement, this gives an estimated current annual global emission of over 2 billion tonnes of CO_2 from cement manufacture; rising to over 5 billion tonnes of CO_2 in 2050.

For example, recent research work [Quillin: BRE Report 496 (2007)] which examined the implications of the use of calcium sulfoaluminate based cements on matters such as CO_2 reduction and concrete properties concluded that:

- CSA-based cements can be used to produce durable concrete with physical properties comparable to those of otherwise equivalent Portland cement concrete.
- The CO_2 generated from calcination of limestone in these mixes is likely to be more than 80% lower than that of Portland cement (CSA/ggbs/anhydrite blends compared with Portland cement concrete, with the savings achieved depending upon the formulation employed).
- Blends of calcium sulfoaluminate based cement (CSA) with materials such as ground granulated blastfurnace slag (ggbs) and calcium sulfate have good physical properties and are suitable for use in precast concrete manufacture. Their 28-day compressive strengths exceeded those of otherwise equivalent Portland cement concretes, although the early age strengths were lower.
- CSA based cements require raw materials that may not be widely available.
- CSA-based low carbon cements might be made with a range of compositions and in blends with other materials. Appropriate formulations of these cements might therefore be made with locally available raw materials. However, the implications of such variations on durability and performance would need to be evaluated.
- Work is required to investigate the suitability for use of materials such as pfa in blends with CSA cements and to identify the effect of cement composition on the rate and extent of activation.

From the above it will be seen that whilst the use of a material such as CSA cement potentially offers a number of benefits, especially in respect of lowering the carbon footprint of concrete; it introduces a range of issues which need to be considered and resolved before such a material could be considered for wide scale use. These activities include establishing the long-term performance of these materials, in order to give users and specifiers confidence in their durability, and developing appropriate codes and standards for their use.

However, many other considerations will also need to be addressed. These include cost, the need for the material to have handling characteristics that are suitable for a volume construction material, compliance with health and safety legislation, have setting times that allow placement, a satisfactory rate of strength gain to allow subsequent work to progress, and so on.

The barriers to the uptake of these materials as structural concretes for primary structural elements are potentially more onerous than for so called non-structural applications such as claddings. It may be that their use in the later applications, which are still demanding situations, will occur first.

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The technical development of geopolymers is another aspect of the continuing evolution of concrete technology, potentially offering another way of addressing the goal of reducing the carbon footprint associated with concrete construction.

Other developments in cementitious materials include the use of ultra-high strength concretes. Although these materials have very high cement contents, the amount of material required within structural members is reduced relative to normal strength grade concrete. Potentially this leads to an overall reduction in the overall CO_2/carbon footprint of the structural and some non-structural elements, with associated reductions in CO_2 emissions associated with the transport of materials, the size of the foundations required for the structure etcetera. Although their long-term durability and through-life performance have yet to be established, they are very dense low permeability materials. Accordingly, rates of carbonation and chloride ingress would be expected to be very slow/small. However, the use of ultra-high strength concretes may remain only a small proportion of the overall usage of concrete, so their overall impact may remain modest.

It is perhaps also important to note that in various parts of the world the availability of materials such as pfa and ggbs (at an appropriate quality) may become restricted as locally available supplies become fully utilised. This may change the dynamics of the market for these materials if they become harder or more expensive to obtain. Clearly the need to transport such materials over greater distances would increase their overall CO_2/carbon footprint.

Various aspects of recent developments in concrete technology are also discussed in the proceedings of conferences such as:

- 8th International Symposium on "Utilisation of High-Strength and High-Performance Concrete", 27– 29 October 2008, Toshi Center Hotel, Tokyo, Japan.
- *fib* Symposium "Concrete: 21st Century Super Hero – Building a Sustainable Future", 22-24 June 2009, London.
- 9th American Concrete Institute International Conference on "Superplasticisers and Other Chemical Admixtures", October 2009, Seville, Spain.
- 10th American Concrete Institute International Conference on "Recent Advances in Concrete Technology", October 2009, Seville, Spain.

5.19.5 Further development and application of service life design principles

Principles and service life design options for achieving durable concrete structures have been established. Their successful application requires further work on a number of topics. These activities include:

- Clarifying the basis of the deemed to satisfy options for concrete mixes for specified service life situations given in current codes and, via a calibration exercise, the actual reliability levels being provided in various situations by the deemed to satisfy options presented in current codes.
- Development of a procedure able to unambiguously measure the properties of cover concrete, with the characterisation being able to account of both the contribution of material and the influence of construction quality. This approach concerns the use of durability index tests (e.g. water absorption, oxygen permeability, chloride transport) that potentially could be used to classify a structure into a durability class.

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- The adequacy of the available service life models representing deterioration processes and that the existing models do not properly consider the influences associated with matters such as structural detailing and the quality of maintenance.
- Further work to calibrate service life models and test methods against the field performance of concrete structures. Better guidance is needed on the interpretation of the results from existing test methods and on how to use the results in service life models.
- Examining the influence of the quality of execution upon durability and procedures for updating estimated service life after the completion of structure.
- Establishing what remedial/structure management action should be taken in respect of structures that do not meet the specified requirements of durability specifications and how the associated contractual and through-life ownership management and cost aspects should be evaluated.
- Rational criteria for durability limit states, such as the end of service life and acceptable crack width; together with consideration of code guidelines on these matters which influence the quantity of reinforcement that needs to be provided (which can have significant economic implications).
- Clarification of what constitutes an acceptable probability of exceeding a particular limit state (such as the end of service life).
- The development and use of the Birth Certificate and Re-birth Certificate concept. This type of documentation has an important role to play in the life-cycle management of concrete structures. Potentially Birth and Re-birth certificates could provide a valuable means of communication between the various stakeholders during the design and construction phases (to help ensure an appropriately durable structure was created) and then as a basic element of a predictive life-cycle management system for concrete structures.
- Evaluation of life cycle cost and life cycle carbon (CO_2) burden, perhaps as a proxy for the associated environmental burden/cost.

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Abbreviations

Abbreviations employed in this document include:

AAR	Alkali-aggregate reaction
ASR	Alkali-silica reaction
BRE	Building Research Establishment Ltd
BS	British Standard
BSI	British Standards Institution
CBDG	Concrete Bridge Development Group
CC	Consequence class
CCL	Condition control class
CE	CE Mark
CEB	Comité Euro-International du Béton
CEN	Comité Européen de Normalisation [European Committee for Standardisation]
CIB	Conseil International du Bâtiment [International Council for Research and Innovation in Building and Construction]
CIRIA	Construction Industry Research and Information Association
CONREPNET	Thematic network on performance-based remediation of reinforced concrete structures: CONcrete REPair NETwork
CPD	Construction Products Directive
CPF	Controlled permeability formwork
DEF	Delayed ettringite formation
DSL	Design supervision level
EA	Environmental aggressivity
EC	European Commission
EN	European Norm (a European Standard)
ENV	European Pre-standard (voluntary use)
EXC	Execution class
fib	fédération internationale du béton
FIP	Fédération Internationale de la Précontrainte
FRP	Fibre reinforced polymer
ggbs	Ground granulated blastfurnace slag
HPC	High performance concrete
HSC	High strength concrete
ISO	International Standardisation Organisation
LCA	Life cycle analysis/assessment
LCC/LCCA	Life cycle cost/Life cycle cost analysis
LIFECON	Life cycle management of concrete infrastructures for improved sustainability, competitive and sustainable
LIFECON	Life cycle management of concrete infrastructures for improved

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LMS	sustainability, competitive and sustainable: Life-cycle maintenance and management planning system
LMS	Life-cycle maintenance and management planning system
MADA	Multi-attribute decision aid
MCDM	Multiple criteria decision making
MC-SLD	(*fib*) Model code for service life design
MC2010	(CEB-FIP) Model Code 2010
MC90	*fib* Model Code 1990
MFU	Most frequently used value
mk	Metakaolin
NDE	Non-destructive evaluation
NDP	Nationally determined parameter
NDT	Non-destructive testing
NR	Not required
PBA	Performance-based approach
PC	Portland cement
pfa	Pulverised fuel ash
PL	Protection level
prEN	pre-normative European Standard (a provisional standard issued for comment)
QA	Quality assurance
QC	Quality control
QFD	Quality function deployment
RAMS	Reliability, availability, maintainability and safety
RH	Relative humidity
ROC	Robustness class
RTD	Research and technical development
SCC	Stress corrosion cracking
SCC	Self-compacting concrete
sf	Silica fume or microsilica
SLS	Serviceability limit states
TR	Technical report
TSA	Thaumasite form of sulfate attack
ULS	Ultimate limit state
w/b	Water-binder (ratio)
w/c	Water-cement (ratio)
WLC	Whole life cost

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Glossary

Keyword	Definition
Birth certificate	A document, report or technical file (depending on the size and complexity of the structure concerned) containing engineering information formally defining the form and the condition of the structure after construction. The document/report should provide specific details on parameters important to the durability and service life of the structure concerned (e.g. cover to reinforcement, concrete permeability, environmental conditions, quality of workmanship achieved etc) and the basis upon which future knowledge of through-life performance should be recorded. This framework should provide a means of comparing actual behaviour/performance with that anticipated at the time of design of the structure. The document/report should facilitate ongoing (through-life) evaluation of the service life which is likely to be achieved by the structure. ***Source: fib Bulletin 44: 2008***
Condition control	Concerns the activities and measures taken as a part of the conservation process for the structure. It involves various activities including condition survey, condition assessment, condition evaluation, decision-making and the execution of any necessary interventions; performed as a part of the conservation or maintenance process for the structure. ***Source: fib MC2010***
Conservation	The overall through-life process for conserving the condition of a structure. It seeks to ensure that the condition of a structure remains within satisfactory bounds to meet the performance requirements for a defined time; namely structural safety and functional performance requirements, which may include considerations such as aesthetics. Conservation activities may involve restoring the current condition of a structure to a satisfactory state (or seek to do so), or include preventive measures which seek to ensure that the future condition of a structure remains within satisfactory bounds, or aim to achieve improvements in current condition to meet revised performance requirements; including consideration of future deterioration. ***Source: fib MC2010***
Construction	The overall physical process whereby materials and/or components are brought together to create a building, an item of infrastructure or other man-made facility or asset. The process is deemed to include any necessary preparatory works (e.g. excavation, landfill, etcetera) and finishing works required to be carried out at a particular site or location to facilitate the creation of the desired entity (e.g. bridge etc). ***Source: fib Bulletin 44: 2008***
Cumulative knowledge of through-life performance	The increase in data and information on the performance of a structure or other man-made facility or asset during its service. This particularly concerns the evolution of certain properties or parameters relevant to the durability of the structure, but could also apply to other parameters such as the magnitude and number of structural loadings (especially if fatigue effects were of potential concern) or to data upon the characteristics of the environment(s) affecting the structure. ***Source: fib Bulletin 44: 2008***
Damage	Physical disruption or change in the condition of a structure or its components, brought about by external actions and influences, such that some aspect of either the current or future functionality of the structure or its components will be impaired. ***Source: fib Bulletin 44: 2008***
Degradation	A worsening of condition with time. ***Source: fib Bulletin 44: 2008***
Defects	A specific *deficiency* or inadequacy in the structure or its components which materially affects their ability to perform some aspect of their intended function either now or at some future time. ***Source: fib Bulletin 44: 2008*** An unacceptable condition which may be in-built or may be the result of deterioration or damage. ***Source: EN 1504: Part 9: 1997***

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Keyword	Definition
Deficiency	Lack of something, possibly arising as a result of an error in design, specification or construction, which affects the ability of the structure to perform some aspect of its intended function, either now or in the future. Often concerned with specific issues, such as strength or ductility, but may be more general in nature and concern matters such as durability. ***Source: fib Bulletin 44: 2008***
Deterioration	A worsening of condition with time, or a progressive reduction in the ability of a structure or its components to perform some aspect of their intended function – see *degradation.* ***Source: fib Bulletin 44: 2008***
Diagnosis	Identification of the cause or explanation of the mechanism(s) by which a phenomenon affects the behaviour or condition of a structure or its components based upon an *investigation* of the signs and indications exhibited therein. The term is typically applied to forms of *deterioration* and *degradation* or other mechanisms causing an alteration in the expected or desired behaviour of the structure or its components. ***Source: fib Bulletin 44: 2008***
Durability	Capability of a structure or any component to satisfy with planned maintenance the design performance requirements over a specified period of time under the influence of the environmental actions, or as result of a self-ageing process. ***Source: ISO 13823: 2006***
Environment	The characteristics of the conditions of the atmospheric or surroundings to the structure affecting and influencing its future durability and performance. In the context of this report, generally these are taken to be factors influencing durability (rather than say structural loadings associated with wind or wave effects). These factors need to be taken in to account during planning of service life, design and construction of a particular structure or asset. Environmental factors may need to be considered at different scales ranging from macro level (affecting the overall structure), meso level (affecting say an individual element or component) down to micro level (localised influences affecting a small part of a structure, e.g. a particular joint). ***Source: fib Bulletin 44: 2008***
Functionality	The ability of the structure to perform the purpose for which it was designed. This may be evaluated under various headings and consideration would normally be given to a number of issues affecting either the whole structure, or parts thereof. The issues would typically include the ultimate load case(s) and possibly also various limit state cases (e.g. deflection, vibration, thermal movements, etc.). ***Source: fib Bulletin 44: 2008***
Hazard	An occurrence which has the potential to cause deterioration, damage, harm or loss. ***Source: fib Bulletin 44: 2008***
Ingress	The entry of substances into structural and/or non-structural components of the fabric of a building or structure. Often the term "*ingress*" is associated with the entry of substances which cause deterioration (e.g. chlorides into reinforced or prestressed concrete, sulphates and carbon-dioxide (CO_2) into concretes, etc.). ***Source: fib Bulletin 44: 2008***
Inspection	A primarily visual examination, often at close range, of a structure or its components with the objective of gathering information about their form, current condition, service environment and general circumstances. ***Source: fib Bulletin 44: 2008***
Intervention	A general term relating to an action or series of activities taken to modify or preserve the future performance of a structure or other man-made facility or asset during its service, usually in the context of works to improve its durability and extend its anticipated service life. *Interventions* may be planned or unplanned, but if the former the activity would tend to be classified as a *maintenance intervention*. Thus *intervention* activities might be instigated for the purposes of *repair, rehabilitation, remediation* etc of the structure concerned. They may also be undertaken as a *pro-active intervention* (applying some form of treatment/taking action prior to damage becoming visible) or as a *reactive intervention* (taking action after damage has become visible, e.g. cracking or spalling of concrete). ***Source: fib Bulletin 44: 2008***

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Keyword	Definition
Investigation	The process of inquiry into the cause or mechanism associated with some form of *deterioration* or *degradation* of the structure and the *evaluation* of its significance in terms of their current and future functionality. The term may also be employed during the *assessment* of *defects* and *deficiencies*. The process of inquiry might employ sampling, *testing* and various other means of gathering information about the structure, as well as theoretical studies to evaluate the importance of the findings in terms of the functionality of the structure. ***Source: fib Bulletin 44: 2008***
Maintenance	A (usually) periodic activity intended to either prevent or correct the effects of minor *deterioration, degradation* or mechanical wear of the structure or its components in order to keep their future functionality at the level anticipated by the designer. ***Source: fib Bulletin 44: 2008*** Recurrent or continuous measures which provide protection. ***Source: EN 1504: Part 9: 1997***
Monitoring	To keep watch over, recording progress and changes with time; possibly also controlling the functioning or working of an entity or process. *Structural monitoring* typically involves gathering information by a range of possible techniques and procedures to aid the management of an individual structure or class of structures. It is often taken to involve the automatic recording of performance data for the structure and possibly also some degree of associated data processing. Strictly this does need to be so, there being a variety of means of gathering appropriate data. ***Source: fib Bulletin 44: 2008***
Passive/Passivity	The state in which, due to protective oxide film, steel does not spontaneously corrode. ***Source: EN 1504: Part 9: 1997***
Penetration	The entry of substances, especially moisture, into structural and/or non-structural components of the fabric of a building or structure. In many instances the term "*penetration*" is used interchangeably with the term "*ingress*" (see above), but it may also be used in the context of evaluating the depth to which a deleterious agent has *penetrated* the component concerned (e.g. chlorides have *penetrated* to the depth of the reinforcing steel). The term "*penetration"* may also be associated with the introduction of agents which will help extend the useful life of the structure (e.g. the introduction of resins or corrosion inhibitors into concrete, etc). ***Source: fib Bulletin 44: 2008***
Protection	An action or series of actions undertaken to seek to defend a structure from the effects of further or future *deterioration* by providing a physical or chemical barrier to aggressive species (e.g. chloride ions) or other deleterious environmental agents and loadings upon the in-service performance and durability of a structure. Typically this will often be provided by surface coatings, impregnation treatments, overlays, membranes, electro-chemical treatments, enclosure or surface wrappings applied to the concrete structure, elements or parts thereof. ***Source: fib Bulletin 44: 2008*** A measure which prevents or reduces the development of defects. ***Source: EN 1504: Part 9: 1997***
Re-birth certificate	Similar to the *Birth certificate* for a structure, but relates to the information and circumstances associated with a project for the repair/remediation/refurbishment of the structure or a part thereof to extend its anticipated service life. ***Source: fib Bulletin 44: 2008***
Robustness	The ability of a structure subject to accidental or exceptional loadings to sustain local *damage* to some structural components without experiencing a disproportionate degree of overall distress or collapse. *Robustness* is an indication of the ability of a structural system to mobilise alternative load paths around an area of local *damage*. It is related to the strength and form of the structural system, particularly the degree of redundancy within the structural system. ***Source: fib Bulletin 44: 2008***

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Keyword	Definition
Risk	The combination of the likelihood of occurrence of a particular hazard and the consequences thereof. ***Source: fib Bulletin 44: 2008***
Service Life	Period of time after installation during which a building or its parts meet or exceed the performance requirements. ***Source: ISO 15686: Part 1: 2000*** The actual period of time during which a structure or any of its components satisfy the design performance requirements without unforeseen major maintenance and repair. ***Source: ISO 13823: 2006*** • **Design service life**: the specified period of time for which a structure or a component is to be used for its intended purpose without major repair being necessary. ***Source: ISO 13823: 2006*** • **Technical service life:** the actual time in service until a defined minimum acceptable performance (functional) state is reached, perhaps due to deterioration associated with the service environment. This might be the initiation of corrosion (see below). • **Functional service life:** the actual time in service until the structure becomes obsolete due to changes in the performance (functional) requirements. • **Economic service life:** the actual time in service until replacement of the structure is economically more advantageous than keeping it in service.
Serviceability Limit State	Corresponds to conditions beyond which specified service requirements for a structure or structural element are no longer met. ***Source: fib Bulletin 34: 2006***
Ultimate Limit State	Corresponds to conditions associated with collapse or with other similar forms of structural failure. ***Source: fib Bulletin 34: 2006***
Working Life	The assumed period for which a structure or a structural element is to be used for its intended purpose without major repair being necessary. ***Source: ISO 2394: 1998.***

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