COWI

Design and Construction of Segmental Concrete Bridges for Service Life of 100 to 150 Years

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SUMMARY

Future demands for service life performance of concrete bridges pose multidisciplinary challenges on the designer and the contractor to master structural-, materials-, construction-, and maintenance properties. The growing demand for environmental awareness is an additional element of such designs. The demands for long service lives reflect also on the owner through new challenges regarding service life design basis and corresponding acceptance criteria.

For segmental bridge construction the strive towards savings in materials consumption and reduced self weight have led to the adoption of more complex high performance materials and more sophisticated member geometries. This raises the demand for high quality workmanship and execution competence, which is not always fully recognized and respected neither by owners and designers nor by the supervision teams. Hence such high performance materials may easily lead to low performance structures.

The challenge of providing factual service life designs documenting 100 to 150 or more years design life can be achieved and at times this is surprisingly easy. This new integrated approach identifies a new focus in our design procedures - an important change in design paradigm is needed. Such a changed design focus may lead to dramatically improved service life performance - and will greatly increase the competitiveness of structural concrete. This new perspective will also reflect on revised engineering university curricular.

THE BRIDGE HERITAGE - NOT TO FORGET !

Back in history experience had taught the builders to create very lasting bridge structures from small segments placed so the self weight created a load bearing capacity sufficient to carry high loads so even today such bridges may be in use, as illustrated in Figures 1 and 2, representing 2000 and 400 years service life respectively. This can also be achieved in segmental construction today - using the right technology being adapted to the individual situation.

Figure 1: Pont du Gard, build 19 B.C., road bridge section added during restoration 1743. Early segmental bridge construction now 2000 years old.

Figure 2: The Mostar Bridge having lasted more than 400 years surviving loads and earthquakes - but brought down only by war action. The bridge has recently been rebuilt in it original shape.

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In order to develop the basis for real service life designs for segmental concrete bridges the fundamental of the deterioration mechanisms of concrete structures must be understood.

This mechanism understanding represents the key to service life designs. Without respecting such scientifically sound deterioration mechanisms in the detailed design as well as during the execution process, the real service lives achieved in the individual cases will be random.

For this reason the basics in modern understanding of the structure-environment interaction, and the consequences for design and execution, is highlighted in this paper.

STRUCTURE-ENVIRONMENT INTERACTION

Concrete is the most versatile and robust construction material available and has therefore obtained a dominating position in construction. Thus it becomes an economic disaster when urban dwellings, large bridges, or major marine structures deteriorate just after a few years in service. With increasing frequency such examples have been reported from the 70´ies and on. The reasons are very complex but fortunately the main causes have now been identified.

It is essential to have these causes highlighted with the aim of adjusting - and in some cases rectifying - design methods, construction procedures, material compositions as well as maintenance and repair procedures, to ensure more reliable structures in the future.

Durability requirements - and solutions

During the past years clients have asked for bridge-, tunnel- and marine structures to be designed to satisfy a specified service life, this being typically 100 and 120 years, and in particular cases 200 and 300 years.

Currently, durability is generally "ensured" simply by adopting deem-to-satisfy requirements, which have no rational relationship to the real service life being achieved, - and the service life as such is not defined in a manner being operational in a design situation.

There is no generally agreed methodology available today on the basis of which such designs can be made and the results be verified. In fact, the present design methods do not consider the time factor on other effects than creep and shrinkage and their ULS and SLS effects.

The Pantheon in Rome, Figure 3, with its famous spherical concrete dome, represents an about 2000 years old concrete structure, just as the aqueduct in Figure 1. This structure also documents that very long service life can be achieved for concrete structures when they are designed intelligently.

Figure 3: Pantheon, Rome. The spherical dome is a 2000 years old concrete structure in full service today.

Characteristics of concrete structures

With respect to deterioration, concrete structures have some important characteristic properties, which differ fundamentally from structures made from other structural materials.

These properties are the following [1]:

> The quality of the concrete and the designed durability performance of the structure are only assumed properties at the design stage.
>
> The true quality and performance characteristics of the structural concrete are determined through the actual execution process during construction on site. Hence, the very short time period of construction (hours, days and weeks) constitutes the most important phase, where the required durability performance of the finished structure is determined.

To manage these special properties of concrete structures integration is needed of a durability performance based design concept approved by the owner, a conscious execution process, and a planned inspection and maintenance program. The initial design shall take the approved future maintenance strategy into account.

Durability and service life definition

A structure is considered durable when it performs satisfactorily and maintains acceptable appearance for as long as the owner and the user need the structure. However, such a definition is not operational as basis for design, maintenance and repair.

The operational way of designing for durability is to define durability as a service life requirement. In this way the non-factual and rather subjective concept of "durability" is transformed into a factual requirement of the "number of years" during which the structure shall perform satisfactorily without unforeseen high costs for maintenance [2]. In this way the time factor is introduced as a design parameter.

Designing for a specified service life requires knowledge of the parameters determining the ageing and deterioration of concrete structures. Hence, the precondition is to have scientifically sound data and mathematical modelling available of the:

Environmental loadings.

Materials and structural resistances, including transport mechanisms for substance moving into and within concrete, and deterioration mechanisms of concrete and reinforcement.

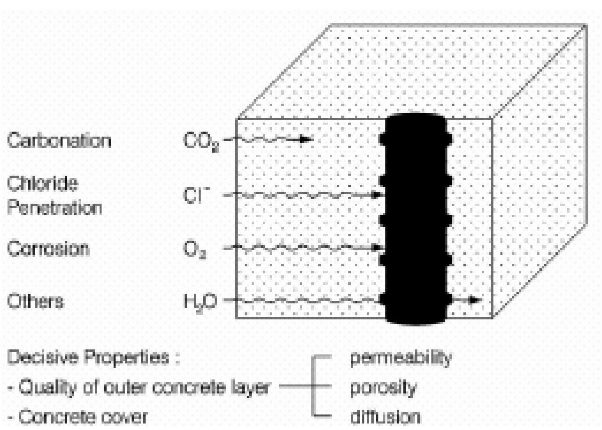

Figure 4: Importance of the penetrability of the outer concrete layer, and the thickness of the cover on the reinforcement to protect the structure against ingress of aggressive substance and deterioration of concrete and reinforcement - see the corner effect on Figure 5.

Therefore, it is evident that the quality of the outer concrete layer exposed to the aggressive environment becomes the one single most important quality determining parameter, Figure 4, [3].

With respect to controlling the time to corrosion initiation of the reinforcement, the thickness of the concrete cover becomes an additional design parameter.

This is the only rational way of performing a quantified service life design for new concrete structures - as well as a residual service life design for existing structures.

Figure 5: With moisture and aggressive substance penetrating the surface as illustrated in Figure 4, it becomes evident that edges are exposed from more than one side, thus leading to concentration of the nasties and early deterioration starting at the corners - the so-called "corner effect".

The corner effect is an issue of particularly importance within precast segmental design. due to the tendency to adopt smaller dimensions and minimizing concrete covers.

Codes and standards as Design Basis

For the everyday buildings and normal structures the national codes and regulations will have defined society's service life requirements - often not explicitly but implicitly through the standards and codified design requirements.

However, it is often forgotten that complying strictly with the performance requirements stated in codes and standards will only provide the minimum quality and performance being acceptable to society, and the assumed service life is in general only of the order of 50 years for buildings and say 75 years (AASHTO) or optimistically 120 years (British Standard) for bridges.

In addition, the knowledge base and experience represented by codes and standards are at least 10 years and most often 30 or more years old. New achievements, products, and procedures will usually not be included in such documents and may even be considered not acceptable, regardless of the level of confirming documentation available.

For many special structures additional requirements would be required if truly long-term performance and service life of the structures are needed.

This aspect is often completely overlooked by owners and clients, on the one hand side demanding, say 100 or 120 years service life, and on the other hand side insisting on a design in strict compliance with current codes and standards.

In this respect codes and standards may well become serious obstacles in achieving reliable long service lives and ensuring limited demand for future maintenance.

In such cases a project specific Design Basis shall be developed as one of the first design activities. This Design Basis shall include possible additions and modifications to the usual codes and standards where such needs have been identified based on present day understanding of durability technology.

Service life design and life cycle costing

The owner shall recognize that all structures - regardless of building material - will age and deteriorate with time. Hence, he must clarify his needs up front regarding design service life. When doing so, his decision has not only impacts on the short term cost of creating the structure but just as much on the long term costs for maintaining and repairing the structure to comply with his long term performance requirements.

The main issue when deciding upon a specific service life is to clarify the event, which will identify the end of the service life.

The requirement for a specific service life performance of a structure is closely associated with the short and long-term costs of this requirement. The owner must therefore acknowledge that he has to take decisions on both the service life and on the associated performance requirements, and he must accept both the short and the long-term costs - and savings - associated with his decisions.

Therefore, life cycle cost optimization - e.g. formulated as an optimization of the net present value - becomes an integral part of a service life design to be accepted by the owner.

In fact, without a rational evaluation of future costs in a comparable manner to rate alternative solutions, preferably as a life cycle costing (LCC), the service life design will most often not be an attractive design procedure. This is mainly because any serious savings in future maintenance costs will not be adequately considered to compensate for any increased initial construction costs, although such future savings may be very substantial.

Environmental loading

With respect to service life design one of the first, and most important decisions to be taken by the designer is the determination of the exposure conditions for which each member of a structure shall be designed, as the structural form itself has decisive influence on the future micro-climate to be expected. Different parts of a structure may be in different exposure conditions. Obvious examples are the submerged, the tidal, the splash and the atmospheric zones of a marine structure, but also different geographic orientations (north / south / east / west, or seaward / landward orientation) may be in different exposure classes. Even very local differences can be taken into account such as vertical faces, horizontal surfaces facing upward (risk of ponding) or facing downward (protected against wetting by rain), and the exposure of edges and outgoing corners, see Figure 5.

The environmental classifications indicated in codes can only be taken as a first guideline towards the general level of aggressivity prevailing for the individual structure.

In this respect a separation between the aggressivity towards concrete and the aggressivity towards reinforcement is a first valuable improvement already adopted in several codes, including the Eurocodes.

Materials and structural resistance

Having identified the environmental aggressivity the next step of the durability design is to identify the relevant degradation mechanisms. Mathematical models describing the time dependant degradation processes and the material resistances are needed. The big step forward towards performance related durability design is that these models enable the designer to evaluate the time-related changes in performance depending on the specific material and the environmental conditions.

Among the deterioration mechanisms relevant for concrete structures chloride induced reinforcement corrosion is by far the most serious problem, particularly in the hot humid and saline environments around the world, like in Florida, California as well as in the Persian Gulf [3-4] and in the Far East, just as in regions with extensive use of de-icing salts.

Deterioration mechanisms

The two-phase diagram illustrated in Figure 6 may model the development in time of nearly all types of deterioration mechanisms of concrete structures.

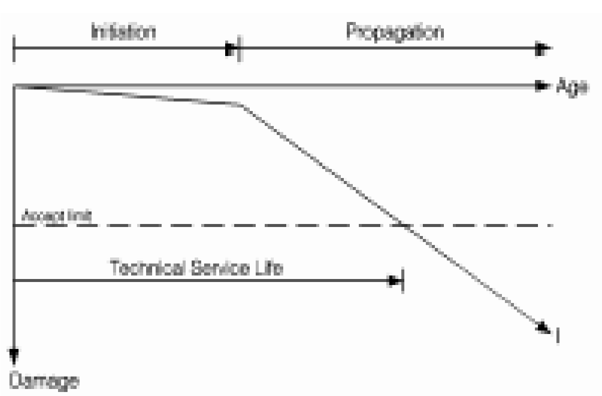

Figure 6: Service life of concrete structures. A two-phase time related modelling of deterioration. [Tuutti model (1982)]

The two phases of deterioration are the following:

> ***The initiation phase.*** During this phase no noticeable weakening of the material or the function of the structure occurs, but the aggressive media overcomes some inherent protective barrier. Carbonation, chloride penetration and sulphate accumulation - the latter two accelerated by cyclic wetting and drying - are examples of such mechanisms determining the duration of the initiation period.

> ***The propagation phase.*** During this phase an active deterioration develops and a loss of function is observed. A number of deterioration mechanisms develop at an increasing rate with time. Reinforcement corrosion is one such important example of propagating deterioration. The propagation phase may be divided into several events.

Figure 7 shows in principle the performance of a concrete structure with respect to reinforcement corrosion and related events. In general, points 1 and 2 represent events related to the serviceability of the structure, point 3 is related to both serviceability and ultimate limit states, and point 4 represents collapse of the structure.

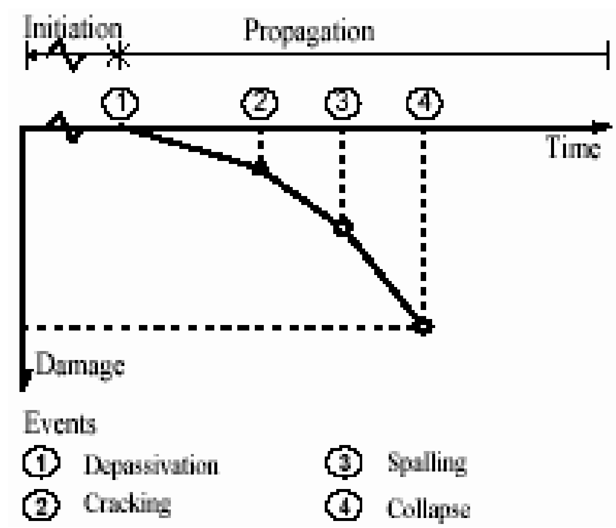

Figure 7: Events related to the service life, and detailing of the propagation phase.

Effect of Temperature

The temperature level is decisive for the rate of transporting aggressive substance into and within concrete. Therefore, the temperature is a decisive factor regarding the rate of deterioration of concrete structures. Chemical and electro-chemical reactions are accelerated by increases in temperature.

A simple rule-of-thumb says that an increase in temperature of 10 ^{o}C causes a doubling of the rate of reaction.

This factor alone makes hot humid tropical environments considerably more aggressive than temperate climates. The effect of temperature can be clearly demonstrated by comparing the damages in the picture in Figure 8 with the damage in the picture in Figure 9. In the former case the average yearly temperature is approximately 30 °C higher than in the latter case, which would lead to a 2 x 2 x 2 = 8 times faster deterioration in the Gulf compared to the rate of deterioration in the Nordic Countries. The pictures are clear documentation of this dependency on the temperature.

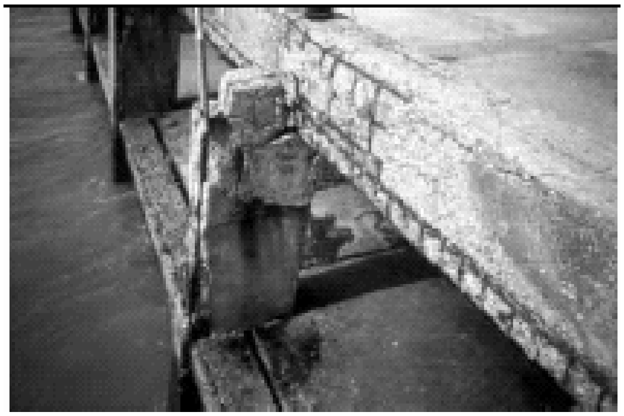

Figure 8: Reinforced concrete jetty in the Gulf exhibiting extensive chloride induced corrosion damage with delamination already after 2-3 years, and having reached a stage of failure and collapse after 7.5 years when this picture was taken.

Figure 9: Bridge piers exposed to a temperate Nordic marine environment. Extensive damage in the splash zone after 18 years due to chloride induced reinforcement corrosion.

Concrete resistance to chloride ingress

To focus on the main principles of modern service life design the calculations have been simplified by defining the nominal service life of new structures to be equal to the initiation period. This means that the time for the chlorides to reach the reinforcement and induce depassivation and initiate corrosion is equal to the nominal design service life.

The initiation phase ends when the chloride concentration at the reinforcement reaches a critical threshold value initiating corrosion. Carbonation of concrete can be treated in a similar manner.

Depassivation does not necessarily represent an undesirable state, as illustrated in Figure 6. However, this event must have occurred before corrosion will begin.

DESIGN

Structural design versus durability design

When designing a structure today the designer first defines the loads to be resisted. As these loads usually vary, he applies some safety factors, to be on the safe side. These factored loads must then be resisted by the structure through selecting a combination of structural systems, element geometry, material types and materials' strengths.

When it comes to durability design the situation is entirely different. It seems to be acceptable without question to use a grossly over-simplistic approach. The codes provide only qualitative definitions of exposure and they fail to define the design life in relation to durability. In particular, they fail to define and quantify the durability limit states that must be exceeded for the design life to be ended.

Previous approaches fail to recognize that, in relation to durability, it is not the properties of the materials or components alone that define performance, but the condition of the structure in its environment as a whole, and its individual need for intervention. This performance can be defined by functional requirements such as fitness-for-purpose, which includes issues such as deflections, cracks and spalling, vibrations, aesthetics and structural integrity.

Service life requirement vs. design service life

A service life requirement for the design of a new structure must be formulated in a format which can be operational as part of the design basis. This means that usually simplifications must be adopted in the design, thus leading to a Design Service Life, which may differ from the required service life. Often the design service life may be shorter than the real service life. The reason is that the transition between the initiation phase and the propagation phase often is more precisely defined than some degree of acceptable deterioration.

This can be exemplified as follows:

> The service life requirement shall be quantified - say 100 years! - or why not 150 or 200 years?
>
> The end of the service life must be defined also in a quantified and verifiable manner. The end of the design service life may then be defined by, say corrosion initiation with respect to reinforcement corrosion, or crack initiation due to either alkali aggregate reactions or due to bulk freezing or achieving exposed aggregates due to sulphate or acid attack, etc.
>
> The level of reliability of the design service life shall also be defined as precisely as possible.
>
> The prior approved type, detailing and frequency of maintenance shall be adopted as this may have a decisive influence on the solutions chosen to achieve the design service life. Examples may be a dependence on surface coatings and the frequency of replacing such coatings, or adopting cathodic protection and the frequency of its maintenance, or selecting corrosion resistant reinforcement in a corrosive environment and thus accept long intervals between inspections, etc.

The real service life may be different, and longer than the design service life. Firstly, there will be a certain part of the propagation phase which can be added to the design service life if this is based on the initiation phase. Secondly, any decision of changes in the type, detailing and frequency of future maintenance may prolong the real service life, in principle close to indefinitely depending on the costs being acceptable to the owner at that time.

For an existing structure, as well as for a repaired structure similar quantified and verifiable criteria may be defined, and the residual service life evaluated or designed.

From the above it becomes clear that in this process of developing a rational design for service life the owner must be directly involved, and he shall take a number of decisive decisions. This involves as well the short- and long-term cost implications of the decisions taken. This means that the LCC aspects also play a decisive role in the design process. If this cooperation and involvement of the owner is not achieved, a true service life design will not be realistic. Instead, the classical low-bid syndrome will prevail, and no improvement in quality, performance, and reliability will be achieved.

Service life design and a sustainable built environment

Environmental aspects together with the growing strive towards a sustainable built environment encompassing the structural concrete profession will be dependent on a successful overall adoption of a rational scientifically based design procedure for service life.

Design strategy

In principle two basically different design strategies for durability can be followed [6]:

A. Avoid the degradation threatening the structure due to the type and aggressivity of the environment.

B. Select an optimal material composition and structural detailing to resist, for a specified period of use, the degradation threatening the structure.

Modelling of deterioration processes is only relevant for Strategy B. An outline of a procedure for Design Strategy B could be the following:

> Start with the definition of the performance and service life criteria related to the environmental conditions to be expected.
>
> The next important element is the realistic modelling of the actions (environment) and the material resistance against these actions.
>
> Based upon the performance criteria, performance tests 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 important, the in situ quality.
>
> From this approach the design procedure can be established.

Strategy A and Strategy B can of course be combined within the same structure but for different part with different degrees of exposure (foundations, outdoor exposed parts, indoor protected parts, etc).

SERVICE LIFE DESIGN, STRATEGY A

Example: Segmental bridge (pier) at Progreso, Mexico

One new issue has evolved during the last few years, which may influence the broad - but not always successful - application of HPC (see later in this paper). This is related to the increased availability and competitiveness of non-corrodible reinforcement bars to avoid reinforcement corrosion in heavily chloride containing environments [1]. The non-corrodible reinforcement bars are made from either glass, aramid or carbon fibres, or galvanised reinforcement, epoxy coated reinforcement or - as described in detail in [1] - stainless steel reinforcement.

A very convincing Strategy A design for a long service life is presented by the 65 year old 2.2 km long concrete pier - or segmental bridge - out into the Mexican Gulf at Progreso in Mexico, reinforced with stainless steel reinforcement, see Figure 10.

Figure 10: The 2.2 km long pier into the Gulf of Mexico at Progreso, built 65 years ago and in full operation today carrying heavy container lorries back and forth from a new container terminal built further out at sea. The piers are reinforced with stainless steel reinforcement as described in [1].

The arch spans are simply supported on the pile heads and have been cast span by span in situ. By selecting this form, the arches could be made unreinforced - benefiting from the well-known arch action just as the ancient long-term durable structures illustrated in Figures 1 and 2.

Only the pile heads have been reinforced, and this was with smooth stainless steel reinforcement with a grade corresponding to today's AISI Grade 304, a so-called medium grade stainless steel.

Nevertheless, no corrosion has taken place within the structure, despite the harsh environment and poor quality local aggregates, which had to be used in this construction. The chloride levels, at the surface of the reinforcement were more than 20 times the traditionally assumed corrosion threshold level.

The Progreso Pier is the best example of a reliable long-term service record using stainless steel reinforcement in a highly corrosive environment.

Figure 11: 65 year old stainless steel reinforced pier at Progreso in Mexico still fully intact without maintenance whereas the remains in the foreground is what is left of an only about 30 year old pier reinforced with ordinary black steel reinforcement, see Figure12, [17].

Figure 12: Close-up of the land based remaining parts of the “new” pier of which part of the remaining piles are shown in Figure 11. This pier was reinforced with ordinary black steel reinforcement and lasted only about 10 years.

A newer, only 35 years old parallel pier has already perished due to reinforcement corrosion of the ordinary carbon steel reinforcement used in this structure, as seen in the foreground of Figure 11 and the remaining parts on land is shown in Figure 12, [17].

The old Progresso Segmental Bridge (Pier) into the Gulf of Mexico is in full use today - it now also serves a new container terminal and tourist cruise ship port facility built further out into the Gulf. Daly it provides the only link for all visiting Caribbean cruise ship tourists and for all the heavy container loaded trucks bringing goods back and forth between the port and container terminal, and the shore - and the Progreso Segmental Bridge has never had any need for structural repairs or strengthening.

This bridge has now served more than half the 100 year service life so many other concrete bridges have failed to reach - and this without structural maintenance.

SERVICE LIFE DESIGN, STRATEGY B

Example: The Great Belt Link, Denmark

The approach of the service life design following strategy B is to select intelligently an appropriate number and types of co-operating measures to ensure the required service life.

This is considered a multi-stage protection design strategy, or a multi-barrier approach [3-4]:

1. Identify the type and aggressivity of the environment in which the structure shall operate.
2. Forecast the possible movement and accumulation of the aggressive substance.
3. Determine which transport mechanisms govern (permeation, diffusion, capillary action) and which parameters control the mechanisms.
4. Select barriers that can co-operate in slowing down or prevent the transport and accumulation processes.

This was a so-called 1st-Generation service life design approach introduced first time for the 100 year service life design for the Great Belt Link in Denmark [4], see Figures 13 and 14.

The multi-stage protection comprised:

1. The annular grout.
2. 400mm dense high performance concrete.
3. Epoxy coated 3D welded reinforced cages using the fluidized bed dipping technique.
4. Cathodic protection installed some time in the future if ever needed.

Later this design has been evaluated using the reliability-based service life design - the so-called 2nd-Generation service life design methodology, see below, and currently it seems as if 150 years service life could be expected.

Figure 13: Great Belt Link, East Bridge. Denmark. Central span L=1416m. Design Life: 100 years.

Figure 14: Great Belt Link, East Tunnel. Denmark. Design Life: 100 years

Reliability-based service life design, Strategy B

The theories of probability and reliability in structural design have been developed and matured remarkably during the past five years. These theories have been transformed from the level of research and development to now being directly applicable and operational in practical engineering design. The methodology has been internationally recognized and used for many decades as basis for the structural safety design through the well-known semi-probabilistic LRFD (load-and-resistance-factor-design).

However, the factors and mechanisms governing the durability and performance of structures throughout their service life have only recently been developed in similar ways. This has among others been achieved through a European research project 1996-1999 "DuraCrete", "Probabilistic performance based durability design of concrete structures" [5].

This has allowed the treatment of transport and deterioration mechanisms to be modeled on a probabilistic level and introduced in the general service life design of concrete structures. Thus, design for safety and for durability can be performed using similar procedures. This opens the eyes of the owners now being able - or forced - to take decisions regarding his required long-term performance of their structures and then accept the consequences regarding maintenance and costs.

This new durability design methodology is based on the reliability theory as traditionally used in structural design. The purpose of a reliability analysis is to determine the probability of a given event, e.g. the event, which marks the end of the service life - the so-called 2nd Generation service life design methodology.

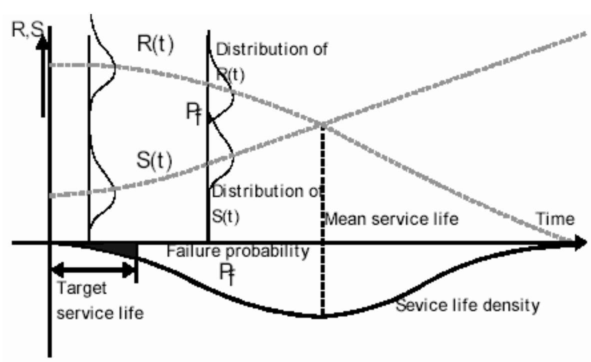

Figure 15: Probability of corrosion initiation and target service life [7].

This formal end - or design end - of service life may not necessarily be the real end of the useful life of the structure as illustrated in Figures 6 and 7. Depassivation of the reinforcement is such an example, and this stage is often used as the formal end of the design life for the design of a new structure, a service life limit state, as stated previously. In Figure 15 a schematic representation of the problem is shown [7]. The problem can be solved by well-known reliability methods.

The DuraCrete Design Guide [5] aims at obtaining a sufficient level of safety of the design service life with respect to the considered events.

Deterministic versus probabilistic service life design

The merits of the probabilistic approach to durability design are illustrated by the following example of a marine structure [1, 8]. Two different environments are considered, representing yearly average temperatures of 10 ^{o}C (exemplified by Northern Europe) and 30 ^{o}C (exemplified by the Gulf Countries) respectively. The design requirement is 50 years service life. For simplicity the service life is also in this example defined as the length of the initiation period, i.e. the time until corrosion initiation due to chloride ingress.

Figure 16 depicts the required concrete covers in each of the two environments, based on a traditional deterministic approach.

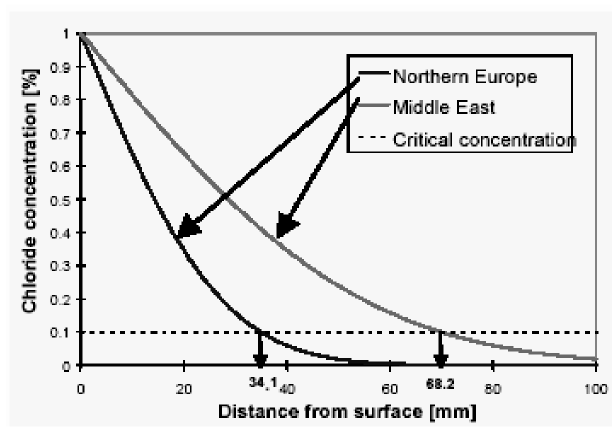

Figure 16: Deterministic approach. Required concrete cover to ensure 50 years service life and assuming a chloride threshold value of 0.1% by weight of concrete.

Figure 17 highlights the fact that the deterministic approach only provides a 50% probability of achieving the required 50 years corrosion free service life. This fact is often overlooked in usual design for durability. 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 Figure 17.

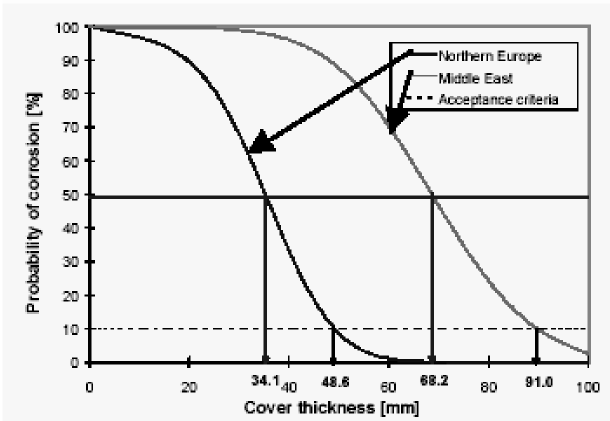

Figure 17: Probabilistic approach. The deterministic approach provides only 50% probability of avoiding corrosion at the age of 50 years. Accepting 10% probability of having corrosion initiated after 50 years results in considerably larger covers.

The deterministic approach used here is based on mean values of the governing parameters. In the probabilistic approach the relevant distribution functions with the mean values of these parameters, and their known or assumed uncertainties (coefficients of variation), are used.

The North American service life design program LIFE 365 is in principle similar to the DuraCrete design methodology, but as known today it is a deterministic design, thus not providing an overview of the uncertainties associated with such service life designs.

The direct service life calculation following the full probabilistic design will mainly be used for large special structures with a required particularly long service life.

Two current examples are presented in the following.

Examples: Major sea-based road link on bridges and in immersed tunnel

The design of a major sea based road link in Eastern Asia comprises a set of cable stayed bridges with approach spans, and an immersed tunnel, Figure 18.

Bridges and the tunnel should be designed for a 100 years service life. A project specific Design Basis was developed as an initial part of the design, and details agreed with the client and owner. Chloride induced reinforcement corrosion was identified as the governing deterioration mechanism in the design for service life, as all other potential deterioration mechanism were solved through a Strategy A approach (avoidance of deterioration).

Figure 18: Sketch of road link with bridges and an immersed tunnel designed for 100 years service life using the full probabilistic service life design methodology of DuraCrete, [5].

The following operational basis for the service life design was adopted:

a. The design life is 100 years.

b. The initiation period represents the design life. Thus corrosion initiation is defined as the nominal end of service life.

c. A 90% probability of not having any corrosion initiated before the 100 years has passed, corresponding to 10% probability of premature corrosion initiation. Or, in other words, adopt the corresponding reliability index (β = 1.3) as design basis. (The Eurocode, EC 1, indicates a value for β_{SLS} = 1.5 - 1.8 (6.7% - 3.6% probability of corrosion initiation).

d. Design the minimum concrete cover using the DuraCrete methodology [5]. This will provide the minimum cover complying with the above design criteria.

The surface chloride concentration being generated by the local marine environment is the force driving chlorides into the concrete and towards the reinforcement. This value was determined based on extensive data from similar type exposures.

Determination of the design quality of the concrete with respect to its chloride penetrability is the basic protective parameter against premature corrosion initiation and shall be determined at the initial stage together with the design compressive and tensile strength needed for the structural design.

The following parameters are therefore decisive for the design for service life [5]:

a. The design surface chloride concentration (Cl^-_s).

b. The background chloride concentration foreseen in the concrete mix, (Cl^-_0).

c. The chloride diffusivity (D_{Cl^-}).

d. The critical chloride concentration (Cl^-_{cr}) triggering corrosion on the reinforcement (the threshold value).

e. The ageing factor (α) representing the ability of the concrete to develop an increased denseness with time, represented by a decreasing diffusion coefficient with increasing age.

Bridges

The input to the service life design:

1. Identification and quantification of the environmental exposure.

   XS 1: Atmospheric zone (above level +8m).

   XS 2: Submerged zone (below normal low water level).

   XS 3: Tidal and splash zone (between normal low water level and level +8m).

2. Determination of the design quality of the concrete.

a. In the atmospheric zones

   i. Cl^-_s = 2.0% by weight of binder

   ii. Cl^-_0 = 0.1% by weight of binder

   iii. $D_{Cl-} = 2x10^{-12}\ m^2/s$ (= 63 mm^2/year)

   iv. Cl^-_{cr} = 0.9% by weight of binder

   v. α = 0.40

b. In the submerged zone

   i. Cl^-_s = 2.5% by weight of binder

   ii. Cl^-_0 = 0.1% by weight of binder

   iii. $D_{Cl-} = 2x10^{-12}\ m^2/s$ (= 63 mm^2/year)

   iv. Cl^-_{cr} = 2.2% by weight of binder

   v. α = 0.30

c. In the tidal and splash zones

   i. Cl^-_s = 4.0% by weight of binder

   ii. Cl^-_0 = 0.1% by weight of binder

   iii. $D_{Cl-} = 2x10^{-12}\ m^2/s$ (= 63 mm^2/year)

   iv. Cl^-_{cr} = 0.7% by weight of binder

   v. α = 0.40

For each parameter the distribution function is as determined in the DuraCrete procedure [5], and the coefficient of variation is based on data from practice.

3. Design the minimum concrete cover using the DuraCrete methodology.

a. In the atmospheric zone:

Minimum concrete cover: c = 38mm, (β = 1.30)
c = 40mm, (β = 1.50)
c = 44mm, (β = 1.80)

b. In the submerged zone:

Minimum concrete cover: c = 20mm, (β = 1.30)
c = 22mm, (β = 1.50)
c = 25mm, (β = 1.80)

c. In the tidal and splash zone, see Figure 19:

Minimum concrete cover: c = 59mm, (β = 1.30)
c = 63mm, (β = 1.50)
c = 68mm, (β = 1.80)

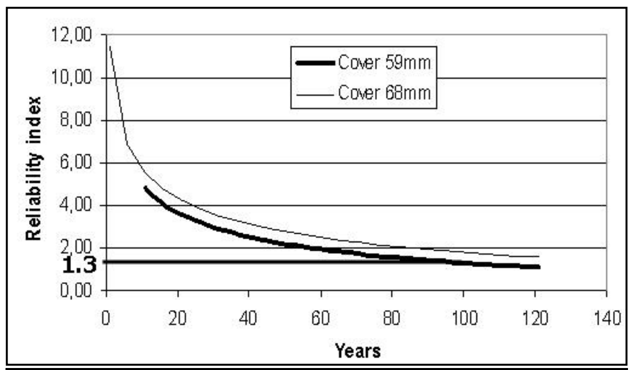

Figure 19: Development with time of the reliability index for the tidal and splash zone of the bridge.

4. Design crack width

Provide sufficient amount and distribution of reinforcement to ensure well distributed tensile and flexural cracks. Cracking of loaded reinforced concrete members is inevitable and is a natural phenomenon of concrete structures. However, the crack width as developed on the concrete surface has limited or no influence on the time to depassivation and on the rate of possible corrosion, except on horizontal surfaces with ponding salt-containing water (seawater). However, crack widths shall not be left uncontrolled for several reasons. Therefore, a sufficient amount of minimum reinforcement shall be provided to ensure a good distribution of cracks and that yielding of the reinforcement does not occur.

Two situations shall be considered:

1. Cracking caused by imposed deformations.

2. Cracking caused by imposed loadings. The required minimum area of reinforcement is calculated as the maximum of either the reinforcement ensuring that cracking does not lead to sudden failure, or the reinforcement which would be able to ensure the development of a stabilized crack pattern.

The final verification of crack width shall be based on the final total area of reinforcement and the SLS-based steel stresses, but still using the minimum concrete cover. The maximum acceptable crack width thus calculated shall be within 0.20mm.

Immersed tunnel

For the immersed tunnel segments a similar calculation was made, and 4 alternative chloride diffusion coefficients (concrete penetrability) were analyzed:

a. D_{Cl-} (a) = $2x10^{-12}$ m^2/s (= 63 mm^2/year)

b. D_{Cl-} (b) = $3x10^{-12}$ m^2/s (= 94 mm^2/year)

c. D_{Cl-} (c) = $4x10^{-12}$ m^2/s (= 125 mm^2/year)

d. D_{Cl-} (d) = $5x10^{-12}$ m^2/s (= 156 mm^2/year)

For the inside surfaces of the tunnel this led to the theoretical concrete covers in Figure 20.

	D_{Cl}- (a)	D_{Cl}- (b)	D_{Cl}l- (c)	D_{Cl}- (d)
β = 1.3	45mm	55mm	**63** mm	70mm
β = 1.5	47mm	57mm	66mm	75mm
β = 1.8	51mm	63mm	73mm	81mm

Figure 20: Concrete cover on the tunnel inside depending on the chosen chloride diffusion coefficient, D_{Cl}-, and on the reliability index, β.

For the outside - seawater exposed - surfaces of the tunnel this led to the theoretical concrete covers in Figure 21.

	DCl- (a)	DCl- (b)	DCl- (c)	DCl- (d)
β = 1.3	20mm	24mm	**27**mm	30mm
β = 1.5	22mm	26mm	30mm	33mm
β = 1.8	25mm	30mm	34mm	37mm

Figure 21: Concrete cover on the tunnel outside, directly exposed to seawater, depending on the chosen chloride diffusion coefficient, D_{Cl}-, and on the adopted reliability index, β.

From Figures 20 and 21 it turns out that the required concrete cover on the inside surfaces of the tunnel shall have a considerably larger cover to ensure the 100 years design life, than the surface directly exposed to the seawater. This has caused some confusion among tunnel designers, but understanding the transport and deterioration mechanism leads naturally to this conclusion.

In practice the designer shall choose a convenient cover, and for purely practical reasons a general cover of 75mm was chosen on all surfaces in this case. The added dead load was also beneficial by slightly reducing the buoyancy of the segments.

The adopted average flexural crack widths for final design were related to the difference in theoretical cover thickness:

Surfaces with hydrostatic water pressure:

(cover = 27mm): 0.20mm

Surfaces with atmospheric exposure:

(cover = 63mm): 0.30mm

***fib* Model Code for Service Life Design (MC-SLD)**

Currently ***fib*** (The International Federation for Structural Concrete) Commission 5 "Structural Service Life Aspects" has two active Task Groups developing guidance documents on service life design.

Task Group 5.6 "Model Code for Service Life Design" developing a code-type document.

Task Group 5.7 "Service Life Design Guide" developing a designers guide to design for durability and service life respecting the requirements of the MC-SLD.

The MC-SLD (TG 5.6) is divided into five chapters, Figure 22:

1. Introduction/General
2. Basis of Design
3. Verification of Service Life Design
4. Execution and Quality Control
5. Maintenance and Condition Control

The flow chart in Figure 22 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 has identified four different levels of sophistication in the performance based design:

1. A ***full probabilistic design***, corresponding to design strategy B.

2. A ***partial factor design*** (semi-probabilistic), corresponding also to design strategy B, but with factors calibrated with level 1 above.

3. A ***Deem-to-Satisfy design*** corresponding to methods in current codes, but also corresponding to design strategy B and with requirements calibrated, to the extent possible, with level 1 above.

4. ***Avoidance of deterioration***, corresponding to design strategy A.

The Design Guide (TG 5.7) will provide background descriptions and guidance assisting the designer in selecting the appropriate design strategy in individual cases. This will include detailed descriptions on how the owner or client shall be part of the design process. In particular the owner shall take decisions defining the design service life verification method and fixing the acceptance criteria so this becomes operational in the design process, as described also in this paper.

Currently ***fib*** is working on a full revision of the CEB/FIP Model Code 1990. The draft is expected to be presented at the fib congress in Naples in June 2006. The 1990 Model Code is mainly dealing with structural design of concrete structures. The present work aims at including aspects of execution, materials and conservation in a consistent manner, as well as service life design as summarized in this paper.

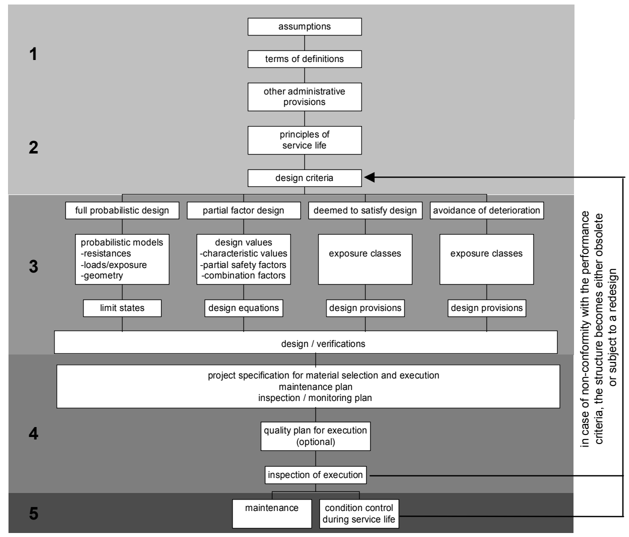

Figure 22: Flow chart of MC-SLD and the related chapters 1-5. Chapter 3 identifies the four different levels of sophistication adopted in the MC-SLD

The MC-SLD prepared by Task Group 5.6 and the Design Guide prepared by Task Group 5.7 are both intended to be published as a separate ***fib*** bulletin and in part serve as input to the full ***fib*** New Model Code concerning service life aspects, and as a supportive guidance document.

MATERIALS

High performance concrete (HPC)

The continuous demand for increased strength and improved durability of concrete structures has led to the development of HPC. This development has had three main objectives in mind:

1. Protect the reinforcement against corrosion; in particular provide protection against ingress of chlorides by creating dense impermeable concrete in the cover zone with very low penetrability of aggressive ions such as chlorides, sulphates and of CO_2.

2. Resist deterioration of the concrete itself when exposed to the aggressiveness of the environment such as sulphates, seawater and other chemical attacks, as well as resist freeze/thaw attack.

3. Provide adequately high strength to fulfil the structural requirements.

HPC for normal type structures will usually have a relatively high cementitious binder content (blended cements), a low water/cement ratio, say in the range of 0.35 - 0.40, and a high contents of plasticizer and superplasticizer. Such concrete can conveniently be used for bridges, marine works, offshore structures, high rise buildings etc. where the strength requirements usually remain within the range of say 50 - 80 MPa.

One drawback has been that these more refined concrete mixes become more sensitive towards the actual handling during execution. They set high demands on the competence, experience and workmanship of the workforce. To varying degrees these concretes differ from the long term known types of structural concrete.

A typical HPC-mix is illustrated in Figure 23.

One drawback has been that these more refined concrete mixes become more sensitive towards the actual handling during execution.

The increased sensitivity of HPC compared to normal concrete relates to the mixing, transport, placing, compaction and curing processes. HPC requires an experienced and competent workforce and high quality workmanship to achieve the potential benefits, but this is not always available on site.

Figure 23: A typical mix for a HPC with a maximum water-binder ratio of 0.35. A three-powder binder combination is used, comprising OPC, flyash and silica fume. Such a mix is extremely sensitive to correct execution and curing. All additives and admixtures have to be mixed into the bulk of the concrete, but mostly only needed in the cover zone - A noticeable waste in all aspects. (Photo: N. Thaulow).

HPC concrete in the "better" end of the strength scale can be very difficult to place and compact, and the risk of honeycombs, particularly in the cover concrete, increases. In addition, the quality and efficiency of compaction is extremely dependent on the individual person handling the vibrator. Hence, the better the concrete mix, from a durability point of view, the greater the risk of having inferior or bad execution leading to reduced quality in the final structure. This fact is seldom respected on site. This inconsistency is due to the dominating influence of the execution process on the final performance of the structure.

To varying degrees these concretes differ from the long-term known types of structural concrete through the following [2]:

The dosing and mixing become more complicated and sensitive, even with respect to timing and sequence of adding the different ingredients.

The placing requires special methods and routines as such mixes can be rather tixotropic (cohesive) and sticky.

Compaction is more demanding as the vibration shall be more intensive and requires more vibration energy.

The denseness of the concrete, when correctly placed and compacted, minimises bleeding, and the available water is so limited, that protection against evaporation shall be introduced promptly following levelling and trowelling of horizontal surfaces exposed to drying, in order to avoid plastic shrinkage cracking. Normal curing compounds are usually not sufficiently effective.

The high content of cementitious material will most often generate more heat than normal concrete, thus increasing the risk of thermal cracking. However, this depends on the cementitious material used in the individual cases, where e.g. slag cement has reduced risk of thermal cracking due to a slow rate of hydration.

The autogenous, or chemical shrinkage, is more pronounced due to the low water/cement ratio, and this sets much stronger demands on controlling the temperature differences in hardening concrete if thermal cracking shall be avoided. The limiting temperature differences generally accepted may have to be halved, and in extreme cases reduced to 1/3 for HPC to avoid unacceptable thermal cracking.

HPC requires air entrainment to be frost resistant according to the generally adopted (rather severe) freeze-thaw tests. However, such mixes with high contents of superplasticizer are usually difficult to air entrain, and the air may easily disappear during the compaction due to the increased vibration energy needed. Hence, HPC may be more sensitive to freeze-thaw actions, i.e. less frost resistant than normal type structural concrete.

The sensitive elements of HPC listed above are not necessarily valid for all types and uses of such concretes. It is however necessary to have these potential problems in mind when designing structures based on the application of HPC, taking the realistically available competence of the local workforce into account when making the selections.

One new issue has evolved during the last few years, which may influence the broad - but not always successful - application of HPC. This is related to the increased availability and competitiveness of non-corrodible reinforcement bars to avoid reinforcement corrosion in heavily chloride containing environments. SSR, as described in detail in this paper, is currently the most convincing solution to the reinforcement corrosion problems, see Figures 10 - 12, and Figures 24 - 28, [1].

Potential environmental and sustainability related drawbacks using HPC

The main driving force in introducing HPC has been objective no. 1 above, the protection of the reinforcement against corrosion from ingressing aggressive substance, particularly chlorides. However, the availability of stainless steel reinforcement as described later in this paper will in nearly all cases solve the problem thus reducing the need for HPC with its possible execution related drawbacks.

Solving the objective no. 1 above will require a highly alkaline concrete mix and a dense impermeable concrete in the cover protecting the reinforcement. This leads to the following:

The more pure Ordinary Portland Cement in the mix, the more calcium hydroxide will be available in the hardened concrete to provide and sustain the high alkalinity ensuring a high threshold level against corrosion initiation.

Obtaining the required strength is generally not a problem, only seldom will it be possible to utilize the very high strengths which can be obtained adding large quantities of pozzolanic admixtures, using very low water/binder ratios, and adding a mixture of chemical additives to ensure workability.

Having to add these costly additives and admixtures to ensure a dense and impermeable concrete cover has the general consequence that these have to be added to the whole bulk of the concrete - but realistically speaking they are to a large part wasted, as they are only needed in the cover zone.

These issues seem greatly overlooked by the design and construction industry - but they have adverse effects on environmental preservation and are counterproductive to a sustainable construction environment.

The nature of deterioration of concrete structures also highlights the fact that high performance concrete does not necessarily provide high performance concrete structures, as detailed in [9].

This conflict - and its practical consequences - is not easily understood by the classical research communities, but painfully experienced by many contractors and structures´ owners/operators.

Corrosion resistant reinforcement

Corrosion resistant reinforcement is the most recent measure to eliminate the risk of reinforcement corrosion, also in the most corrosive chloride containing environments.

The adoption of corrosion resistant steel reinforcement (CRSR) is the simplest solution as this not only solves the corrosion problem in a fool-proof manner, but also leaves the site activities nearly unchanged. This is particularly advantageous as serious changes to the site operations always introduce difficulties and uncertainties.

Using non-metallic reinforcement may solve the corrosion problem, but most such reinforcements cannot be adjusted on site, some may be brittle (e.g. carbon fiber bars) and sensitive to impacts from a vibrator, walking on the reinforcement should usually be avoided, and most disturbing from a practical point of view is that all these types of reinforcements are light and shall be anchored to the formwork to avoid them floating.

Normal reinforcement - also termed mild steel, black steel, or carbon steel reinforcement - is very efficiently protected against corrosion when cast into a good quality alkaline and chloride free concrete.

This is the well-known unique benefit of using reinforced concrete structures in building and construction. Only when carbonation reaches the level of the reinforcement, or more seriously, when chlorides in sufficient quantity reach the surface of the reinforcement will the passivating effect be eliminated and corrosion may start.

Particularly serious are the situations where the initial concrete mix has been polluted with chlorides from the aggregates, the mixing water or chloride-based accelerators.

Recent year's developments within the area of non-corrodible or corrosion protected reinforcement for concrete structures are opening promising new doors in the fight against reinforcement corrosion. The following products with different degrees of resistance against corrosion are available:

Stainless steel reinforcement.

Epoxy coated reinforcement.

Hot dip galvanized reinforcement (zinc coating). This application is very limited, but may be a fully viable protection in concrete exposed to carbonation. In general zinc coating is not considered adequate - or cost-effective - for structures exposed to chlorides. Zinc coating is not discussed further in this paper.

Non-metallic reinforcing bars such as reinforcing bars from glass fibers, aramid fibers, or carbon fibers. The non-metallic reinforcing bars will probably for many more years only have limited applicability due to the major differences needed when constructing on site. They may have a potential within the pre-casting industry, and is not discussed further in this paper.

Stainless steel reinforcement (SSR)

The use of SSR in zones being exposed to high chloride concentrations is considered a highly reliable solution following design Strategy A, as described previously. This can ensure a very long problem-free service life in that part of the structure, provided the concrete itself is made sufficiently resistant to avoid other types of deterioration such as alkali-aggregate reactions, sulphate attack, or salt scaling.

In addition, there are regions where the chloride contamination is so widespread that all aggregates and mixing water is more or less chloride contaminated. Sometimes a 10 - 20 year service life has become the accepted norm in such regions, or continued repair works have been the accepted solution. Using SSR may often solve this problem completely.

Used selectively in the most exposed zones of the structure the increased costs per kg SSR compared to the costs of normal steel will most often have only marginal or negligible effect on the overall initial construction costs. In addition the service life costs will be reduced considerably due to savings in future repair and maintenance [11-12].

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 of reinforced concrete structures remain practically unchanged, a fact that is very attractive to the traditionally very conservative construction industry.

Of particular importance is the often-overlooked fact, that SSR can be coupled with normal mild steel reinforcement (carbon steel) without causing galvanic corrosion [11 - 14]. The reason is that the two types of steels reach nearly the same electro-chemical potentials when cast into concrete. This leads to the possibility to use SSR only in those parts of the structure where this is considered necessary, and then reinforce the remaining parts with ordinary mild steel reinforcement. Such highly exposed zones needing SSR could be splash zones of marine structures, foundations in contaminated soils, lower parts of columns above ground, balconies, etc.

The Progreso Pier in Mexico, see Figures 10 - 12, is an ideal and a remarkable example of the long-term reliable durability performance using stainless steel reinforcement (SSR).

This direct solution to the corrosion problem has during the past few years gained new strong momentum, particularly as it has been documented that SSR and black steel can be mixed, and be in metallic contact without causing galvanic corrosion. Hence, the SSR need only be used in the directly exposed surfaces, as illustrated in the examples below.

As it is recognized that the most serious durability problem for concrete structures is reinforcement corrosion it becomes evident that [1]:

> *"The reliable and readily availability of stainless steel reinforcement at reasonable and foreseeable prices may change - or revolutionize - major parts of the building sector in aggressive environments, simply by solving the corrosion problem."*

The main reason is also the added value which follows from the possibility of accepting the use of locally available materials, even with chloride contamination, and also accepting the qualifications of the local workforce as it is, and still produce highly durable, robust, and long lasting reinforced concrete structures with need for only minimal maintenance - when designed intelligently.

Examples of the practical use of SSR

The Stonecutters Bridge in Hong Kong, Figure 24, is to be a remarkable landmark for the Hong Kong Harbor. The architectural pylons are heavily reinforced with multi-layer of Ø 50mm bars. As corrosion resistance enhancement the outer layer of reinforcement is SSR, the remaining reinforcement is ordinary black steel.

Figure 24: Stonecutters Bridge, Hong Kong. The outer layer of reinforcement in the pylons is Stainless steel reinforcement; the remaining reinforcement is ordinary black steel reinforcement. Construction has just started. Design: COWI A/S in Joint Venture with Arup. (Model photo).

Similarly, the Shenzhen Corridor Bridge (design: Arup) linking Hong Kong with the mainland China has also adopted the same approach; see the close-up of the two-layer solution in Figure 25.

Figure 25: Illustration of the selective adoption of SSR in the outer layer of reinforcement in a highly corrosive environment. The reinforcement layers behind are ordinary black steel reinforcement.

Also within the building industry has the CRSR solution been adopted, as illustrated in a recent adoption of stainless steel reinforcement for a large new Building Complex currently under construction in the Persian Gulf region. SSR was introduced from somewhat below groundwater level to level +5m in the outermost structures directly exposed to the salty ground water and seawater spray of the Gulf waters. All remaining parts are reinforced with normal black carbon steel, see Figure 26.

At the same time the reinforced concrete seawall protecting both the existing important buildings and the adjacent new Building Complex is being replaced using SSR throughout, see Figure 27.

Figure 26: Overview of the construction site of a seaside Building Complex. Stainless steel

reinforcement is used in the outer exposed structures from below ground water level up to level +5m.

Figure 27: Stainless steel reinforcement used in the precast structural members to replace a corrosion damaged seawall protecting the existing important buildings and the new Building Complex (Figure 26) from the Gulf.

Figure 28: Replacing edge beams and parapets on Danish bridges and crash barriers on the motorways exposed to de-icing salt. SSR is introduced to avoid future repairs as well as avoiding the serious delays and inconveniences to the users, an issue becoming more and more in focus with the growing awareness of the costly societal consequences of such delays.

Finally, an additional benefit of using SSR is the fact that stainless steel is a poorer cathode than carbon steel [13]. Therefore, SSR can be beneficial in those repair cases where ordinary carbon steel has corroded to such an extent that local replacement or added reinforcement is needed as part of a repair. In this way the traditional problem of new corrosion developing on the reinforcement adjacent to the repaired area (incipient anodes) - and initiated in part by the repair - can be reduced or fully avoided.

A current example of such replacements can be seen in the replacement of corrosion damaged bridge edge beams on Danish motor- and highway bridges and on crash barriers, using stainless steel in the edge beam and parapet replacements see Figure 28.

Epoxy coating of reinforcement

Epoxy coating of reinforcement has been used in North America since the mid 70'ies. This technology has now for many years been critically discussed.

The nearly unavoidable fine cracking occurring during bending, the pinholes occurring in the coating, and the inferior protective ability of the patch repaired zones and cut ends have led epoxy coated reinforcement to be a less attractive solution - mildly speaking. It also has the side effect that this technology will prevent the possible future use of cathodic protection, leaving no alternative but replacement of damaged members if corrosion develops. Currently, the coating industry is working on enhancing the technology, but still without the possibility to enhance the key source of uncertainty, namely the individual execution phases. Hence, the most acceptable application of epoxy coated reinforcement would probably be in the precasting industry - if used at all.

North American experience has thrown serious doubts on this approach, when following the traditional procedure of coating straight bars individually, then cutting them to length and bending them to the required shape, see Figure 29 and 30 from the Florida Keys. In several states epoxy coating is not allowed by the local Departments of Transport and in Ontario, Canada, SSR is taking over within bridge construction and bridge repairs.

The first public report on failing performance of epoxy coated bars was from January 10th 1992 [15], which concluded that the ***"Epoxy coated rebar technology is flawed"***.

This caused a major disturbance within the North American Continent due to pure commercial and biased reactions from the producers of epoxy coated reinforcement and their organizations. At that time the technology was in the process of spreading rapidly to Europe, the Middle East, mainly the Gulf Countries, and to some parts of the Far East.

Figure 29: Bridge on the Florida Keys. Critical corrosion of epoxy coated reinforcement

Figure 30: Bridge on the Florida Keys. Critical corrosion of epoxy coated reinforcement leading to splitting of the V formed supports now strengthened by prestressed bolts

Using epoxy coated single bars would be to ***"put all the eggs in one basket"*** as it was termed.

The North American experience with the traditional technology of epoxy coating together with additional testing and site investigations, among others in Ontario, Canada, [16], has lead to this technology not gaining foothold in Europe.

This technology is now slowly being phased out, also in the very corrosive environments of the Middle East and Gulf Countries, see Figures 31 - 33.

Figure 31: Seawall reinforced with epoxy coated reinforcement-heavily damaged by reinforcement corrosion after about 12 years

Figure 32: Close-up of the corroded epoxy coated reinforcement shown on Figure 31.

Figure 33: Details of the corroded epoxy coated reinforcement shown on Figure 31. The loose layer of epoxy on the corroding bar is clearly visible.

CONSTRUCTION

Interaction between design and execution

Already at the design stage possible means of construction shall be considered and fixed, as this will influence the durability design.

It is important that this interaction between the foreseen execution and the provision to provide durable structures is identified at an early stage of the design in order to optimize the design and prepare the structure for easy inspection and maintenance [18].

Robustness in design and construction

One of the main obligations of the conscious designer is to adapt the design to the conditions under which the structure is to be constructed, operated and maintained. A degree of robustness adopted in the design can be very advantageous to avoid that the structure becomes very sensitive to local variations in the qualities assumed in the design, such as variations in compaction and cover. Such build-in robustness can enhance the future performance and durability of the structure.

The execution stage

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 correct concrete quality and thickness of the concrete cover - ***it is as simple as that!***

The following two issues relate to the resistance of the concrete cover to ingress of aggressive substance from the environment:

The compaction, in order to ensure uniform, dense and strong covers.

The type and duration of curing, i.e. controlling moisture levels and temperature differences in order to limit or avoid early age cracking.

In addition, the following two recent developments are expected to obtain considerable influence on the quality and service life of future exposed structures:

Permeability controlled formwork liners (PFL).

Self compacting concrete (SCC).

Means of providing the required thickness of the concrete cover with spacers is an integral, but often neglected part of ensuring durable structures

Because the execution phase itself has dominating influence on the final quality and performance of the structure the on site supervision and quality control is an essential part of the construction process.

Prestressing and prestressed concrete structures pose specific execution problems not dealt with in this paper.

Compaction of concrete

Figure 34: Example of reinforcement detailing which does not respect the need for reliable casting and compaction of concrete. Particularly the quality of the concrete cover, the "skin", is in danger

Adequate compaction of the concrete in the cover may be difficult to achieve due to the limited space and the need for the cover concrete to be moved through the outer layer of reinforcement. This movement of the concrete may cause "sieving" of the concrete if the spacing of the reinforcing bars is small or the concrete is stiff. When designing the reinforcement layout the realistic ability to compact the concrete on site shall be respected. Figure 34 illustrates a situation where this has not been achieved.

The compaction by vibrators shall be achieved through vibrating the concrete inside the reinforcement cage. Experience has shown that vibrating directly in the cover concrete may lead to inferior compaction of the concrete, e.g. by leaving porous traces from the vibrator.

The problem requires particular attention in the case of dense high performance concretes with mineral admixtures due to the tixotropy of such low water/cement ratio concretes with high amounts of plasticizer and superplasticizer.

The use of external form vibrators to compact the concrete, and in particular the cover concrete, is also a questionable procedure. The reason being, that the entrapped air voids in the concrete tend to move towards the location of the vibrator and accumulate to form porous concrete. With correct use of poker vibrators such air bubbles are extracted from the concrete when slowly lifting the vibrator head out of the concrete.

Curing

Curing of the concrete is part of the hardening process, which ensures an optimal development of the fresh, newly cast concrete into a strong, impermeable and durable hardened concrete in the cover zone free from plastic shrinkage and thermal cracks. During this initial stage of the life of the concrete, it is necessary to:

1. Use an appropriate hardening process. Casting must be planned such that the required strength at the time of form stripping is achieved.

2. Ensure against damage from drying. Premature drying-out of the concrete surface should be avoided, as this may lead to large plastic shrinkage cracks as seen on Figure 35.

3. Ensure against damage through early freezing. The concrete must not freeze until a required minimum degree of hardening has been achieved.

4. Ensure against damage from thermal stresses. Differential movements due to thermal differences across the section or across a construction joint between hardened and newly cast concrete should not lead to cracks.

Figure 35: Plastic shrinkage cracks on bridge deck.

The increased sensitivity to too early drying out of some types of cement and concrete (composite and blended cements; chemical and mineral admixtures) has accentuated the need to develop simple and rational heat and moisture curing procedures. Several comprehensive tools are now available to design the appropriate types of moisture and heat curing.

According to experience with Ordinary Portland Cement concrete, it is recommended to stay within the following limits for temperature stresses:

A maximum of 20 ^{o}C temperature difference over the cross-section during cooling after stripping

A maximum of 10 - 15 ^{o}C difference across construction joints and for structures with greatly varying cross-sectional dimensions.

Concretes with very low water/cement ratio, such as high performance concrete, may be particularly sensitive to heat and temperature control during the hardening process, and values for temperature differences should be lower - and at times considerably lower - than indicated for Ordinary Portland cement concrete above.

The heat balance to be controlled is sensitive to changes in the selected level of insulation. In practice it is often necessary to decide at short notice whether to strip formwork or whether possible additional or reduced insulation of a hardening cross-section has to be made.

In short, good curing is needed to profit from a good concrete mix. Bad curing destroys an otherwise good concrete mix. And good curing cannot compensate for a bad concrete mix. All efforts to ensure an optimal heat and moisture curing may be in vain, if the initial quality of the concrete mix is inferior.

In practice temperature profiles can be calculated, and the whole curing process can be designed prior to casting the concrete.

Controlled Permeability Formwork Liner (PFL)

Recognising the dominant influence of the quality of the outer concrete skin to protect the concrete as well as the reinforcement against penetration of aggressive substance has lead to the development of a special permeable formwork liner, which can be either flexible and tissue-like, or stiff like a plastic board. Such liners are able to improve the quality of the outer few mm or cm of the concrete cover by controlling the water/cement content and enhancing the curing in this thin outer zone.

Self-compacting concrete (SCC)

The need for durable concrete in an aggressive environment leads automatically to concrete with optimised mix composition and a low water/cement ratio. Such concrete is difficult to compact and the risk of honeycombs, particularly in the cover concrete, increases. In addition, the quality and efficiency of compaction is extremely dependent on the individual person handling the vibrator.

Hence, the better the concrete mix is, from a durability point of view, the greater becomes the risk of having inferior or bad execution leading to reduced quality in the final structure. This inconsistency is due to the dominating influence of the execution process on the final quality and performance of the structure regardless of the good quality of the initial concrete mix.

The development of a concrete mix where the placing and compaction has minimal dependence on the available workmanship on site would therefore improve the true quality of the concrete in the final structure. This has been a main driving force in recent year's development of SCC.

With the aid of a range of chemical admixtures and optimal grading of the aggregates concrete with low water/cement ratio can be made to flow through complicated form geometry and around complex reinforcement layout, without segregation. The form can be filled and a uniform compaction without honeycombs can be achieved, also in the cover zone of the concrete, with no or only minimal additional contribution to the compaction and levelling of the concrete from the workforce on site.

The flowing concrete will exert an increased pressure on the form, which shall be taken carefully into account when designing the formwork. The form pressure may at times be close to the hydrostatic pressure, but with the concrete density of 2.4!

In addition, SCC will "escape" through any hole or slot in the formwork, see Figure 36, so the formwork for SCC shall be tight.

The use of SCC is also an environmentally friendly technology as the noise level from vibrators is nearly eliminated and the concrete workers need only minimal work with the vibrators, with all the adverse effects vibrating concrete has on the body, such as "white fingers".

Figure 36: Self Compacting Concrete, SCC. Left: uniformly filling the form by pouring the concrete into one end of the form. Suddenly the concrete stopped rising in the form. Right: The reason being that a small slot in the form let the SCC flow out of the form! Conclusion: Formwork for SCC shall be (water-) tight, and also stronger than usual to resist the increased form pressure.

Spacers

The minimum concrete cover specified for a design is usually the value used to design the expected service life based on assumptions regarding the penetration of de-passivating and corrosive substance to the reinforcement. Therefore this minimum value shall be ensured in the final structure by taking the relevant tolerances into account in the selection of type, dimension and spacing of spacers.

Spacers shall be designed according to c_{nom} and not to comply only with $c_{min.}$ This fact is often overlooked in practice!

Figure 37: Small concrete spacers with large spacing of spacers. Such spacers are not adequate to ensure a reliable concrete cover, as shown on

Figure 38: The small concrete spacers shown on Figure 37 have no stability and cannot be fixed adequately. In view of the importance of achieving the correct concrete cover, this situation is not acceptable.

Figure 39: A concrete box girder bridge built about 20 years ago. A view of the underside of the box 22m above the sea is shown on Figure 40.

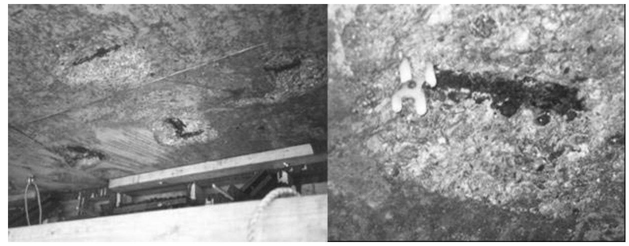

Figure 40: A systematic set of spalled concrete is clearly visible of the underside of the box girder of Figure 39, as seen to the left. The reason is documented on the close-up to the right: The use of plastic spacers. Plastic spacers are not compatible with concrete and should not be used in highly corrosive environments.

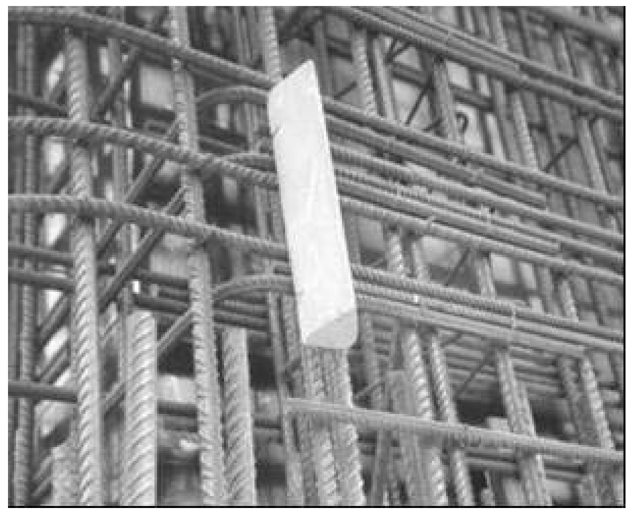

Figure 41: Solid and reliable type of concrete spacer with reliable fixing, and with strength to resist even very high concrete pressures, and still with no or only marginal imprint on the finished concrete surface.

In aggressive environments spacer material should preferably have good adhesion to the concrete. The spacer geometry and fastening shall ensure good and stable positioning on the reinforcement. Spacers with geometry and fixing as shown on Figure 37 should not be acceptable.

The reason why such tiny spacers as shown on Figure 37 should not be acceptable is clearly illustrated on Figure 38 from the same structure.

The spacer material shall have good bond to the concrete and shall have similar hygro-thermal deformation characteristics as concrete.

In this respect plastic spacers are not compatible with the surrounding concrete, in the sense that they have no direct adhesion and that they have different temperature coefficients (factor ten) than concrete, and furthermore age under exposure to air, sun and marine environment, see Figure 39 and Figure 40.

In aggressive environments high quality concrete spacers shall be the preferred option and it is important to ensure that the spacers are of the same high quality as required for the structural concrete itself. Hence, high quality concrete spacers with reliable fixing, as shown on Figure 41 shall be used in exposed areas.

THE HANDING-OVER SITUATION service life updating

The "Birth Certificate"

In order to document the fulfillment of the design specifications and verify the subsequent performance, the Quality Assurance documentation for design and execution should be enlarged to include information gathered during the operation and use of the structure.

An Operation and Maintenance Manual specific for each structure shall include such additional documentation. This Manual should be prepared by the Designer and shall include all information from the structural design and the construction being relevant for the future inspections and maintenance. This Operation and Maintenance Manual shall also include recommendations regarding type and frequency of future inspections and should highlight possible sensitive or critical parts of the structure which are assumed beforehand to need particular attention during use.

When the structure is handed over to the Owner, the initial Operation and Maintenance Manual will constitute a "***Birth Certificate***" of the structure [1]. Information from future inspections and all other relevant events such as accidental impacts are then filled in as they occur. Depending on the nature and contents of such future information the type, frequency and selected special areas of concern shall be revised or updated by the Owner following his needs at that time.

CONCLUSIONS

The complexity of designing well performing, low maintenance and long-lived concrete structures has been presented in this paper. It highlights the multidisciplinary set of problems to be solved by the designer in order to ensure truly long service life with minimal maintenance of concrete structures.

However, the descriptions of the individual measures needed to achieve this goal have also been shown to be relatively simple - in most cases surprisingly simple - and using well-known methods, materials and technology. The real challenge is in fact twofold:

1. The owner shall formulate his performance requirements to the structure in a format that can be translated into a quantified design basis, taking the long term effects or consequences, including acceptable needs for maintenance, into account technically as well as in the economic evaluations.

2. The designer and the contractor need to combine readily available knowledge from design, construction, materials technology and deterioration mechanisms into an integral solution adapted to the individual structure in its foreseen environment.

Our daily terminology using "durable concrete" and discussing "High Performance Concrete" is misleading. In fact, no one cares very much about high performance concrete (except maybe the cement, concrete and admixture producers) but what we all need is ***High Performance Concrete Structures***, and that is a completely different challenge, as highlighted in this paper.

Figure 42 is an excellent confirmation of this statement; it is a recent picture from the inside of the 2000 years old concrete dome of Pantheon shown in Figure 3. Could this long service life be due to the fact that this dome is not reinforced with corrodible reinforcement? This is in fact an unreinforced structure!

A contemporary example of a similar solution, though of a different level of exclusivity, could be the capping beam of the seawall illustrated in Figure 43. The heavily reinforced capping beam was so damaged by reinforcement corrosion after just 4-5 years that replacement was necessary. A replacement with unreinforced concrete has now served problem-free for nearly 15 years.

It could seem as if the producers and suppliers have monopolized durability research for concrete structures throughout the past several decades. Focus has only been on the concrete and its refinements to ensure all aspects of corrosion protection of reinforcement.

This has led to more expensive types of concrete resulting also in more execution sensitive concretes causing distress at the construction site, increasing the probability of early age cracking, as well as leading to additional drying shrinkage cracking and load induced cracking due to increased brittleness.

Figure 42: The inside of the 2000 years old concrete dome of the Pantheon in Rome shown in Figure 3.

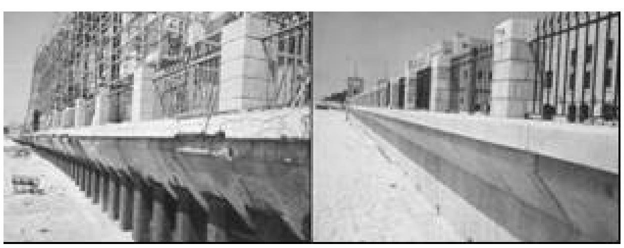

Figure 43: A reinforced concrete capping beam on top of a steel sheet pile seawall. To the left, the extensive reinforcement corrosion after 4-5 years requiring replacement of the capping beam. To the right the new capping beam, but now made from unreinforced concrete. This new structure has now served problem-free for more than 15 years.

In general the more costly and execution sensitive HPC is only needed in the outer concrete cover. However, that this concrete has to be used throughout the full sections has been overlooked by the design industry. The consequences have been a real global waste of expensive mineral and chemical additives and admixtures, often at the same time causing execution and curing difficulties - and often also leading to inferior quality and insufficient durability of the finished structure.

Special corrosion preventive additives to concrete, like inhibitors and pore blockers, also have to be added to the bulk of the concrete, though only needed near the reinforcement. This represents a similar costly waste.

It seems a one-sided approach to ensure the service life of corrosion-threatened structures by looking only at the concrete mix and coatings. This does not enhance environmental protection due to often increased need for future maintenance and repairs, and does not contribute to a sustainable society.

Very little has been done towards solving the corrosion problem through the choice of type of reinforcement. The efforts towards adopting non-metallic reinforcement have intrigued the researchers, but will change the execution conditions considerably, and seem therefore mostly adaptable to the precasting industry.

The potentials of adopting non corroding steel reinforcements have only during the past few years gained momentum, both on the designers side, due to the obvious advantages in design and execution, and on the steel reinforcement producers side, due to the foreseen growing markets. At this time such solutions seem to become very promising in the ongoing strive towards long lasting durable concrete structures and at the same time enhancing the concrete construction industry's efforts towards a sustainable built environment.

Finally, the competence to fully understand the durability related problem-complex and to achieve optimized integrated performance based designs of concrete structures will have to start with adapting this into the engineering educational curricular.

A new design paradigm is needed for the design and execution of concrete structures.

It shall no longer be the rule for large concrete structures that a new structure is a repaired structure.

This is a precondition if the competitiveness of concrete structures shall be increased and structural concrete thus remain the solid and reliable foundation for future societal prosperity.

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